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FLOOD HAZARD, SEDIMENT MANAGEMENT, AND WATER FEATURE ANALYSES, HAHAMONGNA WATERSHED PARK PASADENA, CA

Prepared for

Takata Associates

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1. INTRODUCTION

The Hahamongna Watershed Park (also known as the Devil's Gate Reservoir) is a 300-acre area at the foot of the San Gabriel Mountains, in Pasadena, California (Figure 1.1). The basin is upstream of Devil's Gate Dam on Arroyo Seco, one of the main tributaries to the Los Angeles River (Figure 1.2). Located at the foot of the most geologically dynamic mountains in the world, the basin receives periodic high-intensity floods that carry very high sediment loads. The basin is currently used by several stakeholder groups for multiple purposes. Los Angeles County Public Works Department (LACDPW) owns Devil's Gate Dam and operates it for flood control and sediment management. LACDPW has a permanent easement within the basin for the purposes of flood control and water conservation. The City of Pasadena owns the park area, manages its recreational use, and also uses the basin for groundwater recharge and municipal water supply. The basin is adjacent to the Jet Propulsion Laboratory (JPL), a NASA research center affiliated with Caltech University.

With the increasing recreational use of the basin the City of Pasadena (the City) has begun a process to develop a Master Plan to enhance the open-space, natural habitat, and aesthetic qualities of the area. This is an excellent opportunity for the City to develop a first-class open-space area within urban Pasadena and greater Los Angeles. The City has hired the landscape architecture firm Takata Associates to develop the Master Plan. Philip Williams & Associates, Ltd. (PWA), consultants in hydrology, has been hired to provide supplemental information on hydrology and geomorphology within the Hahamongna Watershed Park to inform the on-going master planning process.

This report summarizes results from PWA's study, beginning with an outline of PWA's findings and recommendations. The objectives of the study are presented, along with a description of relevant site characteristics. The main hydrologic opportunities and constraints on the master planning process are reviewed, and finally, results from PWA's analyses are described as they relate to the study objectives. It is intended that recommendations for the Master Plan will promote self-sustaining natural riparian habitat while addressing issues related to flood hazards, erosion control, sedimentation, stream stabilization, and water resource management within the park.

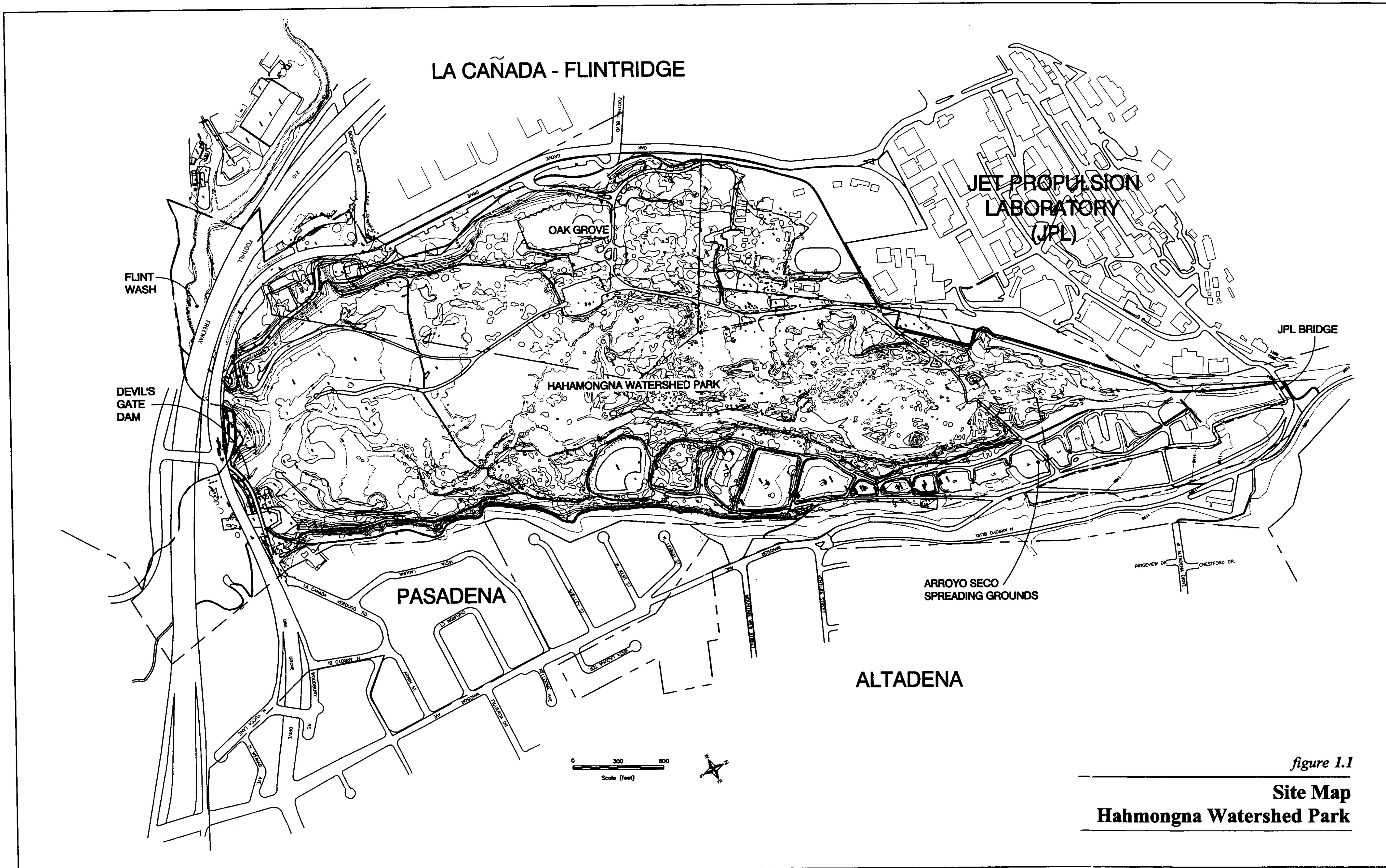

figure 1.1

Site Map
Hahmongna Watershed Park

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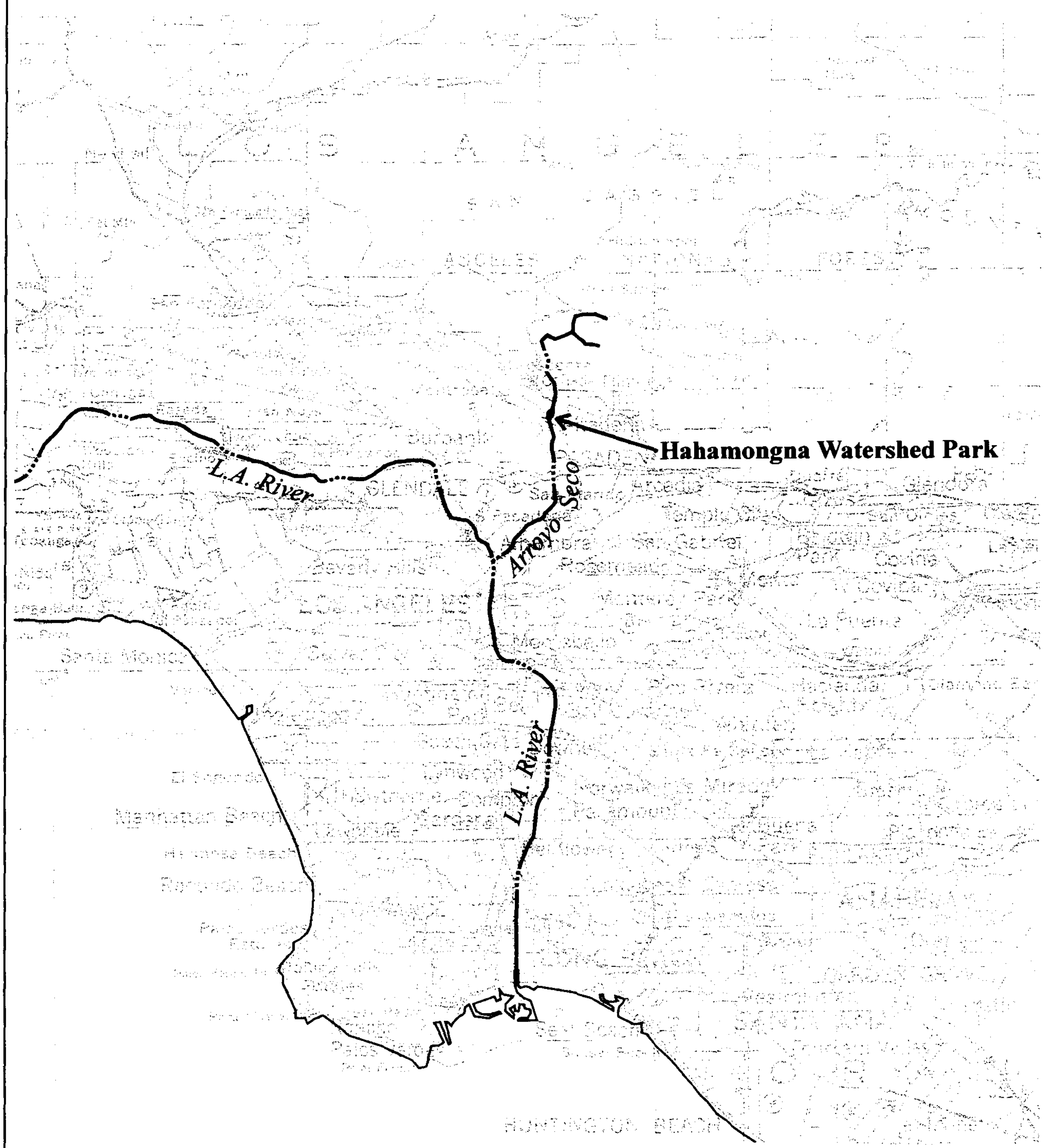
figure 1.2

Location Map



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2. FINDINGS AND RECOMMENDATIONS

2.1 FINDINGS

Several important findings resulted from this study:

a. The Hahamongna Watershed Park is located in the actively managed flood control operation zone of Devil's Gate Dam. As a result, stored water temporarily inundates significant areas of the park during large flood events.

b. In addition, there are risks of channel migration, erosion, and deposition by debris flows, especially in the upper part of the park area. The Arroyo Seco active channel zone—composed of the areas where the channel has been located over time—was found to cover a large portion of the Hahamongna Watershed Park.

c. Approximately 35% of the original reservoir capacity (4601 acre-feet) has been lost due to sedimentation over the past 80 years, despite significant maintenance excavation by LACPWD. For any given year the sediment delivery to the park is difficult to predict. However, over the long-term, the average annual sediment delivery to the basin is approximately 90 acre-feet. In order to maintain flood control capacity in the reservoir an active on-going sediment removal program is required.

d. Modeling results and historical geomorphic analysis show that during most flood events there is significant sedimentation in two areas: 1) in the upper portion of the park where the channel rapidly widens adjacent to recharge ponds 2 through 9; 2) downstream, adjacent to the Devil's Gate Dam. During smaller events sedimentation in the reach between these areas seems to be minimal. During large events sedimentation occurs throughout the park, including the two areas described and the reach between them.

e. A water feature is physically feasible for the Hahamongna Watershed Park. A water feature located near the dam would likely require little implementation engineering or maintenance. However a water feature at this location would be seasonal and temporary due to limited runoff and LACDPW's flood control capacity sediment removal activities. Furthermore, a water feature near the dam would not provide significant aquatic habitat due to its seasonal nature and high suspended sediment concentrations and could not be used for active recreation. Smaller water features in the upper area of the park would provide superior aquatic habitat, recreational opportunities, and could be full of water year-round. However, this type of water feature would require significant engineering and maintenance efforts, and would provide a type of aquatic habitat that is not native to the Hahamongna Watershed Park.

f. Sustainable riparian and wetland habitats are dependent on a seasonally high groundwater table in the park. Groundwater table levels are affected by seasonal runoff, sedimentation, reservoir operations, surface water spreading, and groundwater pumping.

g. The City of Pasadena currently diverts virtually all low flows (up to 25 cubic feet per second (cfs)) in Arroyo Seco from a point upstream of Hahamongna Watershed Park. Diverted flows are routed through a network of pipes to the Arroyo Seco Spreading Grounds, a series of ponds along the east side of the park used for groundwater recharge.

h. Since the cost of pumping groundwater (approximately $91/acre-foot) is relatively low in comparison to the cost of imported Metropolitan Water District (MWD) water (approximately $431/acre-foot), the City of Pasadena has an incentive to pump groundwater for municipal water supply, and the City also has an incentive to maximize groundwater recharge credit with the Raymond Basin Management Board (RBMB). The RBMB oversees the management of groundwater resources in the Pasadena area.

i. The Arroyo Seco Spreading Grounds may not be the most efficient or cost-effective way to recharge groundwater in the Hahamongna Watershed Park area. Hydraulic conductivity rates in the ponds have been found to be much lower than in other areas of the park. This is likely due to siltation and maintenance compaction. Furthermore, significant leakage from one of the ponds was discovered during a site visit, allowing ponded water to flow back into the Arroyo Seco channel without percolating.

j. It was found that increased groundwater recharge might be achieved in the Hahamongna Watershed Park if natural flows are restored to the Arroyo Seco channel and if ponding was allowed to occur regularly adjacent to the dam. Relative to the existing spreading grounds, a larger ponding area could be achieved with ponding at the dam and regular reservoir capacity excavation would maintain high hydraulic conductivities. Furthermore, natural flows in the Arroyo Seco channel could percolate most, if not all, of the low-flows currently diverted to the spreading grounds.

k. Restoring natural flows to the Arroyo Seco channel through the Hahamongna Watershed Park and obtaining groundwater percolation credit for these flows and ponded water at the dam would require a significant adjustment to the adjudication of water rights for the City of Pasadena. Furthermore, runoff inflow and outflow from the park would have to be estimated more precisely to accurately quantify groundwater percolation credit.

l. Partial re-establishment of "natural" riparian habitat—historic pre-dam habitat—within the park seems feasible if natural flows are restored to the Arroyo Seco channel through the park, and if LACDPW sediment maintenance excavation is localized leaving undisturbed areas for habitat development. However, without these actions the opportunity for areas of "natural" riparian habitat will likely be minimal and incidental to the overall project. With increased protection from human disturbance, native non-riparian habitat will likely establish in the park. Habitat will likely also establish around any ponded water features in the park, although this habitat may not be part of the natural (historical) ecosystem of the area.

2.2 RECOMMENDATIONS

Conflicting resource management objectives, such as LACDPW's flood control objectives and the City of Pasadena Department of Water and Power's groundwater recharge objectives, limit the ability of the City of Pasadena Parks and Natural Resources Division to achieve their stated habitat restoration goals. However, given these conflicting objectives, the following hydrologic and geomorphic recommendations describe PWA's understanding of the most feasible way to realize the opportunity for open space and natural habitat restoration within the Hahamongna Watershed Park:

1. Park facilities and improvements need to take into account hydro-geomorphic hazards mapped in the figures accompanying this report. Structures should either be built outside of the designated flood and debris hazard zones or they should be constructed such that they are effectively "flood-proof," i.e. can tolerate inundation without damage.

2. Sediment maintenance excavation should generally be limited to two areas, one in the upper portion of the Park and one directly adjacent to the dam. These two areas correspond to the two primary areas of sediment deposition (Figure 7.27). Ideally, both excavation areas should be excavated using a performance-based management strategy. For the greatest cost-effectiveness excavation should not take place on a prescribed schedule of prescribed quantities but should occur after sediment has deposited to a set tolerance elevation. The quantity of excavation should return the area to a preferred design elevation. Tolerance and design elevations are suggested in Section 7.3.3.3. Since this performance-based removal strategy will allow more vegetation growth than a prescribed annual or biennial removal strategy it is crucial that agency permits allow un-mitigated cyclical vegetation destruction in the excavation areas. Without this permission performance-based management is

likely infeasible due to mitigation requirements. If appropriate permits can not be obtained, an annual or biennial excavation schedule could be adopted in the two designated areas.

3. The primary function of Devil's Gate Dam and its accompanying reservoir easement is flood control. As such the dam should be operated so that downstream flow requirements are met and flow-assisted sediment transport from the reservoir is maximized (to maintain reservoir capacity and reduce sediment excavation requirements). This also implies an operation strategy that keeps the dam's release gates relatively free of sediment blockage. As secondary goals the dam should be operated to minimize water-surface elevations in the Hahamongna Watershed Park during large flood events (to protect park elements), while at the same time seasonally and temporarily holding water to maximize the potential for groundwater recharge. Current dam operations during flood events seem to adequately balance these considerations. However, the opportunity for flow-assisted sediment transport after large flood events using residual flows in Arroyo Seco should be examined in greater detail, as should the potential for seasonal temporary ponding behind the dam for increased groundwater recharge. A sediment transport monitoring program at Devil's Gate Dam should also be instituted to better determine flow-assisted sediment transport quantities.

4. The main water feature within the park should be the main channel of Arroyo Seco. Historically, Hahamongna Watershed Park was primarily a riparian zone. A flowing stream is likely the only water feature that would be sustainable within the park. Sustaining a different type of water feature within the park, such as a pond, would likely require a significant amount of engineering and maintenance effort to convey water to the correct location and to keep it free of sediment. The exception to this might be a water feature located downstream, adjacent to Devil's Gate Dam. However, it is recognized that a water feature adjacent to the dam would have to be seasonal and temporary due to extreme fluctuations in runoff and LACDPW excavation requirements.

5. There is potential for restoring a riparian zone within Hahamongna Watershed Park if natural flows are restored to the Arroyo Seco channel through the park and groundwater percolation is allowed to occur adjacent to the dam. A water feature at the dam could serve both as a sediment excavation site and the primary groundwater percolation site in the park. This water feature would also provide a higher local water table, encouraging habitat upstream. The water feature itself would not provide significant aquatic habitat due to high suspended sediment concentrations in pond water. It is recognized that such a water feature would be seasonal and temporary due to extreme fluctuations in runoff quantity and LACDPW's requirement to excavate sediment from their easement to maintain flood control capacity.

6. If the City is inclined to continue water diversions from the Upper Arroyo Seco and maintain separate upstream percolation ponds, these ponds should be concentrated upstream, north of the existing pond 11, perhaps on both sides of the proposed upstream excavation area. The outlet structures of the downstream-most percolation ponds should be directed back into the creek. Between the percolation pond outlet structures and the downstream excavation area near the dam there should be a reach of relatively undisturbed creek channel and floodplain (Figure 7.27). This reach would serve as the main riparian habitat zone in the park. Although less-desirable than simply halting upstream diversions, this strategy could allow for the creation of a limited amount of semi-natural riparian habitat. This reach would be subject to sedimentation during larger events and would therefore require periodic maintenance excavation. However, the frequency of this maintenance is expected to be less than for Areas 1 and 2.

7. Pumping water back from a temporary water feature near the dam to the Arroyo Seco Spreading Grounds is feasible and may increase the amount of groundwater recharge achieved at the spreading grounds. This would provide a means of percolating flood-waters that were originally too sediment-laden to divert from the upper Arroyo during a flood event. Sediment would settle out near the dam and then clearer water could be pumped to the spreading grounds. However, PWA does not recommend this since it would require significant engineering and cost to implement and maintain, and since it would add structural and mechanical elements to the park area.

3. OBJECTIVES

The following sections describe the three specific objectives of PWA's sediment and water resource management study: evaluating flood hazards within the park, development of a sediment maintenance strategy for the park, and assessing the feasibility of a water feature in the park.

3.1 FLOOD HAZARDS

As part of the Master Plan, several new structures and landscaped areas are proposed in the park, including an interpretive center, picnic and camping areas, athletic fields, and pedestrian and equestrian trails. A major park planning objective is to minimize risk of flood hazards in the park. These hazards include damage to park features due to inundation from reservoir ponding, channel overflow, or debris flows, and damage to park features from either channel migration or reservoir sedimentation.

One important purpose of PWA's study is to assess hydrologic and geomorphic hazards throughout the basin so structures and landscaped areas can be located with a clearer understanding of the risks associated with any given site. Results from this study are used to locate areas of high hazard risk, so that vulnerable park elements can be located elsewhere. These areas are determined through the results of hydraulic and sediment transport modeling within the basin, as well as the examination of historical records of channel configuration to identify the active channel zone.

