A10/D002 File

Mussetter Engineering Inc.

Memorandum

To: **Mr. Dan Aguirre, Wilson and Company**

CC: **Mr. Doug Hughes, Mark Goodwin & Associates, Inc.**
Mr. John Kelley, AMAFCA

Date: February 9, 2005

From: Stuart Trabant, Bob Mussetter

Project: 04-31

Subject: **DRAFT** Comments for Design Features, Main Branch Calabacillas Arroyo

Mussetter Engineering, Inc. (MEI) has completed a preliminary evaluation of the proposed channel protection measures for the main branch of Calabacillas Arroyo upstream from Swinburne Dam. The locations and general layout of the protection measures that were analyzed are essentially the same as those shown on Sheet 1 of 1 in the Master Plan for the Arroyo Vista Subdivision that was prepared by Mark Goodwin and Associates (Goodwin) in June 2004. Several of the details, including the crest elevations and widths of the grade-control structures, the configuration of the McMahon Boulevard bridge opening, and widths between the spurs have been modified based on discussions at the October 12, 2004 meeting, and subsequent telephone conversations and email messages. One important change from the Goodwin plan is the elimination of Grade Control Structure (GCS) No. 5, at the upstream end of the reach. Other specific changes included reducing the bottom widths of the grade-control structures GCS#2 and GCS#4 to 140 feet and GCS#3 to 100 feet. The geometry through the bridge is a 7-foot deep rectangular channel with a bottom width of 100 feet (perpendicular to the direction of flow) intended to convey the 10-year peak flow. The rectangular channel is bound by a 15-foot left bank bench, 12-foot right bank bench, and 2H:1V sloping abutments (**Figure 1**). This memorandum provides a brief discussion of the findings from this preliminary evaluation that will provide a basis for adjustments to the overall plan that will better meet the needs of the adjacent landowners, AMAFCA, and the City of Albuquerque.

A one-dimensional (1-D) hydraulic analysis of the project reach was conducted using the U.S. Army Corps of Engineers HEC-RAS software package (USACE, 2004) to evaluate the changes in hydraulic characteristics associated with the design. The analysis indicates that the design will result in relatively minor changes in the hydraulic characteristics from existing conditions in most areas, but backwater conditions will occur upstream from the McMahon Bridge crossing for flows greater than the 10-year event. At the peak of the 100-year flood, the increase in water-surface elevation is nearly 4 feet (**Figure 2**.)

A sediment continuity analysis was performed using results from the hydraulic analysis to evaluate the vertical stability of the study reach under existing and design conditions, and to provide input to the lateral stability evaluation. To facilitate the analysis, the overall study reach was subdivided into 8 subreaches extending from upstream of the county line to Swinburne Dam (**Table 1**). Sediment rating curves (i.e., relationships between bed-material transport capacity and discharge) were developed for each subreach based on the typical bed material gradations and hydraulic results for existing conditions, design conditions with the bed at the existing grade, and design conditions with the bed at the estimated equilibrium grade, using procedures discussed in MEI (2000; Section 4.1). The sediment- continuity analysis was performed by comparing the bed-material transport capacities for each subreach with the total bed-material sediment supply. If the bed-material transport capacity in a particular subreach for a given storm exceeds the supply, degradation is indicated, and if the supply exceeds the capacity, aggradation is indicated. The total supply to a given subreach includes the supply from the upstream channel and from any tributaries along the subreach. The continuity analysis was performed for the 2-, 5-, 10-, 25-, 50-, and 100-year events and for average annual conditions under the following conditions:

A. Existing conditions, existing hydrology,
B. Existing conditions, future hydrology,
C. Design conditions at existing grade, future hydrology, and
D. Design conditions at the equilibrium slope, future hydrology.

Table 1. Summary of subreaches used in the sediment-continuity analysis.

Subreach	Upstream Station (ft)	Downstream Station (ft)	Length (ft)
1	30330	32526	2196
2	28094	30330	2236
3	26343	28094	1751
4a	23800	26343	2543
4b	22985	25000	2015
5a	22985	23800	815
5b	22150	22985	835
6	19845	22150	2305

The computed aggradation/degradation volumes from the sediment-continuity analysis were converted to average aggradation/degradation depths by dividing the volumes by the subreach length and width (**Figure 3**). It should be noted that the sediment-continuity analysis results for Scenario A (existing conditions and existing hydrology) represents a short-term scenario, where the sediment supply to any given reach was derived from both tributaries and the supply from the next upstream subreach. The results for the future conditions hydrology scenarios (Conditions B, C, and D) represent a long-term scenario, where the transport capacity for a given subreach was compared to the upstream supply to the overall project reach and local tributaries. This comparison effectively assumes that the capacity of the channel throughout the project reach has adjusted to the upstream supply to the overall reach. The results indicate that under existing topographic conditions and existing hydrology, the system is essentially in balance except for the subreach in Swinburne Pool (Subreach 5b), which is aggradational. Under the existing topographic conditions and future hydrology, the system is degradational throughout the project reach (Subreaches 2 through 5a), with Subreaches 4b and 5a being the most strongly degradational. Under the design conditions at existing channel grade with future hydrology, slightly more degradation than under existing conditions is indicated in Subreaches 2 and 3, less degradation is indicated in Subreaches 4a and 4b, and significantly more degradation is indicated in Subreach 5a. Under the design scenario at equilibrium slope with future conditions hydrology, the analysis indicates that the degradation is significantly reduced in all subreaches except Subreach 5a. The bed in this subreach has a considerable amount of erosion-resistent caliche, and it has been assumed that the caliche will provide grade control that will prevent significant degradation. It is important to note, however, that caliche is not non-erodible, and degradation could occur in this area over the long-term. The thickness of the calche layer is also not known. If it is relatively thin, erosion through the caliche could cause rapid degradation in this reach. While MEI is comfortable with the assumptions that have been made to date on this issue, we strongly recommend that this area be monitored very carefully in the future, and corrective actions, including additional grade control constructed, if warranted. It should be noted that the overall degradational tendencies under the equilibrium slope conditions is a likely somewhat conservative, at least over a short- to medium-term time-frame, because it assumes that the entire upstream channel, including reaches upstream from the county line have adjusted to the watershed sediment supply associated with the assumed 2038 development conditions. During the period when the upstream channel is adjusting to development, the sediment load will be substantially higher than the supply 2038 conditions over the long term. In fact, depending on the spatial distribution of the development as it occurs, the short- to medium term sediment yields may actually be higher than the existing supply.

