SEDIMENTATION MANUAL

2nd Edition

Los Angeles County Department of Public Works
March 2006

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Water Resources Division
March 2006

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CHAPTER

1

Introduction

1.1 ACKNOWLEDGEMENT

The first edition of the Sedimentation Manual (1993) has been reformatted to be consistent with the 2006 Hydrology Manual. The methods from the Sedimentation Manual have not changed. This second edition of the Sedimentation Manual contains updated references to the 2006 Hydrology Manual and does not share appendices.

A group consisting of Isaac Gindi, Mariette Schleikorn, William Ward, Belinda Kwan, Loreto Soriano, Glenn Howe, Mahdad Derakhshani, Hartun Khachikian, Martin Moreno, and Allen Ma prepared the first edition of this manual under the principal direction of Sree Kumar and David Potter. An overview committee comprised of Eric Bredehorst, Alan Bentley, Chander Garg, Sree Kumar, Iraj Nasseri, and David Potter reviewed the contents of the Manual. Mr. Garvin Pederson, Mr. Reza Izadi, and Mr. Michael Anderson supervised the entire project. Laurel Putnam, Mooler Ang, Michael Miranda, Sanjay Thakkar, Phat Ho, and Darrell Yip also provided assistance.

1.2 PURPOSE AND SCOPE

This manual establishes the Los Angeles County Department of Public Works' sedimentation design criteria. The procedures and standards contained in this manual were developed mostly by the Hydraulic/Water Conservation Division of Los Angeles County Department of Public Works as the need arose to design erosion control structures, sediment retention structures, and channels carrying sediment laden flows. These sedimentation techniques are applicable in the design of local debris basins, storm drains, retention and detention basins, and channel projects within Los Angeles County.

Some sections of this Manual were previously part of Public Works' Hydrology Manual. When the Sedimentation Manual was developed, all information in the Hydrology Manual (March 1989 Edition) related to sedimentation was transferred into this manual. The hydraulic and structural design considerations are covered in Public Works' Hydraulic Design Manual (March 1982 Edition) and Public Works' Structural Design Manual (April 1982 Edition). For detailed debris basin design, refer to Public Works' Debris Dams and Basins Design Manual.

The Sedimentation Manual Appendices contain reference material, information, and design examples.

Public Works distributed copies of the first edition of the 1993 Sedimentation Manual and Appendix to members of the Land Development Advisory Committee (LDAC) for their review. The members who responded indicated that they had no comments on the Sedimentation Manual. This second edition reformats the manual and updates references to the 2006 Hydrology Manual.

1.3 FACTORS AFFECTING SEDIMENT PRODUCTION

Sediment production from a watershed is a function of several variables. The most evident variables in the County of Los Angeles are: vegetative cover, rainfall intensity, slopes of the watershed, geology, soil type, and size of drainage area. Figure 1.3.1 shows the result of sediment production after the 1969 storms.

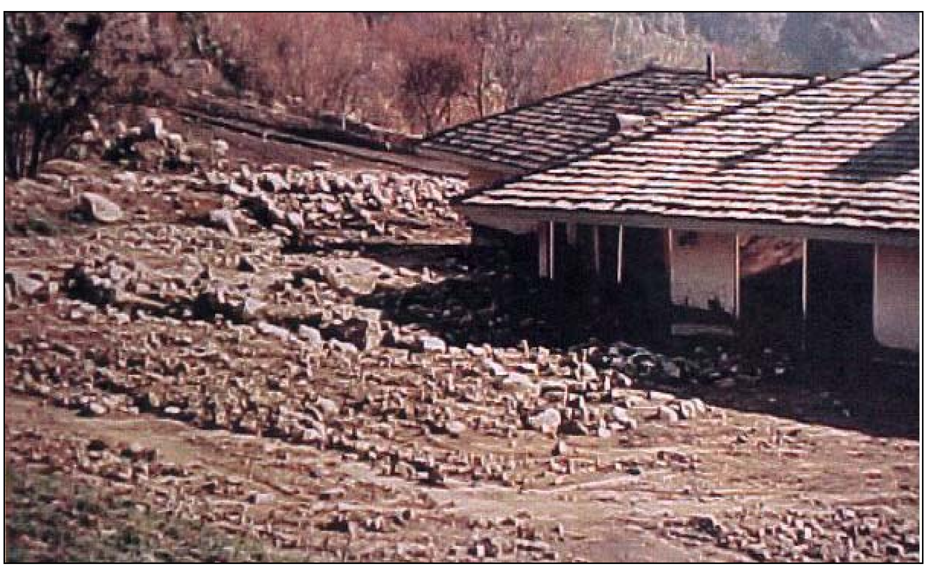

Figure 1.3.1
Sediment Production: Glencoe Heights, 1969

Fire greatly increases the amount of runoff and erosion from a mountain watershed. A recently denuded watershed will produce greater than normal sediment volumes due to higher runoff caused by a lack of vegetation and lowered infiltration rates. The inclusion of sediment in runoff results in a greater total discharge. This is referred to as bulking. Figure 1.3.2 shows a burned watershed below San Dimas Dam after the 2002 fires.

Figure 1.3.2
Burned Watershed Below San Dimas Dam After 2002 Fires

Flood flows from a denuded watershed can transport large quantities and sizes of sediment. Sediment production from a major storm has amounted to as much as 120,000 cubic yards per square mile of watershed. Boulders up to eight feet across have been deposited in valley areas a considerable distance from their source. Sediment discharge from a major storm can be equal to the actual storm runoff, that is, runoff bulked 100 percent.

1.4 FACTORS AFFECTING SEDIMENT TRANSPORT

Sediment transport depends on several factors such as particle size, shape, specific gravity, flow velocity, and depth. The ability of a stream to transport sediment increases as discharge increases and as streambed gradient increases. The three forms of sediment movement evident in the County of Los Angeles are discussed below.

General Sediment Transport

Sediment is transported as bed load or suspended load. Bed load is mostly transported by sliding, rolling, and bouncing over the bed. Suspended load is the portion of the load that is supported by turbulent eddies. Suspended load includes the finer portion of the bed material, which is only intermittently suspended within the flow. It also includes wash load, which consists of particles too fine to settle to the channel bed. Figure 1.4.1 shows high velocity flow, downstream of San Dimas Dam, which is capable of moving large amounts of sediment.

Figure 1.4.1
Flow Downstream of San Dimas Dam

Mud Floods

A flood in which the water carries heavy loads of sediment, generally between 20 to 45 percent by volume, is referred to as a mud flood. Mud floods typically occur in watercourses or on alluvial fans discharging from mountainous regions, although they may occur on less mountainous flood plains as well. Conventional hydraulic analysis using momentum, energy, and continuity theories are applicable, provided appropriate parameters are used.

Mudflows

A mudflow is a specific subset of landslides where the flow has sufficient viscosity to support large boulders within a matrix of smaller-sized particles. Mudflows may be confined to drainage channels or may occur unconfined on hill-slopes and alluvial fans. The concentration of sediment is generally higher than mud floods (typically 45 to 60 percent by volume). Mudflows are generally treated as viscoplastic fluids. Analysis requires use of the non-Newtonian theory.

The hydromechanics of mud floods and mudflows are not covered in this manual. Figure 1.4.2 shows the aftermath of mudflow in Upper Shields Debris Basin.

Figure 1.4.2
Upper Shields Debris Basin
March 3, 1978

CHAPTER

2

Public Works' Policy on Levels of Flood Protection

2.1 POLICY FOR SEDIMENT IN FLOW

A Public Works memorandum that established the policy on levels of flood protection for hydrologic design is included in Chapter 4 of the 2006 Hydrology Manual. That policy provides instructions on which design storm or rainfall frequency to use in developing runoff rates. This section discusses the additional requirements if flow includes sediment.

Capital Flood Protection

The following facilities and structures must be designed for the Capital Flood. The Capital Flood is the burned and bulked (where applicable) runoff from a 50-year frequency design storm falling on a saturated watershed. For fire factors see Chapter 6 of the 2006 Hydrology Manual. Section 3.4 of this Manual contains information on flow bulking.

Natural Watercourses

The Capital Flood level of protection applies to all facilities, including open channels, closed conduits, bridges, and dams and debris basins, that are constructed to transport or intercept sediment laden floodwaters from natural watercourses. Dams that are under the State of California (D.S.O.D.) jurisdiction must also meet the Probable Maximum Flood criteria found in Section 4.4 and Section 5.5 of the 2006 Hydrology Manual.

A natural watercourse is typically a path along which water flows due to natural topographic features. Refer to Section 4.2 of the 2006 Hydrology Manual for more detail. Figure 2.1.1 shows a natural portion of the San Gabriel River, below Morris Dam.

Figure 2.1.1
San Gabriel River
Below Morris Dam

Sediment Retention Facilities

The Capital Flood level of protection applies to all retention basins and detention basins designed to intercept sediment-laden floodwaters. Sediment retention basins must be designed to handle the design sediment volume. Refer to Chapter 3 for information on sediment production and delivery and to Chapter 4 for details on sediment control facilities.

Culverts

The Capital Flood level of protection applies to all culverts that pass sediment-laden flood waters under public roads.

Facilities with Tributary Areas Subject to Sediment Production

For any facility, apply the Capital Flood to all undeveloped tributary areas that are likely to produce sediment, whether or not the areas are likely to burn.

2.2 SANTA CLARA RIVER & MAJOR TRIBUTARIES - DRAINAGE POLICY

The Santa Clara River Basin is the second largest of the eight moderately developed drainage basins in Southern California and a major source of sediment for the beaches along the coast. In addition, the groundwater basins that underlie the Santa Clara River are an important source of water for the valley. It is important that the groundwater basins continue to be recharged by streambed percolation.

Therefore, the following standards have been adopted by the Department of the Public Works to maintain, as closely as possible, the environmental balance that exists in the Santa Clara River Basin. Note these standards supersede all previous standards and reports written for the Santa Clara River Basin.

1) The design of flood protection facilities for the Santa Clara River shall be based on the following:
   a) Public Works Capital Flood flow rates (50-year rainfall Q, burned and bulked only).
   b) Soft bottom waterways with levees.
   c) Protective levees and additional facilities such as drop structures or stabilizers as required, shall be designed using the Public Works criteria.

2) The design of flood protection facilities for major tributaries of the Santa Clara River that have been mapped by the Public Works as floodways (see Figures 2.2.1 and 2.2.2) or have a burned and bulked flow rate[1] of 2,000 cfs or greater as determined by Public Works' Capital Flood hydrology shall be based on items b) and c) above.

3) The design of flood protection facilities for tributary streams to the Santa Clara River that have existing flood control improvements shall be compatible with these existing facilities. See Table 2.2.1.

