ERDC/CHL SR-00-3

Coastal and Hydraulics Laboratory

US Army Corps of Engineers®
Engineer Research and Development Center

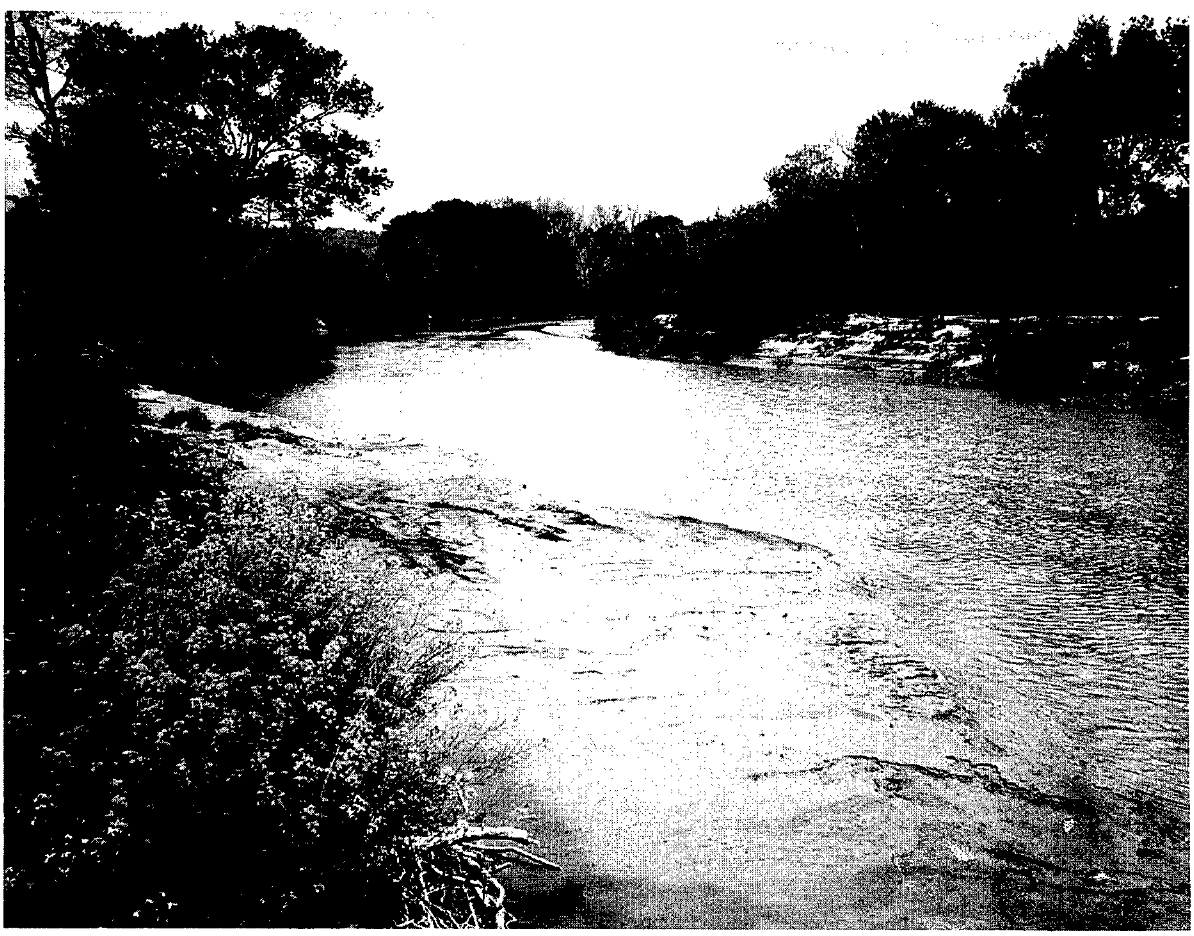
20000920 055

Channel Stability Problems, Pajaro River, Watsonville and Pajaro, California

U.S. Army Engineer Committee on Channel Stabilization Report of the 63rd Meeting

Ronald R. Copeland and Dinah N. McComas, editors

August 2000



ERDC/CHL SR-00-3
CCS-00-1
August 2000

Channel Stability Problems, Pajaro River, Watsonville and Pajaro, California

U.S. Army Engineer Committee on Channel Stabilization Report of the 63rd Meeting

by Ronald R. Copeland, Dinah N. McComas, editors

Coastal and Hydraulics Laboratory
U.S. Army Engineer Research and Development Center
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Final report

Prepared for U.S. Army Corps of Engineers
Washington, DC 20314-1000

Engineer Research and Development Center Cataloging-in-Publication Data

United States. Army. Corps of Engineers. Committee on Channel Stabilization. Meeting. (63th : 1998 : San Francisco, California)

Channel stability problems, Pajaro River, Watsonville and Pajaro, California : U.S. Army Engineer Committee on Channel Stabilization report of the 63th meeting / by Ronald R. Copeland and Dinah N. McComas, editors ; prepared for U.S. Army Corps of Engineers.

94 p. : ill. ; 28 cm. — (ERDC/CHL ; SR-00-3)(CCS ; 00-1)

Includes bibliographic references.

1. River channels -- California. 2. Channels (Hydraulic engineering) -- California. 3. Levees -- California. 4. Pajaro River (Calif.) I. Copeland, Ronald R. II. McComas, Dinah N. III. United States. Army. Corps of Engineers. IV. Engineer Research and Development Center (U.S.) V. Coastal and Hydraulics Laboratory (U.S.) VI. Title. VII. Series: CCS ; 00-1. VIII. Series: ERDC/CHL SR ; 00-3.

TA7 E8 no.ERDC/CHL SR-00-3

Contents

List of Figures

Executive Summary

The San Francisco District is in the process of planning and conducting rehabilitation work on the U.S. Army Corps of Engineer Pajaro River Flood Control Project. The project, located in both Santa Cruz and Monterey Counties, California, consists of setback levees on both sides of the river's main channel. The project protects valuable agricultural lands and the cities of Watsonville and Pajaro. The project levees were overtopped during floods in both March 1995 and February 1998 causing extensive property damage and damage to the flood control project itself. Both floods exceeded the project's design discharge. Project damages include severe erosion of the main channel to the point where it has reached the levee toe.

The San Francisco District requested that the U.S. Army Engineer Committee on Channel Stabilization provide guidance, insights, and/or recommendations that would point the rehabilitation effort in the right direction. The function of the Committee is to act as advisors to the District, providing the benefit of its members' experiences. Committee recommendations are in no way binding on the District which has responsibility for project design. The Committee was provided with historical information and conducted a site visit to observe existing conditions.

It was the consensus of the Committee that the original design concept was a good one in that it had operated successfully for almost 50 years. Project overtopping occurred when the discharge exceeded design values and extensive erosion occurred when protective vegetation was removed from the benches and the banks of the main channel. It is therefore the Committee's recommendation that rehabilitation efforts focus on restoring the original setback levee configuration, using reliable bank protection measures where appropriate and vegetative protection as much as possible. Allowing vegetative growth in the flood conveyance channel introduces the requirement for a detailed maintenance plan that is both technically and economically feasible.

The Committee commented on the appropriate level of repair work to be conducted under Public Law 84-99 authority. PL 84-99 funding is intended to repair "flood-caused damage." The Committee defined repairing flood-caused damage as returning the channel to a condition where the levee integrity is restored. This would involve restoring the bench so the levee is set back a minimum of 30 ft, which was the minimum design setback, or armoring the levee and toe to prevent erosion. Design criteria for the rehabilitation work needs to meet current standards. This would include the levee slope and levee source material. Conveyance for the design discharge must be maintained. Finally, the benefits from the rehabilitation must be greater than the costs.

The Committee recommended the following actions:

a. Lay out an appropriate stable planform for the main channel of the river. This should be the channel's long-term stable footprint reflected by preflood conditions. Rehabilitation works should conform to this long-term planform. Natural deviations from this long-term planform would indicate that additional rehabilitation works should be constructed.

b. Erosion of the main channel that has progressed to a point adjacent to the levee should be repaired. One option is to cover the entire face of the levee with riprap. This includes providing an appropriate foundation, filter, and toe protection as recommended in U.S. Army Engineer Manual 1110-2-1913 and EM 1110-2-1601. Another option would be to use a combination of riprap and bioengineering protection. Proper design would require determining the applied shear stresses on the upper portion of the levee and then selecting a bioengineering treatment that has a critical shear stress which exceeds the applied shear stress. A third option would be to build river training works to return the bench to its original 30-ft-wide dimension. This could be accomplished using spur dikes or a longitudinal dike system with tiebacks. A variety of construction materials have been used for these purposes. Rock has been shown to be the most reliable.

c. For cases where there is still a bench adjacent to the levee — but it is less than 30-ft wide — toe protection is recommended. This could take the form of weighted toe placement of riprap. Vegetative treatments above the weighted toe would be appropriate.

d. The design conveyance of the channel should be maintained. This will require the development of a vegetation management plan that integrates both environmental and operational considerations. It also requires that the sponsors and resource agencies work together to develop a feasible monitoring and maintenance plan. It should be recognized that the vegetative features associated with this project may increase the design uncertainty. This is attributed to: increased uncertainty associated with determining hydraulic roughness through vegetation; variability in vegetation density, height, and distribution that will occur regardless of the intents of the maintenance plan; and unexpected future staff and/or funding limitations and/or environmental issues that may affect implementation of the maintenance plan.

Attendees

U.S. Army Engineer Committee on Channel Stabilization

Meg Burns - CENAB-EN-GH 410-962-6761
Ronald Copeland - CEWES-CR-R - Chairman 601-634-2623
Richard DiBuono - HQUSACE-CW-EH 202-761-8511
J Craig Fischenich - CEWES-EE-A 601-634-3449
Rob Grubbs - HQUSACE-CW-OE 202-761-5090
Jim Lencioni - CENWS-ED-TB-HH 206-764-3595
Tom Pokrefke - CEERD-HV-A 601-634-2650
John Remus - CENWO-ED-HB 402-221-4620
Ed Sing - CESPD-ET-EW 415-977-8117
Mike Spoor - CEORH-ED-G 304-529-5514
Scott Stonestreet - CESPK-DE-D 916-557-7719

U.S. Army Engineer District, San Francisco

Carlos Hernandez - CESPN-PE-EH 415-977-8590
Tom Kendall - CESPN-PE 415-977-8532

San Francisco Division

Donald Bergner - CESPD-ET-ET 415-977-8101

Monterey County Water Resources Agency, CA

Narayan Thadani 408-755-4865
Russ Yoshimaru 408-755-4803

Santa Cruz County, CA

Don Hill 408-454-2637
Terry Reynolds 408-477-3921
Steven Stiles 408-454-2816

City of Watsonville, CA

David Koch 408-728-6046

Northwest Hydraulic Consultants

Ed Wallace 916-371-7400

Lee and Pierce

Jim Slater 408-758-0096

1 Background

Jim Lencioni

Senior Hydraulic Engineer
U.S. Army Engineer District, Seattle

The Committee on Channel Stabilization was convened to review the present and future status of the federal flood control project on the Pajaro River near the towns of Watsonville and Pajaro, California. This levee project was designed and constructed by the Corps of Engineers in the late 1940's and is maintained by the local project sponsors, Santa Cruz and Monterey Counties.

The Federal project consisted of setback (30-ft minimum from the channel banks) levees along the lower approximately 11.5 miles of the Pajaro River and approximately 2.5 miles along Salsipuedes Creek upstream from it's confluence with the Pajaro River. The levee system was designed to contain a discharge of 22,000 cfs downstream from the Pajaro-Salsipuedes confluence and 19,000 cfs upstream from the confluence, with a freeboard of 3 ft between the computed design discharge water surface elevation and top of levee elevation (assumed Manning roughness 0.035). A short (about 9 years) hydrologic record from the U.S. Geological Survey stream gauge at Chittenden (located upstream from the Pajaro-Salsipuedes confluence) was used to evaluate the hydrologic characteristics of the river basin during project design. At the time of design and construction of the project, the design discharges were considered to be the 2 percent (50-year frequency) peak annual flow. With addition of the subsequent hydrologic record through 1997, the original design discharge presently represents about the 7 percent (14-year frequency) peak annual flow event.

The project has been subjected to four floods exceeding the original design discharge capacity since it's construction—the latter two occurring in 1995 (approximately 22,000 cfs) and 1998 (approximately 28,000 cfs) and the other two in 1956 and 1958 (approximately 24,000 cfs each). The two large floods in the mid 1950's caused some levee erosion, but did not overtop the levees. The 1995 and 1998 floods both caused erosion damage to the levees and overtopped the levee leading to subsequent levee failures at the overtopped locations. There has evidently been no significant levee threatening erosion damage or overtopping of levees with any other flood events during the project's history even though peak discharges of up to about 18,000 cfs occurred.[1]

Maintenance of woody vegetation on the overbank setbacks between the river channel banks and levee was allowed to significantly deteriorate until the floods of 1995 occurred. Significant clearing of the woody vegetation was undertaken upstream of the Highway 1 bridge crossing after that flood event and resulted in a channel capacity that probably exceeded the original design condition. Briefing and data prepared by staff from Northwest Hydraulic Consultants (Santa Cruz county consultant) indicated that the originally constructed channel width had decreased over time until the recent flood events occurred which appeared to have increased the width back approaching, but not equal to, the original constructed dimensions. Lack of vegetation maintenance and/or decreased channel dimensions had become so severe that, according to San Francisco District staff, by 1993 channel conveyance was reduced to the point where a discharge of only about 6,600 cfs created water surface elevations to within about 4 ft of the top of levee.

Subsequent to the more recent flood events, various areas of levee erosion have been repaired either by flood fight operations or PL 84-99 projects. SPN and the local sponsors have identified other locations where levee erosion has occurred and are in the process of determining the best approach to use in making such repairs under PL 84-99 authority. In addition, a Section 216 planning authority study is ongoing to review a more systemwide approach which could eventually lead to redesign of the project to increase the existing level of protection. SPN requested the committee meeting to solicit guidance/advice regarding the integrity of the existing levees, the design of appropriate bank erosion protective works and integration of the PL 84-99 and Section 216 authority design processes.

[1] Note: all discharges as measured at the Chittenden gauge.

2 Comments by Committee Members

Meg Burns

Hydraulic Engineer
U.S. Army Engineer District, Baltimore

Numerical hydraulic modeling of the project should be performed, using current sections, existing Manning n-values, and the current top-of-levee profile. Questions that need to be answered include whether or not the project is hydraulically deficient and what the new overtopping profile looks like with the "resurfacing" work. It is necessary to determine a politically acceptable level of conveyance (protection), and then see how much vegetation can be allowed within the existing channel. High water marks from 1995 (with lots of vegetation inside the levees) and 1998 (with little vegetation inside the levees) can be used to determine the historic range of hydraulic roughness. These data can be used to evaluate the sensitivity of the water surface profile to changes in roughness. The potential for (and impacts of) future deposition should be assessed.

The seepage problems with the levees need to be addressed. The levees also appear to have other problems including:

a. Too steep on land side (and possibly also on the river side).

b. Reports of considerable settlement.

c. Close to San Andreas fault (possibility of severe damage if earthquake occurs while levees saturated).

d. Silty material in levees may not meet today's standards.

e. Top width does not appear to be sufficient.

These problems need to be evaluated, and decisions made as to whether they need to be rectified as part of the rehabilitation.

I liked Scott Stonestreet's idea of plotting floods by volume versus frequency rather than peak flow versus frequency. This might show that the 1995 flood (for instance) put the most stress on the levee system in resisting seepage.

Slope failures of banks: avoid exacerbating failure conditions by controlling seepage if possible, and not surcharging the bank by planting trees or placing rock near the top of bank.

The benches appeared to be much higher than the ground outside the levees, indicating substantial deposition. The new surveys should be examined to determine whether this is actually the case.

Small patches of invasive species (*Arundo*) should be killed now before they get out of control.

Ronald Copeland

Research Hydraulic Engineer
Coastal and Hydraulics Laboratory
U.S. Army Engineer Research and Development Center

Operation and maintenance

A vegetation management plan that satisfies both environmental and conveyance objectives should be developed. Regulatory and resource agencies should concur with the plan so execution can be timely. An example of a good operation and maintenance plan is one developed by the Sacramento District for the Wildcat and San Pablo Creek Project in Contra Costa County, California. An outline of the procedure is presented here; details are contained in the Operation and Maintenance Manual (May 1990).

To achieve conveyance criteria, specified cross sections are to be monitored annually in the spring of the year after the flood season. The location, extent, and size of vegetation growth are to be documented at each of these cross sections during the inspection. In addition, these cross sections are to be surveyed to determine sediment removal and/or vegetal and debris clearing requirements. Each specified cross section is to be monitored as follows:

a. Survey each cross section. Plot onto previously provided, as-constructed, cross-section geometry. Determine the net area of sediment deposition by comparing the as-constructed condition to the condition surveyed at the time of inspection.

b. Determine the effective roughness condition by comparing existing vegetation conditions to the pictorials presented in the manual. Pictorials for each cross section show estimated hydraulic roughness at various levels of maturity. Examples are shown in Figure 1. As the vegetation matures, photographs should be taken to replace the pictorials. The roughness condition and appropriate n-value for each cross section are to be collectively determined by representatives from the Corps of Engineers and Contra Costa County.

An alternative method to evaluate the n-value for a particular cross section is to subdivide the cross section into regions of similar roughness conditions. Each subdivision is evaluated to determine a subsection n-value, wetted perimeter, and hydraulic radius. A composite n-value is then calculated. The Sacramento District suggested using the following compositing equation.

$$n = \frac{PR^{\frac{5}{3}}}{\sum_{i=1}^{N}\left(\frac{P_i R_i^{\frac{5}{3}}}{n_i}\right)}$$

where

P = the wetted perimeter

R = the hydraulic radius

N = the number of subdivided sections

This equation, which is equivalent to the conveyance method, assumes that the total discharge of the cross section is equal to the sum of the discharges calculated for the subdivided areas. Another compositing equation is the alpha method recommended in EM 1110-2-1601. The conveyance and alpha methods are among the compositing equations contained in the SAM hydraulic design package. Water-surface elevations can be quickly calculated using the SAM package. These compositing methods are appropriate for overbank flow or shallow flow conditions. However, in a channel with an alluvial bed and vegetated banks the equal velocity compositing method (also in SAM) is more appropriate. In the case of the Pajaro River it may be necessary to use the equal velocity method in the channel to determine a channel composite roughness which would then be used in the conveyance method to calculate total composite roughness.

c. Using the determined n-value and the net area of sediment deposition, the percent loss or reduction in freeboard can be determined for each cross section from figures developed for each cross section similar to Figure 2 shown here. A 50 percent or greater reduction in freeboard will require uniform maintenance of the entire representative reach. In no case will the maintenance result in over-excavation of the as-constructed channel

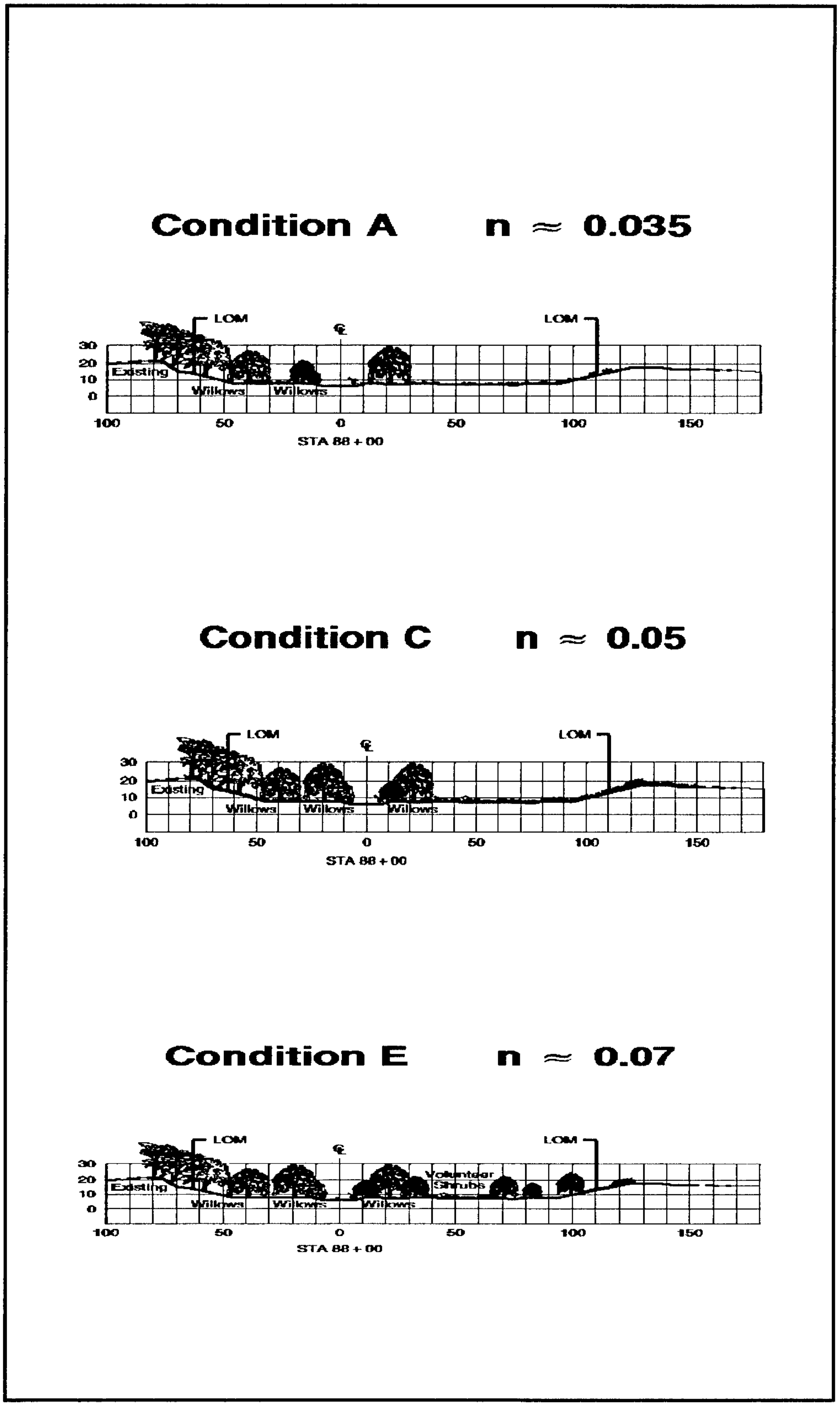
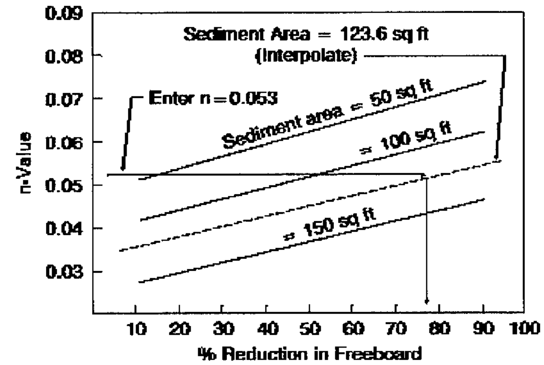


Figure 1. Pictorials for estimation of roughness coefficients Wildcat Creek (U.S. Army Engineer District, Sacramento 1990)

Monitoring Procedure

Step 3



- **Using freeboard reduction curves determine reduction in freeboard as shown above**

Figure 2. Monitoring Step 3 - use figure to determine reduction in freeboard as shown (U.S. Army Engineer District, Sacramento 1990)

or remove more vegetation than necessary to return to the design vegetation condition as presented in the manual.

Naturally, no plan is without challenges. In the case of Wildcat Creek, problems with the operation and maintenance plan developed. The main problem was that the plan was not followed. Inspections were not made as planned and maintenance was not conducted as scheduled. Another problem was that the vegetation did not grow as expected as shown in the pictorials.