Since water levels in the basin largely are dictated by the operation of the Devil's Gate Dam, a sub-objective was to qualitatively examine dam operations and suggest ways in which Dam operation could be adjusted to maximize benefits to the park area. However, the re-operation of Devil's Gate Dam must be considered in light of the important constraints on dam operations, including LACPWD flood control criteria and downstream channel issues. Operation of Devil's Gate Dam is integrally linked to flooding and sediment transport processes within the basin.

3.2 SEDIMENT MAINTENANCE STRATEGY

Massive quantities of sediment are seasonally deposited in the Hahamongna Watershed Park by flood waters. Managing this large inflow of sediment to the basin is an important planning issue for the Watershed Park. Currently sediment management and removal are the responsibility of LACDPW. The LACDPW's sediment removal objective is the restoration of the flood control capacity of the reservoir. An important objective of this study is to produce an integrated strategy for dealing with the large amounts of sediment entering the park from the upper watershed. The strategy must identify preferred locations, frequencies, and quantities for sediment maintenance excavation, and it should maximize opportunities for flow-assisted sediment transport through the dam, which reduces requirements for excavation. The strategy must also be in harmony with the various other objectives of the Master Plan, including flood control, groundwater recharge, recreational use, and natural habitat within the basin.

3.3 WATER FEATURE FEASIBILITY

Previous years of planning for the Hahamongna Watershed Park have revealed a strong desire among community members and stakeholders for the creation of a water feature within the park. This objective is somewhat flexible: the water feature may be permanent or seasonal, and may take either a deep-water lake form or a shallow-water wetland form. Access to the water feature is desirable for passive recreational activities such as fishing and bird watching. In the past, suggestions for the form of this water feature have included a seasonal lake immediately upstream of Devil's Gate Dam and/or a smaller pond/wetland in the upstream portion of the basin. However, presently the LACDPW has clearly stated that any kind of recreational water feature in their flood control easement is unacceptable.

For this study, PWA's objective was to preliminarily examine the feasibility of implementing and maintaining a recreational water feature in the Watershed Park. Factors such as dam operations, flood control, safety, pumping requirements, and sedimentation should be considered in relation to the potential for a recreational water feature. The preferred water feature would be one that is passive, or self-sustaining, requiring very little effort to implement or maintain.

4. SETTING

A suite of natural processes influences the physical functioning of steep chaparral drainage basins in southern California, such as the Arroyo Seco Watershed. In this section, the setting of the Hahamongna Watershed Park is described. Key hydrologic and geomorphic characteristics of the park are examined. Interactions between regional phenomena (e.g., climate, geology) and processes at smaller spatial and temporal scales (e.g., fire, floods, debris flows) are described, with a focus on their effect on erosion and sedimentation within the drainage basin. Figure 4.1 illustrates the relationship between physical setting and watershed processes operating at a range of spatial and temporal scales.

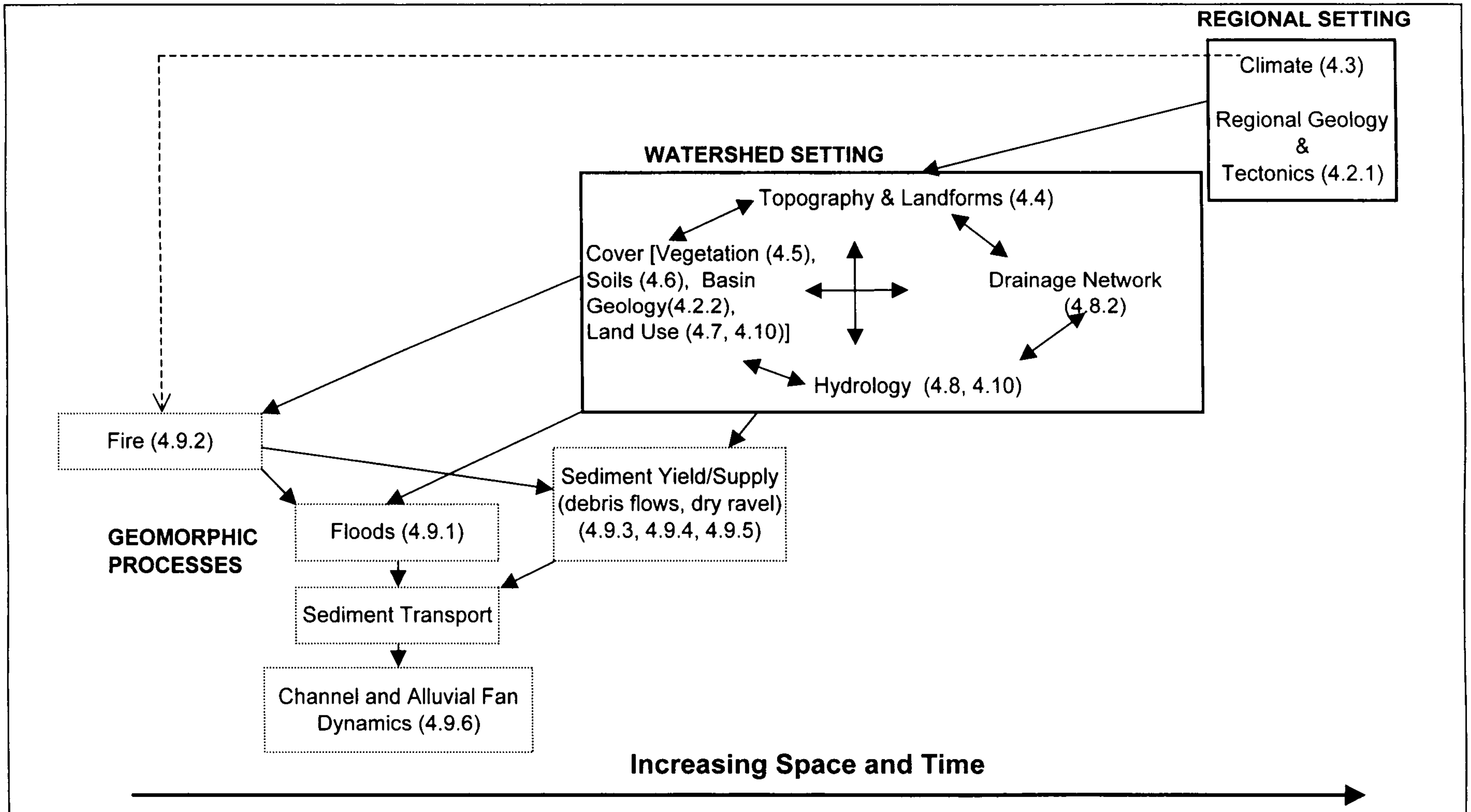
4.1 SITE LOCATION

The study site, the Hahamongna Watershed Park, is located on the northern edge of the City of Pasadena, between the communities of Altadena (to the east) and La Cañada-Flintridge (to the west), in the greater Los Angeles area. The site is located at the foot of the San Gabriel Mountains in the geomorphically active alluvial fan zone. Arroyo Seco, a major tributary to the Los Angeles River, flows through the middle of the site. Portions of the proposed park site lie within LACDPW's Devil's Gate Dam and Reservoir facility. This facility consists of Devil's Gate Dam and an upstream reservoir easement for flood control and water conservation. Figure 1.2 shows the site location.

4.2 GEOLOGIC HISTORY

4.2.1 Regional Geology

Large-scale changes in the interaction of the continental plate and the oceanic crustal plate caused widespread deformation throughout western North America beginning approximately 20 million years ago. A structural depression formed and led to the near continuous deposition of sediments over the submerged Los Angeles Basin over the past 20 million years (Fall, 1981). Besides marking the onset of uplift in the San Gabriel area, the period from latest Oligocene through early Miocene time (30 to 20 million years ago) also saw the inception of movement along most of southern California's major faults.



This factors shown in this figure are not exhaustive, but represent those key concepts described in this report.

Numbers in parentheses refer to sections of the text.

NOTE: Solid arrows indicate general relationships between factors. Dashed arrow indicates specific relationship between factors.

figure 4.1

Conceptual Diagram of Relationships Between Watershed Characteristics

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Tectonic activity has been intermittent since the late Pleistocene epoch (~0.5 million years ago). Uplift of the southern California area within the last few hundred thousand years has caused the basin to emerge, marine deposition to cease, and deformation of preexisting bedrock.

Fluvial sedimentation, largely the work of the Los Angeles, San Gabriel, and Santa Ana rivers, dominated the Los Angeles coastal plain and continued to fill the basin floor with alluvium. The result of this long history of marine and terrestrial deposition has been the accumulation of an over 5.9 mi (9,450 m) thick (at its deepest point) layer of sediment (Fall, 1981) that conceals most of the rocks in the lower Los Angeles basin. Exposed intrusive and metamorphic bedrock is generally limited to steep hills throughout the landscape. Coincident with regional uplift, alternating periods of alluviation and valley cutting are indicated by paired terraces juxtaposed above present day channel beds.

4.2.2 Watershed Geology

Smith (1986) provides an extensive review of the geology specific to the Pasadena area, including the Arroyo Seco watershed upstream of Devil's Gate Reservoir. The rocks of the region found within the Arroyo Seco watershed can be grouped into two distinct types: crystalline "basement" rocks (undifferentiated igneous and metamorphic rocks underlying sedimentary rocks of primary interest), and recent sedimentary deposits. The majority of rock within the upper Arroyo Seco Basin are crystalline basement rocks, ranging in age from Cretaceous to PreCambrian (65+ million years old). These have been classified by Smith (1986) as diorite, granodiorite, and gneiss. The sedimentary strata in the Arroyo Seco basin are mainly of Quaternary age (<1.5 million years old), and consist of uplifted and abandoned stream terraces alongside Arroyo Seco and the Devil's Gate Reservoir.

The closest known fault is located approximately 1 mile north of Devil's Gate Dam in the Sierra Madre Fault Zone (SMFZ). The Main Branch of the Fault Zone intersects Arroyo Seco at an elevation of approximately 2100 feet NGVD (National Geodetic Vertical Datum) and trends northwest through the entire basin. Shorter parallel faults associated with the SMFZ have been identified below Gould Mesa at the Jet Propulsion Laboratory and approximately 0.5 miles farther upstream near the Los Angeles Vina Hospital and Sanatorium (Smith, 1986). The most recent surface rupture due to earthquakes on these faults is thought to have occurred in Pre-Holocene time (greater than 11,000 years ago).

4.3 CLIMATE

The region surrounding the Arroyo Seco basin has a Mediterranean-type climate, with warm, dry summers and mild, relatively wet winters. Rainfall distribution is highly seasonal, with nearly all precipitation falling during the winter months. Rainfall varies dramatically with respect to elevation.

Temperatures in the Los Angeles region are usually moderate. Winter temperatures rarely drop below freezing. Average daily minimum temperatures for January at Los Angeles civic center and Mount Wilson (the highest peak in the San Gabriel range) are 48 degrees and 35 degrees Fahrenheit respectively. The summer months occasionally bring temperatures above 100 degrees Fahrenheit and are normally without rain. Average daily maximum temperatures for July are 84 degrees and 80 degrees Fahrenheit for the two locations respectively.

Hot summer and early fall winds from the Mojave Desert, north of the San Gabriel Mountains, occasionally blow through the Hahamongna Watershed Park area. Known as "Santa Ana" winds, under certain conditions these winds can greatly increase the spread of fire in San Gabriel Mountain watersheds such as the Arroyo Seco. As such they can play an important role in the delivery of massive quantities of sediment to the alluvial fan zone of the San Gabriels (Section 4.9.2).

4.4 TOPOGRAPHY

The San Gabriel mountains are currently being uplifted at a rate of about 24 inches per century, and denudation rates are around 4 to 8 inches per century (Scott and Williams, 1974). This suggests a "youthful" mountain range where down-cutting has not yet reached equilibrium with uplift rate. This rapid uplift has rejuvenated the drainage system, producing high relief and pervasive drainage dissection (Cooke, 1984).

Retzer *et al.* (1951), in a study of 46 mountain watersheds in the Los Angeles River catchment of the San Gabriel Mountains, gave the average slope as 68%, and reported that slopes exceeded 70% in almost two-thirds of the area mapped. These data emphasize the steepness of the San Gabriel mountain slopes and their potential instability, given that many exceed the normal angle of repose of unconsolidated material (~70%). As a result, the valley-side slopes in the mountains are exceptionally active environments in which rates of debris production and removal are extremely rapid by comparison with those of surrounding areas and of different climatic regions (Cooke, 1984). Many of the canyons along the San Gabriel range front also have vertical-walled inner gorges, as much as 130 feet deep.

4.4.1 Watershed Topography

The Arroyo Seco basin is characterized by sharp contrasts in terrain that range from the high relief of the rugged San Gabriel Mountains to the alluvial surfaces in the valley below. The highest point in the watershed is Strawberry Peak at 6,164 feet NGVD. In this upper mountainous portion of the watershed, the hillslopes are deeply dissected and form very steep side canyons with rectilinear or convex-concave hillslopes. The steep gradients promote efficient delivery of hillslope materials to the stream channels. Within the mountainous watershed, channel cross-sections are commonly V-shaped.

4.4.2 Watershed Longitudinal Profile

A longitudinal profile of an alluvial channel shows the elevation of the channel thalweg with distance along the thalweg. The channel slope along the profile reach can be readily observed. Channel slope is a useful predictor of channel dynamics, since particular types of channel processes and morphologic features dominate the alluvial system for discrete slope intervals. A longitudinal profile is also useful in identifying specific points of abrupt slope change or "knickpoints" along the channel. Knickpoints generally develop in response to the discharge regimen and by the structure and composition of the bed and bank materials of the river.

A longitudinal profile of the Los Angeles River up from the Pacific Ocean through the Arroyo Seco to Mount Lawlor is shown in Figure 4.2. The profile shows a steep, concave channel slope along the uppermost 6 miles (~10 km) of the river, until the edge of the mountain front is encountered. Downstream of this for the next 30 miles (~50 km), the channel slope decreases more gradually. For the lowermost 20 miles (~30 km) to the Pacific Ocean, the Los Angeles River profile shifts to nearly flat. The Devil's Gate Dam is situated within the transitional zone from the steep mountainous fronts to the fringing alluvial plains.

When this information is plotted as slope verses distance (Figure 4.3), local peaks and troughs in the channel slope are apparent. The two slope peaks in the mountainous portion of the Arroyo Seco watershed are both associated with local across-channel structures. These features are unlikely to pose any important large-scale constraints on channel processes.

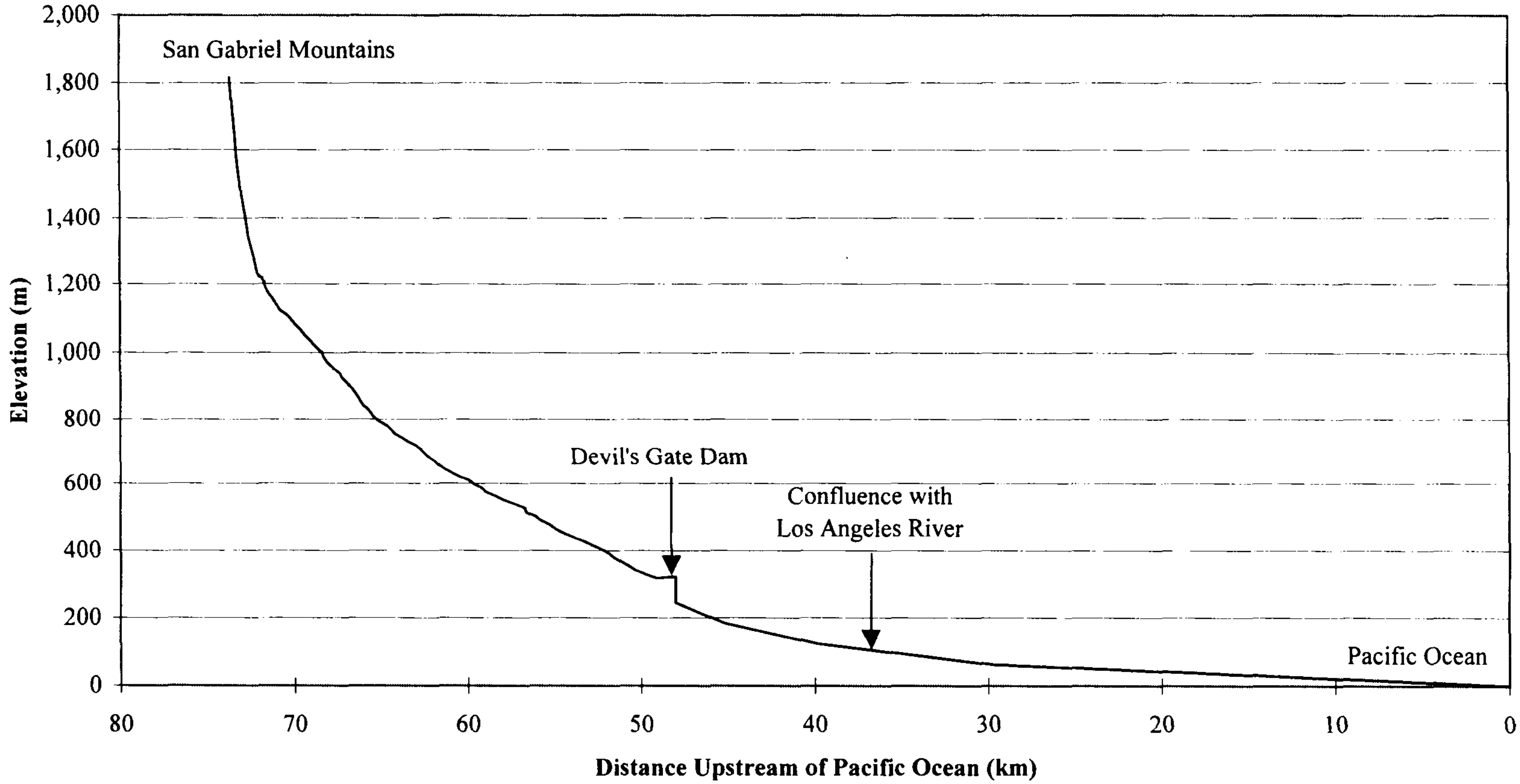

figure ***4.2***

Longitudinal Profile of the Los Angeles River and Arroyo Seco Watershed

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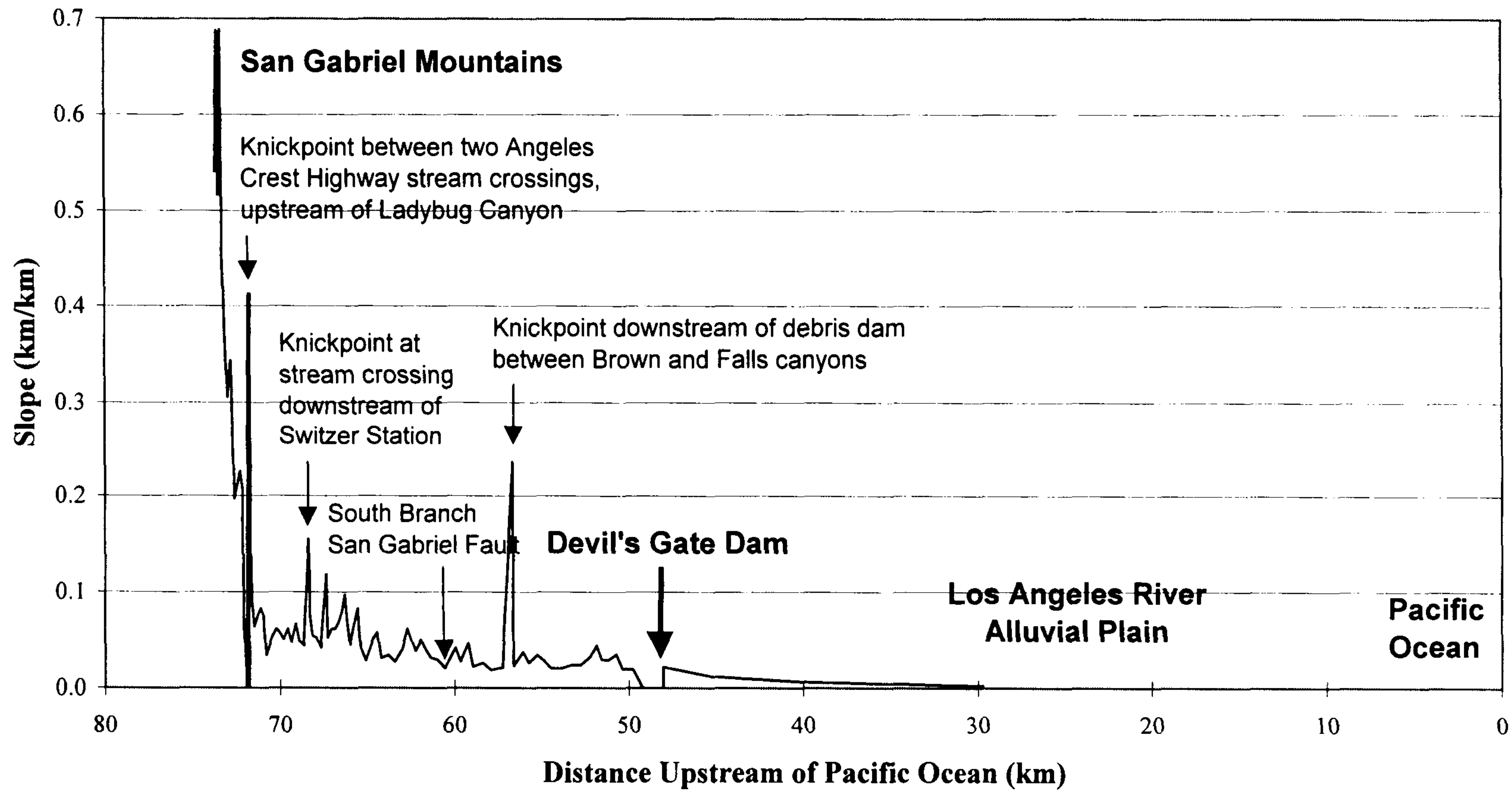

figure ***4.3***

Channel Slope along Longitudinal Profile of L.A. River and Arroyo Seco

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4.4.3 Park Topography

Hahamongna Watershed Park is situated in what was formerly the Arroyo Seco canyon, much like the areas upstream of the JPL bridge and downstream of Devil's Gate Dam. After the dam was constructed, sediments began to accumulate behind it. This deposition raised the ground-surface in the reservoir area and created a broad flat sediment plain between the canyon walls. Today this flat sediment plain gently slopes from an upstream elevation of approximately 1100 feet NGVD at the JPL bridge to a downstream elevation of approximately 988 feet NGVD at the dam face. The former canyon walls slope steeply up from the sediment plain at its edges. The sediment plain itself is quite irregular due to erosion and historical excavation within the reservoir.