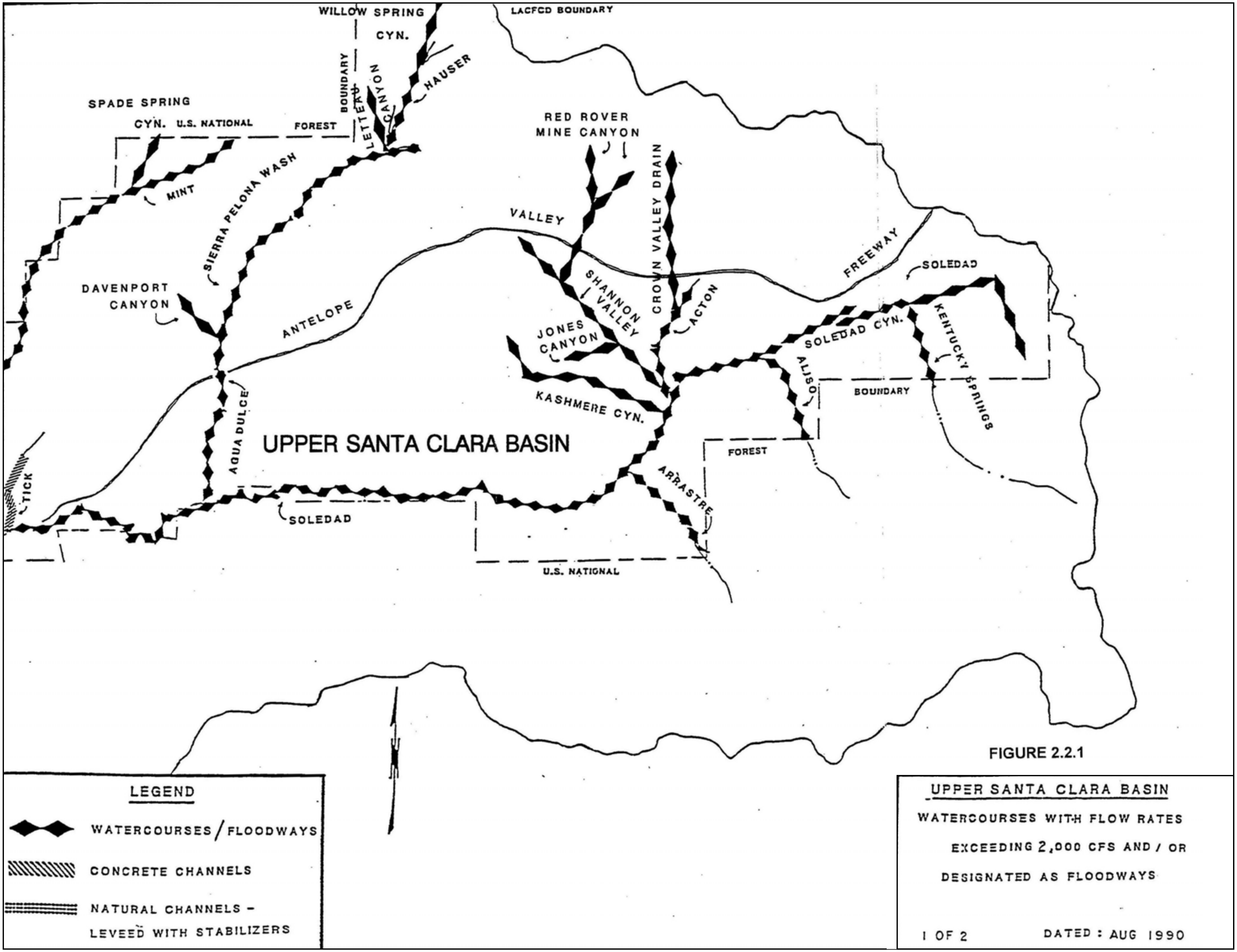

FIGURE 2.2.1

LEGEND

WATERCOURSES / FLOODWAYS

CONCRETE CHANNELS

NATURAL CHANNELS - LEVEED WITH STABILIZERS

UPPER SANTA CLARA BASIN

WATERCOURSES WITH FLOW RATES EXCEEDING 2,000 CFS AND / OR DESIGNATED AS FLOODWAYS

1 OF 2 DATED: AUG 1990

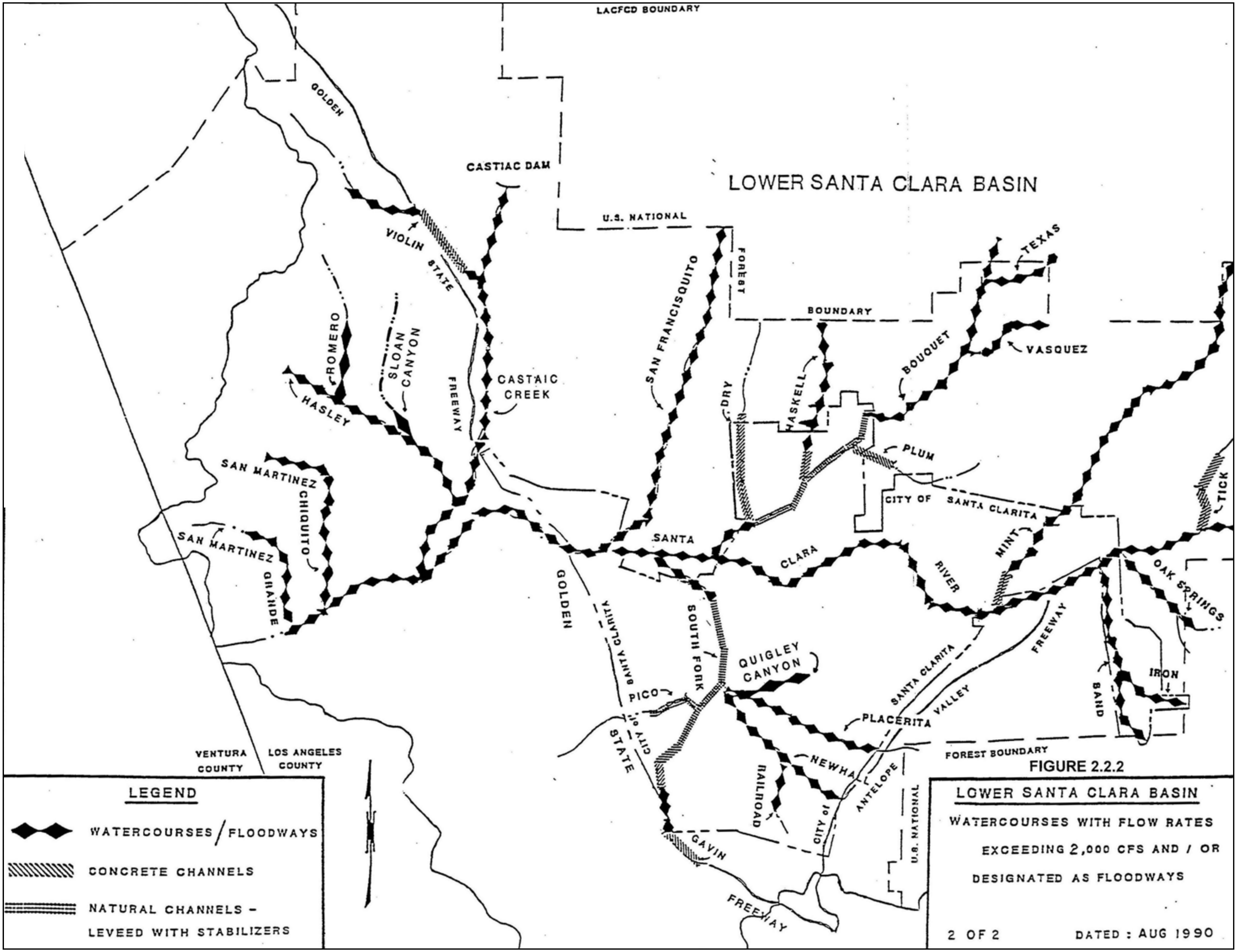
LACFCD BOUNDARY
LOWER SANTA CLARA BASIN
GOLDEN
CASTIAC DAM
U.S. NATIONAL
FOREST
BOUNDARY
VIOLIN
STATE
ROMERO
SLOAN CANYON
FREEWAY
CASTAIC CREEK
HASLEY
SAN FRANCISQUITO
DRY
HASKELL
BOUQUET
TEXAS
VASQUEZ
PLUM
CITY OF SANTA CLARITA
TICK
SAN MARTINEZ CHIQUITO
SAN MARTINEZ GRANDE
SANTA CLARA RIVER
MINT
OAK SPRINGS
GOLDEN
CITY of SANTA CLARITA
SOUTH FORK
QUIGLEY CANYON
PICO
SANTA CLARITA VALLEY
FREEWAY
IRON
SAND
PLACERITA
STATE
RAILROAD
NEWHALL
ANTELOPE
CITY of
U.S. NATIONAL
FOREST BOUNDARY
GAVIN
FREEWAY
VENTURA COUNTY
LOS ANGELES COUNTY
LEGEND
WATERCOURSES / FLOODWAYS
CONCRETE CHANNELS
NATURAL CHANNELS - LEVEED WITH STABILIZERS
FIGURE 2.2.2
LOWER SANTA CLARA BASIN
WATERCOURSES WITH FLOW RATES EXCEEDING 2,000 CFS AND / OR DESIGNATED AS FLOODWAYS
2 OF 2
DATED : AUG 1990

Figure 2.2.2

4) The soft bottom waterways shall be designed to maintain equilibrium between sediment supply to the waterway and sediment transport through the waterway. In cases where a soft bottom waterway is subject to significant deposition due to high sediment supply or significant erosion due to lack of sediment supply, then the drainage concept shall be discussed with the Public Works prior to submitting plans.

The following criteria was added in response to comments made by public on the previous policy:

1) Covered sections of natural bottom channels shall primarily be limited to street crossings.

2) Whether a bridge or a culvert is required for a road crossing over a soft-bottom channel depends on the flow rates and the magnitude of debris. Short culverts may be acceptable under certain cases, but in general, bridges shall be anticipated.

Figure 2.2.3 shows debris caught on a railroad bridge in the South Fork tributary of the Santa Clara River, which is a result of bulked flows.

Figure 2.2.3
Santa Clara River
South Fork

Table 2.2.1
Drainage Facilities for the Santa Clara River and Major Tributaries

Main River / Tributary	Current Improvement	Compatible Future Channel Improvement
Santa Clara River	Soft bottom with protective levee	Soft bottom with stabilizers where necessary
Tick Canyon	Lower reach-concrete channel	Upper reach-concrete channel with debris control
Mint Canyon	Lower reach-concrete channel	Middle reach-concrete channel Upper reach-soft bottom with stabilizers
Bouquet Canyon	Middle reach-soft bottom with stabilizers	Lower and Upper reaches-soft bottom with stabilizers
Dry Canyon	Lower reach-concrete channel	Upper reach-concrete channel
Haskell Canyon	Lower reach-concrete channel	Upper reach-soft bottom with stabilizers
Plum Canyon	Lower reach-concrete channel	Upper reach-concrete channel with debris control or soft bottom with stabilizers
South Fork -Santa Clara	Lower reach-soft bottom with stabilizers Middle reach-concrete channel	Lower reach-soft bottom with stabilizers Upper reach-concrete channel with debris control.
Pico Canyon	Lower reach partly soft bottom with stabilizers partly concrete channel	Upper reach-soft bottom with stabilizers
San Francisquito	Lower reach-soft bottom with stabilizers	Upper reach-soft bottom with stabilizers
Violin Canyon	Lower reach-concrete channel	Upper reach-concrete channel with debris control.
Castaic Creek	Below I-5 Freeway-soft bottom with protective levee	Above I-5 Freeway-soft bottom with stabilizers or concrete channel.

[1] Public Works' Capital Flood Flow Rates (50-year rainfall Q, burned and bulked)

CHAPTER 3

Sediment Production and Delivery

3.1 INTRODUCTION

Los Angeles Basin, Santa Clara River Basin, and Antelope Valley are divided into zones that yield similar volumes of sediment under similar conditions. These Debris[1] Potential Area (DPA) zone delineations are found in Appendix A.

Sediment production from a watershed is a rate at which sediment passes a particular point, usually expressed as cubic yards / square mile / storm. The sediment production is dependent upon many factors such as: rainfall intensity, geology, soil type, vegetative coverage, runoff, and watershed slope. Figure 3.1.1 shows a house buried by debris produced in Glencoe Canyon.

Figure 3.1.1
Glencoe Canyon, Glendora

A Design Debris Event (DDE) is defined as the quantity of sediment produced by a saturated watershed significantly recovered from a burn (after four years) as a result of a 50-year, 24-hour rainfall amount. The concept of DPA zones and Debris Production (DP) curves for determining watershed sediment production was introduced after the 1938 storms. Each DP curve and DPA zone represents particular types of geologic, topographic, vegetative, and rainfall features. These curves have been modified several times since inception of the concept.

A rate of 120,000 cubic yards / square mile / storm has been established as the design debris event for a one square-mile drainage area in DPA 1 zone. This rate is used as a design value for debris basins in areas of high relief and granitic formations characterizing the San Gabriel Mountains and Verdugo Hills. Other mountain areas in the County have been assigned relatively lower sediment potentials based on historical data and differences in topography, geology, and rainfall. Studies of sediment flow records indicate that areas less than one square-mile are expected to produce a higher rate of sediment production and areas greater than one square mile a lower rate.

In designing sediment retention facilities, use the DP curves to determine sediment production. Section 3.3 contains debris production equations for undeveloped watersheds, partially developed watersheds, watersheds with multiple DPA zones, and partially controlled watersheds.

In cases where slides or unstable slopes are found in the watershed, additional capacity may be required in the sediment retention facility. The additional capacity must be determined by a registered geologist and approved by Public Works' Geotechnical and Materials Engineering Division.

3.2 SEDIMENT PRODUCTION ZONES AND CURVES

The Los Angeles Basin has five sediment production curves, the Santa Clara River Basin has four curves, and the Antelope Valley has eight. See the debris production curves in Appendix B.

The use of DPA 7 in the Los Angeles Basin is limited to undeveloped areas with slopes less than 20%.

3.3 SEDIMENT DELIVERY

The following sections show the procedures to determine sediment production from watersheds with different characteristics. Sediment production is used for the selection and sizing of sediment control/conveyance structures. See Example 1 in Appendix D.

Undeveloped Watershed

Use the following procedure to determine sediment production at the outlet of an undeveloped watershed that completely falls within the boundaries of one DPA zone:

1) Identify the DPA zone from the maps in Appendix A.

2) Determine the drainage area (A) in square miles.

3) Determine the Debris Production Rate (DPR) from curves in Appendix B-1, 2, or 3, corresponding to the DPA zone and the drainage area found in steps 1 and 2 above. For areas smaller than 0.1 square mile, use the same DPR for 0.1 square mile.

4) Calculate the total Debris Production by multiplying the Debris Production Rate, from step 3, by the drainage area, from step 2. Equation 3.3.1 is used for single undeveloped watersheds within a single DPA Zone.

For a single watershed use Equation 3.3.1:

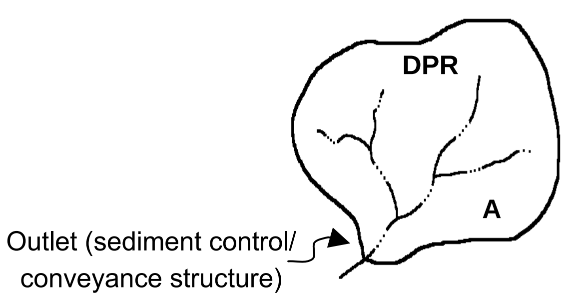

Figure 3.3.1
Debris Production for a Single Watershed

Equation 3.3.1

$$DP = DPR_{(A)} \times A$$

Where: DP = Debris Production in yd^3
DPR = Debris Production Rate in yd^3/mi^2

For multiple watersheds having a common outlet use Equation 3.3.2:

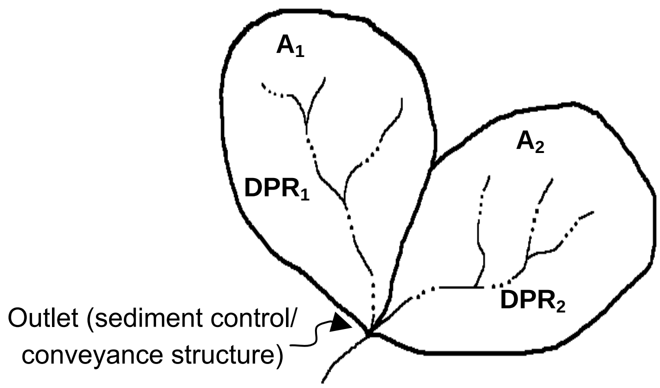

Figure 3.3.2
Debris Production for Multiple Watersheds

$$DP = (DPR_{1(A_1)} \text{ x } A_1) + (DPR_{2(A_2)} \text{ x } A_2)$$

Equation 3.3.2

Where: DP = Debris production in yd^3
$DPR_{i(Ai)}$ = Debris production rate based on area A_i in DPA zone i in yd^3/mi^2
A_i = Drainage area in mi^2

Partially Developed Watershed

Developed areas such as house/commercial pads, paved streets and parking areas, and maintained permanently landscaped areas that are not subject to burning (e.g. golf courses, cemeteries, parks) are considered non-debris producing. Other features such as a geologically non-erosive rock may be considered non-debris producing if supported by a geologic report. Use Equation 3.3.3 to calculate the total sediment production.

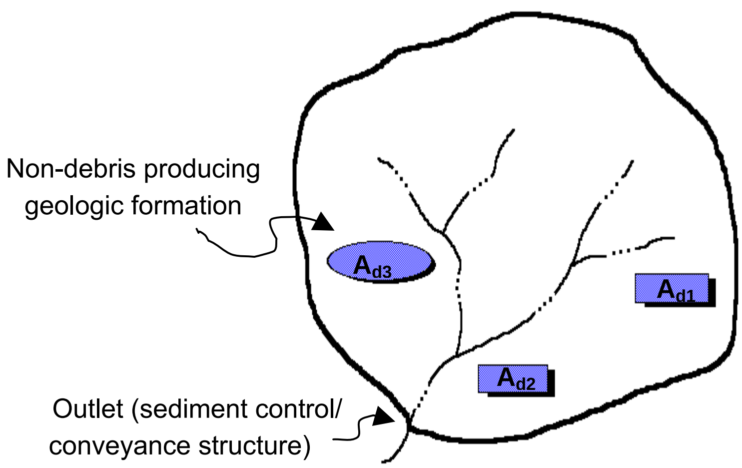

Figure 3.3.3
Debris Production for a Partially Developed Watershed

$$DP = DPR_{(A)} \times A_u\left(\frac{A_u}{A}\right) + DPR_{(A_u)} \times A_u\left(\frac{A_d}{A}\right)$$

$$A_d = A_{d_1} + A_{d_2} + A_{d_3}$$

$$A_u = A - A_d$$

Equation 3.3.3

Where:
- DP = Debris production in yd^3
- $DPR_{(A)}$ = Debris production rate based on the total drainage area A in yd^3/mi^2
- $DPR_{(Au)}$ = Debris production rate based on the total undeveloped drainage area A_u in yd^3/mi^2
- A = Total drainage area including developments in mi^2
- A_u = Total undeveloped area in mi^2
- A_d = Total developed area (existing only) in mi^2

Watersheds with Multiple Debris Production Zones

For an undeveloped watershed in two DPA zones use Equation 3.3.4.

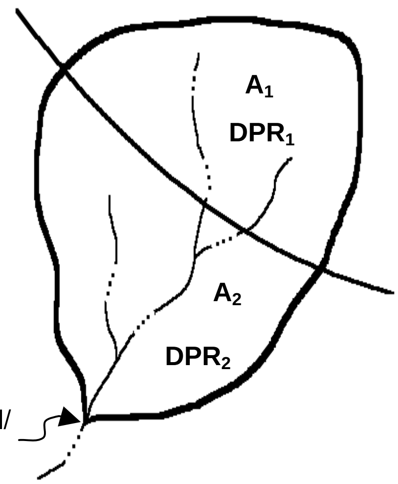

Figure 3.3.4
Debris Production for an Undeveloped Watershed in Two DPA zones

Equation 3.3.4

$$DP = DPR_{1(A_1+A_2)} \times A_1\left(\frac{A_1}{A_1+A_2}\right) + DPR_{1(A_1)} \times A_1\left(\frac{A_2}{A_1+A_2}\right) + DPR_{2(A_1+A_2)} \times A_2\left(\frac{A_2}{A_1+A_2}\right) + DPR_{2(A_2)} \times A_2\left(\frac{A_1}{A_1+A_2}\right)$$

Where:
- DP = Debris production in yd^3
- $DPR_{i(Ai)}$ = Debris production rate for drainage area A_i in DPA zone i in yd^3/mi^2
- A_i = Drainage area in mi^2

For a partially developed watershed in two DPA zones use Equation 3.3.5.

