Manchester Avenue Conservation Bank

Preliminary Hydrology & Design Alternatives

Final Report

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Prepared for

The Center for Natural Lands Management
4367 Coronado Avenue
San Diego, CA 92107

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1.0 INTRODUCTION

The purpose of this task was to conduct a preliminary hydrology and hydraulic study and to develop conceptual erosion control alternatives to address the erosion problem in the upper reach of the main creek (referred to as the Project Reach) within the Manchester Avenue Conservation Bank.

2.0 APPROACH & METHODS

2.1 RESEARCH & SITE VISIT

HDR researched and reviewed available information on the watershed conditions and drainage patterns that affect the creek on the west side of the Conservation Bank. Information was collected during a visit to the City of Encinitas and through correspondence with City staff. The research produced topographic mapping (2-foot intervals) and aerial photos of the project area from years 2001 and 2005. Also, drainage plans of the residential areas surrounding the Preserve were used to confirm the watershed boundaries shown in Figure 1.

A portion of the Encinitas Landfill is tributary to the Project reach. The landfill has been inactive for many years and is owned and managed by San Diego County. In the last couple of years, a sediment basin was built at the landfill directly upstream from the Preserve. No as-built information could be found for existing improvements at the Encinitas Landfill, including the relatively new sediment basin. However, hydrology calculations for the landfill area were previously done for a May, 2002 study titled "Sediment Basin Evaluation, Inactive Landfills" prepared by URS for the County of San Diego Department of Public Works.

On October 16, 2006 HDR visited the site to investigate the extent of and possible contributors to the existing erosion problems, as well as to identify possible erosion control alternatives. A photo-log of the site visit is provided in Appendix A.

2.2 HYDROLOGY

HDR conducted a hydrology study to determine the 2-, 5-, 10-, 25- and 100-year peak flow rates for the Project Reach, per the San Diego County Hydrology Manual (2003) procedures. The calculations were made using CivilDesign Corporation's CivilD software (see Appendix B). The tributary watershed was delineated based on the topographic mapping and development plans obtained from the City of Encinitas. The topography and drainage plans show that very little of the residential areas around the Preserve drain to the project reach.

For this project, the watershed was broken into three areas as shown in Figure 1. For Area 1, the west side of the Encinitas Landfill, the time of concentration was taken to be 10.5 minutes and the runoff coefficient was taken to be 0.3 in order to be consistent with the previous hydrology work done for the URS report. A check using the San Diego County Hydrology method confirmed that these values are reasonable. Because design documents could not be obtained for the sediment basin at the landfill site, our analysis does not take any dampening effects it may provide into consideration. For Areas 2 and 3, soils information was obtained from the San Diego Hydrology Manual and the existing land use was input as "undisturbed".

Source: City of Encinitas Aerial Photo, 2005; San Diego County Hydrology Manual, 2003 | G:\Projects\384198_CenterForNaturalLandsMgmt\47890_ManchesterAveConservation\map_docs\mxd\Fig_1_Hydrology | Last Updated: 1-10-07

Hydrology Map

FIGURE 1

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Table 1 shows the cumulative peak runoff results from each watershed area for a wide range of storm frequencies.

Table 1. Total Peak Runoff

Area	Total Peak Runoff (cfs)					
	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
Area 1 (Pt 2)	11	16	17	20	23	26
Area 2 (Pt 3)	15	21	24	28	32	36
Area 3 (Pt 4)	21	30	34	40	46	52

2.3 HYDRAULIC MODELING

Cross-sections were taken at fifty-foot intervals along the project reach for use in a U.S. Army Corps of Engineers, Hydrologic Engineering Center's River Analysis System (HEC-RAS) model (see Appendix B). See Figure 2 for the cross-section locations and the extent of the model. Figure 2 shows topography, aerial photography and the cross section locations for the project reach. The 2001 topography is shown in the figure because the channel shape observed during our site visit (nearly vertical banks with a relatively flat bottom) is more consistent with the older data than that taken in 2005. Further design of this project may require a more detailed survey of the project reach. For more discussion on our use of the available topography, see Figures 3 and 4.

The model was run for the range of flows shown in Table 1 and for several roughness conditions (n-values) in order to see the streams sensitivity to these factors. The results show that the scour problem in the stream is relatively in-sensitive to changes in storm frequency, roughness, and channel shape. See Figures 3, 4, 5, 6, and 7 for analysis of the HEC-RAS model results.

Source: City of Encinitas Topography, 2001 | G:\Projects\364198_CenterForNaturalLandsMgmt\47690_ManchesterAveConservation\map_docs\mxd\Fig_2_Hydraulics | Last Updated : 1-10-07