The most recent topographic map of the site is an aerial survey from November 1995, and was done by LACPWD (Figure 4.4). There are many distinctive topographic features apparent within the Hahamongna Watershed Park. The Arroyo Seco Spreading Grounds are elevated along much of the eastern side of the reservoir. In that vicinity there are also horse stables, an informal baseball field, and several maintenance roads providing access to the spreading grounds. Oak Grove, a recreational area, is located west side of the reservoir. The Arroyo Seco channel runs through the middle of the reservoir, distinct and well-defined in some portions of the park, broad and undefined in others.

The outlet of Flint Wash, a tributary drainage to the reservoir, is located directly west of Devil's Gate Dam. North of this outlet is a relatively uniformly graded area where LACPWD has conducted maintenance excavation. Depressed portions of this area contain ponded water. In the southeastern corner of the park the outfall of a large stormwater culvert has created an additional distinct channel that drains toward the dam.

4.4.4 Park Longitudinal Profile

To obtain a detailed longitudinal channel profile through Devil's Gate Reservoir, the Arroyo Seco thalweg (the lowest point in the channel) was traced on the 1995 CAD map. Thalweg elevations were then plotted as shown in the lower part of Figure 4.5. Where the channel braids (splits into multiple channels) in the upper half of the basin, separate profiles for each of the two braids were constructed for comparison. The average channel slope through the reservoir is 0.023, which is typical for a meandering channel. The Arroyo Seco channel braids where channel slope is limited between 0.0005 and 0.01 over a distance of approximately 2000 feet. It is common for natural channels to braid at these lower slopes.

figure 4.4

1995 Topography
Hahamongna Watershed Park

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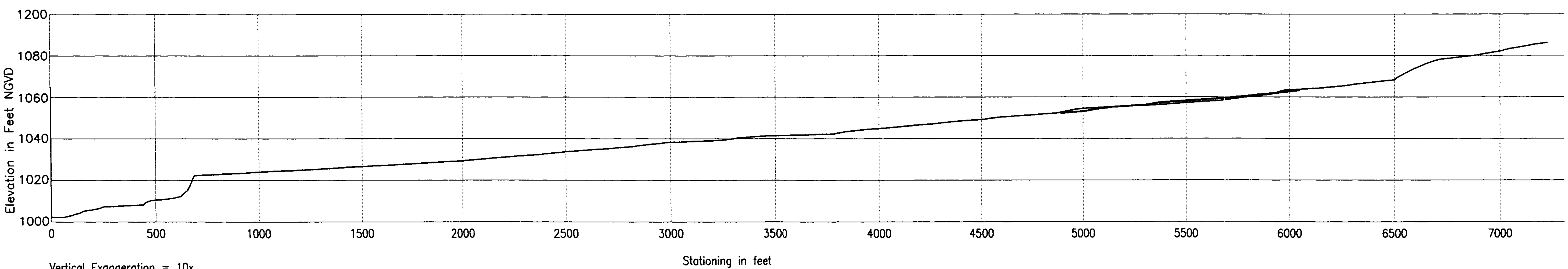

figure 4.5

Thalweg and Profile
Hahamongna Watershed Park

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4.5 SOILS

The soils in the Arroyo Seco watershed are closely related to the underlying rock types. The underlying intrusive igneous rocks of the upper Arroyo Seco watershed generally weather to coarse, sandy soils. Because slopes in the Arroyo Seco basin are often steep and unstable, soils are usually shallow, poorly developed, relatively young, and highly permeable.

There is no detailed soil survey encompassing the Arroyo Seco basin currently available from the Natural Resources Conservation Service (NRCS, formerly known as the Soil Conservation Service or SCS). However, a general soil report and map developed by the SCS is available for Los Angeles County. According to the report, soils in the Arroyo Seco watershed are sandy loams and gravelly sandy loams of the Vista-Amargosa association and occur on slope between 30 and 50 percent (USDA NRCS, 1969). These soils of the Vista-Amargosa association are generally between 14 and 38 inches deep, are well drained, have moderately rapid subsoil permeability. There do not appear to be significant areas of soils within the Arroyo Seco watershed that are subject to expansion or collapse (hydro-compaction)(Smith, 1986).

4.6 LAND USE

The Hahamongna Watershed Park area has been subject to the general urban expansion experienced by the whole greater Los Angeles area over the past 100 years. Urban communities have grown around the Watershed Park, including La Cañada-Flintridge to the west, Altadena to the east, and urban Pasadena to the south. The NASA-Caltech Jet Propulsion Laboratory is located directly northwest of the reservoir. The upper watershed area of Arroyo Seco, north of Hahamongna Watershed Park, is approximately 90% U.S. Forest Service Land and has not been subject to significant development.

Land-use within the Hahamongna Watershed Park itself includes flood management and groundwater recharge. LACDPW has a flood management easement that covers portions of the park and the majority of the groundwater recharge facilities (the Arroyo Seco Spreading Grounds). Sediment is periodically excavated from this area to maintain reservoir capacity and downstream flood control. The City of Pasadena, Department of Water and Power, operates and maintains the Arroyo Seco Spreading Grounds. These ponds are used to recharge the local groundwater aquifer beneath the park.

The Hahamongna Watershed Park and the upper Arroyo Seco watershed have also increasingly been used for recreation over the past 20 years. Hiking, mountain-biking, horse-back riding, and picnicking are all popular recreational pursuits within the basin. The two multi-purpose fields are also regularly used, as is the frisbee golf course on the west side of the park.

4.7 HYDROLOGY

4.7.1 Precipitation

Hahamongna Watershed Park is located approximately at the boundary between two different precipitation regions as designated by the LACPWD: the San Gabriel Mountains and the San Gabriel Valley (LACPWD, 1996). These two regions receive an approximate average of 27.5 inches and 17.6 inches of precipitation per year, respectively.

Several precipitation gages operated by the County are near the project site (Figure 4.6). The table below lists County precipitation stations near the project site and in the Arroyo Seco watershed. Precipitation amounts during recent water years (October to September) are given for each station. Data from the gage at the Devil's Gate Dam were used to evaluate hydrology within the park for recent years. The period of record for each gage is relatively short and is not always complete (LACDPW, 1996b, 1998). Table 4.1 emphasizes yearly and between-station variation in precipitation based on data from these stations.

TABLE 4.1 Precipitation for Stations Near Hahamongna Watershed Park and the Arroyo Seco Watershed

Station No.	Station Name	Station Elevation (feet NGVD)	Precipitation Amounts (inches)		
			1995	1996	1997
683B	Sunset Ridge	2110	43.38	21.44	21.70
612B	Pasadena Chlorine Plant	1160	42.32	19.52	21.01
453D	Devil's Gate Dam	980	29.78	19.90	10.23
280C	Flintridge-Sacred Heart	1600	44.20	20.00	-
1261	Los Angeles Canada Reclamation Plant	1800	44.25	20.30	21.26
726C	Angeles Crest Guard Station	2300	53.98	25.70	26.66
1152	Clear Creek Ranger Station	3625	48.67	24.67	26.16
57B	Camp Hi Hill (Opids)	4250	69.90	31.10	-
338C	Mt. Wilson-Observatory	5709	61.80	34.32	32.41

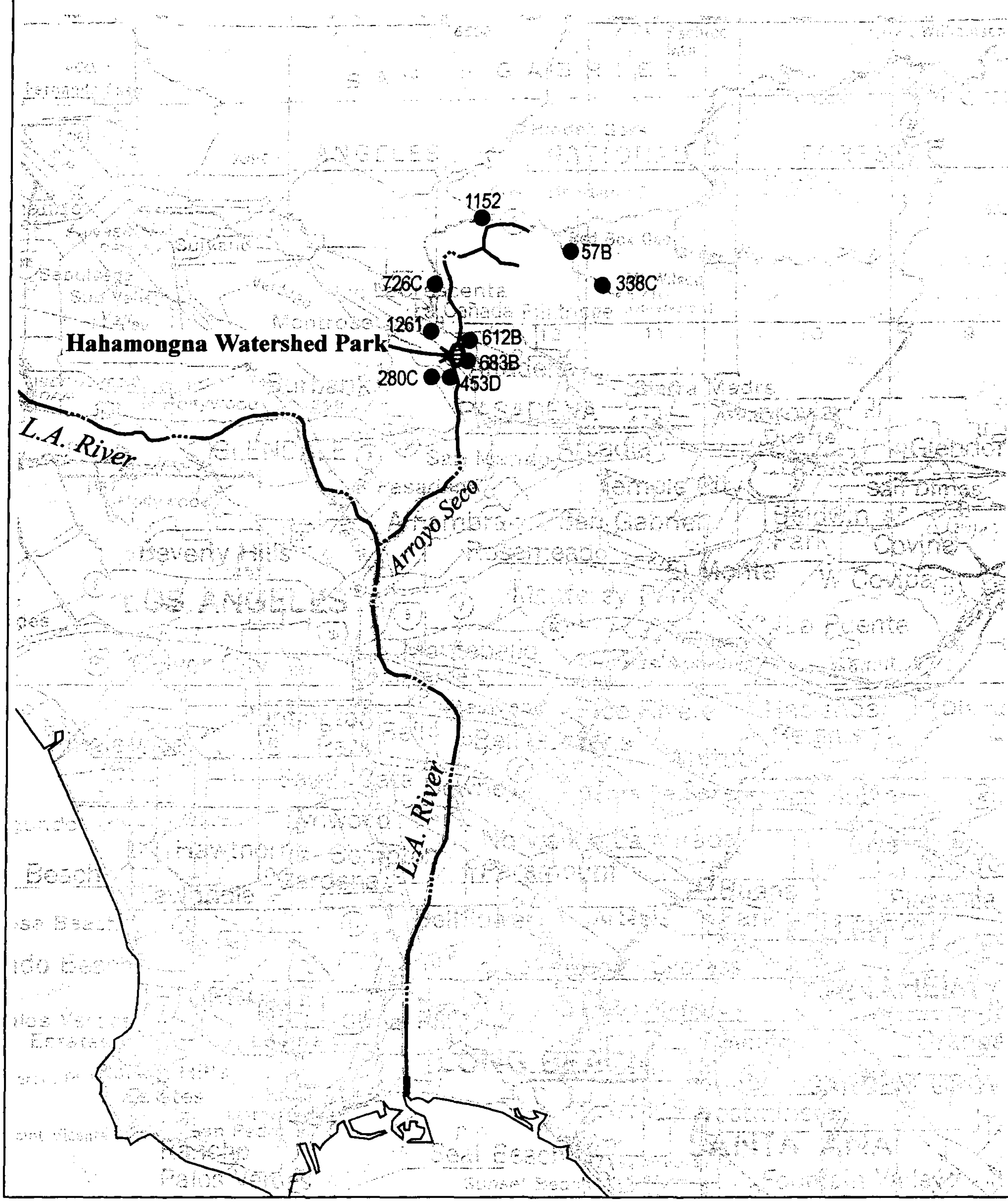
Source: LACDPW, 1996b, 1998

figure 4.6

LACDPW Precipitation Gages

Source: LACDPW

● = precipitation gages



Additional precipitation gages in the region are operated by the National Weather Service, and surround the Arroyo Seco basin (Figure 4.7). These gages have a more complete and extended period of record. Table 4.2 emphasizes monthly and between-station variation between the three nearest NWS gages.

TABLE 4.2 Average Precipitation Measured at National Weather Service Climate Station (in)

Month	Average precipitation measured at National Weather Service Climate Station (in)		
	Pasadena (046719)	Tujunga (049047)	Mt. Wilson No. 2 (046006)
January	4.31	4.11	7.24
February	4.41	4.37	8.57
March	3.52	4.31	6.26
April	1.45	1.54	2.65
May	0.38	0.47	0.71
June	0.14	0.06	0.12
July	0.03	0.02	0.04
August	0.12	0.16	0.44
September	0.37	0.63	1.02
October	0.61	0.39	1.33
November	1.91	2.34	4.80
December	3.09	2.43	5.38
Annual	20.3	20.8	38.6
Period of Record	1927-1998	1966-1987	1961-1990

Source: NWS Western Regional Climate Center, 1999.

4.7.2 Natural Drainage

Approximately 21.75 square miles of the Angeles National Forest drains to the Hahamongna Watershed Park at the JPL bridge boundary. Arroyo Seco, ultimately a tributary to the Los Angeles River, is the main drainage through this upper watershed area. Multiple smaller tributaries drain to Arroyo Seco in the upper watershed, including streams in the following canyons: Ladybug, Cloudburst, Daisy, Colby, Little Bear, Bear, Long, Dark, Twin, Brown, Pine, Falls, Agua, Fern, El Prieto, and Millard. Figure 4.8 shows a map of the upper watershed areas that drain to the Hahamongna Watershed Park.

figure 4.7

National Weather Service Precipitation Gages

● = precipitation gages

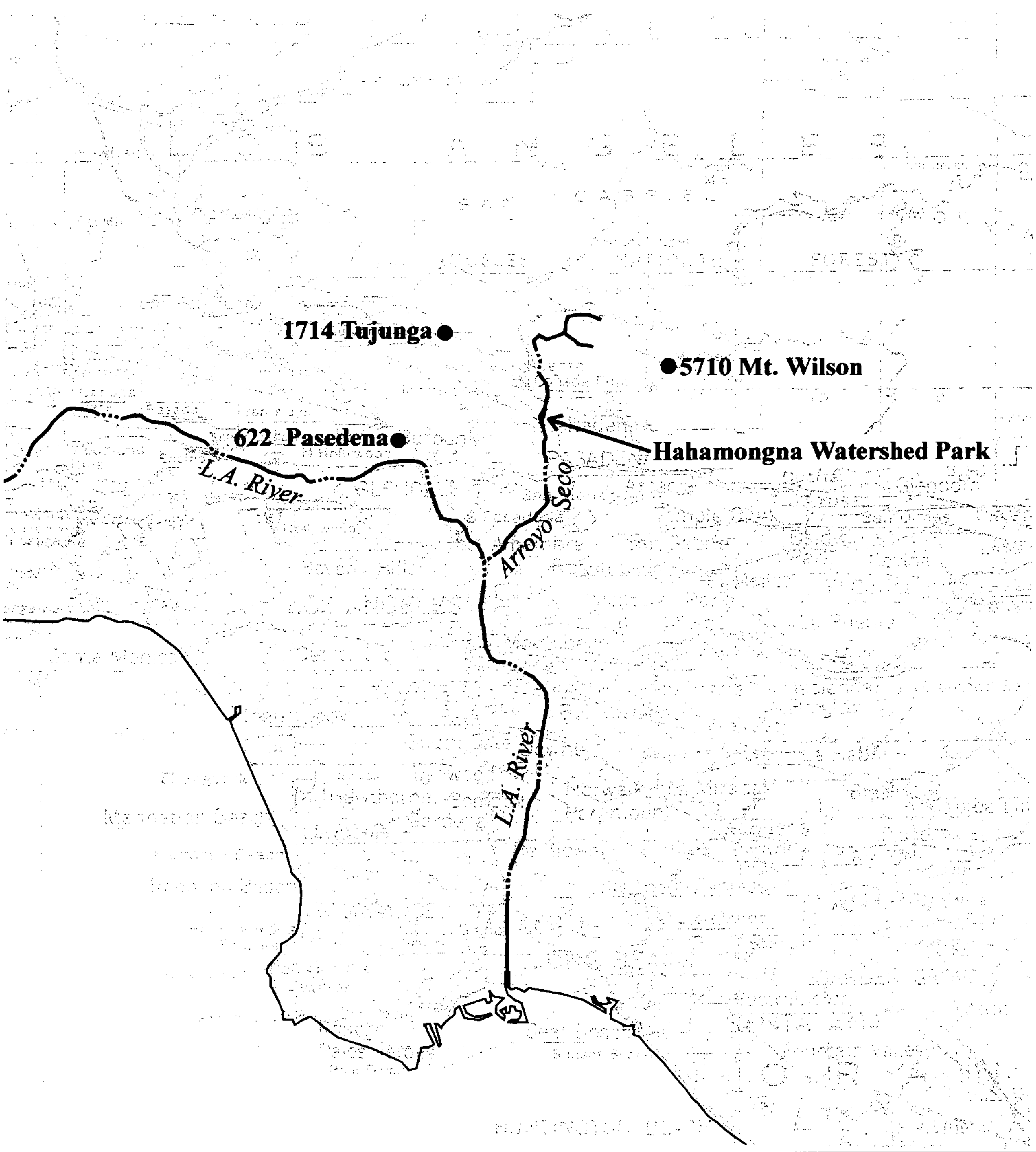
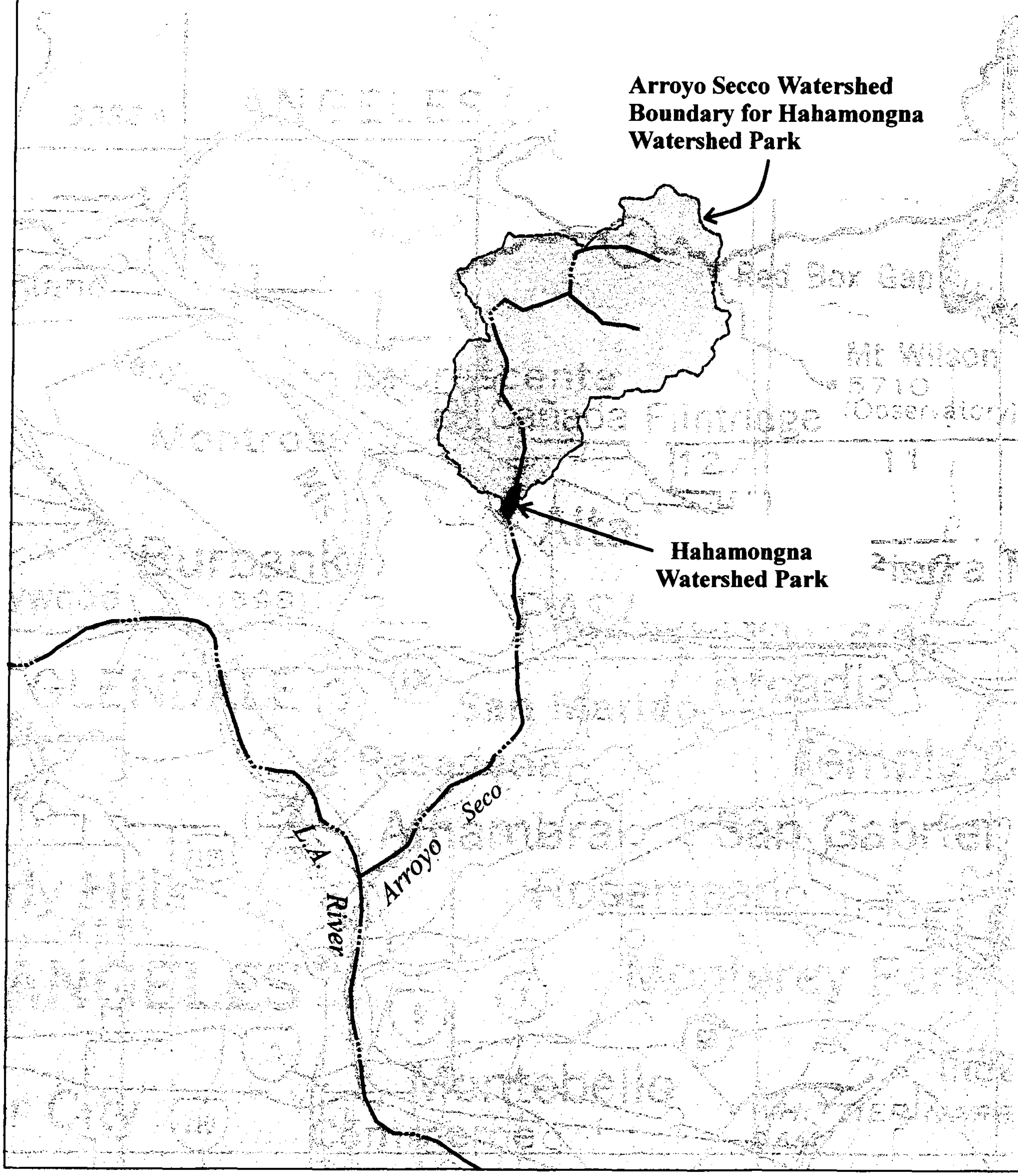