To facilitate the evaluation of project conditions, the various design features, including the spur dikes, grade-control structures, and McMahon Bridge crossing, were overlaid onto the most recent available aerial photography (2000) and topography (2003) (**Map 1**). The spur dikes are labeled 1 through 9 from down- to upstream, with subscript L for the left bank spurs and subscript R for the right bank spurs. The design top of bank was laid out with a channel top width of about 290 feet, based on a channel bottom width between the spurs of 180 feet at the existing bed elevations, 8-foot high spurs at 2H:1V sideslopes, and a spur length of 40 feet. (The 180-foot bottom width is slightly greater than the 165-foot width of the spurs shown on Sheet 1 of 1 on the Goodwin Master Plan.)

The potential erosion setback for the design top of bank was evaluated by comparing the erosion envelope line that represents the maximum lateral migration distance (Δ_{max}) between the hardpoints (i.e., spurs and grade-control structures) with the flow expansion angle downstream from each structure. The erosion envelope was established based on Figure 3.24 of AMAFCA's Erosion and Sediment Design Guide (Mussetter et al., 1994). Guidelines from the Federal Highway Administration HEC-20 (Lagasse et al., 1995) indicate that, for impermeable dikes with the suggested design geometries, the flow expansion angle relative to the direction of flow is about 18 degrees. For purposes of this analysis, it was assumed that the bank is subject to erosion in any areas that are within the line represented by the expansion angle downstream from each structure and the maximum erosion envelope. Areas where bank erosion could lead to flanking of the design features are highlighted in Map 1. Protective measures for the highlighted areas could include either:

A. Tying the structure back into the bank beyond the maximum lateral migration distance or limit of the flow expansion angle from the upstream structure, whichever is least,

B. Protection of the upstream bank to the location where the limit of flow expansion intersects the bank, or

C. Reducing the distance between the spurs.

A summary of the tie-back distances (Option A) and upstream bank protection lengths (Option B) is summarized in **Table 2**.

A number of additional items regarding the location and alignment of the structures should be addressed before finalizing the design, including:

1. The hydraulic, sediment-continuity, and lateral migration analyses were based on spur geometries with 40-foot lengths extending from the bank into the flow path (perpendicular to the direction of flow). Initial calculations indicate that scour depths at the nose of the spurs during a 100-year event could range from about 15 to 21 feet at the existing grade, and could reach total scour depths of up to 28 feet, measured from the existing grade, under equilibrium slope conditions. The computed scour depths may be reduced by shortening the length of the spurs.

2. Spurs 9L and 9R are located about 550 feet downstream of the right bank tributary outfall that will be protected or reinforced as part of the design. If the right bank spur is located in the channel (Map 1), a significant portion of the right bank fill upstream of Spur 9R will need to be protected. Because Spur 9L is located on the outside of the downstream portion of a large bend, significant tie-back or upstream bank protection will be required.

3. Because the alignment of GCS#4 results in flow impingement on the left bank upstream from Spur 6L, the bank will likely require protection (even if the spur were tied into the bank by the suggested distance of 114 feet). If the structure was rotated counter-clockwise by 10 to 20 degrees, the length of bank protection could be reduced while maintaining proper flow alignment through the structure.

4. The current alignment of Spurs 5L, 5R, and 4L will direct the majority of flow into the upstream face of the right bridge abutment. Improved alignment of the flow into the bridge opening could be achieved by removing or significantly reducing the length of Spur 4L, adding Spur4R adjacent to Spur 4L, and shifting Spurs 5L and 5R about 70 degrees toward the left bank. (The flow alignment could also be improved by shortening Spur 5L and lengthening Spur 5R).

5. It is MEI's understanding that the entire right bank between GCS#3 and Spur 3R will be protected. To minimize the potential for bend scour below GCS#3, the upstream portion of this bank should be parallel to the direction of flow exiting the structure (see Map 1). Since the bank protection will extend to the upstream face of Spur 3R, it may be more cost-effective to eliminate Spur 3R and extend the riprap a short distance (50 to 80 feet) to a location coincident with the nose of the proposed spur. Spur 3L does not appear to provide significant flow direction or bank protection and could probably be eliminated from the design.

After you have had a chance to review these results and comments, please call so that we can discuss changes to the protection plan that should be evaluated in the next iteration.

References

Lagasse, P.F., Schall, J.D., Johnson, F., Richardson, E.V., and Chang, F., 1995. Stream Stability at Highway Structures. Hydraulic Engineering Circular No. 20, Second Edition, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., Publication No. FHWA-IP-90-014, November. 144 p.

Mussetter, R.A., Lagasse, P.F., Harvey, M.D., and Anderson, C.A., 1994. Sediment and Erosion Design Guide, prepared for Albuquerque Metropolitan Arroyo Flood Control Authority by Resource Consultants and Engineers, Inc.

Mussetter Engineering, Inc., 2000. Calabacillas Arroyo Prudent Line Study and Related Work – Development of Prudent Line for the Main Branch Upstream of Swinburne Dam, Albuquerque, New Mexico. Prepared for Albuquerque Metropolitan Arroyo Flood Control Authority.

U.S. Army Corps of Engineers, 2004. HEC-RAS River Analysis System, Version 3.1.2. User's Manual.

