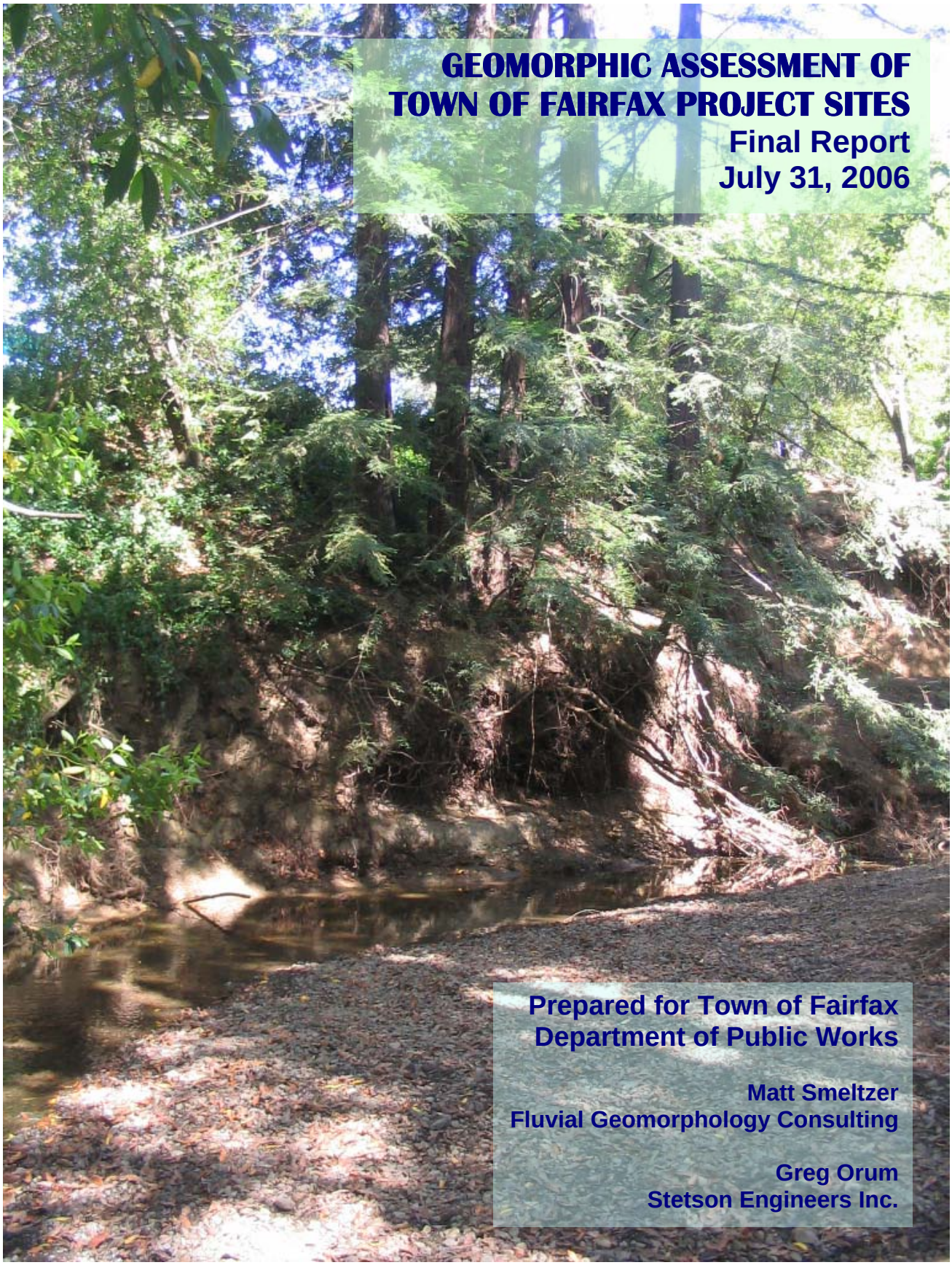

GEOMORPHIC ASSESSMENT OF TOWN OF FAIRFAX PROJECT SITES

Final Report
July 31, 2006

Prepared for Town of Fairfax Department of Public Works

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F L U V I A L G E O M O R P H O L O G Y C O N S U L T I N G

GEOMORPHOLOGY HYDROLOGY LANDSCAPE ARCHITECTURE PLANNING ENGINEERING DESIGN

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Cover photo:

Rapidly eroding left bank at outside bend of Fairfax Creek in Peri Park, June 4, 2006.

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SUMMARY

Fluvial Geomorphology Consulting (FGC) was contracted by the Town of Fairfax to conduct fluvial geomorphic assessments of Upper San Anselmo Creek and Fairfax Creek for evaluating the potential of biotechnical bank stabilization and riparian and aquatic habitat protection and enhancement as part of necessary repair work at six Town project sites that experienced damage due to flooding on December 31, 2005:

San Anselmo Creek
Spring Creek Culvert Replacement
Creek Road Bridge Foundation Reconstruction and Bank Repair
Pastori Avenue Storm Drain Outlet Repair

Fairfax Creek
Sir Francis Drake Blvd Culvert Outlet Repair
Peri Park Tennis Courts Bank Repair
Peri Park Footbridge Replacement and Bank Repair

Stetson Engineers assisted FGC in conducting detailed field geomorphic assessments of Upper San Anselmo Creek and Fairfax Creek in the vicinity of the project sites. The FGC-Stetson field team evaluated the December 31, 2005 flood's overall impact on the riparian corridor, and compiled field observations of bank erosion and other field data for putting the six Town of Fairfax project sites into an appropriate watershed- and reach-scale physical perspective for forming site-specific design recommendations.

The field team used available hydrology and hydraulic model data and field-measured December 31, 2005 flood high water marks (HWMs) to estimate flood depths at each site and the approximate return interval of the flood. The December 31, 2005 flood depth may be used as a minimum design elevation for bank erosion protection at vulnerable sites. The team also mapped the alluvial soil types exposed in creek banks near the project sites to characterize typical geotechnical characteristics for site design evaluation, as would affect appropriateness and design flexibility of biotechnical techniques. The field team also identified and measured the width of San Anselmo Creek and Fairfax Creek where the channel is wide enough to allow fine sediment to deposit along channel margins. The local average of these measured channel widths is reported as the recommended minimum design bankfull channel width for project sites.

Finally, FGC identified potential flood flow conveyance constrictions at existing culverts and within natural channel section, and inventoried potential flood management improvement projects in the Town of Fairfax. FGC considered the

potential design implications at the six Town project sites of implementing these and other potential flood management improvement projects.

This report first summarizes the field methods and uses field data highlights to briefly characterize the overall geomorphology and general project design considerations for San Anselmo and Fairfax Creeks. The complete field data inventories, summaries, air photo site location exhibits, hydrology, hydraulic, and sediment stratigraphy data are included as appendices. This report then provides site descriptions and site-specific design recommendations for each of the six Town of Fairfax project sites.

METHODS

Field basemaps

FGC prepared air photo field basemaps from the County of Marin 2004 digital orthophotos overlain by the 2004 5-ft interval topographic contours, property boundary data, and a channel centerline alignment with longitudinal channel centerline stationing according to the Town of Fairfax Flood Insurance Study (FEMA 1977).[1] Using the 1977 FIS channel centerline stationing allows the field-measured December 31, 2005 high water mark (HWM) data to be compared directly to FEMA-measured bed elevations and calculated flood depths for various return interval flood discharges. Any changes to the planform alignment or total length of the creek centerlines from 1977 to 2006 would be negligible. Stetson Engineers (2000) found that changes to the bed elevation between 1977 and 1999 at FEMA-measured HEC-2 model cross-sections were minor.

Appendix A and Appendix B contain air photo field basemaps that locate the individual Town of Fairfax project sites and show the distribution of existing stabilization structures and recent bank erosion conditions along the San Anselmo Creek and Fairfax Creek corridors, respectively.

Field distance measurement

Matt Smeltzer of FGC and Greg Orum of Stetson Engineers walked the 13,580-ft length of San Anselmo Creek within the Town of Fairfax limits on May 9-11 and May 23. The field team walked the 9,400-ft length of Fairfax Creek within the Town of

[1] Note that the 2004 5-ft contour interval data are not accurate enough within the riparian corridor for individual project site planning or design purposes.

Fairfax limits on June 3-4. The field team used a hip-chain to measure distance along the channel and tabulate the locations of individual sites, and locations of other measurements and characterizations according to 1977 FIS centerline stationing. The hip-chain measurements for individual locations are accurate to within +/- 1-2 ft. Differences between hip-chain measured distance and 1977 FIS centerline stationing distance were determined between each successive culvert/bridge, and the hip-chain distances were rectified to correspond exactly to FIS stations at culvert/bridge inlets and outlets. Typical differences corrected by rectification were on the order of 1-10 ft over 1,000 ft hip-chain measured distance.