Site Plan

FIGURE 2

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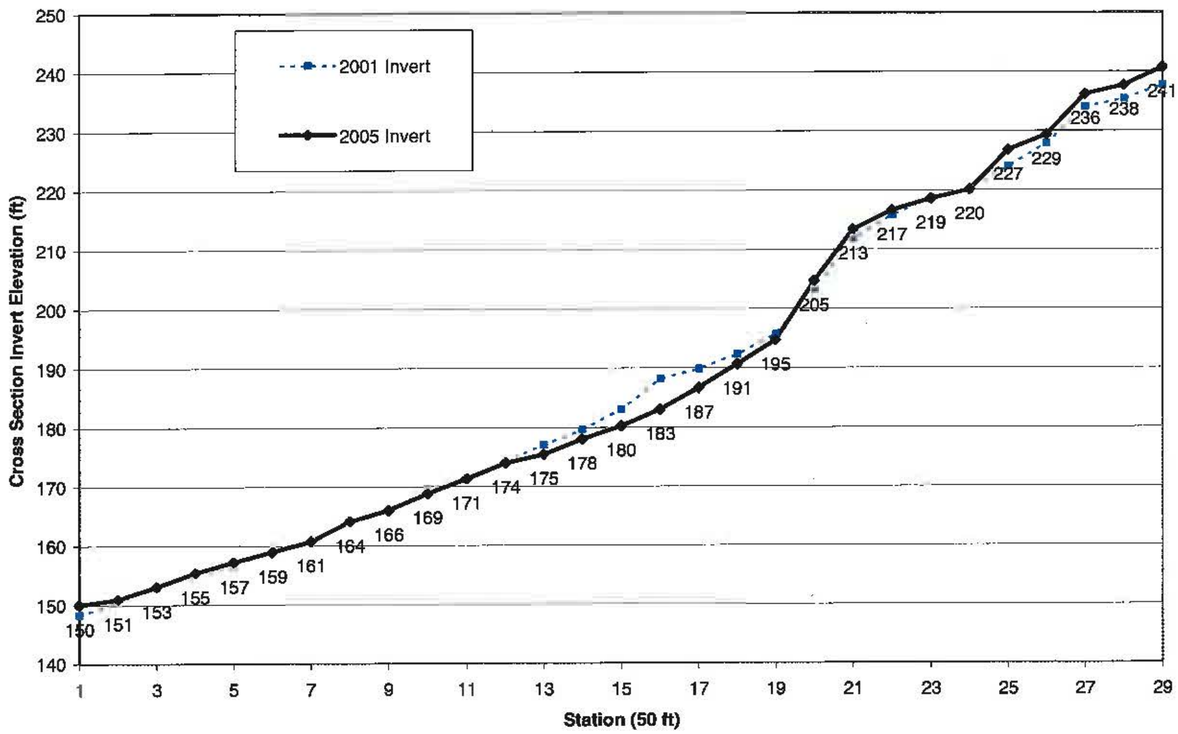


Figure 3. Stream Invert Profile

Figure 3 shows profiles of the stream invert along the project reach from the 2001 and 2005 data. The profile shows that the most severe erosion has occurred from cross section 20 approximately 900 feet downstream to cross section 12. In this reach high flow velocities are contributing to erosion at the toe of the channel which is resulting in the problematic sloughing of the banks shown in the pictures in Appendix A. The reach downstream from cross section 12 is also experiencing velocities that will cause scour; however, the stream bed degradation is less pronounced during the window of time shown in the profile.

From the Encinitas Landfill (cross section 29) down to the foot bridge (Cross-section 21) the channel appears to have been fairly stable during the four year window, despite seeing high flow velocities (see Figures 5 and 6). This may be due to the presence of harder, more erosion resistant materials at the channel toe in this reach.

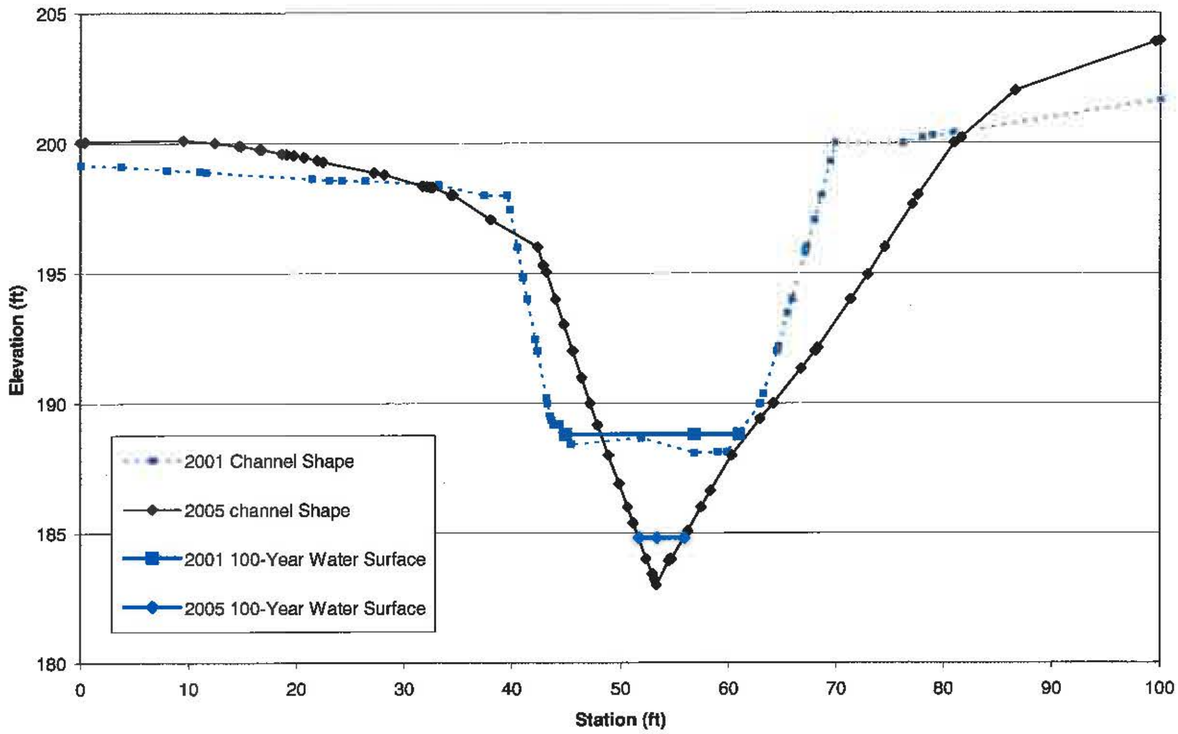


Figure 4. Example Cross Section (Cross Section 16)

Figure 4 shows cross section 16 and the 100-year water surface elevations for both the 2001 and the 2005 models. Models were built from cross sections from both the topographic data sets because there was a significant difference between the two regarding the channel shape. See Figure 4 for reference. The 2005 data appears to have one elevation point along the stream invert that forms a triangular channel shape with the banks, while the 2001 data shows very steep banks with a relatively flat bottom. This difference is probably due to the 2005 data set having less elevation data points from the channel bottom than the 2001 data set.