figure 4.8

Arroyo Seco Upper Watershed Draining to Hahamongna Watershed Park



After passing under the JPL bridge, Arroyo Seco meanders through the Hahamongna Watershed Park to Devil's Gate Dam. Approximately 10.15 square miles drain directly to the Hahamongna Watershed Park between the JPL bridge and Devil's Gate Dam. Direct drainage to the park is mostly through stormwater culverts from local municipalities, although one open channel, Flint Wash, drains to the southwest corner of the basin. There is a total watershed area of 31.90 square miles at the dam.

4.7.3 Runoff and Flood Frequency

The United States Geological Survey (USGS) maintains a stream gage on Arroyo Seco above Hahamongna Watershed Park, "Arroyo Seco Near Pasadena, CA" (USGS Gage #11098000). The gage is positioned on the right bank 0.7 miles east of Angeles Crest Highway and 5.5 miles northwest of Pasadena. The historic flow record includes December 1910 to January 1913 (fragmentary), April 1913 to November 1915, and April 1916 to the present. Since November 1938, the gage has been located in the same place, with an upstream drainage area of 16.0 square miles. Average annual runoff for the period of record is 7,290 acre-feet. The mean annual discharge rate at the gage is 10.1 cfs. A flood frequency analysis was conducted for this gage using standard Bulletin 17B procedures. Results of this analysis are presented in Section 4.9.1.

The second stream gage that is relevant to this study is located immediately downstream of Devil's Gate Dam and is operated by LACDPW ("Arroyo Seco Below Devil's Gate Dam," F277-R). Approximately 32.5 square miles of watershed area drain to this gage. Drainage to this gage includes runoff from upstream of the dam and from areas draining to the short reach of Arroyo Seco downstream of the dam but upstream of the gage. The period of record for this gage is 57 years, from 1942 to the present. Streamflow data for the gage is in electronic format from 1989 to present, and is in hard copy for the remainder of the years. PWA obtained all available gage data in electronic format (15 minute readings) and selected storm runoff data from 1969, 1978, 1980, and 1983. With the assistance of the LACDPW, these years were selected since large flood events occurred in each. Summary statistics for this gage were unavailable. Flow frequency was not calculated for this gage, since electronic data was only available for a small portion of the entire flow record.

4.7.4 Evapotranspiration

The Hahamongna Watershed Park is located in arid southwestern California. As a result evaporation and evapotranspiration rates are relatively high. Table 4.3 shows average monthly pan evaporation data for a nearby monitoring station, and equivalent evapotranspiration rates using a pan coefficient of 0.7.

TABLE 4.3 Average Monthly Pan Evaporation, LACDPW Descanso Gardens Station (NO. 1071B-E) & Estimated Monthly Potential Evapotranspiration, Devil's Gate Reservoir

Month	Average Pan Evaporation (inches)	Average Potential Evapotranspiration (inches)
January	2.06	1.44
February	2.21	1.55
March	3.06	2.14
April	3.88	2.71
May	4.46	3.12
June	5.43	3.80
July	7.13	4.99
August	6.94	4.86
September	5.83	4.08
October	4.39	3.07
November	3.09	2.16
December	2.29	1.60
Annual	**50.77**	**35.54**

Sources: LACDPW, 1999; PWA analysis, 1999.

4.8 GEOMORPHIC PROCESSES

In the Arroyo Seco watershed, active geomorphic processes include floods, fire, debris flows, and dry ravel. These processes are temporally and spatially variable, and dependent on a complex interaction between other watershed characteristics (Figure 4.1). Floods, fires, and debris flows are events that change the physical condition and function of a watershed over a relatively short time scale, and therefore are considered to be "catastrophic." Floods and fire often play a critical geomorphic role by causing hillslope and channel materials to reach a critical stability threshold. Floods and fire thus play an important role in the frequency and magnitude of sediment transport and net sediment yield. Non-catastrophic hillslope processes, such as dry ravel, are critical to sediment supply to the alluvial fan environment. The following sections describe the geomorphic setting of the Hahamongna Watershed Park and the main geomorphic processes that effect this setting.

4.8.1 Floods

Flood events are a crucial geomorphic process in the Hahamongna Watershed Park. USGS flow records from the Arroyo Seco gage indicate several large flood events. The maximum peak instantaneous discharge recorded at the stream is 8,620 cfs and occurred on March 2, 1938 (mean daily flow of 2,700 cfs). In addition, large mean daily flows occurred on January 23, 1943 (1,760 cfs); January 25, 1969 (3,210 cfs); March 4, 1978 (1,400 cfs); and March 2, 1983 (1,530 cfs). PWA conducted a peak flow flood frequency analysis for this gage and then scaled the analysis to quantify flood frequency at the upstream face of Devil's Gate Dam. The program HEC-WRC was used for this analysis, which utilizes standard Bulletin 17b methods (U.S. Water Resources Council, 1981). Results of the flood frequency analysis are shown in Table 4.4.

Of the major historic floods, none were preceded by a wildfire within three years of the flood. Thus, the immediate effect on sedimentation of a large flood immediately following a wildfire has not been tested within the historical record at the Hahamongna Watershed Park. However, it is likely that climatic variables are more important to the generation of floods than the reduced soil infiltration rates often attributed to wildfire (Section 4.9.2). The effect of wildfire likely has some effect on the magnitude and timing of peak flows.

4.8.2 Fire

Greater than 90% of the Hahamongna Watershed lies within the Angeles National Forest (ANF), and is administered by the U.S. Forest Service (USFS). The Angeles National Forest (ANF) has an extensive wildfire history, in part due to the vast stands of mature (>20 years of age) chaparral which are extremely flammable. Other contributors to wildfire include rugged terrain which makes fire fighting difficult, Santa Ana winds which quickly spread fire once they have started, and a Mediterranean climate which promotes ignition. In addition, the frequency of fire has been increased by the number of people using the upper watershed for recreational purposes.

Ecological evidence suggests that fire has played a prominent role in chaparral communities for millions of years (Rice, 1982). Relative to controlled fires, wildfires in chaparral tend to burn at a high intensity. During such a hot fire, chaparral plants create a water repellent layer at shallow depth in the soil. Following formation of a water-repellent layer, a drainage basin that might have a hydrologically active mantle 5 feet thick when unburned may have its effective thickness reduced by fire to an inch (Krammes and Osborne, 1969). As a result of the reduction in plant cover and infiltration capacity of the soil, surface and shallow subsurface runoff is increased and flood peaks are magnified.

TABLE 4.4 Flood Frequency Analysis for Arroyo Seco at USGS Gage and at Devils Gate Dam

Drainage	Watershed Area[1] (sq. mi.)
Upstream of Gage	16.05
Upstream of Dam	31.90

Exceedance Probability (1/Years)	Recurrence Frequency (Years)	Expected Flow at Gage[2] (cfs)	Expected Flow at Dam[3] (cfs)	Exponent[4]
0.002	500	17300	30592	0.83
0.005	200	12500	22104	0.83
0.010	100	9550	16887	0.83
0.020	50	7040	12364	0.82
0.050	20	4410	7692	0.81
0.100	10	2860	4920	0.79
0.200	5	1670	2834	0.77
0.500	2	556	912	0.72
0.800	1.25	168	275	0.72
0.900	1.11	86	141	0.72
0.950	1.05	48	79	0.72
0.990	1.01	15	25	0.72

All discharges are bulked with sediment.

Notes:

1 Computed by GIS analysis of USGS 30 Meter Digital Elevation Model

2 Computed using peak annual discharges at USGS Gage #11098000 for 1914-96 and HEC-WRC software (Bulletin 17b Methodology)

3 Computed by $Q_{dam} = Q_{gage} * (A_{dam} / A_{gage})$^Exponent (see Waanan & Crippen 1977)

4 Waanan, A.O. and J.R. Crippen, 1977. Magnitude and Frequency of Floods in California. B6 U.S. Geologic Survey Water-Resources Investigation 77-21

Chaparral, the dominant vegetation type in the Arroyo Seco watershed, typically burns every 20 to 40 years because of accumulation of enough fuel to carry fire readily (City of Pasadena, 1996). Historical fires in the watershed are consistent with this temporal pattern. Since 1878, when fire history records were first systematized, six major fires (>3 percent of Hahamongna Watershed burned) have been documented within Arroyo Seco. The most recent large fire, the Woodwardia Fire, occurred during October 13-22, 1959. The Woodwardia Fire resulted in the burning of approximately 10,000 acres within the Arroyo Seco watershed. Other historical fires include a small event in 1955 that burned approximately 400 acres near Little Bear Canyon, and a 1975 Mill Fire that burned approximately 800 acres near Strawberry Peak and Mount Josephine. Table 4.5 lists these major events and the associated area burned for each documented fire.

TABLE 4.5 Major Wildfires in the Arroyo Seco Watershed

Year	Acres Burned within Arroyo Seco Watershed [1]	Percent of Undeveloped Watershed [1,2]
1896	6,385	42
1934	3,743	25
1955	424	3
1959	10,729	71
1975	809	5
1979	1,328	9

[1] From County of Los Angeles Department of Public Works, Hydraulic/Water Conservation Division, "Devil's Gate Dam & Reservoir Hydrologic Reanalysis," August 1993.

[2] The undeveloped portion of the Devil's Gate Dam watershed is listed as 15,107 acres in the August 1993 study.

Since the majority of Arroyo Seco watershed lies within the ANF, the USFS is responsible for the active fire suppression as well as fire prevention. The USFS has constructed a fire frequency map that indicates the number of times that portions of the watershed have burned. The map shows that 61% of the watershed has burned twice since 1878, 29% has burned once, 5% has burned 3 or 4 times, and the remaining 5% has never burned (LACDPW, 1993).

Based fire history in the ANF, the USFS determined that when 65% or greater of the watershed area is covered with mature chaparral, the chance of an extremely large wildfire is nearly certain (LACDPW, 1993). As a result, a USFS program is being initiated in which portions of "high fuel" vegetation will be artificially burned each year to maintain the mature chaparral coverage to less than 65% of the basin area, thereby reducing the risk of large uncontrollable wildfires in the Hahamongna Watershed (LACDPW, 1993). The Land and Resource Management Plan for the Angeles National Forest establishes that no more than 20 percent of the watershed in any one year and no more than 40 percent of the watershed in the 0 to 5 vegetative age class should be burned at any time (David Kerr, USFS, personal communication).

In their August 1993 report, it was estimated by the LACDPW that this mosaic (prescribed burn area) approach would take some time to implement, particularly for the following reasons: 1) environmental agencies have restricted burning without appropriate environmental documentation; 2) USFS funding has limited the amount of controlled burning possible; 3) at the inception of the mosaic program from 1988 to 1993, only about 1,000 acres were burned annually, as compared with the modeled optimal goal of about 3,500 acres per year.

Since 1986—the inception of USFS planning for prescribed burn forest management in the Arroyo Seco watershed—at least three prescribed burns took place in the watershed near the Devil's Gate reservoir. The three documented burns occurred on the following dates and were of varying extent: 1) April 8, 1997, 194 acres; 2) July 31, 1998, 81 acres; and 3) February 23, 1999, 20 acres (David Kerr, USFS, personal communication). An additional burn of 56 acres was proposed for April 1999.

4.8.3 Debris Flows

Debris flows (and mudflows) commonly occur in the San Gabriel Mountains where thick deposits of fine-grained alluvium become saturated near the end of a storm period. Other types of hillslope processes such as dry ravel generally do not convert directly to flows. Instead hillslope processes might provide source material to or act to trigger debris flows at the time of major storms (Scott and Williams, 1974). In some instances hillslope processes might be transformed into debris flows. The occurrence of fire in a watershed may encourage soil movement, and thus the entrainment of material by debris flows; however, the interaction between fire and debris flows is unclear (Cooke, 1984). In fact, the development of the water-repelling layer (Section 5.9.2) and the fact that root structures are still intact following fires may cause a low rate of slope failure in a recently burned watershed.

In the mountains, debris flows can scour away all hillslope material from soil to boulders and even erode into the underlying bedrock. Due to their fluidity, debris flows transport material through mountainous terrain and even well beyond the mountain fronts. Material transported by debris flows can fill up channels and contribute to the filling of reservoirs and the development of alluvial fans. The destructive potential of debris flows in residential areas is a principle motivation for the construction of debris-basins. Debris flows pose the most serious hazards in the vicinity of alluvial fans, since flows are unpredictable in the complex topography of the fan.

4.8.4 Dry Ravel

Dry ravel is the most ubiquitous erosional process occurring on chaparral-covered slopes exceeding the movement-threshold of about 30 degrees. This process, sometimes termed "dry creep" (Krammes, 1965), is the downslope movement by gravity of individual grains or aggregates of soil. An inventory of chaparral zones in southern California by Rice (1982) indicates that about 25 percent of the area is subject to dry ravel. The proportion of drainage basins that are steep enough for dry ravel is higher, where approximately two-thirds of the slopes are probably steep enough for dry ravel (Krammes and Osborne, 1969). Rice (1982) maintains that with unburned chaparral, the dry-ravel rate is about 0.29 acre-ft/mi^2 (1.4 m^3/ha) per year.

Dry ravel is initiated by many types of small disturbances, such as the movement of fauna along hillsides. However, one of the principal triggers of dry ravel is the movement of vegetation during periods of strong foehn winds (a warm, dry wind blowing down the side of a mountain).

Characteristically, following a fire, dry ravel accumulates during summer and fall on the flatter portions of burned slopes and in ephemeral channels (Rice, 1982). Large flood events during winter then convey sediments to the major channels and ultimately to the alluvial fan. Scott and Williams (1978) describe a conceptual model of headwater-basin sediment transport in which channel in-filling by dry ravel and sheet erosion during dry and moderate years alternates with channel scour by debris flows during major wet-year storms. The scenario may be repeated several winters after the fire as the rills formed during each runoff period are refilled by dry ravel between storms and during summers. Thus, the transport of sediment from its source in the upper watershed to its destination on the alluvial fan can be a multi-seasonal process.

Data from the Arroyo Seco support this strongly seasonal pattern, and demonstrate that dry-season hillslope processes in Arroyo Seco contribute as much sediment to channels as do fluvial processes (Anderson *et al.*, 1959) . The immediate effect of the autumn 1959 fire was to consume the forest litter that has been serving as temporary barriers to the downslope movement of hillslope materials. Accelerated ravel occurred within minutes of the passage of the fire and produced debris cones blocking stream channels within a few hours. Ultimately, this resulted in about 8.2 acre-ft/mi^2 (39 m^3/ha) of dry-ravel erosion during the first 3 months. Five years of monitoring the Arroyo Seco drainage showed an average dry-season erosion rate of 0.2 acre-ft/mi^2 (0.96 m^3/ha), which was 55 percent of the total surface erosion measured (Anderson *et al.*, 1959). Thus, about half of the wet-season erosion was actually dry ravel occurring between winter rainstorms.

Fire also accelerates dry ravel by the creation of a water-repellent layer, which causes the soil surface to dry more quickly and, therefore, to be subject to dry ravel for a greater portion of the wet season. Although fire-related water repellency is detectable for a long time it ceases to affect runoff or erosion significantly within 2 or 3 years after a fire (Rice, 1982). Surface erosion is a rarity in unburned chaparral drainage basins because litter usually protects the soil surface. Even without that protection, the thick, hydrologically active mantle can store and transmit large volumes of water. Furthermore, rainfall intensities rarely exceed infiltration rates (Rice, 1982).

Landsliding is relatively uncommon in the Arroyo Seco watershed, especially considering the steepness of slope and magnitude of relief. Most of the larger landslides are ancient and show no evidence of reactivation under Holocene climatic conditions (Smith, 1986).

4.8.5 Regional Annual Sediment Yield

The average annual sediment yield is the rate at which sediment is eroded from a watershed. It is calculated using the sum of the large contributions from major storms plus the individual contributions of lesser storms and periods of low flow, all divided by the number of years of record.

The primary natural factors that determine sediment yields in the Arroyo Seco basin include hillslope cover, topography, hydrology, and fire (Figure 4.1). The conditional and independent probabilities associated with these processes have resulted in a sediment regime dominated by a few very large events, many insignificant ones, and low predictability over the short-term.

The long-term sediment yield of the San Gabriel mountains is also the subject of many studies exploring gross landscape change. Estimated effects of human developments and artificial control structures on erosion suggest that overall regional erosion has been altered very little during the past 50 years (Taylor, 1981). Average annual denudation rates are summarized by Booker (1998; Table 4.6) and are highly variable. This variability is due in part to fire history and the type of hillslope processes most active in the study area (e.g., dry ravel versus debris flows). The volume of sediment transported and then deposited by a single debris flow event tends to exceed average annual yield volumes (due primarily to dry ravel processes) by one order of magnitude (Table 4.6). Measured annual sediment yields for the San Gabriel area range over four orders of magnitude, from 0.3 to 4,421 acre-ft/yr (Table 4.6).

Comparisons of aggregate upland sediment yields and coastal sediment deliveries on major river systems suggest that under recent natural conditions alluvial rivers in the southern part of the region are depositional along their floodplains, with only a fraction of the aggregate sediment yields being delivered to the shoreline. Locally, annual catchment sediment yields have varied more than four orders of magnitude during the past five decades.