Creekbank sediment stratigraphy mapping

Where bank sediment was exposed or could be discerned and distinguished beneath vegetation cover, the field team measured and tabulated the vertical height of individual bedrock and alluvial sediment material units (facies) using the adjacent water surface elevation as zero datum, such that the sum of all material unit heights at a measurement point is an approximation of the bank height. The field team distinguished between finer, unconsolidated silty-clayey-sand alluvium typically forming the upper banks (A horizon), from the denser materials typically forming the lower banks nearer the water surface, including partially consolidated sandy-clayey-gravel alluvium (B horizon), and consolidated clay hardpan (C horizon), and sandstone, shale, or chert bedrock, where present. Coarse, unconsolidated colluvial materials occurred where banks were formed from landslide and debris flow deposited materials.

Appendices E and F contain the facies and bank height field data plotted on the adapted 1977 FIS channel bed elevation and flood longitudinal profiles.

Stormwater drainage outlet inventory

The field team identified the location, dimensions, and general condition stormwater drainage outlets and natural tributary streams discharging to the study reaches. The field team tabulated the creek centerline station (ft) of the drain outlet, the approx. size and type of drainage structure, and noted the structural condition of the outlet. The field team also considered whether or not the drainage outlet design or structural condition causes any long-term ongoing or recent affects on bank stability.

Appendix H contains an inventory of existing stormwater drainage outlets discharging to San Anselmo Creek and Fairfax Creek within the Town of Fairfax limits, including a brief description of the design and structural condition of the facilities.

Equilibrium channel bankfull width estimates

The field team identified creek segments in the study reaches where the active channel is wide enough to allow fine sediment to deposit along the channel margins, typically in straight reaches or along inside bends where there are narrow gravel bars and/or floodplain bench surfaces. The team tabulated the channel width along measured along a line approximately 2-3 ft above the water surface, from the opposite bank to the inside edge of the fine sediment deposition. The average of these data values can be taken as a proxy for "equilibrium" bankfull channel width, and used as a target, minimum design value for bank stabilization and other bank modification projects in the riparian corridor. Equilibrium width data are summarized in Appendix L and minimum design active channel width is discussed in the general design considerations section of this report.

Existing active floodplain inventory

The field team also identified active floodplains – approximately flat, variably vegetated, gravelly or sandy alluvial deposits flanking the channel – that were apparently inundated by the December 31, 2005 flood. A surface was assumed to be an active floodplain if the height of the surface above the water surface was less than the height of December 31, 2005 high water marks in the vicinity. The team tabulated the beginning and ending point creek station (ft) and average heights above the water surface of individual floodplains.

Appendix I contains an inventory of existing active floodplain units on left bank and right banks of San Anselmo Creek and Fairfax Creek within the Town of Fairfax limits.

Canopy enhancement site inventory

The field team identified sections of the study reaches where riparian canopy enhancement projects are recommended to create new, or increase existing overhead riparian tree canopy cover needed for reducing large areas of unobstructed solar penetration into the open water column. The team also considered reaches dominated by non-native and/or apparently senescent mature riparian trees as potential candidate canopy enhancement project sites. The team tabulated the beginning and endpoint point creek station (ft) of individual canopy enhancement sites, and characterized the existing condition of the creek bank and bank stabilization structures within the creek segment. Site descriptions included

general considerations for planting the site, thought to be sufficient for project conceptualization and first-order cost estimates for grant proposals.

Appendix J contains a summary inventory of recommended canopy enhancement projects on San Anselmo Creek and Fairfax Creek within the Town of Fairfax limits.

December 31, 2005 high water marks inventory

The field team identified and measured the height above the water surface of existing December 31, 2005 flood HWMs.[2] The team tabulated the creek station (ft) and HWM height for individual HWMs and plotted the HWMs as flood water surface elevation data points on the longitudinal channel bed profile charts adapted from the FIS. This allows direct comparison of the actual water surface elevation profile of the December 31, 2005 flood to FEMA-calculated water surface elevation profiles for various estimated return interval discharges. The field team did not inspect top of bank areas in floodprone reaches to identify any additional HWMs outside of the main channel.

Appendix K contains an inventory of December 31, 2005 HWM data collected along the San Anselmo Creek and Fairfax Creek corridors within the Town of Fairfax limits. The December 31, 2005 HWMs and estimated peak flood water surface elevation are plotted on longitudinal profile charts adapted from the 1977 FIS for direct comparison to FEMA-calculated water surface elevation profiles contained in Appendices E and F.

Flood flow constrictions

To the extent feasible in known or suspected floodprone reaches, the field team compared the HWM heights to adjacent bank heights to roughly identify and delimit flood flow exit points and overflow zones. The field team attempted to identify multiple HWMs where present immediately upstream from bridge culverts in floodprone reaches, to determine the severity and extent of the backwater effect exerted by the culvert, if any, during the December 31, 2005 flood.

The field team also inspected the upstream and downstream conditions in the vicinity of culvert and bridge structures in floodprone reaches. The team measured and tabulated the width and height of culvert openings, and characterized the probable hydraulic conditions affecting culvert capacity compared to theoretical

[2] HWMs are reliable indicators of maximum instantaneous flood water surface elevation, such as floated vegetative debris attached to stout tree limbs, or attached to existing bank stabilization structures.

maximum capacity, including: the potential effect of vegetation and gravel bars in entrance and exit flow paths and flow approach angle on the entrance and exit energy losses and effective flow width within culverts; the potential effect of hanging utilities within culverts on effective opening height; and, the potential effect of non-existent or poorly designed wingwalls on entrance and exit energy losses. The bridge/culvert hydraulic characterizations compiled in this report are intended to inform but not completely substitute for hydraulic model calculations generally needed for flood management improvement project design.

The field team also identified reaches of San Anselmo Creek and Fairfax Creek within the Town of Fairfax limits where the bankfull channel width was substantially narrower than the approximate "equilibrium" bankfull channel width and/or the top of bank channel width was substantially narrow than upstream and downstream channel sections – narrow sections that appeared by visual inspection to possibly cause a flood backwater effect similar to that of an undersized bridge/culvert opening. Examples of potential flood constriction sites included reaches where an existing bank stabilization structure appeared to be constructed within former waters of the U.S. and thereby substantially reduced the width of the channel, and locations in relatively narrow or existing structure-constrained reaches where a group of mature riparian trees occupy more than approximately 25 percent of the cross-section area below flood stage.

The potential flood management improvements of tree removal or modification of an existing encroached bank stabilization structure were among the many considerations the team took into account in forming an "ACTION" or "NO-ACTION" recommendation and specific design recommendations for a potential bank stabilization project site.

Appendix L lists existing culvert opening dimensions and measured minimum channel widths within artificially narrow channel sections. Appendix G contains preliminary recommended flood management improvement projects. Note that the reported lists of potential flood flow conveyance constraints and recommended flood management improvement projects are neither exhaustive nor informed by formal hydraulic model and cost-benefit analyses.

Existing bank stabilization structure delineation and classification

The field team delineated the length of the left bank and right bank[3] of the study reaches into either individual existing "structures" or existing "sites" according to

[3] Note that the engineering/geomorphology convention for determining left bank and right bank is looking downstream, whereas the fisheries biology convention is looking upstream.

where there are existing bank stabilization structures and where there are not. The field team tabulated the creek station (ft) at the beginning and ending points of all existing bank stabilization structures, including distinguishing between adjacent bank stabilization structures of different construction types or apparent ownership. The most common structure types include:

RR BANK	Rock rip-rap placed on the bank slope to a height more than 4 ft above the water surface.
RR TOE WALL	Rock rip-rap placed on the bank slope to a height less than or equal to 4 ft above the water surface.
VERT CONC RET WALL	Vertical or near vertical poured concrete retaining wall up to a height more than 4 ft above the water surface, with or without an L-shaped footing or concrete apron near the water surface elevation.
VERT TIMBER RET WALL	Vertical or near vertical wall constructed of vertical timber piers and horizontal timber planks, with or without a concrete footing or RR toe wall protection at the base of the wall near the water surface.
GABION WALL	Steeply, typically 1(H):2(V) sloped retaining wall formed by stacking onto the bank horizontal courses of rectangular prism-shaped heavy-gage galvanized wire baskets containing facing class or light class rip-rap ballast.
RUBBLE WALL	Near vertical, typically 1(H):4(V) sloped bank erosion protection formed by stacking dry or grouted broken concrete pieces on the bank.
SACKRETE WALL	Variably-sloped retaining wall or bank erosion protection formed by stacking 40-lb to 120-lb paper bags of dry concrete or cement mix in horizontal courses on the bank, typically pinned together with vertical steel rebar reinforcement.
CULVERT/BRIDGE	Any of several roadway crossing structures including natural- and concrete-bottom rectangular concrete box culverts, arch culverts, and large-diameter corrugated metal pipe (CMP), with or without vert

conc ret wall or other type of wing walls forming both banks upstream and downstream.

The field team tabulated the estimated height above the water surface of each individual structure and its general structural condition. The field team also identified failed or failing portions of structures and considered whether or not modification or replacement of failed/failing structures would reduce erosion pressure on affected upstream and downstream sites or enhance the riparian canopy in a canopy-limited reach.