Building HEC-RAS models from both showed that the difference in the two geometries did not significantly change the flow conditions in the channel. Even for the 100-year flood, the main channel fully contains the flow for the entire project reach. Figure 4 also shows the significant scour that occurred between 2001 and 2005 at cross section 16.

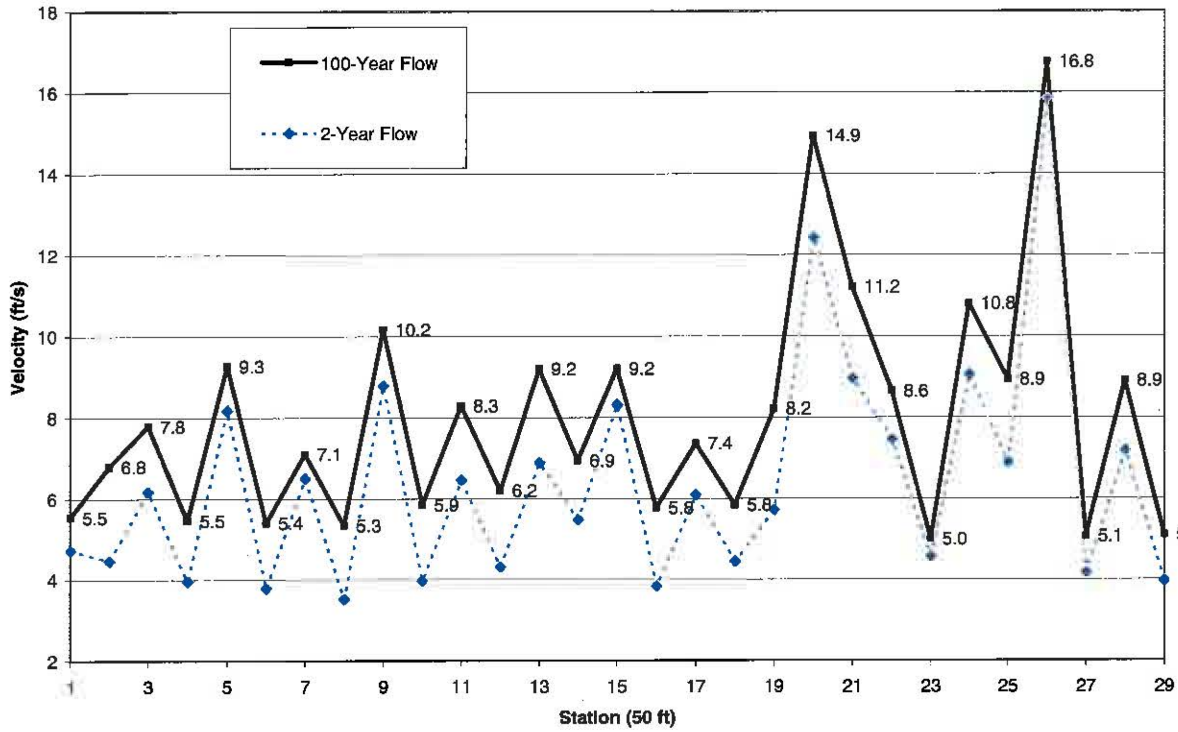


Figure 5. Velocity Analysis (2001 Topo, n=0.03)

Figure 5 shows the channel velocity from the 2001 model at each cross section for both the 100-year and 2-year frequency floods. The n-value was taken to be 0.03 to be on the conservative side for calculating velocity. Additional data for intermediate floods (5-, 10-, 25-, 50-year) was not included for clarity. Figure 5 shows that downstream from cross section 20, the flow velocities vary from 4 to 10 ft/s for all storms considered and velocities only increase by approximately 1 to 2 feet per second between the 2-year and the 100-year storm.

Scour potential in streams is a complex phenomenon affected by water velocity, water depth, soil characteristics, the sediment loading of the water, and the localized effects of bends and obstructions. While a complete scour analysis would require more information and is beyond the scope of this report, there are suggested maximum velocities that have been published for various soil types and sediment loading conditions. The soils in the project reach are defined as Soil Mapping Unit CsD (loamy sand) by the San Diego County Hydrology Manual. The range of maximum permissible velocities for this soil varies from 1.7 to 2.5 ft/s. This data shows that scour will occur in the stream during regular events. Because the additional costs for materials will be small, we propose designing the project for a 100-year flood.