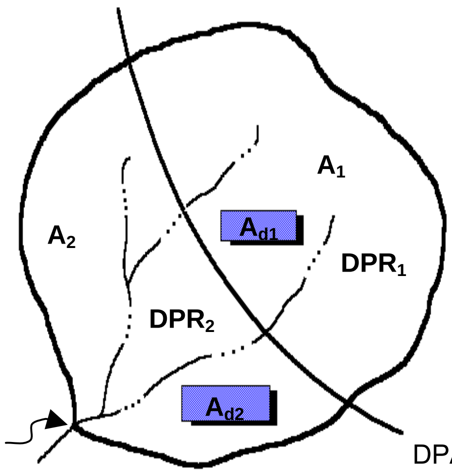

Figure 3.3.5
Debris Production for an Undeveloped Watershed in two DPA zones

$$DP = DPR_{1(A_1+A_2)}\,(A_1 - A_{d_1})\left(\frac{A_1 - A_{d_1}}{A_1 + A_2}\right) + DPR_{1(A_1-A_{d_1})}\,(A_1 - A_{d_1})\left(\frac{A_2 + A_{d_1}}{A_1 + A_2}\right) + DPR_{2(A_1+A_2)}\,(A_2 - A_{d_2})\left(\frac{A_2 - A_{d_2}}{A_1 + A_2}\right) + DPR_{2(A_2-A_{d_2})}\,(A_2 - A_{d_2})\left(\frac{A_1 + A_{d_2}}{A_1 + A_2}\right)$$

Equation 3.3.5

Where:
- DP = Debris production in yd^3
- $DPR_{i(Ai)}$ = Debris production rate for drainage area A_i in DPA zone i in yd^3/mi^2
- A_i = Drainage area including development in mi^2
- A_{di} = Developed area in area A_i in mi^2

Watersheds with Existing Sediment Control Structure

Use the following procedure to determine sediment production from a watershed partially controlled by an existing sediment control structure that meets the Public Works standards:

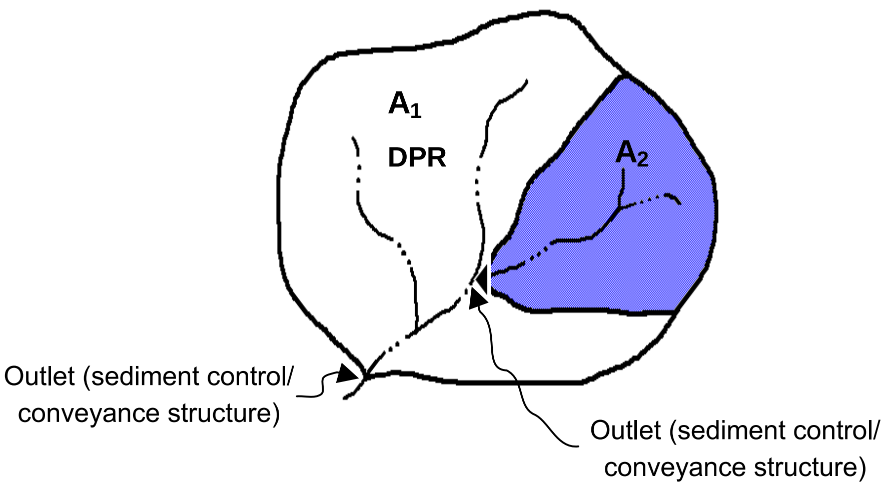

Figure 3.3.6
Debris Production for a Watershed with a Sediment Control Structure.

Follow steps (1) through (3) in the "Undeveloped Watershed" portion of Section 3.3. The equation to calculate the total sediment production depends on the condition of the existing sediment control structure.

(a.) Adequately sized:

$$DP = DPR_{(A_1+A_2)}\, A_1 \left(\frac{A_1}{A_1 + A_2}\right) + DPR_{A_1}\, A_1 \left(\frac{A_2}{A_1 + A_2}\right)$$

Equation 3.3.6

(b.) Undersized:

$$DP = DPR_{(A_1+A_2)}\, A_1 \left(\frac{A_1}{A_1 + A_2}\right) + DPR_{A_1}\, A_1 \left(\frac{A_2}{A_1 + A_2}\right) + DPR_{A_2}\, A_2 - C$$

Equation 3.3.7

Where:
- DP = Debris production in yd^3
- $DPR_{(Ai)}$ = Debris production rate based on area A_i in yd^3/mi^2
- A_i = Drainage area in mi^2
- C = Capacity of sediment control structure in yd^3

3.4 BULKING AND BULKED FLOW HYDROGRAPH

Bulking

Bulking is the increase in flow rate due to inclusion of sediment in the flow. This condition applies primarily to mountain areas subject to wildfires that destroy the vegetative cover protecting the soil. It also applies to watersheds in mountain areas with loose surface material that is likely to produce sediment. Figure 3.4.1 is an example of a burned watershed near Placerita Canyon Road. This watershed will potentially produce a bulked flow rate during a storm.

Figure 3.4.1
Placerita Canyon Road
after the Foothill Fire
October 10, 2004

The peak bulking factor curves in Appendix B show the proportion of the bulked flow rate to burned flow rate during the peak of the flood hydrograph or to the clear flow rate if the watershed has no potential to burn. These curves are used to design channels in a sediment producing area where a debris basin does not exist. Example 1 in Appendix D illustrates use of these curves.

The procedures for determining bulking factors for watersheds with different characteristics are similar to the procedures for determining sediment production explained in Section 3.3. To determine bulked flow rates, Q_B, use the equation listed below for the appropriate case.

For single undeveloped watersheds (see Figure 3.3.1):

$$Q_B = BF_{(A)} \times Q_{(A)}$$

Equation 3.4.1

For multiple undeveloped watersheds having a common outlet (see Figure 3.3.2):

$$Q_B = BF_{1(A_1)} \times \left(\frac{Q \quad A_1}{A_1 + A_2}\right) + BF_{2(A_2)} \times \left(\frac{Q \quad A_2}{A_1 + A_2}\right)$$

Equation 3.4.2

For partially developed watersheds (see Figure 3.3.3):

$$Q_B = BF_{(A)} \times \left(\frac{Q_{(A)} \quad A_u}{A}\right)\left(\frac{A_u}{A}\right) + BF_{(A_u)} \times \left(\frac{Q_{(A)} \quad A_u}{A}\right)\left(\frac{A_d}{A}\right) + \left(\frac{Q_{(A)} \quad A_d}{A}\right)$$

Equation 3.4.3

For a watershed with multiple debris production zones (see Figure 3.3.4):

$$Q_B = BF_{1(A_1+A_2)} \times \left(\frac{Q \quad A_1}{A_1 + A_2}\right)\left(\frac{A_1}{A_1 + A_2}\right) + BF_{1(A_1)} \times \left(\frac{Q \quad A_1}{A_1 + A_2}\right)\left(\frac{A_2}{A_1 + A_2}\right) +$$

$$BF_{2(A_1+A_2)} \times \left(\frac{Q \quad A_2}{A_1 + A_2}\right)\left(\frac{A_2}{A_1 + A_2}\right) + BF_{2(A_2)} \times \left(\frac{Q \quad A_2}{A_1 + A_2}\right)\left(\frac{A_1}{A_1 + A_2}\right)$$

Equation 3.4.4

$$Q = Q_{A_1+A_2}$$

Where:

Where:	Q	= Clear or burned discharge in cfs
	Q_B	= Bulked or burned and bulked discharge in cfs
	$BF_{i(Ai)}$	= Bulking factor based on area A_i
	A_i	= Drainage area in mi^2
	A_u	= Total undeveloped area in mi^2
	A_d	= Total developed area in mi^2

For a partially developed watershed in multiple DPA zones (see Figure 3.3.5):

$$Q_B = BF_{1(A_1+A_2)}\left(\frac{Q\ (A_1 - A_{d1})}{A_1 + A_2}\right)\left(\frac{A_1 - A_{d1}}{A_1 + A_2}\right) + BF_{1(A_1-A_{d1})}\left(\frac{Q\ (A_1 - A_{d1})}{A_1 + A_2}\right)\left(\frac{A_2 + A_{d1}}{A_1 + A_2}\right) + \left(\frac{Q\ (A_{d1})}{A_1 + A_2}\right) + BF_{2(A_1+A_2)}\left(\frac{Q\ (A_2 - A_{d2})}{A_1 + A_2}\right)\left(\frac{A_2 - A_{d2}}{A_1 + A_2}\right) + BF_{2(A_2-A_{d2})}\left(\frac{Q\ (A_2 - A_{d2})}{A_1 + A_2}\right)\left(\frac{(A_1 + A_{d2})}{A_1 + A_2}\right) + \left(\frac{Q\ (A_{d2})}{A_1 + A_2}\right)$$

Equation 3.4.5

For a watershed with an adequately sized, existing control structure (see Figure 3.3.6):

$$Q_B = BF_{(A_1+A_2)}\left(\frac{Q\ A_1}{A_1 + A_2}\right)\left(\frac{A_1}{A_1 + A_2}\right) + BF_{(A_1)}\left(\frac{Q\ A_1}{A_1 + A_2}\right)\left(\frac{A_2}{A_1 + A_2}\right)$$

Equation 3.4.6

For a watershed with an undersized, existing control structure (see Figure 3.3.7):

$$Q_B = BF_{(A_1+A_2)}\left(\frac{Q\ A_1}{A_1 + A_2}\right)\left(\frac{A_1}{A_1 + A_2}\right) + BF_{(A_1)}\left(\frac{Q\ A_1}{A_1 + A_2}\right)\left(\frac{A_2}{A_1 + A_2}\right) + BF_{(A_2)} \text{ x } \left(\frac{Q\ A_2}{A_1 + A_2}\right)$$

Equation 3.4.7

Where:

Where:	Q	= Clear or burned discharge in cfs
	Q_B	= Bulked or burned and bulked discharge in cfs
	$BF_{(Ai)}$	= Bulking factor based on area A_i
	A_i	= Drainage area in mi^2
	A_u	= Total undeveloped area in mi^2
	A_d	= Total developed area in mi^2

Appendix B has the bulking factor curves for the Los Angeles Basin, the Santa Clara River Basin, and the Antelope Valley area.

Bulked Flow Hydrograph (Sediment Transport Studies Only)

The bulked flow hydrograph is used for fluvial analysis and flood regulation studies. The bulked flow discharge can be obtained from the following equation:

$$Q_b = Q_s + Q_w$$ **Equation 3.4.8**

Where: Q_b = Bulked flow discharge
Q_s = Sediment discharge
Q_w = Water discharge (clear or burned).

This equation assumes that the peak of the sediment hydrograph coincides with the peak of the clear or burned water hydrograph.

To distribute the total design sediment volume (as described in Section 3.3) throughout a hydrograph, Public Works uses the following equation:

$$Q_s = a \times (Q_w)^n$$ **Equation 3.4.9**

Where: a = Bulking constant (fixed throughout the hydrograph)
n = Bulking exponent (fixed throughout the hydrograph)

Assume values of n to solve for a. The total sediment volume determined from the computed sediment hydrograph is then compared with the total volume obtained from the sediment production curves in Appendix B-1, 2, or 3. The value of n is then adjusted until the total volume under the sediment hydrograph is approximately equal to the total volume obtained from Appendix B-1, 2, or 3.

Consult with Public Works for additional guidelines if analysis of this type is needed.

3.5 GENERAL FORM EQUATIONS – DEBRIS PRODUCTION RATES & BULKING FACTORS

These equations are the general form of the equations in Sections 3.3 and 3.4 and can be used for multiple DPA zones. The number to the right of each equation corresponds to the number of the equation in Section 3.3 or 3.4. The postscript "g" shows that this is the general form of the equation.

$$DP = DPR_{(A)} \text{ x } A$$ **Equation 3.3.1g**

$$DP = \sum \left(DPR_{i(A_i)} \text{ x } A_i\right)$$ **Equation 3.3.2g**

Where:
- DP = Debris production, in yd^3
- $DPR_{i\,(Ai)}$ = Debris production rate based on area A_i in DPA zone i in yd^3/mi^2
- A_i = Drainage area in mi^2

$$DP = DPR_{(A)} \text{ x } A_u \left(\frac{A_u}{A}\right) + DPR_{(A_u)} \text{ x } A_u \left(\frac{A_d}{A}\right)$$ **Equation 3.3.3g**

$$A_d = \sum \left(A_{d_1} + A_{d_2} + A_{d_3} + \ldots + A_{d_n} \right)$$

$$A_u = A - A_d$$

Where:
- DP = Debris production in yd^3
- $DPR_{(A)}$ = Debris production rate based on the total drainage area, A, in yd^3/mi^2
- $DPR_{(Au)}$ = Debris production rate based on the total undeveloped drainage area, A_u, in yd^3/mi^2
- A = Total drainage area including developments in mi^2
- A_u = Total undeveloped area in mi^2
- A_d = Total developed area (existing only) in mi^2

$$DP = \sum \left[DPR_{i(A)} \times A_i \left(\frac{A_i}{A} \right) + DPR_{i(A_i)} \times A_i \left(\frac{A - A_i}{A} \right) \right]$$ **Equation 3.3.4g**

$$DP = \sum \left[DPR_{i(A)} \left(A_i - A_{d_i} \right) \left(\frac{A_i - A_{d_i}}{A} \right) + DPR_{i(A_i - A_{d_i})} \left(A_i - A_{d_i} \right) \left(\frac{(A - A_i) + A_{d_i}}{A} \right) \right]$$ **Equation 3.3.5g**

Where:
- DP = Debris production in yd^3
- $DPR_{i(Ai)}$ = Debris production rate for drainage area A_i in DPA zone i in yd^3/mi^2
- A = Total drainage area in mi^2
- A_i = Drainage area including development in mi^2
- A_{di} = Developed area in area A_i in mi^2

$$DP = \sum \left[DPR_{i(A)} \left(A_i - A_{c_i} \right) \left(\frac{A_i - A_{c_i}}{A} \right) + DPR_{i(A_i - A_{c_i})} \left(A_i - A_{c_i} \right) \left(\frac{(A - A_i) + A_{c_i}}{A} \right) \right]$$ **Equation 3.3.6g**

$$DP = \sum \left[DPR_{i(A)} \left(A_i - A_{c_i} \right) \left(\frac{A_i - A_{c_i}}{A} \right) + DPR_{i(A_i - A_{c_i})} \left(A_i - A_{c_i} \right) \left(\frac{(A - A_i) + A_{c_i}}{A} \right) + DPR_{(A_{ci})} \left(A_{ci} \right) - C_i \right]$$ **Equation 3.3.7g**

Where:
- DP = Debris production in yd^3
- $DPR_{(Ai)}$ = Debris production rate based on area A_i, in yd^3/mi^2
- A = Total drainage area in mi^2
- A_i = Drainage area in mi^2
- A_{ci} = Controlled drainage area within A_i in mi^2
- C_i = Capacity of sediment control structure in yd^3

$$Q_B = BF_{(A)} \text{ x } Q$$ **Equation 3.4.1g**

$$Q_B = \Sigma \left[BF_{i(A_i)} \text{ x} \left(\frac{Q \quad A_i}{A} \right) \right]$$ **Equation 3.4.2g**

$$Q_B = BF_{(A)} \text{ x} \left(\frac{Q \quad A_u}{A} \right) \left(\frac{A_u}{A} \right) + BF_{(A_u)} \text{ x} \left(\frac{Q \quad A_u}{A} \right) \left(\frac{A_d}{A} \right) + \left(\frac{Q \quad A_d}{A} \right)$$ **Equation 3.4.3g**

$$Q_B = \Sigma \left[BF_{i(A)} \text{ x} \left(\frac{Q \quad A_i}{A} \right) \left(\frac{A_i}{A} \right) + BF_{i(A_i)} \text{ x} \left(\frac{Q \quad A_i}{A} \right) \left(\frac{(A - A_i)}{A} \right) \right]$$ **Equation 3.4.4g**

$$Q_B = \Sigma \left[BF_{i(A)} \left(\frac{Q \ (A_i - A_{d_i})}{A} \right) \left(\frac{A_i - A_{d_i}}{A} \right) + \right.$$

Equation 3.4.5g

$$\left. BF_{i(A_i - A_{d_i})} \left(\frac{Q \ (A_i - A_{d_i})}{A} \right) \left(\frac{(A - A_i) + A_{d_i}}{A} \right) + \left(\frac{Q \ (A_{d_i})}{A} \right) \right]$$

$$Q_B = \Sigma \left[BF_{i(A)} \left(\frac{Q \ (A_i - A_{c_i})}{A} \right) \left(\frac{A_i - A_{c_i}}{A} \right) + \right.$$

Equation 3.4.6g

$$\left. BF_{i(A_i - A_{c_i})} \left(\frac{Q \ (A_i - A_{c_i})}{A} \right) \left(\frac{(A - A_i) + A_{c_i}}{A} \right) + \left(\frac{Q \ (A_{c_i})}{A} \right) \right]$$

Where:
- Q = Total clear or burned discharge in cfs
- Q_B = Bulked or burned and bulked discharge in cfs
- $BF_{(Ai)}$ = Bulking factor based on area A_i
- A = Total drainage area in mi^2
- A_i = Drainage area in mi^2
- A_u = Total undeveloped area in mi^2
- A_d = Total developed area in mi^2
- A_{ci} = Controlled drainage area within A_i in mi^2

$$Q_B = \sum \left[BF_{i(A)} \left(\frac{Q\ (A_i - A_{ci})}{A} \right) \left(\frac{A_i - A_{ci}}{A} \right) + \right.$$

$$BF_{i(A_i - A_{ci})} \left(\frac{Q\ (A_i - A_{ci})}{A} \right) \left(\frac{(A - A_i) + A_{ci}}{A} \right) + \left(\frac{Q\ (A_{ci})}{A} \right) + \qquad \textbf{Equation 3.4.7g}$$

$$\left. BF_{(A_{ci})} \left(\frac{Q\ (A_{ci})}{A} \right) \right]$$

Where:

Where:	Q	= Total clear or burned discharge in cfs
	Q_B	= Bulked or burned and bulked discharge in cfs
	$BF_{(Ai)}$	= Bulking factor based on area A_i
	A	= Total drainage area in mi^2
	A_i	= Drainage area in mi^2
	A_u	= Total undeveloped area in mi^2
	A_d	= Total developed area in mi^2
	A_{ci}	= Controlled drainage area within A_i in mi^2

Figure 3.5.1 shows sediment deposition at the confluence of Whitney and Elsmere Canyons at San Fernando Road on October 20, 2004.