The Pajaro River project has the advantage of historical highwater marks, known discharges, and photographic records of vegetation in the channel. Using a HEC-2 or HEC-RAS backwater model it is possible to calculate composite roughness values for known vegetation conditions. It is likely that photographic records could be used to develop n-value exhibits for the operation and maintenance manual.

Planform design

The flood control levees were apparently aligned following the existing Pajaro River planform at the time of construction. The levees were not set back sufficiently to contain the natural meander belt. This design strategy requires that the natural process of lateral migration be controlled by bank stabilization. A stable channel design therefore requires identification of a long-term planform for the main channel. Channel stability is defined herein as the ability to pass the incoming sediment load without significant degradation or aggradation. The following procedure for determining planform in alluvial rivers has been developed as part of the Flood Damage Reduction and Channel Restoration Research Program.

When hydrologic and sediment conditions are steady and the existing channel is stable, the existing channel geometry, wavelength, and sinuosity should be maintained in any stable channel design scheme. This may be the case for the Pajaro River, although no long-term aggradation or degradation studies were reported by the District. In lieu of a long-term sedimentation investigation, the proposed methodology may be used to confirm that the existing planform represents a stable configuration.

a. Determine the design width of the channel. The design width is related to the idealized "bankfull width" which is the channel top width that occurs when the channel-forming (dominant) discharge occurs. In terms of frequency this discharge generally varies between the 1.5 and 2 percent chance exceedance annual peak flow, but may be outside this range. Current research from the Flood Damage Reduction and Channel Restoration Research Program suggests that the effective discharge is the best representation of the channel-forming discharge. The effective discharge is the increment of discharge that transports the most sediment on an annual basis. This discharge may be determined by integrating a sediment transport rating curve with the annual flow-duration curve. This calculation requires a knowledge of the flow-duration characteristics, bed material size distribution, and a sediment rating curve (either measured, calculated, or a combination thereof). It is important to attempt to verify this channel-forming discharge with field indicators of bankfull discharge.

Several techniques are available for determining the design width as a function of the channel-forming discharge in stable alluvial streams. In order of preference they are:

(1) Develop a width versus effective discharge relationship for the project stream. This can be accomplished by measuring average width in stable reaches where the effective discharge can be calculated. These channel reaches may be in the project reach itself or in reference reaches upstream and/or downstream from the project reach. This is referred to as the analogy method. If there is no significant lateral inflow and if a stable reach can be found within the project reach, then a single measurement may be sufficient. This also assumes that the banks are composed of similar material in the project and reference reaches and that there are no significant hydrologic, hydraulic, or sediment differences in the reaches. This technique is inappropriate for streams where the reference reaches are in disequilibrium.

(2) Find stable reaches of streams with similar hydrologic, hydraulic, and sediment characteristics in the region and develop a hydraulic

geometry relationship for width versus effective discharge. This technique is also inappropriate for streams where the reference reaches are in disequilibrium.

(3) If a reliable width versus effective discharge relationship cannot be determined from field data, analytical methods discussed in Step 2 may be employed to obtain a range of feasible solutions. If the channel width is constrained because of right of way limits, select the required width and be prepared to provide bank protection.

The composition of the bank is important in the determination of a stable channel width. It has been shown that the percentage of cohesives in the bank and the amount of vegetation on the bank significantly affect the stable channel width. General guidance is available in U.S. Army Engineer Manual EM-1110-2-1418 (1994). Currently under development at ERDC are hydraulic geometry predictors for various stream types with different bank characteristics. These predictors will include confidence limits and may be used for general guidance when site specific data cannot be obtained. Figure 3 shows a generalized width predictor which was developed from data collected by Brice from meandering sand bed streams throughout the United States.

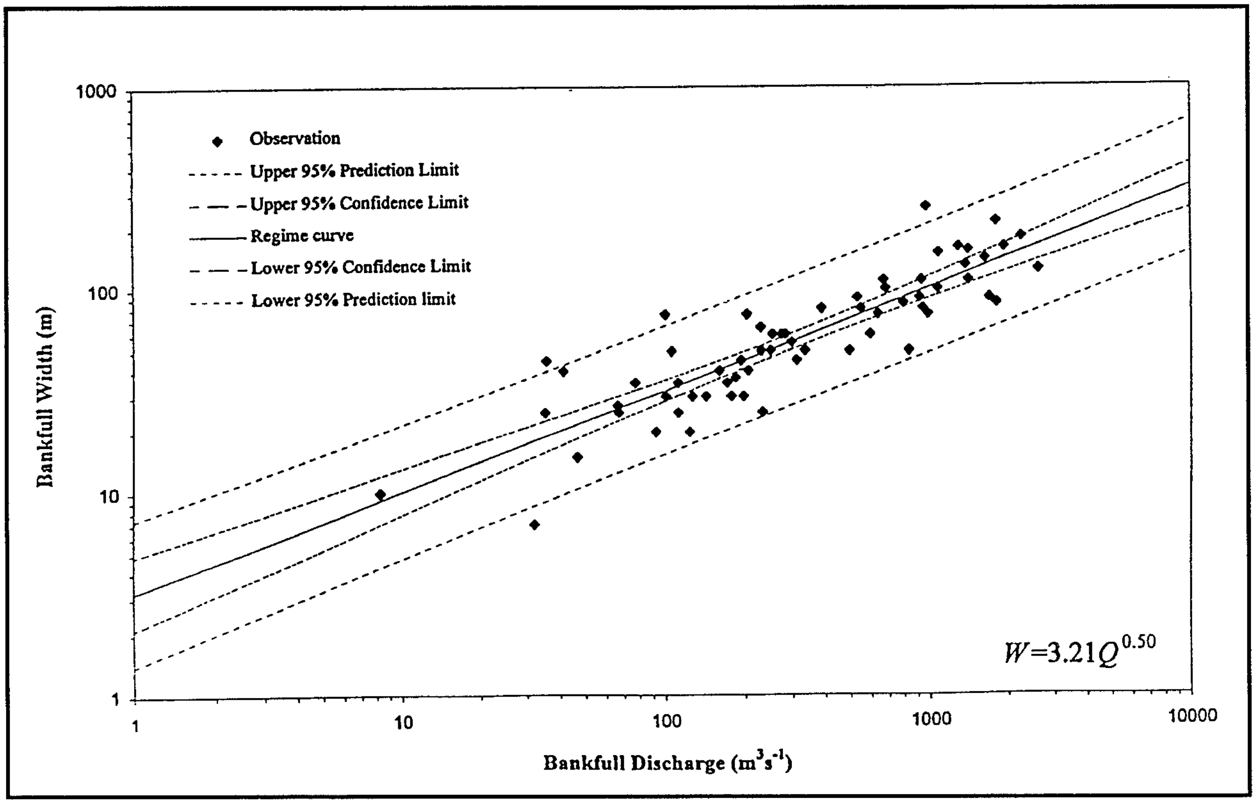


Figure 3. Hydraulic geometry width predictor for sand bed streams (Brice data)

b. *Calculate a stable channel slope and depth.* This step insures that channel geometry is capable of transporting the inflowing sediment load through the project reach. In sand bed streams, such as the Pajaro River, sediment transport is typically significant and an analytical procedure that considers both sediment transport and bed form roughness is required. Analytical approaches calculate the design variables of width, slope, and depth from the independent variables of discharge, sediment inflow, and bed-material composition. Three equations are required for a unique solution of the three dependent variables. Flow resistance and sediment transport equations are readily available. This procedure proposes using a hydraulic geometry width predictor as the third equation. The stable-channel analytical method in the U.S. Army Engineer hydraulic design package SAM may be used to determine a depth and slope for the width selected in Step 1. This method assumes a fully mobile sand bed and uses the Brownlie sediment transport and roughness equations.

c. *Determine a stable channel meander wavelength for the planform.* The most reliable hydraulic geometry relationship for meander wavelength is wavelength versus width. As with the determination of channel width, preference is given to wavelength predictors from stable reaches of the existing stream either in the project reach or in reference reaches. Lacking data from the existing stream, general guidance is available from several literature sources. An example from EM 1110-2-1418 is shown in Figure 4.

d. *Calculate the channel length for one meander wavelength.*

$$\textit{meander length} = \frac{\textit{wavelength X valley slope}}{\textit{Slope}}$$

e. *Layout a planform using the meander wavelength as a guide.* One way to accomplish this task is to cut a string to the appropriate channel (meander) length and lay it out with the appropriate wavelength on a map. Another, more analytical approach, is to assume a sine-generated curve for the planform shape as suggested by Langbein and Leopold and calculate x-y coordinates for the planform. This rather tedious numeric integration can be accomplished using a computer program such as the one in the SAM hydraulic design package. The sine-generated curve produces a uniform meander pattern. A combination of the string layout method and the analytical approach would produce a more natural looking planform.

Check the design radius of curvature to width ratio, making sure it is within the normal range of 1.5 to 4.5. If the meander length is too great, or if the required meander belt width is unavailable, grade control may be required to reduce the channel slope.

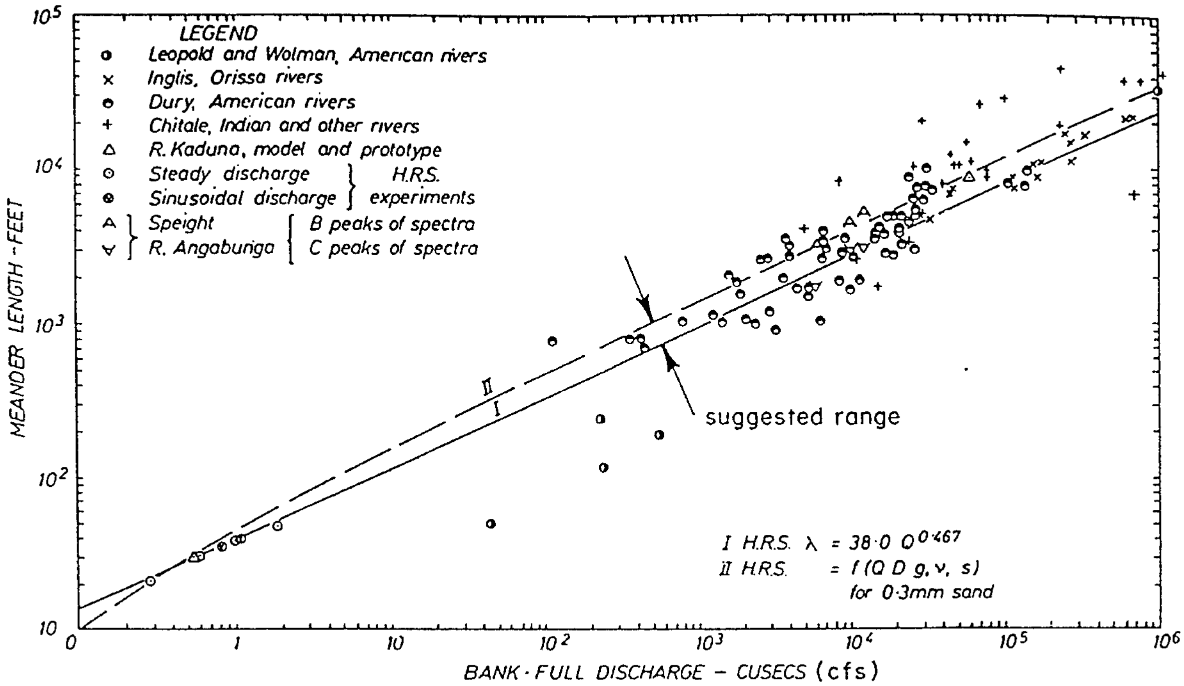


Figure 4. Hydraulic geometry meander wavelength predictor (Ackers and Charlton 1970 - EM 1110-2-1418)

f. Conduct a sediment impact assessment. The purpose of the sediment impact assessment is to assess the long-term stability of the new vegetated channel in terms of aggradation and/or degradation. This can be accomplished using a sediment budget approach for relatively simple projects or by using a numerical model which incorporates solution of the sediment continuity equation for more complex projects. With a sediment budget analysis, average annual sediment yield with the design channel is compared to the average annual sediment yield of the existing channel. Large differences in calculated sediment yield indicate channel instability. This step is especially important in the Pajaro River because the vegetated channel is part of a flood damage reduction project. In such cases it may be necessary to design a channel that is less than ideal in terms of channel stability in order to achieve flood control benefits. Typically, a compound channel design provides the best combination of benefits.

The most reliable way to determine the long-term effects of changes in a complex mobile-bed channel system is to use a numerical model such as HEC-6. River systems are governed by complicated dependency relationships where changing one significant geometric feature or boundary condition affects other geometric features and flow characteristics both temporally and spatially. Changes at any given location in a stream system are directly related to the inflow of sediment from upstream. This makes the application of the sediment continuity equation essential to any detailed analysis. The most significant of these relationships and the continuity of sediment mass are accounted for in the numerical model approach.

Sediment accumulation

It is necessary to make a determination of sediment accumulation rates on the benches to develop a vegetation management plan and to estimate maintenance costs. Sediment has been accumulating on the benches of the Pajaro River over the years, which was apparent at several locations along the river during the field inspection. The rate of sediment accumulation on the benches is a function of the amount of sediment washed into the flood control project and the magnitude of the deceleration of velocity on the benches. Accumulation is accelerated as vegetation density increases. Estimating the quantity and rate of sediment deposition is difficult because the fine sediment depositing on the bench is "wash load" and not related to the bed material found in the channel. This means that the only way to determine how much fine sediment is entering the system is to measure it. According to U.S. Geological Survey information, sediment data were collected at the Chittenden gauge between 1978 and 1992. It is unfortunate that the data collection was apparently discontinued before the two major floods in 1995 and 1998. Consideration should be given to reactivating this sediment gauge. It will be difficult to develop a reliable method for calculating sediment deposition with various densities of vegetation. Deposition is a function of localized velocities rather than the average velocities used to calculate water-surface elevation. For this reason it is recommended that a field monitoring program be developed to measure sediment accumulation on the benches at several locations with a variety of vegetation densities. Sediment accumulation rates could then be correlated with inflowing sediment loads for a variety of vegetation conditions. This program should be established in a timely fashion to be able to use the results in the planning and/or design process.

Bank stabilization approaches

General guidance for design, construction, and monitoring of streambank protection projects is provided by the *WES Stream Investigation and Streambank Stabilization Handbook* (1997) by David Biedenharn, Dave Derrick, Charles Elliott, and Chester Watson. A copy of this handbook was provided to the San Francisco District. Excerpts covering design guidance for longitudinal stone toe, dikes and retards, and deflecting methods are included in Appendix C.

J. Craig Fischenich

Research Hydraulic Engineer
Environmental Laboratory
U.S. Army Engineer Research and Development Center

The San Francisco District, Monterey and Santa Cruz Counties, the consultants and others involved with the Pajaro River Project are to be complimented for their initiative and efforts to date. I was impressed with the level of thought already invested and the competence of those involved. I believe that the project is generally headed in the right direction and that much of what remains is details (albeit important ones).

I'll try to keep my comments brief. Organizationally, I've divided my comments into three sections: general observations and suggestions; recommendations to restore levee integrity; and recommendations for long-term management.

General observations and suggestions

Based on the information provided, it appears that the project has performed as intended. I see no need for significant design changes or reconstruction and would discourage notions that the level of protection afforded by the project must be increased. An economic analysis will provide more information about this but, even if economically justified, raising the level of protection without further levee setbacks should be approached cautiously. I'm concerned that raising the levees would place excess stress on the system from both a hydraulic and geotechnical perspective.

Assessing stream processes on the basis of a single site visit is difficult, but I think the evidence supports the description of deposition and erosion given by the consultants during the briefing. It appears that the project was originally constructed with a conveyance area approximating its current condition, but that considerable deposition occurs during the "normal" flow years. The deposited sediments form large benches that are periodically (about every 10 years) removed by high flows. The process of removal appears to be geotechnical failures (rotations and block failures) followed by removal of the talus material by scour. No doubt standard hydraulic erosion contributes to bank loss during high flows, but the predominant loss is likely mass wasting as the water surface drops. Piping and suffusion contribute to subsequent material loss, as do shallow slides after the material dries.

The bank and bench material is very silty, contributing to the drainage problems that lead to failures. The sedimentation processes are significantly influenced by vegetation in the channel. Vegetation plays a central role in the deposition of sediments on streambanks and flood plains. The capacity of flowing water to transport bed material load increases approximately with the sixth power of the velocity. Vegetation dramatically retards near bed and bank velocities by increasing the local flow resistance. This effect can and does promote deposition of the bed material load, particularly on streambanks but also on the benches formed within the levee.

The stabilizing benefits of vegetation can be a strong inducement for their incorporation into flood control projects. Leaves and stems of plants intercept rainfall and reduce surface erosion both from runoff and from overbank flooding. Vegetation, primarily woody plants, also helps prevent mass movement, particularly shallow sliding in slopes. The roots of many woody species reinforce soil particles and substantially improve the tensile strength of the underlying soil mass. A root-reinforced soil behaves as a composite material in which elastic fibers of relatively high tensile strength (roots) are embedded in a matrix of relatively plastic soil. Tractive forces between the roots and the soil add shear strength to the composite. Vertical root systems can also penetrate through the soil mantle into firmer strata below, thus anchoring the soil to the slope and increasing resistance to sliding. Roots also modify the soil moisture content of the soil, thus increasing slope stability, and can eliminate geotechnical failures related to high pore water pressure. Compared with unvegetated stream banks, soils in vegetated banks are drier and better drained. Anchored and embedded stems can act as buttress piles or arch abutments in a slope, counteracting shear stresses and preventing soil sliding around and between vegetation components. The weight or surcharge of large trees exerts a stress component perpendicular to the slope that tends to increase resistance to sliding. The downslope component of stress imparted from surcharge can also have a destabilizing influence on the slope, however, and this must be weighed against its benefits. Likewise, there are other destabilizing influences of vegetation. Of generally minor concern is the alleged tendency of roots to invade cracks, fissures, and channels in a soil or rock mass and thereby cause local instability by wedging or prying action. Of greater concern is the destabilizing influence from turning moments exerted on the soil mass as a result of strong winds or flowing water moving across the vegetation. This can become particularly troublesome when the turning forces are sufficient to uproot the vegetation and expose the underlying soil to further erosion. Thus, the effect of vegetation on soil stability is the sum of the root reinforcement, soil moisture modification and buttressing benefits minus the root wedging and overturning drawbacks, with consideration for both stress components of surcharge.

Recommendations to restore levee integrity

The first order of business must be the restoration of the levee integrity. I hope this is done within the context of a longer-term management plan. But either way, areas damaged by the 1998 floods must be repaired. Concurrent with this effort, I recommend that an experienced geotechnical engineer evaluate the levee cross section and material integrity. Floods in 1995 and 1998 suggest the levees are adequate, but they look too narrow and steep given the source of material and the proximity to major faults.

I'm in general agreement with the recommendations of the Committee regarding restoration and stabilization of damaged reaches, so I won't revisit those here. I was asked to provide specific information about a couple techniques and that information follows.

One option contemplated for stabilizing the benches in cases where the bench width is less than 30 ft but outside a line extended on a 3:1 slope from the top of the levee is the use of bioengineering techniques on the bench face. Biostabilization techniques to reinforce slopes and streambanks, popular in the 1930's in the United States, have seen a resurgence in recent years here and in southeast Asia, and have been used for centuries in Europe. Soil bioengineering is the use of live and dead plant materials in combination with natural and synthetic support materials for slope stabilization, erosion reduction, and vegetative establishment. Soil bioengineering techniques can be very useful in multi-objective erosion control projects, such as the Pajaro, because some techniques can be used to concurrently control erosion and provide environmental benefits (habitat and aesthetics, for example).

Bioengineering techniques best suited to stabilizing the benches on the Pajaro are those that promote the development of dense brushy vegetation growth with extensive and deep root systems. The best bets are brush mattresses, wattling, brush layering, and perhaps posts.

a. *Brush mattresses.* A brush mattress, sometimes called brush matting or a brush barrier, is a combination of a thick layer (mattress) of interlaced live willow switches or branches and wattling. Both are held in place by wire and stakes. The branches in the mattress are usually about 2 to 3 years old, sometimes older, and 5 to 10 ft-long. Basal ends are usually no more than about 1.5 in. in diameter. They are placed perpendicular to the bank with their basal ends inserted into a trench at the bottom of the slope in the splash zone, just above any toe protection, such as a rock toe. The branches are cut from live willow plants and kept moist until planting. The willow branches will sprout after planting, but care should be taken to obtain and plant them in the dormant period, either in the late fall after bud set or in the winter or early spring before bud break. A compacted layer of branches 4 to 6 in.-thick is used and is held in place by either woven wire or tie-wire. Wedge-shaped construction stakes (2 by 4 by 24 in. to 2 by 4 by 36 in., diagonal cut) are used to hold the wire in place. A gauge and suitable type tie-wire is No. 9 or 10 galvanized annealed. It is run perpendicular to the branches and also diagonally from stake to stake and usually tied with a clove hitch. If woven wire is used, it should be a strong welded wire (2 by 4-in. mesh). The wedged-shape stakes are driven firmly through the wire as it is stretched over the mattress to hold it in place. The wedge of the stake compresses the wire to hold the brush down.

b. *Wattling.* Wattling is a sausage-shaped bundle of live, shrubby material made from species that quickly root from the stem, such as willow and some species of dogwood and alder. These bundles are laid over the basal ends of the brush mattress material that was placed in the ditch and staked. Wattling bundles may vary in length, depending on materials available. Bundles taper at the ends and this is achieved by alternately (randomly) placing each stem so about one-half of the basal ends are at each end of the bundle. When compressed firmly and tied, each bundle is about 15 to 20 cm in diameter in the middle. Bundles should be tied with hemp binder twine or can be fastened and compressed by wrapping "pigtails" around the bundle. Pigtails are commonly used to fasten rebar together. If tied with binder twine, a minimum of two wraps should be used in combination with a non-slipping knot, such as a square knot. Tying of bundles should be done on about 38-cm centers. Wattling bundles should be staked firmly in place with vertical stakes on the downhill side of the wattling not more than 90 cm on center and with the wedge of the stake pointing upslope. Also, stakes should be installed through the bundles at about the same distance, but slightly offset and turned around so their wedge points downslope. In this way, the wedged stakes, in tandem, firmly compress the wattling. Where bundles overlap, an additional pair of stakes should be used at the midpoint of the overlap. The overlap should be staked with one pair of stakes through the ends of both bundles while on the inside of the end tie of each bundle.

c. *Brush layering.* Brush layering, also called branch layering, or branch packing, is used in the splash zone, but only in association with a hard toe, such as rock riprap, in the toe zone. It can also be used in the bank zone as discussed later. This is a treatment where live brush that quickly sprout, such as willow or dogwood species, are used in trenches. Trenches are dug 2-6 ft into the slope, on contour, sloping downward from the face of the bank 10 to 20 deg below horizontal. Live branches are placed in the trench with their basal ends pointed inward and no less than 6 in. or more than 18 in. of the tips extending beyond the fill face. Branches should be arranged in a crisscross fashion. Brush layers should be at least 4 in.-thick and should be covered with soil immediately following placement and the soil compacted firmly.

Specifications for these techniques are not standardized. Following are excerpts from specifications for successful projects, and details from the descriptions above should help in preparing specifications for the Pajaro.

a. *Wattling.*

(1) Materials. Wattling bundles should be prepared from live, shrubby material, species that will root advantageously, such as *Salix* (willow), etc.

(2) Bundle size. Wattling bundles may vary in length, depending on materials available. Bundles should taper at the ends and should be 1 to 10 ft longer than the average length of stems to achieve this taper. Butts should be no more than approximately 10 in. in diameter. When compressed firmly and tied, each bundle should be 8 in., plus or minus 2 in., in diameter.