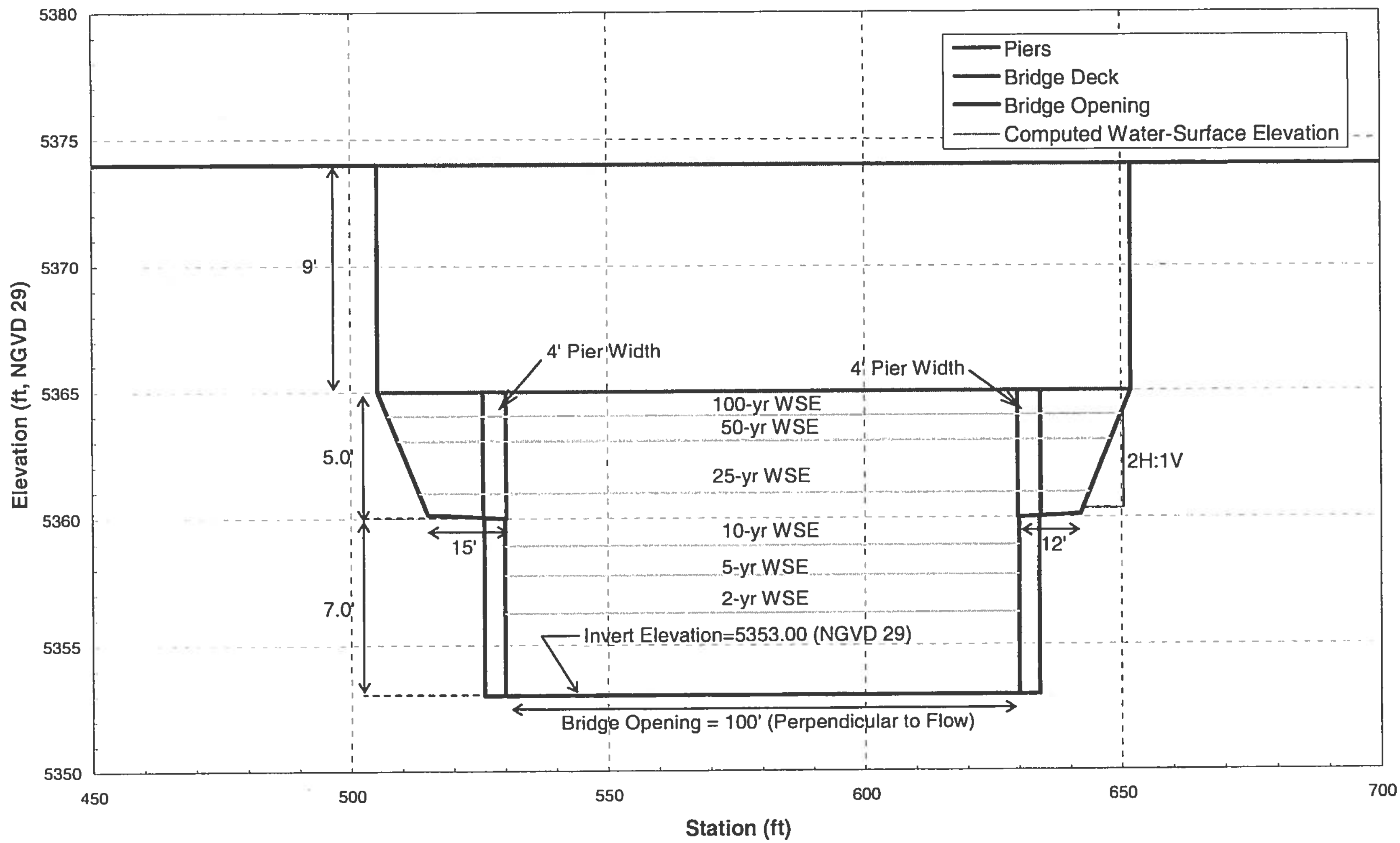

Figure 1. Upstream face of McMahon Bridge crossing (perpendicular to the direction of flow.)

