Assessment of Structural Flood-Control Measures on Alluvial Fans

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1. INTRODUCTION

The Federal Emergency Management Agency (FEMA) prepares Flood Insurance Rate Maps (FIRMs) based on the results of Flood Insurance Studies (FIS). Those studies determine the areas with a i-percent annual chance of being inundated. The flood is called the **Base** (100-year) Ffood by FEMA. The FIS must evaluate the existing flood conveyance system, including installed flood-control measures. The determination of the inundated area may depend on whether flood-control measures protect part of the floodplain. With increased development in the arid western portions of the United States, more people are being exposed to the extreme flood hazards associated with flash floods, mud and debris flows, high flow velocity, channel avulsion, erosion and channel migration on alluvial fans. The dominant fluvial processes operating on alluvial fans are episodic in nature and are poorly understood. Nevertheless, one must consider whether flood-control measures perform well on alluvial fans, and if so, whether their performance can be evaluated.

1.1 Geomorphology of Alluvial Fans

Alluvial fans are depositional landforms, developed over a geologic time scale, located at the base of mountain ranges where ephemeral mountain streams emerge onto the lesser slopes of the valley floors (French, 1997). They are usually conical, or fan shaped in plan view. On topographic maps, alluvial fans appear as contour lines that are concentric about a canyon mouth (fan apex). Figures l-l and 1-2, respectively, are plan and profile views of an idealized alluvial fan. The figures also illustrate some of the terms used in this report.

The broad use of the term “alluvial fan” in the geologic literature does not imply the existence of any specific hydraulic processes. Trends in deposition and erosion are episodic and locally variable on alluvial fans.

There are fluvial systems which are not on the typical conical fan but maintain some of the characteristics of systems on alluvial fans, Arroyos are examples of distributary drainage networks with flow characteristics similar to those of a typical fan.

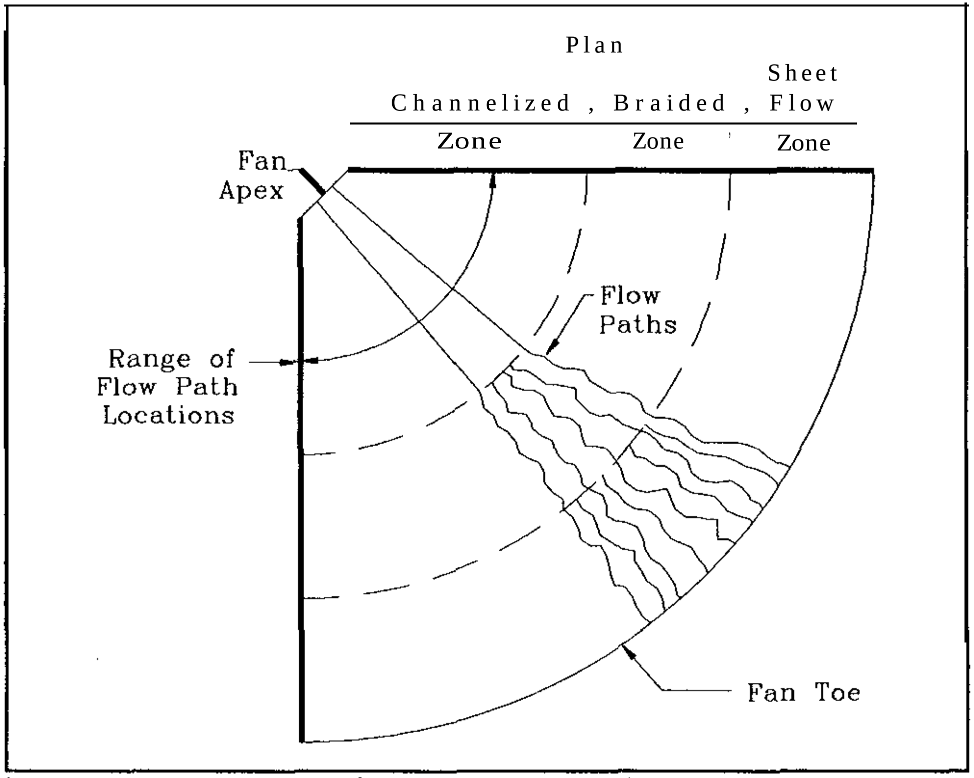


Figure 1-1 Plan View of an Idealized Alluvial Fan

The sediments deposited on alluvial fans are generally coarse grained, being composed of sand, gravel, and boulders. The depositional region is relatively close to the source region when compared with riverine situations. Alluvial fans are most common in arid or semi-arid areas where there is little vegetation; however, they also occur in polar and humid regions (Lecce, 1990 and MacArthur, et al., 1992). There is considerable discussion in the geomorphologic literature (Lecce, 1990) regarding the formation of alluvial fans.

Alluvial fans in the arid and semi-arid areas of the Southwestern United States are the focus of this study. In these areas, the important factors that contribute to the formation of alluvial fans are the hydrology, the geology and soil type, and the vegetative cover. Formation of an alluvial fan requires a source of sediment, a mechanism for moving that sediment, and an abrupt change in topographic slope and channel width that leads to shallow unbounded flow and sediment deposition. The relatively steep slopes often found on alluvial fans (3 to 20 percent) provide a high energy environment for the movement of water and sediment. The unbounded lateral dimension and rapid depositional nature of alluvial fans support frequent avulsions and flow spreading laterally on the fan surface. The hydrology of these areas is characterized by ephemeral (episodic) streams that only carry water and sediment during significant rainfall events. The predominant rainfall events that

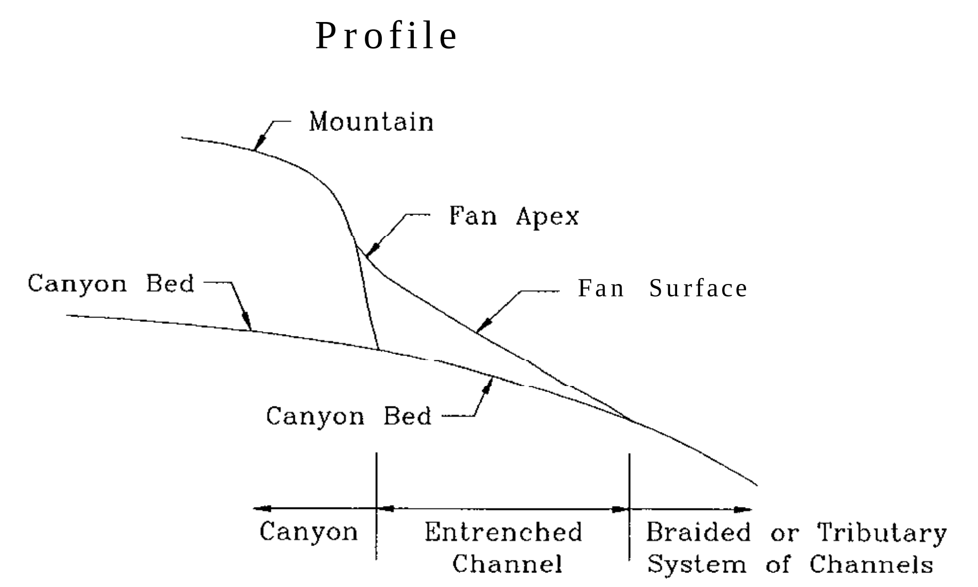


Figure 1-2 Profile View of an Idealized Alluvial Fan

form alluvial fans in these areas are localized thunderstorms. These storms may occur only once every several years over the watershed contributing to any particular fan (French, 1987).

During the periods between large storms, unconsolidated sediment accumulates within the watershed and stream channels from minor storms, weathering, dry ravel erosion, earthquakes, etc., providing an abundant supply. This combination of steep slopes, abundant supply of sediment, and intense short-duration precipitation can produce mud and debris flows capable of moving large amounts of sediment onto the evolving fan. Debris flows can occasionally form “sieve deposits” on the fan (Rachocki, 1981). Sieve deposits occur when the water filters into the underlying coarse deposits, depositing the recently moved material. The stream channel system on alluvial fans is typically braided, which is both a characteristic of alluvial fans and an important factor in their formation (Rachocki, 1981). Beaty (1990) states that an “average” alluvial fan in the White Mountains of Southern California and Nevada could be formed in about 750,000 years by three “average” debris flows every 1,000 years,

During a flood event, the flow may abandon the path lt has been taking and follow a new one. The occurrence, termed an avulsion, can result from floodwater overtopping a channel bank and creating a new channel. The overtopping may be caused by the sudden deposition of sediment and/or debris, or by the undercutting and subsequent failure of a channel bank. Because points below the avulsion may be in the path taken by the flood flow, either before or after the avulsion occurs, the probability of those points being inundated by the flood is greater than if the avulsion had not occurred. Through multiple avulsions over geologic time, the fan aggrades uniformly so that it tends to exhibit a concentric, semi-circular contour.

Changes in flow and/or sediment supply to the fan can greatly affect the morphology of the apex and fan surface. If sediment supply from the upstream watershed to a fan is reduced due to changes in rainfall patterns or increased vegetation, incision of a channel will begin at the apex. When sediment transport capacity ***or competence*** of the flow exceeds the sediment load available at the apex, the flow will scour the fan surface and create an incised channel. As long as stream competence exceeds sediment supply, channel deepening and widening will continue. A return to higher sediment productivity in the watershed, that mayresult from forest fires or increased rainfall, will often cause the channel to backfill. In that case, erratic flood channels and sediment deposition will again dominate. If the reduction in sediment supply is permanent, an incised channel or entrenchment of the fan surface results. Normal depositional patterns on the fan are altered by the entrenchment in such a way that little deposition occurs near the apex, fan building commences downstream where the entrenchment ends, and flows radiate laterally over the fan surface. A new, secondary fan is established with its apex at the end of the entrenchment.

Significant watershed and fan characteristics that influence flood behavior include:

- watershed slope
- watershed soil type, vegetation and land use
- forest fire frequency
- rainfall intensity and duration
- longitudinal and lateral fan slope and shape
- fan sediment type and vegetation
- existence of an entrenched channel(s)
- presence of rock outcrops or sills in the channel
- location, density, geometry of development and/or structures (roads, fences, railroads, buildings) on the fan

The influences and potential affects of these fan characteristics on the hydraulics and the sediment transport capability of flood waters must be considered when placing and designing structures on alluvial fans,

1.2 Floods on Alluvial Fans

Floods on alluvial fans, in the geographic regions discussed here, are generated by both localized convective storms and large scale cyclonic storms. Convective storms are the more common cause in Nevada, Arizona, Utah, and New Mexico; large cyclonic storms, generally, produce the flooding on alluvial fans in Southern California. Flooding on alluvial fans is infrequent, rapid, debris laden, and of high velocity. These floods can fill flood-control channels with debris, or erode elevated structures such as roadways, railroads and pipelines that cross the fan. Alluvial fans usually have multiple, braided channels that are subject to avulsion factors such as human activity modifying the landscape between floods, changes in vegetative growth, debris accumulation, eolian sediment deposits, etc. Brush fires in the contributing watershed can have a major impact on the availability and delivery of water and debris. Freshly burned watersheds can produce 10 to 100 times more sediment per unit runoff than unburned catchments (MacArthur, 1983). Precipitation and streamflow data for alluvial fan floods are sparse because alluvial fan flooding is typically caused by infrequent and intense storms. Long, dry periods between floods contribute to a rapidly declining public awareness of the flood hazard. In addition, most fans had not been developed; therefore, there are little or no long-term, historic flood records available for specific fans.

FEMA (1989) has identified the following flood hazards that may be encountered on alluvial fans:

- High velocity flow (15 to 30 ft/s) that can produce significant hydrodynamic forces on structures
- Erosion/scour to depths of several feet
- Deposition of sediment and debris to depths of 15-20 feet during a single event
- Debris flows and their associated impact forces and large sediment loads
- Mudflows and their associated deposits and need for removal
- Inundation, with its associated hydrostatic (buoyant) forces on structures (these forces are often affected by high sediment concentrations leading to modification of the effective fluid density and viscosity)
- Flashflooding, which means that there is little (ii any) warning time for evacuation and emergency actions to protect property
- Little or no long-term data; (event frequency and duration criteria must often be estimated from regional relationships based on limited data)

1.3 Purpose, Scope, and Study Limitations

The FIA requires an assessment of the effectiveness of various structural approaches to flood control in alluvial fan Special Flood Hazard Areas (SFHAs). The purpose of this study is to document how installed flood-control measures have performed during major floods and to present current methodologies for assessing the performance and adequacy of the measures. Non-structural measures were not within the scope of this study, nor were methodologies for mapping alluvial fan hazards. This report is aimed at providing guidance to floodplain managers for use in assessing the adequacy of structural flood-control measures on alluvial fans (primarily improved channels, detention storage, diversions, and bypasses) for protection against the Base Flood. This report is not intended to be a design manual. Considerable study and testing, which are beyond the scope of this report, are required for proper design.

It must be recognized that the state-of-the-science and our understanding of mobile boundary hydrodynamic processes on alluvial fans are limited. These complex processes often evade theoretical attempts to characterize flow depth, location, orientation, velocity, sediment-carrying capacity, and event predictability with a high degree of accuracy. Present state-of-the-science methods depend on empiricism, experience, field observation, and the application of traditional clear-water assessment methods that have been modified to account for flow bulking and the unpredictable; and often episodic, nature of alluvial fan processes.

This report documents flood experiences with flood-control structures on alluvial streams. While the emphasis is on alluvial fans, many of the potential flood hazards and performance problems for fans are the same as those on alluvial stream projects throughout the Southwest. It is intended that documented flood problems will lead to an awareness of potential causes for failure, and the required analyses to quantify important design parameters. The information and examples presented herein, are taken from field reviews of projects, interviews, and from various reports, papers, and publications listed in Chapter 4 of this report. The three basic approaches used in this study were to:

1) Prepare an inventory of project experience;
2) Conduct a library search for related studies and reports; and
3) Summarize engineering assessment procedures for determining the adequacy of alluvial fan flood-control structures.

1.4 Definitions

There are many terms used in this report and in the literature that are descriptive of alluvial fans and streams; sediment, debris, and mudflows; structural flood-control measures and their features; and the nature of the flood risk. Some of the terms are in general public usage, while others are technical and may not carry the same meaning among professionals. A glossary was developed for this project and it is included as Appendix A. The definitions are intended to be brief and informative; therefore, in some cases they may not be technically complete. The intent is to simply explain technical terms so the reader can better understand this report.

1.5 Acknowledgements

The Hydrologic Engineering Center (HEC) was authorized to perform the study for FEMA under Interagency Agreement ENW-90-E-3265, Project No. 136669, undated. Subsequent project orders extended the contract duration to June 30, 1993.

This study was performed under the management of Vern Bonner, Chief, Training Division, HEC. Mr. Darryl W. Davis was the Director of the Hydrologic Engineering Center during this investigation. The HEC study team included: D. Michael Gee, Project Engineer; Richard Hayes, field experience/case studies; Gary Brunner, engineering analysis; and Eric Butler, library research. Dr. Robert MacArthur, Resource Consultants and Engineers, Inc., prepared two chapters, expanded several others, and prepared a draft manuscript. All parties contributed to the study report. The final document editing was provided by FEMA.

Dr. Frank Tsai was the FEMA project officer during the study. His support and guidance, along with the reviews from FEMA staff, were most helpful. Also, the support and information from the many Federal, State, and Regional offices is sincerely appreciated.

2. CASE STUDY REPORTS

The following case studies present reviews of flooding problems and, in some cases, failures of flood-control project features. Some of the examples are not specific to alluvial fans; however, they all represent the type of flooding problems and project failures that must be addressed in successful projects on alluvial fans. Some earlier flood experience has been incorporated into U.S. Army Corps of Engineers (USACE) and other agency standards and guidance documents. Most of the following information comes from office reports that are not readily available. Because there is limited access to these documents, major portions of the documents that meet the objective of this report have been paraphrased or copied in this chapter. Quotation marks and traditional reference marks have not been used because the information is taken from a single report. Full credit for the investigations and presented information belongs to the writers of the referenced reports.

2.1 Los Angeles County 1938 Flood

Location: Los Angeles County, California

Reference: "Report on Engineering Aspects - Flood of March 1938" (USACE, 1938).

While this is an old report, it documents flood-related problems concerning: channel bank protection; super-elevations at bends; transportation and deposition of debris in channels; action of debris basins; and dam operation and size of spillways. While the experience gained from studies of this type have been incorporated into the USACE Engineer Manuals, the summary of types and causes of failure are worth presenting here because they represent items that must be considered in the design and construction of flood-control measures. Excerpts on the subjects of bank protection, debris basins, and dam operation are presented here.

Flood Experience

The series of storms that occurred in the Los Angeles County Drainage Area during the period of February 27 to March 3, 1938, caused the largest flood discharges on record. At many locations, the estimated peak discharge exceeded the previous record by two or three times. While a considerable amount of data were collected, there was a lack of direct measurement of discharge during the peak period. The flood was of short duration and the rapid change in stage made it difficult to collect field data.

Bank Protection

The following, are seven main classifications of bank protection:

1) Pipe-and-wire fence
2) Riprap (dumped rock)
3) Rock paving (hand-placed)
4) Wire and rock mattress
5) Gunite slope paving
6) Reinforced concrete open channel
7) Reinforced concrete closed conduit

Pipe-and-Wire/Pile-and-Wire Fences. Fence-type protection was used extensively on the Rio Hondo and San Gabriel River and tributaries. It seemed to have played an important part in preventing bank erosion and confining the flood within channels in straight, or nearly straight, reaches. However, large sections failed on the outside of bends, or where some obstruction directed the main current against the fences, The weakness of this form of protection was its inability to deflect cross-flows sufficiently to prevent bank erosion on the back of the fence, and the ease with which pipe or piles were broken off or pulled out when encountering the full force of the current. In general, the stream alignment was assisted and the tendency to meander retarded in some cases, but this form of protection was ineffective at critical locations,

Riprap Bank Protection. With few exceptions, riprap may be classed as derrick stone or dumped rock. Numerous failures of this type of levee occurred largely on the outside of bends or in the vicinity of an exceptional disturbance. The failures appear to have been started at the toe rock. The maximum velocity in the lower river was probably In excess of 18 ft/s. The irregularity of the pavement is believed to have set up scour along the toe, chopping the toe rock and cutting into the earth levee.

Rock Paving Bank Protection. The review of several creeks with this protection showed no damage with velocities up to 17 R/s and rather sinuous channel alignment. The upper Los Angeles River, wRh a design capacity of 40,000 ft3/s, had an estimated 60,000 ft% and flow velocity in excess of 20 ft/s. The failures there were attributed directly to poor channel alignment, disturbances created by side inflows, and ponding of local drainage behind levees. There was some indication that failure may have commenced immediately above the toe rock. Other failures seemed to have commenced about half-way up the slope. Once the stone was removed, water entered the fine sand of levee fill, and undercut the upper section. Failure progressed downstream until a breach of sufficient capacity to bypass the levee was made.

Wire and Rock Mattress Protection. Two types of protection included one in which the toe of the slope is protected by a wire and rock mattress and the upper slope by wire laid on the slope, and the second in which the wire and rock mattress covers the entire levee face. These types of revetment withstood the flood as long as flowing water did not get behind them. Where this occurred, the rock mattress was rolled up and carried away.

Gunite Slope Paving. Extensive failures of this class of protection occurred along the Los Angeles River. Failure seems to have been the result of water getting into the levee through cracks, causing settlement and breaking out a small section after this breach was made. The high velocity flows ripped off the thin gunite slabs.

Reinforced Concrete Open Channel. No difficulties were experienced with this type of improvement, with the exception of several breaks in small channels and the Verdugo channel in the vicinity of a bridge, where a section of channel had been left unlined during the reconstruction.

Reinforced Concrete Closed Conduit A great many small structures of this class were rendered useless by plugging with debris; however, only one important structural failure was noted. A double-barrelled conduit, each 18 feet wide and 6 feet tall, became severely blocked with debris, causing the flow to blow off the cover slab.

Debris Basins

The 1933 fire and the disastrous New Year's Day 1934 Flood hastened the adoption of the basin method of debris control. At the time of the March flood, there were 16 debris basins in the Los Angeles County area. During the years 1933 to 1935, most of the areas involved were burned. As it takes 5 years or more to re-establish cover after a burn, the state of the cover in the drainage area was inadequate for the 1938 storm.

The debris basins functioned very satisfactorily within the limits of their capacity. In the areas wlth steep slopes, the material eroded was much larger and coarser than in the areas with flatter slopes. It was noted, as a typical case, that material accumulated to the crest of the outlet structure and was at a fairly uniform grade line to approximately the top of the inlet structure. When this condition was reached, a large amount of material passed over the outlet spillway during the peak inflow period. In one example, the debris slope backed up over the top of the inlet structure and piled up in the approach channel. In general, the action of all the debris basins was judged satisfactory.

Outlet Channels. A common phenomenon was noted at a majority of the basins: a pulsating discharge from the basin to the outlet resulted in the formation of a standing wave, which proceeded down the channel at a high velocity.

Dam Operation

At the time, most of the reservoirs were small, local projects with small capacity and limited outlets, and so they had little effect on large flood peaks. The reservoirs serve mainly as debris basins and conservation structures. Although no appreciable damage to the structures resulted, the outlet and storage capacity of several structures were materially reduced by debris.

2.2 Southern California 1969 Floods

Location: Southern California

Reference: "Report of Engineering Aspects - Floods of January and February 1969 in Southern California" (USACE, 1974).

The floods that struck Southern California during January 18 to January 26, and February 20 to February 26, 1969, were the most damaging floods of record in six counties. Although past floods may have equaled, these floods were more damaging because of the intensive development that had taken place in recent years adjacent to unimproved or partly improved stream channels. More than 100 persons lost their lives in the floods, and property damage totaled more than $213 million. These were also the first great floods to occur since construction of the complex systems of flood-control projects by the USACE. This report presents information on each project sustaining flood damage or encountering problems not foreseen in design, on flood magnitudes, and on flood-damage data to permit future re-evaluation of practices and criteria. The report presents project-oriented information on existing improvements sustaining flood damage, plus the performance of flood-control improvements, special problems, and evaluation of existing design criteria. The following paragraphs summarize the performance review, special problems, and presents the evaluation of design criieria.

Performance of Flood-Control Improvements

Only improvements sustaining flood damage where the probable cause of damage could be determined are discussed. The information is presented by the following four project types:

1) levees
2) channels
3) debris basins
4) reservoirs

Levees. The performance of USACE levees that sustained damage is discussed under the categories: single-levee projects and double-levee projects. All the levees were protected with stone revetments on the channel side.

Sing/e-Levee Projects. The single-levee projects sustaining flood damage were Santa Maria Valley, the Santa Clara River levee, the Lytle Creek levee, the Mill Creek levee, and the Banning levee. Projects, both with and without groins, successfully withstood most of the damaging effects of the high-velocity floodflows. The major damages sustained by the levee revetments were probably caused by meandering flows that undermined the stone toe protection at isolated points. The erosion and subsequent slumping along the Santa Maria Valley levees, where the levee revetment was ungrouted, were clearly caused by undermining of the revetment toe as a result of impinging cross-stream flows. The severe scouring that occurred along the Mill Creek levees, where the levee revetment was grouted, was also caused by impinging cross-stream flows. Other failures were due to excessive streambed degradation and insufficient depth of the revetment toe.