(3) Bundle construction. Stems should be placed alternately (randomly) in each bundle so approximately one-half the butt ends are at each end of the bundle.

(4) Bundle tying. Bundles should be tied on not more than 15-in. centers with a minimum of two wraps of binder twine or heavier tying materials with a non-slipping knot. Tying should be done with strapping machines as long as the bundles are compressed tightly.

(5) Timing of preparation. Bundles should be prepared not more than two days in advance of placement when kept covered and in the shade. If provisions are made for storing wattles in water or they are sprinkled as often as needed to be kept moist, covered, and in the shade, they may be prepared up to seven days in advance of placement.

(6) Grade. Grade for wattling trenches should be staked with an Abney level or similar device, and should follow slope contours (i.e., they should be horizontal).

(7) Spacing. Wattling should be spaced at no more than 6 ft on center.

(8) Installation. Bundles should be laid in trenches dug to approximately one-half the diameter of the bundles. Bundles should be placed with ends overlapping at least 12 in. The overlap must be sufficient to allow the last tie on each bundle to overlap.

(9) Staking. Bundles should be staked firmly in place with vertical stakes on the downhill side of the wattling not more than 24 in. on center and with stakes through the bundles at not more than 36 in. on center. When bundles overlap between two previously set guide or bottom stakes, an additional bottom stake should be used at the midpoint of the overlap. The overlap should be "tied" with a stake through the ends of both bundles and inside the end tie of each bundle. (Note: alternate staking designs consisting of "duck-bill"-type anchors and one-fourth-in. steel cable attached with cable saddles or gripples have been used by the author in high energy conditions.)

(10) Stake materials. Stakes may be made of live willow stems greater than 1 in. in diameter or they may be construction stakes (2 by 4 by 24 to 2 by 4 by 36, cut diagonally) or a mixture of the two. Reinforcing bar may be substituted only as specified below.

(11) Backfilling. Wattling should be covered immediately and seeped. Workmen are encouraged to walk on the wattling as work progresses to further work the soil into the bundles. Ten to 20 percent of the bundle should be left exposed when all construction

is completed. This allows better rooting and helps intercept water and detritus.

(12) Staking. All stakes should be driven to a firm hold and a minimum of 18 in.-deep. Where soils are soft and 24-in. stakes are not solid (i.e., if they can be moved by hand), 36-in. stakes should be used. Where soils are so compacted that 24-in. stakes cannot be driven 18 in. deep, three-eights- or one-half-inch reinforcing bar should be used for staking. When rebar is used, the tops should be bent over to hold the wattling in place.

(13) Progression of work. Work should progress from the bottom of the slope to the top and each row should be covered with soil and packed firmly behind and into the bundle by tamping or walking on the bundles or by both these methods.

(14) Prevention of drying damage. Exposure of the wattling to sun and wind should be minimized throughout the operation. Trenches should be dug only as rapidly as the wattling is being placed and covered to minimize drying soil in the trench and backfill.

b. Brush layering.

(1) Materials. Live brush of willow species should be used. When there is a shortage of willow, up to 50 percent of the brush may be of non-rooting species. When non-rooting species are used they should be mixed randomly with the rooting species.

(2) Time of work. Work should be done during the planting season specified for woody plant species, i.e. during the rainy season unless otherwise specified.

(3) Size (length of brush). Length of brush should vary according to the particular installation and should be specified on the plans. The length may vary from 2 - 6 ft to full-length brush. Hand trench brush layering used for small gully repair should be from 2 - 4 feet long. Brush layering in new fill will vary to the size of the fill but should usually be full-length brush.

(4) Vertical spacing. Vertical spacing should be as specified on plane.

(5) Hand trenching. Hand trenching should start at the bottom of the slope as for wattling placement. Trenches should be dug 24 - 36 in. into the slope, on contour, and with a downward slope of 10 - 20 deg.

(6) New fill. Brush layering should be placed on successive lifts of well-covered fill.

(7) Placement. Brush should be placed with butts inward with no more than 12 in. or less than 6 in. of the tips extending beyond the fill face. Brush should be arranged randomly, perpendicular and at angles of up to 30 deg from the perpendicular to the slope face, i.e., in a criss-crossed manner. Brush should be 3 - 4 in.-thick in hand trenched placement work and 5 - 6 in.-thick in fill work. Thickness should be measured after compression by the covering soil.

(8) Covering. Brush layers should be covered with soil immediately following placement and the soil compacted firmly. Covering may be done by hand or with machinery.

(9) Interplanting. Interplanting of woody plants (transplants and/or unrooted willow cuttings) and grasses should follow placement of the brush layering as specified for the site.

A technique was suggested by the Committee for cases where the bench was eroded to the levee toe. Longitudinal stone sections and tiebacks or deflectors can be used to re-establish the benches by promoting alternate sediment deposition and vegetation establishment that will reform the benches and protect the levee bank. Guidelines for sizing stone for the longitudinal section is provided in EM 1110-2-1601. Height of the structure should be approximately 3 ft above the existing bed and must contain sufficient material to effectively launch into local scour areas without failing the structure. (Scour estimation techniques are presented in Appendix B.) The tiebacks can be constructed to the same elevation as the top of the longitudinal section and will effectively increase roughness and promote sediment deposition. They are usually made of rock, but other materials such as logs or trees can be used. Tiebacks can also be designed to serve as deflector dikes if they are constructed by sloping the structure from the top of the bench elevation to the elevation of the longitudinal stone section. The tiebacks should be spaced no less than the length of the upstream tieback structure nor further than four times the length of the upstream tieback. Actual spacing within these constraints should be based on the following:

$$S = f\left(\frac{R_c}{W}\right)$$

where

S = spacing

R_c = radius of curvature

W = water surface width when tiebacks are just overtopped

Recommendations for long-term management

In addition to formulating plans to address immediate levee integrity, the District should develop a long-term management plan for the Pajaro Project. Based on my observations and the information provided, I believe the project has performed as intended and will continue to do so. I would anticipate a sequence of deposition, vegetation establishment, erosion and levee overtopping on a frequency consistent with that observed for these processes since the levees were constructed. I think this sequence should be allowed to continue.

Healthy riparian vegetation tends to stabilize streambanks; provides shade that prevents excessive water temperature fluctuations; performs a vital role in nutrient cycling and water quality; improves the aesthetic and recreational benefits that can be derived from a project; and is immensely productive as wildlife habitat. Vegetation should be permitted on the interior benches of the project to obtain some of these benefits. Not all species or assemblages of vegetation provide these benefits, however, and there is a tradeoff in flow conveyance. Techniques discussed during the meeting to evaluate conveyance impacts should be applied and careful consideration given to formulating a sound vegetation management plan.

Hydraulic impacts of vegetation are a function of the flow conditions, the vegetation density, the flow depth relative to the vegetation type, and the species composition of the vegetation. In general, grasses and pliable herbaceous vegetation offer less resistance than stiff, woody vegetation. Resistance increases with increasing flow depth until vegetation is overtopped, after which it decreases with increasing flow depth – so short vegetation is preferable to tall vegetation except in the uppermost margins of the project. The report "Hydraulic Impacts of Vegetation," by the author was provided to the District during the Committee meeting and offers sufficient guidance to evaluate impacts associated with herbaceous vegetation (the n-VR method is recommended). Hydraulic impacts associated with woody vegetation can be determined by computing a $C_d Veg_d$ value from calibrated conditions for the design event in the overbank areas using:

$$C_d Veg_d = \frac{2gn^2}{R^{4/3} k_n^2}$$

where

$C_d Veg_d$ = the bulk drag-density term for the vegetation

g = gravity constant

n = Manning's resistance coefficient for calibrated flow

R = hydraulic radius for calibrated flow

k_n =unit conversion for Manning's equation (1 for SI, 1.486 for English)

The above equation can be rearranged to evaluate resistance for other overbank flow conditions by changing the hydraulic radius, or can be used to assess changes in flow conditions attributable to vegetation thinning (reduced density). The above equation assumes the woody vegetation is not completely overtopped. More rigorous techniques and more complicated algorithms are required if the vegetation is overtopped. Note that when vegetation occupies only a portion of the flow area, the conveyance method in HEC-2 will overestimate conveyance because losses associated with the open channel vegetation interface are not accounted. This shouldn't present a problem when using C_dVeg_d values estimated from calibrated n values. If other techniques are used to determine n values, increase estimated n values by 10 - 15 percent to account for losses at the interface. (The 10 - 15 percent value was computed for flows 2 ft below the tops of the levees on the Pajaro and may not be applicable to other conditions or locations).

The proposal to place sycamore and cottonwood trees on the upper bench should be reconsidered. The proposed 40-ft spacing, combined with the relatively low environmental values of these species will limit their benefit. Attached in Appendix A are tables from a report by the author that list species with acknowledged environmental benefits. Consideration should be given to using species other than sycamore and cottonwood. If the desired environments can be met by woody vegetation on the banks (face) of the benches rather than the tops, it would be preferable to manage the vegetation in a manner that would maintain herbaceous vegetation on the benches. This will minimize the potential for failure of the levees due to concentration of flow lines between vegetation and the levee face, or root penetration and subsequent failure of the levee section. Maintaining the bench relatively free of woody vegetation may require an intensive monitoring and maintenance program (though not necessarily an expensive one).

The proposed (by the county's consultant) management plan for the Project includes a comprehensive planting plan to re-establish vegetation within the channel margin. This may be unnecessary as the pioneer and successional species that will naturally occupy these regions will likely be consistent with the desired vegetation community. If the proposed planting plan is implemented, I recommend that an irrigation system (preferably drip) be installed as well. Cost estimates for the plants should include a contingency to replace at least 50 percent of the plants.

James Lencioni

Senior Hydraulic Engineer
U.S. Army Engineer District, Seattle

Site inspection observations and discussions

Visual inspection of the Santa Cruz County side of the project from upstream of Thurwachter Road to the end of the project was accomplished from accessing the top of the levee. A cursory observation of the Monterey County levees was accomplished while driving along the Santa Cruz County levee on the opposite river bank. The entire reach observed was typified by lateral main channel erosion into the originally constructed bench between the levee and channel bank (30 ft minimum bench width as originally constructed). This erosion varied throughout the reach and in some locations extended almost into the levee embankment.

My impression was that the majority of the channel bank erosion observed appeared to be related more to geotechnical failures of the bank rather than hydraulic (velocity) erosion. This type of damage is usually associated with conditions where high pore pressures have occurred in the banks resulting from inadequate pressure relief during relatively rapid decreases in channel water elevations. Santa Cruz County's consultant presented some computed hydraulic data that revealed mean channel velocities of about 5.5 to 8.5 fps in the main channel and 0.5 to 3.5 fps on the benched overbanks for discharges resulting in out-of-bank flow. Such velocities appear relatively commensurate with the type of damage observed.

There appeared to be a good amount of main channel bank erosion both downstream from the Highway 1 bridge where massive vegetation clearing had not been accomplished after the 1995 flood as well as upstream of the bridge where vegetation clearing had been accomplished. This suggests to me that the presence of vegetation on the benches, in itself, was relatively insignificant as far as providing protection against lateral channel migration.

In some areas evidence suggested that the levee profile may have been raised as much as about 3 ft in the near past without commensurate embankment placement to bring the levee cross section back to original design and construction geometry. The top width was not much greater than about 10 ft and side slopes were much steeper than 1V to 2H.

My visual impression in most areas was that the elevation of the ground on the bench riverward of the levee was higher than the ground line on the landward side of the levee. If so, this would suggest that deposition was occurring on the bench which could impact the projects' level of protection. However, Santa Cruz County's consultant said that recent survey data indicates that ground elevation is essentially the same on both sides of the levee.

Some locations showed evidence of past riprap protection on the levee. However, construction plans do not indicate riprap to have been included in the original project design. Although no record exists of such protection, discussion with SPN and the local sponsor suggested that this riprap was probably the remnants of damage repaired following the floods which occurred in the 1950s and had subsequently been buried by sediment. Also, some areas of levee on both sides of the river included slope protection in the form of what appeared to be unusually small sized rock and/or manufactured fabric overlain by rock. In the case of the fabric-rock protection, all such areas observed evidenced failure.

Conclusions/recommendations

Following are the conclusions and recommendations I offer as a result of the inspection and discussions:

a. The project has functioned acceptably as designed over its approximately 50-year life. Levee overtopping occurred in 1995 and 1998 when discharges exceeded design capacity. Although some damage has occurred to the levees, the project has safely contained floods approaching, and somewhat exceeding, its design capacity. That being stated, some areas now exist following the 1995 and 1998 floods where repairs are needed to ensure continued levee integrity in the future and to maintain the original design level of protection. Some areas exist where lateral channel migration has moved the channel to within a distance constituting a threat to future levee integrity. In addition, some past repairs should be re-evaluated to determine whether they need to be revised to ensure levee integrity, particularly the bank protection attempts previously made using manufactured textile material which are not adequate and should be repaired with an acceptable design.

b. Given the proven ability of the channel to laterally migrate between the levees, some method of ensuring levee integrity over its entire length appears rather obvious. The line of defense may be either at the levee or at the channel bank. Guidance contained in EM 1110-2-1601 should be used to design riprap slope protection on the riverward face of the levee embankment if that is where the line of protection is selected. If the selected method of levee protection is set at the channel banks with reliance on the existing bench landward of the levee, or re-establishment of the bench, the 30 ft minimum bench width established in the original design should be taken as the minimum necessary to ensure levee integrity. In some areas, conditions (i.e., water velocity, existing materials, etc.) appear conducive to utilizing some bioengineering and/or non-traditional techniques-- vegetation, coir, flattened channel side slopes, longitudinal toe protection--in lieu of riprap slope protection, albeit with a somewhat greater uncertainty in effectiveness. There is very little published and proven design guidance available regarding most of these techniques, therefore their predictability is considered less certain than that of riprap. I have had very little practical experience in the application of such techniques, although some members of the Committee apparently have used them in the past with a reasonable success rate. My experience with instances where integrated vegetated slope protection has been successful is in rather low energy conditions (mean velocity less than about 5 fps) and even then properly engineered riprap toe protection is considered to be a necessity. Under the Section 32 Program, the Seattle District obtained good results in stabilizing a one-half-mile length of seriously eroding bankline experiencing geotechnical failures relatively similar to the Pajaro by mildly sloping (1V to 3H) and vegetating the bank with various species of native woody vegetation. A key element of this design included a riprap blanket extending up to the ordinary high water line with weighted riprap toe and properly designed rock filter. This project has functioned successfully for almost 20 years when exposed to numerous design-condition flow events. Unfortunately, funds were not available to monitor the engineering aspects of the project, i.e., impacts on sediment transport characteristics, conveyance, etc.

c. The designed channel width was not in concert with the river's sediment load as evidenced by the channel width decreasing over time, and channel planform does not seem to have been considered in the original design. The channel width/depth geometry should be designed to provide a stable channel geometry considering the effective (channel forming) discharge. The effective discharge can be determined by integrating the annual flow duration curve with the channel's sediment transport characteristics. Analytical procedures with which to estimate the effective discharge and stable channel geometry are contained in the computer package SAM. Design of levee protection works incorporating re-establishment of the levee setback (bench width) distance should consider the desired planform geometry in combination with the channel geometry required to conform to the effective discharge. EM 1110-2-1418 also discusses procedures to evaluate channel stability and planform.

d. I do not question claims that vegetation promotes bank stability. However, the fact that bank migration into the benches both in areas where vegetation was not cleared (downstream of Highway 1) and cleared (upstream of Highway 1) leads me to be quite skeptical of any claims suggesting that bench vegetation by itself is adequate to preserve the integrity of the levee. Where vegetation is incorporated into the protection scheme, the design of the vegetation components needs to be approached cautiously and conservatively especially from standpoint of conveyance. This implies that a technical evaluation is required to balance the density and type of vegetation with the required conveyance of the channel to meet design criteria. Recent research at the ERDC and by others have led to development of some analytic techniques to compute roughness resulting from vegetation. However, I believe that these techniques inherently contain a relatively high degree of uncertainty, therefore sensitivity evaluation of assumptions used in these techniques are required when evaluating conveyance in a vegetated regime. A detailed vegetation management/maintenance plan must be developed in close coordination with environmental interests and the local entities who will ultimately be responsible for accomplishing the maintenance to ensure that the conveyance criteria is maintained over time. Regarding future maintenance, some statements made by the local sponsors seem to indicate that maintenance costs are of a significant concern to them. Therefore, I would be extremely leery of relying on present day maintenance plans and agreements to ensure project capacity over time.

e. A numerical water surface computation model using existing and/or design condition channel geometry and top of levee elevations needs to be developed to evaluate project conveyance both for any PL 84-99 repairs as well as future, more wide-range planning and design investigations.

f. SPN requested guidance on assignment of percent failure risk for the levee Probable Failure-Point (PFP) and Probable Non Failure (PNF) water surface elevations. This is a question to be answered by a detailed geotechnical investigation of the levee. In my opinion, the existing levee stability needs to be evaluated if for no other reason than the observed levee prism does not appear to conform to present day standards. The top width and side slopes appeared much too narrow and steep, respectively, and some question as to the suitability of the levee embankment material and construction would also appear to require answering. In determining the PFP and PNF water elevations, consideration needs to be given to duration and drawdown time as well as elevation of water exposure.

Tom Pokrefke

Research Hydraulic Engineer
Coastal and Hydraulics Laboratory
U.S. Army Engineer Research and Development Center

Relative to the overall project, the Corps of Engineers should thoroughly look at the entire system for solutions. Since the project was designed for 22,000 cfs (downstream of the confluence with Salsipuedes Creek) and has passed 28,800 cfs (February 1998 flood) it appears that the project is functioning as designed; however, maybe the design flows need to be revisited. It appeared from the site visit and discussions that the vast majority of the damage was from levee overtopping which was designed for 22,000 cfs by a flow of 28,800 cfs. Areas flooding is definitely not a popular or politically acceptable situation, which is why the hydrology needs to be revisited.

Existing channel conditions

The concerns relative to the proximity of the some of the bank lines to the levees are real and need to be addressed. There were two obvious conditions that I observed during the field trip - the channel sinuosity and channel width. I made some rough computations using some of the maps provided and a relatively crude scale to make the measurements. What I determined was that from Murphy Crossing to Salsipuedes Creek the sinuosity was about 1.30 and from Salsipuedes Creek to the Highway 1 Bridge the sinuosity was 1.02. Based on maps provided it appeared the channel width (top bank to top bank) varied significantly over the project length. It appeared that one of the narrowest sections was immediately downstream of the Southern Pacific Railroad Bridge and it should be noted that there appeared to have been significant shoaling (possibly from backwater effects) upstream of this narrow section.

I reviewed Table 2, "Levee Damage Sites/Pajaro River & Salsipuedes Creek" which was provided to the Committee as read-ahead material (see Appendix D). Considering only the "A" and "B" classification (Class "C" were additional county sites) which were emergency repair and monitoring sites, respectively, I determined that many of the sites were on the outside of bends where the 1998 flood probably overscoured the bendways during the higher-flow event. It appeared, looking at the maps, that the higher flows were attempting to move in a straighter line down the channel while at the same time trying to handle (or scour) the alternate bar channels within the channel. In fact to me, it appeared the remnant low-flow channel planform, particularly in the reach from Salsipuedes Creek to Highway 1, was an alternate bar pattern within the modified channel.

Considerations for solutions

Ultimate channel design has to address the hydrology. Once that is addressed and determined, then some planform layout can be determined. It appears that some low-flow channel within the main channel needs to be incorporated in the design. The key here is that the alignment of that low- and medium-flow channel needs to fit within the overall design and not be aligned to cause bank scour. In other words, the design for the lesser events must use the revetted and erosion-resistant banks for the overall project. It would be helpful to get as many historical aerial photographs and hydrographic surveys together into one database and determine the planform that the river has taken over the years. If the 1998 flood was a highly unusual and high energy event, then the scour and resulting planform will be reworked over the next few "normal" years. Any available data after the 1955, 1958, and 1995 floods should shed some light on what happened during and following those events. Based on the presentations at the Committee meeting by the District and contractor for the City of Santa Cruz, the Pajaro River has not been actively meandering. Therefore, it could be anticipated that the active bank erosion will continue to occur mainly within the channel.

The large disparity between the sinuosity upstream and downstream of Salsipuedes Creek needs to be addressed. If significant straightening has taken place downstream of the Creek and the planform is in essence locked in place, then revetment needs to be in place for the high-flow events to protect the levees. The banks can be protected for the lower-flow events using toe protection and lesser methods, but the high-flow events need first class bank protection. Upstream of Salsipuedes Creek the required planform can be protected probably in essentially the existing alignment. In some reaches, such as on the left bank where the old piles remain (about Sta 530+00, I think) the bank is going to have to be re-established.

I'm sure that Dr. Copeland can suggest some programs, such as SAM, that will help determine required slopes which can be used to determine required channel length and sinuosity. Mr. Ed Sing probably has some suggestions in this area also. All of this will have to be determined so that the decision can be made relative to the use of P 84-99 funds. During the meeting it appeared that the District has been given some latitude in long-term fixes of hot spots. The need for re-establishment of some of the bench vegetation must be incorporated within the fix also.

John Remus

Hydraulic Engineer
U.S. Army Engineer District, Omaha

Preinspection briefing

The presentations by the District and the Counties were very helpful. The data presented suggest that the channel was not straightened to any great degree during initial construction. However, examination of the May 27, 1931, aerial photo shows some slight remnants of channel meander, or at least flood erosion out in the floodplain. This would suggest that the 1931 channel might not be the "stable" channel configuration, but the one that was in place at that time. Also, by 1931 the valley appeared to be almost completely under cultivation. If one assumes that current agricultural practices (land leveling, reclamation after floods, etc.) are indicative of past practices, then it is safe to assume that land damaged in the floods prior to the project being put in place would have been reclaimed and the channel re-established along existing property lines. Therefore, I do not feel that the 1931 channel is necessarily a naturally stable channel alignment.

Examination of the cross sections and the discharge history indicate that the channel has reacted about as would be expected. The floods were confined to a very narrow corridor between the levees. As a result the deposition from each flood was increased over the natural deposition. As the accretion on the riverward berms increased, the bankfull discharge also increased.

In the absence of regular large flood events, the alluvial processes of erosion and deposition were also altered. Basically the erosion was confined to a narrower low-flow channel, which lead to erosion of the bed (degradation). This further slowed the lateral meander process allowing the vegetation to become established on the riverward berm. This vegetation further increased berm deposition. This would appear to have diminished channel capacity. However, with the information provided it is difficult to determine the changes in channel capacity from 1955 to 1995.

The Counties indicated that the levee failures in 1995 and 1998 were the result of over topping, and not failure of the levee due to erosion of the bank or general levee instability. However, both the District and the Counties indicated that without the flood fight efforts, the levees probably would have failed due to erosion of the bank or instability problems.