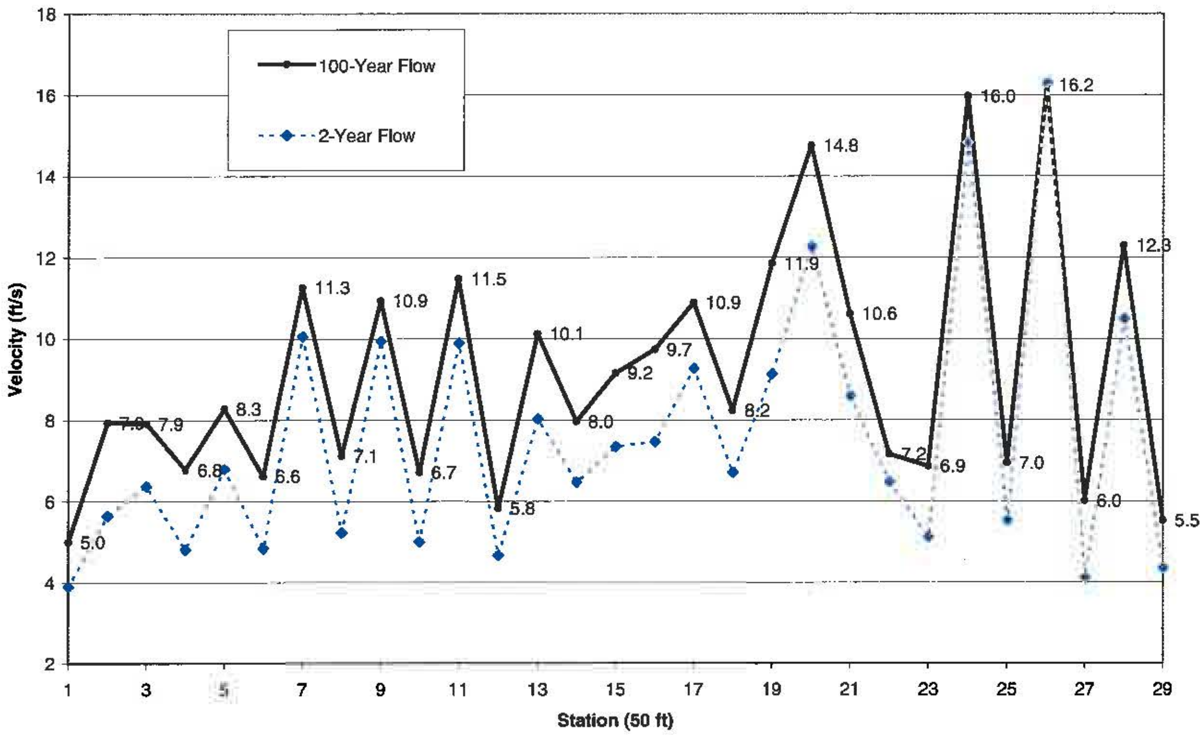


Figure 6. Velocity Analysis (2005 Topo, n=0.03)

Figure 6 shows the channel velocity from the 2005 model at each cross section for both the 100-year and 2-year frequency floods. The n-value was taken to be 0.03 to be on the conservative side for calculating velocity.

Figure 6 shows that the flow velocities are relatively consistent between the 2001 and 2005 HEC-RAS models. Downstream from cross section 20, the flow velocities vary from 5 to 11.5 ft/s for all storms considered and velocities only increase by approximately 1 to 2 feet per second between the 2-year and the 100-year storm.

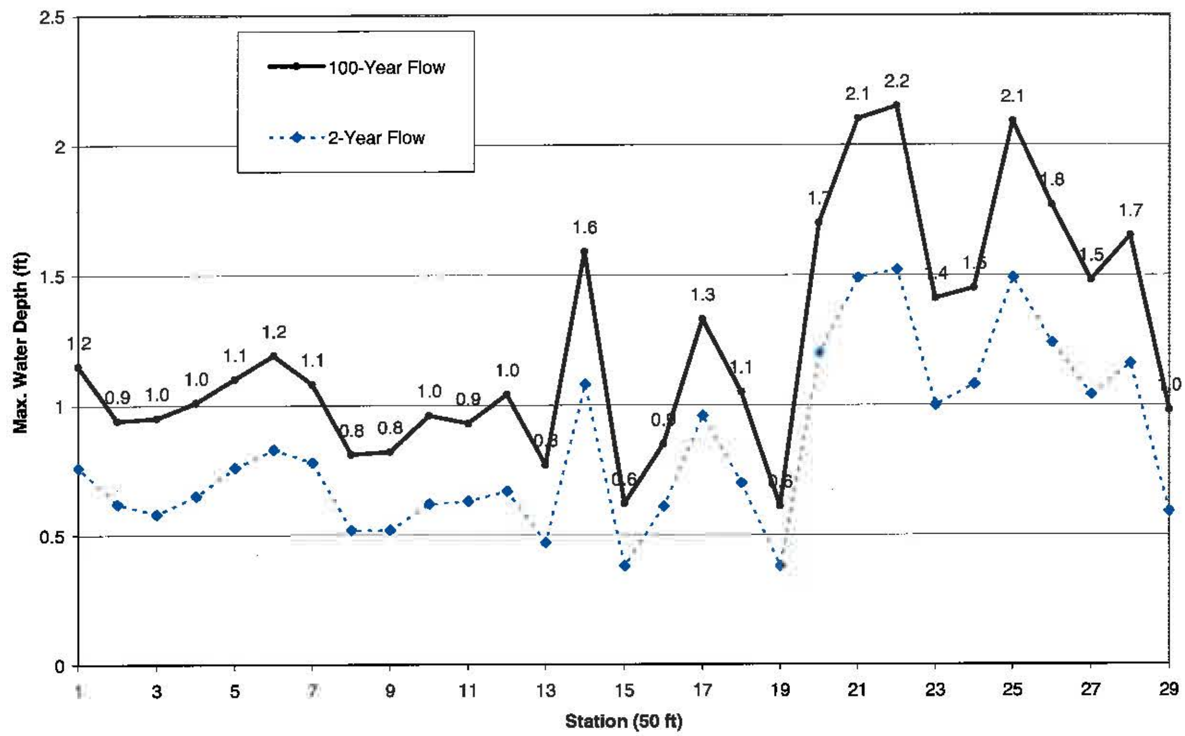


Figure 7. Water Depth Analysis (2001 Topo, n=0.05)

Figure 7 shows the water depth at each cross section for both the 100-year and 2-year frequency floods. The n-value was taken to be 0.05 to be on the conservative side for calculating water depth. Downstream of cross-section 20, the water depth varies from 0.75 feet to 1.5 feet for a 100-year flood. The height of longitudinal toe protection would only need to be raised by 0.5 feet to increase from 2-year to 100-year flood protection.