Where there were multiple structures on one section of creek bank, the delineation is according to the structure type nearest to the water surface. Typical structures appear to consist of materials placed directly on the pre-project creekbank surface, such that the structure would have encroached into then-existing waters of the U.S. by a minimum amount comparable to its thickness, typically 3-4 ft. The field team noted instances where individual structures appeared to encroach more than that typical amount, and characterized the physical effects of encroachment on upstream and downstream sites, if any.

The individual structures are numbered sequentially increasing from the downstream to the upstream end of the Town of Fairfax limits (e.g., S-1, S-2, etc.). Appendix G contains location data and brief text descriptions of individual structures. Appendix A and Appendix B contain air photo maps that show the locations of individual structures along both banks of San Anselmo Creek and Fairfax Creek, respectively.

Site stability delineation and classification

Wherever there are not structures, the still natural banks are considered “sites” that are delineated according to apparent recent and immediate future erosion/stability conditions. The field team tabulated the creek station (ft) at the beginning and ending point of each site according to the following classification:

BR BANK	Stable native sandstone, shale, or chert bedrock exposed in bank to a height above the adjacent water surface exceeding 3 ft.
BR BANK/OC	Stable native sandstone, shale, or chert bedrock exposed both on the channel bed and on the bank to a height above the adjacent water surface exceeding 3 ft.

CLASS A	Creek channel banks show none or negligible recent bank erosion or upper bank slump failures.
CLASS B	Creek channel banks show frequent and/or recent minor bank erosion and/or upper bank slump failures that overall appear somewhat self-stabilized or self-stabilizing and *do not* appear subject to additional major erosion that would damage existing public or private property or threaten to erode mature riparian trees important to the existing tree canopy cover in the reach.
CLASS B*	Creek channel banks show recent major bank erosion and/or upper bank slump failures that *do not* appear self-stabilized or self-stabilizing and instead appear subject to additional major erosion in the immediate future, especially as would damage existing public or private property and/or threaten to erode mature riparian trees important to the existing tree canopy cover in the reach.
CLASS C	Creek channel banks with severely undermined existing mature riparian trees at mid-bank or near top of bank that, if not structurally stabilized or heavily pruned, appear likely to erode into the channel in the immediate future and as a result significantly reduce the existing tree canopy cover in the reach, and/or threaten to form flood constricting debris jams, and/or damage public or private property.

The individual sites are numbered according to stability class increasing from the downstream end to the upstream end of the Town of Fairfax limits (e.g., A-1, A-2, etc.). Appendix G contains location data and brief text descriptions of individual sites. Appendix A and Appendix B contain air photo maps that show the locations of individual sites along both banks of San Anselmo Creek and Fairfax Creek, respectively.

ACTION / NO-ACTION recommendations

The field team considered the overall watershed-scale physical effects of the December 31, 2005 flood and multiple reach-scale and site-specific factors to

provide a best available information ACTION or NO-ACTION recommendation for each of the more than approximately 700 individual structures and sites delineated within the Town of Fairfax limits.

ACTION was generally recommended at sites or existing severely failed or failing structures where bank stabilization appeared necessary to prevent against immediate future bank erosion of such a severe nature that the erosion would likely: significantly damage existing public infrastructure or private property; reduce existing riparian tree canopy cover in canopy limited reaches; and/or increase bank erosion pressure at nearby vulnerable sites.

ACTION was also generally recommended at structures and sites if it would feasibly eliminate flood flow constrictions that appeared from HWM measurements and other field observations to cause a local backwater effect contributing to flooding of public and private property adjacent to the riparian corridors on December 31, 2005.

NO-ACTION was almost always recommended at stable sites and existing bank stabilization structures in overall fair to good condition. NO-ACTION was also generally recommended at sites with frequent minor or recent erosion or failing structures that appeared to be self-stabilizing and therefore not threatening to existing public or private property, or canopy-forming trees. NO-ACTION was also often recommended at severely eroding sites in cases where no property or important canopy-forming trees appeared threatened – i.e., stabilization projects that appeared to only reduce chronic sediment delivery to the channel. NO-ACTION was recommended at sites where stabilization appeared infeasible due to limited construction equipment access or difficult geotechnical conditions.

ACTION/NO-ACTION recommendations for all existing structures and sites are listed in Appendix G. Corresponding site locations are shown in Appendices A-B. Ground photographs of many existing sites are compiled according to creek station (ft) in Appendices C-D.

Site-specific design recommendations

The field team prepared brief conceptual bank stabilization and flood management improvement design recommendations for all ACTION sites and a small percentage of NO-ACTION sites. The design recommendations are intended to be detailed enough for feasibility analysis, cost estimate development, master plan development, and to inform design evaluation and permitting by regulatory agencies. The design recommendations neither suffice for design drawings, site analyses, and project descriptions typically needed for regulatory permitting, nor for geotechnical,

structural, and hydraulic engineering analyses typically needed for detailed design, cost-benefit, and factor of safety calculations.

The site-specific design recommendations were generally formulated assuming the most biotechnical bank stabilization technique would be preferable by the landowner and regulatory agency personnel, but only if it appeared feasible to implement at a site. Biotechnical techniques are most successful when it is feasible to lay back (i.e., excavate and off-haul excess bank material) and/or build up (e.g. place mechanically stabilized earth, soil lifts) the bank from near the water surface elevation (WSE) or ordinary high water (OHW) up to the top of bank (TOB) to achieve a maximum feasible, stable slope for the site.[4] However, it appeared feasible to lay back the bank to a stable approx. 1.5(H):1(V) or lower design slope at only a small percentage of actively eroding sites. First, the individual landowner's priority for protecting existing TOB property in its existing condition cannot be determined by the assessment – laying back the bank generally requires conversion of flat, generally landscaped terrace property to relatively steep creek bank property, typically perceived as a "loss" of private property. Second, bank erosion focuses at the lower and mid-bank portion of the banks below the maximum flood depth at a site. The upper bank may intact at an actively eroding site, particularly if it is densely vegetated by existing mature CA native riparian canopy-forming trees – only in high priority bank stabilization situation would it be preferable to lay back the bank at the expense of existing canopy-forming trees near the TOB. The design site-specific design recommendations are optimistic in the sense that they generally err on the side of assuming loss of TOB private property is negotiable at any given site to the extent that it increases the feasibility of environmentally best, and generally lower cost, biotechnical bank stabilization techniques. There are still other limitations for the suitability of these site-specific design recommendations that are discussed in the subsequent section of this report.

Limitations of ACTION/NO-ACTION and site-specific design recommendations.

The ACTION/NO-ACTION and site-specific design recommendations provided in this report necessarily suffer from a number of important limitations. First, the private landowner's priority for TOB private property protection and willingness to sacrifice a few to several horizontal feet of property to increase long-term bank stability and success of riparian vegetation establishment could not be assessed.

[4] In general, the steeper the bank, the less stable it is. Naturally stable banks composed of native alluvial soils occur along San Anselmo Ck and Fairfax Ck, where the bank slope ranges between 1(H):1(V) (steeper, less stable) to 2(H):1(V) or less (less steep, more stable). The maximum stable slope at any given site depends on the bank material, bank height, stabilization material, channel position, and other factors.

Second, individual properties are closely spaced along the creek corridor and exact property boundaries could neither always be inferred from fences and other existing structures, nor from property boundary maps of unknown accuracy. The feasibility of site-specific design recommendations is therefore often subject to cooperation between multiple neighboring owners. For example, a landowner cannot lay back the bank on their property from near vertical to a gradual slope without impacting the stability of her immediate downstream neighbor.

Third, site-specific design recommendations could not be logically determined in reaches where there were multiple recommended ACTION sites on multiple properties, particularly in narrow reaches and/or reaches in the vicinity of evidently undersized culverts likely to be replaced to improve flood management. For example, the upstream portion of the approximately 300 ft-long reach of Fairfax Ck between Spruce St and Merwin Ave culverts is narrow with near-vertical actively eroding banks threatening numerous existing canopy-forming native riparian trees along both banks. The reach needs to be made wider for the banks to be made stable using biotechnical techniques, but the extent to which each of the multiple owners is willing to allow existing TOB landscaped terrace property and trees to be modified by individual or coordinated project cannot be assessed. This situation is complicated by the poor alignment, site configuration, and limited flood flow capacity of the existing downstream Merwin Ave culvert. If Merwin Ave culvert were replaced with the objective to safely passing the 100-year flood discharge through the reach, the existing multiple ad-hoc bank stabilization structures upstream and downstream from the culvert site would need to be severely modified and/or removed to allow the overall site to be configured properly. Culvert replacement would therefore have major design implications for the upstream 300 ft-long reach.

Another example is Peri Park, where multiple recommended ACTION sites were delineated. Biotechnical techniques are feasible at the site, but would require a large-scale channel and floodplain restoration component through the park site in order to be made long-term stable. Yet, the alternatives are unsightly and expensive vegetated rip-rap or vegetated log crib wall banks, which may not be consistent with public desire for maintaining a natural appearance in the park. Site-specific design recommendations with the park require a comprehensive reach-scale creek and floodplain restoration and management plan.