TABLE 4.6 Equivalent Soil Loss for Different Fire Areas[1]

Reference	Fire Location	Active Processes & Notes	Equivalent Soil Loss (mm or mm/yr, specified)	Projected Volume of Sediment Yielded if Applied to Arroyo Seco (acre-feet or acre-ft/yr, specified)[2]
LONG-TERM BACKGROUND EROSION RATE				
Bruington (1982)	San Gabriel Mnts.	Long term erosion rate, 1936-1980	2.18 mm/yr	146 acre-ft/yr
Scott *et al.* (1978)	Transverse Ranges	Long term erosion rate	2.3 mm/yr	154 acre-ft/yr
Taylor (1981)	**Devil's Gate Reservoir**	**Measured average denudation rate for erosional areas in catchment; based on 54 years of measurement from 1920 - 1974**	**1.63 mm/yr for Devil's Gate basin (1.11 mm/yr for mountainous San Gabriel area based on regression)**	**109 acre-ft/yr (74 acre-ft/yr)**
Wells (1985)	San Gabriel Mnts.	Long term erosion rate, 1961 - 1981	1.4 mm/yr	94 acre-ft/yr
SHORT-TERM EROSION RATE, UNBURNED				
Howard (1982) Rice *et al.* (1979)	San Gabriel Mnts.	1966 dry ravel 1969 flood year	2.11 mm/yr (unburned) 29.8 mm/yr (unburned)	141 acre-ft/yr 1996 acre-ft/yr
Wells (1981)	San Gabriel Mnts.	Erosion plots, Fern Canyon 1938-1941	1st year: 1.9 mm 2nd year: 0.2 mm 3rd year: 0.1 mm	127 acre-ft/yr 13 acre-ft/yr 7 acre-ft/yr
Wells (1981)	San Gabriel Mnts.	Erosion plots, Tanbark Flat 1938-1941	0.004 mm/yr	0.3 acre-ft/yr

TABLE 4.6 (continued)

Reference	Fire Location	Active Processes & Notes	Equivalent Soil Loss (mm or mm/yr, specified)	Projected Volume of Sediment Yielded if Applied to Arroyo Seco (acre-feet or acre-ft/yr, specified)[2]
BURNED: SEEDED/UNSEEDED				
Bruington (1982)	San Gabriel Mnts. Glendora & Zachau debris basins	1933 1968 1975	35 mm 66 mm (seeded) 66 mm (seeded)	2344 acre-ft/yr 4421 acre-ft/yr 4421 acre-ft/yr
Florsheim (1987)	Transverse Range	Dry ravel & shallow slips	0.3 mm/yr (seeded)	20 acre-ft/yr
Krammes & Hill (1963)	San Gabriel Mtns.	Erosion plots, sheetwash & rilling	3.4 mm/yr (seeded) 3.3 mm/yr (unseeded) 6.1 mm/yr (unseeded)	228 acre-ft/yr 221 acre-ft/yr 409 acre-ft/yr
Wells (1985)	San Gabriel Mnts.	Fire 1962 - 1964 Fire 1979 - 1981 Flood 1969	5.83 mm/yr (seeded) 14.29 mm/yr (seeded) 13.46 mm/yr (unburned)	391 acre-ft/yr 957 acre-ft/yr 902 acre-ft/yr
Wells (1987)	San Gabriel Mtns.	Hidden Springs 1978 debris torrent	25 mm/yr (seeded)	1675 acre-ft/yr
Wells (1987)	San Gabriel Mtns.	Carter Canyon debris torrents 1978	10 mm/yr (seeded)	670 acre-ft/yr
SINGLE EVENT DEBRIS FLOW EROSION RATE				
Scott (1992) & Colman (1951)	San Gabriel Mnts. Montrose, CA	Debris torrent	30.6 mm	2,050 acre-ft
USDA (1954)	San Gabriel Mnts., Wolfskill Canyon	Single debris torrent	16.4 mm	1,099 acre-ft

TABLE 4.6 (continued)

Reference	Fire Location	Active Processes & Notes	Equivalent Soil Loss (mm or mm/yr, specified)	Projected Volume of Sediment Yielded if Applied to Arroyo Seco (acre-feet or acre-ft/yr, specified)[2]
ASSORTED HILLSLOPE PROCESSES (DRY RAVEL, ETC.)				
Doehring (1965)	San Gabriel Mtns.	Dry ravel, rill & channel activity	25 mm/yr	1675 acre-ft/yr
Howard (1982)	San Gabriel Mnts.	Shallow failures	10 mm	670 acre-ft
Howard (1982) and Anderson (1959)	San Gabriel Mnts.	Unburned, dry ravel	0.03 - 0.4 mm/yr	2 - 27 acre-ft/yr
Scott (1992) & Krammes (1969)	San Gabriel Mnts. Santa Ana Mnts.	9.7 % overland flow as % of rain 9.9% overland flow as % of rain	34.6 mm/yr 5.2 mm/yr	2318 acre-ft/yr 348 acre-ft/yr

[1] Adapted from Booker 1998. See that text for full references.

[2] Based on a drainage area o 31.9 mi^2 for Arroyo Seco at the Hahamongna Watershed Park. Calculated by multiplying drainage area by soil loss to obtain volume.

4.8.6 The Alluvial Fan Environment

When hillslope materials are mobilized to the valley, they commonly are deposited at the canyon-valley interface where there is a sudden reduction in channel slope. There the deposits form a roughly semicircular arc, referred to as an *alluvial fan*. The Devil's Gate Reservoir is situated at the opening of the Arroyo Seco canyon along the upper portion of this alluvial fan environment.

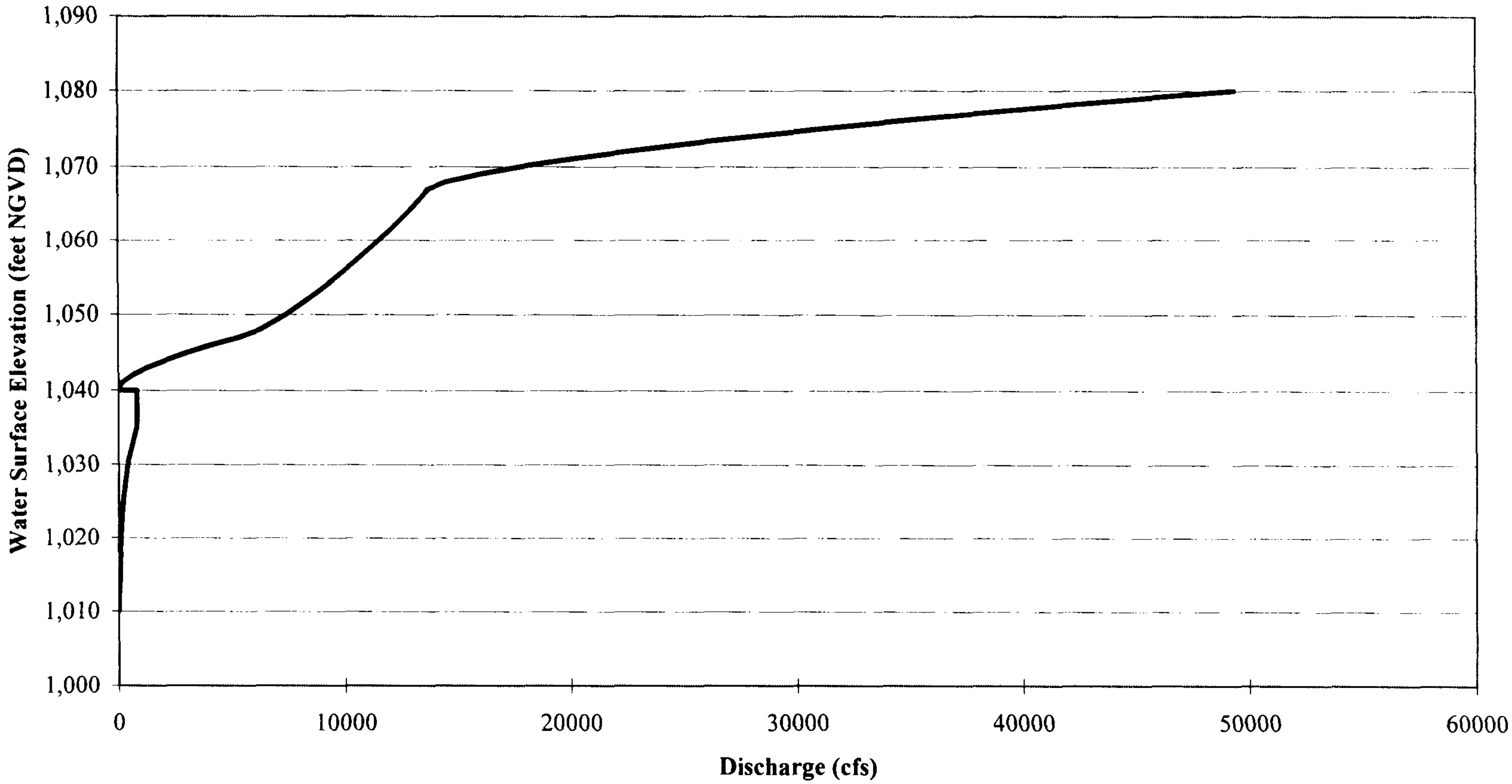
Alluvial fans are complex and potentially destructive geomorphic environments, since they occur at the interface between high hillslope sediment supply and the beginning of extensive fluvial reworking. Processes that have caused extensive damage on alluvial fans include lateral scour in existing channels, the formation of new channels by sudden redirection of flow at the fan apex, and inundation by debris flows and mudflows. Since the Hahamongna Watershed Park is located in an alluvial fan environment these types of channel processes are common there.

4.9 WATER RESOURCES HISTORY

4.9.1 Devil's Gate Dam

Devil's Gate Dam is operated in a relatively simple way, and has been operated this way since at least 1977. Under all flow and sediment transport situations, the lowest elevation outlet gate is kept open until water levels behind the dam rise to elevation 1,010 feet NGVD. In this way flow-assisted sediment transport through the dam is maximized without compromising flood protection, thereby reducing the amount of sediment accumulation and the subsequent required excavation in the reservoir. Through this operating strategy, storage capacity is maximized for use during major storm events.

During relatively large storm events, when water-levels exceed elevation 1,010 feet NGVD, the lowest outlet gate is closed and other gates—such as the 7-foot by 10-foot slide gates in the tunnel—are used to make releases. Closing the lowest gate once water levels reach 1,010 feet NGVD causes debris and sediment to settle out of suspension farther away from the dam, reducing clogging at the lowest outlet structure. Generally, once water-levels behind the dam reach 1,040.5 feet NGVD (the elevation of the spillway crest), all gates are closed and releases are made only through the spillway ports. The second spillway, an ogee spillway, has a crest elevation of 1,067 feet NGVD and is the final outlet structure for the most extreme events. The spillways were completely reconstructed with the dam retrofit in 1995. Figure 4.9 shows LACDPW's current operating curve for Devil's Gate Dam.



Source: LACFCD
Note:

figure 4.9

Devils Gate Dam Operation Curve

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dam_ops_via_mike.xls

4.9.2 Water Supply

Beginning in 1891 the Pasadena Lake Vineyard Land and Water Company constructed several underground tunnels in the Hahamongna Watershed Park area. Cut through alluvium, these tunnels provided a significant amount of water. This water was sold to the City of Pasadena as municipal water supply. By 1903 a total of 4730 linear feet of tunnel had been constructed. Between 1897 and 1904 a sub-surface dam was constructed at Devil's Gate to increase the percolation of water into the tunnels (City of Pasadena Water and Power Department, 1994). In 1912 the City of Pasadena Water Department was formed and incorporated the Pasadena Lake Vineyard Land and Water Company, along with the Devil's Gate tunnel network. Between 1913 and 1919 the tunnels yielded an average of approximately 3400 acre-feet of water per year to the City water supply system (City of Pasadena Water and Power Department, 1994).

Between May 1919 and June 1920 the Devil's Gate Dam was constructed. The dam was built for the joint purposes of increasing water supply through the City of Pasadena tunnels and providing flood control for Arroyo Seco, a major tributary to the Los Angeles River. Between 1920 and 1928 the tunnels yielded an average of approximately 2300 acre-feet per year, and this during a relative dry period. Relatively high water yields from the Devil's Gate tunnels were attributed to the holding of water behind the new dam. After 1929 the water yield of the tunnels declined steadily, until 1938 when a large flood and debris event from the recently-burned upper watershed rendered water percolating into the tunnels non-potable. From that time water from the tunnels has been used exclusively for irrigation purposes. Today the relatively small amount of water yielded by the tunnels is used to irrigate Brookside Golf Course, downstream of the dam.

In addition to water from the Devil's Gate tunnels, the City of Pasadena actively diverts water from the Arroyo Seco. The City of Pasadena has a historic right to divert up to 25 cfs for water supply. The City maintains a diversion intake upstream of the JPL bridge. In decades past diverted water was routed to the Behner Treatment Plant (near the northern end of the Hahamongna Basin), treated, and then received directly into the municipal water supply system. During this period LACDPW operated a series of spreading ponds along the east side of the basin, used to recharge the over-pumped alluvial aquifer of the Raymond Basin. These basins are called the Arroyo Seco Spreading Grounds and historically received water when flow in the Arroyo Seco exceeded the City's 25 cfs diversion right. With the advent of more stringent water quality standards, the City's direct diversions to the municipal water supply system were discontinued. As a result the City began diverting water into the spreading ponds operated by LACPWD. The Behner Treatment Plant was left unused and currently remains this way; however, there is a possibility that it will be renovated in the future so that water treated there would be able to meet current water quality standards.

Since 1998 the City has taken over operation and maintenance of the Arroyo Seco Spreading Grounds from LACDPW and continues to use its 25 cfs diversion right to route water through the percolation ponds. The

City of Pasadena receives groundwater pumping credit equal to approximately 50% of their diversion by percolating water in this way. Groundwater pumping and percolation in the Raymond Basin (an adjudicated groundwater basin) is overseen by the Raymond Basin Management Board (RBMB).

The City currently obtains approximately 40% to 50% of their municipal water supply from groundwater pumping from the Raymond Basin with most of the remaining municipal demand being met by water from the Metropolitan Water District (MWD). The City's pumping costs for one acre-foot of groundwater is approximately $91 while MWD water costs the City approximately $431 per acre-foot. Therefore, the City has a financial incentive to maximize groundwater pumping, and therefore groundwater percolation credit.

4.9.3 Groundwater

The Hahamongna Watershed Park is situated over part of an unconfined groundwater aquifer known as the Monk Hill Basin. Together the Pasadena Subarea, the Santa Anita Subarea and the Monk Hill Basin make up a larger unconfined aquifer called the Raymond Basin (Figure 4.10). The Raymond Basin aquifer is approximately 40 square miles in area and underlies much of the City of Pasadena. It is bounded to the north by the San Gabriel Mountains, to the south and east by the San Gabriel Valley, and to the west by the San Rafael Hills. The Monk Hill and greater Raymond Basin aquifers are composed largely of unconsolidated alluvial sediments (conveyed by runoff processes), ranging to a maximum thickness of approximately 1,100 feet. Below the Hahamongna Watershed Park, the alluvial aquifer is composed of relatively coarse sediments from Arroyo Seco. These coarse sediments make the aquifer very permeable. Water percolates from the surface to the groundwater relatively quickly and groundwater flows at relatively high rates.

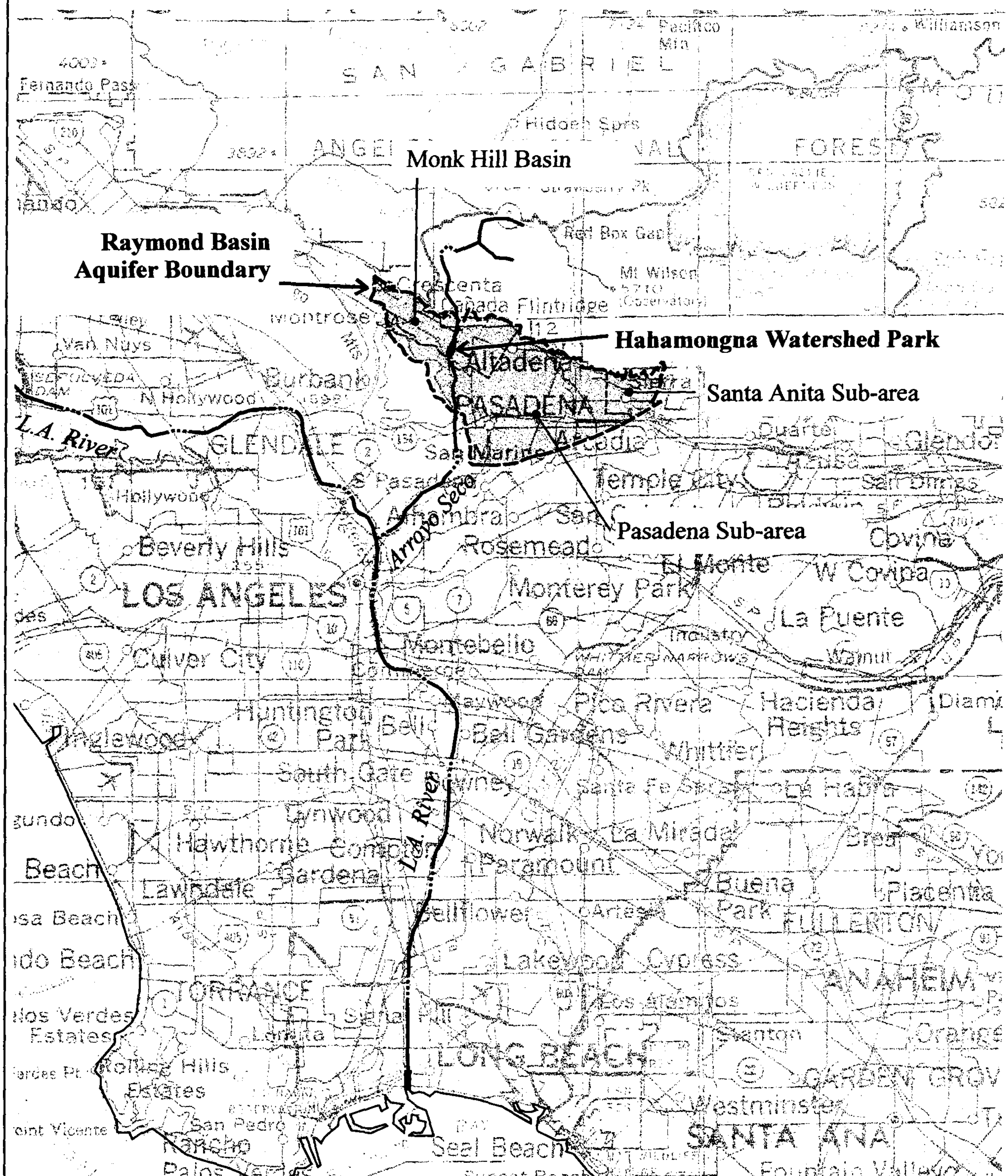
There are many groundwater wells in the Hahamongna Watershed Park area, some for water supply and some for monitoring groundwater contamination from JPL (Section 4.10.7). Figure 4.11 shows a map of many of the groundwater wells proximal to the park, and Figure 4.12 shows historic groundwater elevation data for several of these wells. Average groundwater elevations in the vicinity of the Hahamongna Watershed Park are between 900 feet NGVD and 1,000 feet NGVD, with significant seasonal fluctuations.

The first groundwater wells were drilled in the Raymond Basin in 1881, following the commencement of the Southern California land development boom in 1880. Water from these early wells was used for irrigated agriculture and municipal water supply. From this time on the City of Pasadena was a primary groundwater user in the Raymond Basin. By 1908 approximately 141 wells were in operation in the Raymond Basin.