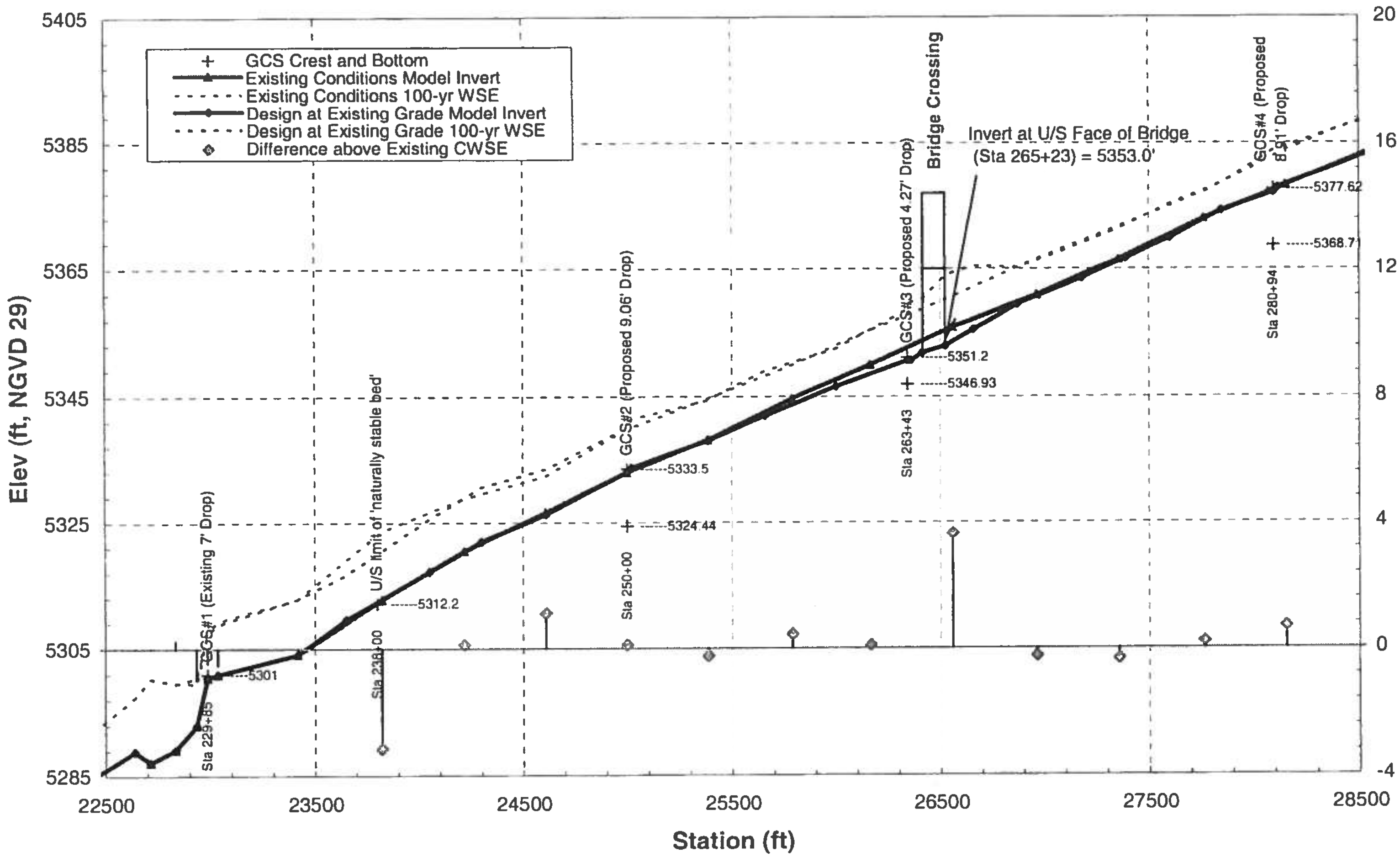

Figure 2. Computed 100-year water-surface profiles through the project reach.

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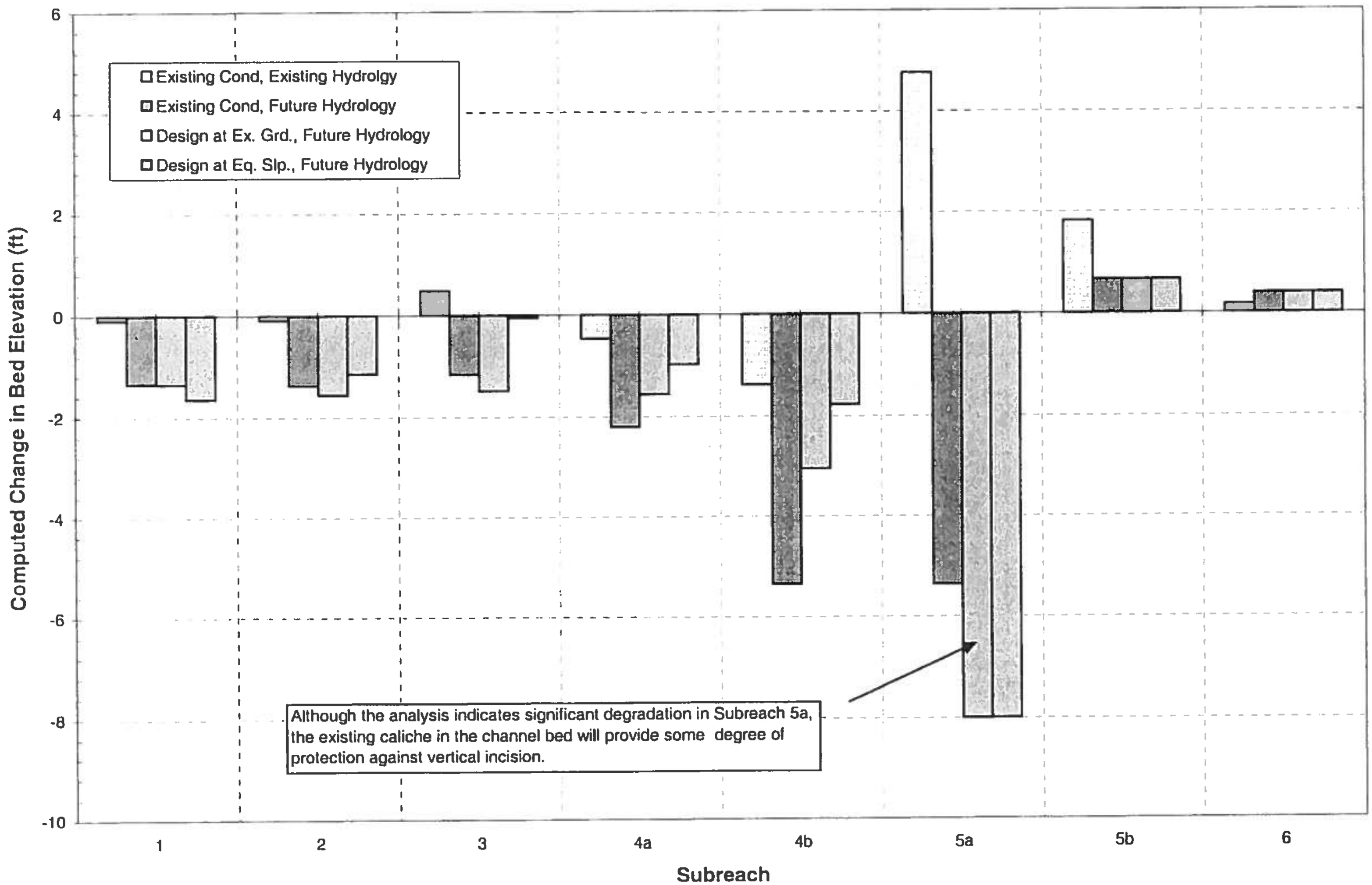
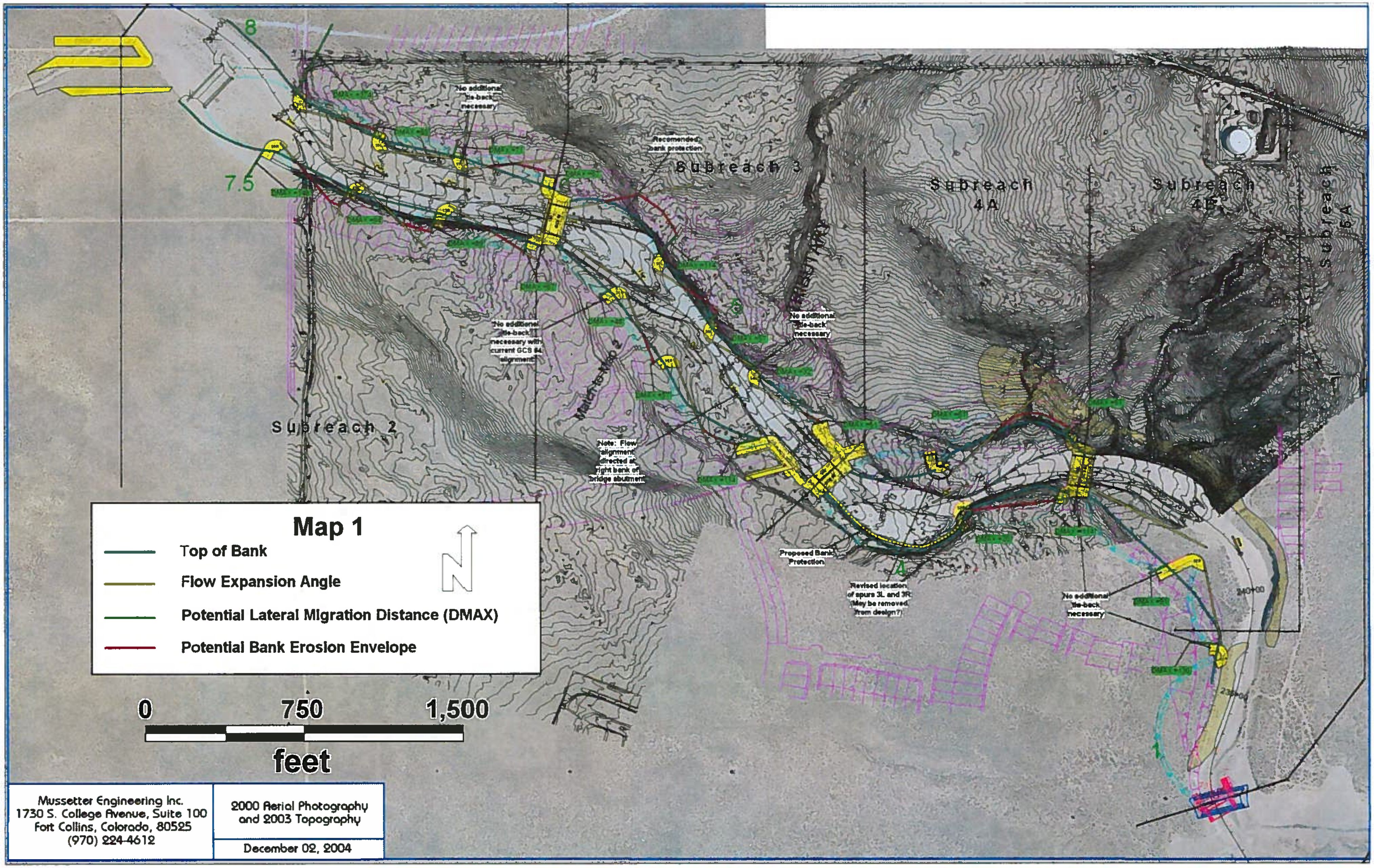