Site inspection

The project was viewed from the Santa Cruz County side of the river. The District and counties highlighted a number of sites where they felt erosion was threatening the levee and areas where they had repaired and/or performed flood fight efforts. Following are my observations during the site inspection.

a. Based upon the information provided in the pre-inspection briefing and field observations, it would appear that the high banks have eroded past the "original" high bank in a relatively few locations. (Figure 5) these are: (1) Sta 132+00 to 142+00 RB; (2) Sta 164+00 to 166+00 LB; (3) Sta 176+00 to 178+00 LB; (4) Sta 219+00 to 224+00 RB; (5) Sta 243+00 to 247+00 LB; (6) Sta 275+00 to 276+00 LB; (7) Sta 296+00 to 297+00 LB; (8) Sta 323+00 to 324+00 LB; (9) Sta 333+00 to 345+00 LB; (10) Sta 374+00 to 376+00 RB; (11) Sta 415+00 to 417+00 LB; (12) Sta 422+00 to 423+00 LB.

b. There is no consistent planform for either the low-flow channel or the bankfull channel, particularly downstream of the confluence of Salsipuedes Creek. The flood planform is confined to the levee alignment. It would appear that the bankfull channel is trying to lengthen itself. This may indicate that the 1931 alignment is not a naturally stable situation, and/or there has been a change in the hydrologic input to the system. In any case the unstable situation is placing pressure on the banks, especially in the straighter reaches.

c. The locations where the banks have been stabilized are very steep. Also, the stone, although apparently well graded, is inconsistent in size. The erosion control mats have been installed incorrectly and/or have been applied to situations for which they were not intended (see Figure 6).

d. There are some areas that appear to be very stable and the erosion seems to be under control. An example of this is the reach between Sta 425+00

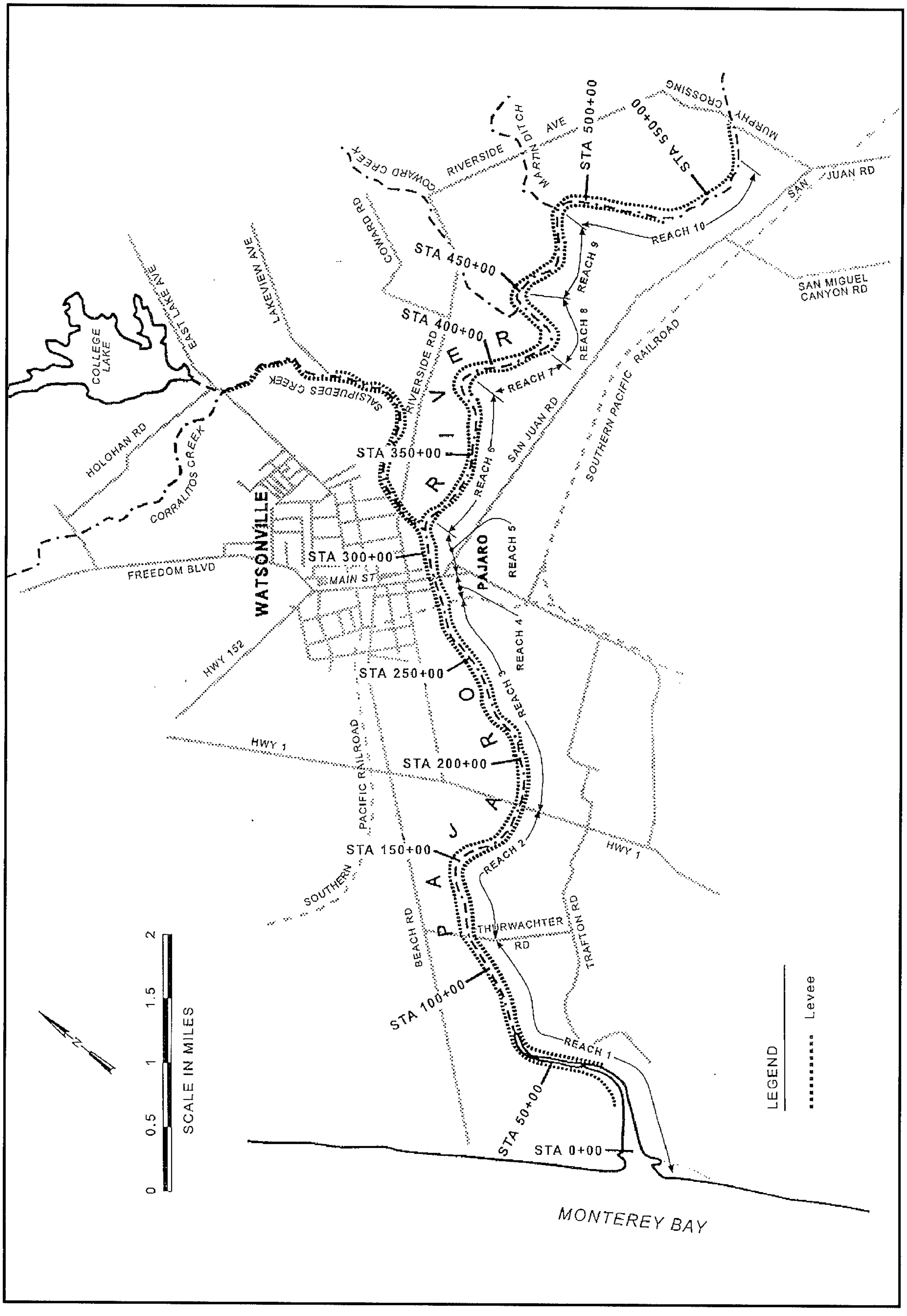


Figure 5. Location map

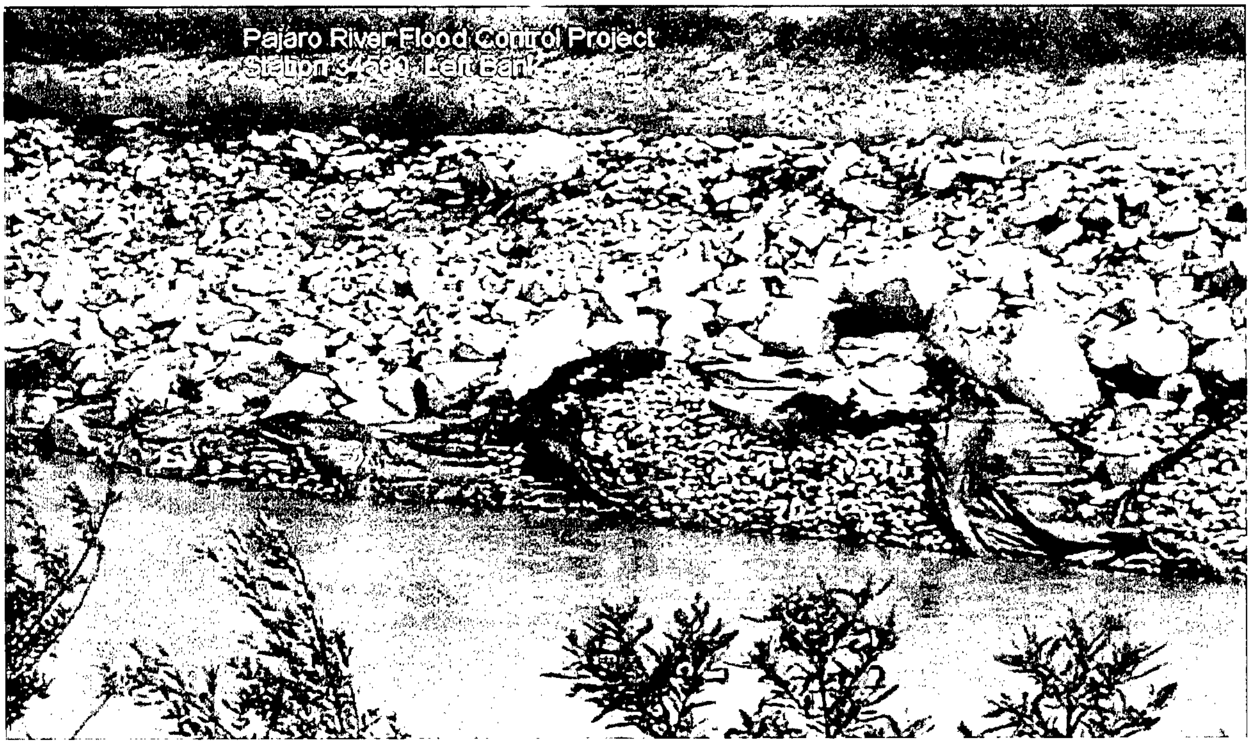


Figure 6. Incorrect installation of erosion control mats

and 430+00. It is important to note that it appears that the high banks in this area have either eroded in the past, or have never accreted in the first place. In these areas the riverward berms are narrower than the counties would like but seem to be stable. The low-level bench between the high bank and the low flow channel appears to be stable. These areas are also in the upper portion of the project where the channel is a series of straight reaches. Also, according to the information provided by Northwest Hydraulics, for the reach above the confluence of Salsipuedes Creek the bed sediment is much coarser. This will also impact stability.

Evaluation of Corps and County plans

Because the original design mentioned a 30-ft riverward berm as a minimum for the project, the District should use that as a guide for scheduling repairs and preventative maintenance. In areas where the berm is greater than 30 ft, the District needs to look at the planform to determine if the bank should be stabilized or allowed to erode in order to create greater channel capacity. The Counties indicated that there was a limited amount of funding available for maintenance and that relocating the levees was not an option from the local's perspective. However, it is important to note that as the floods are confined, the maintenance increases. Primarily the velocities along the levee are increased, deposition in the overbank increases, and reaction time to erosion problems is decreased.

Recommendations

The Committee looked at two different scenarios for bank stabilization: first, where the bank has eroded up to the toe of the levee (within 10 ft); and second, where the bank has eroded to within 30 ft of the levee toe. For the first case the Committee developed three alternatives for stabilizing the banks (Figures 7 and 8). Alternatives for the second case are shown in Figure 9.

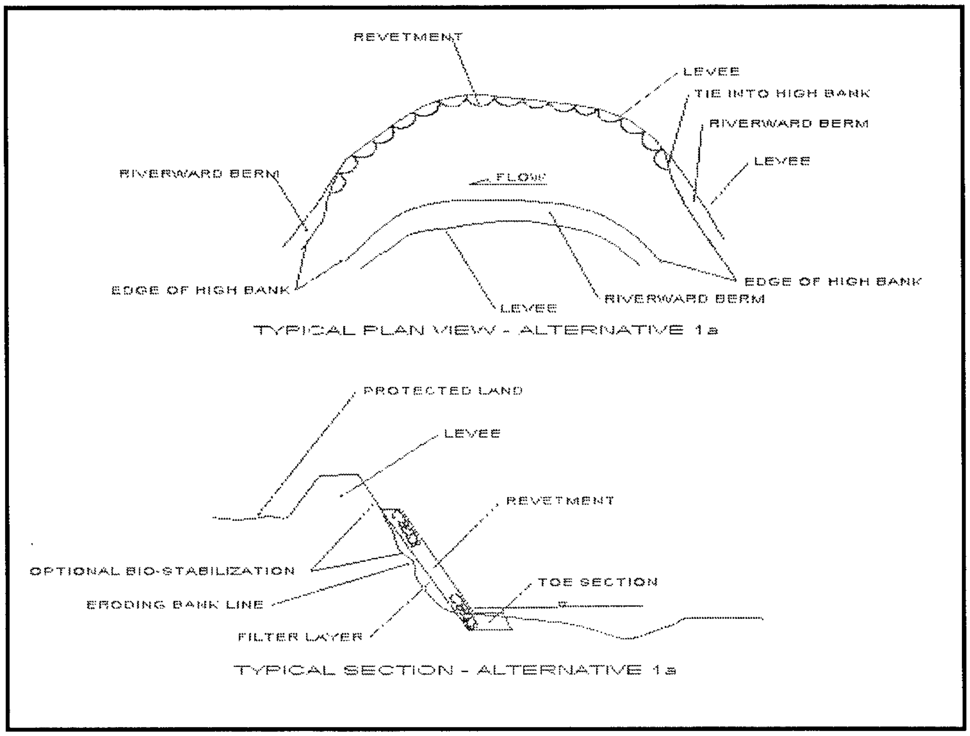


Figure 7. Alternative 1a

a. Of the alternatives for stabilizing the banks that have eroded to the toe of the levee, Alternative 1b (peak stone revetment with horizontal tie backs) is my recommended plan. This alternative will provide protection for the toe of the bank without promoting excessive deposition. However, there are risks associated with leaving a vertical bank adjacent to a levee. These risks included subsequent geotechnical failures, continued erosion during prolonged high-flow events, and decreased seepage paths. To counter some of these impacts, biostabilization could be employed above the horizontal tiebacks, or every second or third tieback could be a sloping tieback. The tiebacks should be perpendicular to the peak stone revetment, and should be spaced at 100-250-ft intervals depending on the degree of curvature in the bend. The tighter the bend the closer the spacing.

The peak stone revetment and tiebacks are shown in the figures as being constructed of stone, and this is the preferred method due to long-term reliability. However, the structures could be constructed of woven wire

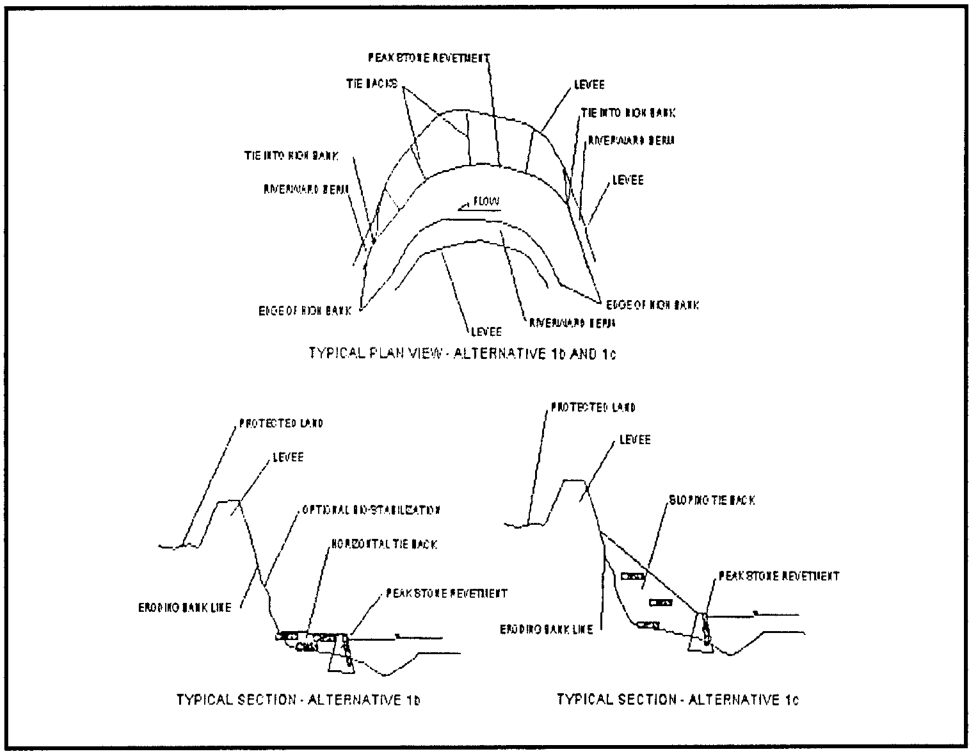


Figure 8. Alternative 1b and 1c

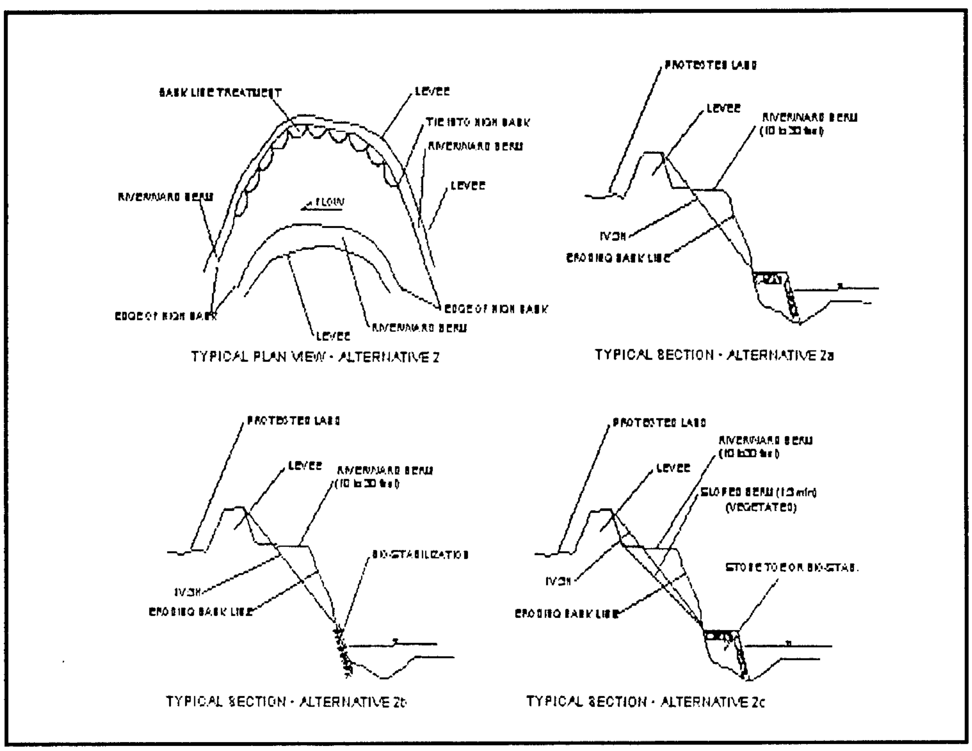


Figure 9. Alternative 2

or wood fence cribs filled with hay bales or other inexpensive material. Cribs are more labor intensive to construct and require more intensive maintenance in the first few years after construction, and they will not last as long as stone.

The height of the peak stone revetment can vary depending on the site details, but generally the tighter the bend the higher the revetment. A good rule of thumb is to construct the peak stone revetment to the elevation of the flow that is exceeded only 25 percent of the time. However, this elevation may be too high to allow vegetation to establish.

b. For the second case, my preferred alternative is the stone toe (Alternative 2a). This alternative provides a proven technique for stopping erosion at the toe, and would require the least amount of maintenance. The toe should be one-third to one-half the height of the high bank and should be built wide enough to account for any anticipated scour. Some erosion of the high bank above the stone toe is expected but shouldn't be a problem if the vegetation is managed properly. Sloping of the berm, as suggested by Mr. Carlos Hernandez, while adding some conveyance does not seem necessary. If the river is constricted, it will erode the berm.

Other alternatives

The District and Counties should revisit the levee setback option as a viable alternative. Levee setback could reduce maintenance and increase the level of protection. Realizing that the sponsors don't want the setbacks, they need to be made aware that $500,000 per year is not likely enough to maintain the project, and that if the project is not maintained, they (and the Corps) will be in a reactionary mode all the time. Bottom line, inadequate maintenance equals zero benefits.

Ed Sing
Hydraulic Engineer
U.S. Army Engineer, South Pacific Division

Past performance

It is important to note that the project levees have effectively conveyed the design discharge of 19,000 cfs (upstream from Salsipuedes Creek) in past flood events. In fact, four flood events have exceeded the design discharge since project construction. During two of those events, the levees failed from overtopping at greater than design discharge.

Maintenance

Although local representatives expressed a desire for a maintenance-free project, there is no such thing as a maintenance-free flood control project. Thus, one should expect some damage after a significant flood event and required repairs for this damage.

Repeated flood damage repairs

I do not believe there is any simple answer/reason why repeated and costly repair efforts on the project levees have been required in the past few years. Several explanations have been advanced, but I do not believe that any one of these should be considered the sole cause. In addition, I believe that additional consideration may be necessary to determine if some of these explanations have merit. Explanations that have been advanced include: a) major hydrologic events exceeding design capacity within multiple years, b) removal of dense vegetation in the channel after the 1995 flood event causing significant increases in flow velocities in the river, c) lack of structural slope protection measures (other than the "jacks" at selected locations) in the original design, d) a "domino" effect of high velocity flows "bouncing" off protected banks and attacking unprotected banks, e) loss of the berm in front of the project levees through repeated exposure over the years to erosive forces, and f) river trying to re-establish an equilibrium planform after (some) straightening of the river during project construction.

PL 84-99 repairs

There was a great deal of discussion as to which repairs could be affected under the PL 84-99 authority and which could not. It is clear in ER 500-1-1 that PL 84-99 repairs are only for damage caused by the flood. Repairs made under emergency (i.e., flood fight) conditions should be evaluated to ensure their long-term integrity and conformance to current design criteria. Any repairs should be designed and constructed to be consistent with the long-term channel planform. As the analyses required for determination of this long-term planform will probably be quite lengthy (extending beyond and outside of the scope of the PL 84-99 analyses), a best estimate of the planform limits should be made for use in developing the PL 84-99 repairs.

Some interest was expressed in the use of bioengineering slope protection measures. Although used successfully under many flow situations, the suitability of these type measures to this project reach must be carefully assessed. This assessment, as with any engineering analysis, should be performed by those with experience in this type installation.

Apparently after the 1998 flood, a portion of the project levees was "restored" to the original design profile. Recent topographic surveys indicate that some portions of this restoration may have actually exceeded the original design grade of the project levee. This is of concern due to the potential for induced flooding that this effective levee superiority may induce. An assessment should be made to ensure that the top of levee profile is in conformity with the original design.

Mike Spoor
Geotechnical Engineer
U.S. Army Engineer District, Huntington

Presentations, site visits, and discussions with local interests, consultants, and District staff were most important to understanding the project function, necessary repairs and environmental components, and proposed changes and additions to further reduce flood damages.

Evaluations, analysis, and costs will be better defined for planning, design, construction, and operation responsibilities as the District obtains survey, explorations, monitoring, maintenance, and repair data.

Since this project is located near the San Andreas Fault, detailed geotechnical characterization of levee fills and foundations and adjacent floodplain and bench soils is essential for determination of probable failure and erosion conditions and occurrences.

During the reconnaissance of 27 May 1998, topography was encountered which seemed indicative of flood flow erosional undercutting, oversteepening, related upslope failures, and truncation of benches. These erosional and failure surfaces were observed along some reaches of embankment slopes, banks, and benches which were mantled with blocks and slabs of failed soil. These failed soils may have resulted from recessional loading when the river receded from flood crests more rapidly than the silty sand to sandy silt soils could drain. Additional failures were caused by seepage-initiated internal erosion, piping, enlargement of these features and collapse of overlying soils. Some soil blocks, at and adjacent to banks and bench, were defined by open tension cracks. As these cracks intercept surface and/or ground waters, cleft pressures may cause further displacement. Secondary currents, within areas mantled with failed soils and displaced trees and debris, may cause additional erosion of adjacent inplace soils.

The levee geometries seemed to be partially defined by erosion and failure related oversteepening and seepage features, settlement, and additional fill placement within crest and slope areas. The slopes often appeared to be steep and the crest narrow when compared with design cross sections. Seepage and failures did not expose sandy gravel or gravelly sand fill material. Silty fine sand and sandy silt, similar to adjacent agricultural banks, and bench soils, were referenced as borrow areas being obtained during the construction of these levees. Failure features also contained woody debris which may have been windrowed during clearing, grubbing, and excavation to form levee fill placement surfaces.

Emergency repairs, including flood fight dumping of stone and placement and weighting of HDPE membrane, may cover other areas of bench and levee embankment failures and failed soil accumulations. Some of the dumped rock was undersized, poorly graded, or placed on overly steep slopes and will continue to launch. The seepage control membrane was displaced at locations of laps. Improper placement and anchoring sand bags on redundant membranes and dumping of stone on geosynthetic filters may result in launching.

Conditions which have occurred as a result of flood fight and other emergency actions should be fully evaluated and necessary repairs or reconstruction effected. Relevant design guidance and construction methods should be used. Excavation, to form suitable placement surfaces and slopes within inplace soils, and fill and placement of suitable filter materials and stone to stable geometries with adequate transition and toe components is necessary to complete these repairs. Where seepage paths have been shortened by erosion and failure of adjacent banks, benches and embankments, cutoff and drainage interception and control features should be further evaluated by the District. The proposed treatments would effect necessary extents of levee protection by preventing additional fill, bench and bank failures or by retaining failed soils and deposited sediments. Vegetation is a necessary treatment component except for areas with height of bench bank or bank and levee stone slope protection. Adventitiously rooted inundation-tolerant woody plantings and structures would provide protection from river flow erosion and would retain failed soils and fine sediments. However, vegetative treatments and environmental components require extensive and costly maintenance. Concerned entities should be in agreement and fully committed as regards all significant project repairs and restoration.