Around 1913 overdraft of the Raymond Basin Aquifer began. Through the 1920s, over-pumping combined with a relatively dry period caused groundwater levels to drop significantly and caused the failure of some

figure 4.10

Raymond Basin Aquifer

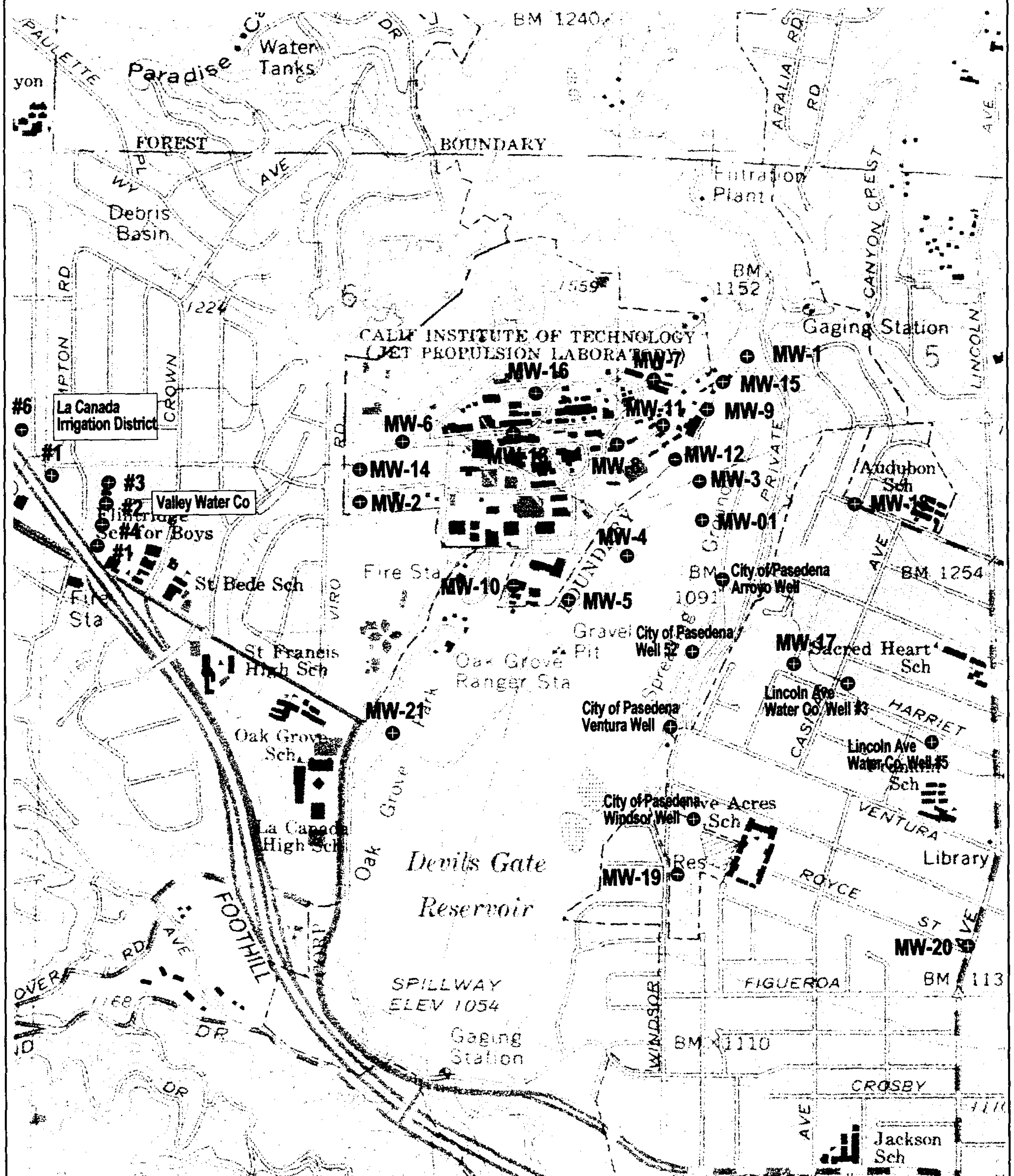


PWA

figure 4.11

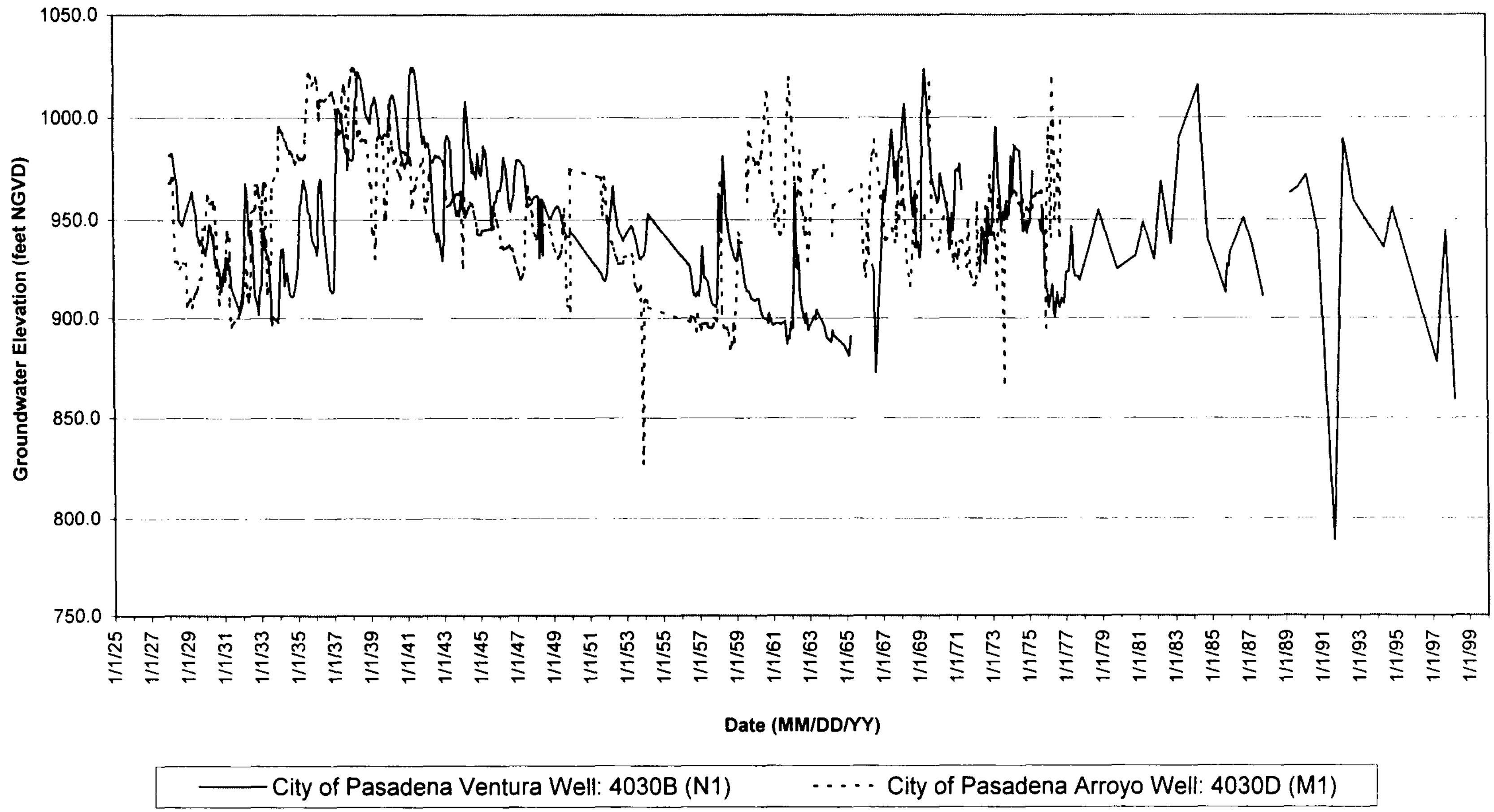
Groundwater Wells Near Hahamongna Watershed Park

⊕ = well location



1310 wells.cdr

PWA



Source: California Department of Water Resources
Note:

figure ***4.12***

Historic Groundwater Levels, Hahamongna Watershed Park

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wells. Increasing pressure on groundwater resources in the Raymond Basin ultimately led to an adjudicated division of groundwater rights within the basin in 1944. The estimated safe yield of the basin was increased in 1955 from its original adjudicated amount. In 1974 a modification to the Raymond Basin judgement was made allowing water users to gain groundwater pumping credit for spreading diverted surface waters in spreading ponds to recharge the aquifer. At this time the City of Pasadena began receiving credit for spreading diverted water into the Arroyo Seco Spreading Grounds, located within what is now the Hahamongna Watershed Park.

4.9.4 Sediment Management

Historically, check dams have been constructed by the USFS in the steep side canyons of the Arroyo Seco watershed, above the Hahamongna Watershed Park, with the intention of reducing erosion during storm events by slowing down debris flows following fires and floods. Post-wildfire growth of vegetation in this sediment further slows down flood waters thereby reducing scouring and incision of the canyon floor. Reduced sediment flows have a beneficial impact on water quality. The impact of these check dams on sediment management and delivery to the Hahamongna Watershed Park is unclear. Over the long-term, the sheer volume of sediment eroding from the watershed is far more than can be contained behind check dams alone. However, reduced erosion during storm events, caused by the check dams, may have decreased overall sediment transport and delivery to the park. Brown Canyon debris basin, a debris basin also located in the upper watershed of the Arroyo Seco, may also have had an historical impact on sediment delivery to the park area, trapping significant volumes of sediment before it entered the reservoir area. However, it is unlikely that the Brown Canyon debris basin has any impact on current sediment delivery to the reservoir since the basin is virtually full of sediment to its spillway elevation.

Since the construction of Devil's Gate Dam, the LACDPW has actively removed sediment from its reservoir easement located upstream of the dam. Sediment was removed on an as-needed, economic efficacy basis. Removal has consisted of sluicing (utilizing inflow and machinery to wash and push accumulated sediment through the dam's lowest gate) or excavation. Table 4.7 shows the approximate quantities of sediment removed since the dam was constructed in 1919. The table also shows estimated sediment deposition and sediment storage within the reservoir. In general, the total amount of sediment removed from the reservoir is less than the total amount that has deposited there, thus accounting for the overall decline in the reservoir's active storage capacity. Over the entire sediment maintenance record approximately 20% of the total sediment deposited has been sluiced. The lack of high inflows outside of storm events limited the amount of sediment that sluicing operations could remove. It should be noted that this record of sediment management is not complete. Significant mining, illegal dumping of sediment, and unquantified sediment removal have added uncertainty to the record.

TABLE 4.7 Historical Sediment Management at Devil's Gate Reservoir[1]

Date	Reservoir Capacity (acre-feet)[2]		Volume Sluiced (acre-feet)	Volume Excavated (acre-feet)	Volume Deposited (acre-feet)[3]	Accum. Sediment Production (acre-feet)	Sediment in Storage (acre-feet)
	Spillway at 1054 feet	Spillway at 1040.5 feet					
October 1919	4601	N/A	0	0	0	0	0
September 1934	4127	N/A	0	47	521	521	474
June 1935	3996	N/A	0	0	131†	652	605
June 1938	2967	N/A	0	0	1029‡	1681	1634
January 1942	2728	N/A	644	24	907	2588	1873
December 1943	2504	N/A	65	18	307†	2895	2097
Fall 1948	2561	N/A	75	46	64	2959	2040
July 1952	2636	N/A	256	85	266	3225	1965
September 1955	2709	N/A	0	73	0	3225	1892
December 1959	2839	N/A	0	175	45	3270	1762
May 1962	2750	N/A	0	431	520	3790	1851
September 1966	2598	N/A	51	369	572	4362	2003
February 1969	2106	N/A	0	20	512	4874	2495
March 1969	1875	N/A	0	0	231	5105	2726
November 1969	2002	N/A	119	8	0	5105	2599
December 1971	1928	N/A	0	143	217‡	5322	2673
October 1973	2186	N/A	0	293	35	5357	2415
March 1977	2502	N/A	0	462	146	5503	2099
March 1978	2460	N/A	0	149	191	5694	2141
July 1978	2434	N/A	0	0	26†	5720	2167
December 1978	2748	N/A	0	314	0	5720	1853
February 1979	2692	N/A	157	76	289	6009	1909
March 1980	2790	N/A	0	281	183	6192	1811
July 1981[4]	2869	N/A	0	199	120†	6312	1732
September 1982	2820	N/A	0	60	109‡	6421	1781
April 1983	2775	N/A	0	33	78	6499	1826
June 1988[5]	2869	N/A	0	127[5]	325[5]	6531	1731
February 1992	2975	N/A	0	105[6]	0	6531	1626
July 1992	2887	N/A	0	0	88†	6619	1819
April 1993	2903†	N/A	0	0	65[7]	6684	1884
November 1995	3060	N/A	0	120[8]	0	6684	1764

TABLE 4.7 (continued)

[1] All values here area are from table obtained from the LACDPW. Only minor additions have been made by PWA, as shown by crossed items. Drainage areas on record with LACDPW are as follows:

31.9 mi^2 uncontrolled October 1919 - December 1935.

30.6 mi^2 uncontrolled December 1935 - October 1936

30.4 mi^2 uncontrolled October 1936 - December 1942

24.4 mi^2 uncontrolled December 1942 - Date

Change in uncontrolled drainage area due to construction of the Los Angeles County Flood Control District debris basins and Browns Canyons Barrier constructed by the U.S. Forest Service in 1942.

[2] Spillway was modified in 1997.

[3] Using El. 1,054.0 ft, except where noted.

[4] Approximately 14 acre-feet of broken A.C. pavement was in storage on the day of the survey.

[5] Quantity excavated is from records to January 1985. Unknown additional amounts were subsequently excavated. Illegal dumping of sediment has also occurred. "Debris Deposited" may not be the actual amount from watershed erosion.

[6] Amount excavated by City of Pasadena's permittee (per Larry Harsha, City's Department of Water & Power)

[7] Analysis at El. 1,020 ft indicates 65 acre-feet of sediment inflow.

[8] Excavation tonnage records indicate 120 acre-feet were removed by the 1994 excavation project.

†Volume deposited associated with a single peak flow event, as described in Section 7.1.3.2.

‡Volume deposited associated with two to three peak flow events (highest peak selected for calculation), as described in Section 7.1.3.2.

4.9.5 Flood Management

In 1914 a devastating flood occurred in Los Angeles County, primarily the result of flood-waters originating in the San Gabriel Mountains. The flood caused over $10 million in property damage and claimed many lives. As a result, the Los Angeles County Flood Control District (LACFCD) was formed, later becoming part of the LACDPW. Their mandate was to provide flood protection for L.A. County. To begin fulfilling this mandate the LACFCD initiated construction on multiple dams in the San Gabriel Mountains. Devil's Gate Dam was the first of these. As mentioned in Section 4.10.1 Devil's Gate Dam was built for the dual purposes of water conservation (increased yields from underground tunnels) and flood control. However, flood control seems to have been the more significant purpose of the two.

When it was originally constructed the dam created a reservoir with approximately 4601 acre-feet of active storage capacity. Although a program of regular sediment removal was practiced, over the years sediment accumulation in the basin gradually reduced the active storage of the reservoir. In 1997 the dam was rehabilitated to meet the seismic stability and spillway capacity requirements of the State Department of Water Resources, Division of Safety of Dams. A new spillway was constructed at a lower elevation (lowered from 1,054 feet NGVD to 1,040 feet NGVD) to allow larger floods to pass safely, and sub-surface fractures were filled with concrete.

Downstream of Devil's Gate Dam the Arroyo Seco flows through a short canyon section of channel and then emerges at Brookside Golf Course where flows enter a concrete trapezoidal flood channel. The design capacity of this downstream channel is approximately 8000 cfs. Whenever possible, Devil's Gate Dam is operated such that downstream flows do not exceed this design capacity.

4.9.6 Percolation Ponds and Recharge

The City of Pasadena operates the Arroyo Seco Spreading Grounds, a series of ponds along the east side of Hahamongna Watershed Park designed to facilitate percolation of diverted surface water into the groundwater reservoir (Figure 1.1). As discussed in Section 4.10.2, by percolating surface water into groundwater storage the City of Pasadena obtains groundwater pumping credit with the RBMB.

The Arroyo Seco Spreading Grounds were first operated in 1948. They cover a total area of approximately 15.1 acres, of which approximately 13.1 acres is wet at any one time. Total storage available in the ponds is approximately 30 acre-feet, with an estimated percolation capacity of 18 cfs. This is equivalent to approximately 1.4 cfs per wetted acre.

The accumulation of fine sediment particles in the percolation ponds tends to reduce percolation rates over time. Measures are taken to prevent this, such as not diverting water to the ponds during high sediment transport flood events. Furthermore the ponds are excavated approximately annually to remove fine sediments and restore hydraulic conductivity of the soils. However, despite these efforts at minimizing fine sediment accumulation, the hydraulic conductivity of the ponds remains orders of magnitude lower than in other nearby areas of the basin (Converse Consultants West, 1995).

5. OPPORTUNITIES AND CONSTRAINTS

5.1 OPPORTUNITIES

5.1.1 Groundwater Recharge Efficiency

In planning the Hahamongna Watershed Park there may be an opportunity to enhance the efficiency of the City's groundwater recharge program. The Arroyo Seco Spreading Grounds may not provide the most efficient means of recharging groundwater in the park. On PWA's January 1999 site visit, flow piping was observed from pond #1. Water flowed through the adjacent bank, and back into the main river channel, short-circuiting the percolation pond process and eroding the bank separating the pond from the river. Water meant for percolation was actually rejoining flow in the river instead. Furthermore, the hydraulic conductivity (percolation rate) of the ponds has been observed to be low relative to alluvium in other parts of the basin (Converse Consultants West, 1995). This difference could be due to the accumulation of fines in the ponds or compaction from maintenance activities. On the 18 January 1999 site visit flow in the river was observed to "disappear" approximately half way between the JPL bridge and the dam, indicating that for low flows the majority of water left in the channel may recharge the alluvial aquifer naturally. Questionable pond recharge efficiencies and the potential for more effective natural alternative groundwater recharge strategies may provide an opportunity to decommission the existing recharge ponds while maintaining or increasing groundwater recharge.

Realizing new efficiencies in groundwater recharge within the park could provide an opportunity for increased revenue for the City of Pasadena. Groundwater percolation credit could be increased, reducing the City's requirement to purchase expensive MWD water for municipal supply. If it is found that there are more effective ways to achieve groundwater recharge in the park than by using the existing recharge ponds (for example by periodically holding water behind the dam) there could be an opportunity to save park maintenance costs by eliminating the City's recharge ponds and diversion structures. This would also expand the areas available for other park use, such as natural habitat.

5.1.2 Fire Suppression Water Supply

If a water feature is created behind Devil's Gate Dam it could provide a significant benefit to the U.S. Forest Service in their fire suppression efforts in the Arroyo Seco watershed area. In discussions with representatives of the City of Pasadena, the USFS has indicated a desire for water to be available from behind

Devil's Gate Dam. Water held in the reservoir can be readily used to fill vessels that are conveyed to and emptied over fire zones by helicopter. Benefits would be maximized if it was possible to hold water through the late summer and early fall months.

5.1.3 Devil's Gate Dam Operations

As part of the Hahamongna Watershed Park master plan process there may be an opportunity to reconsider the way Devil's Gate Dam is operated in order to maximize benefits for the park area. However, it should be noted that any alterations to the dam release schedule must comply with LACDPW flood control constraints, discussed in Section 5.2.3. There are two main benefits that could result from reconsidering dam operations.

Firstly, flood hazards could be reduced in the park. Flood hazards are a significant constraint on park design and infrastructure (Section 6). Any reduction in flood hazard within the park area would be valuable. However, LACDPW's responsibility for flood-control downstream of the dam in the Arroyo Seco channel and the corresponding management constraints for Devil's Gate Dam must take priority over reducing flood hazard in the park.

The second benefit that could potentially be realized from a reevaluation of dam operations is increased flow-assisted sediment transport through the dam. Sediment has been sluiced mechanically through Devil's Gate Dam occasionally over the 80-year life of the dam and for the past 20 years flow-assisted sediment transport has been pursued when feasible. Orienting dam operations to maximize flow-assisted sediment transport would reduce the amount of sediment deposition in the basin, and thereby reduce the amount of expensive excavation required to maintain active flood storage in the basin.

5.1.4 Riparian Habitat

The City of Pasadena and other surrounding communities have expressed a desire to have the Hahamongna Watershed Park support as much natural habitat as possible. Since the historic, or "natural," habitat at the project site was riparian habitat, an important master plan objective is to maximize riparian habitat within the basin. This desire for increased natural riparian habitat provides a significant opportunity for habitat restoration in the park area.

5.2 CONSTRAINTS

5.2.1 Land Use

Land uses adjacent to and in the park pose constraints on the development of PWA's sediment management plan and on the potential location and configuration of a water feature within the park. A primary constraining land-use in the park is the LACDPW's flood-control and water conservation easement. The flood-control and water conservation function of the reservoir area behind Devil's Gate Dam must take priority in park planning. The proximity of residential areas and JPL must also be considered in PWA's recommendations. Recreational land-use within the park must be considered in recommending sediment excavation locations. Also public safety and liability issues are important constraints on sediment removal activities and a water feature within the park.