Figure 3.5.1
Sediment Deposition - Confluence of Whitney and Elsmere Canyons at San Fernando Road October 20, 2004

[1] The term "debris" is used in this manual to be consistent with past practice but it means sediment.

CHAPTER

4

Sediment Control

4.1 INTRODUCTION

This chapter discusses the type of structure acceptable to Public Works for sediment control, which depends on the volume of sediment to be delivered to the site. This, in turn, depends on the Debris Potential Area (DPA) zone for the particular watershed. Table 4.1.1 is used to determine the type of structure. See Chapter 3 for methods of computing the sediment production volume. Where sediment production is less than 250 cubic yards, sediment control is generally not needed. Design the conveying storm drain following the closed conduit bulked flow design criteria listed in Section 5.5. As stated in the State Water Code, Division 3, Section 6000-6452, certain dams are under State jurisdiction. The State may have additional requirements for the design of the facility. Figure 4.1.1 shows Englewild Debris Basin during cleanout.

Figure 4.1.1
Englewild Debris Basin
Post-Storm Cleanout
February 2003

Table 4.1.1
Debris Control Structures Based on Debris Production

	Type of Structure	
Total Sediment Production (cubic yards)	DPA zone 1-4 requirement	DPA zone 5-11 requirement
20,000 or greater	Debris Basin	Debris Basin
5,000 to 19,999	Debris Basin	Elevated Inlet
1,000 to 4,999	Debris Basin or Elevated Inlet *	Desilting Inlet
250 to 999	Desilting Inlet *	Inlet with bulked flow drain
less than 250	Inlet* with bulked flow drain	Inlet with bulked flow drain

* The use of elevated or desilting inlets and bulked flow drains in DPA zones 1 through 4 will only be approved by Public Works in special circumstances. The steepness of the watershed, presence of boulders, and higher sediment and mudflow potential in these DPA zones results in a greater risk of plugging the storm drain and damaging the desilting wall.

Figure 4.1.2 shows the Upper Shields Debris Basin used for sediment control.

Figure 4.1.2
Upper Shields Debris Basin
March 3, 1978

4.2 GENERAL DESIGN CONSIDERATIONS

Location and Alignment

Locate all sediment retaining facilities in the existing watercourse. Align dams perpendicular to the original flow paths as shown in Figure 4.2.1. In order to insure maximum capacity, place the longer dimension of the basin along the flow line of the watercourse. If this distance is short in relation to the width, the intended capacity may not be attained.

Cone Slope

Sediment-laden flood flow, when reaching a sediment retaining facility, deposits the sediment up to spillway elevation and forms a delta or cone sloping upward from the spillway. For design purposes, this cone may contain up to, but no more than, one-half the capacity of the basin; this is called cone capacity. Figure 4.2.1 shows the cone capacity. The slope of the cone (S_D) is taken as one half of the average natural slope of the stream (S_N). The cone slope (S_D) should not exceed five percent (0.05).

In cases where the stream branches as it moves upstream from the debris dam, cone calculations are to be made along the individual profile lines of each branch. Depending upon the stream configuration, the profiles may branch from either the spillway crest or perhaps upstream of the crest. Hence, it is possible to have two different cone slopes. In these cases, the cone lines drawn perpendicular to the profile lines will intersect showing the configuration of the final cone surface as shown in Figure 4.2.2.

Level Capacity

The basin capacity up to the spillway elevation is called the "Level Capacity." Level capacity shall be at least one-half the capacity of the basin. Figure 4.2.1 shows the level capacity and cone capacity.

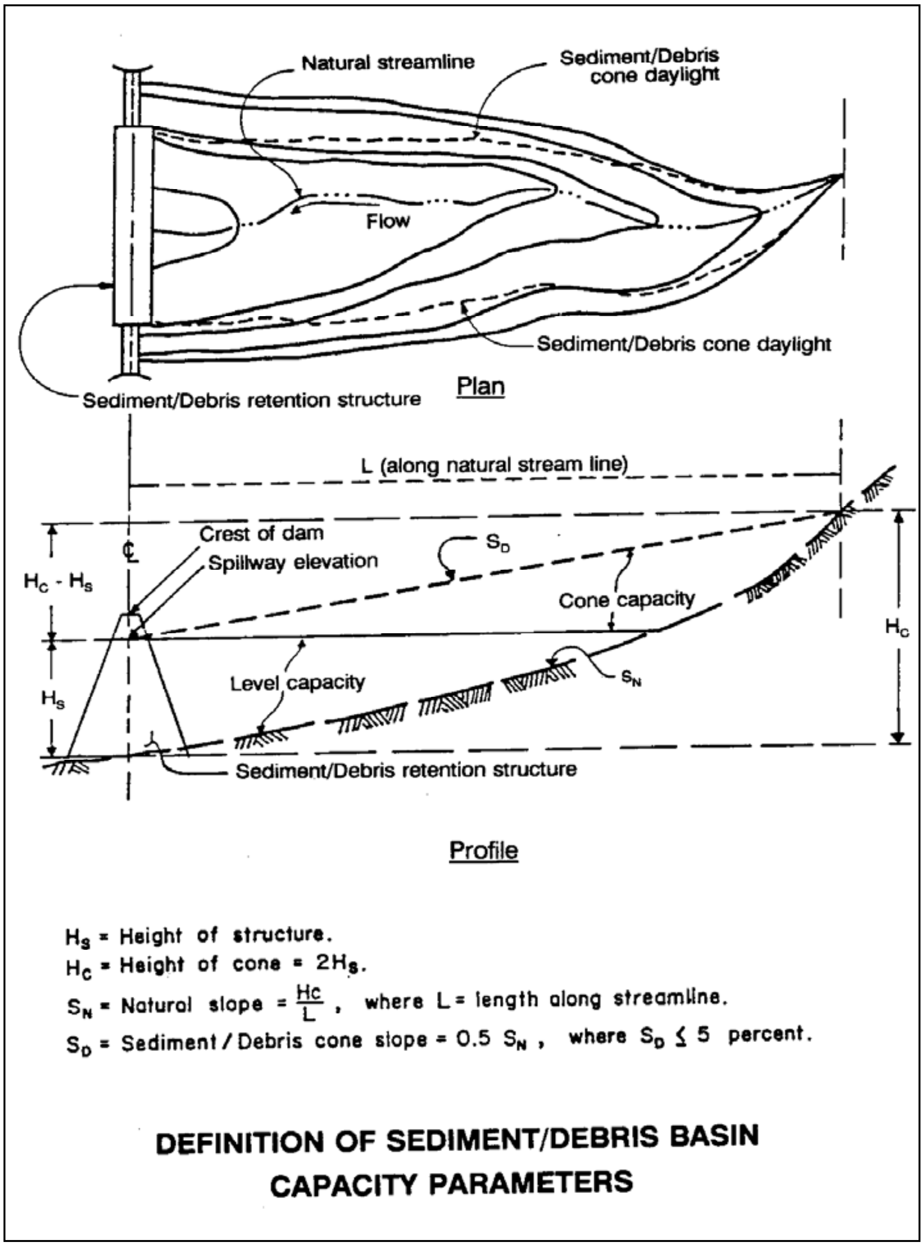

Figure 4.2.1
Definition of Sediment/Debris Basin Capacity Parameters

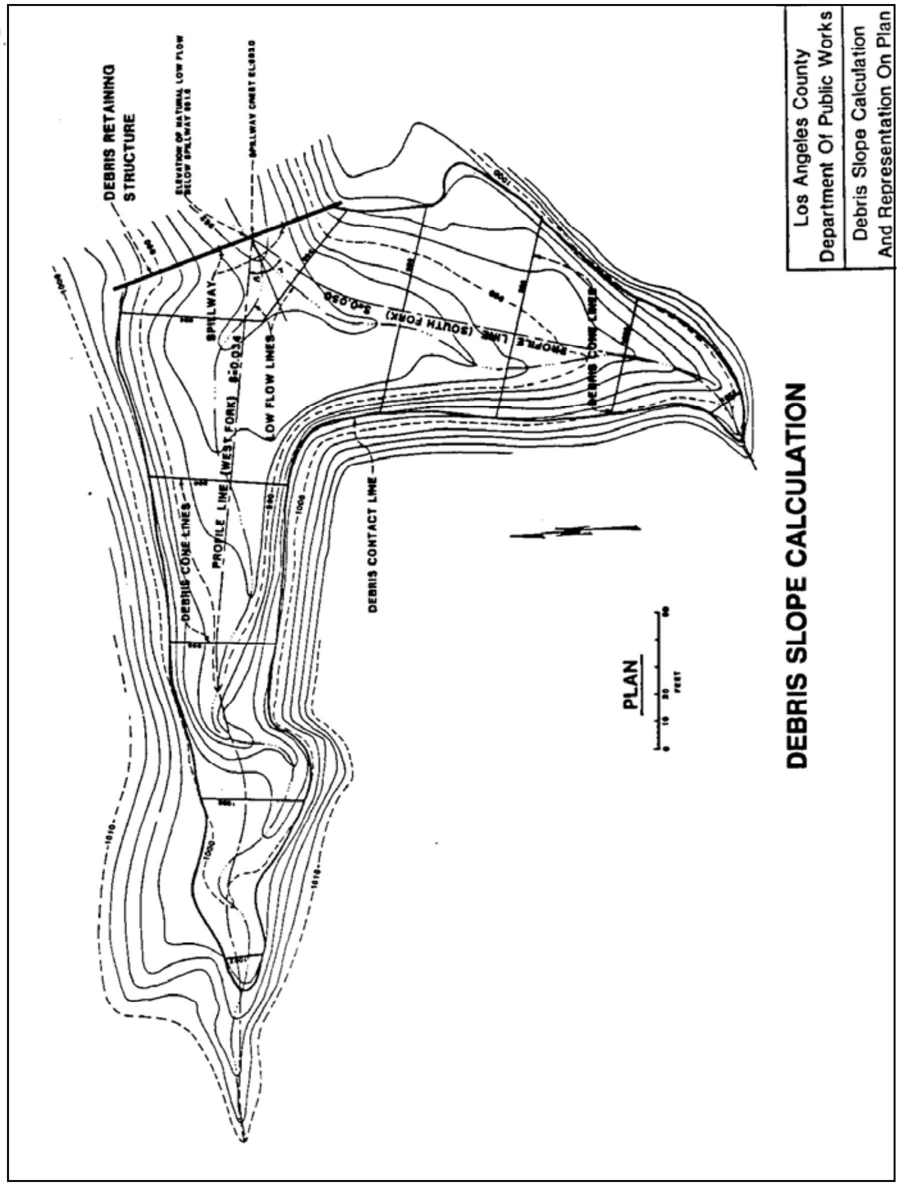

Figure 4.2.2
Debris Slope Calculation

Momentum Overflow

In the 1969 and the 1978 storms, some locations experienced unexpected events where significant amounts of sediment overflowed the spillway or dam before the basin was full. This type of event has been referred to as "Momentum Overflow."

It is believed that there are many contributing factors to this phenomenon. Some of the important factors are: rainfall amounts and intensity; watershed size, slope, shape, and condition (burned or unburned); soil composition; Debris Potential Area zone; debris basin shape; total versus cone capacity of the basin; slope of the upstream dam face; and the spillway location.

The likelihood of "Momentum Overflow" is reduced if the following design criteria are met for the sediment retaining facility:

- The cone slope is limited to a maximum of five percent.
- The level capacity is large enough to accommodate at least 50 percent of the debris event.

4.3 STANDARD SEDIMENT CONTROL METHODS

Appendix E includes a table comparing the design criteria for debris basins, elevated inlets, and desilting inlets.

Debris Basin

Public Works' Debris Dams and Basins Design Manual provides the specific design criteria for a debris basin. Appendix D contains a debris basin design example.

The criteria listed below amends the criteria given in Public Works' Debris Dams and Basins Design Manual.

- The horizontal alignment should be located in the original watercourse where the dam is perpendicular to the flow path. The longer dimension of the basin shall fall along the flow line.

- For the design of the outlet tower and conduit, refer to the section on Outlet Works in Public Works' Debris Dams and Basins Design Manual.

- Gage boards are required on basins under State Jurisdiction. Sediment lines need to be painted on the tower, marking from the lowest port invert suffice for all others. See the section on Gage Board Pipe Support in Public Works' Debris Dams and Basins Design Manual.

- The earth embankment slope, upstream and downstream, should be less than or equal to 3H:1V. Steeper slopes require a complete geotechnical stability analysis. Refer to the section on Earthen Dam Design in Public Works' Debris Dams and Basins Design Manual for more information.

- The embankment crest top width of the berm over the inlet shall be 20-feet paved with 3 inches of asphalt concrete. A berm width of 15-feet may be approved if geological analysis is provided to support the reduction.

- The facing slab shall be 6-inch concrete or gunite with No. 5 reinforcing steel at 18-inch spacing each way. See the section on Earthen Dam Design, Protection for Dam Slopes in Public Works' Debris Dams and Basins Design Manual.

- For trash barrier design, refer to the Debris Barrier section in Public Works' Debris Dams and Basins Design Manual.

- For access road and ramp design, refer to the Access to Dam and Basin section in Public Works' Debris Dams and Basins Design Manual. Access roads with 12-foot wide paving (3-inch asphalt concrete on 4-inch crushed aggregate base) within a 15-foot easement with a minimum turning radius of 40 feet can be used for structures with capacity less than 20,000 cubic yards. Access ramps are required. Unpaved ramps require slopes less than 10 percent. Paved ramps (3-inch asphalt concrete on 4-inch crushed aggregate base) require slopes less than or equal to 12 percent.

- For fencing, totally secure the basin area and inlet by 5-foot high fencing per APWA standard drawing 600-0.