Fourth, site-specific design recommendations do not strictly consider the landowner's ability or willingness to pay for the cost of design, permitting, and construction. Construction cost is generally minimized by recommendation of the most biotechnical (generally least expensive) technique. However, more expensive stabilization techniques were recommended at sites where the channel was too narrow to allow for a biotechnical design approach, or where existing TOB riparian canopy-forming trees appeared to be a priority. In addition, the recommendations

often include optional cabled rip-rap large woody debris (LWD) structures to encourage scour pool formation for habitat enhancement and deposition of gravel bars to create planting sites for CA native willow and alder along the toe of erosion prone banks. However, few, if any, local design engineers are familiar with design of cabled rip-rap LWD structures, and few local contractors are experienced with installation. Marin Municipal Water District (MMWD) may be the most experienced, having installed several cable-anchored LWD structures in the Lagunitas Creek watershed.

Fifth, close proximity of housing and existing trees can prevent access to TOB for typical construction excavation equipment needed to cheaply and quickly construct projects according to the site-specific design recommendations. Physical access for construction equipment could not always be assessed from the creek. The alternative of operating equipment in the channel to access the site from an upstream or downstream location increases potential impacts to fish habitat.

Sixth, the ACTION/NO-ACTION and site-specific design recommendations have not been informed by results of concurrent Biological Assessment of the study reaches. Both the geomorphic and biological assessments should be considered in planning, design evaluation, and permitting for individual proposed projects.

Seventh, the recommendations were not prepared using geotechnical engineering calculations to estimate the slope stability at the site and factor of safety against bank failure under both existing conditions and proposed design conditions. Geotechnical analysis requires soil boring samples be taken at depth from the site and sent to a testing laboratory to have the soil's strength and cohesion characteristics quantified as input for standard geotechnical engineering calculations that explicitly consider earthquake loading and fully saturated soil conditions.

Eighth, the recommendations include actions that would likely ameliorate apparent existing flood management problems, but the flood management related recommendations are only preliminary. The actual number, locations, detailed design, and sequence of flood management improvement projects needed in the watersheds needs to be informed by formal hydraulic model, geotechnical, engineering cost-benefit analyses.

Due to the limitations discussed above, neither the ACTION/NO-ACTION nor the site-specific design recommendations can substitute for a complete site analysis and design. The strength and value of these recommendations is their comprehensive watershed-scale formulation following the December 31, 2005 flood. The priority embodied in the ACTION/NO-ACTION recommendations is therefore as true as possible. The feasibility of implementing biotechnical techniques at an individual site embodied in the site-specific design recommendations is informed by the

performance of similar existing structure and natural channel banks at various channel positions along the corridor during the December 31, 2005 flood.

GENERAL DESIGN CONSIDERATIONS FOR SAN ANSELMO AND FAIRFAX CREEK SITES

Hydrology and Probable Flood Depth

Table 1 summarizes the estimated 10-yr, 50-yr, 100-yr, and 500-yr return interval peak flood discharges at various points along the San Anselmo Ck and Fairfax Ck channel. Appendices E-F show that the December 31, 2005 flood HWM elevation profiles (also tabulated in Appendix K) roughly correspond to the FEMA-calculated 50-yr return interval flood water surface elevation profiles for both creeks. This suggests that the peak discharge of the December 31, 2005 flood at the Town limits was about 3,000 – 3,300 on San Anselmo Ck and 1,400 – 1,500 on Fairfax Ck.

Table 1

Estimated 10-year to 500-year Peak Flows within Town of Fairfax
(Source: Federal Emergency Management Agency, 1977)

	Drainage Area (sq mi)	Peak Discharge (cfs) 10-yr	50-yr	100-yr	500-yr
San Anselmo Creek					
imm d/s fr Fairfax Creek trib	9.1	1,970	3,100	3,500	4,500
imm d/s fr Deer Park Creek trib	5.0	1,080	1,780	2,100	3,000
imm d/s from Wood Lane Creek trib	4.2	930	1,620	1,900	2,780
at FIS model cross-Section P	3.7	800	1,420	1,590	2,350
at Cascade Preserve (Town Limits)	3.1	725	1,300	1,480	2,100
Fairfax Creek					
at confluence with San Anselmo Ck	4.1	850	1,450	1,720	2,400
imm d/s fr Bothin Ck	3.4	690	1,200	1,450	2,000
at White Hill School (nr Town Limits)	1.8	450	770	960	1,600

The December 31, 2005 flood depth can be determined for any given site by interpolation between measurements tabulated in Appendix K, or by visual inspection of HWMs and FEMA-calculated flood water surface elevation profiles charted in Appendices E-F. For most sites, the actual elevation of the December 31,

2005 water surface can be used as the minimum design top elevation for new proposed bank stabilization structures intended to prevent against erosion by direct flow attack.

Minimum recommended design channel width

The field team measured reach-average bankfull widths at 13 locations along the study reach where there were with well-established channel bed forms, relatively stable channel banks, and evidence of recent fine sediment deposition along channel margins. The average of these values, 28.5 ft, is the minimum recommended design bankfull channel width in at San Anselmo Creek sites.

The field team identified only three reaches with qualifying conditions in the Fairfax Creek study reach. The average of these values, 18.0 ft, is the minimum recommended design bankfull channel width in at Fairfax Creek sites.

It is generally not recommended to allow new proposed bank stabilization structures to encroach into the channel to the extent that the post-project bankfull channel width would be less than the recommended minimum. However, multiple reach-scale factors need to be taken into account at any given site. For example, some reaches are substantially narrower than the recommended minimum width with or without the proposed project. These reaches are naturally unstable and should be made stable not with multiple individual stabilization projects, but with single, reach-scale, coordinated multiple ownership projects that maximize opportunity to widen the channel and restore gravel bars and floodplain benches. The opportunities for widening the creek will typically be at inside bend channel positions – it may not be feasible to require sites at outside bend channel positions to maintain near vertical bank slopes. Appendix L lists reach-average minimum bankfull channel widths in narrow reaches the field team identified as potential flood flow constrictions. These data can be used to identify many of the existing narrow reaches in the watersheds.

Conversely, it may be prudent to allow new proposed bank stabilization structures to encroach into the channel in reaches where the post-project bankfull channel width will still be less than the recommended minimum. This would particularly be true at sites with very steep or near vertical banks where the landowner or agency personnel put a high priority on protection existing TOB private property uses or existing mature native riparian canopy-forming trees near the TOB.

Sedimentology and geotechnical characteristics

The field team identified and measured natural alluvial and fill sediment facies exposed in channel banks along the study reaches. Appendices E-F plot the top elevations of the various exposed strata at the measurement locations, from which the stratigraphy profiles can be roughly interpreted.

San Anselmo Creek banks are typically cut in hillslope bedrock along the southern edge of the valley or terrace alluvium forming the narrow valley floors. The alluvium consists of an upper horizon (A horizon) of fine-grained relict floodplain deposit of silty-clayey-sand which is very erosion prone due to its typically low strength and cohesion characteristics. A discontinuous 3-6 ft-thick layer of sandy-gravel alluvium (B horizon) underlies the relict floodplain materials. These are former channel bed materials exposed by long-term and recent channel bed incision. B horizon gravels are typically relatively dense and erode much slower than overlying A horizon fines. Beneath the gravels, virtually everywhere exposed along the toe of the banks or shallowly underlying the channel bed of San Anselmo Creek is a medium to high density brown-colored clay (C horizon), that is the extremely weathered sandstone bedrock underlying the valley floor and much of the watershed.[5] The clay is very erosion resistant and serves, along with exposures of resistant native bedrock, to regulate the rate of channel bed incision and widening along reaches of San Anselmo Creek between culverts and other grade controls.

In some cases, either exposed dense gravel alluvium or weathered bedrock clay can provide a suitably stable foundation for biotechnical bank stabilization techniques designed to reduce continuing erosion of the overlying fines. Where feasible, the stability of the gravel and/or clay toe material can be improved by installing cabled RR-LWD structure(s) along the toe of the bank to deflect erosion pressure from the bank and encourage deposition of modern gravel along the toe of the bank where willow and alders can be planted and established. The depth that the channel bed has incised into the gravel and clay horizons varies along the reach, from only 1-2 ft at places to 6-10 ft in others. The December 31, 2005 flood depths varied from 5-6 ft in some reaches, to 8-10 ft in others.

The upper horizon (A horizon) is typically 10-15 ft-deep and almost everywhere forms the upper banks of San Anselmo Creek. Moderate to high flood flows are deeper than the top of the exposed clay and gravel materials along most of the reach and can erode the toe of the overlying fine materials, and induce major slump

[5] Probably Cretaceous sandstone of the Franciscan formation. Where there is resistant bedrock exposed in the bed and banks, it is almost always grey shale, or red chert (metamorphic sandstone), both of which are also typical of the Franciscan formation and weather much slower than the brown sandstone.

failures of the weak A horizon materials. Even where the exposed height of the relatively dense clay and gravel materials is higher than the depth of moderate and some high flood flows, it is common for slow, progressive erosion of relatively dense clay and gravel along the toe to slowly, ultimately undercut and trigger upper bank failures in the overlying A horizon materials.