5.2.2 Groundwater Recharge Expansion

The City currently has the right to divert up to 25 cfs to the Arroyo Seco Spreading Grounds and gain proportional groundwater pumping credit from the RBMB. However, the City currently only has spreading capacity to handle an 18 cfs diversion. Since groundwater is the City's most economical source of municipal water supply the City's Water and Power Department has the master planning objective of at least maintaining, and potentially expanding, the amount of groundwater recharge credit they receive from percolation in the Hahamongna Watershed Park. The City's maximum goal would be to expand percolation capacity in the park to accommodate a diversion of 32 cfs. This increased diversion would include a purchased water right of 6.9 cfs from the Lincoln Avenue Water Company. A more moderate goal for the City would be to expand spreading capacity to handle the 25 cfs diversion right currently owned by the City. Their minimum requirement is that a spreading capacity of 18 cfs be maintained.

As part of this study one of PWA's constraints was to take into account the City's water resource goals when making planning recommendations. PWA's recommendations must accommodate the City's goal of maintaining, or expanding if feasible, recharge capacity within the Hahamongna Watershed Park.

5.2.3 Water Rights

As mentioned in Section 5.2.2, the City of Pasadena currently has the right to divert up to 25 cfs from the Arroyo Seco channel upstream of Hahamongna Watershed Park to the Arroyo Seco Spreading Grounds and gain proportional groundwater pumping credit from the RBMB. The City's adjudicated agreement, by which their right to proportional groundwater recharge credit is guaranteed, was established over 40 years ago and was established through a difficult negotiation process. Since the City's water rights constitute a substantial resource benefit to the City, PWA's recommendations must not compromise the maintenance of these water rights. Furthermore, it should be understood that any recommended significant changes to the way the City uses this water right and gains groundwater credit may necessitate a re-negotiation of the City's original adjudicated agreement, which could be a lengthy and difficult process. These water rights issues act as constraints on the recommendations of this study.

5.2.4 Flood Control

Several flood control criteria constrained PWA's recommendations in this study. Currently LACDPW is required, whenever possible, to keep flows downstream of the dam below 8000 cfs, the capacity of the downstream channel. To accommodate this downstream flood-control function, as well as water conservation activities, LACDPW also requires that the overall capacity of the reservoir below elevation 1040.5 feet NGVD be maintained at the level calculated in 1995, the most recent survey of the reservoir. Furthermore, the risk of clogging important dam release gates with debris should not be increased by PWA recommendations. Therefore, to prevent debris plugging, LACDPW requires that the reservoir be managed such that water may be ponded behind the dam at elevation 1020 feet NGVD.

5.2.5 Economics

An economic objective for the City is to maintain the amount of revenue generated within the park, from parking, groundwater recharge, and/or other sources. Similarly, construction costs and long-term maintenance costs should be minimized for the overall park master plan. PWA considered these economic constraints in making recommendations for the park Master Plan.

6. FLOOD HAZARDS ANALYSIS

6.1 RESERVOIR AND FLUVIAL INUNDATION HAZARDS

6.1.1 Hydraulic Modeling

In order to assess potential flood hazards in the Hahamongna Watershed Park, a hydraulic model was constructed to estimate flooding extent for existing and proposed conditions. The MIKE 11 model was chosen for the flood hazards analysis because it utilizes a fully dynamic flood routing scheme, is appropriate for estimating inundation behind a dam, and has been validated world wide with exceptional success.

6.1.2 MIKE 11 Hydrodynamics

The MIKE 11 model can simulate one-dimensional movement of water and sediment in multiply connected networks of channels. By using MIKE 11 it is possible to simulate the dynamic flooding and conventional sediment transport behind Devil's Gate Dam.

MIKE 11 hydrodynamics (HD) are governed by the fully dynamic de St. Venant equations (de Saint Venant, 1871). The differential equations are solved by approximation using a six-point implicit staggered-grid finite difference scheme. The unknowns are the water discharge and water surface elevation (WSE), which are solved for at each point in time and space, dependent on the time step selected and channel cross-section spacing.

Data required for the HD calculations in the dam include spatial data such as channel geometry and roughness, and plan view connectivity. Boundary conditions required are inflow discharge time series at the upstream model boundary and the Flint Wash tributary inflows, and a water surface vs. discharge rating curve at the Dam.

For a complete description of all MIKE 11 model component equations and numerical solution schemes see Appendix A.

6.1.2.1 *Hydraulic Model Formulation*

Four significant flood events were chosen for the flood hazard and inundation analysis: the 2-, 10-, and 50-year recurrence interval events according to USGS gage data, and the LACDPW's design storm event, the Capital Storm, based on a 50-year rainstorm and saturated watershed conditions. As mentioned, the 2-, 10-, and 50-year flood peaks (912 cfs, 4,920 cfs, and 12,364 cfs respectively) were estimated based on a statistical analysis of the USGS Gage on Arroyo Seco (as described in Section 4.8.3). The Capital Storm flood peak (20,026 cfs) is based on LACDPW rainfall-runoff analysis. The LACFCD Capital Flood hydrograph shape was scaled down to create inflow hydrographs for the 2-, 10-, and 50-year events.

The MIKE 11 model domain is between the upstream boundary at the JPL Bridge and Devil's Gate Dam, including approximately 2,800 feet of Flint Wash. The model represents the active channels (Arroyo Seco and Flint Wash), the dam bathymetry, and the outlet structures as a connected network of branches (2 channels), and a control structure. The rating curve used at the downstream boundary was provided by LACFCD and is shown in Figure 4.9. For one proposed ("preferred") conditions simulation (using the 50-year (Capital) event) the reservoir was assumed to be partially full (elevation 1030 feet NGVD) at the beginning of the simulation to evaluate the effect this might have on water levels in the park and on flow-rates downstream of the dam.

The hydraulic model is composed of two main branches, Arroyo Seco and Flint Wash. The two branches are stationed separately, in meters, descending from upstream to downstream. Resistance values were estimated by field inspection and set to a Mannings' "n" of 0.04 for all computational nodes.

Each branch is physically represented in the MIKE 11 model as a series of cross-sections. For the existing conditions scenario, cross-sections were extracted from the November 1995 topographic survey and supplemented at the JPL bridge with information from PWA's January 1999 topographic survey. Existing conditions cross-sections were modified as described in Section 7.3.3.3 to represent the "preferred" park conditions topography. Plan view cross-section locations are presented in Figure 6.1.

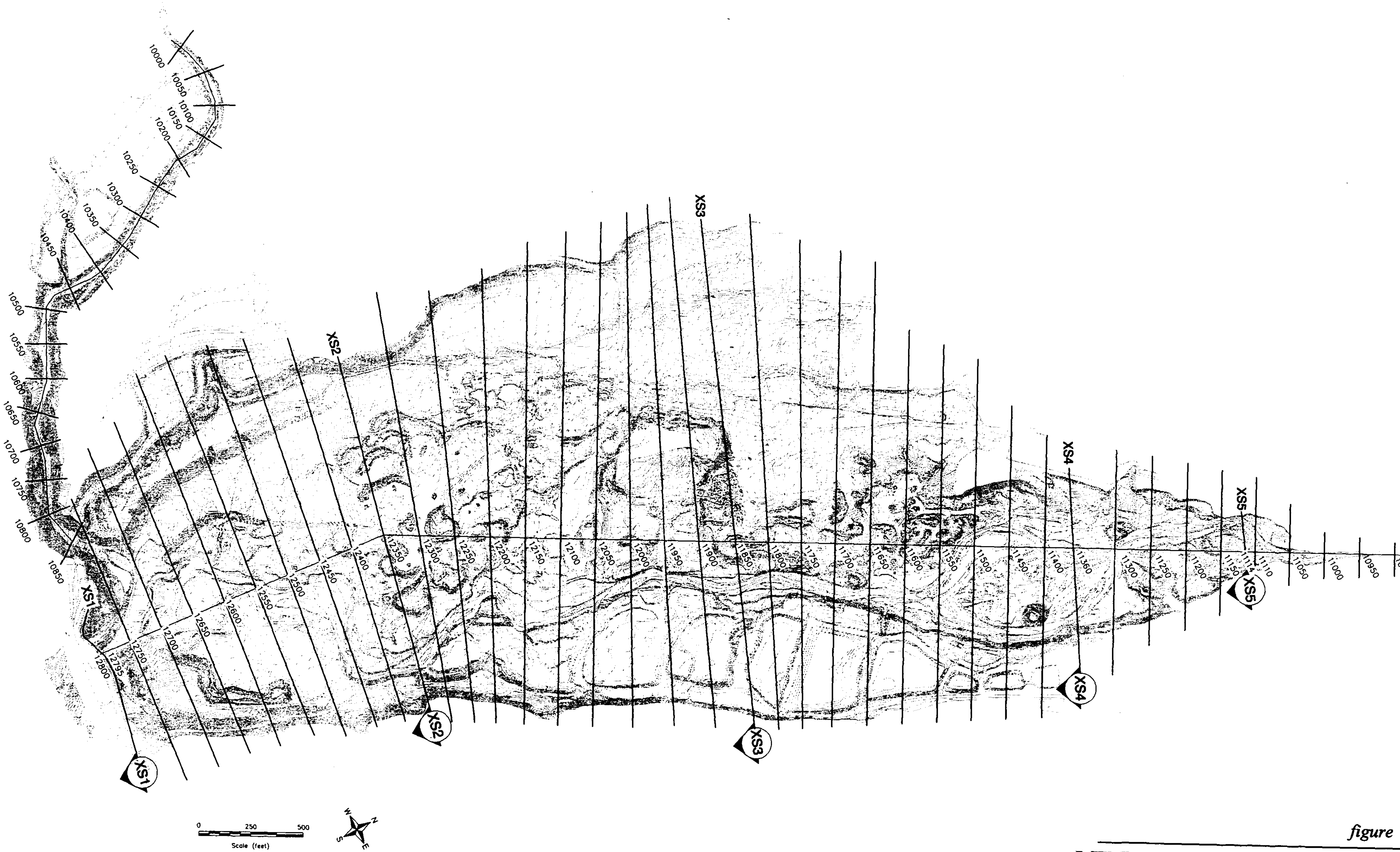

figure 6.1

MIKE 11 Cross Section Transects
Hahamongna Watershed Park

D:\projects and CAD\1310 Haha\Master Plan\Sectins.dwg

6.1.2.2 *Hydraulic Model Results*

The MIKE 11 hydrodynamic simulation yielded one-dimensional estimates of water surface elevation, discharge, and velocity at all computational nodes. The estimated extent of inundation under existing conditions for the various modeled design storms is presented in Figure 6.2. This inundation map represents the estimated maximum water surface elevations during the design storms. However, it should be noted that water surface elevations were calculated assuming a fixed channel bed. That is, channel morphological changes during the flood events were not accounted for. Since morphological changes can be extremely large and unpredictable during flood events, especially in the upper portion of the park (which resembles an alluvial fan), the flood hazard zone may actually cover more area than is shown. For example, it is likely that the Arroyo Seco Spreading Grounds will be inundated during a 50-year or Capital flood event. Table 6.1 presents the maximum water surface elevations for each computational node in the model for all four design events. The table can be referenced to the cross-section map (Figure 6.1). Design storm inflow hydrographs are shown with modeled outflow hydrographs and WSE's at the dam face for the 10-year, 50-year, and Capital storm events under existing conditions on Figures 6.3 through 6.5. It should also be noted that relative to the uncertainties associated with actual existing flood hazards, especially in the upper part of the basin, there would not be any significant change in flood hazard areas under proposed grading conditions. Under proposed conditions flood hazards would remain appreciable, but relatively unchanged from existing conditions.

If the inundation hazard map or the hydrodynamic model results are used as a tool for locating and designing structures, they should be used conservatively as the modeled inflow hydrographs are based on a flood frequency analysis which contains uncertainty, and since morphological channel changes during flood events were not accounted for. All structures should either be flood proofed, or elevated above acceptable inundation hazard zones within the park.

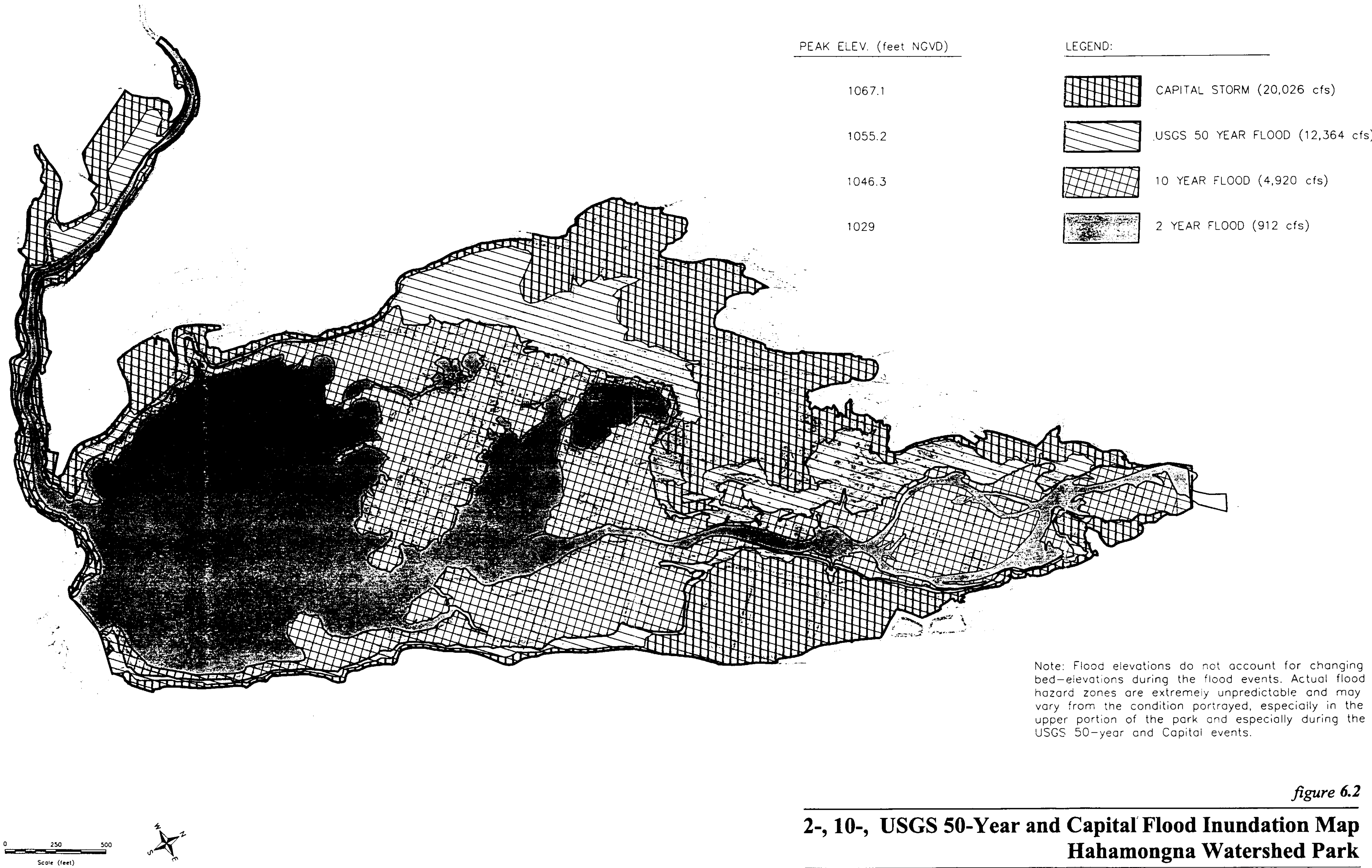

figure 6.2

2-, 10-, USGS 50-Year and Capital Flood Inundation Map
Hahamongna Watershed Park

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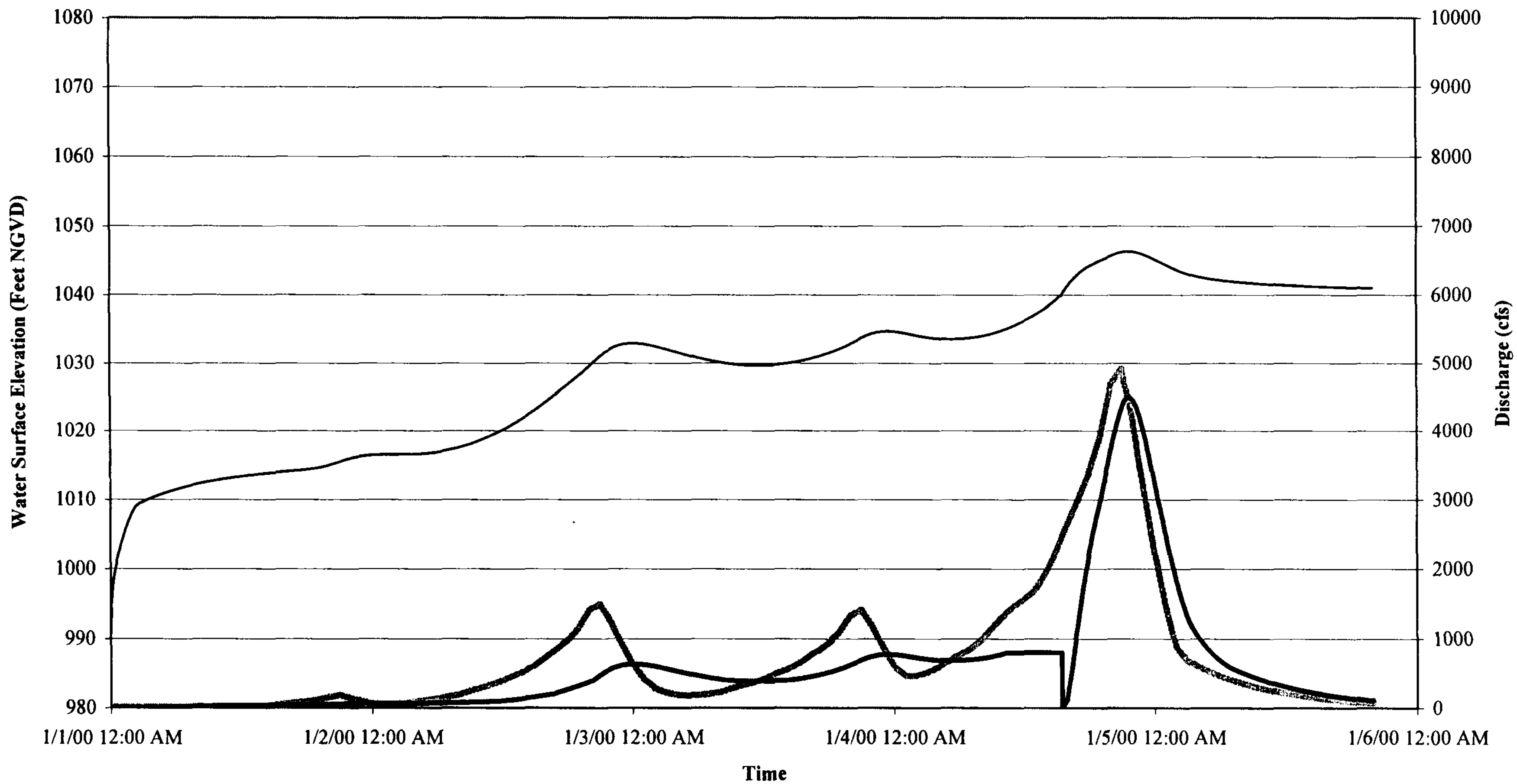
TABLE 6.1 Maximum Water Surface Elevation for 2-, 10-, 50-Year, and Capital Design Storms, Existing Conditions.