Investigation of effects of earlier floods revealed the ineffectiveness of the triangular-shaped toe protection on the grouted-stone revetment. This type of toe protection was used for the Lytle Creek levee and Muscoy Groin 4, as shown in Figure 2-l. Investigation of these two improvements after the 1969 floods, revealed the streamward displacement of the quarrystone toe protection that had originally been placed on the grouted-stone revetment. The scattering of the displaced quarrystone away from the revetment and the absence of the bulk of the quarrystone in the eroded

Figure 2-1 Scour Adjacent to Muscoy Groin 4 (Photo 36 - Scour depth *of about three feet below the foe of* grouted-stone facing.)

section at the toe indicated that this design is ineffective in providing additional protection against excessive scour along the toe.

Double-Levee Projects. These projects consist of the San Gabriel River approach channel (which is also discussed under the section below titled Channels), the Riverside levees, and a 5-mile reach of the Santa Maria Valley levees. The San Gabriel approach and the Riverside levees performed well. Except for local scour near the stabilizers in the San Gabriel River approach, little degradation or aggradation occurred. The excellent performance of both projects reflected the efficiency of proper earth-bottom channel design. It was noted that this type of performance of channels with alluvial streambeds is possible where the upstream sediment supply is sufficiently large to replace the material transported out of the improved reach, and where the channels do not have excessive base widths that permit meandering. The reach of double levees in the Santa Maria Valley levees project is an example of a leveed channel with an excessive base width. The levees were damaged by impinging cross-stream flows.

Channels. Channels sustaining flood damage were the Los Angeles River, the San Gabriel River, and the Devil Creek diversion. Only the cause of damage on the San Gabriel River channel could be clearly defined. In general, little damage occurred in reaches of fully-lined concrete or grouted-stone channels.

The sloping drop structures in the San Gabriel River approach channel, upstream from the Santa Fe Reservoir, withstood the floodflows satisfactorily, except for isolated structural damage. The drop structures generally performed efficiently. The sloping grouted-stone aprons of the drop structures did not fail despite the abrasive actions of the moving rocks and gravel carried by the floods. The highly abrasive action of the floodflows was clearly evidenced by the sheared conditions of the stone projections on the surface of the sloping aprons.

In the reach of the San Gabriel River channel from Santa Fe Dam to Whittier Narrows Reservoir, severe damage was sustained by revetments near the stilling basins of the drop structures. Grouting of the revetments would have prevented much of the damage that occurred.

Debris Basins. A review of the debris basins indicated wide variations in the amount of debris and sediment trapped during the floods. The greatest amount of sediment and debris was produced from areas extensively burned in the fires of 1968. Generally, the USACE debris basins' accumulation was well below their total capacity. Six basins, constructed by local interests, were filled with mud and debris during the floods. Aithough property downstream from the debris basins was severely damaged by mud and debris after the debris basins were filled, the damage would have been much greater if the basins had not trapped the bulk of the material.

Reservoirs. Prado Reservoir was the only USACE reservoir that experienced serious problems in making planned releases; this was due to damage to downstream levees. Many non-Federal reservoir projects sustained damage during the floods. The damage included plugged-up outlet works, destroyed water-supply systems, damaged irrigation-outlet systems, undermined spillways, and eroded outlet channels. In addition, the capacities of the non-Federal reservoirs were severely reduced by sediment and debris deposition,

Special Problems

Sediment Transport. Sediment transported by the 1969 floods and deposited along channels and levees and in reservoirs and debris basins, caused serious damage to flood-control improvements. The collection of sediment data by the U.S. Geological Survey (USGS) and the data collected on sediment deposition during the 1969 floods, permitted an evaluation of the formulas used in design of USACE projects. The comparisons applied Du Boys' formula, Einstein's bed-load method plus suspended load, Colby's curves, and the Toffaleti procedure. The results were plotted on a log-log display of flow in ft^3/s vs. sediment in tons/d. Generally, the methods underestimated the sediment transport, compared with the USGS field data. The Einstein method gave the closest agreement to the measured data, even though the two differ by several hundred percent. The Toffaleti method gave the closest agreement for discharges, less than 1,000 ft^3/s, but departs radically from the curve based on USGS data for greater discharges.

Gravel Pi. Gravel pits and gravel-extraction operations in streambeds caused severe damage to USACE flood-control improvements during the 1969 floods. The extensive gravel-pit operations in the streambed of the Santa Clara River and the stockpiling in the floodplain of materials from those operations accentuated the meandering qualities of the river. In reaches where complete failure of the levee occurred, the high-velocity floodflows of the meandering river were deflected toward the levee by a natural high bank on the opposite side of the stream.

As a result of virtually uncontrolled mining of sand and gravel in the streambed of Lytle and Cajon Creeks, even insignificant flows caused serious degradation of the streambed. As mentioned above, gravel pits accentuated the meandering stream. When floods occurred, the headcutting action of floodflows moving into the pits scoured the streambed in an upward direction and attacked nearby flood-control improvements. The scouring action of the floodflows eventually caused failure of the levee toes.

The experience gained during the 1969 floods emphasizes the need for the establishment of controls on gravel mining in the floodways of future projects. Such controls are usually administered by the flood-control district responsible for operation and maintenance of the project. Suggested requirements include:

- No stockpiling of any kind and no other obstructions will be permitted in the floodway.

- No excavation will be permitted within a strip extending 200 feet streamward from the levee-control line.

- No excavation will be permitted below a plane originating a distance 200 feet horizontally and 10 feet vertically from the levee-control line, and extending to the channel side on a slope of 1 ft vertically, for every 20 feet horizontally.

- No excavation in the floodway will exceed a depth of 50 feet below the elevation of the corresponding point on the levee-control line.

- All extraction operations will be conducted in accordance with plans that have received prior official approval of the USACE, and which will provide for continuous pits within the property of any one operation. “Leapfrog” operations will not be permitted, and the continuous pits will not be sinuous, with respect to either line or grade.

Temporary excavation not conforming to the above rules will not be permitted.

. Depth of new pits on the shore side of the levee shall not extend below a plane passing through the present ground surface at a point 60 feet south of the levee-control line, and dropping toward the south at a l-on-l 0 slope.

. Depth of new excavation in existing pits shall not extend below a plane starting at the point described above, and dropping toward the south at a l-on-5 slope.

Evaluation of Existing Design Criteria

Levees. The severe damage sustained by levees during the 1969 floods indicates that continued investigation and analyses should be made of design criteria pertaining to the depth of revetment toes. The insufficient depth of revetment toes was probably the major cause of levee damage to the Santa Maria Valley, the Santa Clara River and Mill Creek levees, and the Lytle and Cajon Creeks channel improvements. For levees where flow impingement is likely to occur during floods, the depth of the toe protection should not be less than the depth of the anticipated scour below the invert. When rigid revetments are used, the toe protection should be either an extension of the rigid lining to a depth below the estimated level of scour or a system of gabion mattress. The use of stone protection is discouraged because it was washed away during the 1969 floods.

Channels. Current freeboard criteria was judged adequate. The conclusion was based on the absence of any evidence or report of spillage over the tops of the channel walls at any time during the floods.

The riprap side-slope protection for earth-bottom channels functioned satisfactorily during the floods. Detailed analyses, made after the floods, of the thickness and grade of the riprap in place on three streams, indicated that the recommendation in Civil Works Engineering Bulletin 52-15 are reliable and conservative.

Sediment transport through earth-bottom channels and a series of drop structures had been evaluated using Du Boys’ equation. While detailed analysis could not be made because information was not available on the quantity of sediment passing through Whittier Narrows Dam, an evaluation of a reach with seven reinforced concrete-drop structures indicated that the method is adequate. Additionally, the method serves as a guide in determining the maximum degradation and aggradation of an invert during a design flood and, in turn, the design depth of toe protection and the heights of levees.

The use of sloping bridge-pier extensions to reduce debris deposition was judged successful. No debris accumulated on the extensions provided on bridge piers in the Rio Hondo channel, which was the only channel with the extensions and the maximum discharge near the design discharge.

Debris and sediment deposition in concrete channels was not a problem where debris basins were present. Extremely large amounts of sediment were deposited in the rectangular sections of the Devil Creek diversion channel, which did not have a debris basin. The design of concrete channels in drainage areas capable of producing large amounts of debris and sediment during floods, should include provisions for trapping such material at the head of the channel or moving the material through the channel. An alternative method for conveying all debris and sediment throughout the length of a fully lined channel is to design the channel with a steep grade and a trapezoidal configuration to maintain sufficient depth and velocity to move the material.

Deb&Basin Capacities

The criteria currently used by the USACE, Los Angeles District to determine design capacities was judged satisfactory. The criteria used are described in the paper titled "A New Method of Estimating Debris-Storage Requirements for Debris Basins" (Tatum, 1963).

Reservoir Sedimentation

Reservoir surveys made by the USACE, Los Angeles District after major storms, provide data on the rate of debris and sediment inflow to existing reservoirs. The data collected provide valuable information for use in estimating sediment allowances for future projects. Guides for estimating sediment yield are contained in the paper titled "Factors Affecting Sediment Yield in the Pacific Southwest Area" (PSIAC, 1968).

2.3 Riverside County 1980 Flood

Location: San Jacinto River Levee and Bautista Creek Channel, Riverside. California.

Reference: Engineering Report, “Report on Levee Failures and Distress San Jacinto River Levee and Bautista Creek Channel, Riverside County, Santa Ana River Basin, California” (Engineer Team, 1980).

This report reviews project design, construction, subsequent modifications, flood history, project performance, investigation of potential causes of failure, probable cause of failure, and remedial measures taken. Sections on project design, construction, and performance are presented here, along with the causes of levee failures and conclusions. The section on potential causes, lists the items that should be considered in levee design and construction.

Background

During February 1960, flooding caused the San Jacinto River flood-control project to undergo distress. Levees on both San Jacinto River and Bautista Creek reaches were breached, as evidenced in the aerial mosaics. Because of this occurrence and at the request of the USACE, Los Angeles District, an Engineer Team was formed and tasked with determining the probable cause or causes of failure; recommending remedial construction measures; and making recommendations as to the application of this experience to existing and future projects. The report sections describing the project design, construction, project performance, causes of levee failure, and conclusions are presented here.

Project Description

The San Jacinto River levee and the Bautista Creek channel improvements are located in Riverside County. They consist of a 3.7-mile levee on the left side of San Jacinto River, a 1.3-mile levee on the left side of Bautista Creek, and a 3.25mile concrete-lined channel on Bautista Creek upstream from State Highway 74. The federal cost of constructing this project was $3 million. The project units are designed to protect San Jacinto, Hemet, Valle Vista, and nearby agricultural areas. Since their completion in November 1961, the units have been maintained by the Riverside County Flood Control and Water Conservation District (RCFC&WCD). During the 1969 floods, they prevented damages estimated at $1.3 million.

Project Design

The bases for design are included in the following reports prepared by the USACE, Los Angeles District:

- Design Memorandum No. 1, "Hydrology for San Jacinto River and Bautista Creek Improvements," July 1959;
- Design Memorandum No. 2, "General Design for Bautista Creek Channel," September 1959; and
- Design Memorandum No. 3, "General Design for San Jacinto River Levee," September 1960.

Hydrology. The standard project flood (SPF) was used as the basis for design. The flood was developed in accordance with guidelines presented in Civil Works Engineer Bulletin No. 52-8, dated March 26, 1952. The standard project storm, general winter type, was employed for the drainage area tributary to the San Jacinto River levees. This storm is based on the assumed occurrence of a storm equivalent to that of January 1943, transposed and centered over the area tributary to the pertinent area. The standard project storm, local type, was used for the drainage tributary to the Bautista Creek improvement. This storm is based on the assumed occurrence of a storm equivalent in magnitude to that of March 1943, transposed and centered over the area.

The resulting SPF peak discharges are 88,000 ft^3/s for the San Jacinto River improvement and 16,500 ft^3/s for the Bautista Creek improvement. The SPF peak discharge for San Jacinto River is about 50 percent larger than the peak discharge that occurred during the flood record of February 1927.

Hydraulics. The hydraulic design was based on the theoretical analyses and design practices previously approved for similar projects, The design conformed to the criteria, which applied at the time, published in chapters of the Civil Works Construction Engineer Manual and Civil Works Engineer Bulletin No. 52-l 5.

Design Memorandum No. 3 describes the proposed plan of improvement and functional characteristics. The pre-project San Jacinto River channel flood-control levees, were constructed by local interests and were protected on the channel side with pipe-and-wire fencing. The estimated channel capacity was about 8,000 to 20,000 ft^3/s, and the slope ranged from 0.00526 to 0.00935 feet/ft

The levee along Bautista Creek was built in a reach where local interests had constructed sand levees and pilot channels. The channel sides were protected with pipe-and-wire fencing. The capacity of the pre-project Bautista Creek channel was about 75 percent of the design flood flow, and the slope of the channel ranged from 0.0100 to 0.0182 feet/ft.

Water-surface computations were made by the reach method, using Manning's R. The computations were made on the basis of a design discharge of 86,000 ft^3/s in San Jacinto River downstream from the confluence with Bautista Creek, and a design discharge of 16,500 ft^3/s in Bautista Creek. The maximum water-surface computations to determine levee heights were based on an *n* value of 0.040. Depths ranging from 5.7 to 13 feet were computed for San Jacinto River; and from 3 to 6.6 feet, for Bautista Creek. The maximum mean velocities used to determine the slope and toe protection were based on an *n* value of 0.025. Velocities ranging from 7.3 to 15.5 ft/s were computed for San Jacinto River; and from 9.4 to 16.9 ft/s, for Bautista Creek. The water surface for San Jacinto River was computed based on the assumption that the existing left levee would be removed and the existing right levee would remain in place; however, for Bautista Creek, the water surface was computed based on the assumption that flow would be contained in an area bounded on the left by the levee, and on the right by high ground.

A minimum freeboard of 3 feet above the computed water surface is provided along both streams. Superelevation was computed by the formula $V^2 T/gRc$, where: V is the velocity of flow, T is the top width of flow, g is the gravitational constant, and Rc is the radius of the curve. The superelevation of the water surface ranged from 0.2 to 1 ft.

Confluence computations were based on a flow of 74,000 ft^3/s in San Jacinto River upstream from the confluence, and a flow of 12,000 ft^3/s in Bautista Creek. This combination produces the maximum water-surface elevation in the confluence for the design discharge in San Jacinto River downstream from the confluence.

Under the project document plan, the thickness of the revetment would range from 2 feet at the top of the levee, to 5 feet at the toe of the levee and the revetment would be underlain by a 1-ft layer of filter material. The adopted stone revetment, a 1.5-ft layer of riprap over a 6-inch fitter blanket, is shown in Figure 2-2. (reprint of report Figure 2). The revised thicknesses were based on the then "present-day criteria."

Depth of toe was an item of considerable concern during the design of the project, as indicated by a review of District records. The adopted depths of toe

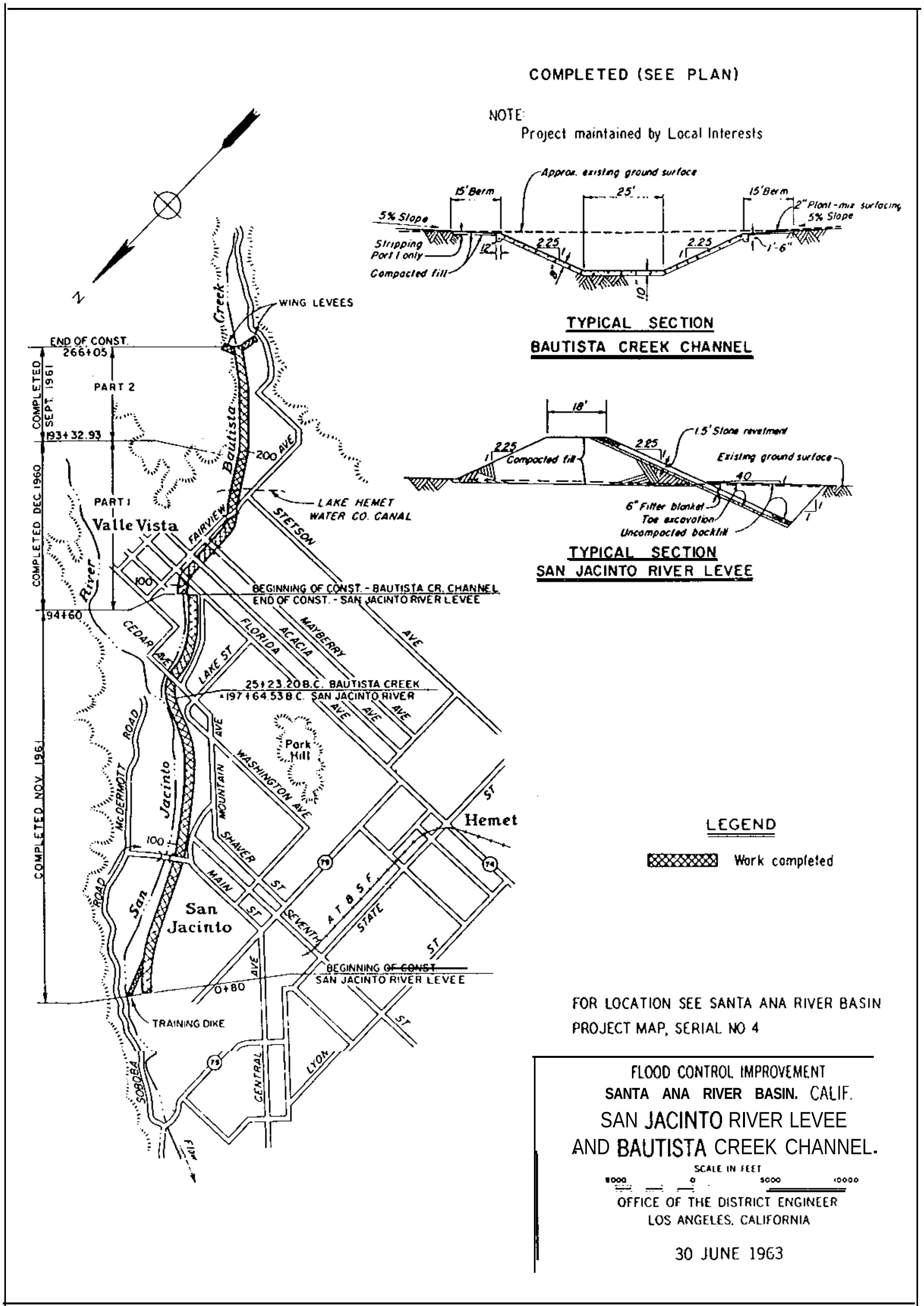


Figure 2-2 Typical Channel and Levee Sections

for the Bautista Creek channel and the San Jacinto River levee are 5 and 10 feet, respectively, below the low point of the streambed.

Embankment and Foundation. The foundation materials are principally silty sands, sand-silty sands, and silts, with occasional gravel and cobbles. The upper 6 to 12 feet are loose to medium-dense. Groundwater was not found in any of the test holes that were drilled to a maximum depth of 35 feet along the project reach. The 1957 well records indicated that groundwater was about 10 feet below the streambed at the downstream end of the project levees, and 60 feet below the streambed at the upstream end. A typical embankment section is shown on the project map in Figure 2-2. Analysis of the slopes was based on drained strengths, Using the infinite slope method, the factor of safety for the end of the construction condition is 1.4. Steady seepage and drawdown conditions were not analyzed because the influence of seepage into the levee fills and foundations was considered to be negligible due to short-duration flows.

Project Construction

The dates for the completion of construction of the various reaches of the San Jacinto River levee and the Bautista Creek channel are presented in Figure 2-2. The Bautista Creek Channel Project is a concrete-lined trapezoidal channel with an energy dissipator at the downstream end. The portion of the Bautista Creek channel downstream of the concrete channel is a left-bank levee with a typical section similar to that shown for the San Jacinto River levee. It was constructed as part of the San Jacinto River Levee Project.

The Bautista Creek levee has a maximum height of 10 feet and the stone revetment toe is 8 to 9 feet below the line of backfill at the face of the levee. This distance corresponds to 5 feet below the low point of the streambed. The levee section was built with streambed materials and borrow from an existing levee. These materials were placed in 12-inch layers, compacted with four passes of a 50-ton rubber-tired roller.

The borrow for the San Jacinto River levee was obtained by removing about four miles of existing levee between Cedar Avenue and the downstream end of the project. The remainder of the levee fill came from streambed materials similar to the foundation materials previously described. Construction of the levee was the same as for Bautista Creek. The construction-control data show that the densities varied from 98 to 106 percent of the standard American Association of State Highway and Transportation Officials (AASHTO) maximum density.

Riprap. Stone for the project was obtained from the Bernasconi Pass Quarry and the Juaro Quarry. The stone tested had a bulk-specific gravity of 2.71 to 2.76 and an apparent specific gravity of 2.73 to 2.78. The construction control riprap gradations, taken at the plant located at the quarry, are not representative of the stone gradation on the levee, in part, because of segregation that results from handling and placement. It has been verified that a jaw crusher was used to control the maximum size of stone, but it is not known whether a screen was used to remove the finer stone throughout the production. The stone was transported to the levee crown in end dump trucks and then was dumped into a "skip" that was crane-operated. The skip was used to place the stone and drag the slope.

Project Performance

Before the February 1980 Flood Since the completion of the project, high flows have occurred in 1965, 1966, 1969, and 1978. In November 1965, a multiple (10) corrugated metal pipe and dip crossing with concrete overflow at Main Street were washed out. During the February 1969 storms, the Bautista Creek channel was degraded. Afterwards, the seven stabilizers previously mentioned were constructed. Five of the stabilizers were damaged during the 1978 storm and were repaired in 1978 by a RCFC&WCD contract. The RCFC&WCD has kept a record of degradation and aggradation in Bautista Creek and has furnished a drawing showing streambed profiles at various times. Severe degradation of the streambed (about 10 feet), was noted before the floods of 1969. The RCFC&WCD has noted that the energy dissipator derrick stone has been repaired since the original construction.

A review of the aerial mosaics presented in Design Memorandum No. 3 and post-construction aerial photographs, indicates that topographic features have directed flows into the San Jacinto River levee in the general vicinity of the February 1980 breach. A long-time resident of the area commented after the break that it was the third time that the water broke through the same reach. The first two breaks occurred in locally constructed levees before the construction of the USACE levee.

February 1980 Flood Rainfall occurred over the watershed for nine consecutive days, from February 13 to February 21, 1980. Mean seasonal precipitation ranged from about 14 inches at San Jacinto to about 45 inches at San Jacinto Peak, averaging about 20 inches over the total area.

The peak discharge of February 21, 1980, in San Jacinto River above Bautista Creek is 17,300 ft^3/s, about a 30-year flood. The estimate of a 6,000 ft^3/s discharge on Bautista Creek represents about a 70-year flood. Based on these two discharges, the peak discharge, which occurred at the San Jacinto levee, is estimated to be about 25,000 ft3/s, representing a flood recurrence interval of about 25 years,

During the February 1980 **Flood.** On February 21, 1960, the Bautista Creek and San Jacinto River levees were breached. The breach in the Bautista Creek levee extended from approximately sta. 61+00 to sta. 59+00. The breach in the San Jacinto River levee extended from approximately sta. 169+00 to sta. 154+00, before flood fighting operations controlled the erosion. At several other locations erosion occurred, generally below the "line of backfill."