The alluvial stratigraphy of Fairfax Creek is different than that of San Anselmo Creek. There is apparently no distinct gravel horizon between the fine-grained floodplain deposited materials and the underlying clay (weathered bedrock). Moreover, it is very difficult to distinguish the overlying floodplain deposited material from the underlying weathered bedrock clay. The underlying weathered bedrock clay is exposed in the bed and banks throughout the entire study reach. The clay appears to be sandier, with a lower density, cohesion, and strength than the corresponding San Anselmo Creek clay. Fairfax Creek's current bed incision rate could be expected to be higher than for San Anselmo Creek, but it is more densely checked by box culverts, dams, and some resistant brown sandstone bedrock (evidently resulting from locally higher degree of metamorphism) outcropping in the bed between Azalea Drive and Spruce Street.

By comparison to San Anselmo Creek stratigraphy, it appears that the Fairfax Creek watershed, with its shallow deposit of floodplain fines, underwent channel bed incision and floodplain abandonment much later and more incompletely than Upper San Anselmo Creek. It is possible that the pre-European Fairfax Creek valley floor was dominated by a dense redwood forest with its shallow root system reinforcing shallow channel banks cut in the shallow floodplain deposit. Upper San Anselmo Creek cuts deeply against the southern edge of the valley floor where the oversteepened canyon walls periodically contribute massive landslide deposits to the valley floor that would have temporarily dammed the valley and initiated valley cut-and-fill sequences explaining both the depth of the relict floodplain deposits and the presence of the relict channel bed gravels.

SPECIFIC DESIGN RECOMMENDATIONS FOR TOWN OF FAIRFAX PROJECTS

The following specific design recommendations for the six Town of Fairfax 2006 proposed project sites are prepared in the context of: (1) post-flood field assessments of the larger study reaches; and (2) author interpretation of the current agency-accepted practices and standards for bank stabilization and culvert replacement. All of the six Town of Fairfax proposed project sites are priority projects as objectively determined in the context of the overall field assessments. All of the sites failed during the December 31, 2005 flood, creating conditions that compromise infrastructure stability and public safety.

Note that the design recommendations are based on Geomorphic Assessment only. The Town projects should be designed using these design recommendations as guidelines but not replacements for site analyses. The Town projects should also be designed, reviewed, and evaluated in the context of the Biological Assessment still underway for the sites.

Spring Creek Culvert Replacement

Spring Creek is a tributary to San Anselmo Creek draining a portion of the south-facing slope of Pam's Blue Ridge. At the time of the May 2006 field surveys, Spring Creek appeared to have the highest baseflow of any of the smaller tributaries to Upper San Anselmo Creek (i.e., except for Cascade Creek, Carey Camp Creek, Wood Lane Creek, Deer Park Creek, and Fairfax Creek).

Spring Creek flows from the steep hillslope area onto a gradually-sloped colluvial valley floor surface where it is a relatively shallow-depth gravel-bedded stream bounded by hillslopes and residential properties. The bankfull channel width is approximately 10-12 ft at its outlet and its active channel width is approximately 9-11 ft in the vicinity of the site.

Spring Creek then flows beneath Cascade Drive within an existing undersized and/or poorly designed culvert. The December 31, 2005 flood reportedly exceeded the existing culvert capacity and overflowed Cascade Drive, requiring road closure that potentially restricted residential and emergency vehicle access.

The Town of Fairfax plans to replace the existing culvert with a larger and/or more appropriately configured culvert beneath Cascade Drive in 2006. Important design objectives and general considerations for culvert replacement include:

- Providing capacity to pass the estimated 100-year peak flow discharge without overflowing Cascade Drive;

- Selecting culvert type, width and opening height, design slope, and design inlet and outlet locations and elevations that will create flow depths and velocities within the culvert and entrance and exit flow areas that meet minimum and maximum criteria for passing steelhead and Coho salmon which will possibly reinhabit Upper San Anselmo Creek during the design lifetime of the replacement culvert;

- Providing rip-rap slope protection or vertical concrete retaining wing walls that will protect the inlet and outlet of the replacement culvert while also

maintaining suitable flow hydraulics for fish passage, minimizing the extent of required hardscape bank protection, and allowing for riparian vegetation establishment.

The existing culvert site is not listed in the recent Corte Madera Creek watershed crossing inventory and fish passage evaluation study report (Ross Taylor and Assoc., 2006). However, the existing culvert appears too steep to provide for sufficient flow depths and maximum flow velocities to meet fish passage criteria.

Although it is reasonable to presume that the existing culvert inlet elevation was constructed at the then-existing pre-culvert bed elevation, the inlet appears higher than appropriate or necessary when viewed in the reach-scale context of the field-observed longitudinal channel bed profile through the crossing. That is, the shallow-depth, gradually-sloped channel reach immediately upstream from the culvert appears influenced by a small historical base level increase which may have been imposed, for example, by a more restrictive culvert/fill prism pre-dating the existing culvert. The channel reach immediately downstream is significantly deeper and considerably widened by historical bank failures.

The design replacement culvert slope should be significantly reduced compared to existing. This may be a significant design constraint because:

(1) The culvert outlet elevation cannot be raised without also reducing fish passage suitability. The design replacement culvert outlet elevation should be either the same elevation (grade) as the natural channel bed substrate immediately downstream, or (preferably) less than channel bed grade, so that the outlet is submerged by a relatively deep and long pool under low-flow conditions. The replacement culvert site design could include a boulder weir type structure to intentionally create a pool meeting specific minimum length and depth requirements. If so, the boulder weir structure must be both stable under high flow conditions and not configured to act as a potential fish passage obstruction itself, including if it were to partially fail under high flow conditions.

(2) The existing Cascade Drive roadbed elevation is not high enough above the existing channel bed elevation immediately upstream from the existing culvert inlet to accommodate many culvert types or culvert dimensions without: (a) raising the design finished grade roadbed elevation compared to existing; and/or (b) lowering the design culvert inlet elevation compared to existing.

Given these site constraints, the most naturalistic and long-term sustainable approach to improving flood capacity and fish passage would be to restore the natural channel bed slope through the reach by substantially lowering the bed elevation upstream from Cascade Drive, and raising or keeping the same the bed

elevation downstream from Cascade Drive (natural grade alternative). A replacement clear-span bridge deck replacement or a bottomless arch culvert on reinforced concrete strip footings would span the finished natural sediment channel bed. However, the natural grade alternative would require the channel bed within and upstream from the existing culvert invert to be lowered by 3-5 vertical ft, gradually tapering to 0 ft at a location more than approximately 200-250 ft upstream from the culvert site. This amount of channel bed elevation adjustment would likely require removal of existing mature riparian trees bounding the existing shallow-depth upstream channel. And overall, the natural grade alternative would be an expensive treatment.

An appropriate lower cost alternative to a natural bottom arch culvert may installing a "squashed" CMP culvert with its inlet about 1-2 vertical feet lower than the existing culvert inlet elevation. This inlet lowering will cause comparable channel bed elevation adjustment (lowering) extending about 50 feet upstream, then gradually tapering from 1-2 to zero vertical feet channel bed elevation adjustment over another approximately 100 feet upstream. The length of the upstream channel bed elevation adjustment and required biotechnical bank stabilization could be reduced by installing a boulder weir type or cable-anchored LWD type v-shaped grade control drop structure within tens of feet from the culvert inlet.

However, the width of the culvert should approach approximately 1.3 times the active channel width, or about 12-13 ft in order to provide suitable fish passage conditions. Squashed culverts may not be stable or manufactured with dimensions approximately 12 ft wide by 5 ft high.

Given the above considerations, the best alternative for reasonable cost replacement providing for reasonably good fish passage improvement may be to replace the culvert with a pre-fabricated clear-span steel bridge deck resting on two new reinforced concrete wall abutments, spaced not less than 12-13 ft apart inside-to-inside dimension and both founded on reinforced T-shaped footings. The natural grade alternative appears cost prohibitive so that channel bed between the concrete abutments could be formed from a roughened ramp (i.e., densely packed rip-rap of a range of sizes, including a few large pieces protruding up from the bed into the flow to allow for complex near bed hydraulic conditions during floods similar to a step-pool type channel).

The appropriate balance between optimizing fish passage within the replacement structure and overall project cost needs to be informed by a biological assessment of the potential quality of the fish habitat upstream from the culvert site.

Creek Road Bridge Foundation Reconstruction and Bank Repair

Creek Road Bridge crosses a gradual bend of San Anselmo Creek. The existing crossing is not a flood constriction, but by normal geomorphic processes the December 31, 2005 flood eroded the outside bank (the left bank looking downstream) of San Anselmo Creek over a 122-ft-long reach -- upstream, within, and downstream from Creek Road Bridge.