Figure 3. Computed average annual aggradation/degradation depths for existing conditions (existing hydrology), existing conditions (future hydrology), design conditions at existing grade (future hydrology), and design conditions at the equilibrium slope (future hydrology).

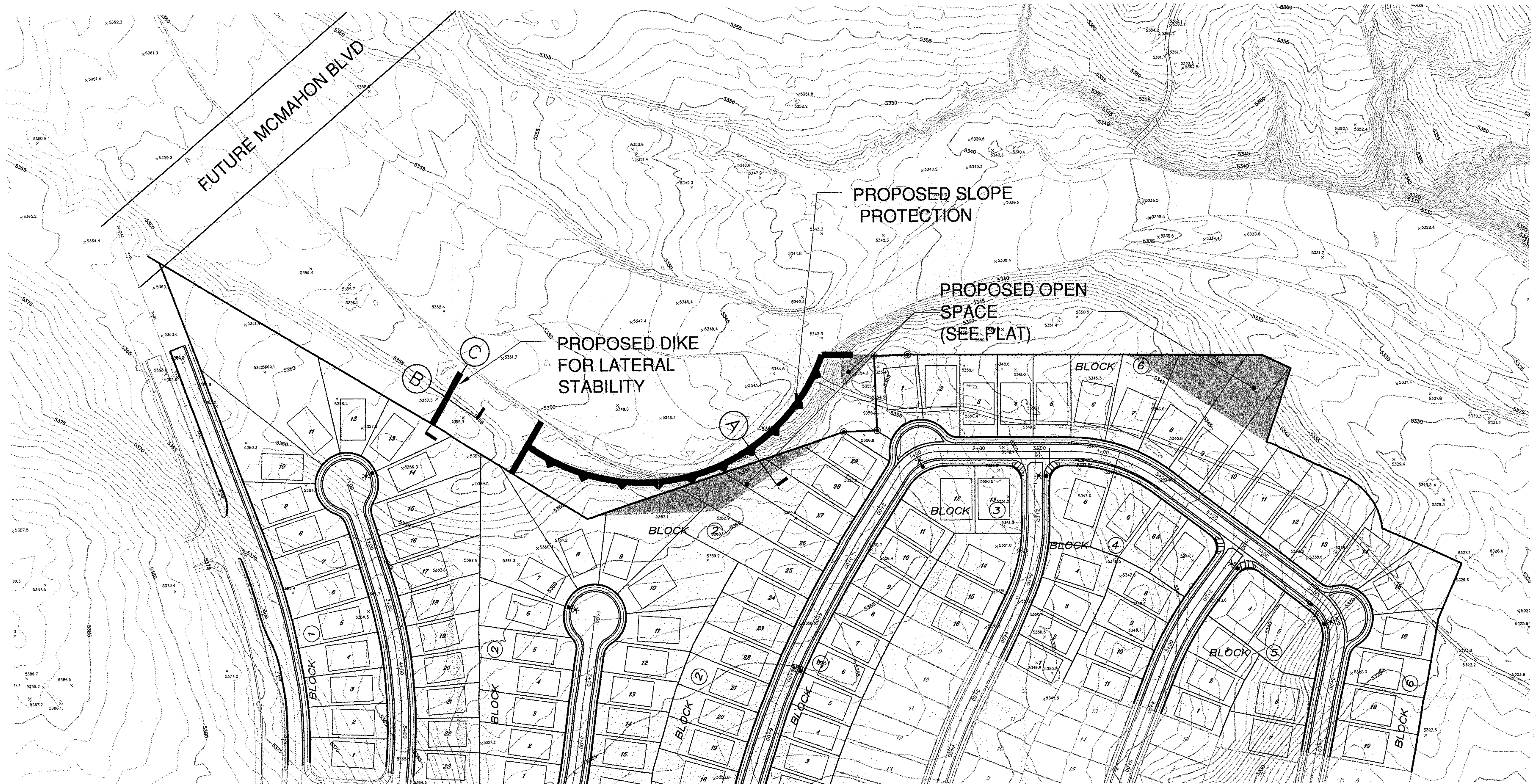
Table 2. Summary of protective measures for design features on the Main Branch Calabacillas Arroyo near the Arroyo Vista Subdivision.