The RCFC&WCD has provided eyewitness accounts of the San Jacinto River levee breach. Excerpts from these eyewitness reports state:

> "Water Master for the Hem&San Jacinto Area of Eastern Municipal Water District...was on Mountain Avenue at approximately 7:00 a.m. and observed a 204 wide breach in the levee at that time and reported to their headquarters."

Other eyewitness accounts following the initial breach give an account of the progress of the failure. An eyewitness account of observations at 7:45 a.m. reports:

> "Levee disintegrating on the upstream side of breach rapidly. Flood through breach surging in river in waves 5 to 10 ft high. .+ 8:30 a.m. Breach $\pm$ 700 ft wide at this time At the location of breach, the main direction of the river flow was $\pm$ 25° to the downstream tangent, as observed."

Eyewitness estimates of the flow through the breach ranged from 75 to 95 percent of the river flow.

Causes of Levee Failures

The Engineer Team considered the following six as possible causes of levee failures, and their application to the subject project:

1) Overtopping
2) Internal erosion (piping)
3) Slides within the levee embankment and/or foundation
4) Surface erosion
5) Undermining of bank protection (scour)
6) Channel configuration

Overtopping. Based on high-water marks, probable maximum height of ride-up, speculative height of waves, and their influence on probable maximum water levels, overtopping did not occur and, therefore, was not a cause of failure.

Internal Erosion (piping). There was no evidence to suggest the occurrence of piping, even though the characteristics of embankment and foundation materials make them susceptible to internal erosion. Observed rodent activity is not considered to be significant. The small differential head does not produce sufficient hydraulic gradient in levee sections to develop piping. Thus, internal erosion (piping) was not a cause of levee failure.

Slides Within the Levee Embankment and/or Foundation. Levee design exploration and stability analyses indicated levee embankment and foundations to be stable. Minor erosion at the landside toe of the levee upstream of the San Jacinto River levee breach is not considered to be significant. The levee has a conservative cross section, embankment and foundation materials have high strengths; and no evidence of through or underseepage exists. Consequently, it is concluded that, because slides did not occur within the levee embankment or foundation, they were not a cause of levee failure.

Surface Erosion. Levee failures can be caused by surface erosion of riprap bank protection because of action from excessive stream currents and/or waves. Surface erosion will then occur when the tractive force produced by flow velocity exceeds the critical tractive force for stone stability. Waves, caused by unstable streambed formations near the bank or flow impingement on the bank (both conditions occurred in San Jacinto River), produce uplift pressures on bank protection stone that, in combination with stream velocity, can cause surface erosion when tractive forces are smaller than critical. Consequently, when riprap bank protection is designed for flow velocity alone and significant waves occur along the bank, surface erosion may occur for flows substantially smaller than the design discharge.

To determine whether surface erosion was a cause of levee failure on San Jacinto River, observations of in-place stone were made and four in-place gradations were taken, as previously noted. Based on visual observations, there was no evidence that significant surface erosion had occurred, although some localized areas of stone were judged to be fine and others to be coarse. The gradations indicate one sample to be undersized with respect to project specifications; however, the original design appears to be following the criteria used at the time of construction, namely, gradation control at the quarry only. Therefore, the areas of undersized stone may be due, in part, to segregation that occurred during handling and placement, Observations and sampling of in-place riprap indicate that, because removal of the bedding layer from beneath the riprap had not occurred, it is an unlikely cause of surface erosion leading to levee failure. Although two of the in-place gradations show the bedding layer to be finer than specified, this condition could have resulted from silting by flow sediments and/or contamination from sampling procedures, since demarcation between bedding and embankment materials probably was not distinct. In any event, it is believed that the finer gradation of the bedding material was not a significant factor in levee failure.

In one trench, where scour depths were near the bottom of the riprap protection, some riprap was located at the scour level riverward of the riprap toe. This stone was either removed from the riprap layer by surface erosion or undermined in the breach area and transported downstream along the scoured streambed. The latter case appears to be the most likely reason for finding displaced riprap.

Based on present criteria (ETL 1110-2-120), a significantly thicker layer or heavier stone would be required to withstand flood velocities. Although no evidence was found that surface erosion was a significant factor in levee failure, the undersized riprap protection compared with present criteria would probably be subject to failure by surface erosion during larger floods up to design-flood magnitude.

Undermining of Bank Protection (scour). Inspection of Bautista Creek upstream of the levee suggests that construction of the concrete channel caused sediments, naturally carried by the creek, to be deposited upstream of the channel inlet. The resultant delivery of relatively sediment-free water to the leveed reach along with the steep slope of this reach (greater than 1 percent) caused general streambed degradation downstream of the concrete channel. The subsequent nearly complete filling from the valley immediately upstream from the concrete channel inlet with deposited sediment and the construction of channel stabilizers by the RCFC&WCD have reduced, and in the upstream part of the reach have reversed, the general tendency of the streambed to degrade.

The RCFC&WCD had documented the general degradation of Bautista Creek through most of the leveed reach. The level of backfill (still evident along much of the levee) provides a reference plane for evaluating the approximate depth of scour and/or channel degradation. Comparing the design depth of riprap toe with the depth of the existing streambed below the backfill reference level indicates that the streambed below the backfill reference level is at about the same level as the riprap toe along much of the levee. Visual inspection of exposed riprap at the streambed tends to confirm that the riprap toe is exposed and damaged in some locations.

Examination of the scour gage data indicates that scour along the levee was approximately to the rock toe, except in the breach area where scour was several feet below the rock toe. These data indicate that scour was 4 to 5 feet below the levee toe at sta. 54+58, upstream and downstream of the breach. Based on observed conditions and scour gage information, it is quite evident that undermining of the bank protection caused the levee failure at Bautista Creek.

During the initial field inspection and preparation of the preliminary report, there was no readily apparent or obtainable information upon which to determine the cause or causes of levee failure at the main breach in the San Jacinto River levee, other than the evidence that most of the river flow impinged on and then flowed along the levee in the areas where the breach subsequently occurred. This evidence suggested the possibility that deep scour occurred along the levee in the area of flow impingement, which undermined the levee toe and caused failure of the levee.

Subsequent excavation and inspection of trenches provided positive evidence of scour depths. A trench, located a short distance downstream of the breach, revealed that the depth of scour was approximately to the bottom of the rock toe. A second trench, located within the breach area and approximately 50 feet riverward of the original levee rock toe, indicated the depth of scour to be approximately at the same level as the bottom of the original rock toe. Considering the magnitude of the 1980 flood compared with other floods that occurred subsequent to completion of the project, it is reasonable to conclude that the maximum depth of postconstruction scour occurred during the 1980 flood. This evidence suggests that the maximum depth of scour at the rock toe resulting from impingement of flow on the levee face during the February 1980 flood was at or below the bottom of the rock toe at the time of the levee breach. Consequently, undermining of the bank protection by scour appears to be the principal cause of the San Jacinto levee failure.

Below the Main Street crossing, the similar evidence of impingement and flow along the levee face suggests that the levee distress there was caused in the same manner as it was for the main breach.

Channel Configuration. The channel configuration appears to have been a significant factor contributing to levee failure, inasmuch as the resulting flow impingement on the levee causes deeper scour at the toe of rock protection. Flow impingement was particularly significant on the left levee of San Jacinto River between sta. 164+00 and sta. 169+00. Upstream from this location, the abrupt junction of Bautista Creek with San Jacinto River and the protection wall upstream of the water-well area resulted in impingement of flows at the upstream end of the right Indian levee with some distress at that point. The upstream end of the Indian levee deflected flows across San Jacinto River to impinge at an angle of approximately 25 degrees on the left levee at the above-referenced failure location. This angle of impingement contributed to 75 to 95 percent of the flow that passed through the levee break. Similar, but less noticeable, irregularities in channel bank alignment farther downstream on San Jacinto River and on Bautista Creek resuited in flow impingement at several locations where levee distress occurred. Therefore, it is evident that channel configuration contributed to levee failures by producing flow impingement on levees that, in turn, produced deeper scour and undermining of the levees.

Conclusions

Based on the information available, the Engineer Team has reached the following conclusions regarding the causes of levee failures:

- Failure of the levees, in whole or in part, was caused by undermining of the levee toe, influenced by flow impingement due to adverse channel configuration,

- There is no evidence that inadequate or improper maintenance contributed to the failure.

- Considering the customary practices and procedures at the time of construction, the project was constructed substantially according to plans and specifications, These procedures did result, however, in riprap levee slope protection that was, at some locations, somewhat smaller than called for in the design.

- The riprap protection was designed based on the criteria in effect at the time. Present criteria would call for a thicker layer of heavier and more uniformly graded riprap.

- The depth of scour was properly recognized in the original design of the levee slope protection as an important design consideration; however, the effect of flow impingement on producing greater depths of scour in certain locations was not recognized, as riprap toe protection was not taken to greater depths in those locations.

  Two factors contributed to the failure of the Bautista Creek levee: (1) inability to provide sufl icient depth of riprap protection to accommodate the increased streambed degradation caused by reduction in sediment load due to the presence of the upstream concrete channel and inlet; and (2) the excessively steep streambed slope in the levee reach.

2.4 Las Vegas 1975 Flash Flood

Reference: "A Brief Hydrologic Appraisal of the July 3-4, 1975, Flash Flood in Las Vegas Valley, Nevada, (Katzer, T.L.; Glancy, P.A.; and Harmsen, L., 1976).

This report focuses on the storm, flood characteristics, sediment transport, and resulting damage of the 1975 Las Vegas flash flood. There were no major flood-control projects in place at this time. (Subsequently *C/ark County completed flood-control facilities in 1987 (Reel & Bond, 1988) and the USACE, Los Angeles District, is proceeding with a Las* Vegas *Wash & Tributaries project that incorporates the earlier County project.)* Portions of the report sections on the flood characteristics and sediment transport are presented here because they provide insight on the consequence of thunderstorm events on alluvial fans and the resulting sediment and floodflow damage. Automobiles, roads, and utilities were severely damaged from this event.

Background

Heavy thunderstorm precipitation on the afternoon of July 3, 1975, between metropolitan Las Vegas and the mountains to the south, west, and north, caused flashflooding in the city area. Total storm precipitation equaled or exceeded 3 inches in some areas. Peak flows of Tropicana Wash, Flamingo Wash, Las Vegas Creek, and Las Vegas Wash were the highest ever determined.

Flood Characteristics

Source Area. The alluvial fan system southwest, west, and north of metropolitan Las Vegas received the greatest amount of precipitation and, therefore, contributed most of the runoff. The complex drainage patterns superimposed on the alluvial surfaces indicate that this type of storm runoff has occurred many times in the past. Much of the alluvial surface area was inundated by shallow sheet flow. The vegetation on the alluvium is sparse to moderate, consisting of desert shrubs and grasses, and is not very effective in retarding flows and promoting infiltration, Thus, as sheet flow moves downslope it tends to become channelized. As flow capacities of major channels are sometimes exceeded, aerially widespread flooding occurs during particularly large runoff events.

Peak Flows. Hydrologically, the July 3, 1975, flood may have been the greatest flood in Las Vegas history. Peak flows in most major drainages exceeded those previously measured or estimated. However, quantitative records are completely lacking on some earlier floods; therefore, the 1975 floodflows may have been exceeded in the past, at least at some sites along some tributaries,

Peak flows at gages were determined and flood hydrographs were developed for four recording stations. Flood peaks generally diminish in a downstream direction in the absence of additional tributary inflow. This reduction in peak flow is at least, in part, the resuit of some of the flow being temporarily stored or retarded on the floodplain because of localized flooding. Some of this localized flooding is frequently caused by flood debris clogging bridge and culvert openings, thereby, reducing channel capacities and forcing some flow out of the main channels,

The peak flow rates per unit area of contributing drainage area, are not particularly great when compared to other flash floods in Nevada; in other floods, peaks as high as 7,000 to 8,000 $(ft^3/s)/mi^2$ from small drainages have been determined by USGS investigations *(data in files of the USGS, Carson City, Nevada).*

Las Vegas Creek probably peaked sometime about 4 p.m., P.D.T, and was the first known tributary to peak on July 3, followed by Flamingo and Tropicana Washes. The first flows reached the Flamingo Wash gaging station at Maryland Parkway at 5:00 p.m., about 5 hours after the storm started, with the peak occurring at 6:30 p.m. and lasting just a few minutes. By 7:30 p.m., the flood crest had dropped about 3 feet and was decreasing rapidly. This was the only gaging station that operated throughout the peak-flow period; however, the gage became inoperative later during the flow recession. No known data fix the time of peak flow on Duck Creek.

Flow Velocities. Mean velocities of peak flows at the indirect-measurement sites are calculated to have ranged from about 2 ft/s on Tropicana Wash near Interstate Highway 15, to as high as 15 ft/s on Las Vegas Wash near North Las Vegas. Maximum point velocities within the cross sections at these sites are unknown, but they are inherently somewhat greater than the average velocity.

One current-meter flow measurement was made during the flood in a channel reach characterized by heavy salt cedar growth at the Las Vegas Wash near Henderson gaging station. The measured stream discharge was 3,500 ft^3/s. Velocities ranging up to 3.4 ft/s in individual vertical sections were noted, and the mean velocity for the entire cross section was 1.38 ft/s. This measurement was made about 3 hours after the peak had passed.

The approximate 4-hour time lag between the start of sheet flow on the alluvial fans (about noon) and the beginning of flooding in the metropolitan area (about 6.5 mi maximum distance), gives a general suggestion of the average integrated flow velocities from points throughout the drainage. The time of travel of the storm runoff, however, is the product of a complex mixture of many factors and is, primarily, affected by storm and land surface characteristics,

Sediment Transport

The intense rainfall and heavy runoff caused a substantial amount of erosion, sediment transport, and sediment deposition. The field-reconnaissance nature of this investigation did not allow any quantitative measurements of erosion or sediment deposition. Also, an unknown fraction of the total sediment transported by the storm runoff was deposited in Lake Mead near the mouth of Las Vegas Wash and, therefore, is not readily accessible to quantitative assessment. This report addresses only some of the more obvious qualitative aspects of sediment erosion, movement, and deposition by the flood.

Erosion. In spite of the reported intense nature of precipitation at many localities from time to time during the storm, subsequent observations did not generally disclose extensive rill erosion of the general landscape; however, many striking examples of ditch, gutter, and gully erosion were seen throughout areas subjected to intensive runoff. Major stream channels also exhibited numerous striking examples of lateral channel cutting and bank caving; however, obvious vertical downcutting along reaches of major channels was not common in and near the metropolitan area, possibly because the major channels are extensively underlain by deposits of caliche (calcite-cemented alluvium) that effectively armor the streambeds against vertical erosion. Vertical scour damage occurred locally at the downstream ends of culverts and similar drainage structures. Some concrete protective aprons or wingwalls were undercut and seriously damaged by the highly turbulent flow. A particularly dramatic example of this type of damage occurred near the mouth of Las Vegas Wash, where concrete box culverts through the high fill of Northshore Road were progressively undermined after turbulence and vertical channel downcutting of flood flow destroyed the effectiveness of the protective riprap armor lining the channel, and mantling the downstream fill slope. Damage at this site continued even long after peak flows had subsided, and the highway fill section required extensive reconstruction to prevent complete failure.

There was severe but typical examples of eroded roads at diverse locations in the Las Vegas metropolitan area. In most situations, roads that were overtopped by heavy flows failed from progressive headward channel cutting through the roadbed. In other places, road-surfacing was laterally displaced in masse by streamflow. A particularly severe example of eroded roadway occurred where Lamb Boulevard was cut by Las Vegas Wash a short distance south of the intersection of Lamb Boulevard and Owens Avenue.

Probably the most pronounced example of vertical and lateral erosion along a major stream channel occurred in the lower reaches of Las Vegas Wash. The site is near the former location of a Geological Survey streamflow gage that was lost when the stream bank eroded during the flood. Recent drastic channel erosion in lower Las Vegas Wash had occurred prior to the July 3-4 flood, but the floodflows greatly accelerated the erosion and were largely responsible for the chaotic results.

The suspended-solids content of Las Vegas Wash at Nothshore Road still showed pronounced effects of the flood 11 days after the peak flow, and had not recovered to “background” levels more than 4 months after the flood.

Lateral channel cutting by overbank floodflows also affected constructed features other than road surfaces. The overbank flow undercut masonry block walls, sidewalks, street curbing, sewer lines, and street signs.

A minor erosion problem, having the potential for serious consequences, was the exposure of a natural-gas line by erosion. The line was constructed on top of the land surface and covered only with a relatively thin blanket of alluvium. The path of the pipeline lies across numerous shallow gullies that drain surface flow down the alluvial fan, creating the potential for exhumation by moderate to heavy surface runoff. An exposed pipeline would be vulnerable to further flood damage and vandalism that could trigger more serious problems.

Sediment Deposits. Sediment deposits created many problems and may actually have caused greater overall economic damage than that damage caused by erosion. One of the most obvious sediment deposits that received early cleanup attention was in Flamingo Wash at the Caesars Palace parking lot. Although the deposit covered only a few acres at most, cleanup probably involved removal of several acre-feet of sediment,

Another obvious problem area of sediment deposition was at Winterwood Golf Course near the junction of Flamingo Wash, and Las Vegas Wash in southeast Las Vegas. The deposits covered many acres, but the depths of most of the deposits are uncertain. Total volume of the deposits was at least several acre-feet.

Sediment was also profusely deposited on numerous streets, highways, lawns, and in homes, businesses, and other buildings. Cleanup of much of this sediment probably accounted for a large part of the cost of the flood damage. Sediment deposition at the delta of Las Vegas Wash in Lake Mead was probably great. The effects of this sediment transport on lake and stream biota are unknown, but may have been significant.

Patti&-Size Distribution of the Transported Sediment, The sediment loads transported by floodwaters consisted of three basic components:

1) Man-made objects
2) Natural organic debris (mostly trees and brush)
3) Natural inorganic particles (mineral and rock material)

Man-made Objects. This component was probably the smallest volume of material transported, but involve the greatest economic impact because of the high financial losses associated with displacement and damage of automobiles and other expensive articles.

Natural Organic Debris. Organic debris probably makes up a minor fraction of the total weight and volume of all sediment transported, but was important because the debris and man-made objects together effectively blocked and clogged culverts. The clogged drainage ways ponded and diverted floodflow, which caused increased flooding and damage. The bulky character of much of the organic debris and the man-made objects, as well as their generally floatable nature, contributes to the clogging problems. Fine-grained organic debris and small man-made objects probably had only minor effects on the floodflow movement.

Natural Inorganic Particles. The nonorganic mineral and rock material made up the majority of the weight and volume of sediment transported and deposited by the flood. Almost all observed sediment deposits, both overbank and in-channel, were dominated by fine-grained sediments (sand, silt, and clay). Undoubtedly, some coarse material moved, but cursory visual inspection suggests that gravel and boulders were only a minor part of the total weight and volume of transported sediment. The main-channel flow commonly displayed the competence to move automobiles, concrete drainage pipe, and other large heavy objects over considerable distances; therefore, if gravel and boulder transport did not occur, it was probably because that size of material was unavailable for transport in most major channels. The particle-size distribution of the sediment apparently moved by the flood was, therefore, controlled more by availability than by the competence of flows required to move it.

Damage

Heavy damage occurred along Flamingo Wash in the vicinity of Caesars Palace, where automobiles were parked in the floodplain, despite several signs warning of flash floods. Several hundred cars were damaged by submersion and collisions when they were moved by the floodwaters. Many of the vehicles were piled up at the entrance to drainage structures under Las Vegas Boulevard South, commonly referred to as "The Strip." The obstructions caused increased backwater, and more cars and a larger area were inundated.

Many automobiles in various parts of the flooded city suffered similar consequences. Several autos were lost when they were driven onto flooded sections of streets and the flows swept the vehicles off the roadways.

Overbank flooding of major creeks caused great damage to buildings that were invaded by the turbid water. Many utility poles tilted to non-vertical positions during the flood. Streets were inundated and later, left coated with sediment, as were lawns and other improved real estate features. Curbs and drainage structures were undermined and pipelines were exhumed and commonly damaged. Sewage plants were inundated and deactivated by mud and water.

2.5 Saddleback Diversion Harquahala Valley Watershed

Location: Maricopa County, Arizona

Reference: Engineering Report, "Saddleback Diversion Harquahala Valley Watershed" (SCS, 1987).

Project Description

Saddleback Floodwater Diversion Channel is a 4.73-mile-long channel that takes the principal spillway outflow from Saddleback Flood Retention Structure (FRS). Approximately 1,900 feet of the channel from the FRS to the Courthouse (McDowell Road) bridge is lined with grouted rock riprap, shown in Figure 2-3. Downstream from the bridge, the unlined channel intercepts ,drainage from an 8.6-square-mile area across an alluvial fan. There are four grouted rock drop structures to maintain grade and to reduce velocity within the channel. The diversion channel outflow is a natural alluvial wash in an undeveloped area. The diversion protects agricultural development adjacent

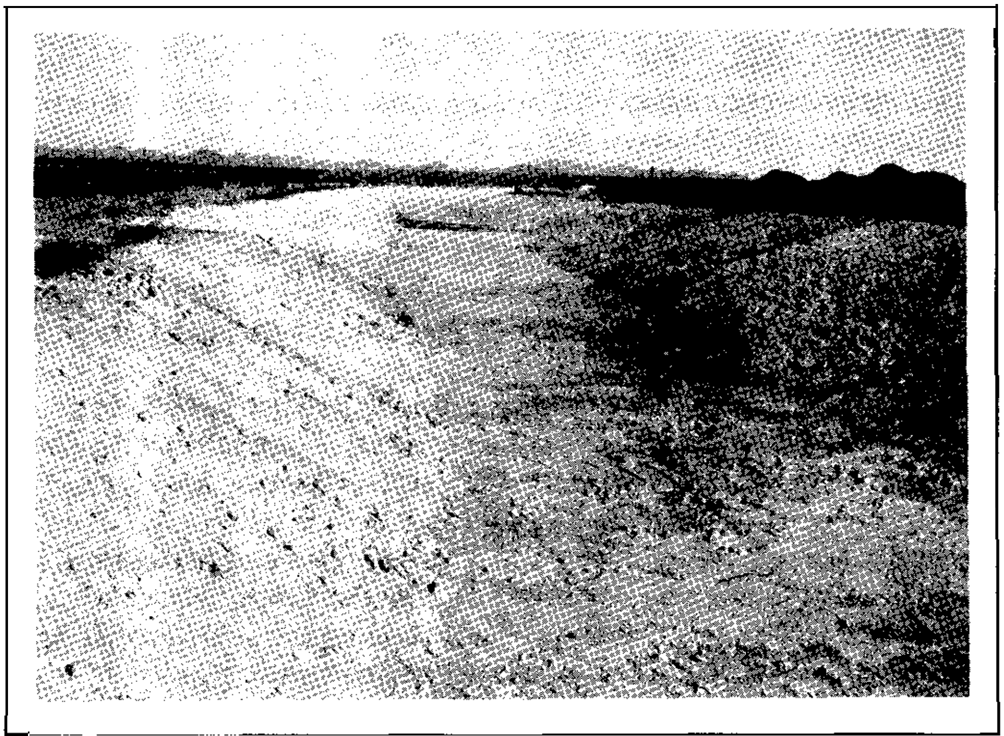


Figure 2-3 Lined Diversion Channel, Looking Upstream Toward FRS

to the diversion. Figure 2-4 is a photocopy of the project alignment on a site photograph, flown in March 1976, (north direction is the bottom of the page).