References to the selection of Probable Non-Failure Point (PNP) and Probable Failure Point (PFP) and the assignment of percent failure of risk to these points was reviewed by the Committee. Failure risks of 15 percent for the PNP and 85 percent for the PFP seem appropriate for recent long duration rainfall and flood events. However, for flood events with a return period of 14 years and durations of 3 to 5 days, and recessional times, from 3 feet below levee crests to toe of embankment, of 3 days, the repaired project should provide protection with a significantly lower failure risk percentage.

3 Conclusions

It was the consensus of the Committee that the original design concept was a good one, in that it had operated successfully for almost 50 years. Project overtopping occurred when the discharge exceeded design values and extensive erosion occurred when protective vegetation was removed from the benches and the banks of the main channel. It is therefore the Committee's recommendation that rehabilitation efforts focus on restoring the original setback levee configuration, using reliable bank protection measures where appropriate and more environmentally beneficial vegetative protection as much as possible. Allowing vegetative growth in the flood conveyance channel introduces the requirement for a detailed maintenance plan that is both technically and economically feasible.

The Committee commented on the appropriate level of repair work to be conducted under the PL 84-99 authority. PL 84-99 funding is intended to repair "flood caused damage." The Committee defined repairing flood caused damage as returning the channel to a condition where the levee integrity is restored. This would involve restoring the bench so the levee is set back a minimum of 30 ft, which was the minimum design setback, or armoring the levee and toe to prevent erosion. Design criteria for the rehabilitation needs to meet current standards. This would include the levee slope and levee source material. Conveyance for the design discharge must be maintained. The benefits from the rehabilitation must be greater than the costs.

The Committee recommends the following actions:

a. Determine an appropriate stable planform for the main channel of the river. A stable planform will result in less stress on the banks and therefore less bank erosion and maintenance. This is especially important in channels depending on vegetation for bank protection. This planform should be the channel's long-term stable footprint reflected by preflood conditions. That is it should be based on a "channel-forming" discharge rather than a flood discharge. This can be accomplished using historical data or by using the methodology outlined by Dr. Ronald Copeland. Rehabilitation works should conform to this long-term planform. Natural deviations from this long-term planform would indicate that additional bank stabilization works should be constructed. Mr. Pokrefke pointed out that the existing Pajaro River planform downstream from Salsipuedes Creek is considerably straighter than the planform upstream which suggests a greater potential for bank erosion downstream.

b. Erosion of the main channel that has progressed to a point adjacent to the levee should be repaired. One option is to cover the entire face of the levee with riprap. This includes providing an appropriate foundation, filter, and toe protection as recommended in EM 1110-2-1913 and EM 1110-2-1601. Another option would be to use a combination of riprap on the lower portion of the levee and a bioengineering protection on the upper portion of the levee. Proper design would require determining the applied shear stresses on the upper portion of the levee, and then selecting a bioengineering treatment that has a critical shear stress which exceeds the applied shear stress. A third option would be to build river training works to return the bench to its original 30-ft-wide dimension. This could be accomplished using spur dikes, or a longitudinal dike system with tiebacks. A variety of construction materials have been used for these purposes. Rock has been shown to be the most reliable. Mr. John Remus provided design guidance and sketches for bank stabilization methods that have been effective in the Omaha District. Mr. Jim Lencioni provided additional guidance for a project in the Seattle District. Dr. J. Craig Fischenich provided detailed information on usage of bioengineering technology and guidance for estimating toe scour and toe protection. Excerpts from the WES Stream Investigation and Streambank Stabilization Handbook, provided as Appendix C, show various examples of combined armor and bioengineering bank stabilization approaches.

c. For cases where there is still a bench adjacent to the levee, but it is less than 30-ft-wide, toe protection is recommended. This could take the form of weighted toe placement of riprap. Vegetative treatments above the weighted toe would be appropriate. Guidance provided by Mr. Remus and Dr. Fischenich are applicable here too. It was pointed out by Mr. Lencioni that bank erosion had occurred both in reaches where most of the vegetation had been removed and in reaches where vegetation removal had not occurred. Thus, vegetation by itself cannot be considered as an effective bank stabilization method.

d. The design conveyance of the channel must be maintained. This will require development of a vegetation management plan integrating both environmental and operational considerations. This will require that the sponsors and resource agencies work together to develop a feasible monitoring and maintenance plan. It should be recognized that the vegetative features associated with this project may increase the design uncertainty. This is attributed to: a) increased uncertainty associated with determining hydraulic roughness through vegetation; b) variability in vegetation density, height, and distribution that will occur regardless of the intents of the maintenance plan; and c) unexpected future staff and/or funding limitations and/or environmental issues that may affect implementation of the maintenance plan. The District was provided with recent guidance relative to estimating hydraulic roughness due to vegetation. These included the draft update to EM 1110-2-1601 based on research conducted in the Flood Damage Reduction Research Program, and papers prepared by Dr. Fischenich and others based on his research conducted in the Environmental Impacts Research Program. A numerical backwater model should be developed, based on current geometry, to establish water-surface profiles for a variety of hydraulic roughness conditions. It is recommended that the operation and maintenance manual contain pictorials and/or photographs that can be used to define the various expected roughness conditions. Estimates of sediment accumulation on the benches with various levels of vegetation density are required. It is recommended that a monitoring program be established to help determine these rates.

e. It is important that maintenance costs are accurately portrayed. Maintenance budgets are limited. There is no such thing as a maintenance free flood control project.

f. A geotechnical evaluation of the existing levees should be made. This should include a detailed geotechnical characterization of levee fills and foundations as well as adjacent floodplain and bench soils. It is the consensus of the Committee that the levees may need to be repaired to bring them up to current design standards. Based on field observations it appeared that at some locations the side slopes were too steep, the top width was too narrow, and that the fill material may be inadequate. Concerns with seepage and settlement were expressed.

The Committee on Channel Stabilization thanks the San Francisco District for hosting this 63rd meeting and for providing us the opportunity to participate in the Pajaro River planning process. This flood control project provides many unique challenges. The combined design objectives of environmental benefits and flood damage reduction are timely and are becoming more and more frequent for Corps of Engineer projects across the country. We were impressed with the District's progress to date and hope to remain informed about design plans.

References

Abt, S. R., Hamilton, G. B., Watson, C. C., and Smith, J. B. (I 994). "Riprap sizing for modified ARS-type basin," *Journal of Hydraulic Engineering,* American Society of Civil Engineers, 120(2), 260-267.

Allen, H. H., and Klimas, C. V. (1986). "Reservoir shoreline revegetation guidelines," Technical Report E-86-13, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Allen, H. H., and Leech, J. R. (1997). "Bioengineering for streambank erosion control," Technical Report EL-97-8, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Allen, J. R. L. (1970). "A quantitative model of grain size and sedimentary structures in lateral deposits," *Geological Journal,* 7, 129-146.

American Society of Civil Engineers. (1965). "Channel stabilization of alluvial rivers," *Journal of the Waterways and Harbors Division,* 91, February.

American Society of Civil Engineers. (1975). *Manual 54: sedimentation engineering.* V. A. Vanoni, ed., American Society of Civil Engineers, New York.

American Society of Civil Engineers. (1977). *Sedimentation Engineering.* V. A. Vanoni, ed., American Society of Civil Engineers Task Committee for the preparation of the Manual on Sedimentation of the Sedimentation Committee of the Hydraulics Division, New York, NY.

Amidon, R. E. (1947). "Discussion of Model study of Brown Canyon debris barriers (by K. J. Bermel and R. L. Sanks)." *Transactions,* American Society of Civil Engineers, 112.

Andrews, E. D. (1986). "Downstream effects of Flaming Gorge Reservoir on the Green River, Colorado and Utah," *Geological Society of America Bulletin,* 97, 1012-1023.

Baird, D. C., and Kluinpp, C. C. (1992). "Study of groins on the middle Rio Grande." *Hydraulic Engineering.* M. Jennings and N. G. Bhowmik, eds., Proceedings of Water Forum '92 in Baltimore, MD, American Society of Civil Engineers.

Bermel, K. J., and Sanks, R. L. (1947). "Model study of Brown Canyon Debris Barrier," *Transactions,* American Society of Civil Engineers, 112.

Biedenharn, D. S. (1983). "Channel response on the Little Tallahatchie River downstream of Sardis Dam," Proceedings of the Rivers 1983 Conference, American Society of Civil Engineers, New Orleans, LA.

__________. (1995). "Streambed degradation protection measures," Training course on streambank erosion and protection, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS (unpublished).

Biedenharn, D. S., Little, Jr., C. D., and Thome, C. R. (1990). "Effects of low drop grade control structure on bed and bank stability," Proceedings of the National Conference on Hydraulic Engineering, American Society of Civil Engineers, San Diego, CA.

Biedenharn, D. S., Combs, P. G., Hill, G. J., Pinkand, C. F., and Pinkston, C. B. (1989). "Relationship between channel migration and radius of curvature on the Red River," *Sediment transport modeling,* S. Y. Wang, ed., Proceedings of the International Symposium.

Blaisdell, F. W. (1948). "Development and hydraulic design, Saint Anthony Falls Stilling Basin," *Transactions,* American Society of Civil Engineers, 113(2342), 483-561.

Blench, T. (1969). *Mobile-Bed Fluviology.* Edmonton, Alberta, Canada: The University of Alberta Press, 28.

Boeters, R. E. A. M., Meesters, H. J. N., Scbiereck, G. J., Simons, H. E. J., Stuip, J., Swanenberg, A. T. P., Verheij, H. J., and Verkade, G. J. (1994). "Waterways with room for nature," Section 1-4, 28th Congress of PIANC in Seville, Permanent International Association of Navigational Congresses.

Bowie, A. J. (1995). "Use of vegetation to stabilize eroding streambanks," Conservation Research Report No. 43, U.S. Department of Agriculture, Agricultural Research Service, Oxford, MS.

Brice, J. C. (1981). "Stability of relocated stream channels," Report to Federal Highways Administration, Report No. FHWA/RD-80/158, Washington, DC, 177.

Brice J. C. (1984). "Planform properties of meandering rivers," *River meandering.* S. Y. Wang, ed., New York, NY. American Society of Civil Engineers, 1-15.

Brookes, A. (1988). *Channelized rivers: perspectives for environmental management.* Chichester, John Wiley & Sons.

Brown, S. A. (1985). "Design of spur-type streambank stabilization structures," FHWA/RD84/101, Federal Highway Administration, McLean, VA.

California Department of Highways. (1960). "Bank and shore protection in California highway practice," State of California, Documents Section, Sacramento, 1970.

Carlson, E. J. (1987). "Drainage from sloping land using oblique drains," Denver, CO: Hydraulics Branch, Division of Research and Laboratory Services, Engineering and Research Center, U.S. Dept. of the Interior, Bureau of Reclamation.

Carlson, E. J., and Dodge, R. A. (1963). "Control of alluvial rivers by steel jetties," *Transactions,* American Society of Civil Engineers, 128(IV), 347-375.

Chow, V. T. (1959). *Open channel hydraulics.* New York, NY, McGraw-Hill Book Company, Inc.

Clay, C. H. (1961). *Design of fishways and other fish facilities.* Department of Fisheries and Oceans, Ottawa, Canada: Queen's Printer.

Clemens, S. (1944). *Mark Twain's Life on the Mississippi.* New York, NY: The Heritage Press.

Collinson, J. D. (1978). *Alluvial sediments. In sedimentary environments and faces.* H. G. Reading, ed., New York, NY: Elsevier.

Cooper, C. M., and Knight, S. S. (1987). "Fisheries in man-made pools below grade control structures and in naturally occurring scour holes of unstable streams," *Journal of Soil and Water Conservation,* 42, 370-373.

Copeland, R. R. (1983). "Bank protection techniques using spur dikes." MP HL-83-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Coppin, N. J., and Richards, I. G. (1990). *Use of vegetation in civil engineering.* London: Butterworths.

Derrick, D. L. (1997a). "Harland Creek bendway weir/willow post bank stabilization project." *Management of landscapes disturbed by channel incision.* Conference Proceedings, S. S. Y. Wang, E. J. Langendoen, and F. D. Shields, eds., Center for Computational Hydroscience and Engineering, University of Mississippi, University, MS.

Derrick, D. L. (1997b). "Twelve low-cost, innovative, landowner financed, streambank protection demonstration projects." *Management of landscapes disturbed by channel incision.* S. S. Y. Wang, E. J. Langendoen, and F. D. Shields, eds., Conference Proceedings, Center for Computational Hydroscience and Engineering, University of Mississippi, University, MS.

Derrick, D. L., Pokrefke, T. J., Jr., Boyd, M. B., Crutchfield, J. P., and Henderson, R. R. (1994). "Design and development of bendway weirs for the Dogtooth Bend reach, Mississippi River," Technical Report HL-94-10, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Dury, G. H. (1964). "Principles of underfit streams," Professional Paper 452-A, 67, U.S. Geological Survey, Washington, DC.

Einstein, H. A. (1972). "Final remarks," *Sedimentation,* H. W. Shen, ed., Symposium to Honor Professor H. A. Einstein, Sedimentation Symposium, Berkeley, CA, 27-7.

Elliott, D. O. (1932). "The improvement of the Lower Mississippi River for flood control and navigation," U.S. Army Engineer Mississippi River Commission and Waterways Experiment Station, Vicksburg, MS.

Elliott, C. M., and Pokrefke, T. J. (1984). "Channel stabilization in a straight river reach," *River meandering.* C. M. Elliott, ed, Proceedings of Rivers '83, American Society of Civil Engineers, New York, NY.

Elliott, C. M., Rentschler, R. E., and Brooks, J. H. (1991). "Response of lower Mississippi River low-flow stages," Proceedings of the Federal Interagency Conference on Sedimentation, Las Vegas, NV.

Farabee, G. B. (1986). "Fish species associated with reverted and natural main border habitats in pool 24 of the upper Mississippi River," *North American Journal of Fisheries Management,* 6, 504-508.

Federal Highway Administration. (1985). "Design of spur-type streambank stabilization structure," U.S. Department of Transportation, Federal Highway Administration, Report No. FHWA/RD 84/101, McLean, VA.

Forsythe, P. (1985). "Performance of a grade control structure system during extreme floods," American Society of Agricultural Engineers Winter Meeting, Chicago, IL.

Fortier, S., and Scobey, F. C. (1926). "Permissible canal velocities," *Transactions,* ASCE no. 1588, 941-984.

Franco, J. J. (1967). "Research for river regulation dike design," *Journal of the Waterways and Harbors Division,* American Society of Civil Engineers, New York, NY, 93(WW 3), August.

Franco, J. J. (1982). "Summary report: model-prototype comparison study of dike systems, Mississippi River: Potamology investigations." U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Nanson, G. C., and Hickin, E. J. (1986). "A statistical analysis of bank erosion and channel migration in western Canada," *Geological Society of American Bulletin,* 97(4), 497-504.

Neill, C. R., and Galay, V. J. (1967). "Systematic evaluation of river regime," *Journal Waterways and Harbors Division,* American Society of Civil Engineers, 93 (WWI), 25-51.

Neilson, F. M., Waller, T. N., and Kennedy, K. M. (1991). "Annotated bibliography on grade control structures," Miscellaneous Paper, HL-91-4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Nunnally, N. R., and Shields, F. D. (1985). "Incorporation of environmental features in flood control channel projects," Technical Report E-85-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Odgaard, A. J., and Kennedy, J. F. (1982). "Analysis of Sacramento River bend flows, and development of a new method for bank protection," IIHR Report No. 241, Iowa Institute of Hydraulic Research, University of Iowa, Iowa City, IA.

Odgaard, A. J., and Kennedy, J. F. (1983). "River bend bank protection by submerged vanes," *Journal of the Hydraulics Division,* American Society of Civil Engineers, 109(8), 1161-1173.

Odgaard, A. J., and Mosconi, C. E. (1987). "Streambank protection by submerged vanes," *Journal of the Hydraulics Division, American Society of Civil Engineers,* 113(4), 520-536.

Osman, A. M., and Thome, C. R. (1988). "Riverbank stability analysis: Part 1, Theory," *Journal of Hydraulic Engineering,* American Society of Civil Engineers, 114(2), 125-150.

Parsons, D. A. (1960). "Effects of flood flow on channel boundaries," *Journal of the Hydraulics Division,* American Society of Civil Engineers, 86(HY4), 21-34.

Petersen, M. S. (1986). *River engineering.* Prentiss-Hall, Englewood Cliffs, NJ.

PIANC. (1987). "Guidelines for the design and construction of flexible revetments incorporating geotextiles for inland waterways," Supplement to Bulletin 57, Permanent International Association of Navigational Congresses, Brussels.

Pickett, E. B., and Brown, B. J. (1977). "Guidelines for monitoring and reporting demonstration projects," IR H-77-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Portland Cement Association. (1975a). *Soil-cement slope protection for embankments: planning and design.* Skokie, IL.

U.S. Army Corps of Engineers. (1981a). "The Streambank Erosion Control Evaluation and Demonstration Act of 1974," Final Report to Congress, Appendix B - Hydraulic Research.

__________. (1981b). "The Streambank Erosion Control Evaluation and Demonstration Act of 1974," Final Report to Congress, Appendix C - Geotechnical Research.

__________. (1981c). "Streambank Erosion Control and Demonstration Project (Section 32)," Final Report to Congress, Appendix F - Yazoo River Basin Demonstration Projects.

__________. (1981d). "Streambank Erosion Control and Demonstration Project (Section 32)," Final Report to Congress, Appendix H - Evaluation of Existing Projects.

__________. (1984). Lower Mississippi Valley Division, Vicksburg, MS, 12 July 1984 memo on revetments from Chief, River Engineering Branch to Chief, Engineering Division.

__________. (1988). Final Supplement III - Environmental Impact Statement, Sacramento River Bank Protection Project. U.S. Army Engineer District, Sacramento and the Reclamation Board, State of California.

__________. (1990). "Construction with large stone," Engineer Manual 1110-2-2302.

__________. (1991). "Hydraulic design of flood control channels," EM 1110-2-1601.

__________. (1993). "Scour and deposition in rivers and reservoirs," (HEC-6).

__________. (1994). "Channel stability assessment for flood control projects," EM 1110-2-1418.

U.S. Department of Agriculture. (1977). "Design of open channels," Tech. Rel. No. 25.

U.S. Soil Conservation Service. (1976). "Hydraulic design of riprap gradient control structures," Technical Release No. 59 and Amendment I dated 10 April 1986, U.S. Department of Agriculture, Washington, DC.

Appendix A
Compilation of Woody and Herbaceous Species Commonly Found in Riparian Systems

Table A1
A Compilation of Woody and herbaceous Species Commonly Found in Riparian Systems

Scientific Name	Common Name	Riparian Zone[1]	Value	Local
		Woody Species		
Abies amabilis	Pacific silver fir	MMF	aesthetics	NW
Abies balsamea	Balsam fir	MLF	timber, wildlife, aesthetics	N, NW
Acacia greggii	Catclaw	AET		SW
Acer macrophylum	Big-leaf maple	MMF	aesthetics	W, NW
Acer negundo	Box elder	MMF	wildlife	N, C, NW
Acer saccharinum	Silver maple	MHF	timber, wildlife, aesthetics	S, NE, C
Acer saccharum	Sugar maple	MHF	timber, wildlife, aesthetics	N, NE, C
Acer rubrum	Red maple	MHF	timber, wildlife, aesthetics, water quality	SE, NE
Aesculus glabra	Buckeye	MMF	timber	NW, N
Aesculus octandra	Yellow buckeye	MHF	timber	E, N
Allanrolfea occidentalis	Iodine bush		wildlife	C, W
Alnus oblongifolia	Alder	MMF	timber (west) wildlife (east)	NW
Alnus rhombifolia	White alder	MMF		NW
Alnus rugosa	Speckled alder	MMF		NW
Alnus tenuiflolia	Thin-leafed alder	AIF	wildlife, aesthetics	SW
Aloysia grattisima	White brush	AIT		S
Amorpha fructicosa	False indigo-bush	MFS		C
Ampelopsis arborea	Peppervine	AIT		C
Artemisia californica	Coastal sagebrush	AIT		SW
Artemisia douglasiana		AIT		W
Asimina triloba	Pawpaw	MHF		SW
Atriplex sp.	Shadescale	AET		W
				(Sheet 1 of 12)

[1] Riparian zone modifers for vegetation.
East and Pacific Northwest: MLF-Mesic low floodplain; MMF-Mesic medium floodplain; MHF-Mesic high floodplain; MTF-Mesic transitional floodplain.
West: AEC-Arid ephemeral channel; AET-Arid ephemeral transition; AIC- Arid intermittent channel; AIF-Arid intermittent floodplain; AIT-Arid intermittent transition; APC-Arid perennial channel; APF-Arid perrennial floodplain; APT-Arid perennial transition.