Feature	Lateral migration distance (ft)		Erosion limited to U/S expansion angle?		Option A: Tie-back distance (ft)		Option B: Upstream bank protection		Comment
	Left Bank	Right Bank	Left Bank	Right Bank	Left Bank	Right Bank	Left Bank	Right Bank	
Spur9	374	148	No	No	374	148	400+	300+	Option B depends on outfall design.
Spur 8	85	94	Yes	No	30	94	150	300	
Spur7	71	89	Yes	No	30	89	0	290	LB tie-back is F/S.*
GCS4	67	67	No	Yes	67	50	270	190	
Spur6	114	48	No	Yes	114	30	300	0	RB tie-back is F/S.*
Spur5	57	57	No	Yes	57	30	200	140	
Spur4	32	NA	Yes	NA	20	NA	0	NA	LB tie-back is F/S.*
Upstream face of Bridge Crossing	51	114	Yes	No	40	114	70	320	Need to shift flow alignment into bridge
Spur3	61	20	Yes	No	30	-	0	-	RB will be protected upstream to GCS3
GCS2	209	91	No	Yes	209	30	310	320	
Spur2	NA	130	NA	Yes	NA	30	NA	0	RB tie-back is F/S.*
Spur1	NA	62	NA	Yes	NA	30	NA	0	RB tie-back is F/S.*

*F/S is Factor of Safety based on previous experience.



Map 1
Top of Bank
Flow Expansion Angle
Potential Lateral Migration Distance (DMAX)
Potential Bank Erosion Envelope
0
750
1,500
feet
Subreach 2
Subreach 3
Subreach 4A
Subreach 4B
Subreach 5A
8
7.5
No additional tie-back necessary
Recomended bank protection
No additional tie-back necessary with current GCS #4 alignment
Match to Map 2
Note: Flow alignment directed at right bank of bridge abutment
Proposed Bank Protection
Revised location of spurs 3L and 3R (May be removed from design?)
Mussetter Engineering Inc.
1730 S. College Avenue, Suite 100
Fort Collins, Colorado, 80525
(970) 224-4612
2000 Aerial Photography and 2003 Topography
December 02, 2004



SCOUR DEPTH CALCULATIONS

References:
Sediment & Erosion Design Guide
Resourse Consultants & Engineers, Inc.
March 1994

Calabacillas Arroyo–Stabilized Development Master Plan
DMG & Assoc., Inc.
March 2004