Runoff from the fan area flows into a collector channel running parallel to the diversion channel. A collector dike separates the collector channel from the diversion channel. The dike has a 12-ft top width and a 3-to-l-side slope toward the collector channel. The collector channels were formed as "lateral swales" to direct the overland flow into side inlets located along the length of the diversion channel. The collector channels were a maximum of 2 feet deep, with approximately, a 4-to-l-side slope toward the natural fan, and bottom widths ranging from 10 to 30 feet. Figure 2-5 shows a plan view of the collector channel (labeled: lateral swale) and side weir inlet, plus a cross-section view (B) of the collector channel and dike.

The original project had 18 side inlets at locations where the diversion dike intercepts natural washes that drain the west slopes of Saddleback Mountain. The side inlets are trapezoidal-shaped weirs, 2 feet deep, with 10 to 60 feet bottom widths, and 3-to-l-side slopes. (See Figure 2-5. As-Built Drawing.) The widths were sized based on the estimated contributing area for each inlet. All but one inlet are protected with grouted rock 2 feet thick. The diversion, side inlets, and collector channels were designed for a 50-year, 24hour storm.

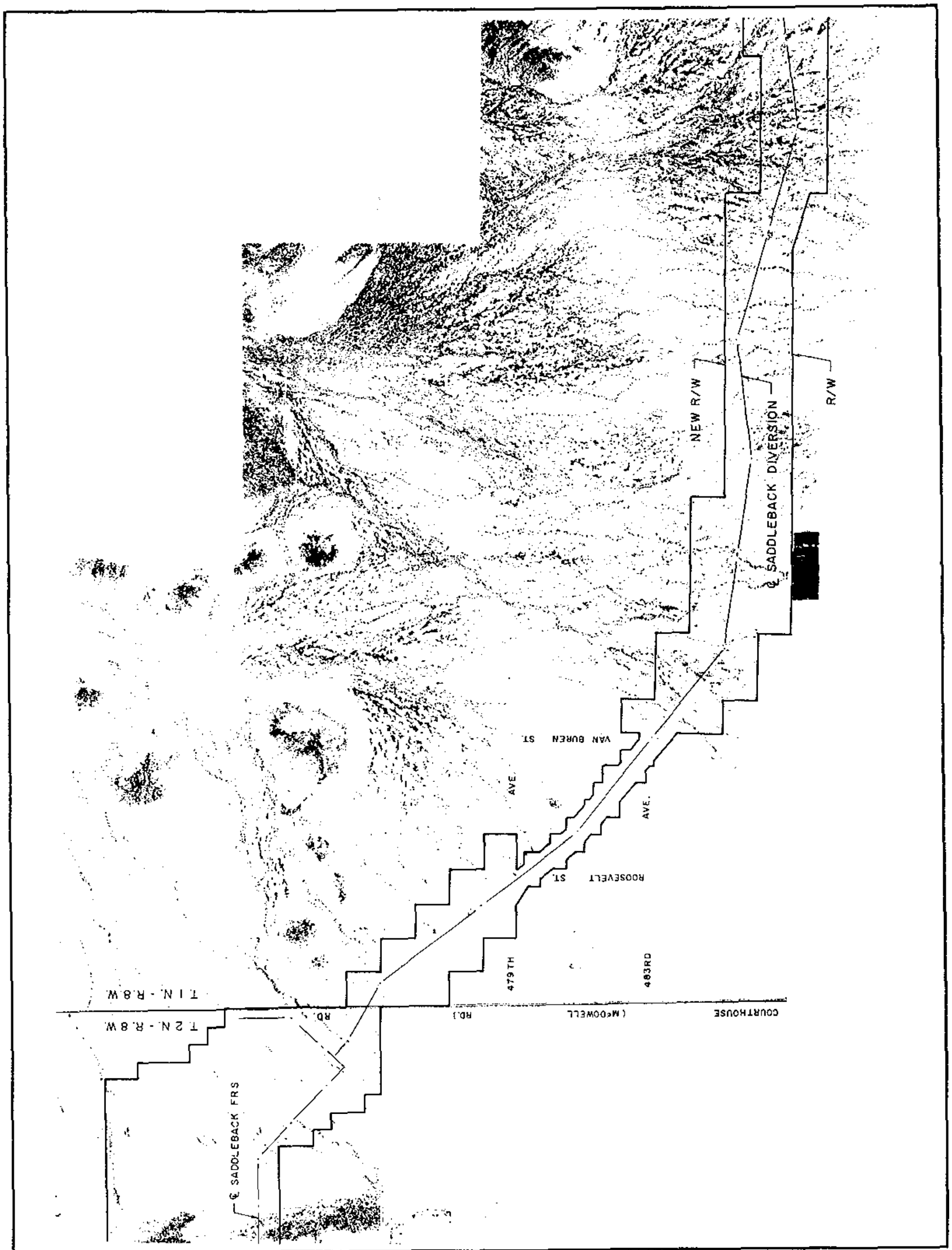
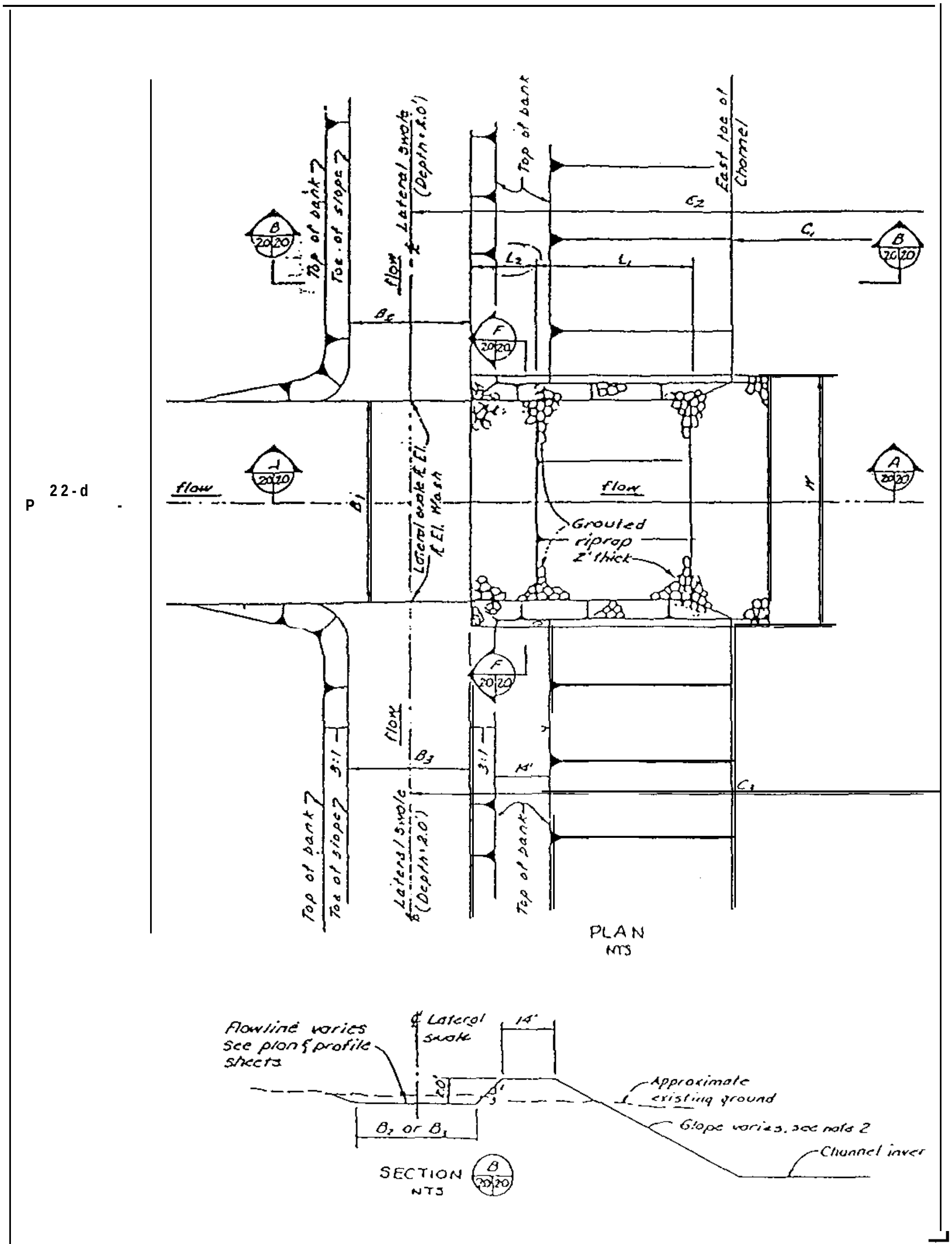


Figure 2-4 Site Photograph Showing Diversion Location Across the Fan



igure 2-5 As-Built Drawing: Collector Channel and Side Weir

Flood Experience

On September 2, 1984, a storm passed through the watershed with an approximate duration of 4 hours. The storm produced an estimated outflow of 739 ft^3/s at the Saddleback FRS outlet and approximately 12,355 ft^3/s at the diversion outlet, The design discharges are 1,120 ft^3/s and 6,060 ft^3/s, respectively. The diversion performed well during and after the storm, with a maximum water level within 1 ft of the top to the diversion dike. There was little erosion damage to the diversion channel.

The runoff from the alluvial fan caused sediment bar development in the collector channels, which caused the collector dike to be overtopped by floodwater; 15 of the side inlets were overtopped or flanked by erosion through the abutments. The damage occurred even in locations where the inflow did not exceed design discharge. Figure 2-6 shows severe erosion to the levee due to overtopping *(Photo No.* 9 from *referenced* report). The grouted side inlet is on the right edge of the photo and a small gully through the levee is at the left of the grouted inlet.

Evaluation

An engineering evaluation was performed by the Soil Conservation Service (SCS) and reported in the referenced Engineering Report. This project summary paraphrases sections of that study report. The project has been

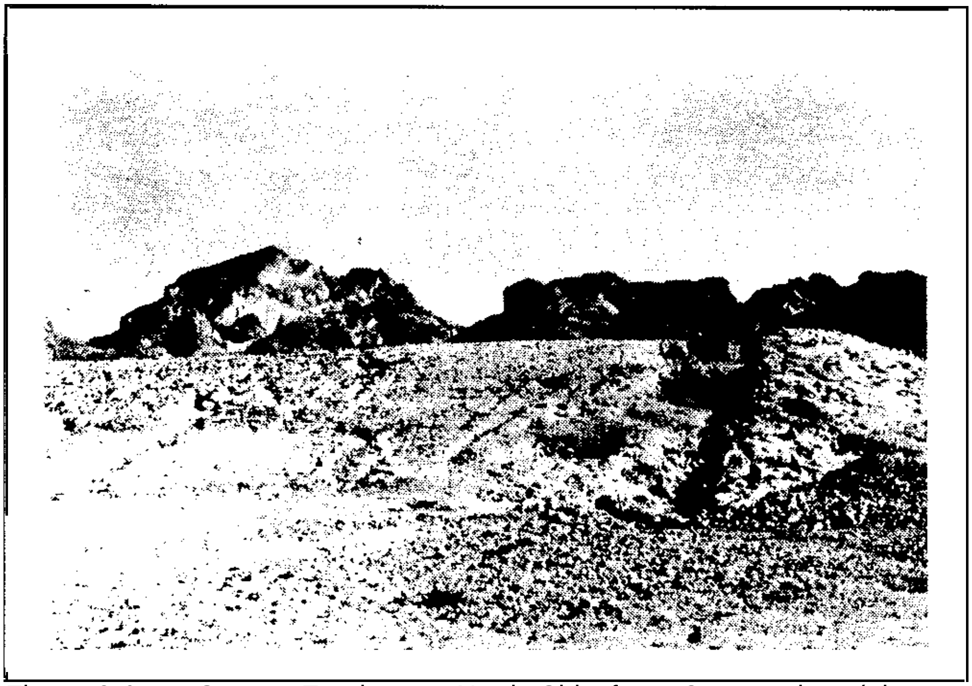


Figure 2-6 Severe Erosion on North Side from Overtopping (photo *No. 9 - Station 80+12 /n/et #7)*

completely repaired and was in excellent condition when the site was visited on November 3,1990.

While the storm event generally exceeded the design capacity at some locations in the project, the accumulation of sediment across the collector channels caused overtopping and breaching of the collector dikes. The storm caused flow braiding on the alluvial fan, erosion, and deposition of sediment in the collector channels. The collectors were not designed with sedimentation considerations, and there was no freeboard added to the dike. The accumulation of sediment deposits in the collector channel reduced the capacity and effectiveness of the collector to distribute the fan runoff to the side inlets.

There were locations on the fan where new drainage channels were formed during the event. Beside the contribution of sediment, these shiis in runoff changed the contributing area to some of the inlets; therefore, some inlets received runoff from more area than expected, while others had a decrease.

The SCS Engineering Report identiiied the following design criteria that need to be re-examined:

- Freeboard requirements for side inlets
- Freeboard requirements for collector channels
- Sediment accumulation in collector channels
- Over-designing collector channels to account for shifting, braided flow on alluvial fans

The project has been repaired with the following treatments:

- Side inlets were repaired and some enlarged or additional inlets were added to increase capacity.
- Collector channels were enlarged to provide for sediment.
- Additional side inlets were placed in locations where new major washes developed.

While the overflow of the collector dike required considerable repair, all the overflowing water was carried by the diversion channel, which performed well during and after the storm.

No probability was assigned to the 1984 storm, but it did exceed the 50-year design discharges at the lower end of the project. With additional capacity, this project should be able to offer flood protection at the 1-percent-annual chance level.

2.6 Lowell Creek Diversion

Location: Seward, Alaska

Reference: Office Reports and Project Files, USACE, Alaska District, Anchorage, Alaska.

Project Description

The project consists of a dam and emergency spillway that diverts Lowell Creek at the apex of the fan into a tunnel such that flows are completely removed from the fan, as shown in Figure 2-7. The drainage area upstream of the project is 4.02 mi^2. The diversion dam is about 400 feet long with a maximum height of 25 feet, The uncontrolled spillway is about 60 feet long, with a crest elevation 4 feet below the top of the dam. Flow overtopping the spillway would follow the former course of Lowell Creek through the City of Seward. The diversion tunnel is a 10-ft horseshoe about 2,000 feet long, on a gradient of 4.3 percent. A sharp drop at the tunnel entrance accelerates the water to a velocity of about 40 ft/s. This high velocity is necessary to ensure that all debris will pass into and through the tunnel. The tunnel is concrete lined throughout and the floor is armored with 40-lb railroad rails welded to channel cross-ties embedded in the floor, The design drawings indicate a minimum concrete thickness of 8 inches, with 8 inches of concrete below the floor rails. The space between the rails is filled with abrasion-resistant concrete. This project was constructed in the early 1940s.

Basis of Design

Hydrology. The project was constructed to replace a previous project, built in 1929, that consisted of a timber flume to carry water and debris through Seward to Resurrection Bay. That flume required heavy maintenance and had deteriorated to such an extent by 1937 that a replacement flood control project was urgently needed to protect Seward. The diversion/tunnel project was probably designed based upon observations of floodflows during the period in which the flume was in place. Hydraulic calculations indicate that the tunnel capacity is 3,200 fts/s and the spillway capacity is 1,600 ft^3/s, giving a total system capacity of 4,800 ft^3/s. Contemporary frequency computations indicate

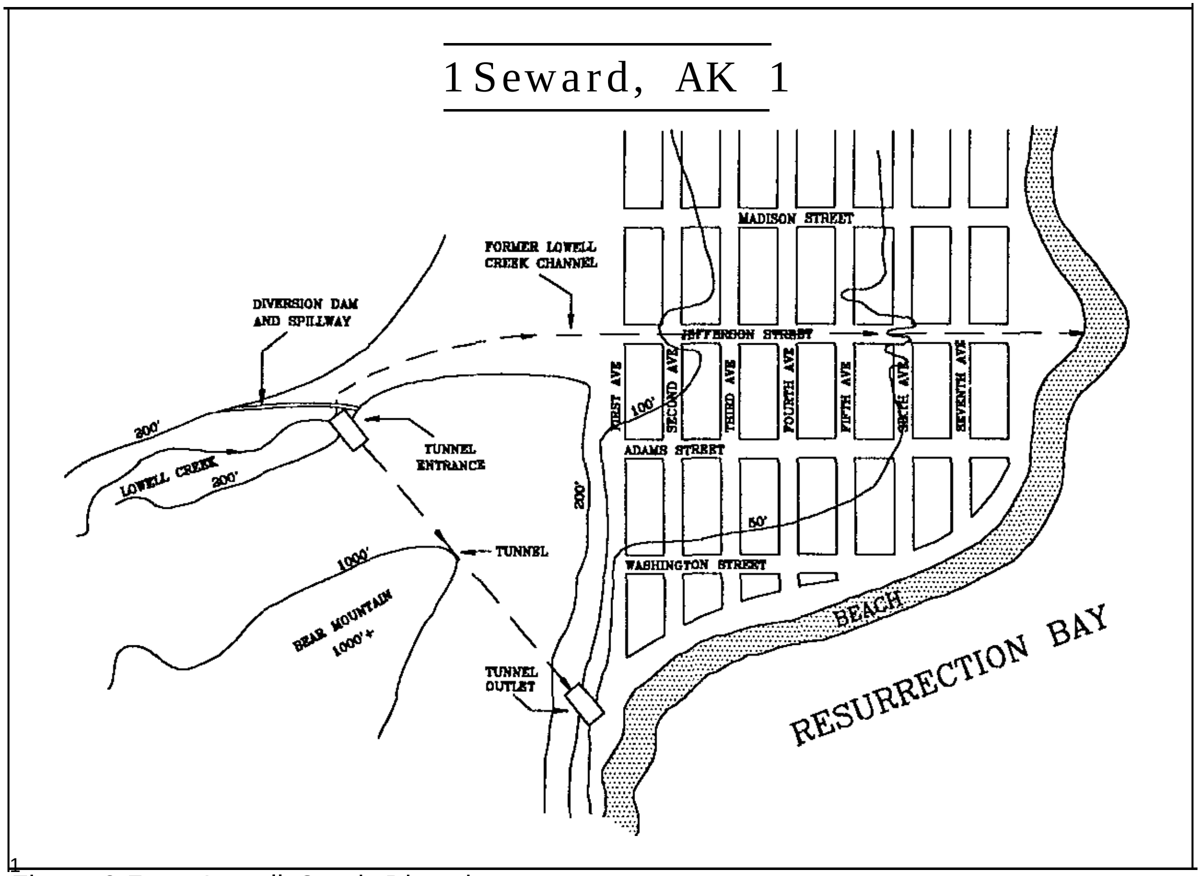


Figure 2-7 Lowell Creek Diversion

that the project is capable of conveying one-half of the PMF (2,200 ft^3/s) without flow over the spillway, provided that the tunnel is not blocked by debris, landslide, or ice accumulation. Note that the USACE, Alaska District uses a procedure to modify frequency curves to reflect the possible occurrence of "surge-release" flood events. These events occur when the channel is temporarily blocked by a landslide, which is subsequently overtopped and breached during the storm.

Hydraulics. No records of the design criteria or methodology were found. It appears that classical hydraulic computations were performed based upon observations of floods carried by the flume. It is not known if the tunnel was designed to flow full at its design flow.

Sediment/Debris Historical information from the period during which the flume was in place was apparently used. The significance of the debris load was recognized by the designers, although little quantitative information was available. It was reported, for example, that 10,000 yd3 of material were deposited in the flume in 11 hours during a flood in 1935.

Flood Experience

The flood of October 1986 is believed to be the largest known flood since Seward was established in 1900. The peak discharge was estimated to be 1,900 ft^3/s, which is approximately the l-percent chance exceedance flood. During this flood, the water surface came to within 6 inches of the spillway crest, due to a landslide above the tunnel entrance that temporarily blocked the tunnel. The City of Seward had an observer at the dam during this event. He reported the conditions and the potential flood area downstream of the spillway was evacuated in 30 to 45 minutes. The tunnel was also partially blocked by a similar landslide in August 1966, when the water level came within 2 feet of the spillway crest. Major damage to the tunnel lining occurred in 1986 and continues to be a problem.

Over the years, the concrete lining of the tunnel has been abraded; reinforcing rails have broken loose: and some of the natural rock has eroded due to the high-velocity movement of large material through the tunnel. After both the 1966 and 1988 floods, large rocks (4 to 10 tons) were found in the tunnel. No major repairs were made to the tunnel until 1968, when loose rails in the center of the invert were removed and replaced. By 1974, much of the concrete replaced in 1968 had eroded and the rails were again coming out. (The rails fail by working loose at the upstream ends, bending upwards, and trapping more debris creating the possibility of a tunnel blockage.) Erosion of a large hole through the tunnel lining in the inlet transition began around 1977. With the loss of the tunnel lining, the tunnel is no longer structurally sound and may not resist the overburden compressive forces, leading to the possibillty of a collapse. In 1981, the State of Alaska appropriated $3 million for tunnel repairs that were performed in 1984. During these repairs, failed rails in the middle third of the tunnel were removed and not replaced. By 1987, the middle third of the tunnel floor had eroded to bedrock at 18 locations. It was noted at that time that the tunnel walls had not sustained significant damage over the 40+ years of operation. By January 1988, an estimated 30 percent of the middle third of the tunnel floor had eroded to bedrock at 44 separate locations, and much of the bedrock had also been scoured. Emergency repairs were made in early 1988 by the USACE. These repairs were made only to damage from the 1986 flood and did not restore the entire tunnel to its original design, Part of the tunnel floor was unrepaired and left covered by a thin (2-inch or less) layer of concrete.

The tunnel outlet, which consists of an open concrete flume 10 feet wide by 100 feet long, has also experienced damage from erosion, including loss of reinforcing rails. The ground level downstream of the outfall to Resurrection Bay has been raised due to the accumulation of material transported through the tunnel. In September of 1982, the material transported through the tunnel during a flood, blocked a road at the end of the tunnel (not shown in Figure

2-7); apparently this event has occurred several times, The diversion of sediment may have had negative impacts in the fan area as well. In 1964, for example, Seward experienced catastrophic landsliding along the distal edge of the fan delta during The Good Friday earthquake. It has been speculated that this landsliding would not have been so severe, or nonexistent, had Lowell Creek continued to deliver sediment to the distal edge of the fan delta.

Several design deficiencies have been identified that need to be addressed in the future.

- No provision was made for flood passage through the area downstream of the spillway. Since construction of the project, this area has become heavily developed to include single-family homes, senior citizens' apartments, and a hospital. The former creek bed is now occupied by a major residential and business street (Jefferson Street).

- There was no emergency plan developed for action to be taken should the tunnel be blocked or long-term impoundment behind the dam occur. (The dam was not designed for long-term impoundment, and the possibility of seepage and subsequent failure exists).

- There was no provision for a monitoring and warning system. Lowell creek is subject to flashflooding, and there is extreme hazard to life and property should the tunnel diversion system fail.

2.7 Fourth of July Creek Levee System

Location: Seward, Alaska

Reference: Office Reports and Project Files, USACE, Alaska District, Anchorage, Alaska.