Table A1 (Continued)				
Scientific Name	**Common Name**	**Riparian Zone[1]**	**Value**	**Local**
		Woody Species		
Baccharis emoryi	Baccharis	AET		W
Baccharis glutinosa	Seep willow	AET		SW
Baccharius salicina	Great Plains false willow	MMF		C
Baccharis sarothroides	Desert broom	AET		W
Baccharius viminea	Mulefat	AIT		W
Betula alleghaniensis	Yellow birch	MMF	timber	N, NE
Betula fontinalis	Birch	MHF		SW
Betula nigra	River birch	MMF	timber, aesthetics	NE
Betula papyrifera	Paper birch	MHF	timber, aesthetics	NE
Betula populifolia	Grey birch	MHF	wildlife	NE
Brickella laciniata	Brickel brush	AET		W
Bumelia lanuginosa	Gum bumelia	MHF	aesthetics	SW
Campsis radicans	Trumpet creeper	AIT		SW
Carpinus caroliniana	American hornbeam	MHF	aesthetics	C, NE
Carya aquatica	Water hickory	MLF	timber, wildlife	SE
Carya cordiformis	Bitternut hickory	MMF	timber, wildlife	C, NE, S
Carya glabra	Pignut hickory	MHF	timber, wildlife	SE
Carya illinoensis	Sweet pecan	MHF	timber, wildlife, aesthetics	S
Carya laciniosa	Shellbark hickory	MHF	timber, wildlife	N, E
Carya lieodermis	Swamp hickory	MIF	timber, wildlife	SE
Carya ovata	Shagbark hickory	MHF	timber, wildlife	E, S, N
Carya pallida	Sand hickory	MHF	timber, wildlife	S, NE
Carya tomentosa	Mockernut hickory	MHF	timber, wildlife	SE
Catalpa bignonioides	Catalpa	MMF	timber, aesthetics	E
Celtis laevigata	Sugarberry	MMF	timber, wildlife, aesthetics	SE, NE, C
Celtis occidentalis	Common hackberry	MMF	timber, aesthetics, wildlife	NE, SE, C
Celtis pallida	Hackberry	AIF	wildlife	SE, SW, C
Celtis reticulata	Desert hackberry	AIF	wildlife	SW
				(Sheet 2 of 12)

Table A1 (Continued)				
Scientific Name	**Common Name**	**Riparian Zone[1]**	**Value**	**Local**
		Woody Species		
Cephalanthis occidentalis	Buttonbush	APC/MFS	wildlife	Nationwide
Cercis canadensis	Redbud	MHF	aesthetics	C, N
Cercidium floridum	Palo verde	AET		W
Cercocarpus betuloides	Mountain mahogany	AET		SW
Chamaecyparis thyoides	Atlantic white cedar	MMF	timber	E, NE
Chilopsis linearis	Desert willow	AET		W
Chrysothamnus nauseosus var. graveolons	Rabbit brush	AET		W, SW
Clematis pitcheri	Pitcher's virgin's bower	AET		C
Conium maculatum	Poison hemlock	AIT		W
Condalia hookeri	Brasil	AET		S
Comus amomum	Silky dogwood	MHF	wildlife, water quality	C, SE
Comus drummondii	Rough-leaf dogwood	MTF	aesthetics	C, N, W
Comus florida	Flowering dogwood	MTF	timber, widlife, aesthetics	E, NE, S, C
Comus stolonifera	Red-osier dogwood	MLF	wildlife, aesthetics	E, SE
Corylus americana	Hazlenut	MHF	timber	E, SE
Crataegus sp.	Hawthorn	MMF	timber	E, C
Diospyros virginiana	Persimmon	MLF	timber, wildlife	SE
Elaegnus angustifolia	Russian olive	MMF		C
Eriastrumdensifolium ssp. sanctorum	Santa Ana River wolly-star	AIC		W
Ericameria pinifolia	Pine goldenbrush	AIT		SW
Eriodictyon trichocalyx	Hairy yerba santa	AIT		SW
Euonymus atropurpureus	Wahoo	MHF		S, SW, C
Fagus grandifolia	American beech	MTF	timber, wildlife, water quality	NE, SE, C
Fallugia paradoxa	Apache-plume	AET		W, SW
				(Sheet 3 of 12)

Table A1 (Continued)				
Scientific Name	**Common Name**	**Riparian Zone[1]**	**Value**	**Local**
		Woody Species		
Forestiera acuminata	Swamp privet	MLF	aesthetics	SE, SW
Forestiera neomexicana	New Mexican olive	AET	aesthetics	W, SW
Forquieria splendens	Ocotillo	AET		W
Franseria dumosa	White bursage	AET		W
Fraxinus velutina	Velvet ash	MLF	timber, water quality	W
Fraxinus americana	White ash	MLF	water quality, aesthetics	C, S, NE
Fraxinus caroliniana	Swamp ash	MFS	aesthetics	E, SE
Fraxinus latifolia	Orgeon ash	MMF	aesthetics	NW
Fraxinus nigra	Black ash	MFS		NE
Fraxinus pennsylvanica	Green ash	MLF	aesthetics	Nationwide
Fraxinus profunda	Pumpkin ash	MMF	timber	NE, SE, C
Gleditsia aquatica	Water locust	MLF	aesthetics	SE, C
Gleditsia triacanthos	Honey locust	MHF	timber, wildlife, aesthetics	SE, C
Gordonia lasianthus	Loblolly bay	MMF	aesthetics	SE, C, NE
Gymnocladus dioicus	Kentucky coffeetree	MHF	timber, aesthetics, wildlife	NE, SE, C
Hymenoclea monogyra	Burrow weed	AET		SW
Ilex decidua	Deciduous holly	MMF/AIF	aesthetics, wildlife	Nationwide
Ilex opaca	American holly	MMF	aesthetics	Nationwide
Itea virginicia	Virginia willow	AIF	aesthethics	W, NW
Juglans cinera	Butternut	MHF	timber, wildlife, aesthetics	N, NE
Juglans nigra	Black walnut	MHF	timber, wildlife	C, E, NW
Juglans major	Nogal walnut	AET	wildlife	W
Juglans microcarpa	Little walnut	AET	wildlife	W
Juniperus californica	Californa juniper	AET		SW
Juniperus virginiana	Eastern red cedar	MTF	timber, wildlife, aesthetics, water quality	SE, E
Larix laricina	Larch	MFS		NE
				(Sheet 4 of 12)

Table A1 (Continued)				
Scientific Name	**Common Name**	**Riparian Zone[1]**	**Value**	**Local**
		Woody Species		
Larrea tridentata	Creosote bush	AET		W
Lepidospartum quamatum	Scalebroom	AET-APT		W
Lindera benzoin	Spice bush	AET		NE, E
Liquidambar styraciflua	Sweet gum	MMF	timber, wildlife	SE
Liriodendron tulipifera	Yellow-poplar	MTF	timber, wildlife, aesthetics, water quality	SE, NE
Lonicera involucrata	Ink berry	AIT		W
Lycium sp.	Boxthorn	AET		W
Lycium torreyi	Wolfberry	AIT		W
Maclura pomifera	Osage orange	MMF	timber, wildlife	S, C
Magnolia grandiflora	Southern magnolia	MHF	aesthetics	SE
Magnolia virginiana	Sweetbay	MMF	aesthetics	NE, SE
Malosma laurina	Laurel sumac	AIT		SW
Menispermum canadense	Canada moon seed	AIT		C
Morus microphylla	Mulberry	AIF	aesthethics	SW
Morus alba	White mulberry	MMF	aesthetics, wildlife	NE, C, S
Morus rubra	Red mulberry	MHF	timber, wildlife	NE, SE, C
Nyssa aquatica	Water tupelo	MFS	timber, wildlife, aesthetics	SE
Nyssa sylvatica v. biflora	Tupelo swamp	MFS	timber, wildlife, aesthetics	SE,
Nyssa sylvatica	Black gum	MFS	timber, wildlife, aesthetics	NE, SE,
Olneya tesota	Ironwood	AET		W
Opuntia littoralis	Coastal prickly pear	AET		SW
Opuntia parryi	Valley eliotis	AIT		SW
Orontium aquaticum	Golden club	AIT		S, E, C
Ostrya rubra	Hophorn beam	MHF		SW
Oxydendrum arboreum	Sour wood	MHF	wildlife	SE, NE
Parthenocissus inserta	Thicket creeper	MMF		C
				(Sheet 5 of 12)

Table A1 (Continued)				
Scientific Name	**Common Name**	**Riparian Zone[1]**	**Value**	**Local**
		Woody Species		
Parthenocissus quinquefolia	Virginia Creeper	MMF		C
Persea borbonia	Red bay	MLF	timber, aesthetics	SE
Philadelphus microphyllus	Mock orange			W
Picea glauca	White spruce	MMF	timber, wildlife, water quality	E, NE
Picea mariana	Black spruce	MMF	timber, wildlife, aesthetics	NW, NE
Picea pungens	Red spruce	MMF	timber, wildlife, aesthetics	NE
Pinus echinata	Shortleaf pine	MMF	timber	SE
Pinus elliotti	Slash pine	MMF	timber	SE
Pinus glabra	Spruce pine	MHF	timber	SE
Pinus rubens	Red pine	MHF	timber	NE
Pinus serotina	Pond pine	MMF	timber	SE
Pinus strobus	White pine	MMF	timber, wildlife, aesthetics	NE
Pinus taede	Loblolly pine	MMF	timber, water quality	SE
Pinus virginiana	Virginia pine	MHF	timber	E
Planera aquatica	Water elm	MFS	asethetics	E
Platanus occidentalis	American sycamore	MMF	timber, aesthetics	N, SE, C
Platanus racemosa	California sycamore	AET	aesthetics	W
Plantanus wrightii	Sycamore	AET		SW, NW
Pluchea sericia	Arrow weed	MHF		SW
Populus acuminata	Narrow leaf cottonwood	APC	aesthetics	SW
Populus angustifolia	Cottonwood	APC		nationwide
Populus balsamifera	Balsam poplar	APC		NW
Populus deltoides	Eastern cottonwood	MMF-AIC	timber, wildlife, aesthetics	N, SE
Populus fremontii	Fremont cottonwood	AIF	aesthetics	SW, NW
Populus grandidentata	Bigtooth aspen	MFS	timber, wildlife	N, NE
Populus sargentii	Plains cottonwood		aesthetics	SW
Populus tremuloides	Quaking aspen	MMF	timber, wildlife, water quality	NE, NW
				(Sheet 6 of 12)

Table A1 (Continued)				
Scientific Name	**Common Name**	**Riparian Zone**[1]	**Value**	**Local**
		Woody Species		
Prosopis juliflora	Mesquite	AET/MHF		C, E
Prosopis pubescens	Screwbean	AET/MHF		C, W
Prunus americana	Wild plum	AET/MHF	wildlife	C, W
Prunus ilicifolia	Holly-leaved cherry	AET		SW
Prunus serotina	Black cherry	MHF	timber, wildlife	C, NE, SE
Prunus virginiana	Common choke cherry	MTF	wildlife	C, SW
Quercus alba	White oak	MTF	timber, wildlife, water quality	NE, C
Quercus bicolor	Swamp white oak	MLF	timber, wildlife, water quality	SE
Quercus falcata var. falcata	Southern red oak	MMF	timber, wildlife, aesthetics, water quality	SE
Quercus falcata var. pagdaefolia	Cherrybark oak	MHF	timber, wildlife, water quality	SE
Quercus imbricaria	Shingle oak	MHF	timber, wildlife, water quality	SE
Quercus laurifolia	Laurel oak	MHF	timber, wildlife, aesthetics	SE
Qurecus lobata	Valley oak	MHF		E
Quercus lyrata	Overcup oak	MLF	timber, wildlife, water quality	C, N
Quercus macrocarpa	Bur oak	MLF	wildlife, aesthetics, water quality	C, SE
Quercus marilandica	Blackjack oak	MHF	timber, wildlife, water quality	E
Quercus michanxii	Swamp chestnut oak	MHF	timber, wildlife, aesthetics	S
Quercus muehlenbergii	Chinkapin oak	MHF	timber, wildlife, water quality	S, E
Quercus nigra	Water oak	MLF	timber, wildlife, water quality	SE
Quercus nuttallii	Nuttall oak	MMF-MLF	timber, water quality	S
Quercus palustris	Pin oak	MMF	timber, wildlife, aesthetics	C, NE
Quercus phellos	Willow oak	MMF/MLF	timber, wildlife, water quality	SE
Quercus prinus	Chestnut oak	MHF	timber, wildlife, water quality	C, NE
Quercus rubra	Northern red oak	MHF	timber, wildlfie, water quality	S, NE
Quercus shumardii	Shumard oak	MHF	timber, wildlife, water quality, aesthetics	C, SE
Quercus stellata	Post oak	MHF	timber, wildlfie, aesthetics	S, SE
				(Sheet 7 of 12)

Table A1 (Continued)				
Scientific Name	**Common Name**	**Riparian Zone[1]**	**Value**	**Local**
		Woody Species		
Quercus velutina	Black oak	MHF	timber, wildlfie, water quality	S, N, SE
Quercus virginiana	Live oak	MHF	timber, wildlfie, aesthetics	S, SE
Rhamnus betulaefolia	Birchleaf buckthorn	AET		W
Rhamnus crocea	Californica redberry	AET		SW
Rhus diversiloba		AIF		W
Rhus integrifolia	Lemonadeberry	AET		SW
Rhus microphylla	Little-leaf sumac	AET		W
Rhus ovata	Sugarbush	AET		SW
Rhus radicans	Poison ivy	MMF/AIF		nationwide
Ribes aureum	Golden currant	AET		SW
Ribes missouriense	Missouri gooseberry	MHF		C
Robinia pseudoacacia	Black locust	MHF	timber, wildlife	E
Rubus allegheniensis	Common blackberry	MHF/AIF	wildlife	C
Rubus hispidus	Swamp dewberry	MMF	wildlife	C
Rubus occidentalis	Black raspberry	AET	wildlife	C
Salix amydaloides	Peach-leaf willow	MLF	aesthetics	SE, C
Salix caroliniana	Carolina willow	MFS	aesthetics	SE
Salix cottettii	Bankers willow	MLF/MFS	aesthetics	SE
Salix exigua	Coyote willow	AET		W
Salix gooddingii	Southwestern cottonwood	AIF/MLF	aesthetics	SW
Salix hindsiana	Sand bar willow	AIF	aesthetics	C, N
Salix nigra	Black willow	MLF	aesthetics	SE, C
Salix purpurea	Purple osier willow	MFS	aesthetics	C
Salix scouleriana	Scouler willow	AET	aesthetics	NW
Salvia mellifera	Black sage	AET		SW
Sambucus canadensis	American elderberry	MHF		C
Sapindus saponaria	Soapberry	MMF		SW
				(Sheet 8 of 12)

Table A1 (Continued)				
Scientific Name	**Common Name**	**Riparian Zone[1]**	**Value**	**Local**
		Woody Species		
Sarcobatus vermiculatus	Grease wood	AET		W, SW
Sassafras albidum	Sassfras	MTF	timber, wildlife, aesthetics	NE, SE
Sherpherdia argentea	Buffalo-berry	AIT		W, SW
Smilax bona-nox	Bull briar	MMF		C
Smilax hispida	Bristly/greenbriar	MMF		SW
Symphoricarpus occidentalis	Western snowberry	MMF		C, NW
Symphoricarpos orbiculatus	Buckbrush	MMF		C
Tamarix pentandra	Tamarisk	APC		W
Taxodium distichum	Baldcypress	MFS	timber, aesthetics, water quality	SE
Taxodium ascendens	Pondcypress	MFS	timber, aesthetics, water quality	SE
Taxus brevifolia	Pacific yew	MMF	timber, aesthetics	NW, N
Thuja occidentalis	Northern white cedar	MFS		NE
Thuja plicata	Western red cedar	MHF		NW
Tsuga heterophylla	Western hemlock	MHF		NW
Tilia americana	American basswood	MLF	timber	NE
Toxicodendron radicans	Kuntze poison ivy	MHF		C
Toxicondendron rydbergii	Redberg poison ivy	MMF		C
Ulmus alata	Winged elm	MHF	timber, aesthetics	S, SE
Ulmus americana	American elm	MMF	timber, wildlife, aesthetics	C, NE, SE
Ulmus crassifolia	American cedar	MMF	wildlife	C, NE
Ulmus pumila	Siberian elm	MMF	timber	C
Ulmus rubra	Slippery elm	MMF	timber	C
Vitis cinera	Graybark grape	MMF		C
Vitis girdiana	Wild grape			S, W, C
				(Sheet 9 of 12)

Table A1 (Continued)				
Scientific Name	**Common Name**	**Riparian Zone[1]**	**Value**	**Local**
		Woody Species		
Vitis mustangensis	Mustang grape	AET		SW
Vitis riparia	River-bank grape	AIT		W
Vitis vulupina	Winter grape	AET		C
Yucca whipplei	Yucca	AET		SW
		Herbaceous		
Agrostis	Bentgrass	MTF		C
Alopercurus sp.	Fox-tail	MHE/AET	wildlife	nationwide
Arundo donax	Giant reed	AIT	aesthetics	SE, SW
Bidens sp.	Beggars-ticks	MLF		C
Bromus diandrus	Ripgut brome	AET	wildlife	SW, S
Bouteloua sp.	Grama	MMF, AIF	wildlife	nationwide
Carex sp.	Sedge	MHF, AET	wildlife, aesthetics	nationwide
Catabrosa aquatica	Brook grass	MTF	wildlife	NW, C
Centrostegia lepioceras	Slender-horned spine flower	AET		SW
Chlorogalum pomeridianum	Soap plant	AET		W
Commelina sp.	Dayflower	MMF		C
Cyperus sp.	Flat-sedge	MLF/AIF		nationwide
Cypres esculentus	Chufa	AIC		W
Desmodium sp.	Tickclover	AIT		C
Distichilis stricta	Salt grass	AIF		nationwide
Echinoochloa sp.	Barnyard grass	MLF		C
Eleocharis sp.	Spikerush	AIF		nationwide
Elymus sp.	Wild rye	MTF	wildlife	N, C, W
Eragrostis pectinacea	Lovegrass	MIT		N, C, W
Erigeron sp.		AET		nationwide
Eriogonum fasciculatum	California buckwheat	AET		SW
				(Sheet 10 of 12)

Table A1 (Continued)				
Scientific Name	**Common Name**	**Riparian Zone[1]**	**Value**	**Local**
		Herbaceous		
Equisetum sp.	Horsetail	MMF/AIF		nationwide
Euphorbia maculata	Spotted spurge	AET		N, C, W
Euphorbia marginata	Snow-on-the-mountain	AET		N, C, W
Festuca pratensis	Meadow fescue	MMF		C
Festuca octoflora	Six-week-fescue	AET		N, C, W
Fimbristylis sp.		AET		nationwide
Galium sp.	Bedstraw	MHF	Wildlife	nationwide
Gaura coccinea	Scarlet guara	AIT		N, C, W,
Glyceria striata.	Fowl manna grass	MHF	Wildlife	C
Helianthus grosseserratus	Sawtooth sunflower	MMF		C
Helianthus petiolarus	Plains sunflower	AIT		N, C, W
Helianthus tuberosus	Jerusalem artichoke	MMF		C
Hordeum sp.	Barley	AIT		nationwide
Juncus. sp.	Rush	AIT		nationwide
Koeleria cristata	Junegrass	AET		N, C, W
Leersia oryzoides	Cut grass	AET		W
Leptocholoa sp.	Sprangle top	MFS		C
Liatris punctata	Blazing star	AET		N, C, W
Luzula sp.	Wood-rush	AET		C, W
Lycopus americanus	American bugleweed	AET		N, C, W
Lysimachia ciliata	Skeleton weed	AET		N, C, W
Lythrum dacotanum	Fringed loosestrife	AIT		N, C, W
Medicago sativa	Alfalfa	AET		N, C, W
Melilotus albus	White sweet clover	AET		N, C, W
Muhlenbergia cuspidata	Plains muhly	AET		C
Muhlenbergia sylvatica	Forest muhly	MLF		C
				(Sheet 11 of 12)

Table A1 (Continued)				
Scientific Name	**Common Name**	**Riparian Zone[1]**	**Value**	**Local**
		Herbaceous		
Panicum sp.	Panic grass	AET		N, C, W
Phragmites communis	Reed	AIT		nationwide
Phalaris arundinacea	Reed canary grass	MLF		nationwide
Phyla cuneifolia	Wedge leaf frog fruit	MMF		C
Phyla lanceolata	Lance leaf frog fruit	MFS		C
Polygonum s p.	Smartweed	AIT		nationwide
Polypogon sp.	Rabbitfoot	AET		nationwide
Potentilla sp.	Cinquefoil	MMF		C
Ranunculus sp.	Buttercup	MLF		C
Rumex crispex	Curly dock	MLF		C
Sanicula canadensis	Canada Sanicle	MHF		NE
Scirpus spp.	Bulrush	MHF		nationwide
Typha latifolia	Cat-tail	MMF		W, S, SW
Viola sp.	Violet	MMF	aesthetics	C,
Xanthium gallica	Cocklebur	AIC		SE, NE
				(Sheet 12 of 12)

Appendix B
Calculation of Scour Potential

Dr. J. Craig Fischenich

Total scour on a river is composed of three components 1) general scour, 2) contraction scour, and 3) local scour. In general the components are additive.

General scour

The change in river bed elevation (aggradation or degradation) over long lengths and time due to headcuts and changes in hydrology, controls such as dams, sediment discharge, or river geomorphology is termed general scour. General scour often occurs during the passage of a flood, but is sometimes masked because sediments deposit to the original lines and grades on the falling stage of the hydrograph. General scour involves the removal of material from the bed and banks across all or most of the width of a channel. This type of scour may be natural or man induced, and requires geomorphologic and sedimentation analyses to quantify. Analytical tools such as HEC-6 are helpful in evaluating long-term general scour.

Contraction scour

The scour resulting from the acceleration of the flow due to either a natural or anthropogenic contraction (such as a bridge), or both, is called contraction scour. This type of scour also occurs in areas where revetments are placed in a fashion that they reduce the overall width of the stream segment. Contraction scour is generally limited to the length of the contraction, and perhaps a short distance up- and downstream, whereas general scour tends to occur over longer reaches.

Laursen's equation (Laursen 1960) given below is often used to predict the depth of scour, y_s, in the contracted section. Laursen's equation for a long contraction will overestimate the depth of scour at the upstream end of the contraction or if the contraction is the result of bridge abutments and piers, but at this time it is the best equation available. Note that the Manning's n ratio can be significant in cases where sand bed channels have variable bed forms (e.g. a dune bed in the uncontracted reach and a plain bed, washed out dunes or antidunes in the contracted reach.

$$\frac{y_c}{y_a} = \left(\frac{Q_c}{Q_a}\right)^{6/7} \left(\frac{W_a}{W_c}\right)^{A} \left(\frac{n_c}{n_a}\right)^{B}$$

$$\textit{and} \quad y_s = y_c - y_a$$

where

y_a = average depth in the main reach

y_c = average depth in the contracted section

W_a = width of the main reach

W_c = width of the contracted section

Q_c = flow in the contracted section

Q_a = flow in the main reach

n_c = Manning n for contracted section

n_a = Manning n for main reach

A and B are transport coefficients from the following:

V*/ω	A	B
<0.5	0.59	0.07
1.0	0.64	0.21
>2.0	0.69	0.37

where

V_* = shear velocity, $(gyS_f)^{0.5}$

ω = fall velocity of the D_{50} of bed material

y = water depth

S_f = energy slope

Local scour

The scour occuring at a pier, abutment, erosion control device, or other structure obstructing the flow is called local scour. These obstructions cause flow acceleration and create vortexes that remove the surrounding sediments. Generally, depths of local scour are much larger than general or contraction scour depths often by a factor of 10. Local scour can affect the stability of structures such as riprap revetments, leading to failures if measures are not taken to address the scour.

Factors affecting local scour include: 1) width of the obstruction; 2) projection length of the obstruction into the flow; 3) length of the obstruction; (4) depth of flow; (5) velocity of the approach flow; 6) size of the bed material; 7) angle of the approach flow (angle of attack); 8) shape of the obstruction; 9) bed configuration; 10) ice formation or jams; and 11) debris.

Width of obstruction has a direct affect on the depth of scour. With an increase in width there is an increase in scour depth. Though most empirical relations do not address this factor, it is probably the obstruction width relative to the channel width that is most important.

Projected length of an obstruction into the stream affects the depth of scour. With an increase in the projected length of an abutment into the flow there is an increase in scour. However, there is a limit on the increase in scour depth with an increase in length. This limit is reached when the ratio of projected length into the stream to the depth of the approaching flow is about 25:1.

Length of a structure has no appreciable affect on scour depth for straight sections, but when the structure is at an angle to the flow the length has a very large effect. At the same angle of attack, doubling the length of a structure increases scour depth by as much as 33 percent. Some equations take the length factor into account by using the ratio of structure length to depth of flow or structure width and the angle of attack of the flow to the structure. Others use the projected area of the structure to the flow in their equations.

An increase in flow depth can increase scour depth by a factor of two or larger. For bridge abutments, the increase is from 1.1 to 2.15 depending on the shape of the abutment. Scour depth also increases with the velocity of the approach flow.

Size of the bed material affects scour depth, though the effect is generally a function of the time exposed to erosive flows. In other words, sediment size may not affect the ultimate or maximum scour but only the time it takes to reach it. Large particles in the bed material such as cobbles or boulders may armor plate the scour hole.

The angle of attack of the flow to the pier or abutment has a large affect on local scour as does the shape of the structure. Structures angled such that they cause flow convergence increase scour whereas structures angled such that they cause divergence of flow lines generally decrease scour. Streamlining structures reduces the strength of the horseshoe and wake vortices, effectively reducing ultimate scour depths.

In streams with sand bed material the shape of the bed (bed configuration) affects the turbulence and flow velocity which, in turn affect the depth of scour. Ice and debris can increase both the local and general (contraction) scour. The magnitude of the increase is still largely undetermined. But debris can be taken into account in the scour equations by estimating the amount of flow blockage (decrease in width) in the equations for contraction scour.

Two simple relations for estimating local scour depths along structures follow. Both have been modified by the author from research conducted by others. The first is based upon Laursen's (1980) approach for scour at a bridge abutment and the second upon Froehlich's (1987) equations for live-bed scour at bridge crossings. Guidance for computing local scour at the toe of a riprap revetment is also given in EM 1110-2-1601. These techniques are based on empirical approaches and have high standard errors of estimates.

Modified Laursen:

$$\frac{y_s}{y_a} = 1.3 \left(\frac{W_O}{y_a} \right)^{0.48}$$

Modified Froehlich:

$$\frac{y_s}{y_a} = 2 \left(\frac{\theta}{90} \right)^{0.13} \left(\frac{W_O}{y_a} \right)^{0.43} F_r^{0.61} + 1.0$$

where

y_s = scour depth

y_a = depth of flow at the structure

W_o = length of structure projected normal to flow.