- For fencing, structural design, hydraulic design, ponding, freeboard, drain size, inlet design capacity, and sediment capacity, refer to the respective section in Public Works' Debris Dams and Basins Design Manual.

Figure 4.3.1 shows a typical debris basin design.

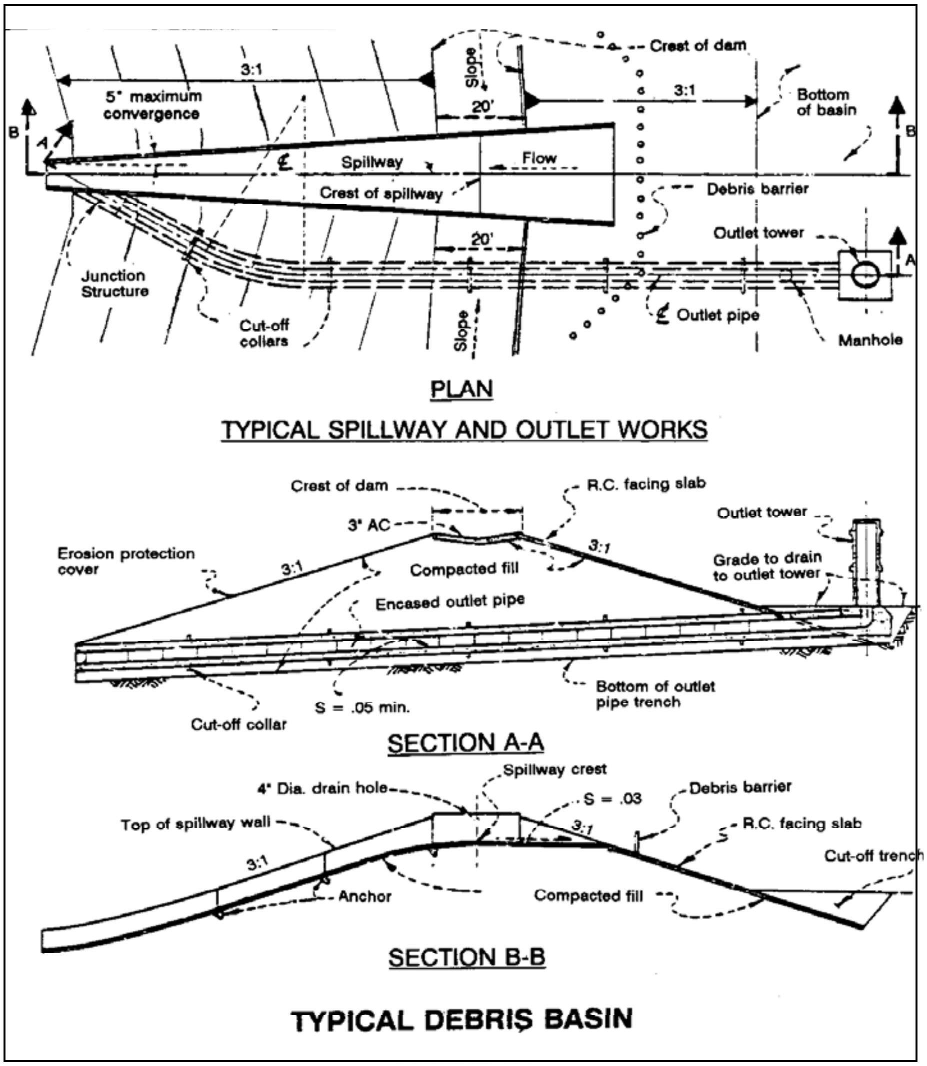

Figure 4.3.1
Typical Debris Basin

Elevated Inlet

Elevated inlets can be used if the conditions listed below are met. The design concept for all elevated inlets must be approved by Public Works prior to proceeding to final plans.

The following general criteria supplements the design criteria given in Public Works' Debris Dams and Basins Design Manual

- The location of an elevated inlet should be on a street or other safe path if available, to convey the water and sediment.

- The horizontal alignment should be located in the original watercourse where the dam is perpendicular to the flow path. The longer dimension of the basin shall fall along the flow line.

- A standard concrete outlet tower and conduit is required except in phased upstream development where corrugated metal pipe (CMP) tower with a concrete base may be substituted. The tower base can be modified to include a cleanout drain with a cover plate to allow flushing of the conduit. Extend the encasement on the conduit to the junction with the mainline or to a point where a 3H:1V slope originating from the intersection of the upstream face and the design headwater elevation meets the conduit, whichever is less.

- Gage boards of sediment lines painted on towers, marking from the lowest port invert can be used.

- The earth embankment maximum berm slope shall be 3H:1V. Steeper slopes require a complete geotechnical stability analysis. Refer to the section on Earth Dam Design in Public Works' Debris Dams and Basins Design Manual for further information.

- The embankment crest top width of the berm over the inlet shall be 20-feet paved with 3 inches of asphalt concrete. A berm width of 15-feet may be approved if geological analysis is provided to support the reduction.

- The facing slab shall be 6-inch thick reinforced concrete with reinforcing steel (no wire mesh) extending to the canyon wall. Air placed concrete is acceptable. Provide facing slabs around the basin wall if the cut and fill method is used to obtain the capacity.

- For trash barrier design, a swinging trash rack is required for conduits greater than 48-inches in diameter. A sloping trash rack per LACDPW 3089-0 can be used for smaller conduits. Discuss with Design Division prior to using a sloping trash rack especially in locations where organic debris may present a significant problem and may lead to clogging up the trash rack. Trash posts spaced at 4-feet or 2/3 the diameter of the conduit, whichever is smaller, are also required at all elevated inlets.

- For access road and ramp design, refer to the Access to Dam and Basin section in Public Works' Debris Dams and Basins Design Manual. A vehicular access road into the basin must be provided at least 12-feet wide within a 15-feet easement, paved with 3 inches of asphalt concrete over 4 inches of crushed aggregate base. Access ramps are required. Unpaved ramps require slopes less than 10 percent. Paved ramps (3-inch asphalt concrete on 4-inch crushed aggregate base) require slopes less than or equal to 12 percent.

- For fencing, refer to the section on Fencing in Public Works' Debris Dams and Basins Design Manual and totally secure the basin area and inlet by 5-foot high fencing per APWA standard drawing 600-0.

- For hydraulic design, base the design of the inlet and storm drain on requirements stated in Public Works' Hydraulic Design Manual.

- The maximum allowable ponding at the drain shall be 3-feet above soffit of the conduit inlet.

- The minimum freeboard at the inlet is 2-feet above the maximum water surface elevation.

- The minimum drain size is 36-inch RCP and the maximum drain size is 84-inch RCP or an equivalent RC Box.

- Design the inlet and storm drain to convey the burned flow rate and the fully developed watershed flow rate, whichever is higher.

- For structural design, refer to the section on Structural Design in Public Works' Debris Dams and Basins Design Manual.

- The maximum allowable capacity of sediment in DPA zones 1-4 is 4,999 cubic yards and in DPA zones 5-11 is 19,999 cubic yards.

If for any reason an elevated inlet cannot meet the requirements, then a debris basin is required. A typical elevated inlet is shown in Figure 4.3.2.

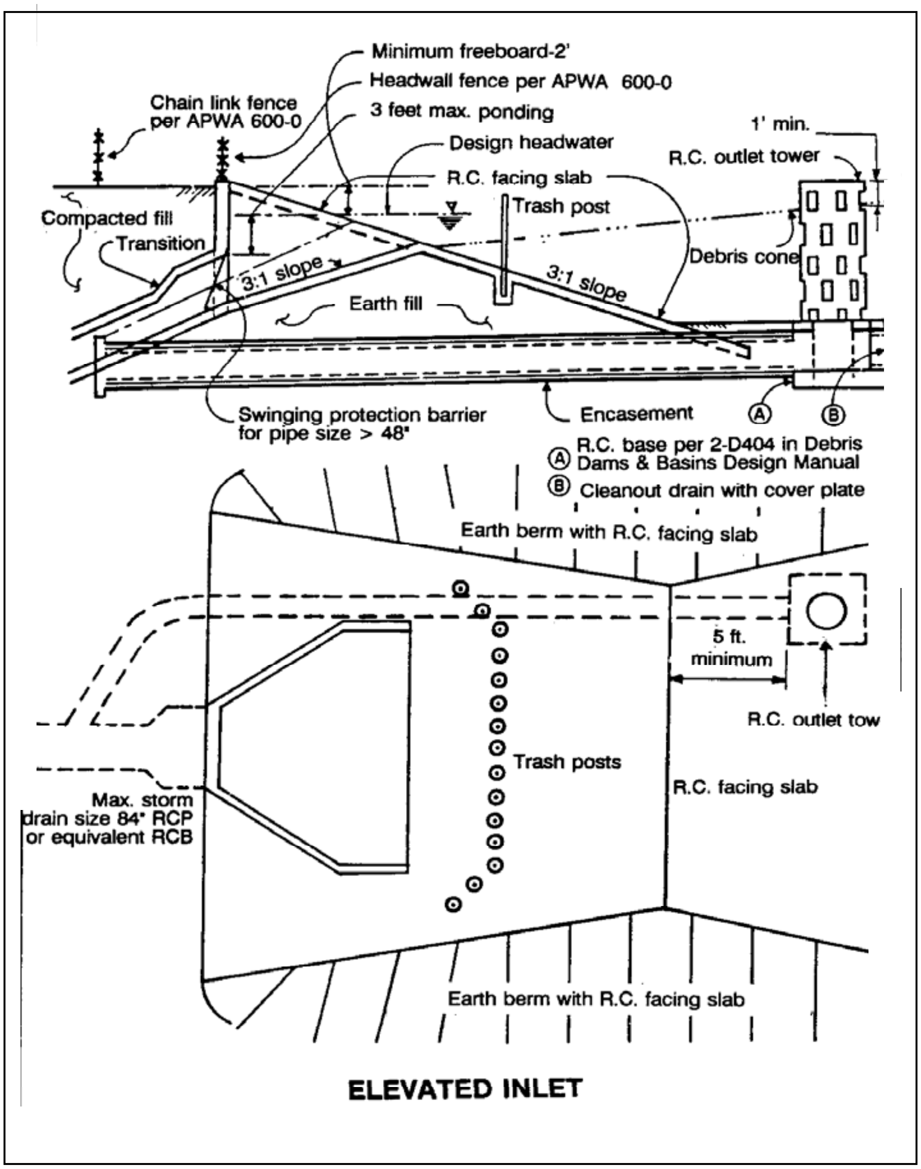

Figure 4.3.2
Elevated Inlet

Desilting Inlet

Desilting inlets can be used if the conditions comply with the requirements for a desilting inlet indicated below. The design concept for this inlet must be approved by Public Works prior to proceeding to final plans.

The following general criterion supplements the design criteria given in Public Works' Debris Dams and Basins Design Manual.

- The location of an elevated inlet should be on a street or other safe path if available, to convey the water and sediment.

- The horizontal alignment should be located in the original watercourse where the dam is perpendicular to the flow path. The longer dimension of the basin shall fall along the flow line.

- A corrugated metal pipe outlet tower and pipe is required upstream of the desilting wall.

- Gage boards of sediment lines painted on towers, marking from the lowest port invert can be used.

- The earth embankment must be protected between the desilting wall and the inlet with a reinforced concrete facing slab. Air placed concrete is acceptable.

- The embankment crest top width of the berm over the inlet shall be 20-feet paved with 3 inches of asphalt concrete. A berm width of 15-feet may be approved if geological analysis is provided to support the reduction.

- The facing slab shall be 6-inch thick reinforced concrete with reinforcing steel (no wire mesh) extending to the canyon wall. Air placed concrete is acceptable. Provide facing slabs around the basin wall if the cut and fill method is used to obtain the capacity.

- For trash barrier design, a sloping trash rack per LACDPW 3089-0 and trash posts spaced at 2/3 the diameter of the conduit are required.

- For access road and ramp design, refer to the Access to Dam and Basin section in Public Works' Debris Dams and Basins Design Manual. A vehicular access road into the basin must be provided at least 12-feet wide within a 15-feet easement, paved with 3 inches of asphalt concrete over 4 inches of crushed aggregate base.

- Access ramps are required. Unpaved ramps require slopes less than 10 percent. Paved ramps (3-inch asphalt concrete on 4-inch crushed aggregate base) require slopes less than or equal to 12 percent.

- For fencing, refer to the section on Fencing in Public Works' Debris Dams and Basins Design Manual and totally secure the basin area and inlet by 5-foot high fencing per APWA standard drawing 600-0.

- For hydraulic design, base the design of the inlet and storm drain on requirements stated in Public Works' Hydraulic Design Manual.

- The maximum allowable ponding at the desilting wall shall be 3-feet above the soffit of the drain.

- The minimum freeboard at the inlet is 2-feet above the maximum water surface elevation.

- The minimum drain size is 36-inch RCP and the maximum drain size is 48-inch RCP or an equivalent RC Box.

- Design the spillway notch and the inlet to pass the burned flow rate and the fully developed watershed flow rate, whichever is higher.

- For structural design, refer to the section on Structural Design in Public Works' Debris Dams and Basins Design Manual. Contact Design Division for additional information.

- The maximum allowable capacity of sediment in DPA zones 1-4 is 999 cubic yards and in DPA zones 5-11 is 4,999 cubic yards.

- The maximum desilting wall height is 6-feet.

- Design the desilting wall to withstand the overflow of the total burned and bulked flow rate.

Under certain favorable conditions, watersheds in DPA 5-11 and producing less than 1,000 cubic yards of sediment can be considered for a sediment-carrying conduit. If a desilting inlet cannot meet the requirements, then an elevated inlet or better is required. A typical desilting inlet is shown in Figure 4.3.3.

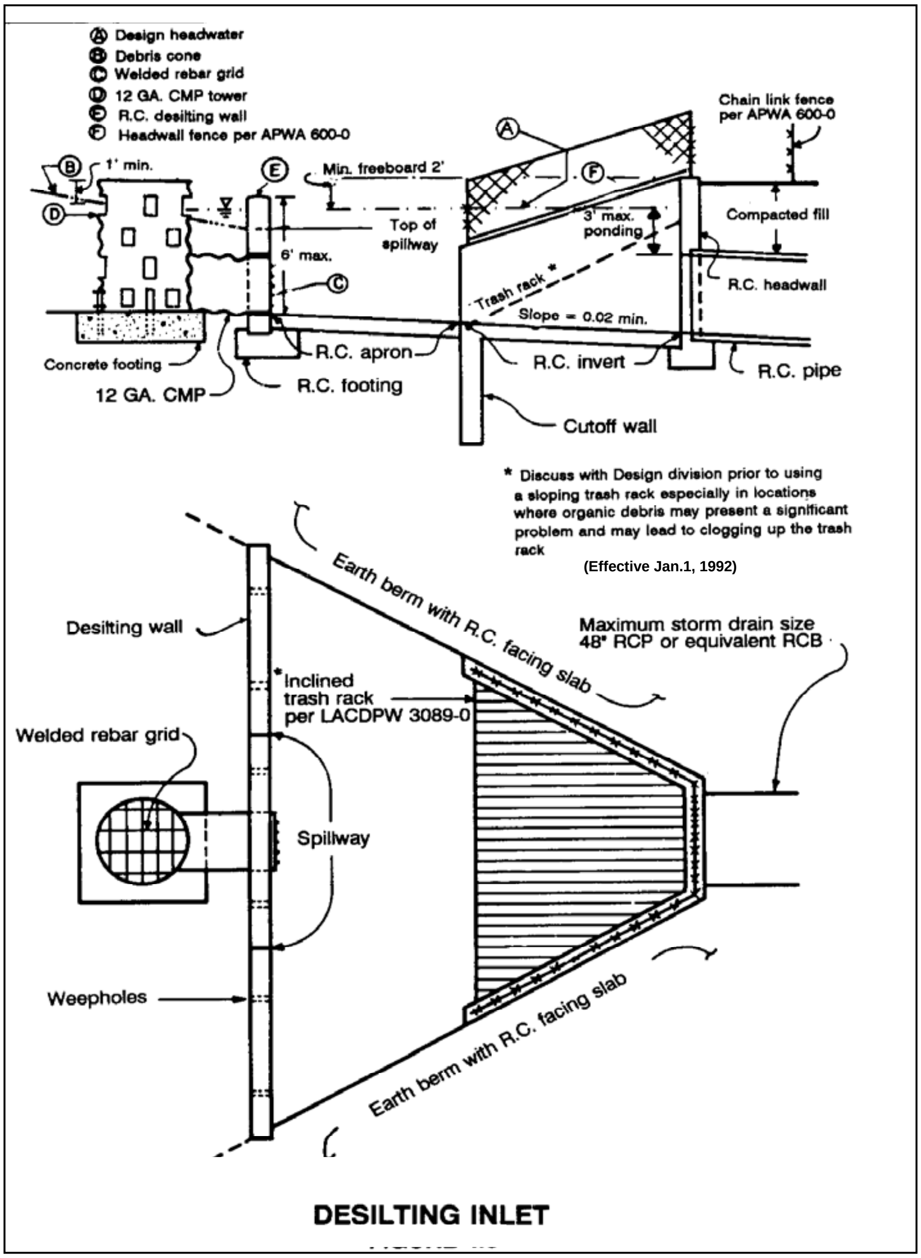

Figure 4.3.3
Desilting Inlet

4.4 OTHER SEDIMENT CONTROL METHODS

Public Works' pre-approval must be obtained at the design concept stage if other sediment control methods are proposed. The design criteria for alternative sediment control methods are described in the following sections.