Project Description

The project is located on an alluvial fan directly across Resurrection Bay from the City of Seward. It consists of a levee system that was constructed to protect land on the fan for development. The levees confine the stream along the southeast margin of the fan delta. This project has reduced the active surface of the fan by about 70 percent. The project drainage area is about 2.5 mi^2. Channel slopes are about 900 ft/mi in the canyons, 200 ft/mi near the apex of the fan, and 50 ft/mi just above the levee system. Fourth of July Creek is a typical glacial-fed stream with low flows occurring in late fall, winter, and early spring, and high flows occurring in summer and early fall. High summer flows carry a large amount of suspended sediment derived from glacial outwash. Low winter flows are mainly derived from groundwater seepage, and carry little sediment. The project was constructed in 1981-82 by the City of Seward.

Flood Experience

A storm in 1988 caused landslides in the canyon of the north branch, which temporarily dammed the stream, resulting in a "surge-release" flood event. Discharge derived from high water marks in the canyon after the debris dam was breached was 30,000 ft^3/s. This is estimated to be about a O.&percent chance exceedance event. This event flowed at an average of 3 feet below the levee crests. Surveys indicated that, during this event, 3 to 4 feet of sediment and woody debris were deposited in the lower floodway. These deposits forced a majority of the flow to impinge on the levee, causing a portion of it to fail during the falling limb of the hydrograph. No damage to the protected areas occurred, however, because flows were low by the time the levee was breached. Damage to the levees was repaired by using larger riprap on the levee face; however, no improvements to the levee toe were made. Between sta. 8+00 and sta. 49+00, about 3.5 feet of scour was observed. In 1989, an estimated lo-percent chance exceedance event occurred. During this event, a log jam in the vicinity of the north diversion dike diverted the stream 90 degrees, causing it to impinge directly into the diversion dike, and scouring the toe and lower portions of the dike. The dike was again repaired with larger riprap. After the 1989 flood, a scour hole 4 feet deep was noted at the confluence of Fourth of July and Godwin Creeks.

Basis of Design

Hydrology. Only a few random-flow measurements are available on Fourth of July Creek. Flow-prediction equations based on multiple regression techniques were used in 1980 for the original levee design. Regression equations developed by the US. Forest Service were used. The levees were designed to withstand the i-percent chance exceedance flood event; the flow rate used is not known. Contemporary frequency analyses by the USACE place the magnitude of this event at 8,600 ft^3/s. Water surface profile computations using HEC-2 show that this event would flow 4 to 5 feet below the levee crests.

Hydraulics. No specific design procedures were identified. From the information available, however, it appears that the levee heights were selected using step backwater computations, probably HEC-2.

Sediment/Debris. During levee design, sediment samples were taken on the lower portion of the fan-delta and bed load computations were performed. Suspended sediment samples were also taken. The estimated suspended load for Fourth of July Creek and data from other Alaskan glacial streams were analyzed; these data correlated quite well with an empirical relation developed by Nordin from data on the Rio Grande. HEC-6 was used to estimate a channel size and configuration that would yield an equilibrium condition with regard to total sediment transport. It appears that this was done for the design flow event. The computed results indicated that the high proportion of larger-sized material in the stream bed provides an armor layer after a small amount of scour at bank-full stage. The original design computations concluded that the total sediment load would pass through the leveed reach with little or no net scour or deposition. The upper basin has been logged several times in the past and stumps, slash, and trees are regularly carried downstream by the higher flows. This debris can and does run aground in bends and shallow areas, causing log jams and still-water areas, where deposition occurs. Changes in local energy gradients at these log jams can cause considerable local scour. The combination of deposition and scour in the area of the grounded debris can cause abrupt changes in flow direction. It appears that the original design did not address the ability of braided glacial streams to migrate laterally and scour locally. This problem may be particularly acute for floods that are less than the design flood where the entire width between the levees may not be occupied by high-velocity flows.

2.8 Gold Creek Channelization Project

Location: Juneau, Alaska

Reference: Office Reports and Project Files, USACE, Alaska District, Anchorage, Alaska.

Project Description

The project consists of about 1,700 feet of reinforced concrete channel with intake and downstream energy-dissipation structures for the portion of Gold Creek passing through Juneau, which is located on an alluvial fan. Thickness of the concrete is 12 inches on the channel bottom and 10 inches on the side walls. The average bottom slope of the channel is 90 ft/mile. The project was initially completed in March 1956 by the USACE. The City of Juneau is responsible for operation and maintenance.

Flood Experience

The project deteriorated somewhat during its first 2 years of operation. The deterioration accelerated rapidly during the 3 year and failure of the structure was likely. The extent of the deterioration indicated a design and/or construction deficiency, so the project was rehabilitated by the USACE. In 1962, two-thirds of the channel length was repaired and a minimum-sized debris basin (1,000 yd^3) was excavated at the upstream end to capture larger sized material that was causing the channel deterioration. Periodic inspection showed that the portion of channel not repaired had continued to erode, with complete erosion of the bottom slab in some places, It was observed that the debris basin generally filled within 1 week after it had been cleaned.

Test panels were installed in the channel in 1963 to help determine the best type of channel lining for the conditions, The five test sections were:

1) Steel plate
2) 70-lb rail with high strength concrete
3) Steel armor grating embedded in high strength concrete
4) Rubberized sheet tar
5) Epoxy resin grout with high strength concrete

Two weeks after application, the rubber tar section had failed completely. After 1 year, the epoxy resin grout panel showed one-half inch of wear and some loss of bond to the original concrete. Four months later, the steel plate was pitted and had lost anchors and fastenings. The steel rail and checker plate showed only minor wear.

An inspection in 1964 identified large material, about 1 ft in diameter, deposited at the lower end of the channel that had obviously traversed the entire length of the paved reach. Repair of the channel and improvement of the headworks to eliminate the main source of debris was completed in June 1968. Repair work to the remaining original channel lining was done in 1975. Repair work was again needed by 1979 and was completed in 1984. The debris basin has been enlarged to 5,000 yd^3 and is periodically cleaned out by the City of Juneau. Other than the necessity for periodic repair of the channel bottom, the project has performed well.

Basis of Design

Hydrology. Precipitation and stream-gaging data are available for the Gold Creek basin from the early 1900s. The original (1949) design was based on traditional flood frequency procedures using these data. Level of protection for the design was SPF. Use of subsequent data and reevaluation in the early 1960s confirmed the SPF magnitude of 4,800 ft^3/s.

Hydraulics. No specific design information was available; however it appears that the design was performed using USACE guidance available at that time for supercriiical flow channels. Note that the channel bottom was super-elevated on curves.

Sediment/Debris. No specific design information was available. From the history of the project, however, it appears that the erosive capability of the large size material passing down the channel was underestimated. The repairs and modifications to the project were based upon observation and field testing.

2.9 Wild Rose Development Project

Location: Riverside County, California

Reference: "Flood Control Improvements on Alluvial Fans" (Schall, D.D.; Sender, D.W.; and Peairs, F.J., 1990).

Project Description

Wild Rose development was under construction when inspected in December **1990.** As a new project, there is no flood experience to document. However, the referenced paper provides a good overview of the alternatives for the project and some general characteristics for those akernatives to be attractive. The project is being developed with a flood control channel to transport both debris and water through the development. The following presents some of the aiternative considerations as presented in the reference. The reference uses the word "debris" as an equivalent to "sediment".

Floodplain Management Alternatives

According to the reference paper, there are four basic ways to manage development on alluvial fans:

1) Mapping and zoning to prohibit development in the areas exposed to flood and debris flow;

2) Developing confined channels that transport flood and debris flows safely through the development;

3) Creating detention storage above the development for debris and transport of the relative clear water through the development; and

4) A combination of 2 and 3, where the debris basin is sized to contain debris from most events and, for larger events, the debris is transported downstream.

Some of the key factors considered for each aiternative include:

- Controlling development by zoning requires accurate mapping of the hazard area and a master plan for the entire alluvial fan. (The "Upper *Indian Bend Wash Regional Drainage and Flood Control Plan" is an example of a master plan; however, it represents a combination solution as described in alternative* 4.) Besides the uncertainty in defining the flood-prone area, the major problem with this alternative is the large land area that would be required to accommodate the potential flood and debris flow. The Wild Rose property did not have adequate land for a natural floodway or open space.

  Transporting both debris and water through a development must be carefully engineered, constructed, and maintained to be successful. A major problem is to maintain debris transport with the potential for deposition contributing to channel blockage and overflow. Maintaining the debris flow has the advantage of contributing sediment to the downstream channel system and, in coastal streams, the sediment supply to maintain beaches.

- Using debris basins and downstream concrete-lined channels is a standard approach to flood control on alluvial fans, particularly in Southern California. The cost of constructing and maintaining the detention basin is generally high, and dam-safety issues are a concern. Dams located immediately upstream from urban development constitute a high risk. If debris loads for extreme events exceed design, the storage is lost and the potential for failure increases. Trapping debris in the reservoir can also cause downstream channel degradation due to the reduced inflow sediment load.

- The combination of debris basin and flood-control channel is fairly common. This alternative usually provides debris control for smaller floods and transport for the larger floods; reducing the construction cost of the basin and the downstream hazard from a failure. Smaller debris basins would require more frequent cleaning to maintain their effectiveness. Temporary debris basins are frequently used in the Southern California watershed after a fire.

  Limited land area precluded the use of "natural" floodways at this project site. The potential site for a debris basin did not provide sufficient storage volume with a conventional earthfill structure.

Transport of debris through the channels was possible because the steep slope would provide high-velocity flow. The final design of the channel facilities were based on a bulking factor near 2, and maintenance of high channel velocity with minimum grade breaks and channel curves to minimize potential deposition of debris. The channel bottom and the lower side slopes were designed with extra concrete to provide a wearing surface and extra reinforcement to support the channel during a debris-flow event.

3. EXPERIENCE WITH FLOOD-CONTROL MEASURES

Chapter 2 provided case studies that demonstrate some of the problems associated with the performance of flood-control measures. Additional information has been gleaned from other reports and books. The following sections summarize this experience for each structural flood-control measure. A general description of the measure and its application is provided. General experience and potential sources of failure are described and, if appropriate, recommendations on applicability are provided.

3.1 Measures to Control Flood and Debris Flow

Structural Flood-Control Measures

Structural flood-control measures attempt to decrease the flow and/or decrease the flooding depth. Measures that decrease the flow include reservoirs and diversions, while measures that decrease flooding depth include channel alterations, levees, and floodwalls.

Structural measures used to control flood and debris flows on alluvial fans have consisted primarily of the eight following:

1) Debris Barriers
2) Debris Basins
3) Detention Basins
4) Flood-Control Channels
5) Diversions and Bypasses
6) Floodwalls
7) Levees
6) Floodwalls and Dikes

Debris Barriers Debris barriers are structures, usually located in the watershed, that stop or reduce the movement of debris down the channel system and onto the fan.

Debris Basins. Debris basins are facilities that are designed to settle out, and provide storage for, coarse material and trash resuiting from a major storm. They are usually located at the upper ends of alluvial fans.

Detention Basins. Detention basins are storage structures, usually uncontrolled, and are designed to reduce the peak flood flow from a drainage basin. Often one basin serves both purposes of debris and detention basin. The two are presented together in this report.

Flood-Contol Channels. Flood-control channels are engineering works designed and constructed to pass flood discharges more efficiently than natural channels, thereby reducing flood stages. Flood-control channels usually follow the natural channel course.

Diversions and Bypasses. Diversions and bypasses are man-made channels designed to provide additional flow capacity or to direct the flow away from developed areas.

Fbodwalls. Floodwalls are vertical walls, usually made with reinforced concrete, oriented parallel to a stream to prevent overflows into developed areas.

Levees. Levees are dikes, usually earthen and parallel to the stream, that are designed to prevent overflows into developed areas.

Fbodwalls and Dikes. Floodwalls and dikes are sometimes used in conjunction with diversions. Also, they may be placed across the fan to divert the flow away from developed areas and reroute it toward local washes or sediment/debris-detention basins before disposing the excess flow into a nearby river or into the local storm drainage system. Examples of floodwalls and channel diversion systems are found in Clark County, Nevada, Bullhead City, Arizona and at Rancho Mirage, California.

3.2 Debris Barriers

Debris barriers are not flood-control measures in the strictest sense, they are barriers designed to block large materials like rocks, boulders, and floating debris (e.g., logs, brush, branches, trash, shopping carts and other large objects that may get entrained by high-velocity flows). Barriers can be a component of a flood-control plan; however, they do not significantly reduce flooding. The barriers can be permanently placed, or can be part of an emergency response after watershed fires or landslides, to retain an expected increase in debris loading. Southern California watersheds have many examples of both permanent and emergency applications. The Oakland Hills fire in November 1991, near Oakland, California, led to the immediate threat of possible landslides and debris flows with the rainy season about to occur. The SCS and the City of Oakland installed several different types of debris barriers throughout the burn area, including debris fences (chain link and wire mesh), silt curtains along the contours on steep slopes, log cribs, and small check dams. Debris barriers encompass several different forms, including fences, small walls, cribs, and check dams.

Debris Fences. Debris fences are typically vertical beams (with or without connecting wire) anchored in a foundation perpendicular to the expected debris flow. Debris fences, by themselves, provide only incidental protection from debris during major events, and are not recommended as a general solution to debris-related damage. The primary purposes of fences are to retard the rate at which debris moves down the slope; to catch a portion of the debris; and to break up the flowing mass, thereby allowing the escape of air that may be trapped under the flow. Such trapped air can serve to reduce the friction between the flowing debris and the ground, resuiting in increased velocity. Debris fences often fail during a debris flow; however, if the mode of failure is a bending of the fence rather than an intact movement or sudden failure downslope, the fence will, generally, still slow the flow and trap a portion of the debris.

The placement of the debris fence has a tremendous effect on the ability of the fence to retard the rate of the flow. If possible, several small debris fences should be constructed in the area where drainage concentrates to serve as debris collectors from small-slope movements and to slow the flow and break up the energy at the inception of the event, preventing the occurrence of larger-scale debris flows further downstream. Fences placed at the toe of steep slopes are likely to fail unless they are properly sized and reinforced, due to the high speeds at which the debris-laden flows can move and the high-impact loading that can occur. Debris fences must be inspected periodically and cleaned or repaired as necessary. Vandalism, such as cutting a fence with wire cutters, is sometimes more difficult to manage than the repairs following small- to intermediate-sized events. Debris fences are best applied in emergencies or as temporary measures, and should not be considered to be permanent flood-control structures (refer to Section 2.4).

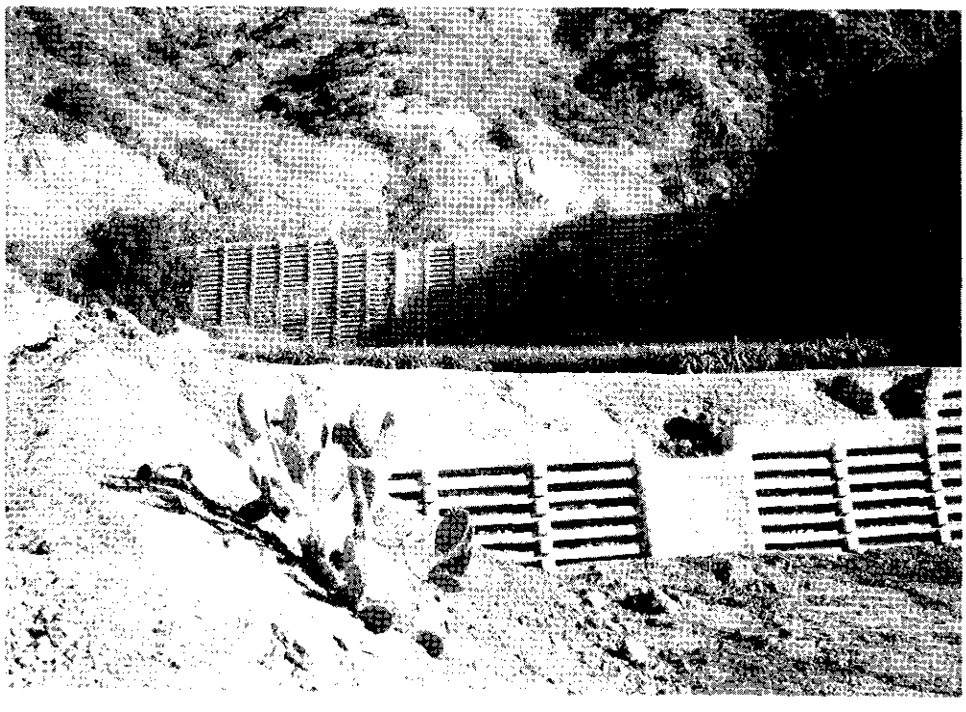
Debris Barrier Walls. Also known as "fire barriers," debris barrier walls are constructed in Southern California across canyon mouths, following watershed fires, in anticipation of debris flows induced by heavy winter rains on unvegetated hillsides. Numerous examples have been constructed by the Los Angeles County Flood Control District (LACFCD) in the San Gabriel Mountains. Debris barrier walls are typically constructed 7 to 15 feet tall, with 2-by-12 inch and 3-by-12 inch timbers supported by 60-lb rails set in a concrete foundation (LACFCD, 1979). Although these structures have been intended to provide temporary protection while protective natural vegetation is re-established in a burned watershed (lasting 3 to 5 years), in practice, they have proven to be quite durable. Examples of filled and unfilled fire barriers in good condition up to 15 years old are found in the San Gabriel Mountains near Glendora, California. Figure 3-1 shows a typical Southern California debris barrier wall.

Figure 3-I Debris Barrier Wall in Southern California

Crib **Barriers.** Crib barriers are constructed like small "check dams" across a channel. Concrete cribs have been used in Southern California by the U.S. Forest Service and by Los Angeles County as debris and flood-retarding structures. Cribs, which are periodically cleaned, function like small-debris basins. Crib structures are often constructed in series. The upper cribs, located in narrow canyons, may have limited access and, as a consequence, are usually not cleaned of debris. Upon filling, the area upstream from the crib becomes a heavily vegetated "wetland" and acts to retard flood flows by virtue of flatter slopes and higher hydraulic resistance. In addition, the filled cribs add stability to the toe of canyon side slopes, thus providing additional debris control.

Current design and application of check dam structures in Southern California has evolved significantly from experience during the past 75 years. During the period of 1914 to 1920, more than 1,500 "loose-rock" check dams were constructed in Los Angeles area watersheds; however, subsequent floods demonstrated conclusively that the "loose-rock" check dams were unable to stand up to the impact and buoyancy forces of debris flows. Following the flood of 1920, "rock-and-wire" check dams, which utilized wire-bound rock mattresses, were constructed. The rock-and-wire check dams performed better, but the continued pounding of debris passing over the structures broke the wire mesh enclosing the rock mattresses. These structures were utilized until numerous failures during the New Years Flood of 1934 lead to their abandonment.

In the late 1930s the U.S. Forest Service designed and built a series of "mortar-rubble" arch (up to 40 feet tall) and gravity dams. These structures have proven to be both durable and effective. In the 1940s the U.S. Forest Service utilized a wide variety of structures and materials, including soil cement, various metal structures, and concrete cribs, for the Arroyo Seco Flood-Control Project. This project has provided valuable cost and performance information on the alternatives employed. The concrete crib (see Figure 3-2 below) emerged from the Arroyo Seco Project as the most favorable of the various alternatives. In the 1950s, the LACFCD and the U.S. Forest Service joined forces to design and construct two experimental projects that included 79 concrete cribs (IACFCD, 1959).



gure 3-2 Concrete Crib Barriers in Southern California

Experience has shown that the grouted spillway section employed on the earliest structures could be damaged by debris. Recently constructed concrete cribs employ a cast-in-place reinforced spillway section that has proven to be more durable. During the past 40 years, Southern California's experience has demonstrated that properly designed and constructed concrete cribs are an effective and durable type of debris barrier. Another application is crib walls built along highways to trap and block debris from entering the roadway. The California Department of Transportation (CALTRANS) "Standard Plans" has design details for concrete, steel, and timber crib wails (CALTRANS, 1988).

Retaining Walls. Retaining walls (6 to 10 feet high) are another common device for protecting single-lot structures in subdivisions. Deflection walls are another form of retaining wall, only placed at an angle other than 90 degrees to the direction of the slope. Observations of retaining and deflection wall performance during large flood events, generally, indicate that wall failures are usually associated with either poor construction methods or inadequate design (the failure to recognize the potential magnitude of impact and buoyancy forces).

Retaining walls constructed of concrete block are commonly employed at this time due to reduced construction costs; however, they are the most susceptible to failure. Steel-reinforced, poured-in-place concrete retaining walls are far more durable than hollow block walls because they do not possess planes of weakness such as those that exist between concrete block and mortar. Also, proper placement of steel reinforcement in poured walls to resist tensile stresses is far more feasible than in block walls, and the bond between the steel and the concrete is superior in poured walls.

3.3 Debris and Flood-Detention Basins

Debris basins may be limited to trapping debris; however, most applications also provide flood detention. Typically, these basins are located in the watershed or near the apex of a fan where flow and debris are confined. The narrow width of the watershed canyon often provides the most attractive sight for locating a dam. Debris basins are often combined with an improved channel downstream from the debris basin. The West Magnesia Debris Basin and Channel provide standard project flood (SPF) protection to Rancho Mirage, California (USACE, 1988). Rancho Mirage below Magnesia Spring Canyon is often shown as a classic example of alluvial fan development (e.g., the cover photo on "Alluvial Fans: Hazards and Management," (FEMA, 1989)). The Magnesia Springs fan was also the basis for a model study conducted by Anderson-Nichols (1981) for FEMA. As shown in Figure 3-3, the USACE

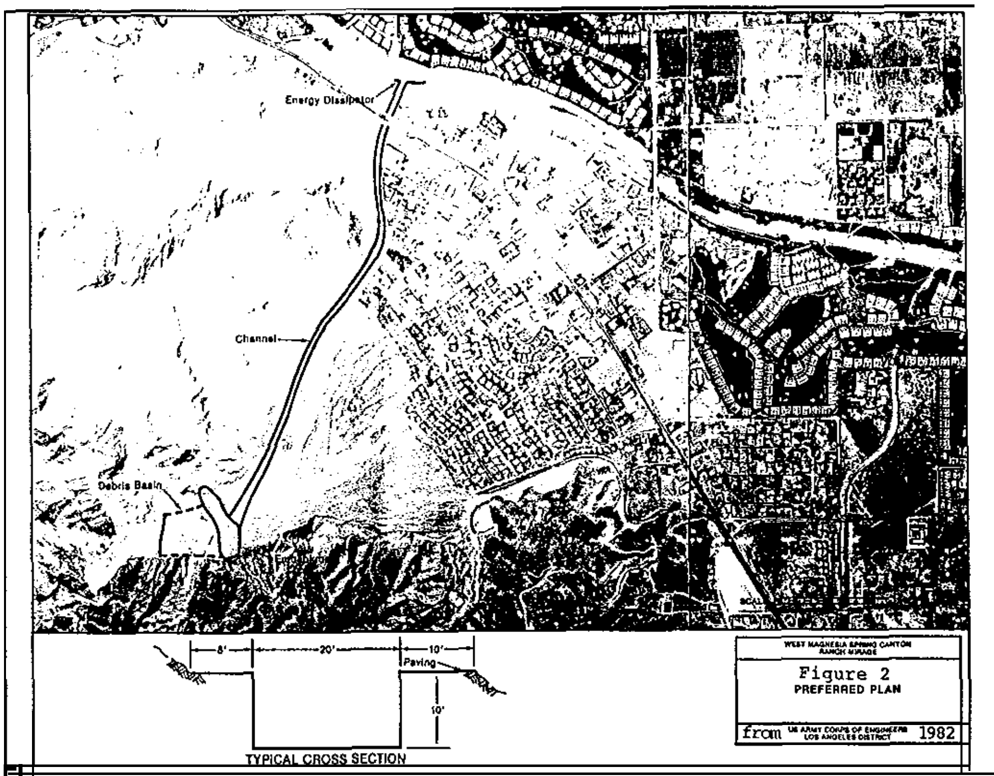


Figure 3-3 West Magnesia Spring Preferred Plan, Rancho Mirage, California

project consists of a debris basin with a spillway and concrete-lined flood-control channel with an energy dissipator at the downstream end, where the channel enters the Whitewater River.