Case: Scour Along A Floodwall

$Ys/Y = 0.73+0.14\pi Fr^2$

Given: Y = 5.8', Fr = 0.85

$Ys/5.8 = 0.73+0.32$

$Ys = 6.1' \approx 6'\pm$

Case: Scour Along Dike Nose

$Ys/Y = 1.1(a/y)^{0.40}Fr^{0.33}$

Given: Y = 7.12', a = 50', Fr = 0.85

$Ys/7.12 = 2.27$

$Ys = 16.2' \approx 16'\pm$

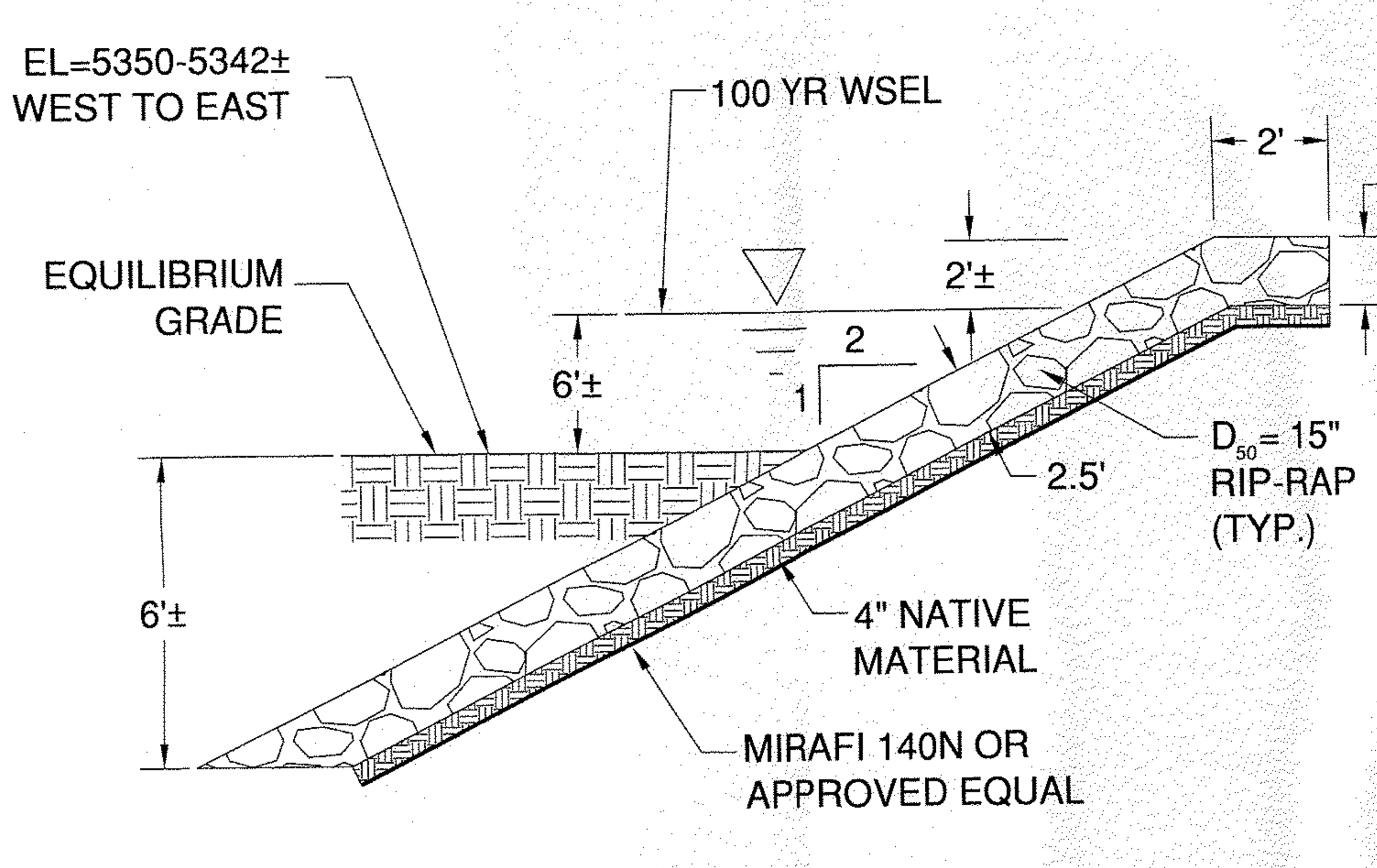
Case: Maximum Spacing Of Spurs

$S = L\cot\Theta$

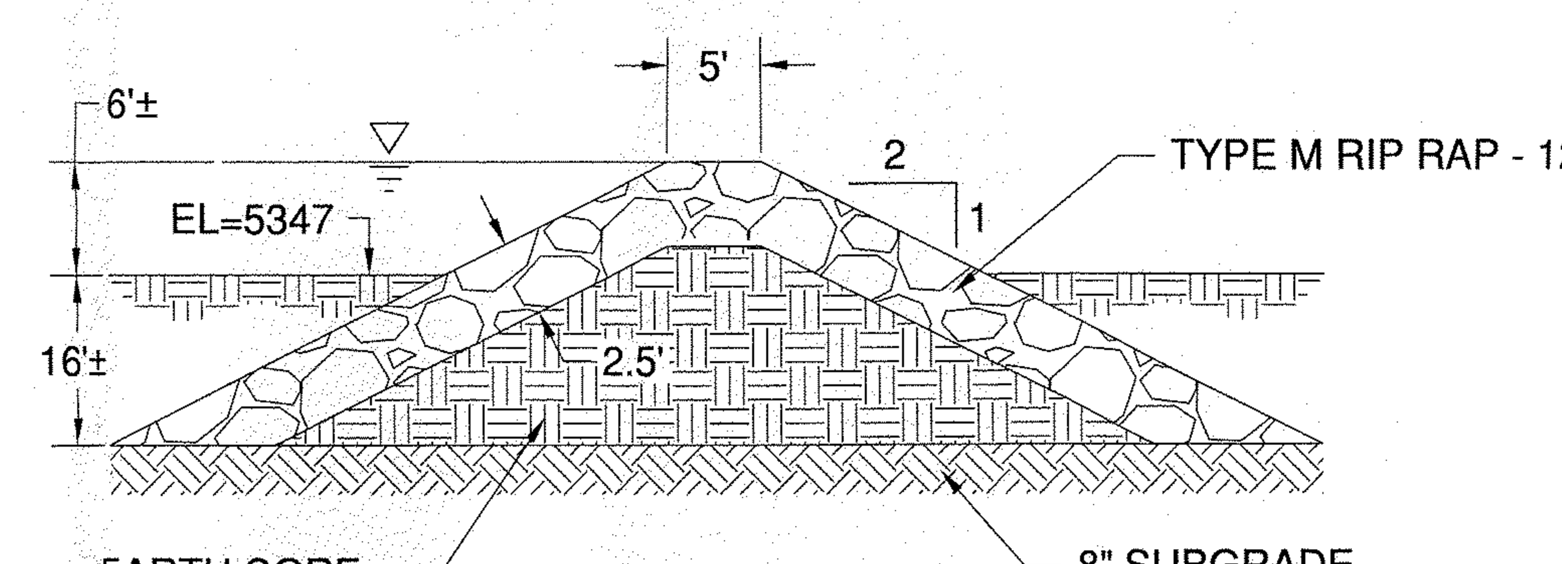
Given: L = 50', $\Theta = 18^\circ$

$S = (50')\cot 18^\circ$

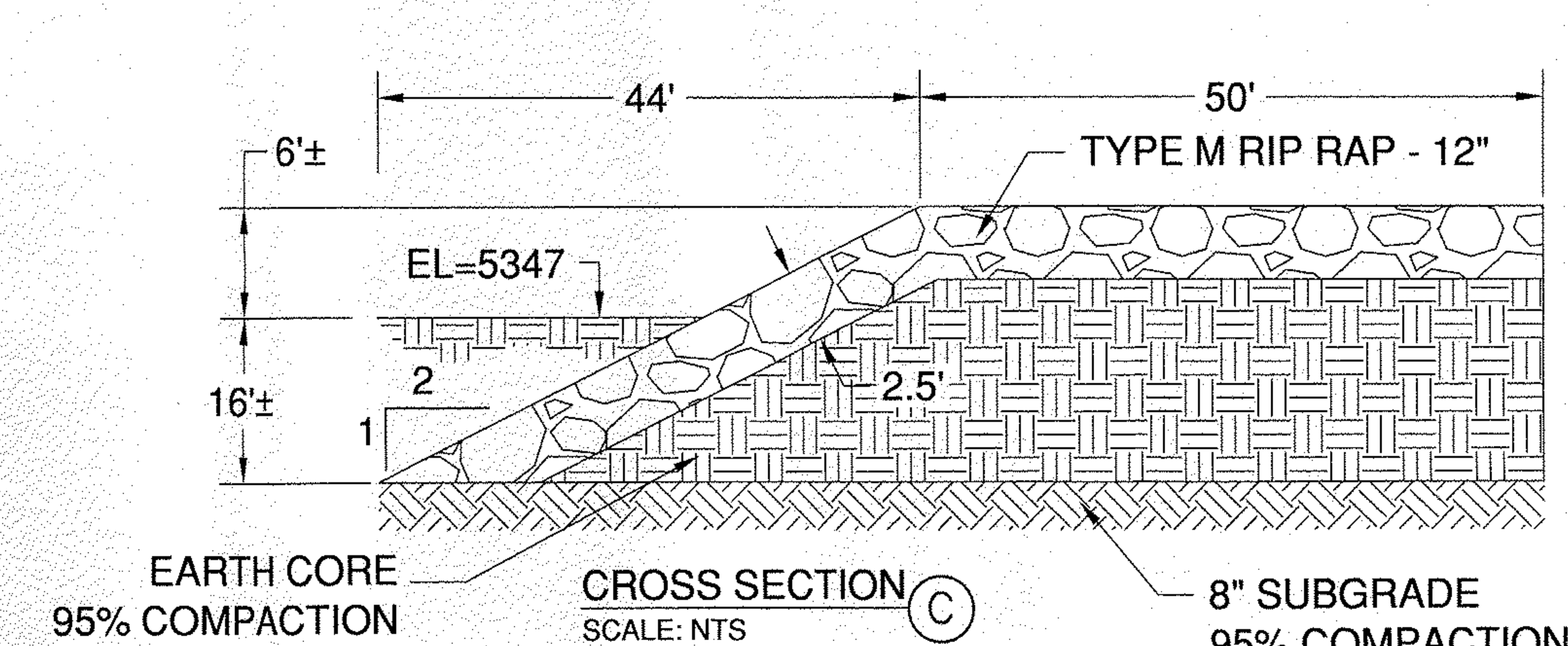
$Ys = 153.9' \approx 150.0'\pm$



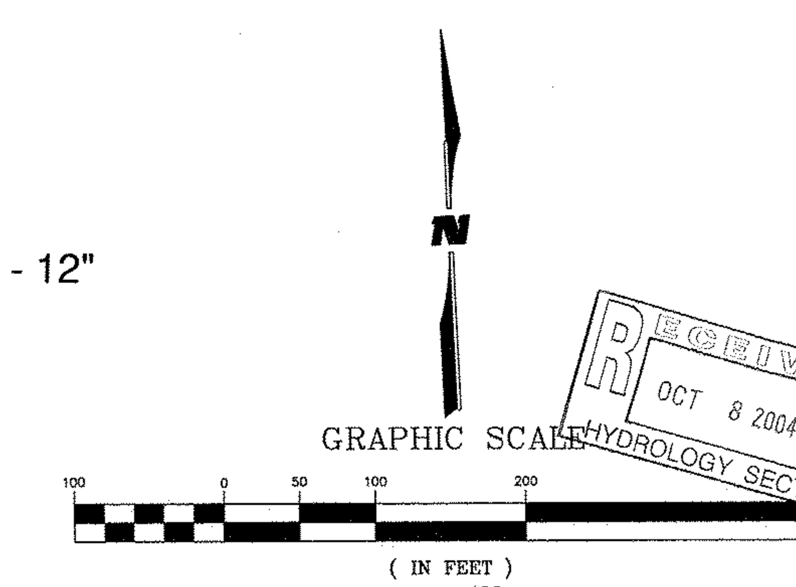
CROSS SECTION (A)
SCALE: NTS



CROSS SECTION (B)
SCALE: NTS



CROSS SECTION (C)
SCALE: NTS



RECEIVED OCT 8 2004 HYDROLOGY SECTION

APPROVED FOR CONSTRUCTION

AMAFCA DATE

WILSON & COMPANY
2600 THE AMERICAN ROAD S.E.
SUITE 100
RIO RANCHO, NEW MEXICO