θ = angle of embankment to flow.

F_r = Froude number of flow upstream of abutment.

The modified Laursen equation is based on sediment transport relations. It gives maximum scour and includes contraction scour. FOR THIS EQUATION, DO NOT ADD CONTRACTION SCOUR TO OBTAIN TOTAL SCOUR AT

THE ABUTMENT. The Modified Froehlich equation does not include contraction scour, but does include a safety factor (+1.0) that effectively accounts for contraction scour in most cases. Values computed from either method should be increased by $y_a/6$ if dunes are the expected bed form.

Design Considerations

When designing a riprap section to stabilize a streambank, the author accounts for scour in one of two ways: 1) by excavation to the maximum scour depth and placing the stone section to this elevation, or 2) by increasing the volume of material in the toe section to provide a launching apron that will fill and armor the scour hole. Preference is generally given to the second option because of ease of construction, cost and environmental impacts associated with excavation of the streambed.

The volume of material added to the toe section must be sufficient to armor to the ultimate depth of scour. The author uses a somewhat conservative approach that assumes the side slope in the scour hole is 1:2 and that the requisite thickness of the launched armor layer should be twice the D_{100} of the riprap gradation. Thus, the volumetric increase in the size of the toe section is given by:

$$Vol = \frac{d_{100}\sqrt{5\,y_s^2}}{13.5}$$

where

Vol = volume (in cy) of riprap required

d_{100} = largest size of stone in the riprap (in ft)

y_s = estimated scour depth (in ft)

Appendix C
Excerpts from WES Stream Investigation and Streambank Stabilization Handbook

Longitudinal Stone Toe

Description

Longitudinal stone toe is another form of a windrow revetment, with the stone placed along the existing streambed rather than on top bank. The longitudinal stone toe is placed with the crown well below top bank, and either against the eroding bankline or a distance riverward of the high bank. Typical crown elevations may vary but are commonly between 1/3 and 2/3 of the height to top bank.

The success of longitudinal stone toe protection is based on the premise that as the toe of the bank is stabilized, upper bank failure will continue until a stable slope is attained and the bank is stabilized. This stability is usually assisted by the establishment of vegetation along the bank.

Advantages

A longitudinal stone toe has the same advantages as a trenchfill and windrow. It also allows for the preservation of much of the existing vegetation on the bank slope, and encourages the growth of additional vegetation as the bank slope stabilizes. An additional advantage is that the treatment is amenable to the planting of additional vegetation behind it.

Disadvantages

A longitudinal stone toe also has the same disadvantages as trenchfill. By definition, longitudinal stone toe protection only provides toe protection and does not directly protect mid and upper bank areas. Some erosion of these mid and upper bank areas should be anticipated during long-duration, high energy flows, especially before these areas stabilize and become vegetated.

Typical applications

Longitudinal stone toe protection is especially suitable where the upper bank slope is fairly stable (due to vegetation, cohesive material, or relatively low flow velocities), and erosion can be arrested by placing a windrow along the toe of the bank. This avoids the wasted effort of disturbing, then rearmoring, an existing stable slope. Small or ephemeral streams are especially suited to this approach.

The longitudinal stone toe technique may be appropriate where the existing stream channel is to be realigned, although for maximum effectiveness the top elevation of the stone must be high enough that it is not overtopped frequently. In this application, it actually functions as a retard.

Design considerations

There are basically two variations of the longitudinal stone toe. These will be referred to as longitudinal peaked stone toe protection, and longitudinal stone fill toe protection. Design consideration for these two stabilization measures are discussed below.

Longitudinal peaked stone toe protection. An efficient design for a longitudinal stone toe is to simply specify a weight or volume of stone to be placed per unit length of streambank, rather than to specify a given finished elevation and cross-section dimensions. This basically results in a triangular shaped section of stone placed along the toe of the streambank. This type of protection is commonly referred to as a longitudinal peaked stone toe protection (Figures C1 and C2). A primary attraction of this treatment is its simplicity. Extensive surveys and analysis during design and construction would reduce that attraction. Since the volume of stone required at each section is determined by the estimated scour depth, simply specifying a volume or weight is all that is required. In the small streams of north Mississippi, longitudinal peaked stone toe protection placed at a rate of 1 to 2 tons per linear foot of streambank has proven to be one of the most successful bank stabilization measures used in that area. This generally results in a height of stone between 3 and 5-ft-high above the streambed. A "typical" cross section can be specified on the drawings, along with a relatively smooth alignment to fit site conditions. During construction, the selected alignment for the structure is flagged, and increments of length are measured as appropriate for the size of delivery vehicles or placement buckets. Design, bidding, and supervision of construction is, therefore, greatly simplified.

With longitudinal peaked stone toe protection, the establishment of vegetation landward of the structure is a critical component for a successful project. Consequently, it is important to maintain as much of the natural

Figure C1a. Longitudinal peak stone toe protection immediately after construction

Figure C1b. Same site one year after

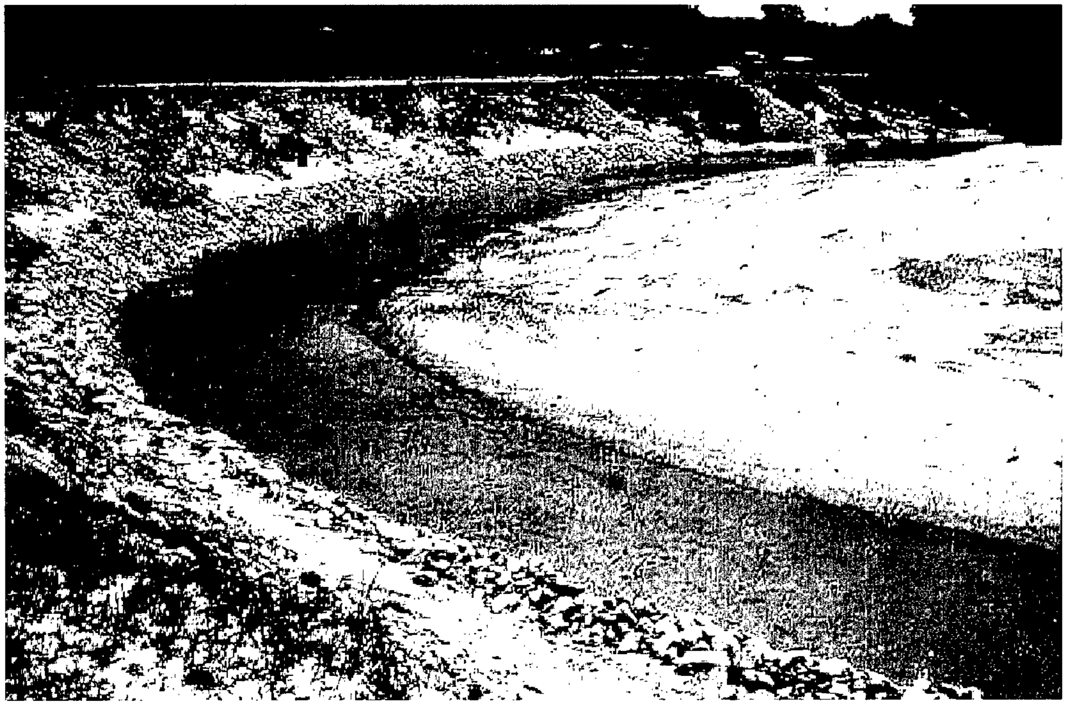


Figure C2a. Typical longitudinal peak stone toe protection with tiebacks

Figure C2b. Typical longitudinal peaked stone toe protection with tiebacks vegetation as possible. If at all possible, the construction site should be approached and the construction work accomplished from the riverward side of the bank to leave the existing upper bank vegetation undisturbed. Longitudinal peaked stone toe protection is easily combined with vegetative treatments for a composite design (Figure C3).

Figure C3. Longitudinal peaked stone toe protection in combination with willow post upper peak protection

The centerline of the longitudinal peak stone toe protection should be constructed along a smooth alignment, preferably with a uniform radius of curvature throughout the bend. The upstream and downstream ends of the structure should be protected against flanking and eddy action.

Where the bank materials are highly erodible, and the adequacy of an unsupported stone placed along the toe of the bank may be marginal, stone dikes can be placed at intervals as "tiebacks" to prevent erosion from forming behind the structure. A spacing of one to two multiples of channel width can be used between tiebacks. At the very least, a tieback at the downstream limit of the structure is recommended.

Longitudinal stone fill toe protection. With longitudinal stone fill toe protection, a top elevation and crown width for the stone are specified, along with bank grading and/or filling to provide for a consistent cross-section of stone. The finished product could just as easily be classified as a thickened stone armor to provide a launchable toe, with the top elevation of the armor being well below top bank elevation. In fact, this method is sometimes referred to as reinforced revetment. There are two basic configurations of longitudinal stone fill toe protection. One method is to place the toefill stone adjacent to the high bank with the tieback stone fill placed in trenches excavated into the high bank as shown in Figure C4. In some instances it may be necessary to place the toefill stone riverward of the high bank as shown in Figure C5. Longitudinal stone fill toe protection is often used as the toe protection with other methods for upper bank protection.

Longitudinal stone fill toe protection can be "notched" in the same manner as a transverse dike or retard in order to provide an aquatic connection between the main channel and the area between the structure and the bank slope.

Other Self-Adjusting Armor

Some armor materials other than stone which have the ability to adjust to scour, settlement, or surface irregularities are:

a. Concrete blocks.

b. Sacks filled with earth, sand, and/or cement.

c. Soil-cement blocks.

Materials which have been occasionally used in the past, but which have serious shortcomings, are:

a. Rubble from demolition of pavement or other source.

b. Slag from steel furnaces.

c. Automobile bodies.

Figure C4. Longitudinal stone fill toe protection placed adjacent to bank with tiebacks

Figure C5. Longitudinal stone fill toe protection riverward of high bank with tie backs

Appendix D
Excerpts from Information Report, U.S. Army Engineer District, San Francisco

TABLE OF CONTENTS

Information Report
For
U.S. Army Corps of Engineers'
Committee on Channel Stabilization

Pajaro River Federal Flood Control Project

1. GENERAL BASIN INFORMATION

The Pajaro River drains an area of approximately 1,300 square miles located in the coastal mountain range of Central California and empties into the Pacific Ocean six miles west of the City of Watsonville. The basin is approximately 88 miles long and 30 miles wide with normal annual precipitation ranging from 13 to 44 inches; the entire basin average is approximately 19 inches. Approximately 25 percent of the basin is cultivated, 45 percent is range land, 25 percent is covered by brush and forest, and the rest is developed. Snowfall is rare and has no noticeable effect on flood runoff. There are six major reservoirs in the basin. All, except for one, were mainly constructed for the purpose of water supply and do not have a major impact on flood flows in the lower portion of the basin during major flood events.

Major tributaries to the Pajaro River include Salsipuedes Creek, which has a basin area of 57 square miles. The creek drains the southern slopes of the Santa Cruz Mountains and meanders through the eastern part of Watsonville before it joins the Pajaro River in Watsonville. Plate 1 displays a map of the Pajaro River Basin.

2. PROJECT DESCRIPTION

a. As Constructed. The Corps of Engineers constructed a flood control project in 1949 along the lower reaches of the Pajaro River where it divides the counties of Monterey (to the south) and Santa Cruz (to the north) to protect Watsonville, the Town of Pajaro and several thousand acres of prime agricultural land.

The 1949 project included levees along the river from its mouth to river mile (RM) 11.8 on the north bank and to RM 10.6 on the south bank. In general, levees were constructed to 5 to 10 feet in height with a 12-foot crown, and 1V to 2.5H river side slopes and 1V to 2H land side slopes. Levees were typically offset from the main channel to allow for benches with vegetation to protect the benches from the river's erosive flows. In turn, the vegetated benches protected the levees, since hard revetment was not intended to be a major project feature. In areas where levees were close to the main river channel, jacks and wire-mesh revetment were used to provide and promote the establishment of vegetation and benches. The project offered much more of a natural-channel-type look than many of the District's other flood control projects. The 1949 project also included construction of levees on Salsipuedes Creek from its confluence at the Pajaro River to RM 2.6 on the west bank to RM 1.7 on the east bank. While the map on Plate 2 shows a footprint of the levee project, the photograph on Plate 3 (dated June 1971) shows an aerial view of the project at Highway 1.

The project was designed to provide safe protection against floods on the Pajaro River up to a discharge of 22,000 cfs below the confluence with Salsipuedes Creek and 19,000 cfs above that point. The design flow capacity on Salsipuedes Creek was 3,400 cfs. At the time, the annual exceedence probability (AEP) was thought to be about 2 percent (50-year event). Based on recent flow frequency analysis, the AEP for these same flows is about 7 percent (river) and 10 percent (creek).

b. Recent Project Modifications By Local Sponsors. The project's local sponsors, Monterey and Santa Cruz Counties, slightly modified the project in the fall of 1997 with the intent to maintain a consistent level of flood protection throughout the project, particularly along levee reaches bordering developed areas. Concerns were raised after high-water-mark data, from a winter storm in 1997, indicated that variations in freeboard along the project were excessive. Consequently, levees along both sides of the Pajaro River were raised an average of 1.5 feet above design grade from RM 3.7 at Highway 1 to RM 6.7, about 4,000 feet upstream of the mouth of Salsipuedes Creek.

c. Present Day. The project today bears little resemblance to the project of old, due to the major damage caused by a flood of record in February 1998. While levees, for the most part, remain intact following extensive emergency repair work, they are now highly vulnerable to scour/erosion damage under high flow conditions, due to the extensive loss of protective river benches and vegetation. Without these protective features, the levees will more than likely require very large amounts of hard-type revetment to maintain project integrity. The photograph on Plate 4 presents a view of the project just upstream of Highway 1.

d. Proposed Improvements Under G.I. Authority. The District is in the latter stage of a conducting a general investigation, or general reevaluation, study under a 1961 project authorization. The study includes plans to modify the levees along Salsipuedes Creek and also extend the tributary project so that it includes Corralitos Creek, a tributary to Salsipuedes Creek. Plans to modify the main stem levees were evaluated during the reconnaissance phase (completed in March 1995), but were found to be economically unfeasible.

Once a preferred plan is selected, it will more than likely call for the raising of existing levees along Salsipuedes Creek and natural banks along Corralitos Creek via floodwalls and/or compacted berms. Typical floodwall and levee heights would be 4 to 6 feet. The project would be designed to minimize the use of unnatural bank protection and would require little, if any, new channelization.

3. RECENT FLOOD DAMAGES

Since construction of the 1949 project, four major flood events have occurred on the

Pajaro River. The peak discharges for these events, all of which exceeded the 19,000 cfs design discharge upstream of the Salsipuedes creek confluence, are presented in the following table:

Table 1
Major Peak Flows / Pajaro River @ Chittenden (1,188 sq.mi.)

Date	Peak Discharge (cfs)	Annual exceedence Probability (%)*	Return Period (years)
December 1955	24,000	6	17
April 1958	23,500	6.5	15.5
March 1995	21,600	7.5	13
February 1998	28,800	4	25

* Based on stream gauge data through February 1998 Storm

While all of the storms caused breaching and/or overtopping of the Pajaro River levees to some extent, the March 1995 storm was by far the most devastating. Floodwaters inundated the entire town of Pajaro and several hundred acres of prime agricultural land causing an estimated $90 million in flood damages. While the City of Watsonville was threatened, it only sustained minor flood damages. Flood waters ponded behind the left (south) bank levee at the State Highway 1 bridge, requiring its breaching in order to drain the large amount of water that had accumulated. Plate 5 provides a map of the March 1995 flood plain.

Although the February 1998 flood event, which is now the flood of record, was considerably larger in flow magnitude than the March 1995 event, flood plain damages were much less. Floodwaters caused a major levee breach at RM 3.35 (right bank), approximately 1500 feet downstream of Highway 1 and well downstream of developed areas. Photos on Plate 6 show the levee shortly after it was breached, and then repaired. Mainly agricultural-type land was flooded (see map on Plate 5), which resulted in flood damages of less than $2 million. However, as previously noted, scour and erosion damage to the project itself was extensive. A total of 12 damage sites were repaired under the Corps' PL 84-99 authority at a cost of nearly $7 million. Because of the extremely wet conditions that prevailed which precluded the use of any biotechnical slope protection, essentially all repair work included the placement of large quantities of rock.

4. PROJECT REHABILITATION WORK UNDER PL 84-99 AUTHORITY

In addition to the 12 damage sites, which were repaired under the Corps' emergency authority, 14 sites were damaged enough to require close monitoring. Furthermore, Santa Cruz County has identified 14 additional sites and has requested that they also be considered for repair. Monterey County has also requested assistance to repair 8 sites, though they are included with the 14 monitoring sites. Most, if not all, of these sites will need to be revisited and evaluated for possible rehabilitation-type work. The following table lists all the sites, while the map on Plate 7 shows site locations.

Table 2
Levee Damage Sites / Pajaro River & Salsipuedes Creek

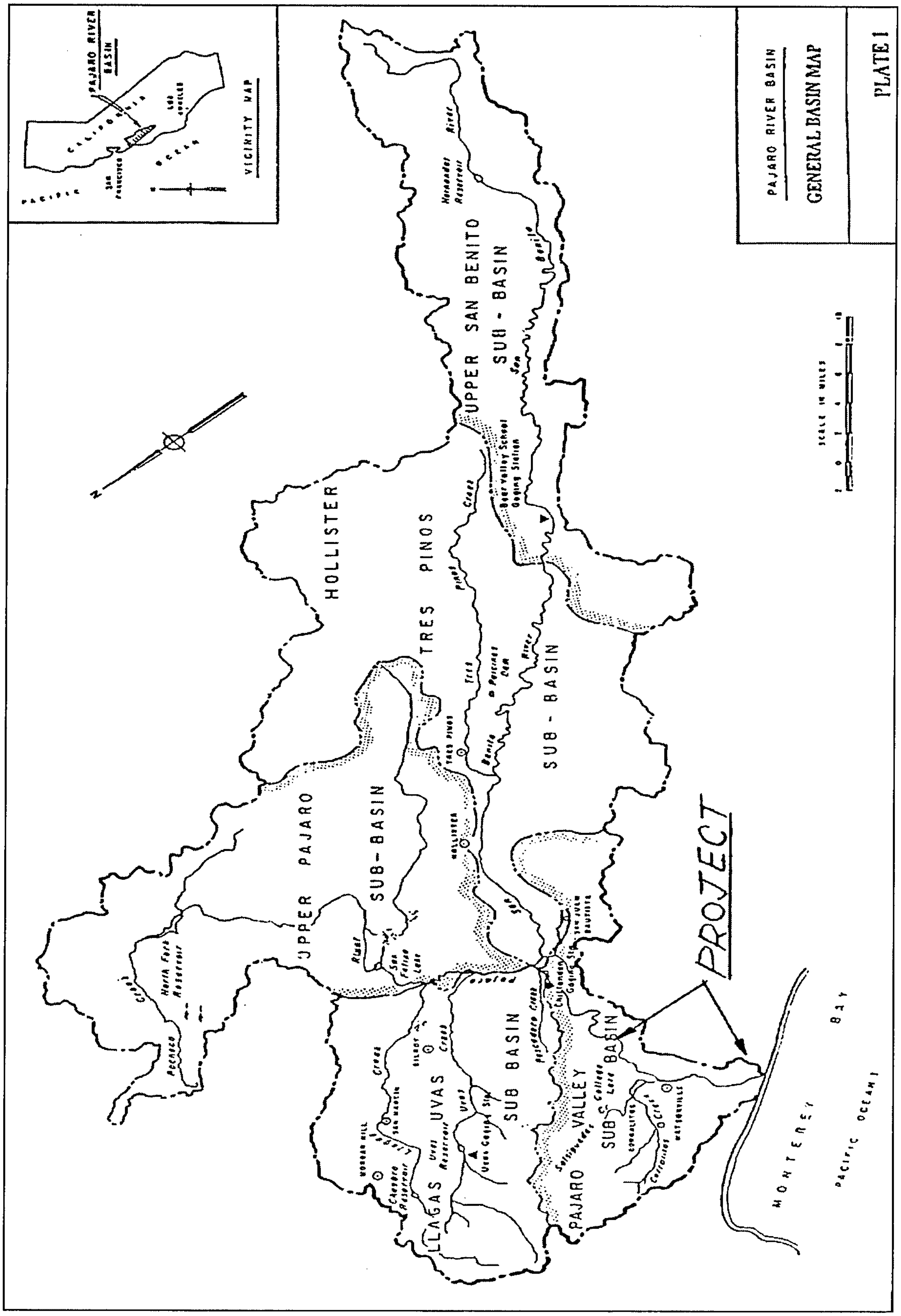
Site No.	Location (river mile)	Station (feet)	Length (lineal ft.)	E (erosion) S (seepage)	Remarks * A / B / C
	2.54 R	134+00	30	S	C
	2.59 R	137+00	120	S	C
16	2.80 R	148+00	300	E	B, C
27	3.1 R	163+00		S	B
1	3.2 R	169+00		Breach	A
	3.3 R	174+00	20	S	C
	3.62 R	191+00	550	E	C
	3.67 R	194+00	35	S	C
3	3.7 L		200	E	A
17	3.75 L		150	E	A
25	4.4 R		550	E	B
19	4.5 L		100	E	B
26	4.6 L		90	E	B
4	5.0 L		1000	E	A
	5.45 R	288+00	20	S	C
9	5.6 L		540	E	A
	6.04 R	319+00	300	E	C
	6.04 R	319+00	20	S	C
11	6.1 L		450	E	A
2	6.3 R			S	B
12	6.4 L		250	E	B
13	6.6 L		370	E	B
	6.65 R	351+00	1300	E	C
7	7.0 L		200	E	A
22	7.0 L		100	E	B
23	7.05 L		200	E	B
8	7.1 L		140	E	A
	7.12 R	376+00	100	E	C
15	7.8 R		230	E	A
	8.05 R	425+00	1500	E	C
14	8.2 L		300	E	B
	8.47 R	447+00	500	E	C
28	8.7 R		260	E	B
24	8.8 L		100	E	B
	8.9 R	470+00	900	E	C
29	9.1 R		70	E	B
5	9.6 R		300	E	A
6	0.1 R (Sal.)		220	E	A
	0.25 R (Sal)		20	S	C
2	0.5 R (Sal.)		400	S	A