The Magnesia Spring Canyon Debris Basin was required at the upstream end of the concrete-lined channel to ensure that the inlet capacity was not reduced due to sand deposition; to minimize the scour of the concrete lining caused by coarse sediments moving at high-velocity; and to ensure the functional adequacy of the outlet structure (USACE, 1983). The USACE, Los Angeles District used traditional procedures to develop the SPF hydrology; however, to size and design the sediment retention basin, the engineers worked with geologists, soil scientists, and local flood-control personnel to evaluate soil erodibility and the potential sediment volume for the SPF. The Tatum Method (1963) was modified to better represent the sediment production for the basin conditions in the area. The sediment production estimate was based on soil type, vegetative cover, slope angle, soil erodibility, and an estimated 10-year Tatum burn recurrence condition. Computed values compared favorably with regional experience. The debris storage requirements were based on past experience that has found that the slope of material deposited after a major flood averages about one-half of the original slope.

The capacity of the debris basin (150,000 yd^3) was determined by calculating the volume between the excavated invert of the basin and the deposition slope projected upstream from the spillway crest at 0.5 of the natural slope," (USACE, 1983).

The spillway and concrete channel sections were designed according to the guidelines defined in the USACE Hydraulic Design of Spillways (USACE, 1965) and Flood-Control Channels (USACE, 1970, revised 1991). The rectangular spillway was designed to pass the probable maximum flood (PMF). The spillway length and height were based on studies of the relationship of debris storage, embankment height, spillway crest length and spillway transition length. The pool drain was located upstream from the spillway, with the intake tower 1 ft above the elevation of the assumed debris level at that location. The drain pipe was sized to operate under inlet control (not under pressure) and to drain the pool within 1day.

Flood protection for the community of Rancho Mirage would not have been adequate without the addition of the West Channel Project. The western side of the development was subject to flooding from the western foothills, A series of diversion dikes divert water and sediments emanating from the western foothills into a diversion channel, which carries the water into a sediment-retention basin that releases the water into the local storm drainage system and drains runoff generated on the surface of the developed fan.

The SCS has developed several Debris/Flood Detention basins in Arizona. A review of the flood experience at Saddle Back Diversion indicated that the Flood-Reduction Structure (FRS) performed well (Chapter 2). This project and the Buckhorn Mesa Watershed flood-diversion and retention structures near Mesa, Arizona (SCS, 1984), are long, earthen, levee-like structures across the fan, capturing runoff from a portion of an alluvial fan. The Buckhorn-Mesa project also uses the levee, shown in Figure 3-4, to divert alluvial fan flow to the FRS. These structures would only be practical where there is sufficient open space to provide for flood inundation during flood events. Because these are dry dams, there is a potential for open-space land use within the storage zone of the reservoir.

The SCS "Earth Dams and Reservoirs" Tech Release No. 60, describes design procedures and provides minimum requirements for planning and designing earth dams and associated spillways (SCS, 1985). The SCS also has "Simplified Method for Determining Floodwater Retarding Storage," Tech Release No. 032, (SCS, 1966). The SCS has developed a covered riser that minimizes potential for debris blockage (from floating debris) on the outlet works; Tech Release No. 29 (SCS, 1965). Both the Saddle Back and Buckhorn Mesa projects have this type of outlet, see Figure 3-5. These outlets are designed for flood water, and would not be appropriate for debris basins,

Figure 3-4 Pass Mtn. Diversion Dam, Mesa, Arizona (Flow trapped on left diverted upward in picture.)

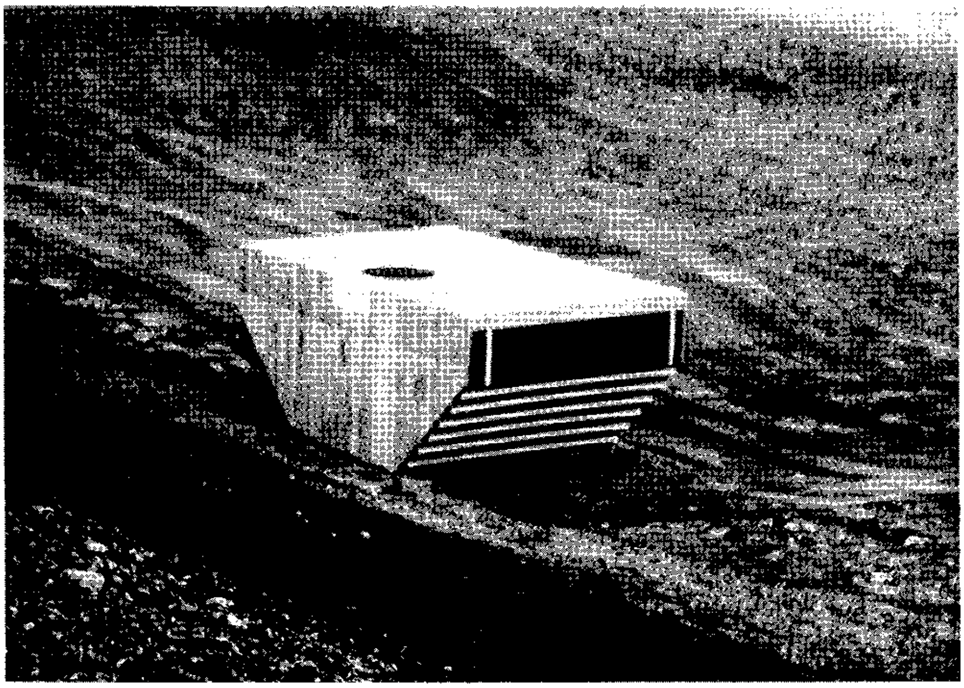


Figure 3-5 Covered Riser Inlet at Signal Butte, Mesa, Arizona

Debris basin outlets must be designed to pass the flow as the basin fills with sediment. The typical outlet works for a debris dam would be a circular tower with slotted openings around the circumference. Several of the projects, reviewed in Chapter 2, set the top of the inlet riser at least 1 ft above the emergency spillway elevation. If the riser is located some distance upstream from the spillway, the upstream slope of the sediment deposit should be considered. Several designs used a slope of one-half the natural upstream slope to estimate the surface slope of deposited sediment in the debris basin. The LACFCD's *Design Manual Debris Dams and Basins* (LACFCD, 1979) provides guidance for design, preparation of plans and specifications, and construction of debris dams and basins.

The Central Arizona Project (CAP) canals are protected, in some locations, by a flood-retention structure across fan outwash areas, just above and parallel to the canal. The outlet works are simple culverts (referred to as "overshoots") that carry the flood flow across the canal, preventing contamination of the canal water. Figure 3-6 shows the flood-retention structure and the inlet to a culvert crossing the CAP. The flow is then released into controlled or natural drainage systems below the canal. While the rural locations support using the natural drainage systems, more developed locations typically use a concrete-lined flood channel. The local experience indicated that flood-control performance has been favorable; however, there have been some problems with settlement. The settlement is probably the result of consolidation of fan

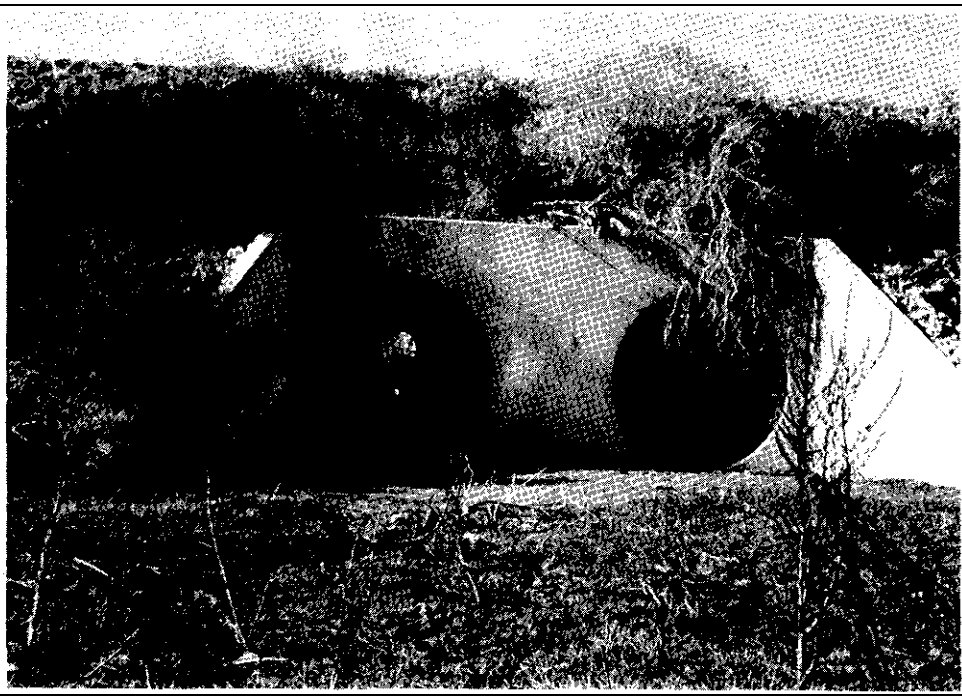


Figure 3-6 Flood Retention Structure and Inlet to Culvert Crossing Central Arizona Project, near Scottsdale, Arizona

material caused by flood inundation. Similar problems have occurred along the California Aqueduct in the Central Valley of California. Portions of the aqueduct right-of-way were flooded to consolidate the material prior to project construction.

The culvert outlets could only be used in those locations where sediment transported by the inflow is low; otherwise, the sediment would settle out from the inflow while it was in storage behind the embankment, and would tend to block the culvert inlet and reduce its flow capacity. Flow at the outlet would be an additional concern. When flow is concentrated, as it would be through the culverts, there is an increased potential for scour and sediment transport at the outlet.

3.4 Flood-Control Channels

Generally, flood-control channels have been developed as highly efficient, concrete-lined channels where right-of-way and real estate costs are high. The rectangular concrete channels around Rancho Mirage are examples. With lower right-of-way costs, larger trapezoidal-shaped channels of natural materials can be utilized, usually with lower construction costs. In alluvial fan areas, there is usually a problem with developing and maintaining unlined stable channels, The typical non-concrete channel will often require rock riprap bank protection, and possibly drop structures to maintain a flatter channel grade. While past design decisions were often based on the trade-off between construction cost and right-of-way and relocation costs, the increased concern over environmental and aesthetic values has expanded the considerations for project formulation. A third category of flood-control channels may be considered as multiple-purpose channels that provide wildlife habitat and/or support public recreational use of a portion of the flood-conveyance area. The Indian Bend Wash of Scottsdale, Arizona (Figure 3-7) is an example of a multiple-purpose development.

Because floods on alluvial fans tend to be heavily laden with sediment and debris, the question is whether to trap the sediment, usually in the headwater area, or transport the sediment through the channel. Heavy sediment loads and high-velocity flows are difficult to manage and often create extensive maintenance problems. Transported and deposited sediments also present significant maintenance problems for landscaped areas and park features in the flood-conveyance area.

Even concrete channels can be heavily damaged by the sediment transported during a flood event and, at the outflow point, there is the impact of the transported sediment on the downstream drainage system. Flood-control channels on alluvial fans must be designed with close attention given

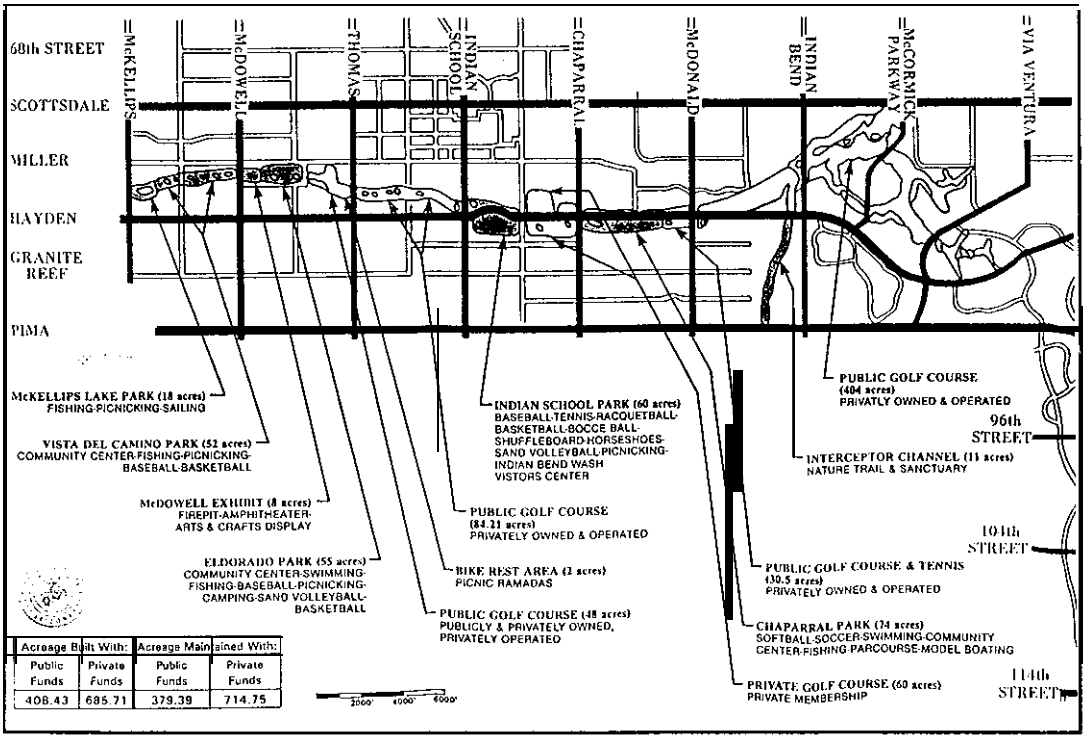


Figure 3-7 Indian Bend Wash Greenbelt Flood-Control Project Map, from the City of Scottsdale, Arizona

to the interception and storage or passing of sediment, particularly the coarser sizes moving at high velocities.

Rock riprap is often used to maintain alluvial channels, especially when high velocities are expected. Figure 3-8 shows a section of Tahquitz Creek with riprap bank protection. The series of flood reports from Southern California (Sections 2.1 to 2.3) describe several causes for riprap failure; however, two, (1) toe failure due to undermining or movement, and (2) riprap failure caused by impinging flow from lateral inflow or poor channel alignment, seem to be the predominant causes of failure.

Chapter 3 of EM 111 o-2-1601 provides design criteria for riprap protection. The USGS has developed a two-volume document, “Rock Riprap Design for Protection of Stream Channels Near Highway Structures” (USGS, 1986).

Grouted riprap has been routinely used in the Denver Urban Drainage and Flood-Control District (DUD&FCD, 1990), particularly in applications where high velocities and tractive forces could pull away the rock in a typical dumped riprap section.” The DUD&FCD Design Notes provides guidance on placing grouted riprap and boulders. Grouted riprap has also been used to protect diversion inlets, as shown in Figure 3-9, and drop structures at SCS and USACE projects.

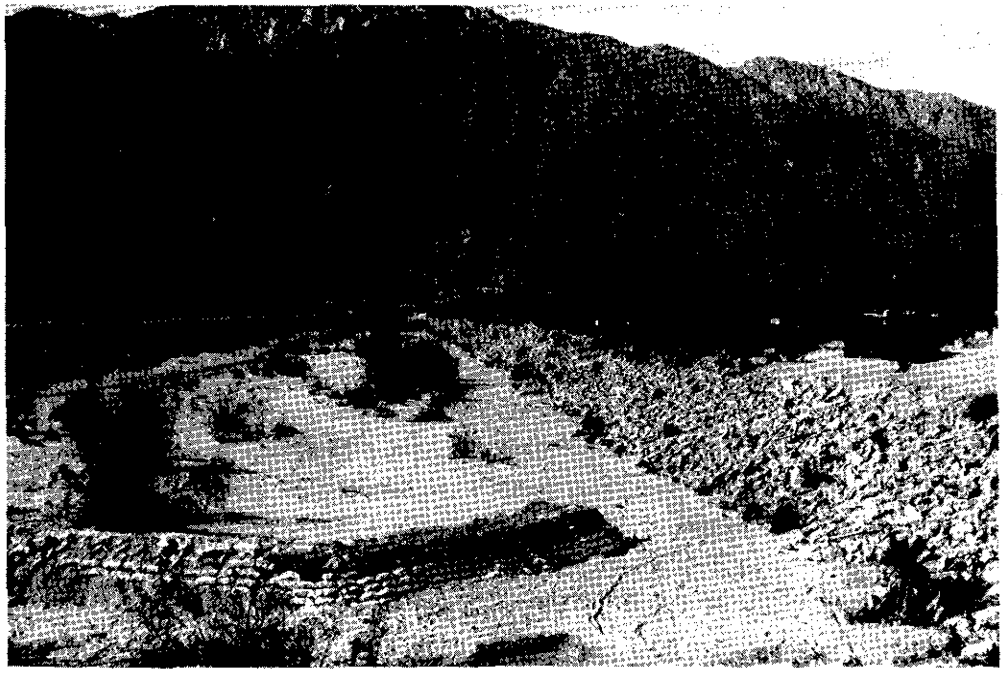
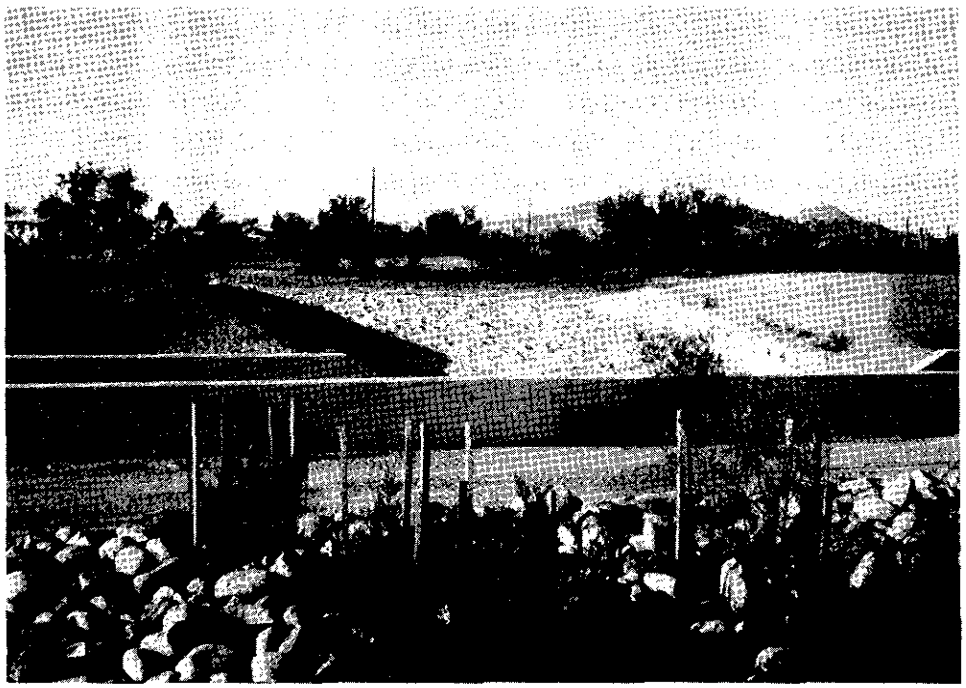


Figure 3-8 Tahquitz Creek, looking upstream from Sunrise Bridge, Palm Springs, California



gure 3-9 Grouted Riprap Inlet into Signal Butte Floodway, Mesa, Arizona

3.5 Diversions and Bypasses

The typical flood-reduction diversion is a side weir or chute that is designed to divert a portion of the river flood flow, while the main channel carries the majority of the flow. The primary effect is to “scalp” the peak flow from the flood wave as it passes the diversion location. The diverted flow could temporarily be detained in a storage area, or transported through an alternative conveyance path. For alluvial fans, the diversion may be used to capture flood flow, distributed across the fan, and to divert it to a location where it is non-damaging, or to transport it through a controlled channel system. The Saddle Back Diversion in Maricopa County, Arizona (Section 2.5) is an example of the capture-and-dived application. In this application, the diversion channel also intercepted alluvial fan flow along the channel. While the flood-retention structure and diversion channel functioned well, there were problems with sediment blocking the designed inflow points along the diversion channel. The planned Alamogordo Diversion Channel Project in New Mexico has sediment basins upstream from the diversion channel on most tributaries (USACE, 1987).

The Day Creek Flood-Control Project in San Bernardino County is comprised of an SPF detention basin, a concrete channel, and a side weir to divert part of the flow to a spreading basin located parallel to the channel. Flow that remains in the channel is carried downstream to a detention basin. Flow leaving the basin is discharged into an open spreading ground for groundwater recharge. The concrete channel in this project is designed to carry water and sediments at velocities ranging from 60 to 70 **ft/s.** Because no major events have occurred since the completion of the project, it remains to be seen whether the concrete and design can withstand such high velocities. It is not recommended that channels be designed to carry water and sediment at high velocities. There are no proven design criteria for these kinds of high-velocity channels. The flood experience at the Lowell Creek Diversion in Seward, Alaska (Section 2.6) demonstrates the erosive power of the rock and sediment transported through the concrete- and steel-lined tunnel.

The SCS Buckhorn Mesa project uses a combination of a dam across a portion of the alluvial fan and a diversion channel to transfer the intercepted flow into the flood retention structure, shown in Figure 3-10. The documented flood experience on this project occurred during construction; therefore, there has not been a significant test of the facility since construction was completed (SCS, 1984). The flood crossings over the Central Arizona Project are similar, with a long dam to intercept the fan flow and culverts to carry the flow across the canal, as shown earlier in Figure 3-6.

Figure 3-10 Signal Butte Flood Retention Structure, Mesa, Arizona

3.6 Muttipurpose Diversion Structures

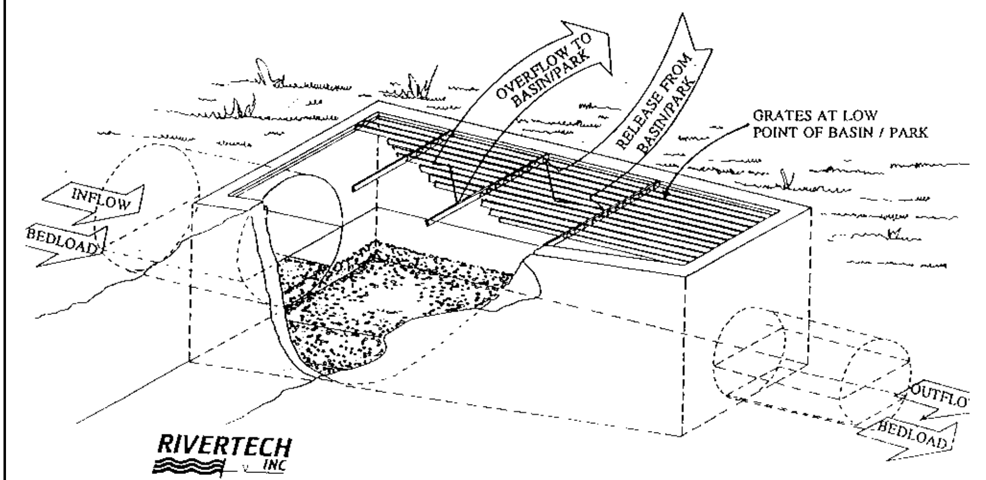
Nouri (1987) describes the design of cost-effective multipurpose diversion structures that provide flood detention during peak-flow periods for a specified range of recurrence intervals, while providing an open park-and-play area for the community the rest of the time. A special inlet and outlet design (Nouri, 1997) also allows a controlled amount of bed material load to be transported through the diversion pipe to the downstream channel in order to provide enough channel stabilizing sediment load into the downstream channel. Figure 3-11 shows a schematic drawing of the components of the overflow structure and how excess flows are released vertically into the detention basin (a park or play ground) while the bed load continues through the pipe outlet to the downstream channel.