Post-flood field evidence suggests that the flood scoured a gravel bar that had been attached to the left bank beneath the Creek Road Bridge, and undermined the toe of the left bank. Bank toe undermining combined with direct flow attack erosion in the mid-bank area contributed to massive bank failure along the entire 122-ft-long reach. The bank failure exposed the left bank abutment of Creek Road Bridge. The abutment appears to be a shallow reinforced concrete spread footing. It also appears possible that runoff from the nearby residential area overwhelmed storm drains and flowed into the creek on the creek bank immediately upstream from the bridge, contributing to bank erosion there.

General Site Design Considerations. The Creek Road Bridge spans an outside bend channel position with an 18 ft-high bank composed of fine (silty-clayey-sand) erosion prone material. The approximately 40 ft-long section upstream from the bridge structure is well-stabilized by emergency rip-rap placement and existing tree roots. Emergency-placed rip-rap along the upstream face of the bridge provides temporary protection for the exposed bank within the bridge structure, and in the approximately 60 ft-long section of the site downstream from the bridge structure for average to moderate high flows.

The exposed banks will likely continue to erode at a slow rate during the 2007 water year if there are average flows. However, if 2007 is a wet year, the additional high flows and long-term saturated bank conditions would likely cause a large failure in the downstream section. The stability of the bridge structure due to abutment foundation exposure cannot be determined from field observations. A full structural-geotechnical evaluation would likely recommend replacement of the exposed spread-type footing with a reinforced concrete wall abutment with an L-shaped footing, or reinforced concrete piers extending down to below the adjacent minimum channel bed elevation and/or stable (dense clay or bedrock) material.

Due to the outside bend channel position, a vegetated rock rip-rap bank is recommended for the entire 122 ft-long repair site. The December 31, 2005 flood crested 9 ft above the base flow water surface elevation in the vicinity of Creek Road Bridge, and showed no backwater effect from the bridge structure. Field observations suggest that rip-rap bank structures create stable bank conditions at outside bend channel positions on San Anselmo Creek if they: (1) are composed of

minimum ¼-ton to ½-ton mixed rip-rap material; and, (2) extend from below the channel bed elevation and extend up the slope at maximum 1(H):1(V) finished surface slope up to the approximate elevation of the December 31, 2005 flood water surface elevation.

It is therefore recommended that the rip-bank bank structure extend down to an elevation not less than 3 ft below the adjacent minimum channel bed elevation and up to an elevation approximately 9.0 ft higher than the base flow water surface elevation. The bank structure should be approximately 12-14 ft-high in total. The bank structure should be vegetated with live willow poles in the lower 3-5 ft-high exposed horizon.

There are few top of bank trees to be saved and closing the riparian corridor tree canopy is a low priority for this site, as there is already sufficient tree canopy in the project reach. The upper bank area above the rip-rap bank structure should be graded to minimum feasible slope given top of bank residential site constraints, hydro covered with biodegradable geofabric (e.g., North American Green C125BN, or equivalent), planted and irrigated. Plantings should include a selected mix of native riparian trees (1-gallon container plants) and mixed sun and shade tolerant native understory plants.

The existing fish habitat quality and use is not known, but the site does not currently provide any notable cover or pool habitat. Site-specific design consideration should be given to not eliminating any quality habitat features found by the concurrent fish habitat study, and to incorporate aquatic habitat improvements to the extent feasible. For example, incorporating large woody debris into the water surface elevation line of the rip-rap bank structure would likely produce a deeper scour pool with more overhead cover. Such structures should be expected to be buried in sand and gravel sediment much of the time, but frequently contribute to flood scour channel formation on the attached gravel bar surface along the perimeter of the rip-rap bank.

Cable-anchoring large woody debris pieces into a stable jam formation at the upstream end of the northerly bridge pier would likely create increased habitat complexity on the scoured gravel bar surface, and may increase the potential to scour a deeper more likely perennial pool along the rip-rap toe line at the downstream end of the site.

If these design recommendations are followed, there will be considerable net fill (import) at the site, particularly in the section within the bridge structure and in the downstream section of the repair site. Federal permits will be required to describe and mitigate for the required excavation and fill within waters of the U.S.

Specific design recommendations upstream from bridge. The 122-ft-long bank repair project reach begins at the downstream end of the 8-ft-high vertical concrete retaining wall at a residential property upstream, about 40 ft upstream from the upstream bridge face. Emergency-placed rip-rap forms an approximately 1(H):1(V) slope from the left edge of the active channel up to the top of the approx. 18 ft-high bank extending about 25 ft upstream from the upstream bridge face. The lower exposed 3-5 ft-high horizon of the emergency rip-rap area should be vegetated as far as feasible, although it looks like it will be difficult to vegetate. The roots of a mature native maple tree rooted near the toe of the bank secure the lower-bank over the 15 ft-long reach closest to the upstream retaining wall. It is recommended that the maple tree to be saved and both the bank upslope from the maple tree roots and the eroded section of exposed tree roots immediately downstream be stabilized by dense, irrigated plantings of live willow poles.

A 24-inch diameter storm drain outlet was observed within the emergency-placed rip-rap formed jetty along the upstream face of Creek Road Bridge. This storm drain will need to be reconfigured as part of the site repair so as to discharge near the top and directly onto the finished rip-rap slope. The alignment of the replacement storm drain culvert outlet should be skewed at least 15 degrees toward the downstream from perpendicular to the rip-rap bank line.

Specific design recommendations within bridge structure. First, the design recommendations for the bank within the structure depend somewhat on whether or not structural-geotechnical analysis of the exposed abutment/footing needs to be replaced. However, it is recommended that the 9 ft-high exposed rip-rap bank be continued through the bridge structure and downstream from the bridge. The existing active channel width within the structure is about 45 ft, substantially more than the 29 ft minimum recommended design active channel width for the San Anselmo Creek study reach. The physical effects on the reach of encroaching into the current active channel with the rip-rap bank are minimal. Therefore, there is considerable design flexibility in the potential configuration of any replacement abutment/footing structures. It may be sufficient, depending on structural-geotechnical engineering recommendations, to reinforce the exposed bank beneath the bridge with the rip-rap bank (i.e., gravity wall) to both prevent future bank erosion and prevent against slope failure beneath the existing exposed abutment/footing.

There are real design advantages to moving the rip-rap bank line and the resulting edge of water toward the creek centerline and away from the exposed abutment: (1) allows for a thicker rip-rap section to provide for a better drained and heavier gravity wall protecting against slope failure beneath the exposed footing; (2) more large woody debris can be safely and stably incorporated in a thicker rip-rap section; (3) aligning the design toe line of the rip-rap bank structure on a line approximately 45 degrees from perpendicular to the bridge face would reduce flood flows impinging

directly on the crescent shaped downstream section of the repair site; and (4) there appear to be top of bank constraints for reaching more stable upper bank slopes in the downstream section of the repair site, so moving the toe line out in the downstream section of the site will be beneficial overall to total site stability.

Specific design recommendations downstream from bridge. The approximately 60 ft-long downstream section of the repair site downstream from the bridge face is the most unstable. The 18-19 ft-high bank section is composed entirely from relatively fine silty-sand floodplain alluvium (Horizon A) material and high flood flows impinge more acutely on the crescent-shaped failure area than upstream sections of the site. It is recommended that the 1(H):1(V) maximum sloped 9.0 ft-high exposed rip-rap bank continue downstream to the end of the site in an alignment that allows for the upper bank to be constructed at a maximum finished slope of 1.5(H):1(V), if feasible given top of bank constraints.

Pastori Avenue Storm Drain Outlet Repair

Street runoff from the December 31, 2005 flood caused the suspended terminal section of an 24-inch CMP to fail and discharge stormwater directly onto the bank beneath the failed culvert, on the left bank (looking downstream) immediately downstream from Pastori Avenue Bridge. The culvert outlet needs to be replaced and the bank stabilized to prevent against another minor bank failure immediately downstream.

The Pastori Avenue Bridge crossing is a concrete-bottom reinforced box culvert that was modified to contain a steel-fabricated denil-type fish ladder in approximately the 1970s. Ross Taylor and Associates (2006) identified the Pastori Avenue Bridge fish ladder as a priority for replacement to improve fish passage in the watershed. Detailed conceptual fish ladder replacement designs are being prepared, including those that would preserve the existing abutments and footings for the bridge, and a “natural grade alternative” that would likely require complete reconstruction of the bridge structure. Construction of any of the alternatives would require biotechnical bank stabilization in the vicinity of the bridge, including the storm drain outlet site.

First, it is recommended that the triangle-shaped apron area beneath the culvert extension be lined with light class rip-rap, up to an elevation to protect the immediately downstream site that is currently covered with geofabric. The existing culvert outlet can be left as is if it discharges onto stable rock rip-rap lining. It is also possible that the culvert outlet could be replaced with a CMP extension discharging directly above the existing vertical concrete wing wall section at an invert elevation approximately 7 ft above the adjacent base flow water surface elevation (pool). The

replacement culvert section should be reinforced in place with steel post(s) embedded and grouted at depth in the light-class rip-rap lining.