Crib Dam

The crib dam structure was originally developed to stabilize streambeds. However, it can replace an earthen dam for debris basins with limited space. The structure is made of a cribbing framework of concrete members and the resulting cells are filled with aggregate. The height is controlled by the allowable stresses in the crib members and is generally not greater than 25 feet. An example of a crib dam is shown in Figure 4.4.1.

Figure 4.4.1
Crib Dam

A design manual for crib dams is currently not available from Public Works. Contact Public Works' Design Division for design details of the structure. For other design details including outlet works, refer to Public Works' Debris Dams and Basins Design Manual.

The following general criteria supplements the design criteria given in Public Works' Debris Dams and Basins Design Manual.

- Design the spillway as wide as possible to provide maximum spreading of the flow, and hence reduce stream energy to a minimum.

- Cap the portion of the crib structure to be used as a spillway with a reinforced concrete cover.

- Place the footing slab and the cribbing of the structure on a 6 horizontal to 1 vertical (6:1) upstream batter (see Figure 4.4.2).

- Construct a six-inch thick reinforced concrete facing slab with a 2 horizontal to 1 vertical (2:1) slope on the upstream face of the dam.

- Provide a sill at distance H+18 feet downstream from the structure to protect the dam from undercutting. Where H is the height of the structure in feet measured from the top of the slab to the water surface at maximum design flow depth.

- Construct a reinforced concrete slab or a grouted riprap slab between the sill and structure.

- Provide a separate channel headworks downstream of the sill to confine and direct the flow.

- Cut-off walls for both the sill and the dam shall be a minimum six feet deep or six inches into bedrock, whichever is less.

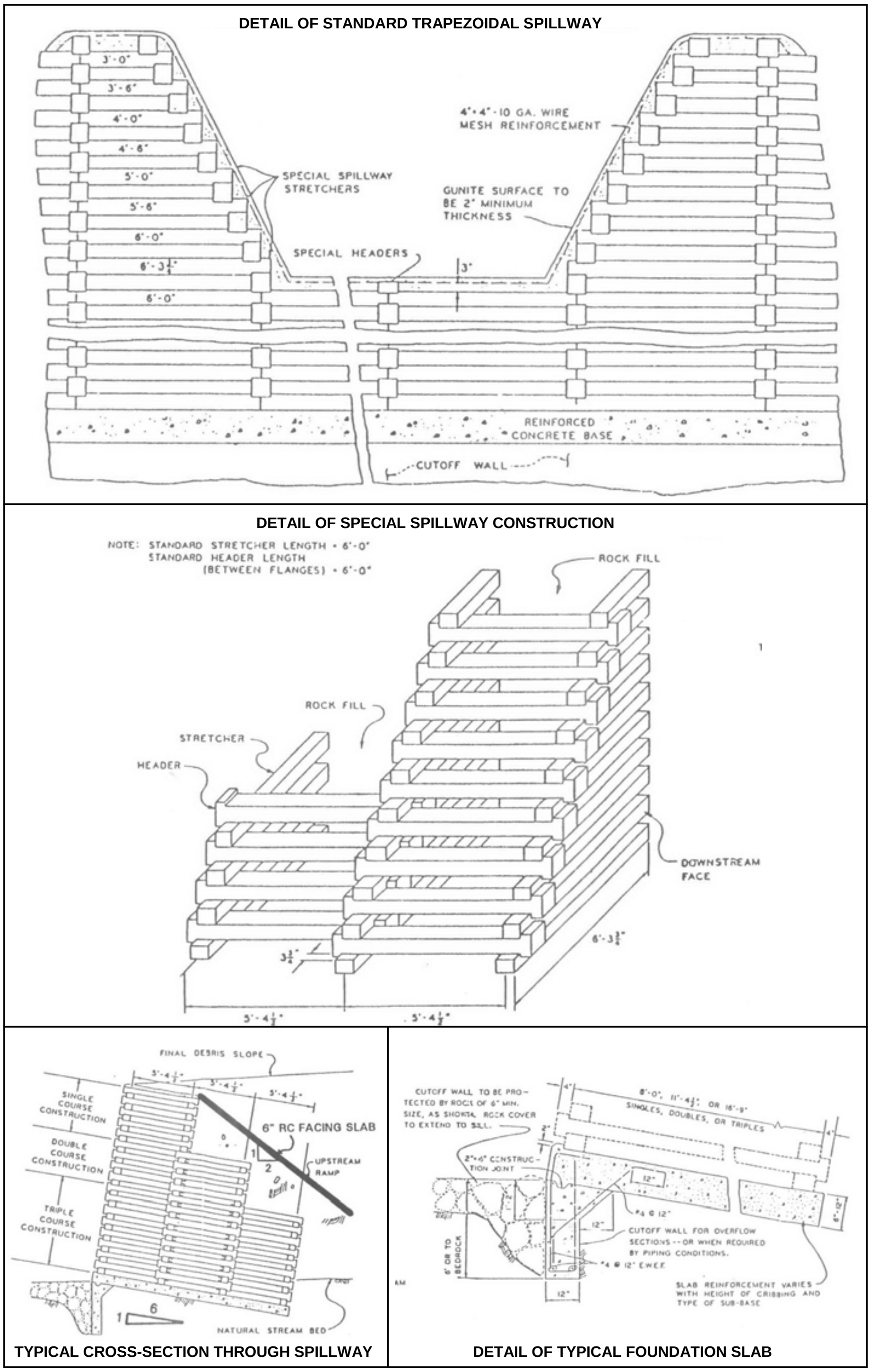

Figure 4.4.2
Crib Dam

Rail and Timber Structure

Rail and timber structures are primarily used as temporary emergency structures erected below recently burned areas where heavy sediment flows may prevent existing facilities from functioning properly. They are not to be permitted as permanent retention structures. They are generally designed and constructed by Public Works and kept in service until the watershed recovers from the burn.

The height of the structure (H) varies to a maximum 15 feet high with a reinforced concrete slab footing as shown in Figure 4.4.2. Refer to Public Works' Standard Plans manual (LACDPW 3085-0) for full design details of the structure.

Design the spillway to pass a Capital Flood peak flow rate, Q burned and bulked.

Provide access into the basin for cleanout purposes. On projects where a road cannot be provided, construct a removable panel in the barrier. For details of the road, refer to Public Works' Debris Dams and Basins Design Manual.

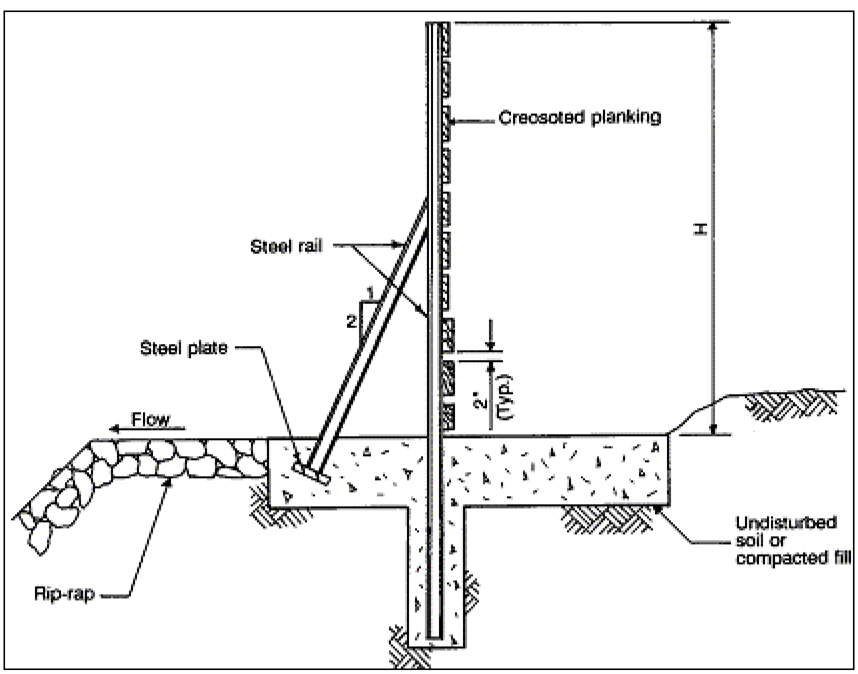

Figure 4.4.2
Rail & Timber Structure

Pit-type Basin

If a standard basin cannot be designed for the required capacity, a pit-type basin may be considered as shown in Figure 4.4.3.

Pit-type basins are generally considered subject to the momentum overflow phenomenon discussed in Section 4.2 and must be approved by Public Works prior to proceeding to final plans.

The type of outlet structure in a pit-type basin, as in any sediment retention basin, depends on the total sediment production. Refer to Appendix E to determine whether a debris basin, an elevated inlet, or a desilting inlet would be required for the design sediment production.

To design the basin capacity, first determine the cone slope then determine the storage ratio. The storage ratio is defined as the ratio of storage capacity below original ground to the total storage capacity (see Figure 4.4.3).

- If the storage ratio is greater than 0.7, the level capacity shall accommodate 100 percent of the design debris event.

- If the storage ratio is between 0.5 and 0.7, the level capacity shall accommodate at least 80 percent of the design debris event.

If the storage ratio is below 0.5, the level capacity shall accommodate at least 50 percent of the design debris event.

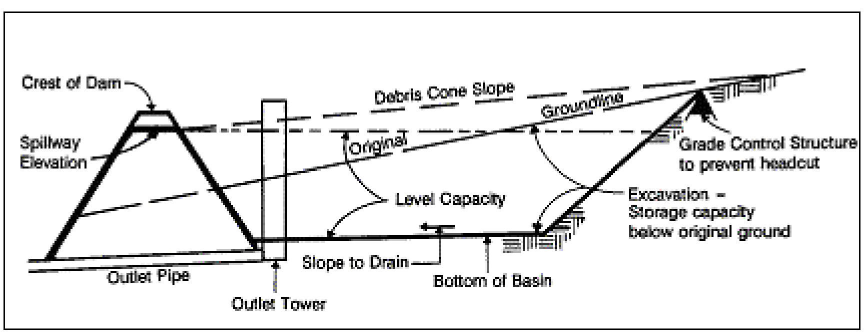

Figure 4.4.3
Pit-Type Basin

4.5 FLOOD RETENTION/DETENTION BASIN

The Public Works generally requires separate sediment and water retaining facilities. However, in special cases where sediment may deposit in a retention/detention basin, a combined facility may be accepted. Do not proceed with the design until approval is received from Public Works.

If Public Works accepts the combined facility, then the basin flow rate capacity is the difference between inflow versus outflow for the design flow rate of the facility. Refer to Chapter 2 for Public Works' policy on Level of Flood Protection and to the 2006 Hydrology Manual for the method of determining the runoff volume. Sediment storage capacity is equal to the design sediment production of the watershed. Determine the design sediment volume using the sediment production curves in Appendix B. The total capacity of the combined facility is the sum of the volume needed to control runoff and sediment. The total capacity must be located below spillway elevation as shown in Figure 4.5.1.

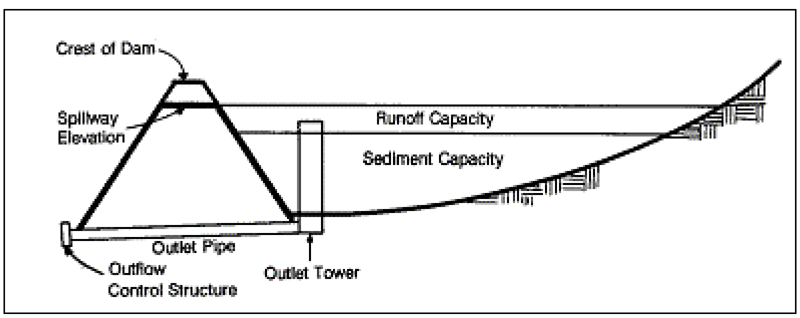

Figure 4.5.1
Flood Retention/Detention Basin

CHAPTER

5

Sediment Transport

5.1 INTRODUCTION

Sediment transport depends on the sediment particle size, shape, specific gravity, and on the flow velocity. Sediment may be transported as bedload or suspended load. Bedload is transported by sliding, rolling, and bouncing over the bed. Suspended load includes the finer portion of the bed material, which is intermittently suspended within the flow, and the wash load, which consists of particles too fine to settle to the channel bed. Figure 5.1.1 shows an example of sediment transport.

Figure 5.1.1
Example of Sediment Transport

Some of the more commonly used methods to determine sediment transport capacity are:

- Meyer-Peter, Muller Equation (MPM)
- Einstein Bed Load Equation
- Einstein Suspended Load Methodology
- Colby Methodology

Human activities can disturb the natural conditions of watercourses. Such activities include developments that encroach on the floodplain, construction of sediment trapping facilities, and gravel mining operations.

Public Works' general policy for the Santa Clara River and major tributaries is included in Section 2.2. This policy promotes the use of soft-bottom channels to pass sediment through the system where practical. Use debris or sediment control and hard bottom (concrete) channels very sparingly, primarily to be compatible with existing improvements.

The most desirable soft-bottom channel is one that does not degrade or aggrade. This channel is said to be in equilibrium. Developments encroaching on the floodplain reduce the channel width and increase the flow velocity. This increases the sediment transport capacity, which leads to invert degradation. Point stabilizers or drop structures may be used to prevent the scour from undermining the levee lining. If a reach is naturally aggrading, channelization can help increase the reach sediment transport capacity to approach the state of equilibrium.

Sediment control facilities and gravel mining operations may significantly decrease the rate of sediment supplied to downstream reaches. This causes the channel bed immediately downstream to erode. A hard-bottom (concrete) channel or soft-bottom channel with a series of drop structures would be necessary to convey the sediment deficient flows.

5.2 SOFT-BOTTOM CHANNELS WITH LEVEES

Under normal conditions, a sediment balanced soft-bottom channel is desired with proper design of the invert slope and channel width.

Conveyance Hydraulics, Erosion, Deposition

Levee failures can be due to general invert scour, bend scour, and/or local scour. Channelization, therefore, needs smooth transitions between varying sections and large radius bends. In addition, bridge abutment protection needs to be tied back or blended into the levee lining.