87124
(505) 898-8021

CITY OF ALBUQUERQUE
PUBLIC WORKS DEPARTMENT
ENGINEERING GROUP

SEVILLE SUBDIVISION – UNIT 3A
CALABACILLAS ARROYO IMPROVEMENTS

Design Review Committee	City Engineer Approval	Last Design Update	Mo./Day/Yr.	Mo./Day/Yr.

City Project No.	Zone Map No.	Sheet	Of
702082	A-10-Z	32	32

AS-BUILT INFORMATION
CONTRACTOR
WORK STAKED BY — DATE
INSPECTOR'S ACCEPTANCE BY — DATE
FIELD VERIFICATION BY — DATE
DRAWINGS CORRECTED BY — DATE

MICRO-FILM INFORMATION
RECORDED BY — DATE
NO.

BENCH MARKS

SURVEY INFORMATION
FIELD NOTES

NO.	BY	DATE

ENGINEER'S SEAL
STEVE J. SALAZAR NEW MEXICO 16241 LICENSED PROFESSIONAL ENGINEER

REVISIONS

NO.	DATE	REMARKS	BY

WILSON & COMPANY, ENGINEERS & ARCHITECTS

DESIGNED BY	SJS	DATE	SEPT. 04
DRAWN BY	DEC	DATE	SEPT. 04
CHECKED BY	SJS	DATE	SEPT. 04

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