3.0 EROSION CONTROL ALTERNATIVES

Based on findings from the site visit and results of the hydrologic and hydraulic models, three conceptual alternatives were considered to address the erosion problem within the Project Reach. Descriptions of the alternatives and Figures 8, 9 and 10 were adapted from the Environmentally-Sensitive Streambank Stabilization (E-SenSS) CD Manual, 2004. The E-SenSS CD Manual was originally funded by the National Cooperative Highways Research Program (NCHRP).

3.1 ALTERNATIVE 1 – LONGITUDINAL STONE TOE BANK PROTECTION

Longitudinal stone toe has proven cost-effective in protecting lower banks and creating conditions leading to stabilization and revegetation of steep, caving banks. Stone toe is continuous bank protection consisting of a stone dike placed longitudinally at, or slightly streamward of, the toe of an eroding bank (see Figure 8). In this application, the rock would be placed continuously on both toes of the channel beginning at cross section 20 then extending approximately 900 feet downstream to cross section 2. The stone toe would be approximately 30-inches thick, 3-feet tall and toed in 1.5 feet. Installing longitudinal stone toe bank protection would replace approximately 0.2 acres of the channel bottom with rock. The success of this method depends upon the ability of stone to self-adjust or "launch" into any scour holes formed on the stream side of the revetment. Longitudinal stone toe is very cost-effective and is relatively easy to construct. It is simple to design and specify and is a thoroughly tested method that has been used in a variety of situations and has been extensively monitored.

This continuous bank protection technique protects the toe from erosion. It is especially effective in streams where most erosion is due to relatively small but frequent events and where the toe is experiencing erosion but the mid and upper banks are fairly stable due to vegetation, cohesive soils, infrequent short-duration inundation, or relatively slow velocities. Stone toe is particularly applicable for ephemeral, narrow, and small- to medium-sized streams. It protects the toe so that slope failure of a steep bank landward of the stone toe will produce a stable angle. Such a bank is often rapidly colonized by natural vegetation.

Longitudinal stone toe has documented environmental benefits. Vegetative cover can become established, even growing through the rock, and can provide canopy and a source of woody debris. Bank grading, reshaping, or sloping is often not needed (existing bank and overbank vegetation need not be disturbed or cleared), nor is a filter cloth or gravel filter needed. If stone is placed from the water side, existing bank vegetation need not be disturbed.

However, longitudinal stone toe only provides toe protection and does not protect mid- and upper bank areas. Some erosion of these areas should be anticipated during long-duration, high energy flows, or until the areas become otherwise protected (biotechnical techniques). Stone toe is not suitable for reaches where rapid bed degradation (lowering) is likely, or where scour depths adjacent to the toe will be greater than the height of the toe.

3.2 ALTERNATIVE 2 – LONGITUDINAL STONE TOE AND SLOPE FLATTENING

Alternative 2 would add slope flattening to the longitudinal stone toe discussed in Alternative 1 (see Figure 9). Flattening or bank reshaping stabilizes an eroding streambank by reducing its slope angle or gradient. Slope flattening is usually done in conjunction with other bank protection treatments, including installation of toe protection, placement of bank armor, re-vegetation or erosion control, and/or installation of drainage measures (see Figure 9). In this case, slope flattening would be used by cutting a more gradual slope behind a longitudinal stone toe structure and re-vegetation with native plants.

Slope angle or inclination is one of the principal determinants of stability. The steeper the slope, the more susceptible it is to both surficial erosion and mass failure. Slope flattening or re-grading is generally one of the first options to consider as a possible remedial treatment; on the other hand, it will only provide temporary protection unless toe erosion (scour) and channel degradation are arrested and controlled as well. Most soils are stable at a slope gradient of 2H:1V or 2.5H:1V. This is also the maximum gradient for successful conventional planting or re-vegetation, at least in the absence of special soil bio-engineering techniques such as live brushlayering, live fascines, or a live slope grating. Hard bank armor, e.g., rock riprap, should not be placed on slopes steeper than 1.5H:1V. Stable artificial or fill slopes can be constructed as steep as 1H:1V (or even higher) by layering geogrid or geotextile reinforcements between successive lifts of soil.

Environmental benefits of slope flattening accrue primarily from the ability to establish vegetation more easily on a streambank that is not over-steepened. A vegetated (or re-vegetated) bank yields both aquatic and terrestrial environmental benefits. Streambank vegetation provides cover, shade, and insect food sources for aquatic organisms near the water's edge. Upper and mid bank vegetation provides cover and habitat opportunities for small mammals and other riparian wildlife. Slope flattening should also result in easier wildlife access to the creek bottom from the top of bank.

Slope flattening to 3H:1V would result in grading of approximately 1 acre of upper bank habitat. If the slope was cut at 2H:1V, the steeper slope would reduce the graded bank area to 0.5 acres. See Figure 2 for the extent of the proposed bank grading.