The vertical concrete wing wall section forming the foundation for this recommended repair is failing. It would be significantly more expensive to replace the wall or to build up vegetated rip-rap bank below the wall. It is not recommended given the cost, the impact to the existing pool habitat at the foot of the wall, and the potential duplicity of the pending fish ladder reconstruction project.

The vertical timber retaining wall along the left bank immediately downstream from the vertical concrete wing wall is also beginning to fail. The closest mature top of bank bay tree to the storm drain repair site is above the upstream, nearly completely failed section of this wall. The tree is dead and will likely be removed to prevent potential damage to the nearby house. The pending fish ladder reconstruction project would likely include biotechnical bank stabilization from the existing abutment, extending downstream to include the failed portion of the timber wall. The existing canopy cover is excellent in this reach, and vegetation of the repair area is not recommended.

Sir Francis Drake Blvd Culvert Outlet Repair

The December 31, 2005 flood caused the downstream-most approximately 18 ft-long segment(s) of an 8 ft-diameter SSP culvert beneath Sir Francis Drake to detach and transport downstream, leaving a severely destabilized site in its place. The majority of the culvert fill and overburden was also evacuated from both streambanks by the flood, leaving a near-vertical raw earthen headwall above the culvert and very steep, bare soil streambanks on either side. The lost culvert section evidently did not compromise the stability of the roadway and pedestrian ROW, and mature top of bank trees appear stable along the roadway above the site. A steep coarse-bedded riffle formed downstream from the site from mixed facing-class and light-class rip-rap, evidently derived from the culvert failure. There are existing light-class rip-rap banks on both banks downstream from new culvert outlet; 16 ft downstream on the left bank, and 22 ft downstream on the right bank.

It is recommended that the raw banks and headwall be stabilized from the new culvert outlet downstream 16-22 ft to the respective left bank and right bank existing rip-rap banks. This could be accomplished in two ways: (1) adding new or refurbished 8 ft-diameter culvert section to the new culvert outlet of a length determined cost effective, not necessarily the same but approximately the same length as the pre-failure culvert segment, and ¼-ton rip-rap slope protection as wing walls blending into the existing rip-rap banks downstream; or (2) keeping the new culvert outlet as is, and reinforcing nearly in current form the steep, bare earth

headwall and streambanks with a standard vertical reinforced concrete headwall-wing wall structure. There are virtually no environmental differences between the two project types, and construction cost and construction access feasibility, should probably be the main design constraints.

The culvert is undersized and caused a backwater effect upstream that one right bank landowner reported was a problem for her site (6 Shemran). It appears from high water mark data and landowner conversations that the backwater caused shallow overflow over the culvert inlet headwall and onto Sir Francis Drake Boulevard. Modification and replacement of individual problem flood constrictions is a goal of the Ross Valley Watershed Management Initiative. It can be presumed that the problem flood constrictions will be addressed in approximately the same order as their priority in terms of the frequency the approximate cost of property damage caused by the December 31, 2005 flood. The extent of the property damage caused by this particular flood constriction was not ascertained by this study. It is possible if not likely that the failure of the culvert outlet was caused by flood overflow originating upstream from the culvert returning to Fairfax Creek over the banks and headwall at the pre-failure outlet. Total culvert replacement would likely only occur if the property damage estimated to have been caused by overflow from the inlet, or public safety concerns re. overflow onto Sir Francis Drake Boulevard, were to make it higher priority candidate for replacement for flood management.

According to Ross Taylor and Associates (2006), the existing (once 175 ft-long, now 156 ft-long) culvert is undersized and in extremely poor condition, but is a poor candidate for complete replacement with grant funding for fish passage improvement because it is a medium-priority fish passage barrier site in the Corte Madera Creek watershed. The entire length of the existing culvert is completely rusted through at the base flow water line and has subsequently been reinforced with concrete. The lower 35 ft-long section of the remaining culvert is polyvinyl coated.

Peri Park Tennis Courts Bank Repair

The December 31, 2005 flood flows significantly eroded the 8-9 ft-high right bank (looking downstream) of Fairfax Creek along nearly the entire length of the Peri Park Tennis Courts, leaving a very steep exposed earth bank that is near-vertical in places. The edge of the existing tennis courts fence line is now at the new top of bank, and extends over the new top of bank in places. The exposed bank material is the typical silty-clayey-sand with low to medium density near the edge of water. Because of the bank's oversteepened condition and relatively weak material, the bank will very probably fail in places along the tennis courts section during the upcoming 2007 or 2008 water years. Establishing vegetation on the bank and along

the toe of the bank would be very difficult because of the channel position, and although it may be sufficient to arrest the rate of bank undermining in upcoming years, it will not be sufficient to prevent against a slump bank failure. The existing tennis court surface shows that it had been settling over time along its edge along the creek before the December 31, 2005 flood. The settlement rate should increase because the bank is steeper than before the flood. The combination of upstream right bank vertical concrete retaining wall and left bank rip-rap bank appears to contribute to recent erosion at the site by increasing flow velocities in the reach and focusing more of the high velocity flow at the u/s end of the site, and at a more acute flow direction angle onto the bank.

It is recommended that the 113 ft-long reach be repaired to prevent against toe undermining, bank erosion by direct flow attack on the mid- and upper bank area, and rotational slump failure. The recommended repair site begins at the downstream end of the upstream grouted stacked broken concrete (rubble) wall on an upstream residential site, and extends 113 feet downstream to the upstream end of the rubble wall near the downstream end of the tennis courts. The downstream site end point is also the beginning point of the Peri Park Footbridge Replacement and Bank Repair site, described in a separate section below.

There are 3-4 mature riparian trees rooted near the top of bank along the tennis courts fence line that provide some canopy cover shading to the reach, but overall the existing canopy cover in the reach is very good and dominated by numerous TOB trees at the left bank. Therefore, saving the existing trees along the tennis court site is probably a low to medium priority. It is common to affix mesh material to chain-link tennis court fences to provide shade and reduce wind through the tennis courts. Affixing this type of mesh to the fence along the top of bank would likely replace shading by the existing trees in the event that the trees cannot be saved as part of the stabilization project.

Erosion by the December 31, 2005 flood increased the bankfull channel width along the repair site. A cross-section measured through the most severe bank erosion location (at the newly exposed light standard footing) shows that the flood deposited some fine sediment at the left edge of the now 18 ft-wide bankfull channel. It is recommended that the project design limit (to the extent geotechnically feasible) construction of hardscape encroaching onto the channel bed in order to maintain a 18 ft-minimum bankfull channel width along the repair site. (Recall that 18 feet is the minimum of all bankfull width measurements made at adequately wide cross-sections in the Fairfax Creek study reach.)

Because the existing post-flood bankfull channel is the same a the suggested 18 ft-minimum, vertical sheetpile or reinforced concrete retaining walls are recommended for stabilizing the bank along the length of the tennis courts repair site. An

acceptable lower-cost, somewhat biotechnical alternative to vertical walls would be a steep live willow pole vegetated ½-ton to 1-ton rip-rap wall, possibly meeting a 1(H):3(V) slope if carefully constructed. Such a wall would likely encroach into the channel a minimum of 3-4 feet more than a vertical wall. If this amount of encroachment onto the bed is allowed along the right bank repair site, then it would result in a comparable amount of bank toe erosion along the left bank across the channel and downstream from the tennis court site, probably causing undermining and failure of numerous mature riparian trees rooted at the toe and mid-bank along the left bank.

Overall, a vertical sheetpile or reinforced concrete wall treatment is probably geotechnically and environmentally superior to a rip-rap wall for this site. If vertical sheetpile or reinforced concrete walls are selected for the design, the plan view configuration of the wall should be an important design consideration. A uniform, straight wall would be appropriate for the upstream 50-60 ft-long section of the wall where it should be expected that a deep lateral scour pool would form as the result of high flows obliquely striking the wall and creating helical flow patterns that would sweep bed sediment from the edge of the wall to form a bar along the left bank immediately downstream. A vertical sheetpile wall is probably superior to either vertical reinforced concrete or steep vegetated rip-rap wall for accommodating 3-4 vertical ft of scour below the base flow water surface elevation without undercutting the toe of the finished structure.

However, there also appears to be an opportunity to effectively widen the channel width at the site without eliminating the existing trees along both banks, by constructing a new low-elevation floodplain on the apparently Town-owned property to the East of left bank. By excavating the now-flat, unvegetated area down approximately 4-6 vertical ft, the area could be converted to an active floodplain overflow area returning flow to Fairfax Creek immediately downstream from the existing pedestrian bridge left bank abutment. The existing TOB trees along the left bank could be preserved by leaving them on an “island” formation between the current active channel and the proposed floodplain restoration site. This alternative solution deserves careful consideration because it would allow the tennis courts site to be reinforced without vertical sheetpile wall treatment, and it would also be consistent with a recommended overall creek and floodplain restoration and management plan needed for the Peri Park site.