Sediment transport may be estimated through use of the procedures listed in Section 5.1. For a given channel width, an equilibrium slope can be calculated in a specific reach to satisfy the sediment continuity relationship where sediment transport through the improved reach is equal to the sediment supply into the reach.

$$Q_{S_{in}} = Q_{S_{out}}$$ **Equation 5.2.1**

Scour Protection (Levee Toe-down)

Toe-down or cut-off depth is the depth to which the bank revetment must be extended below grade to prevent undermining as the bed elevation fluctuates. The requirement for toe-down is the total cumulative channel adjustments possible from long-term degradation, general scour, bend scour, local scour, low-flow incisement, and bed forms. For an example, see Appendix D.

Use a lower Manning's n of 0.025 to estimate scour depth for design of toe-down.

$$Z_{tot} = Z_{deg} + Z_{gs} + Z_{ls} + Z_{bs} + Z_i + \frac{1}{2}h$$ **Equation 5.2.2**

Where:
- Z_{tot} = Total potential vertical adjustment
- Z_{deg} = Long-term degradation, see (a) below
- Z_{gs} = General scour, see (b) below
- Z_{ls} = Local scour, see (c) below
- Z_{bs} = Bend scour, see (d) below
- Z_i = Low-flow incisement, see (e) below
- h = Bed form height, see (f) below

a) Long-Term Degradation (Z_{deg})

The first step in determining long-term degradation is to find the discharge predominantly responsible for channel characteristics. The dominant discharge may be taken as 25% of Public Works' Capital Flood discharge (Q_{cap}).

Long-term degradation (or aggradation) within a particular channel reach may be estimated through use of the equilibrium slope techniques. Equilibrium slope for a channel may be estimated using the following steps:

1. Identify the supply reach, the reach upstream of the channel that supplies the channel with sediment.

2. Compute the hydraulic parameters for the supply reach using the dominant discharge.

3. Using one of the sediment transport methods from Section 5.1 that is appropriate for the stream and the hydraulic parameters from step (2), compute the sediment transport rate for the supply reach. This value is known as the sediment supply rate ($Q_{S\ in}$).

4. Choose an invert slope for the channelized reach, normally milder than the natural slope.

5. Using that slope, compute the hydraulic parameters for the channel (the transport reach) for the dominant discharge.

6. Apply the same sediment transport equation used in step (3) to the transport reach and compute the sediment transport rate through the channel ($Q_{S\ out}$).

7. Compare $Q_{S\ in}$ and $Q_{S\ out}$:

    - If equal, then the slope chosen in step (4) is the equilibrium slope.
    - If $Q_{S\ in} > Q_{S\ out}$, increase the slope and repeat steps (5) and (6).
    - If $Q_{S\ in} < Q_{S\ out}$, decrease the slope and repeat steps (5) and (6).

The curves in Appendix C-1 (A, B, and C) may be used to estimate the equilibrium slope. These curves show the relationship between the percent increase in velocity resulting from channelization and the corresponding change in invert slope. By subtracting that change from the natural slope, you get the equilibrium slope. Each figure consists of four curves to account for various reductions in sediment supply that can result from sediment trapping facilities or gravel mining operations.

When using the curves in Appendix C-1, compute the percent increase in velocity using Public Works' Capital Flood discharge (Q_{cap}), and 25% of Q_{cap}. Use the higher percent increase in velocity to determine the equilibrium slope.

Application of the equilibrium slope calculations requires the identification of a suitable point from which the computed equilibrium slope pivots. If natural geological controls such as rock outcroppings or man-made grade control structures exist, these features can serve as pivot points. For a given reach with such controls, the slope adjustment will always pivot about the downstream control point.

$$Z_{deg} = L\,(S_o - S_{eq})$$ **Equation 5.2.3**

Where: L = Reach length from point of interest to downstream pivot point
S_o = Existing slope
S_{eq} = Equilibrium slope

If the amount of levee toe-down appears excessive because of long-term degradation, consider alternatives such as implementation of grade control structures along the channelized reach.

b) General Scour (Z_{gs})

For a given flood event with a given duration, the volume of the sediment deposited or eroded in a channel reach is simply the difference between the upstream sediment supply rate and the channel sediment transport rate. If the supply rate is greater than the transport rate, the reach aggrades. The aggradation must be considered in the design of the levee freeboard height (FB) (see "Embankment Protection (Levee Height)" in this section). If the transport rate is greater than the supply, general scour will occur. Any scour that results from this phenomenon must be considered in the design of the total levee toe-down dimension (Z_{tot}).

Utilization of a sediment routing model (e.g. QUASED[1], HEC-6[2], FLUVIAL-12[3]) of the stream system is the best method of estimating the potential general scour (or general aggradation) on a reach by reach basis. However, less elaborate methods using rigid bed hydraulic and sediment transport calculations may be used to estimate the imbalance between sediment-transport capacity and sediment supply between adjacent reaches.

The curve in Appendix C-3 may also be used to estimate the general scour for the proposed flow velocity.

c) Local scour (Z_{ls})

Local scour occurs near an obstruction to flow, such as bridge piers, embankments, and contractions. Maximum local scour occurs during peak flow, therefore, use the peak Capital Flood (Q_{cap}) to determine the local scour (Z_{ls}) for the particular obstruction.

Pier Local Scour:

Appendix C-4 shows the relationship between pier width (b), in feet, and local scour (Z_{ls}), in feet, for square-nose piers. The different curves are for different velocities upstream of the bridge piers.

Scour depth adjustment factors (K_1) for pier shape other than square nose are presented in the following table:

Table 5.2.1
Scour Depth Adjustment Factors

Type of Pier	Reduction Factor K_1
Square nose	1.0
Round nose	0.9
Cylinder	0.9
Sharp nose	0.8
Group of cylinders	0.9

The angle of attack of oncoming flow has a significant impact on the potential scour depths. The local scour depth (Z_{ls}) from Appendix C-4 is adjusted by the appropriate factor (K_2) from Appendix C-5. Appendix C-5 shows the relationship between the angle of attack (α), in degrees, and the local scour adjustment factor (K_2). Several curves are shown for different pier length to width ratios (L/b), where L is the length of the pier, and b is the width of the pier, both in feet.

Another adjustment (K_3), is needed to account for debris blockage around the pier.

$$K_3 = \left(\frac{b + d}{b}\right)^{0.65}$$

Equation 5.2.4

Where: d = Debris blockage in feet

Use four feet of debris blockage where a heavy floating debris load can be expected. Otherwise, discuss with Public Works' Water Resources and Design Divisions. See Example 3 in Appendix D.

$$\text{Pier local scour} = Z_{ls} \times K_1 \times K_2 \times K_3$$

Note:

1. Footings supported on soil or degradable rock strata shall be embedded below the maximum computed scour depth.

2. Footings on piles may be located above the lowest anticipated scour level if the piles are designed for maximum scour condition. For earthquake loading, assume only half of the maximum anticipated scour has occurred. For this case, a concept must be approved by Public Works prior to proceeding with design.

Abutment Local Scour:

Estimate the depth of local scour at sloping-wall bridge abutments from the graph in Appendix C-6. The graph shows the relationship between the length an abutment protrudes into the flow path (a), in feet, and the depth of local scour (Z_{ls}), in feet. Several curves are shown for different velocity (V) and depth (Y) combinations.

Appendix C-6 is applicable to non-vertical walled abutments with embankment projection (a) less than 25 times the depth (Y). If the abutment terminates at a vertical wall, then multiply the scour depth (Z_{ls}) estimated from Appendix C-6 by a factor of 2.0.

Figure 5.5.2 shows an example of abutment local scour at the Harding Street Bridge over Pacoima Wash.

Figure 5.5.2
Abutment Scour at Harding Street over Pacoima Wash August 25, 2005

Levee Local Scour:

For soft bottom channels where the flow may possibly carry large debris (tree logs, boulders, etc.), increase the levee toe-down depth by 2 feet to account for local scour.

d) Bend Scour (Z_{bs})

This is the scour induced on the channel bed along the outside banks of channel curves.

Graphs in Appendix C-7A-C show the relationships between the ratio of the channel top width to radius of curvature (W/R) and the bend scour (Z_{bs}), in feet, for three different energy slopes (S_e). Energy slope (S_e) is the slope of the energy gradient. Several curves are shown in each graph for different velocity (V) and depth (Y) combinations.

The secondary currents that create bend scour extend for some distance beyond the downstream end of the channel bend. The relationship between the depth of flow within channel bend (Y), in feet, and the extent of scour downstream of channel bend (X), in feet, is shown on the graph in Appendix C-8. Figure 5.5.3 shows an example of bend scour along the Santa Clara River.

Figure 5.5.3
Bend Scour on
Santa Clara River
August 25, 2005

e) Low Flow Incisement (Z_i)

The best means of estimating the likely depths of incisement is through field inspection by measuring the low flow channel depth. For design purposes use Z_i equal to measured low flow depth, or 2 feet, whichever is greater. Figure 5.5.4 shows an example of low flow incisement along the San Gabriel River.

Figure 5.5.4
Low Flow Incisement on San Gabriel River July 1, 1974

f) Bed Form Height (h)

Bed forms (dunes and antidunes) commonly develop in natural or man-made channels with sand beds. The distance between the mean bed elevation and the trough of the bed form is approximately equal to the distance from the mean bed elevation to the bed form crest, and the sum of these two distances is termed the bed form height.

The relationship between the mean channel velocity (V), in feet per second, and the bed form height (h), in feet, is shown on the graph in Appendix C-9. If the bed form height (h) from Appendix C-9 exceeds the flow depth, use the flow depth instead.

Total Toe-Down Requirement (Z_{tot}):

Levee toe-down, as stated in Equation 5.2.2, is the total of long-term degradation (Z_{deg}), general scour (Z_{gs}), bend scour (Z_{bs}), local scour (Z_{ls}), half the bed form height (½h), and low flow incisement (Z_i).

Compare to the levee toe-down computed using Public Works' Hydraulic Design Manual criteria Section F and use the larger value.

Embankment Protection (Levee Height)

The levees must be designed to contain the design flood plus adequate freeboard.

Freeboard is the vertical distance from the water surface elevation to the top of the levees. Freeboard represents the additional height required to ensure overtopping does not occur from factors not accounted for in the design water surface calculations. These factors include possible long-term aggradation, superelevation at curved channels, and bed forms, in addition to less identifiable components such as separation, excessive turbulence, wave action and variations in loss coefficients.

Use a larger Manning's n to compute water surface elevations for design of levee height. Manning's n cannot be determined based on vegetation coverage alone. It is a function of many other variables including sediment size distribution, surface roughness, channel irregularity, obstructions, channel alignment and slope, and flow characteristics such as discharge, depth, and velocity. These variables change from one site to another; therefore, a generic description of the type and density of vegetation with relation to Manning's n is not feasible. Several references, such as <u>Open Channel Hydraulics</u> by Ven Te Chow, provide methodologies to determine the appropriate Manning's n considering all these variables.

Freeboard allowance is defined:

$$FB = Y_{agg} + Y_{ga} + Y_{se} + \frac{1}{2}h \qquad \textbf{Equation 5.2.5}$$

Where: FB = Total freeboard
Y_{agg} = Long-term aggradation
Y_{ga} = General aggradation
Y_{se} = Superelevation
h = Bed form height (from Appendix C-9)

Superelevation may be determined through application of the appropriate formula listed in Section C-3.1 of Public Works' Hydraulic Design Manual. Other components may be estimated with the same techniques presented under "Scour Protection".

Compare to the freeboard computed using criteria in Section C-4 of the Hydraulic Design Manual and use the larger value.

5.3 SOFT-BOTTOM CHANNELS WITH LEVEES AND STABILIZERS

Appropriate stabilization measures such as drop structures or point stabilizers may be required for soft-bottom channels. Appendix C-2 shows the allowed percent increase in velocity corresponding to the natural slope. Appendix C-2 has three curves to account for reduction in sediment supply that can result from sediment trapping facilities or gravel mining operations. If percent increase in velocity is higher than the allowable (above the curves) then invert stabilization is required. Figure 5.3.1 shows the San Gabriel River, an example of a soft-bottom channel with levees.

Figure 5.3.1
San Gabriel River – Soft Bottom Channel with Levees

Drop Structures

Drop structures (see Figures 5.3.2 and 5.3.3) are generally a conventional design with some type of stilling pool below the drop. The channel invert between the drop structures is graded to the design slope. See Example 4 in Appendix D.

The primary function of a drop structure is to decrease the gradient of a channel to create a condition of equilibrium (sediment inflow equal to sediment outflow). It also controls lateral bank migration and improves bank stability. The recommended maximum nominal height (H) for drop structures is typically five feet.

Place riprap downstream and upstream of the drop structure to reduce the effect of local scour. The mean riprap size is a function of the flow velocity. Appendix C-10 shows the relationship between the bottom velocity and the required riprap size. If channel velocity is beyond the range of the graph in Appendix C-10, an additional energy dissipation measure will be necessary other than riprap.

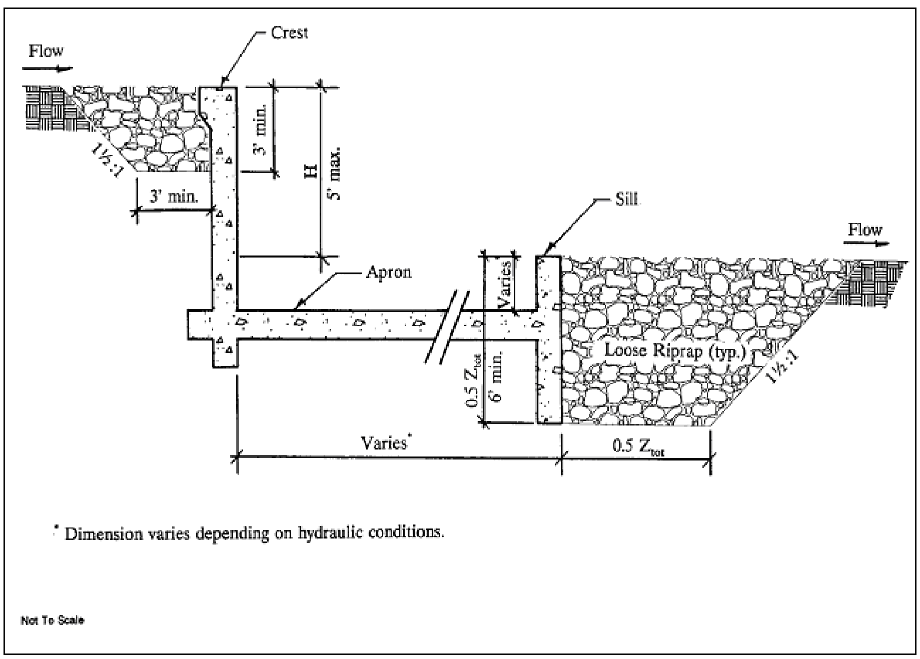

Figure 5.3.2
Drop Structure Drawing

Figure 5.3.3
Drop Structure along San Gabriel River

Point Stabilizers

The primary function of a point stabilizer (see Figure 5.3.4) is to maintain the stability of the natural streambed by controlling headcutting. The stabilizers are set at natural grade and buried to a sufficient depth to account for the scouring action that can occur during peak flows.

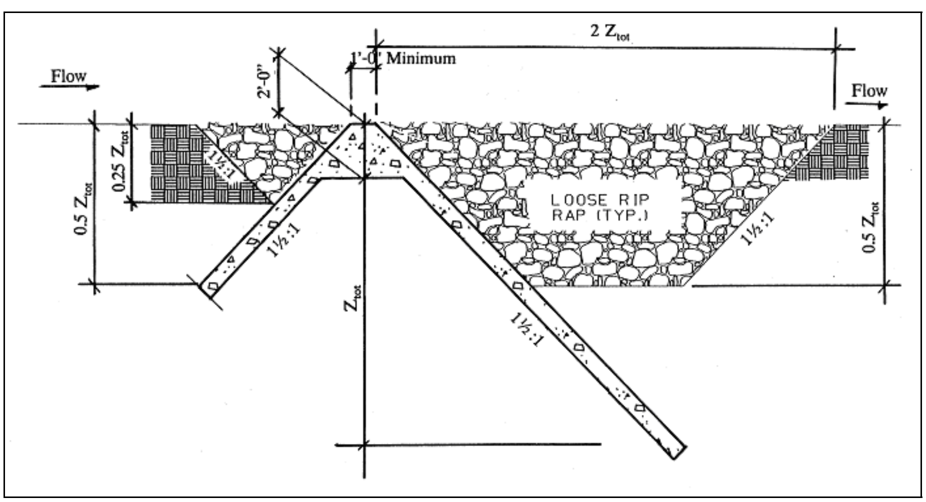

Figure 5.3.4
Point Stabilizer Drawing

Drop Height and Spacing

The design of grade-control structures is dependent upon the existing slope of the channel, the equilibrium slope (design slope) of the channel, the distance downstream to the nearest stable point in the channel, and the estimated scour hole depth below the structure under design flow conditions.