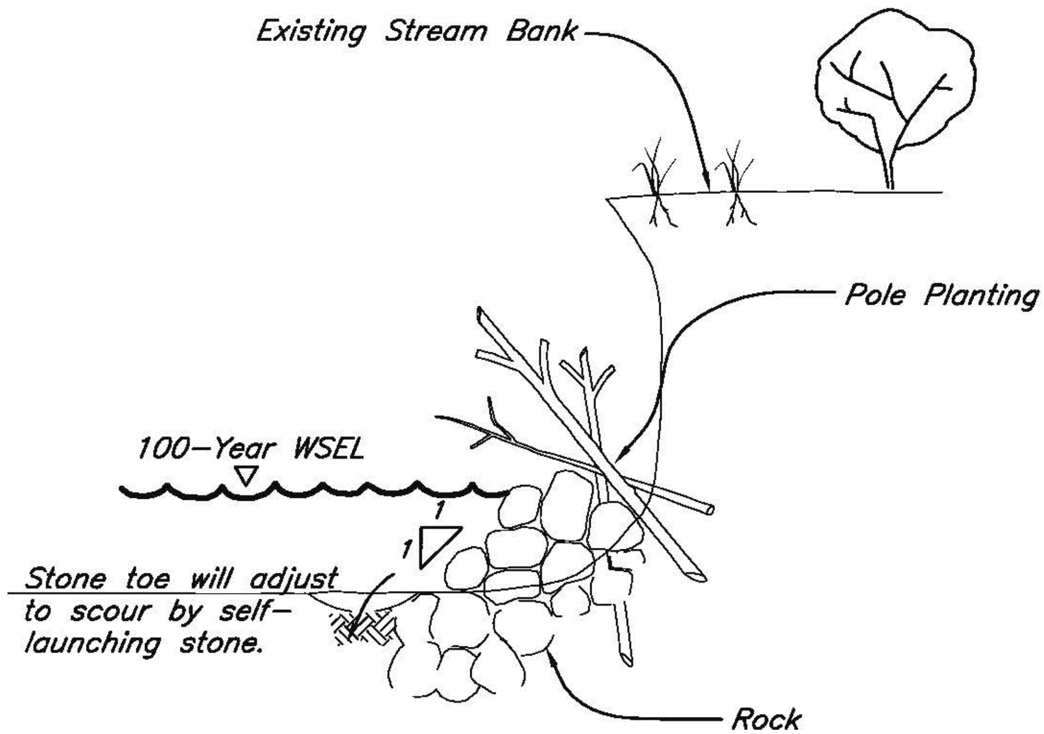
3.3 ALTERNATIVE 3 – ADD GRADE CONTROL STRUCTURES TO ALTERNATIVE 2

Alternative 3 would add rock grade control structures to the longitudinal stone toe and slope flattening concepts discussed in Alternatives 1 and 2 (see Figure 10). Well-constructed weirs can prevent or retard channel bed erosion and upstream progression of knickzones and headcuts, as well as providing pool habitats for aquatic biota. Small weirs or "check dams" are sometimes used to control erosion of gullies and small, ephemeral channels in order to limit sediment movement downstream. Well-designed weirs can prevent headward-progressing bed erosion, which is extremely detrimental to upstream habitats and downstream reaches impacted by sedimentation.

Stone weirs are natural-looking, and create a visual amenity as well as a habitat resource if well designed. The flexible nature of stone allows weirs to deform in response to slight changes in the adjacent channel boundary without failure. If carefully designed, stone weirs are among the few techniques that are useful for stabilizing degrading beds in incising channels. However, designers should plan for future degradation, which will increase the drop height (head loss) over the weir unless the downstream base level is positively controlled.

Perhaps the most important limitation for stone weirs is the maximum drop height or head loss across the weir. In most cases, stone weirs should not produce a change in the energy grade line greater than 2 ft, although very ruggedly designed weirs may approach 5 ft drops. Stone weirs should be inspected annually and after high flow events. Maintenance usually involves removal of sediment deposits, vegetation, trash or woody debris trapped by the weir if they produce undesirable flow patterns and replacement of dislodged stone. In some cases weirs must be reconstructed or replaced with a larger number of weirs to prevent excessive drop heights.

Through interpretation of the topographic data from 2005 and 2001 (see Figure 3), we can see that the most severe scour in recent years has occurred between cross sections 19 and 12. This reach should have the highest priority for installing grade control structures. Alternative 3 proposes installing approximately six structures 3- to 6-feet deep in this reach. This preliminary recommendation is based on conservative assumptions due to incomplete information. Final design and siting of the proposed grade control structures will require additional geotechnical and sediment transport analysis. More information may allow a less conservative design, or it could warrant extending the grade control structures further downstream. Each grade control structure would occupy approximately 60 square feet of channel bottom.