Downstream from the pool scour zone, it would be preferable to design the plan view configuration of the wall to include at least one irregularity projecting slightly into the channel area as opposed to an entirely straight configuration. A plan form that included at least one projection into the edge of flow would prevent against excessive flow velocities entering the downstream footbridge replacement site, and especially the still natural outside channel bend downstream from the footbridge site.

The upstream end of the proposed projection would create a fixed downstream end point for the lateral scour pool, and the low-velocity eddy zone on the downstream end of the projection might allow for a stable gravel bar to deposit where desirable native riparian trees (e.g., alders) could be established. One way to include a rectangular protection would be to outline and save one of existing clumps of mature native riparian trees near the top of bank.

There is also a clump of bay trees near the top of bank near the downstream end of the 113 ft-long repair site, near the southeast corner of the tennis courts fence. This clump may be saved by designing the plan form of the vertical wall structure to slightly deflect flow away from the current bank line toward the center of the channel. Correctly designed, such an irregularity at the downstream end of the wall would also serve to reduce shear stress at the footbridge replacement site immediately downstream and along the still natural outside channel bend immediately downstream on the right bank. Care must be taken in designing the plan form of the downstream end of the proposed wall so as not to deflect too much energy onto the left bank.

In addition, the alignment of the downstream end of the wall will substantially affect the hydraulics in the first bend in the sinuous channel reach downstream from the site, and therefore needs to be consistent with any overall creek management plan under development for Peri Park. The December 31, 2005 flood eroded numerous still-natural unstable banks in the reach downstream from the tennis courts. It is presumed that the preferred approach to managing the creek in the park would be to minimize the use of visible hardscape and preserve natural character created by the intact geomorphic processes. One way to stabilize the high, oversteepened banks along the eastern edge of the park without unsightly hardscape would be build up the banks with mechanically stabilized earth lifts and isolate the finished toe of the bank from chronic bank toe erosion by moving the active channel farther to the west. This is feasible, but would need to be part of a comprehensive channel and floodplain restoration and management plan for the park.

Given the uncertainty as to how the downstream reach may be managed, it may be preferable to end the proposed wall with a straight wall section, and use temporary materials to cause a managed amount of deflection toward the channel off the end of the wall. This could be accomplished by placing a large woody debris root ball projecting into the flow zone at the end of the wall and cable-anchoring it to 2-3 pieces of ½-ton to 1-ton rip-rap. This type of end treatment for the wall would create a scour pool that may cause partial sedimentation at the still-natural outside bend downstream and serve to reduce bank erosion rate. The height of the proposed LWD structure should be not more than 3-5 ft above the base flow water surface elevation, or approximately the same as the right bank height to allow for overbank flows to expand to the right onto the right bank floodplain in the park. The design for

the downstream end of the wall will also depend on the design selected for the footbridge replacement site. For example, the tennis courts site wall may be extended 37 additional feet downstream to stabilize the eroded bank at the footbridge site.

Peri Park Footbridge Replacement and Bank Repair

Significant bank erosion during the December 31, 2005 flood also extended downstream from the tennis courts site and caused the complete failure of the right bank abutment and a clear-span pedestrian footbridge. The footbridge provided the only safe pedestrian crossing connecting between the Town of Fairfax recreational facilities on both sides of the creek. It is recommended that the pedestrian footbridge be replaced with a pre-manufactured clear-span bridge set on vertical reinforced concrete abutments with an 18 ft-minimum inside edge to inside edge opening width.

It is preferable to set the left bank abutment into the finished bank slope to near the existing left bank top of bank location. This would allow the abutment to be covered with natural materials to be visually screened and allow for some future left bank erosion without exposing the abutment to potential failure. However, this would also require that the abutments be spaced farther apart than the18-ft recommended minimum, and likely increase project construction cost. Setting back the left bank abutment is a low to medium priority for the design, and it may depend what standard lengths are available from the selected clear-span bridge manufacturer, and the marginal cost of the standard lengths larger than 18 feet.

The FEMA-calculated 100-year flood water surface elevation is 10 feet above the creek bed elevation at the footbridge replacement site, and the 50-year flood water elevation is 9 feet above the bed (FEMA, 1977). The site is in a floodprone reach of Fairfax Creek where overbank flows naturally occur onto the lower right bank floodplain and are generally preferable to in-channel containment. Therefore, the replacement footbridge structure should either be set on abutments to raise the low chord up to at least 9 ft above the channel bed, or otherwise have a low profile and contain minimal hand-railing structure to allow for flood flows to pass over the through the structure relatively unimpeded.

It is also recommended that the right bank be permanently stabilized upstream and downstream from the replacement footbridge right bank abutment. The 37 ft-long bank repair and stabilization site begins at the upstream end of the stacked rubble wall (to be removed) and extends downstream to the beginning of the still-natural outside channel bend. It is recommended that the right bank be stabilized in this 37

ft-long reach using an extension of the wall from the upstream tennis courts site, either the vertical sheet pile wall or the steep vegetated heavy rip-rap wall.

If the vertical sheet pile wall is the recommended repair technique for the upstream tennis courts site and would also be appropriate for the footbridge site. The vertical wall would provide permanent stabilization for the abutment would allow for the shortest pre-fabricated clear-span bridge structure. If the steep vegetated rip-rap wall is selected for the tennis courts repair, the right bank abutment should be set back near the top of the bank so that the rip-rap wall structure is continuous under and through the structure, and covers the vertical face of the abutment.

There is a lower-cost and possibly environmentally superior alternative bank stabilization design for the footbridge site that should also be considered. That is stabilizing the banks upstream and downstream from the right bank abutment by laying back or constructing with mechanically stabilized earth banks at maximum 1.5(H):1(V) slope, and covering the more gradually-sloped banks with biodegradable geofabric and plantings, including live willow poles along the toe of the bank. This "soft" biotechnical bank stabilization technique would not typically be recommended at the footbridge site, where Fairfax Creek is gradually entering a left channel bend. However, as discussed above, the alignment of the downstream end of the recommended tennis courts wall structure could be made to deflect flow energy coming off the end of the wall toward the center of the channel to allow for a slight eddy zone along the footbridge replacement site. Only this way would it be recommended to use a soft bank stabilization technique at the site. Because this treatment would require the top of bank to be set back to grade to maximum 1.5(H):1(V) bank slope, the right bank abutment would also have to be set back to the new laid back top of bank. This would thereby require the clear-span length the bridge structure to be approximately 8-12 ft longer than the first alternative vertical sheet pile treatment.

For either recommended design, it is further recommended that the downstream end point of the 37 ft-long recommended bank treatment be stabilized by placing large woody debris cable-anchored to 2-3 pieces of1/2-ton rip-rap, such that the structure projects in the downstream direction only 2-3 ft into the flow zone from the finished graded bank, and not above the existing right bank top of bank elevation.

REFERENCES CITED

Federal Emergency Management Agency (FEMA). 1977. Flood Insurance Study, Town of Fairfax, March 1977.

Stetson Engineers. 2000. Geomorphic Assessment of the Corte Madera Creek Watershed. Prepared by Stetson Engineers for Friends of Corte Madera Creek and Marin County Flood Control District, December 2000.

Ross Taylor and Assoc. 2006. Corte Madera Stream Crossing Inventory and Fish Passage Evaluation, Final Report. Prepared by Ross Taylor and Associates for Friends of Corte Madera Creek, February 2006.

LIST OF ABBREVIATIONS

abv	above
adj	adjacent
avg	average
bdge	bridge
bdy	boundary
bkf	bankfull
bk ht	bank height
bldr	boulder
BR	bedrock
ch	channel
C/L	centerline
CMP	corrugated metal pipe
coll	colluvium, colluvial
conc	concrete
conf	confluence
cont	continue
deg	degrees
dep	deposit
d/s	downstream
FGC	Fluvial Geomorphology Consulting
FIS	Flood Insurance Study
fp	floodplain
fr	from
gr	gravel
gw	groundwater
(H)	horizontal
horiz	horizontal
imm	immediately
lt	light

LB	left bank
LEW	left edge water
max	maximum
MT&CC	Marin Town and Country Club
MSE	mechanically stabilized earth (i.e., mechanically compacted soil lifts)
min	minimum
misc	miscellaneous
nat	natural
nr	near
OC	outcrop
O.C.	on center
OHW	ordinary high water
pvt	private
RB	right bank
RCP	reinforced concrete pipe
ret	retaining
REW	right edge water
ROW	right of way
RR	rip-rap
RSP	rock rip-rap slope protection
SD	storm drain
sed	sediment or sedimentation
SEI	Stetson Engineers Inc.
SFD	Sir Francis Drake Blvd
SS	sanitary sewer
SSP	structural steel pipe
TOB	top of bank
tbd	to be determined
tbr	to be replaced
trib	tributary
Q	discharge
u/s	upstream
var.	varies
vert	vertical
(V)	vertical
w	width
w/	with
w/o	without
WSE	water surface elevation
xing	crossing

LIST OF APPENDICES

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