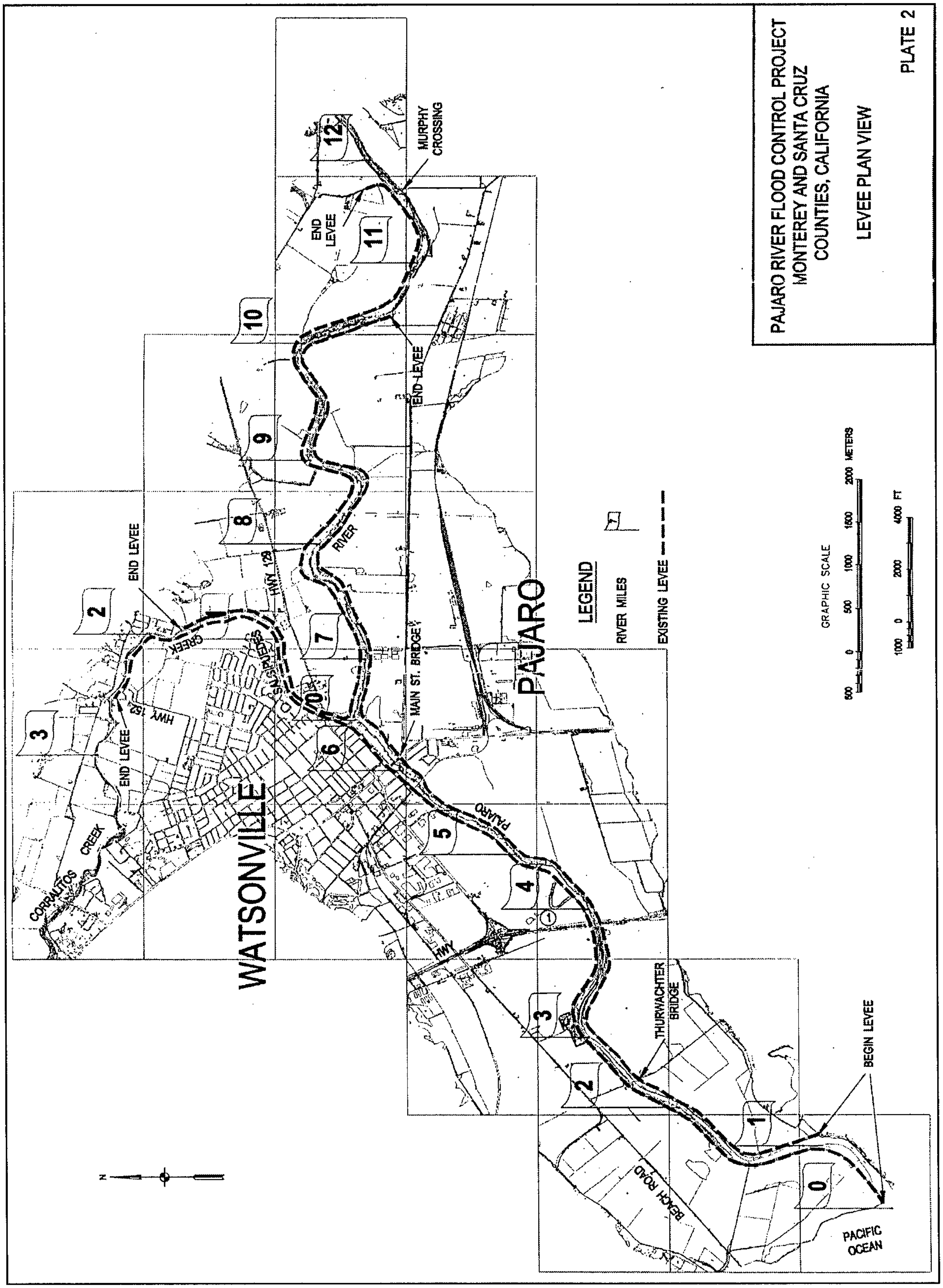
* A – emergency repair site / B – monitoring site / C – additional county site

Study funds have recently been received to prepare a project information report (PIR) that should include several damage sites. It is expected that the report will justify the rehabilitation of several sites, and that repair efforts will have been completed before the next winter in order to inhibit further damage. Unfortunately, because of the extensive damage that has occurred to the project, it is unrealistic to believe that any rehabilitation work that is completed this year will result in a fully sound project – particularly one that is sensitive to both environmental and flood control interest groups. **It is quite evident that the full rehabilitation of the project, if possible, will be time consuming, costly, and complicated. Any guidance, insights and/or recommendations that the Channel Stabilization Committee can provide to "jump-start or point the rehabilitation effort in the right direction" is strongly encouraged.**

5. SECTION 216 STUDY

In addition to the funds received to prepare a PIR, the District has received funds in the amount of $25,000 to evaluate the need to modify the project under the authority of Section 216 of the 1970 Flood Control Act, due to the drastic change in the project's condition. As study funds will not support a detailed risk-based analysis for benefit determination, a reconnaissance level analysis is expected. Key to the analysis will be the selection of appropriate probable nonfailure points (PNPs) and probable failure points (PFPs) for the current levee system and any rehabilitated levee system, and the assignment of the "% failure of risk" to these points. **According to the new EC 1110-2-554, Risk-Based Analysis in Geotechnical for Support of Planning Studies, PGL No. 26-based failure risks of 15% and 85% for the PNP and PFP, respectively are typically much too high. Hence, the Committee's thoughts on what % failure risk to assign the respective points would be most welcomed.**


PAJARO RIVER BASIN
GENERAL BASIN MAP
PLATE 1
VICINITY MAP
PAJARO RIVER BASIN
UPPER SAN BENITO SUB - BASIN
HOLLISTER
TRES PINOS
SUB - BASIN
UPPER PAJARO SUB-BASIN
LLAGAS UVAS SUB BASIN
PAJARO VALLEY SUB BASIN
PROJECT
MONTEREY BAY
PACIFIC OCEAN


PAJARO RIVER FLOOD CONTROL PROJECT
MONTEREY AND SANTA CRUZ
COUNTIES, CALIFORNIA
LEVEE PLAN VIEW
PLATE 2
LEGEND
RIVER MILES
EXISTING LEVEE
GRAPHIC SCALE
WATSONVILLE
PAJARO
CORRALITOS
CREEK
HWY 152
HWY 129
END LEVEE
SALSIPUEDES
MAIN ST. BRIDGE
PAJARO
RIVER
MURPHY
CROSSING
THURWACHTER
BRIDGE
BEGIN LEVEE
BEACH ROAD
PACIFIC
OCEAN

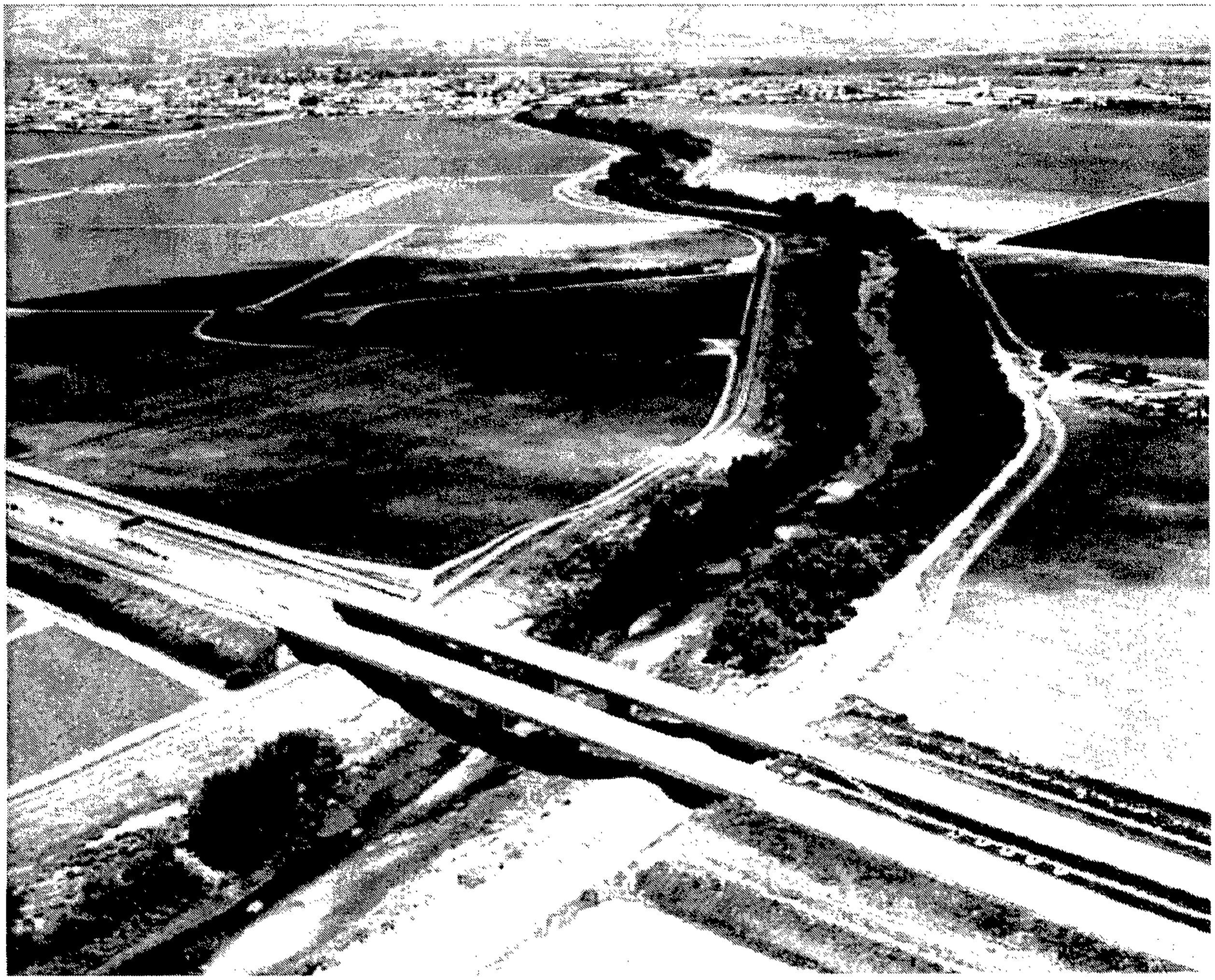
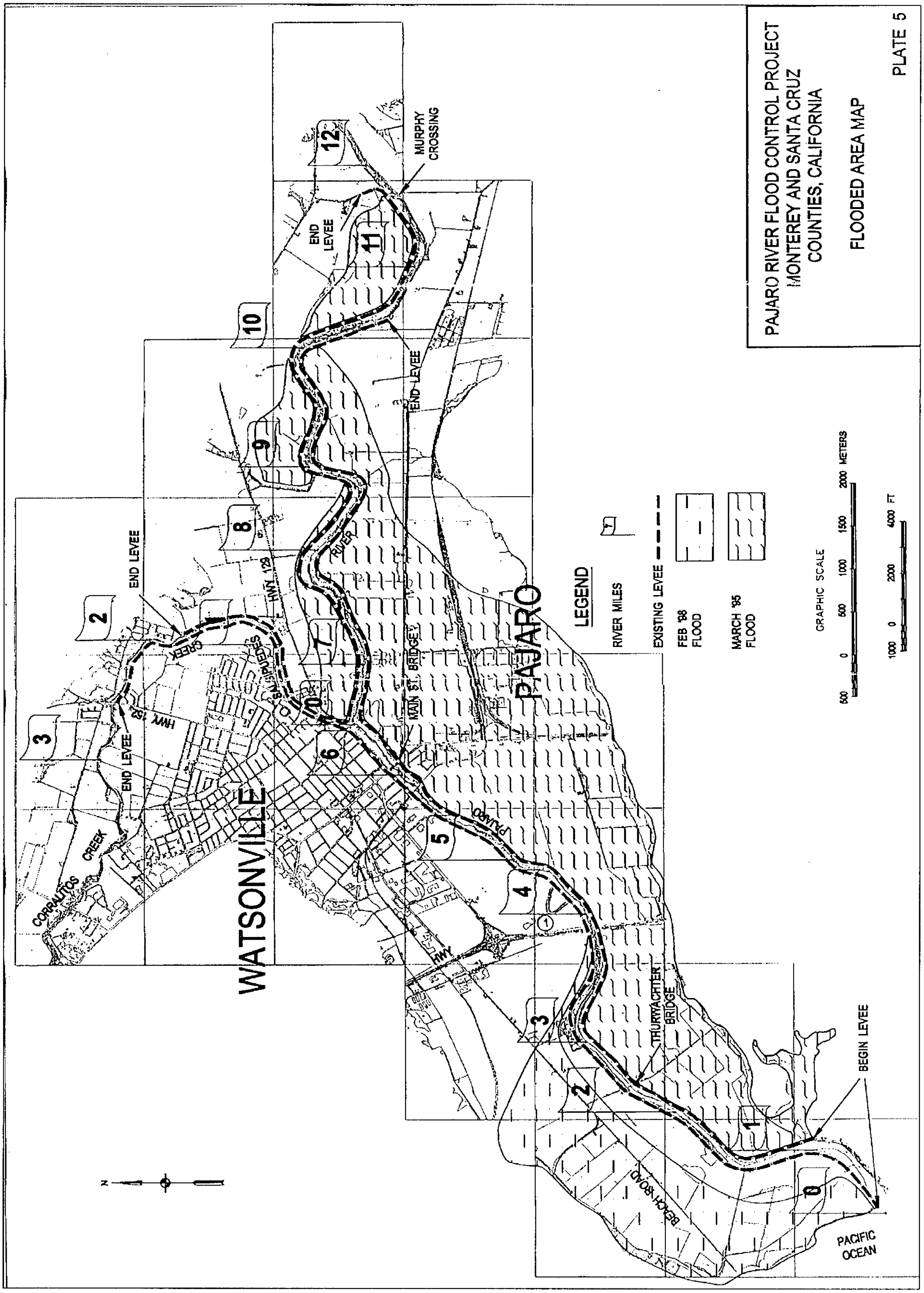


Plate 3
Aerial View of Project at Highway 1, June 1971

Plate 4
Upstream View of Project at Highway 1, May 1998


PAJARO RIVER FLOOD CONTROL PROJECT
MONTEREY AND SANTA CRUZ
COUNTIES, CALIFORNIA
FLOODED AREA MAP
PLATE 5
LEGEND
RIVER MILES
EXISTING LEVEE
FEB '98 FLOOD
MARCH '95 FLOOD
GRAPHIC SCALE
METERS
FT
WATSONVILLE
PAJARO
MURPHY CROSSING
END LEVEE
HWY 129
HWY 152
SALSIPUEDES
CREEK
CORRALITOS
CREEK
MAIN ST. BRIDGE
PAJARO
RIVER
THURWACHTER BRIDGE
HWY 1
BEGIN LEVEE
BEACH ROAD
PACIFIC OCEAN

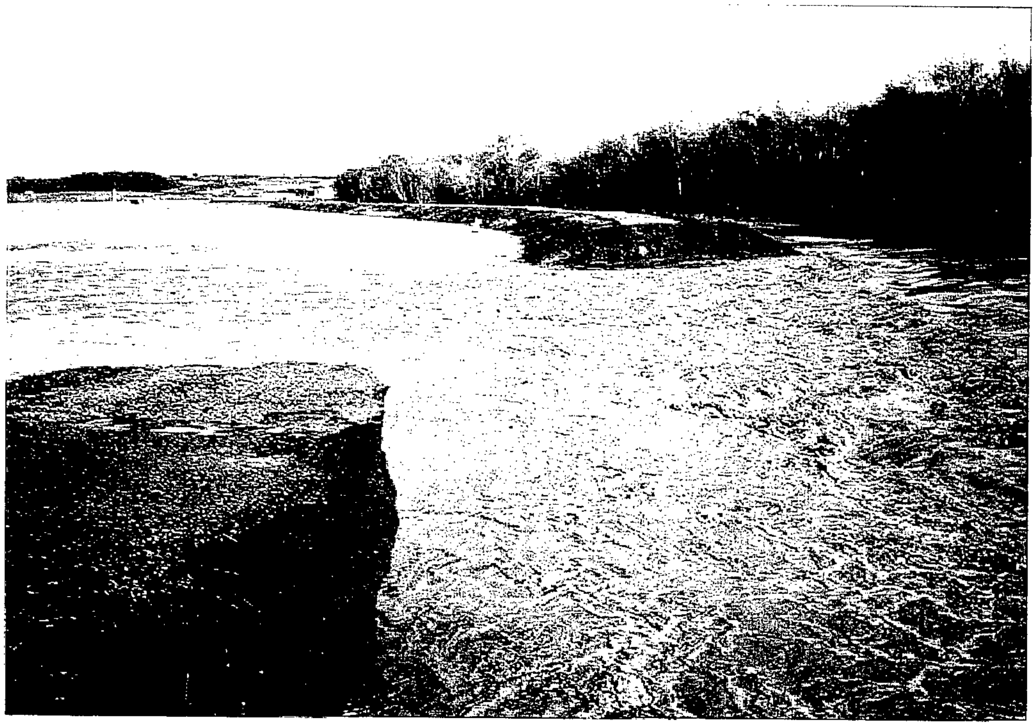
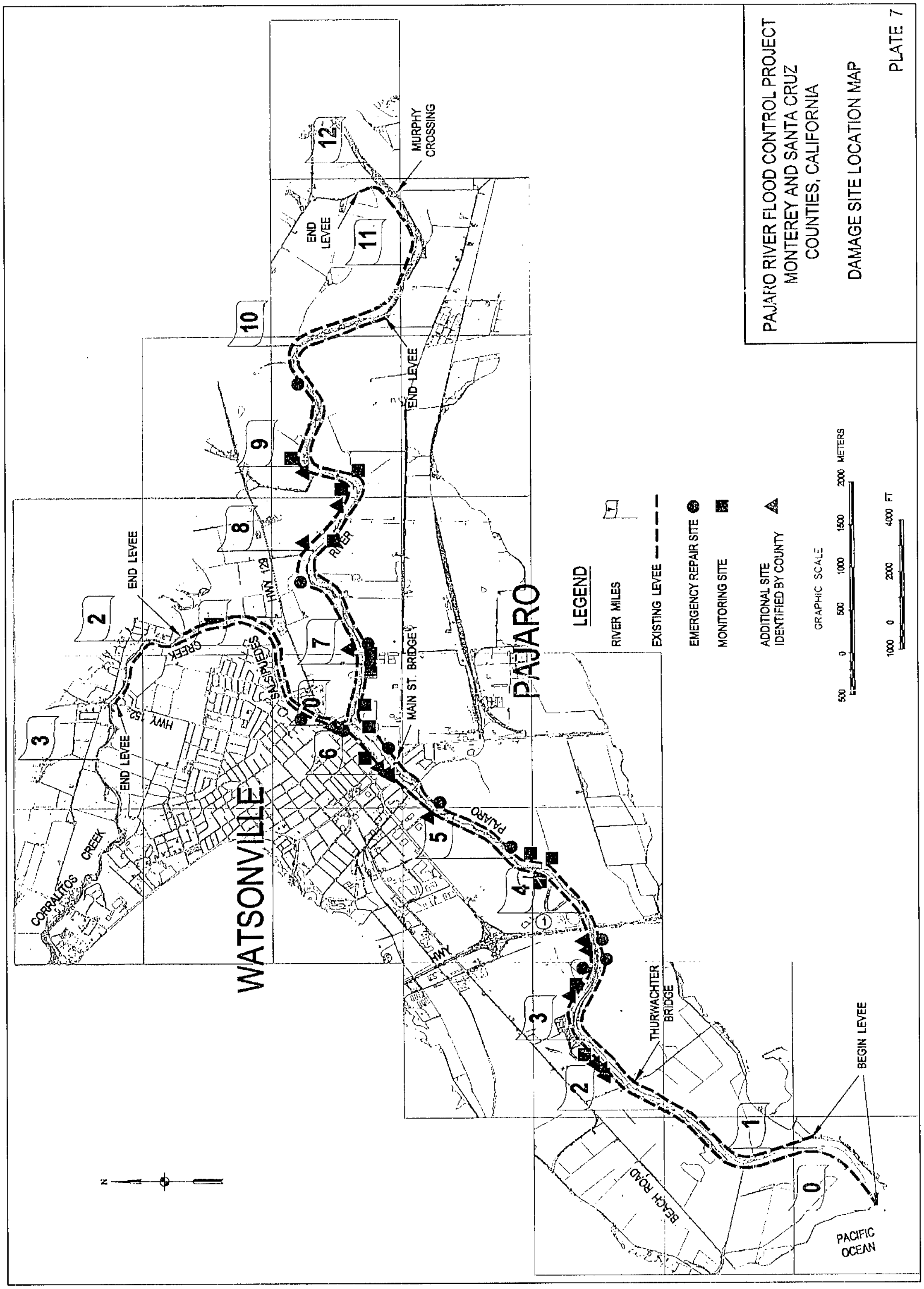


Plate 6
Site 1, Levee Breach and Repair Photographs


PAJARO RIVER FLOOD CONTROL PROJECT
MONTEREY AND SANTA CRUZ
COUNTIES, CALIFORNIA
DAMAGE SITE LOCATION MAP
PLATE 7
LEGEND
RIVER MILES
EXISTING LEVEE
EMERGENCY REPAIR SITE
MONITORING SITE
ADDITIONAL SITE
IDENTIFIED BY COUNTY
GRAPHIC SCALE
500 0 500 1000 1500 2000 METERS
1000 0 2000 4000 FT
WATSONVILLE
PAJARO
CORRALITOS CREEK
SALSIPUEDES CREEK
PAJARO RIVER
HWY 152
HWY 129
HWY 1
MAIN ST. BRIDGE
THURWACHTER BRIDGE
BEACH ROAD
MURPHY CROSSING
END LEVEE
BEGIN LEVEE
PACIFIC OCEAN

REPORT DOCUMENTATION PAGE	*Form Approved* *OMB No. 0704-0188*

1. REPORT DATE *(DD-MM-YYYY)*	**2. REPORT TYPE**	**3. DATES COVERED** *(From - To)*
August 2000	Final Report	

4. TITLE AND SUBTITLE Channel Stability Problems, Pajaro River, Watsonville and Pajaro, California; U.S. Army Engineer Committee on Channel Stabilization Report of the 63th Meeting	**5a. CONTRACT NUMBER** **5b. GRANT NUMBER** **5c. PROGRAM ELEMENT NUMBER**
6. AUTHOR(S) Ronald R. Copeland and Dinah N. McComas, editors	**5d. PROJECT NUMBER** **5e. TASK NUMBER** **5f. WORK UNIT NUMBER**
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) AND ADDRESS(ES) U.S. Army Engineer Research and Development Center Coastal and Hydraulics Laboratory 3909 Halls Ferry Road Vicksburg, MS 39180-6199	**8. PERFORMING ORGANIZATION REPORT NUMBER** ERDC/CHL SR-00-3
9. SPONSORING / MONITORING AGENCY NAME(S) AND ADDRESS(ES) Headquarters, U.S. Army Corps of Engineers Washington, DC 20314-1000	**10. SPONSOR/MONITOR'S ACRONYM(S)** **11. SPONSOR/MONITOR'S REPORT NUMBER(S)**

12. DISTRIBUTION / AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

13. SUPPLEMENTARY NOTES

14. ABSTRACT

The U. S. Army Engineer Committee on Channel Stabilization participated with the U.S. Army Engineer San Francisco District to provide guidance, insights, and/or recommendations regarding the District's rehabilitation work on the Corps of Engineer Pajaro River Flood Control Project. This project, located in both Santa Cruz and Monterey Counties, California, consists of levees on both sides of the river's main channel. The project protects valuable agricultural lands and the cities of Watsonville and Pajaro. The project levees were overtopped during floods in both March 1995 and February 1998 causing extensive property damage as well as damage to the flood control project itself. Both floods exceeded the project's design discharge. Vegetation that had become established in the flood control channel prior to March 1995 had been removed before February 1998. It was the consensus of the Committee that the original design concept was a good one in that it had operated successfully for almost 50 years. Project overtopping occurred when the discharge exceeded design values and extensive erosion occurred when protective vegetation was removed from the benches and the banks of the main channel. The Committee recommended that rehabilitation efforts focus on restoring the original set-back levee configuration, using reliable bank

(Continued)

15. SUBJECT TERMS

Bank stability	Pajaro River	Toe protection
Levees	Stable Channel	

16. SECURITY CLASSIFICATION OF:			**17. LIMITATION OF ABSTRACT**	**18. NUMBER OF PAGES**	**19a. NAME OF RESPONSIBLE PERSON**
a. REPORT	**b. ABSTRACT**	**c. THIS PAGE**			**19b. TELEPHONE NUMBER** *(include area code)*
UNCLASSIFIED	UNCLASSIFIED	UNCLASSIFIED		94	

Standard Form 298 (Rev. 8-98)
Prescribed by ANSI Std. 239.18

14. (Concluded)

protection measures where appropriate and vegetative protection as much as possible. Allowing vegetative growth in the flood conveyance channel introduces the requirement for a detailed maintenance plan that is both technically and economically feasible. This will require that sponsors and resource agencies work together to develop a feasible monitoring and maintenance plan. It should be recognized that the vegetative features associated with this project may increase the design uncertainty.