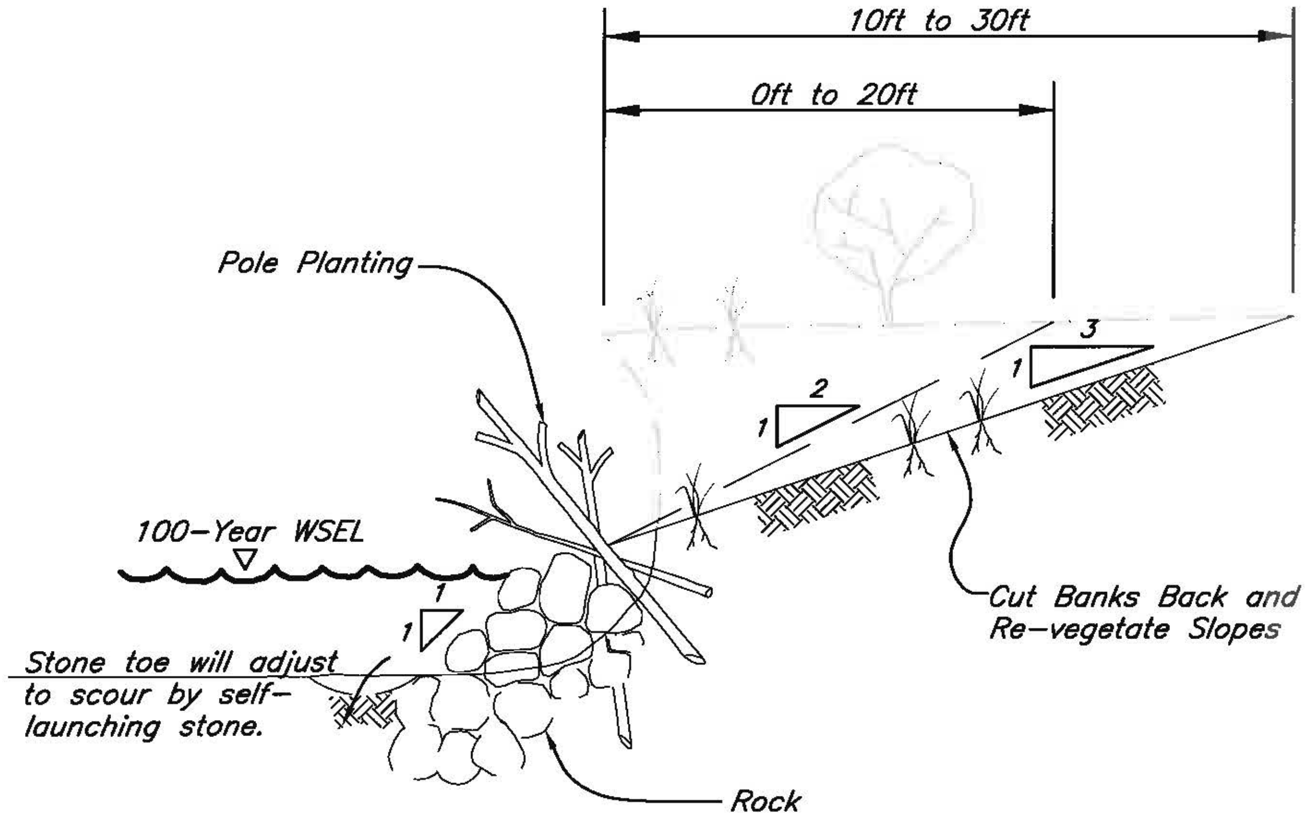
NOTES:

1. Longitudinal stone toe is a good choice when continuous bank protection is needed for the toe, but the mid and upper banks are relatively stable and/or biotechnical practices are suitable.

2. The success of Longitudinal Stone Toe depends on the ability of the well–graded stone to self adjust or "launch" into any scour holes formed on the stream side of the stone toe.