The Nouri design has been implemented at two project sites in Orange County, California, where downstream channels are alluvial riparian streams. The goals of the design were to:

- Maintain downstream channel stability
- Reduce peak discharges along downstream reaches
- Use the detention area as a community park or golf course

Eleven multi-purpose basins employing overflow structures such as those shown in Figure 3-11 have been designed and constructed within the Newport Coast Community Development. Those basins are constructed within two 18-hole golf courses and parks. The basins not only have maintained downstream channel stability, but have also allowed the transport of sediment to the coast for beach sand replenishment. This was one of the design constraints because the beaches in the project area are deficient of sediment.

Another multi-purpose basin employing a similar overflow structure, as shown in Figure 3-11, has been in operation along Handy Creek in the City of Orange since 1986. The design has maintained the stability of Handy Creek, which was degrading prior to construction of the multi-purpose basin. Large debris can plug this type of detention structure, requiring maintenance; however, in areas where the likelihood of large debris accumulation is small, these types of basins appear to function well.



gure 3-11 Details of a Multi-purpose Diversion Structure

3.7 Levees and Floodwalls

Generally, the levees associated with the reviewed flood-control channels have been small portions of the improved flood-control channels. No floodwalls were reviewed during this investigation. Figure 3-12 shows a low concrete wall on the outside bend of West Magnesia Canyon Channel at Rancho Mirage. The channel is protected with gunite-rock riprap.

The flooding consequence for events exceeding a high levee or floodwall design capacity could be more catastrophic than the event would have been under non-leveed conditions due to the potential failure of the wall or levee releasing the high flood stage in a rapid-failure scenario. Instead of the area becoming gradually flooded as the flood wave passes down the channel system, a sudden release of flood flow would result from a levee or floodwall failure. There would be little time for evacuation and other emergency actions.

The issue of channel stability, described above, is applicable to levees because most levees are constructed from the same material as the channel.

gure 3-12 Low Concrete Wall on Outside Bend of Magnesia Canyon Channel, Rancho Mirage, California

Care must be taken to protect the levee faces and toes from erosion by the high-velocity, sediment-laden flows (Whitehouse & McSaveney, 1990). Rock riprap is frequently used for protecting the channel side of the levee; however, there may be strong local interest in a more aesthetically pleasing alternative when the stream runs through an urban environment. For high-velocity channels, concrete lining, soil cement, or grouted rock riprap, is often used to protect the channel and levee. With the low relative roughness of concrete and soil cement, a more efficient channel can be developed, which tends to reduce right-of-way costs.

Geotextiles (sometimes referred to as geofabrics) have been used to protect the levee banks against high-flow velocities. On alluvial fan areas where the soil is fine and where there is a scarcity of well graded riprap material, the geofabric prevents possible piping of the finer materials from the levee through the rock riprap.

3.8 Summary

There are few documented flood events for structural alluvial fan flood-control projects. This review was expanded to include existing projects in alluvial environments. While alluvial fans may be different under natural conditions, the performance (or problems) of flood-control measures on alluvial fans should be similar to other high-velocity alluvial streams. Every situation and project is unique in some way; however, some general conclusions can be drawn from the documented experience.

The traditional flood-reduction project study evaluates the trade-off between flood detention and more efficient flood conveyance. On alluvial fans, the problem expands to include storage and transport of sediment and debris. As described in Section 2.9, the basic structural choices are:

- Develop confined channels that transport the flood and debris safely through, or around, the development
- Develop storage above the development for debris and transport relatively clear water through, or around, the development; or
- A combination of the two.

Flood-Control Channels

Well-engineered and -constructed concrete flood-control channels have generally performed well, as examples in the Los Angeles basin floods show.

Small concrete drainage structures have been severely damaged during flood events (e.g., the Las Vegas flash flood described in Section 2.4).

Sediment deposits in channels, without debris basins, were found at tributaries, and at other locations where there was a decrease in energy slope. Any situation that will cause the inflowing water to slow will probably produce sediment deposition. A "rule of thumb" to consider is to never design a project that requires the flows to change direction or change velocity quickly in a short distance. Sediment erosion, deposition, channel avulsion, and possible structural failure may result from attempting to force flows to change their course abruptly.

Problems with alluvial flood-control channels, that were not overtopped, were generally of two types: (1) destruction of riprap protection, with subsequent project feature damage, and (2) sediment deposits decreasing flow capacity. Riprap and other forms of bank protection tended to fail at the edges (i.e., at inlet, outlet, or toe protection). Failure of the toe protection due to excessive scour occurred in several projects. Riprap failures also occurred due to impinging flow from tributaries, or from flow meander within the channel. Similar problems also occurred on levee protection.

In an overview of stability problems with flood-control channels (USACE, 1990), two major potential stability problems were identified:

- When depths are increased but the original slope is maintained, "velocities at the higher discharges will be increased and the bed and banks may erode, especially if bank stability previously depended on cohesive sediment deposits, armoring or vegetation that was removed in the enlargement process."

- When the cross-section enlargement is too large, and there is a substantial sediment transport load, "the cross section may partly infill with sediment deposits and the calculated flood capacity may not be achieved without maintenance."

Levees may also cause channel sedimentation in streams with high sediment loads by restricting transport and deposition of sand on the overbank areas. More sand is then retained in the channel to deposit further downstream in reaches of flatter slope. This may initiate a progressive upstream-advancing aggradation of the bed. Also, thick deposition of finer suspended sediment on the berm between the river bank and the levee (occurring mainly during flood recessions) may overload the bank to cause slump failures," (ibid, p. 3-5).

Diversions

Sediment deposits were also identified as a problem at the inlets of diversions and interceptor channels in cases where there was "ponding" at the inlet. Again, the decrease in energy slope allowed sediment to deposit to a depth that obstructed flow. Also, the interceptor channel at Saddleback (Section 2.5) had inlets located based on apparent flow paths on the fan. However, there was shifting of the flow paths during the reported flood event, and the new flow paths did not align with the inlets constructed along the interceptor channel.

Stability problems on diversions can be difficult to predict, and depend on the method used to divert the flow. "Sedimentation may occur in the main channel, in the diversion, or in both, as the sediment-carrying capacity of both is likely to be less than that of the existing channel. The division of sediment between the two channels is not necessarily proportional to the division of flow. Further sedimentation problems may arise if there are substantial downstream inflows of sediment that the reduced flows are unable to transport," (USACE, 1990).

When flood flows are diverted into a channel, "but the channel is not deliberately modified to accommodate the increased discharges, serious erosional problems may ensue. The channel tends to respond by widening and deepening, and by flattening slope through upstream degradation and downstream gradation," (ibid, p. 3-9).

Detention Storage

In general, sediment and debris basins have worked well. The critical factors are size, outlet works, and downstream protection. The bigger the storage capacity, the better the chance for the project to perform well (not be exceeded) during a flood event, Even those projects that did fill were considered successful by the reviewers because they prevented the stored volume from moving downstream. If a debris basin is effective, the downstream channel must be protected from the relatively clearer outflow. Also, channel protection is usually required in the vicinity of the overflow spillway.

Debris Barriers

Debris barriers have also been effective in keeping some of the debris from moving downstream, as shown by their extensive use in Southern California. While they are not considered effective for flood control, they should be considered as a component of a total plan. Even when full, they have been credited for reducing the stream slope and thus reducing the rate of debris movement downstream. Localized debris barriers have also been proposed for protection of subdivisions from mudflow in Colorado (Mears, 1977).

Analysis Problem

The Base Flood for determining flood-prone areas is the 1 -percent-chance flood. The problem is to define the flood (including possible flow bulking) and evaluate the performance of the flood-reduction measures to determine whether an area is protected from flooding by this design event.

While the analysis of flood hydrology is difficult due to a lack of recorded data in many alluvial fan areas, the prediction of sediment yield and transport is extremely difficult. The analytical methods used are highly dependent on regional data and experience. Additionally, the analysis must model the flow and debris movement through the area to be developed. The natural conveyance channels are often unstable, and there is considerable uncertainty in the prediction of the size and location of the channel during and after a flood event. Channel avulsions are common during large events.

Every factor affecting the nature of flood and debris problems, plus the development and its susceptibility to flooding, affect the feasibility of flood-reduction options, There is no "cookbook" approach to developing an effective flood-reduction project. Planning and design of flood-control structures on alluvial fans must always consider the effect *of all possible f/ows on the structure as we// as the effects the structure may have on the flow /oca//y and downstream.* While the FIA criteria are based on the i-percent-chance flood, the proper design of any flood-reduction project must consider project performance for the entire range of floods, including floods larger than the Base Flood.

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Appendix A

GLOSSARY

Appendix A. GLOSSARY

AGGHADATION The geologic process by which stream beds, floodplains, and the bottoms of other water bodies are raised in elevation by the deposition of material eroded and transported from other areas. It is the opposite of degradation,

ALGORITHM A procedure for solving a mathematical problem in a finite number of steps that frequently involves repetition of an operation. A step by step procedure for solving a problem or accomplishing an end. A set of numerical steps or routines to obtain a numerical output from a numerical input.

ALUVIAL CHANNEL OR STREAM A stream which flows within bed and banks that are composed of appreciable quantities of the material which it transports. Such streams are sometimes referred to as “authors of their own geometry” because of the interaction between the stream’s flow characteristics (hydraulic and hydrologic) and its shape, both plan form and cross sectional.

ALLUVIAL DEPOSfT Clay, siit, sand, gravel, or other sediment deposited by the action of running or receding water.

ALLUVIAL FAN A conical, or fan shaped deposit at the base of a mountain range where the mountain stream encounters the lesser slope of the valley floor. The deposits are generally coarse and alluvial fans most often occur in arid and semi-arid regions where streamflow is ephemeral and vegetation cover sparse.

ALLUVIAL FAN FLOODING Alluvial fan flooding is flooding that occurs on the surface of an alluvial fan or similar landform which originates at the “apex” and is characterized by high-velocity flows; active processes of erosion, sediment transport, and deposition: and unpredictable’flow paths. Some of the flood hazards associated with alluvial fan flooding are flash flooding, unpredictable flow paths, and a high velocity of flow coupled with the material of the landforms being highly susceptible to erosion.

ALLUVIUM A general term for all detrital deposits resulting directly or indirectly from the sediment transported by (modern) streams, thus including the sediments laid down in river beds, floodplains, lakes, fans, and estuaries.

ANCHOR A series of bolts or metal straps used to secure a structure to its footings or foundation wall so that it will not be displaced by flood or wind forces.

APEX A point on an alluvial fan or similar landform below which the flow-path of the major streams that formed the fan becomes unpredictable and alluvial fan flooding occur.

ARMORING The process of progressive coarsening of the bed layer by removal of fine particles until it becomes resistant to scour. The coarse layer that remains on the surface is termed the "armor layer"'. Armoring is a temporary condition; higher flows may destroy an armor layer and it may re-form as flows decrease.

AVULSION A rapid change in channel direction and form that occurs during catastrophic, rare floods.

A-ZONE **See Special Flood Hazard Area.**

BACKFLOW VALVE **See Check Valve**

BANK MIGRATION Lateral or horizontal movement of the banks of a streamcourse.

BASE FLOOD EVALUATION (BFE) The evaluation for which there is a l-percent chance in any given year that flood levels will equal or exceed it. The BFE is determined by statistical analysis for each local area and designated on the Flood Insurance Rate Maps. It is also known as the 1 **00-year Flood**

BED FORMS Irregularities found on the bottom (bed) of a stream that are related to flow characteristics. They are given names such as "dunes", "ripples", and "antidunes". They are related to the transport of sediment and interact with the flow because they change the roughness of the stream bed. An analog to stream bed forms are desert sand dunes (although the physical mechanisms for their creation and movement may be different).

BED LOAD Material moving on or near the stream bed by rolling, sliding, and sometimes making brief excursions into the flow a few diameters above the bed, i.e. jumping. The term "saltation" is sometimes used in place of "jumping". Bed load is bed material that moves in continuous contact with the bed; contrast with SUSPENDED LOAD.

BED LOAD DISCHARGE The quantity of bed load passing a cross section in a unit of time, i.e. the rate. Usually presented in units of tons per day. May be measured or computed. See BED LOAD.

BED MATERIAL The sediment mixture of which the moving bed is composed.

BED MATERIAL DISCHARGE OR LOAD The total rate (tons/day) at which bed material is transported by a given flow at a given location on a stream. It consists bed material moving as both bed load and suspended load.

BED ROCK A general term for the rock, usually solid, that underlies soil or other unconsolidated, surficial material.

BERM A bank or mound of earth, usually placed against a foundation wall.

BORROW AREA An area where material has been excavated for use as fill at another location.

BOUNDARY ROUGHNESS The roughness of the bed and banks of a stream or river. The greater the roughness, the greater the frictional resistance to flows; and, hence, the greater the water surface elevation for any given discharge.

BRAIDED CHANNEL A stream that is characterized by relatively shallow interlaced channels divided by islands or bars. Bars which divide the stream into separate channels at low flows are often submerged at high flow.

BREAKAWAY WALLS Walls enclosing the are below an elevated structure that are designed to break away before transmitting damaging forces to the structure and its foundation. Breakaway walls are required by NFIP regulations in coastal high-hazard areas (V-Zones) and are recommended in areas where flood waters could flow at significant velocities (usually greater than four feet per second) or could contain ice or other debris.

BUILDING CODE Regulations adopted by local governments that establish standards for construction, modification, and repair of buildings and other structures.

CAULKING Material used to fill joints in a structure, such as around windows or doors.

CHANNEL A natural or artificial waterway which periodically or continuously contains moving water.

CHANNEL STABILIZATION A stable channel is neither progressively aggrading nor degrading, or changing its cross sectional area through time. It could aggrade or degrade slightly, but over the period of a year, the channel would remain similar in shape and dimensions and position to previous times. Unstable channels are depositing or eroding in response to some exterior conditions. Stabilization techniques consist of bank protection and other measures that work to transform an unstable channel into a stable one.

CHECK VALVE A type of valve that allows water to flow one way, but automatically closes when water attempts to flow the opposite direction.

CLAY See TABLE A-l,

CLOSURE A shield made of strong material, such as steel, aluminum or plywood, used to temporarily fill in gaps in floodwalls, levees, or sealed structures that have been left open for day-to-day convenience at entrances such as doors and driveways.

COASTAL HIGH-HAZARD AREA Designated as V-Zone on Flood Insurance Rate Maps, this is that portion of the coastal floodplain subject to storm driven velocity waves of three feet or more in height.

COBBLES See TABLE A-1.

COHESIVE SEDIMENTS Sediments whose resistance to initial movement or erosion is affected mostly by cohesive (chemical) bonds between particles.

COLUMN Upright support units for a building set in predug holes and backfilled with compacted material. Columns will often require bracing in order to provide adequate support. They are also known as posts, although they are usually of concrete or masonry construction.

CONCENTRATION OF SEDIMENT The dry weight of sediment per unit volume of water-sediment mixture, i.e. mg/l. (Note: In earlier writings, concentration was calculated as the ratio of the dry weight of sediment in a water-sediment mixture to the total weight of the mixture multiplied by l,000,000. It was expressed as parts per million, i.e. ppm. Either method gives the same result, within 1 percent, for concentrations up to 16,000 mg/l. A correction is needed for concentrations in excess of that value.) The conversion to mg/l (milligrams per liter) from ppm (parts per million) is as follows:

$$mg/l = K x\ (ppm) = K x\ \frac{\text{weight of sediment} \times 1{,}000{,}000}{\text{weight of water-sediment mixture}}$$

where K is a correction factor.

CONCRETE MASONRY UNIT (CMU) Blocks of concrete used in construction.

CONSOUDATION The compaction of deposited sediments caused by grain reorientation and by the squeezing out of water trapped in the pores.

CONVEVANCE A measure of the carrying capacity of the channel section. Flow is directly proportional to conveyance for steady flow. From Manning's equation, the proportionality factor is the square root of the energy slope.

CRAWL SPACE Low space below the first floor of a house, where there has not been excavation deep enough for a basement, but where there is often access for pipes, ducts, and utilities.

CRITICAL DEPTH If discharge is held constant and the water depth allowed to decrease, as in the case of water approaching a free overfall, velocity head will increase, pressure head will decrease, and total energy will decrease toward a minimum value where the rate of decrease in the pressure head is just counterbalanced by the rate of increase in velocity head, This is the critical depth. More generally, the critical depth is the depth of flow that would produce the minimum total energy head, and it depends on cross section geometry and water discharge.

CRITICAL FLOW The state of flow where the water depth is at the critical depth and when the inertial and gravitational forces are equal.

CRITICAL SHEAR OR TRACTNE FORCE The critical tractive force is the maximum shear force on the bed that will not cause movement of the material forming the channel bed on a level surface.

CROSS SECTION Depicts the shape of the channel in which a stream flows. Measured by surveying the stream bed elevation across the stream on a line perpendicular to the flow. Necessary data for the computation of hydraulic and sediment transport information.

DEBRIS IMPACT LOADS Sudden loads induced on a structure by debris carried by flood water, Though difficult to predict, allowances for impact loads must be made when floodproofing a structure.

DEGRADATION The geologic process by which stream beds, floodplains, and the bottoms of other water bodies are lowered in elevation by the removal of material from the boundary. It is the opposite of aggradation.

DEPOSITION Raising of the stream bed by settlement of moving sediment that may be due to local changes in the flow, or during a single flood event.

DEBRIS FLOW A mass movement of large size material such as boulders with little water visible. They are characterized by a steep front several feet high and typically move in surges down an alluvial fan.

DISTRIBUTARIES Diverging streams which do not return to the main stream, but discharge into another stream or the ocean.

DOMINANT DISCHARGE A particular magnitude of flow which is sometimes referred to as the "channel forming" discharge. Empirical relations have been developed between "equilibrium" stream width, depth, and slope and dominant discharge. It has been variously defined as the bank full flow, mean annual discharge, etc.

DRY FLOODPROOFING A floodproofing method used in areas of low level flooding to completely seal a home against water.

ELEVATION The raising of a structure to place it above flood waters on an extended support structure.

ENTRAINMENT The carrying away of the material produced by erosive action from bed and banks.

EPHEMERAL Existing or continuing for a short time; transitory or temporary.

EQUIUBRIUM LOAD The amount of sediment that a system can carry for a given discharge without an overall accumulation (deposit) or scour (degradation).

EROSION The wearing away of the land surface by detachment and movement of soil and rock fragments through the action of moving water and other geological agents.

EXISTING CONSTRUCTION The structures already existing or under construction prior to the effective date of a community's floodplain management regulations.

FALL VELOCITY The falling or settling rate of a particle in a given medium.

FEDERAL EMERGENCY MANAGEMENT AGENCY (FEMA) This agency was created in 1978 to provide a single point of accountability for all federal activities related to disaster mitigation and emergency preparedness and response.

FEDERAL INSURANCE ADMINISTRATION (FIA) The governmental unit, a part of the Federal Emergency Management Agency, that administers the National Flood Insurance Program.

FILL Materials such as earth, clay, or crushed stone which is dumped in an area and compacted to increase ground elevation.

FLASH FLOOD A flood that crests in a short length of time and is often characterized by high velocity flow. It is often the result of heavy rainfall in a localized area.

FLOOD (For NFIP flood insurance policies) A partial or complete inundation of normally dry land areas from 1) the overland flood of a lake, river, stream, ditch, etc; 2) the unusual and rapid accumulation of runoff of surface waters; and 3) mudflows or the sudden collapse of shoreline land.

FLOOD FRINGE That portion of the floodplain that lies beyond the floodway and serves as a temporary storage area for flood waters during a flood. This section receives waters that are shallower and of lower velocities than those of the floodway.

FLOOD HAZARD BOUNDARY MAP (FHBM) The official map of a community that shows the boundaries of the floodplain and special flood hazard areas that have been designated. It is prepared by FEMA using the best flood data available at the time a community enters the emergency phase of the NFIP. It is superseded by the FIRM after a more detailed study has been completed.

FLOOD INSURANCE RATE MAP (FIRM) The official map of a community prepared by FEMA that shows the Base Flood Elevation, along with the special hazard areas and the risk premium zones for flood insurance purposes. Once it has been accepted, the community is part of the regular phase of the NFIP.

FLOOD INSURANCE STUDY (FIS) A study performed by any of a variety of agencies and consultants to delineate the special flood hazard areas, base flood elevations and risk premium zones. The study is funded by FEMA and is based on detailed site surveys and analysis of the site-specific hydrologic characteristics.

FLOODPWN Normally dry land adjacent to a body of water, such as a river, stream, lake, or ocean, which is susceptible to inundation by floodwaters.

FLOODPWN MANAGEMENT A program of corrective and preventive measures for reducing flood damage, including but not limited to flood control projects, floodplain land use regulations, floodproofing or retrofitting of buildings, and emergency preparedness plans.

FLOODPROOFING Any combination of measures taken on a new or existing structure for reducing or eliminating flood damages.

FLOOD ROUTING The process of tracing, by calculation, the course and character of a flood as it progresses through a river reach or a reservoir.

FLOODWALL A constructed barrier of resistant material, such as concrete or masonry block, designed to keep water away from a structure.

FLOODWAY The central portion of the floodplain that carries the greatest portion of the waterflow in a flood. Obstructions in the floodway will result in increased flood levels upstream.

FLOW DURATION CURVE A measure of the range and variability of a stream's flow. The flow duration curve represents the percent of time during which specified flow rates are exceeded at a given location. This is usually presented as a graph of flow rate (discharge) vs. percent of time that flows are greater than, or equal to, that flow.

FOOTING The enlarged base of a foundation, wall, pier, or column, designed to spread the load of the structure so that it does not exceed the soil bearing capacity.

FOUNDATION The underlying structure of a building, usually constructed of concrete, that supports the foundation walls, piers, or columns,

FOUNDATION WALLS A support structure that connects the foundation to the main portion of the building, or superstructure.

FREEBOARD An additional amount of height used as a factor of safety in determining the design height of a floodproofing or retrofitting method to compensate for unknown factors such as wave action. Certain guidelines and restrictions apply for establishing freeboard on levees and floodwalls in NFIP areas.

FREQUENCY The number of repetitions of a periodic process in a certain time period.