Determine the spacing of the invert stabilizers (D), from the following equation:

$$D = \frac{H}{(S_o - S_{eq})}$$

Equation 5.3.1

Where:
- D = Distance to the nearest downstream stable point
- H = Nominal height of grade control structure, 2' maximum for point stabilizers and 5' maximum for drop structures
- S_o = Existing channel slope
- S_{eq} = Equilibrium channel slope

Provide access ramps between invert stabilizers for channel maintenance.

5.4. HARD-BOTTOM (REINFORCED CONCRETE) CHANNELS

In the following cases, a soft-bottom channel is not feasible, and a concrete channel is needed:

a) Sediment supply to the channel is significantly reduced or eliminated as in the case of a debris basin or a gravel mining operation.

b) The invert slope is so steep that stabilizing the channel is not feasible.

To limit invert abrasion in concrete channels carrying sediment, design the channel based on the following criteria:

a) Velocity of debris carrying flow shall not exceed 40 feet per second

b) Design shall comply with Public Works' Structural Design Manual, Sections G-9 (steel clearances and additional cover over the reinforcing steel)

Concrete channels must be designed to prevent sediment deposition, which would reduce conveyance capacity. Deposited sediment has the dual impact of raising the bed level while increasing the roughness of the channel bed, which increases the channel flow resistance.

The minimum velocity required to keep the channel clear of sediment is known as the limiting deposit velocity (V_l). Graphs in Appendix C-11 show the relationship between the size of sediment for which 85 percent of the sediment is finer (d_{85}) and the limiting deposit velocity (V_l) in feet per second.

Follow the requirements discussed below for the design of concrete channels carrying bulked flow.

Provide a vehicular access road of at least 12-feet wide within a 15-foot easement, paved with 3 inches of asphalt concrete (A.C.) over 4 inches of crushed aggregate base (C.A.B.) on both sides of the channel. For freeboard and the hydraulic design, refer to Public Works' Hydraulic Design Manual. The design capacity of the channel or inlet and drain must be sized to pass the burned and bulked flow rate or the fully developed watershed flow rate, whichever is higher. For structural design, refer to Public Works' Structural Design Manual requirements for sediment carrying channels. The peak flow velocity shall be greater than the limiting deposit velocity for the size of material to be transported, but shall not exceed 40 feet per second. For junctioning, the angle of confluence shall not exceed 5^o 45'. Design the inlet to the concrete channel to accelerate flows into the drain. Provide a minimum slope of 2% for the invert slab.

Figure 5.4.1 shows the Rio Hondo Channel, an example of a hard-bottom concrete channel.

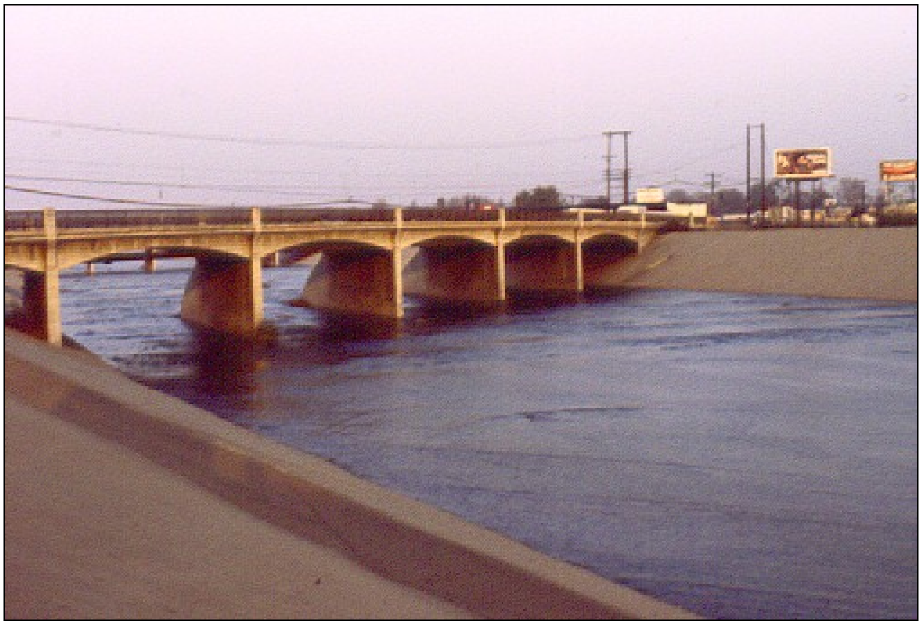

Figure 5.4.1
Rio Hondo Channel
December 15, 1977

5.5 CLOSED DRAINS

The minimum velocity required to keep the conduit clear of sediment is known as the limiting deposit velocity (V_l). Graphs in Appendix C-11 show the relationship between the size of sediment for which 85 percent of the sediment is finer (D_{85}) and the limiting deposit velocity (V_l) in feet per second.

Closed conduits carrying bulked flow may be used according to the conditions in Table 4.1.1 for inlet with bulked flow drain. The design concept must be approved by Public Works prior to proceeding to final plans. Follow the requirements listed below for design of closed conduits carrying bulked flow.

Do not locate a closed conduit drain under homes or other permanent structures. Provide a safe secondary overflow path for water and sediment. The horizontal alignment of the storm drain shall be straight. If bends are unavoidable, the radius of curvature shall be at least 30 times the width of pipe. The central angle shall not exceed 45 degrees. The maximum deviation computed by the ratio: actual length from inlet to outlet/junction over straight line distance from inlet to outlet/junction, shall be less than 1.1. A trash rack per LACDPW 3089-0 is required at the inlet. Trash posts should be spaced at 2/3 the diameter of the conduit or 4 feet, whichever is smaller, are also required.

Provided a vehicular access road of at least 12-feet wide within a 15-foot easement, paved with 3 inches of asphalt concrete over 4 inches of crushed aggregate base. For hydraulic design, refer to Public Works' Hydraulic Design Manual. Pressure flow is not permitted in closed conduits.

Watersheds producing 1,000 cubic yards of sediment or greater require the use of an open channel (see Section 5.4) or a sediment control facility (see Chapter 4). See Figure 5.5.2 for a typical sediment carrying inlet and drain. Figure 5.5.1 shows a sediment-filled culvert.

Figure 5.5.1
Culvert Filled With Sediment
January 30, 1969

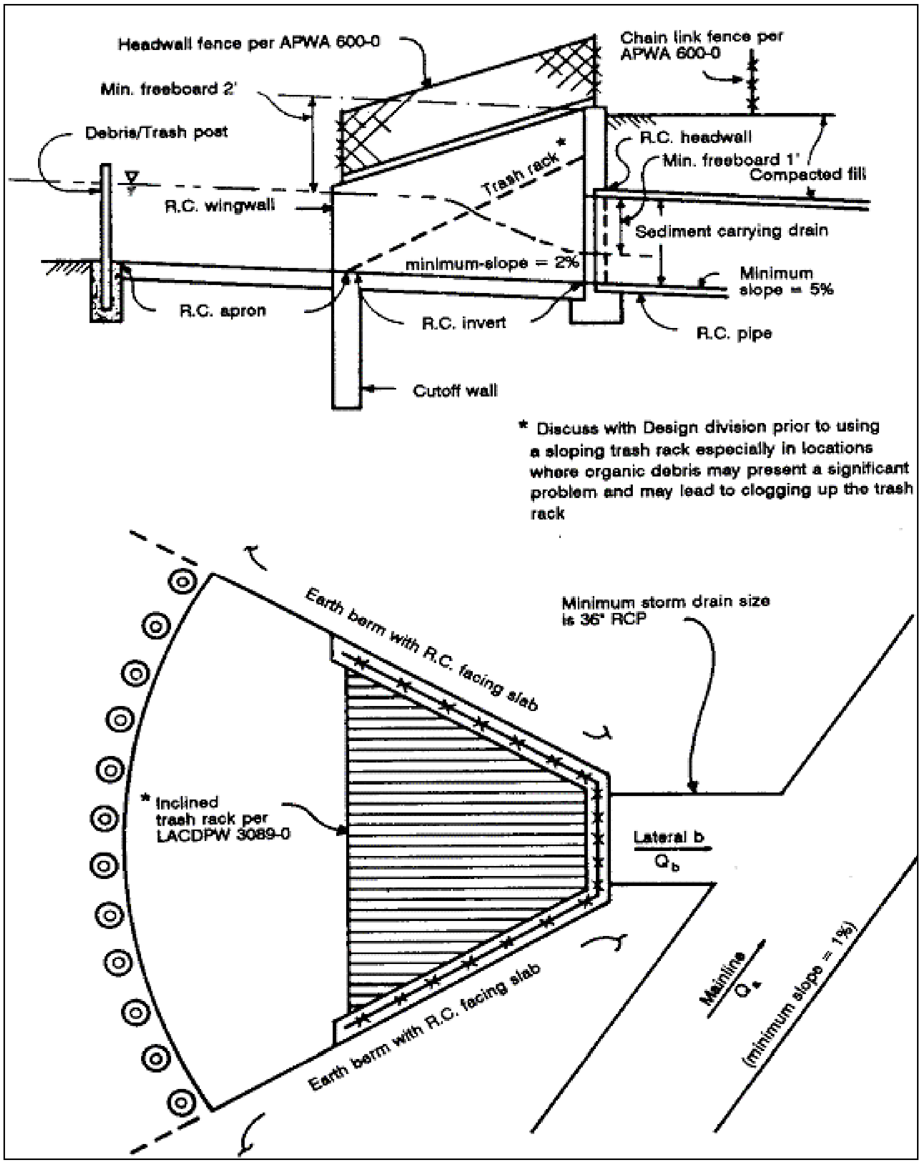

Figure 5.5.2
Sediment Carrying Inlet and Storm Drain

5.6 INLET AND OUTLET DESIGN

Transition Design

Table 5.6.1 lists design considerations for transitions between different types of channels.

Table 5.6.1
Transition Design Considerations

FROM	TO	DESIGN CONSIDERATIONS
Soft-bottom channel	Hard-bottom channel	• Provide adequate cut-off at beginning of concrete channel • Increase inlet slope to accelerate the flow to limiting deposit velocity (V_l) • Provide smooth transition angles • If transition is from an unimproved channel, extend wing walls to the floodplain limits
Hard-bottom channel	Soft-bottom channel	• Use energy dissipation structure to reduce velocities to natural velocity • If concrete channel outlets into an unimproved soft bottom channel, design the outlet to direct the flow to its natural path. Extend wing walls to flood plain limits
Unimproved channel	Stabilized Soft-bottom channel	• Extend wing walls to flood plain limits • Provide invert stabilizer at beginning of stabilized channel to control the grade • Provide smooth transitions
Stabilized Soft-bottom channel	Unimproved channel	• Design the outlet to direct the flow back to its natural path • Provide invert stabilizer at the end of stabilized channel to control the grade
Hard-bottom channel	Hard-bottom channel	• Keep velocities above limiting deposit velocity and below 40 feet per second

Energy Dissipation

Storm drains and channels which outlet into a natural or improved soft bottom channel will generally require an energy dissipater to reduce velocities to a non-erosive magnitude. The type of dissipater structure depends on the approach velocity and the desired natural velocity. Consult Public Works' Design Division for type and design of energy dissipation structure.

In case of sediment laden-flows (bulked flow), the sudden drop in velocity usually causes deposition to occur at the upstream of the energy dissipation structure. Design the dissipater structure to minimize deposition and include provisions for access to remove the deposited sediment.

5.7 FLOODPROOFING OF DEVELOPMENTS IN NATURAL WATERCOURSES

Developments within the natural watercourse boundaries (that have been approved by Land Development Division) requiring flood proofing should follow the criteria in Section 5.2 to determine the scour depth and embankment height of local protection. Developers must prove through use of hydraulic and sediment transport analyses that their development will not have any adverse effect on neighboring properties such as increased flood hazard, scour, or deposition. Contact Land Development and Building & Safety Divisions for Public Works' drainage requirements.

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[1] Quasi-Dynamic Sediment Routing Model - Developed by Simons, Li and Associates, Inc.
[2] Scour and Deposition in Rivers and Reservoirs - Developed by U.S. Army Corps of Engineers
[3] Mathematical Model for Erodible Channels - Developed by Howard H. Chang, Ph.D

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LIST OF SYMBOLS

SYMBOL		DEFINITION
A	=	Total drainage area, including developments
A_i	=	Individual drainage area
A_u	=	Total undeveloped area
A_d	=	Total developed area
A_{di}	=	Developed area, in area A_i
a	=	Bulking constant (fixed throughout the hydrograph)
a	=	Length which abutment protrudes into the flow
α	=	Angle of attack
BF	=	Bulking factor
$BF_{(Ai)}$	=	Bulking factor based on area, A_i
b	=	Pier width
C	=	Capacity of sediment control structure
D	=	Distance to the nearest downstream stable point
d	=	Debris blockage
DP	=	Debris production
DPA	=	Debris potential area
DPR	=	Debris production rate
$DPR_{(A)}$	=	Debris production rate based on the total drainage area A
$DPR_{(Ai)}$	=	Debris production rate based on area, A_i
$DPR_{(Au)}$	=	Debris production rate based on the total undeveloped drainage area, A_u
$DPR_{i(Ai)}$	=	Debris production rate based on area A_i in DPA zone i
d_{85}	=	Size of sediment for which 85 percent of the sediment is finer
FB	=	Total freeboard
G	=	Multiplication factor
g	=	Acceleration of gravity
H	=	Nominal height of grade control structure
H_c	=	Height of debris cone
H_s	=	Height of spillway above natural ground
h	=	Bed form height
L	=	Reach length
L	=	Length of pier
K_1	=	Scour depth adjustment factor
K_2	=	Local scour adjustment factor

SYMBOL		DEFINITION
K_3	=	Local scour depth adjustment factor to account for debris blockage around pier
n	=	Bulking exponent (fixed throughout the hydrograph)
n	=	Manning's roughness coefficient for the channel
Q	=	Clear or burned discharge
Q_B	=	Bulked or burned and bulked discharge
Q_b	=	Bulked flow discharge
Q_{cap}	=	Department's Capital Flood discharge
Q_s	=	Sediment discharge
$Q_{S\ in}$	=	Sediment supply into the reach
$Q_{S\ out}$	=	Sediment transport out of the reach
Q_w	=	Water discharge (clear or burned)
Q_{10}	=	10 year runoff discharge
$\underline{\mathrm{R}}$	=	Radius of curvature
S_o	=	Existing slope
S_e	=	Energy slope
S_D	=	Sediment/Debris cone slope
S_{eq}	=	Equilibrium slope
S_g	=	Specific gravity
S_N	=	Natural slope of the stream
V	=	Velocity of flow
V_l	=	Limiting deposit velocity
V_{lmax}	=	Maximum limiting deposit velocity
W	=	Channel top width
Y	=	Depth of flow
Y_{agg}	=	Long-term aggradation
Y_{ga}	=	General aggradation
Y_{se}	=	Superelevation
Z_{bs}	=	Bend scour
Z_{deg}	=	Long-term degradation
Z_{gs}	=	General scour
Z_i	=	Low-flow incisement
Z_{ls}	=	Local scour
Z_{tot}	=	Total potential vertical adjustment

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[1] This is a partial list.