Alternative 1 - Longitudinal Stone Toe Bank Protection

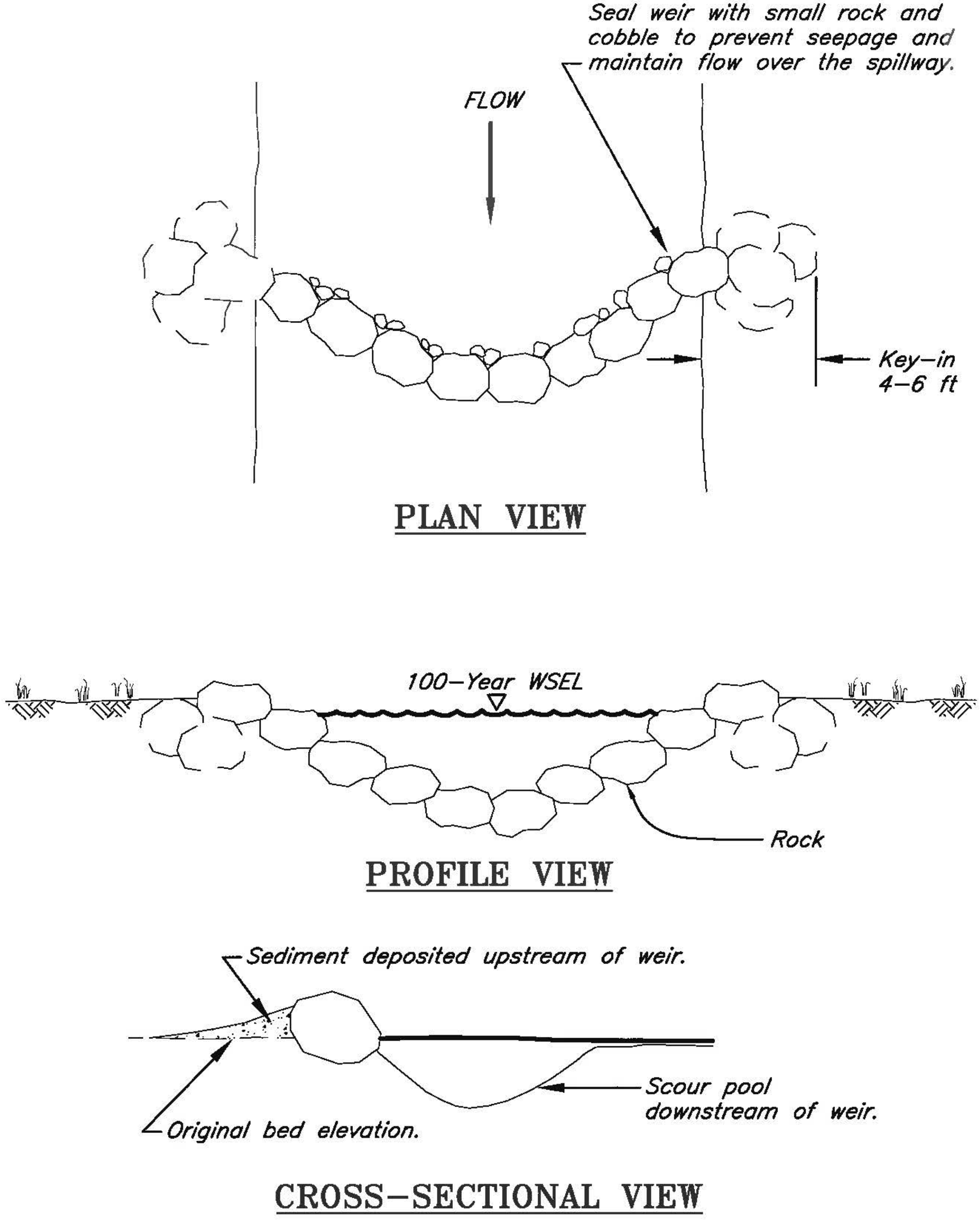
FIGURE 8



- •1V:3H – Maximum suggested slope angle for establishing plantings or seedlings, when used alone.
- •1V:3H – IV:2H – Optimal slope angle range for soil bioengineering.
- •1V:3H or steeper – Roughen stairstep or terrace slope if planting.
- •1V:2H – Maximum suggested slope angle for unreinforced fills.
- •1V:2H or steeper – Biotechnical techniques (combination of stabilization structures, soil bioengineering and geotechnical methods) often needed.
- •1V:1H – Maximum suggested slope angle for unreinforced cuts in clay soil.
- •5V:1H – Typical face angle for rockeries, gabions, crib walls, etc.

Alternative 2 - Longitudinal Stone Toe and Slope Flattening

FIGURE 9



Alternative 3 - Add Grade Control Structures to Alternative 2

FIGURE 10

3.4 PREFERRED ALTERNATIVE

Alternatives 1, 2, and 3 are not mutually exclusive solutions, rather they are progressive steps that address the problems in the project reach in order to stabilize the stream system and, by doing so, enhance the habitat in the Manchester Avenue Conservation Bank. Table 2 breaks down the costs of each alternative. However, the cost estimate does not include costs that may be required for environmental permitting associated with the project.

Table 2. Preliminary Estimate of Probable Cost

	Job No. 47690			Calc No.	00001
Computation					HDR
Project	Manchester Avenue Conservation Bank			***Computed***	GRH
Subject	Preliminary Hydrology & Design Alternatives			***Date***	10-Jan-07
Task	Draft Report			***Reviewed***	
File Name	\\sdg-filsrv01\engdoc\384198_CenterForNaturalLandsMgmgt\47690_ManchesterAveConservation\6_Engineering-Design\[Cost Estimate.xls]Cost Estimate			***Date***	
		QUANTITY	**UNITS**	**UNIT PRICE**	**TOTAL COST**
LONGITUDINAL STONE TOE					
	Excavation for Rock Toe-Down	353	CY	$8.00	$2,824
	Imported Rock : Longitudinal Toe	707	CY	$100.00	$70,700
	Pole Planting	1,800	LF	$10.00	$18,000
	Construction Contingencies (30% of Component Cost)	1	LS	$27,457.20	$27,457
	SUBTOTAL				**$118,981**
SLOPE GRADING 3:1					
	Excavation for Bank Grading : 3 to 1	6,580	CY	$8.00	$52,640
	Soil Preparation	4,356	SY	$2.80	$12,197
	Hydroseeding	4,356	SY	$0.54	$2,352
	Construction Contingencies (30% of Component Cost)	1	LS	$20,156.71	$20,157
	SUBTOTAL				**$87,346**
SLOPE GRADING 2:1					
	Excavation for Bank Grading : 2 to 1	2,800	CY	$8.00	$22,400
	Soil Preparation	1,936	SY	$2.80	$5,421
	Hydroseeding	1,936	SY	$0.54	$1,045
	Construction Contingencies (30% of Component Cost)	1	LS	$8,659.87	$8,660
	SUBTOTAL				**$37,526**
GRADE CONTROL STRUCTURES					
	Excavation for Rock Toe-Down	128	CY	$8.00	$1,024
	Imported Rock : For 6 Grade Control Structures	128	CY	$100.00	$12,800
	Construction Contingencies (30% of Component Cost)	1	LS	$4,147.20	$4,147
	SUBTOTAL				**$17,971**
TOTAL ESTIMATED PROJECT COST					
ALTERNATIVE 1 - LONGITUDINAL STONE TOE BANK PROTECTION					**$118,981**
ALTERNATIVE 2A - LONGITUDINAL STONE TOE AND SLOPE FLATTENING 3:1					**$206,327**
ALTERNATIVE 2B - LONGITUDINAL STONE TOE AND SLOPE FLATTENING 2:1					**$156,507**
ALTERNATIVE 3 - ADD GRADE CONTROL STRUCTURES TO ALTERNATIVE 2A					**$224,298**

APPENDIX A
Photos from Site Visit 10-13-06

1. Storm drain at the end of Taegan Lane (Does not flow into the Conservation Bank watershed.) – Looking West

2. Upstream approach to the pedestrian bridge. – Looking Northwest

3. Downstream of the pedestrian bridge – Looking South

4. Downstream of the pedestrian bridge. – Looking South

5. Downstream of the pedestrian bridge. – Looking South

6. Upstream approach to the pedestrian bridge. – Looking Northwest

7. Upstream approach to the pedestrian bridge. - Looking West

8. First existing grade control structure. - Looking North

9. Downstream of first existing grade control structure. - Looking South

10. Downstream of first existing grade control structure. - Looking South

11. Downstream of first existing grade control structure. - Looking South

12. A drainage path on the west slope of the drainage area. - Looking West

13. Head cutting on the west bank. - Looking Southeast

14. Upstream of second grade control structure. - Looking South

15. Second existing grade control structure (covered by willows). - Looking North

16. Third existing grade control structure. - Looking Northeast

17. Downstream of the major erosion problems. - Looking South

18. Downstream of the erosion problems. - Looking South

19. The existing landfill detention basin and spillway. - Looking North

20. Scour through the embankment of a historical detention basin near the landfill. - Looking West

21. Encinitas Landfill - Looking Northeast

Appendix B
Hydrology Model, Hydraulic Model, & Geographic Information System (GIS) Data