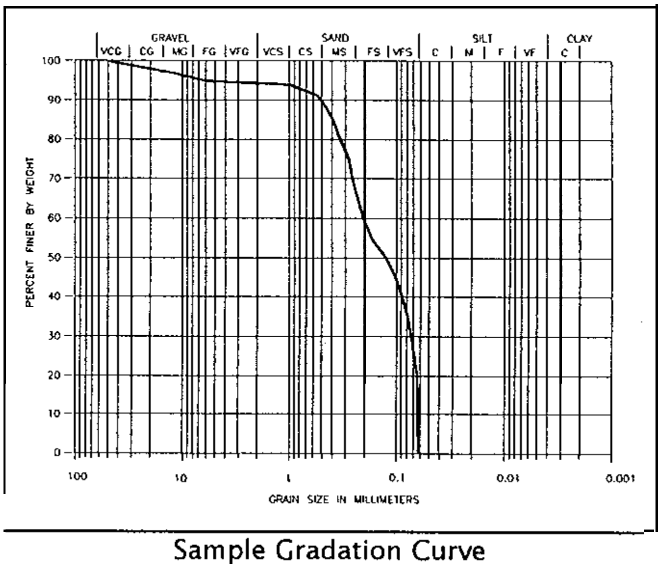
GEOLOGIC CONTROL A local rock formation or clay layer that limits (within the engineering time frame) the vertical and/or lateral movement of a stream at a particular point. Note that man-made controls such as drop structures also exist.

GEOLOGY A science that deals with the history of the earth and its life, especially as recorded in rocks.

GEOMORPHOLOGY The study of landform development under processes associated with running water.

GRADATION The proportion of material of each particle size, or the frequency distribution of various sizes, constituting a particulate material such as a soil, sediment, or sedimentary rock. The limits of each size are chosen arbitrarily. Four different gradations are significant: the gradation of the suspended load, the gradation of the bed load, the gradation of the material comprising the bed surface, and the gradation of material beneath the bed surface.

GRADATION CURVE Sediment samples usually contain a range of grain sizes, and it is customary to break this range into classes of percentages of the total sample weight contained in each class. After the individual percentages are accumulated, a graph, the "gradation curve", shows the grain size vs. the accumulated percent of material that is finer than that grain size. These curves (see example above) are used by movable boundary models to depict the bed sediment material properties (e.g., grain size distribution of the bed material).



Sample Gradation Curve

GRAIN SHAPE FACTOR See PARTICLE SHAPE FACTOR.

GRAIN **SIZE** See PARTICLE SIZE.

GRAIN SIZE DISTRIBUTION (GRADATION) A measure of the variation in grain (particle) sizes within a mixture. Usually presented as a graph of grain diameter vs. percent of the mixture that is finer than that diameter.

GRAVEL See TABLE A-i.

HUMAN INTERVENTION The required presence and active involvement of people to enact any type of floodproofing or retrofitting measure prior to flooding.

HYDRAULIC MODEL A physical scale model of a river used for engineering studies.

HYDRAULICS The study and computation of the characteristics, e.g. depth (water surface elevation), velocity and slope, of water flowing in a stream or river.

HYDRODYNAMIC LOADS Forces imposed on an object, such as a structure, by water moving around it. Among these loads are positive frontal pressure, against the structure; drag effect, along the sides; and negative pressure on the downstream side.

HYDROGRAPH A graph showing, for a given point on a stream or conduit, the discharge,water surface elevation, stage, velocity, available power, or other property of water with respect to time.

HYDROLOGY The study of the properties, distribution, and circulation of water on the surface of the land, in the soil, and in the atmosphere.

HYDROSTATIC LOADS Forces imposed on a surface, such as a wall or floor slab, by a standing mass of water. The water pressure increases with the square of the water depth.

INCIPIENT MOTION The flow condition at which a given size bed particle just begins to move. Usually related to a “threshold” shear stress.

INFLOWING LOAD CURVE See SEDIMENT RATING CURVE.

IN SITU In (its original) place.

INTERIOR GRADE BEAM A section of a floor slab that has a thicker section of concrete to act as a footing to provide stability under load-bearing or critical structural walls.

LEVEE A barrier of compacted soil designed to keep flood water away from a structure.

LIFT A layer of soil that is compacted before the next layer is added in the construction of a fill pad or levee.

LOCAL SCOUR Erosion caused by an abrupt change in flow direction or velocity. Examples include erosion around bridge piers, downstream of stilling basins, at the ends of dikes, and near snags.

MANNING'S EQUATION The empirical Manning's equation commonly applied in water surface profile calculations defines the relationship between surface roughness, discharge, flow geometry, and rate of friction loss for a given stream location.

MANNING'S n-VALUE The coefficient of roughness with the dimensions of T x $L^{-1/3}$. It accounts for energy loss due to the friction between the bed and the water. In fluvial hydraulics (movable boundary hydraulics), the Manning's n value includes the effects of all losses, such as grain roughness of the movable bed, form roughness of the bed, bank irregularities, vegetation, bend losses, and junction losses. Contraction and expansion losses are not included in Manning's n, but are typically accounted for separately.

MATHEMATICAL MODEL A model that uses mathematical expressions (i.e., a set of equations, usually based upon fundamental physical principles) to represent a physical process.

MEANDERING STREAM An alluvial stream characterized in planform by a series of pronounced alternating bends. The shape and existence of the bends in a meandering stream are a result of alluvial processes and not determined by the nature of the terrain (geology) through which the stream flows.

MEAN SEA **LEVEL** The average height of the sea for all stages of the tide, usually determined from hourly height observations over a 19-year period on an open coast or in adjacent waters having free access to the sea.

MITIGATION To make restitution for adverse project impacts.

MODEL A representation of a physical process or thing that can be used to predict the process's or thing's behavior or state.

Examples:

A conceptual model: If I throw a rock harder, it will go faster.
A mathematical model: F=ma
A hydraulic model: Columbia River physical model.

MOVABLE BED MODEL Model in which the bed and/or side material is erodible and transported in a manner similar to the prototype.

MUD FLOW Debris laden water originating on a steep slope carrying such large concentrations of sediment, particularly sands and finer sizes, that it forms a fluid much denser than water and is capable of transporting boulders which are buoyed up by the viscous flow.

NATIONAL FLOOD INSURANCE PROGRAM (NFIP) The federal program, created by an act of Congress in 1968, that makes flood insurance available in communities that enact satisfactory floodplain management regulations.

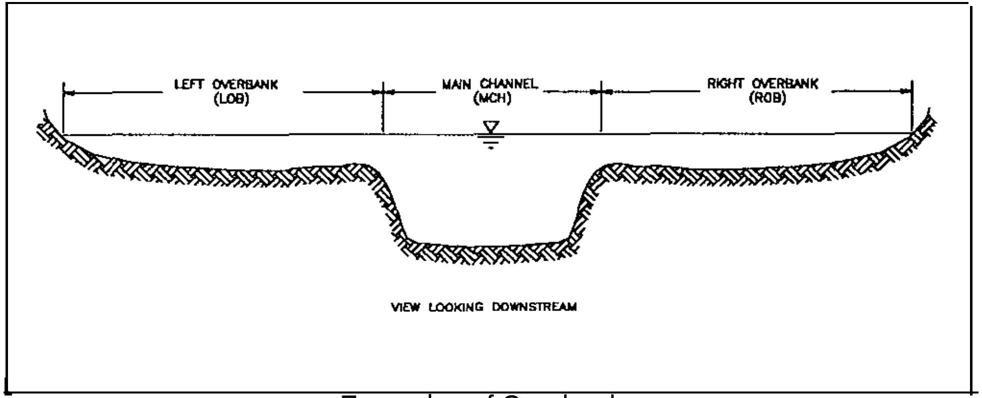
NORMAL DEPTH The depth that would exist if the flow were uniform is called normal depth.

NUMERICAL MODEL A numerical model is the representation of a mathematical model as a sequence of instructions (program) for a computer. Given approximate data, the execution of this sequence of instructions yields an approximate solution to the set of equations that comprise the mathematical model.

ONE-DIMENSIONAL ENERGY EQUATION This equation has the same form as the Bernoulli Equation and the same terms are present. In addition, an α term has been added to correct for velocity distribution.

ONE HUNDRED (100) YEAR FLOOD The flood elevation that has **a** i-percent chance of being equal or exceeded in any given year. It is also known as the Base Flood elevation. **(See Base Flood Elevation.)**

OVERBANK In a river reach, the surface area between the bank on the main channel and the limits of the floodplain. See figure below:



Examples of Overbanks

PARAMETER Any set of physical properties whose values determine the characteristics or behavior of something.

PARTICLE SHAPE FACTOR The particle shape factor of a perfect sphere is 1.0 and can be as low as 0.1 for very irregular shapes. It is defined by:

$$SF = \frac{c}{\sqrt{(a \times b)}}$$

where:

a,b,c = the lengths of the longest, intermediate, and shortest, respectively, mutually perpendicular axes on a sediment particle.

PARTICLE SIZE A linear dimension usually designated as "diameter", used to characterize the size of a panicle. The dimension may be determined by any of several different techniques, including sedimentation sieving, micrometric measurement, or direct measurement.

PERMEABILITY The property of a soil that permits the passage of water under a gradient of force.

PIER An upright support member of a building, with a height limited to a maximum of three times its least lateral dimension. It is designed and constructed to function as an independent structural element in supporting and transmitting building and environmental loads to the ground.

PILE An upright support member of a building, usually long and slender in shape, driven into the ground by mechanical means and primarily supported by friction between the pile and the surrounding earth. Piles often cannot act as individual units, and require bracing to other pilings.

PLANFORM The shape and size of channel and overbank features as viewed from directly above.

POST Long upright support units for a building, set in predug holes and backfilled with compacted material. Each post usually requires bracing to other units. They are also known as columns, although they are usually made of wood.

PROTOTYPE The full-sized structure, system process, or phenomenon being modeled.

QUALITATIVE Relating to or involving quality or kind.

QUANTITATIVE A specific measurement of a quantity or amount.

RATING CURVE See STAGE-DISCHARGE CURVE.

REACH (1) The length of a channel, uniform with respect to discharge, depth, area, and slope, e.g., “study reach’, ‘typical channel reach” or “degrading reach”, etc. (2) The length of a stream between two specified gaging stations.

REGULATORY FLOODWAY As referenced in a floodplain management ordinance, this is the portion of the floodplain needed to discharge the lOO-year flood without increasing the flood elevation by more than a designated height, usually one foot.

RELOCATION The moving of a structure from a flood area to a new location, normally to one where there is no threat of flooding.

REPLICATE To duplicate (a statistical experiment).

RETROFITTING Floodproofing measures taken on an existing structure.

RETROFLOODPROOFING See Retrofitting

RIPPLE Small triangular-shaped bed forms that are similar to dunes but have much smaller heights and lengths of 0.3 m or less. They develop when the Froude number is less than approximately 0.3.

RIPRAP Broken stone, cut stone blocks, or rubble that is placed on slopes to protect the from erosion or scouring caused by flood waters or wave action.

ROUTING MODEL A model (see MATHEMATICAL MODEL and NUMERICAL MODEL) for performing flood routing (see FLOOD ROUTING).

SAND See TABLE A-l.

SATURATION The degree to which voids in soil are filled with water.

SCOUR The enlargement of a flow section by the removal of boundary material through the action of the fluid in motion.

SECONDARY CURRENTS (OR **FLOW)** The movement of water particles on a cross section normal to the longitudinal direction of the channel.

SEDIMENT (1) Particles derived from rocks or biological materials that have been transported by a fluid. (2) Solid material (sludges) suspended in or settled from water. A collective term meaning an accumulation of soil, rock and mineral particles transported or deposited by flowing water.

SEDIMENTATION **A broad** term that pertains to the five fundamental process responsible for the formation of sedimentary rocks: (1) weathering, (2) detachment, (3) transportation, (4) deposition (sedimentation), and (5) diagenesis; and to the gravitational settling of suspended particles that are heavier than water.

SEDIMENTATION DIAMETER The diameter of a sphere of the same specific weight and the same terminal settling velocity as the given particle in the same fluid.

SEDIMENT DISCHARGE The mass or volume of sediment (usually mass) passing a stream cross section in a unit of time. The term may be qualified, for example; as suspended-sediment discharge, bed load discharge, or total-sediment discharge. See SEDIMENT LOAD.

SEDIMENT LOAD A general term that refers to material in suspension and/or in transport. It is not synonymous with either discharge or concentration, It may also refer to a particular type of load; e.g. total, suspended, wash, bed, or material.

SEDIMENT PARTICLE Fragments of mineral or organic material in either a singular or aggregate state.

SEDIMENT RATING TABLES Tables which relate inflowing sediment loads to water discharge for the upstream ends of the main stem, tributaries, and local inflow points.

SEDIMENT TRANSPORT (RATE) See SEDIMENT DISCHARGE.

SEDIMENT TRANSPORT FUNCTION A formula or algorithm for calculating the sediment transport rate given the hydraulics and bed material at a cross section. Most sediment transport functions compute the bed material load capacity. The actual transport may be less than the computed capacity due to armoring, geologic controls, etc.

SEDIMENT TRANSPORT ROUTING The computation of sediment movement for a selected length of stream (reach) for a period of time with varying flows. Application of sediment continuity relations allow the computation of aggradation and deposition as functions of time.

SEDIMENT TRAP EFFICIENCY See TRAP EFFICIENCY.

SETTLING VELOCITY See FALL VELOCITY.

SHAPE FACTOR See PARTICLE SHAPE FACTOR

SHEAR INTENSITY A dimensionless number that is taken from Einstein's bed load function. It is the inverse of Shield's parameter.

SHEAR **STRESS** Frictional force per unit of bed area exerted on the bed by the flowing water. An important factor in the movement of bed material.

SHIELD'S DETERMINISTIC CURVE A curve of the dimensionless tractive force plotted against the grain Reynolds number. For example,

$$U_* \cdot D_s/v$$

where:

U_*	=	turbulent shear velocity
D_s	=	characteristic or effective size of the grains or roughness elements
v	=	kinematic viscosity) and which is used to help determine the CRITICAL TRACTIVE FORCE.

SHIELD'S PARAMETER A dimensionless number referred to as a dimensionless shear stress. The beginning of motion of bed material is a function of this dimensionless number.

$$\frac{\tau_c}{(\gamma_s - \gamma)\, D_s}$$

where:

τ_c	=	critical tractive force
γ_s	=	specific weight of the particle
γ	=	specific weight of water
D_s	=	characteristic or effective size of the grains or roughness elements

SIEVE DIAMETER The smallest standard sieve opening size through which a given particle of sediment will pass.

SILT See TABLE A-l.

SILTATION An unacceptable term. Use sediment deposition, sediment discharge, or sediment yield as appropriate.

SIMULATE To express a physical system in mathematical terms,

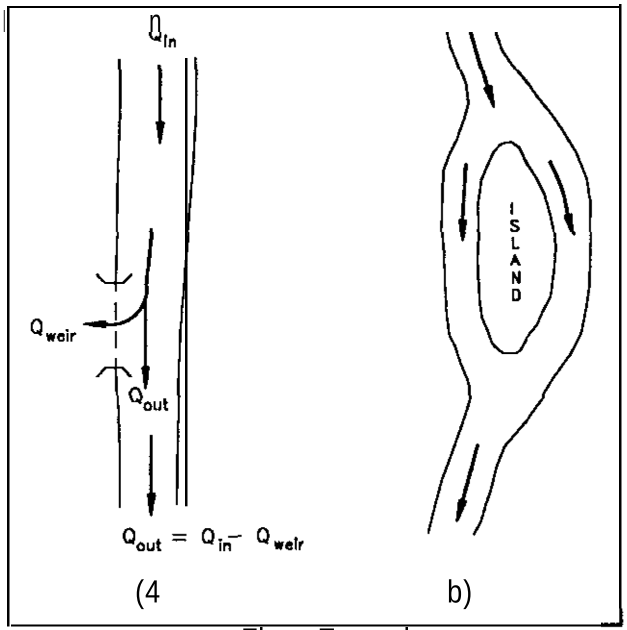
SINUOSITY A measure of meander "intensity". Computed as the ratio of the length of a stream measured along its thalweg (or centerline) to the length of the valley through which the stream flows.

SLAB ON GRADE A structural design where the first floor sits directly on a poured concrete slab which sits directly on the ground.

SORTING The dynamic process by which sedimentary particles having some particular characteristic (such as similarity of size, shape, or specific gravity) are naturally selected and separated from associated but dissimilar particles by the agents of transportation. Also, see GRADATION.

SPECIAL FLOOD HAZARD AREA Portion of the floodplain subject to the l00-year flood, also known as the A-Zone. In coastal regions, this area is subject to velocity wave action of less than three feet.

SPLIT FLOW Flow that leaves the main river flow and takes a completely different path from the main river [Case (a)]. Split flow can also occur in the case of flow bifurcation around an island [Case WI.



Split Flow Examples

STABLE CHANNEL A stream channel that does not change in platform or bed profile during a particular period of time. For purposes of this glossary the time period is years to tens of years.

STAGE-DISCHARGE (RATING) CURVE Defines a relationship between discharge and water surface elevation at a given location.

STEADY STATE MODEL Model in which the variables being investigated do not change with time.

STILE A set of stairs to allow access over an obstruction, such as a floodwall.

STREAM GAGE A device that measures and records flow characteristics such as water discharge and water surface elevation at a specific location on a stream. Sediment transport measurements are usually made at stream gage sites.

STREAM POWER The product of bed shear stress and mean cross-sectional velocity at a cross section for a given flow.

STREAM PROFILE A plot of the elevation of a stream bed vs. distance along the stream.

STRUCTURAL MAT SLAB The concrete slab of a building which includes structural reinforcement to help support the building's structure.

SUBCRITICAL FLOW The state of flow where the water depth is above the critical depth, Here, the influence of gravity forces dominate the influences of inertial forces, and flow, having a low velocity, is often described as tranquil.

SUBSTANTML IMPROVEMENT Any repair, reconstruction, or improvement of a structure, the cost of which equals or exceeds 50 percent of the market value of the structure either: a)before the improvement is started, or b) if the structure has been damaged and is being restored, before the damage occurred.

SUPERCRITICAL FLOW The state of flow where the water depth is below the critical depth, inertial forces dominate the gravitational forces, and the flow is described as rapid or shooting.

SUSPENDED BED MATERIAL LOAD That portion of the suspended load that is composed of particle sizes found in the bed material.

SUSPENDED LOAD Includes both suspended bed material load and wash load. Sediment that moves in suspension is continuously supported in the water column by fluid turbulence. Contrast with BED LOAD.

SUSPENDED-SEDIMENT DISCHARGE The quantity of suspended sediment passing a cross section in a unit of time usually given in tons/day. See SUSPENDED LOAD.

TAIL WATER The water surface elevation downstream from a structure, such as below a dam, weir or drop structure.

THALWEG The line following the lowest part of a valley, whether under water or not. Usually the line following the deepest part or middle of the bed or channel of a river.

TOTAL SEDIMENT DISCHARGE The total rate at which sediment passes a given point on the stream (tons/day). See TOTAL SEDIMENT LOAD.

TOTAL-SEDIMENT LOAD (TOTAL LOAD) Includes bed load, suspended bed material load, and wash load. In general, total sediment load cannot be calculated or directly measured.

TRACTIVE FORCE When water flows in a channel, a force is developed that acts in the direction of flow on the channel bed. This force, which is simply the pull of water on the wetted area, is known as the tractive force. In a uniform flow, the equation for the unit tractive force (i.e., the average value to the tractive force per unit wetted area) is:

$$\tau_0 = \gamma RS$$

where:

τ_o = unit tractive force
γ = unit weight of water
R = the hydraulic radius
S = the slope of the channel.

TRANSPORTATION (SEDIMENT) The complex processes of moving sediment particles from place to place. The principal transporting agents are flowing water and wind.

TRANSPORT CAPACITY The ability of the stream to transport a given volume or weight of sediment material of a specific size per time for a given flow condition. The units of transport capacity are usually given in tons per day of sediment transported passed a given cross section for a given flow. Transport capacity for each sediment grain size is the transport potential for that size material multiplied by the actual fraction of each size class present in the bed and bank material.

TRANSPORT POTENTIAL Transport potential is the rate at which a stream could transport sediment of a given grain size for given hydraulic conditions if the bed and banks were composed entirely of material of that size.

TRAP EFFICIENCY Proportion of sediment inflow to a stream reach (or reservoir) that is retained within that reach (or reservoir). Computed as inflowing sediment volume minus outflowing sediment volume divided by inflowing sediment volume. Positive values indicate aggradation; negative values, degradation.

TRIBUTARY A river segment other than the main stem in which sediment transport is calculated. More generally, a stream or other body of water, surface or underground, that contributes its water to another and larger stream or body of water,

TURBULENCE In general terms, the irregular motion of a flowing fluid.

UNMEASURED LOAD Equipment used to measure sediment transport by sampling the concentration of suspended sediment cannot operate close to the stream bed. The material moving below the lowest point which the sampler can reach is termed “unmeasured load”.

VENTING A system designed to allow flood waters to enter an enclosure, usually the interior of the foundation walls, so that the rising water does not create a dangerous differential in hydrostatic pressure. This is achieved through small openings in the wall, such as a missing or rotated brick or concrete block, or small pipe.

V-ZONE **See Coastal High Hazard Area**

WASH LOAD That part of the suspended load that is finer than the bed material. Wash load is limited by supply rather than hydraulics, What grain sizes constitute wash load varies with flow and location in a stream. Sampling procedures that measure suspended load will include both wash load and suspended bed material load. Normally, that is of sediment particles smaller than 0.062 mm.

WATER COLUMN An imaginary vertical column of water used as a control volume for computational purposes. Usually the size of a unit area and as deep as the depth of water at that location in the river.

WATER DISCHARGE See STREAM DISCHARGE.

WATERSHED A topographically defined area drained by a river/stream or system of connecting rivers/streams such that all outflow is discharged through a single outlet. Also called a drainage area.

WEIR A small dam in a stream, designed to **raise the water level or to divert** its flow through desired channel. A diversion dam.

TABLE A-l [1]
Scale for Size Classification of Sediment Particles

CLASS NAME	MILLIMETERS	MICRONS	PHI VALUE
Boulders	**1 256**	--	**< -8**
Cobbles	**256 - 64**	--	**-8 to -6**
Gravel	**64 - 2**	**--**	**-6 to -1**
Very coarse sand	**2.0 - 1.0**	**2000 - 1000**	**-1 too**
Coarse sand	**1.o - 0.50**	**1000 - 500**	**0 to +1**
Medium sand	**0.50 - 0.25**	**500 - 250**	**+1 to +2**
Fine sand	**0.25 - 0.125**	**250 - 125**	**+2 to +3**
Very fine sand	**0.125 - 0.062**	**125-62**	**+3 to +4**
Coarse silt	**0.062 - 0.031**	**62 - 31**	**+4 to +5**
Medium silt	**0.031 - 0.016**	**31 -16**	**+5 to +6**
Fine silt	**0.016 - 0.008**	**16 -a**	**+6 to +7**
Very fine silt	**0.008 - 0.004**	**8 - 4**	**+7 to +8**
Coarse clay	**0.004 - 0.0020**	**4 - 2**	**+a to +9**
Medium clay	**0.0020 - 0.0010**	**2 - 1**	**+9 to +10**
Fine clay	**0.0010 - 0.0005**	**1 - 0.5**	**+10 to +11**
Very fine clay	**0.0005 - 0.00024**	**0.5 - 0.24**	**+11 to +12**
Colloids	**< 0.00024**	**< 0.24**	**> +12**

[1] **TABLE A-l is taken from EM 111 o-2-4000, March 1988**