Station ID	2-yr Max WSE (ft NGVD)	10-yr Max WSE (ft NGVD)	50-yr Max WSE (ft NGVD)	Capital Max WSE (ft NGVD)
FLINT 10.000	1061.8	1064.0	1069.6	1068.4
FLINT 10.050	1060.1	1062.1	1067.2	1067.2
FLINT 10.100	1057.8	1060.3	1064.9	1067.2
FLINT 10.150	1056.5	1058.9	1062.6	1067.1
FLINT 10.200	1054.4	1057.1	1060.3	1066.9
FLINT 10.250	1052.2	1054.9	1058.0	1067.0
FLINT 10.300	1050.0	1052.5	1055.7	1067.1
FLINT 10.350	1048.5	1050.6	1055.1	1067.1
FLINT 10.400	1047.3	1049.1	1055.1	1067.1
FLINT 10.450	1045.4	1047.4	1055.2	1067.1
FLINT 10.500	1042.4	1046.3	1055.1	1067.1
FLINT 10.550	1040.9	1046.3	1055.2	1067.1
FLINT 10.600	1038.9	1046.3	1055.2	1067.1
FLINT 10.650	1034.3	1046.3	1055.2	1067.1
FLINT 10.700	1030.4	1046.3	1055.2	1067.1
FLINT 10.750	1029.9	1046.3	1055.2	1067.1
FLINT 10.800	1029.9	1046.3	1055.2	1067.1
FLINT 10.850	1029.9	1046.3	1055.2	1067.1
DEVIL'S 10.657	1102.9	1106.1	1109.3	1111.6
DEVIL'S 10.706	1098.0	1100.6	1103.6	1105.7
DEVIL'S 10.755	1094.0	1096.4	1099.0	1101.1
DEVIL'S 10.804	1089.9	1092.4	1094.9	1096.9
DEVIL'S 10.853	1085.8	1088.4	1091.1	1092.9
DEVIL'S 10.902	1081.6	1084.5	1087.2	1089.1
DEVIL'S 10.951	1077.5	1080.5	1083.2	1085.1
DEVIL'S 11.000	1073.4	1076.4	1079.0	1080.9
DEVIL'S 11.038	1070.3	1072.9	1075.0	1076.9
DEVIL'S 11.076	1067.2	1069.4	1071.0	1072.7
DEVIL'S 11.114	1064.0	1065.7	1067.2	1068.5
DEVIL'S 11.150	1058.9	1061.4	1063.2	1066.9
DEVIL'S 11.200	1055.4	1057.4	1058.9	1067.1
DEVIL'S 11.250	1053.1	1054.5	1056.1	1067.1
DEVIL'S 11.300	1050.1	1051.3	1055.2	1067.1

TABLE 6.1 (continued)

Station ID	2-yr Max WSE (ft NGVD)	10-yr Max WSE (ft NGVD)	50-yr Max WSE (ft NGVD)	Capital Max WSE (ft NGVD)
DEVIL'S 11.330	1048.8	1050.2	1055.2	1067.1
DEVIL'S 11.360	1047.5	1049.0	1055.2	1067.1
DEVIL'S 11.400	1045.9	1047.7	1055.2	1067.1
DEVIL'S 11.450	1044.4	1046.8	1055.2	1067.1
DEVIL'S 11.500	1043.5	1046.6	1055.2	1067.1
DEVIL'S 11.550	1041.7	1046.5	1055.2	1067.1
DEVIL'S 11.600	1040.3	1046.4	1055.2	1067.1
DEVIL'S 11.650	1038.8	1046.4	1055.2	1067.1
DEVIL'S 11.700	1037.4	1046.3	1055.2	1067.1
DEVIL'S 11.750	1036.2	1046.2	1055.2	1067.1
DEVIL'S 11.800	1034.5	1046.2	1055.2	1067.1
DEVIL'S 11.850	1032.6	1046.3	1055.2	1067.1
DEVIL'S 11.900	1029.9	1046.3	1055.2	1067.1
DEVIL'S 11.950	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.000	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.050	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.100	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.150	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.200	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.250	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.300	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.350	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.400	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.450	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.500	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.550	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.600	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.650	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.700	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.750	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.795	1029.9	1046.3	1055.2	1067.1
DEVIL'S 12.800	1029.9	1046.3	1055.2	1067.1



Source:
Note:

figure 6.3

MIKE 11 10-Year Hydrodynamic Results: Inflow & Outflow Hydrographs, and Dam Face WSE

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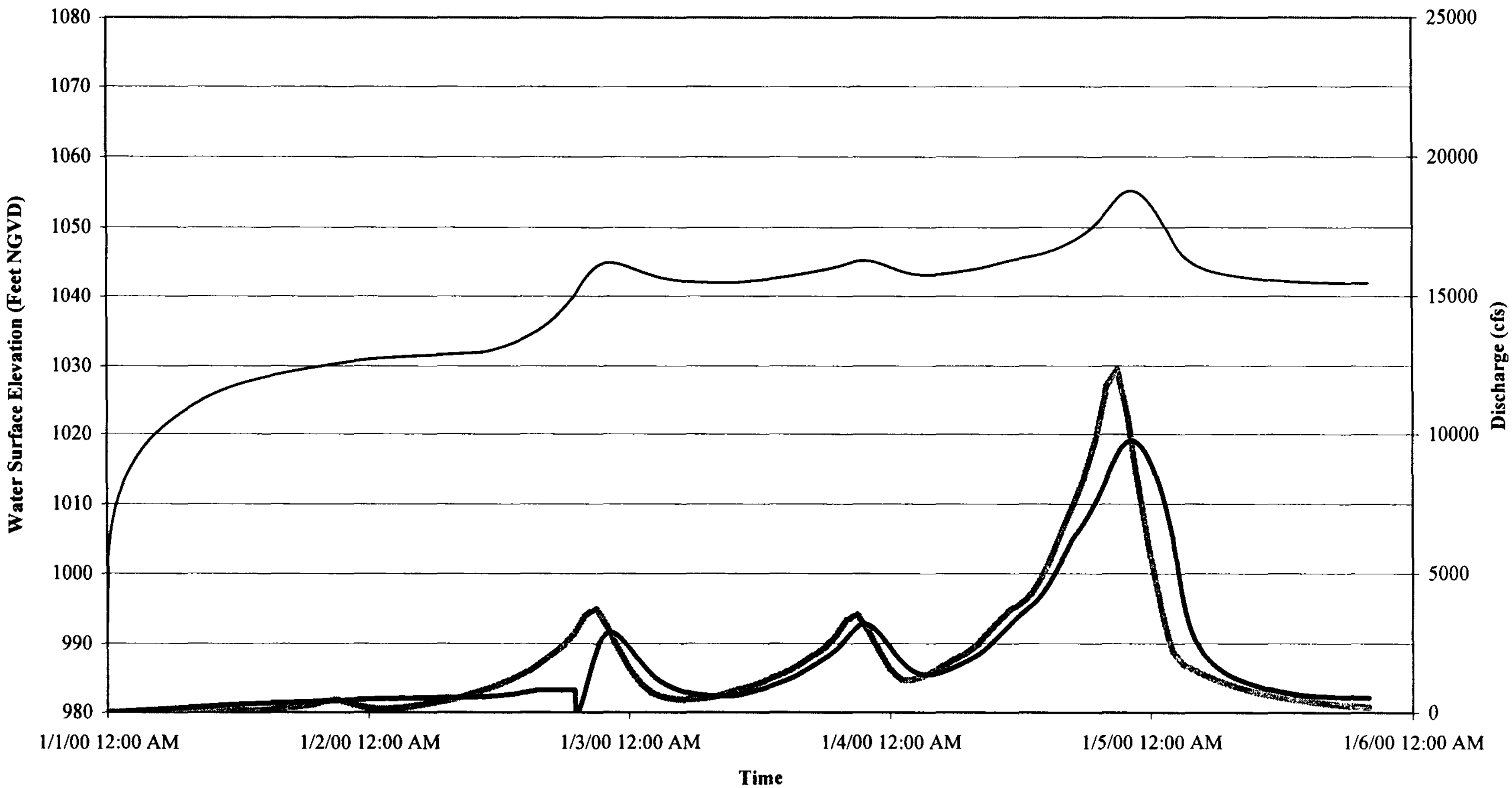

Source:
Note:

WSE at Dam
Inflow
Outflow

figure ***6.4***

MIKE11 USGS 50-Yr Hydrodynamic Results: Inflow & Outflow Hydrographs, and Dam Face WSE

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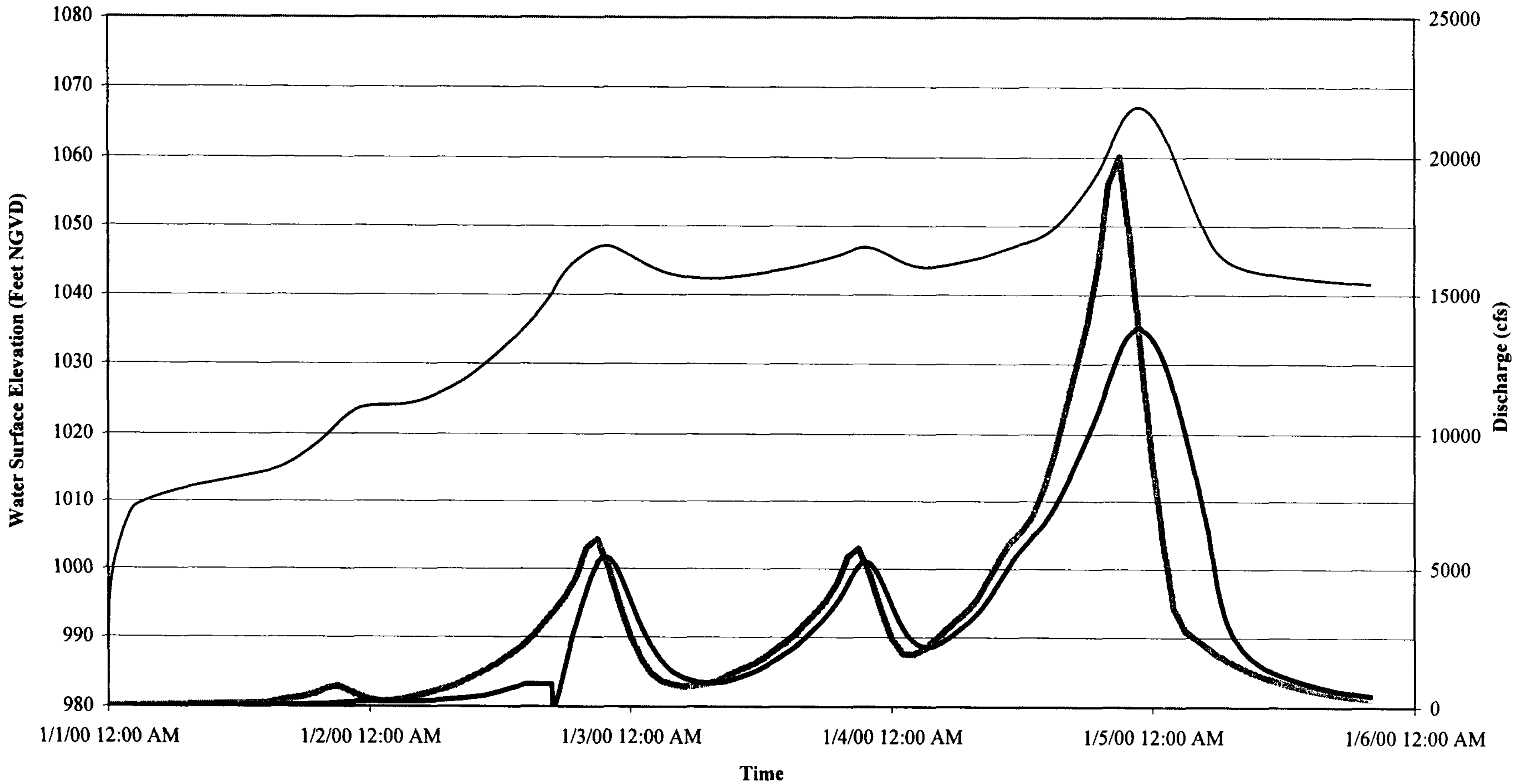

Source:
Note:

WSE at Dam
Inflow
Outflow

figure 6.5

MIKE 11 Capital Flood Hydrodynamic Results: Inflow & Outflow Hydrographs, and Dam Face WSE

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6.2 FLUVIAL EROSION HAZARDS

Bed and bank erosion are natural stream processes of Arroyo Seco that can occur rapidly during floods. Where the stream has a meandering planform pattern, Arroyo Seco typically moves laterally by erosion on one bank and simultaneously by deposition on the other. Where Arroyo Seco is a braided channel, it is extremely unstable, especially where separated by bars of gravel rather than vegetated islands. Following lateral movement of either channel configuration, however, channel geometries (e.g., width and depth) are maintained. That the reservoir is a relatively flat plain in the Hahamongna Watershed Park suggests in part that over time the channel may have been active along all positions of the observed valley flat.

Erosion of the channel banks by channel migration during flood events poses a potential hazard to structures within the floodplain of Arroyo Seco. PWA used photographic and map evidence dating back to the 1930s to analyze the progressive sequence of channel migration and floodplain construction. Ultimately, this information on past migration of Arroyo Seco is used to define those areas where fluvial activity (erosion and deposition) is likely to continue within the Hahamongna Watershed Park.

6.2.1 Historical Aerial Photographs and Topographic Maps

Aerial photographs provided by LACPWD were used to establish the extent of active fluvial zones in the Devil's Gate Reservoir since 1969, concurrent with management activities during this period (Table 6.2). Photographs vary in scale, spatial extent, and degree of inundation in the reservoir. In addition to the low-flow channel position, sediment management activities and vegetation patterns are apparent in the photographs.

TABLE 6.2 Historical Aerial Photographs

Date	Photo Numbers	Scale	Notes (Water Level, Photographic Coverage)
12/10/69	69297-3 69297-5	not shown	Extensive sediment management activities apparent throughout basin. Local ponding in lowermost basin; channel(s) through disturbed upper basin.
2/13/73	73021-22-024 73021-24-026 73021-26-028 73021-28-030	not shown	Lower basin inundated; water level high upper half of basin.
3/20/80	80062-045-3 80062-047-5	1: 6,000	Ponding in lower portions of basin; channel(s) through upper basin.
9/30/82	82182-2 82182-4	1: 6,000	Low flow channel(s) throughout basin.
4/1/83	83083-002 83083-004	1: 6,000	Low flow channel(s) throughout basin.
1/16/98	ADM-1721-1-1 ADM-1721-1-2 ADM-1721-1-3	1: 3,600	Photo coverage of lower half of basin only.

LACPWD also supplied historical topographic maps of the basin dated November 1934 (Map No. 65-T5), June 1938 (Map No. 65-T7), January 1942 (Map No. 65-T8), and July 1981 (Map No. 65-T61). Each map was reviewed at the 1:1,200 scale. These maps were helpful in identifying likely channel paths and localized sediment management activities. These maps were also used in sedimentation analyses described in later sections of this report (Section 7).

6.2.2 Observed Erosional Patterns

Since 1934, natural channel migration in the Arroyo Seco basin has been disturbed by spatially variable sediment management activities within the reservoir. Virtually all of the land surface within the reservoir basin has been altered, removed, or overturned by earth-moving equipment (Cotton/Beland/Associates, Inc., 1988). Historical photographs and topographic maps indicate that sediment management has occurred throughout the upper, middle, and lower basin areas. Thus, it is impossible to assess what aspects of channel migration are primarily fluvial in origin versus anthropogenically influenced.

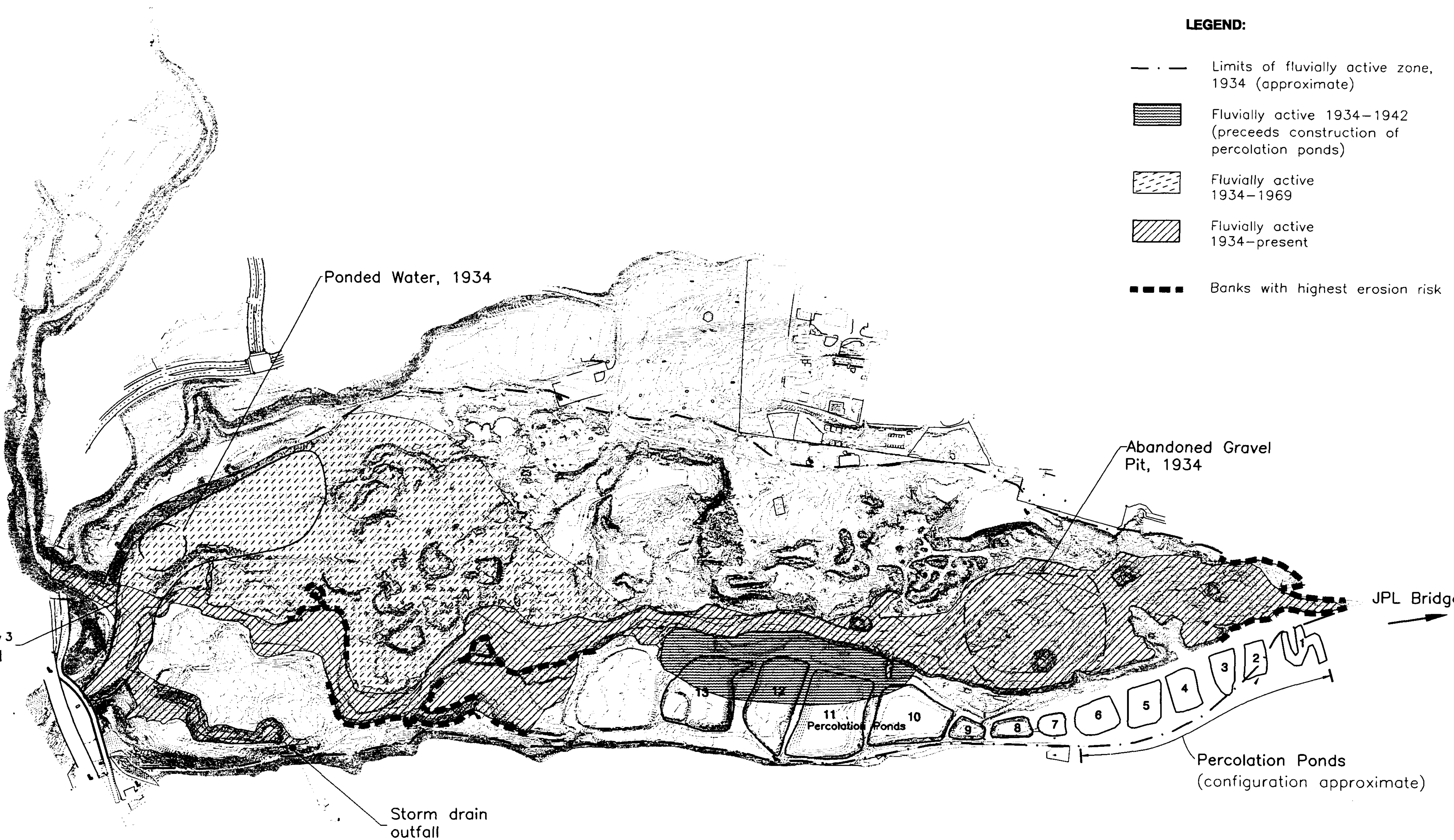
Figure 6.6 indicates the interpreted spatial limits of fluvial activity based on the historical photographs and topographic maps. It is apparent that the gross topography of the reservoir and bank protection along the perimeter of the reservoir has set some absolute limits to channel migration. This is indicated by the "limit of fluvially active zone" line in Figure 6.6. Although the thalweg of Arroyo Seco has more commonly been situated within the central portion of the reservoir, occasionally braids have extended into the broader region.

In the upper half of the reservoir (upstream of percolation pond 13), Arroyo Seco is braided and has typically been contained within a 800-foot wide corridor within the past 65 years. Variations in migration of the active braided channel appear to be strongly linked to sediment and water supply management. The widest active channel area (alongside ponds 6 through 9) coincides with an abandoned gravel pit apparent in the 1934 topographic map. Conversely, sometime between 1942 and 1969, construction of the percolation ponds significantly encroached upon the active channel from the south along the present-day percolation ponds 10 through 13 (Figure 6.6).

In the lower half of the reservoir (downstream of the present-day percolation ponds), channel activity has been less defined and extends across the majority of the lower reservoir. The transition zone between fluvial processes and ponding has been situated in this lower portion of the reservoir. As Arroyo Seco reaches the ponded portion of the reservoir, rapid sedimentation can lead to unpredictable abandonment of a channel segment and subsequent formation of a new channel course. This process of sedimentation is a likely reason for the relatively broad lateral limits of channel migration indicated by historic maps and photographs. If sediment management activities were more extensive and spatially variable in the lower reservoir, this may also have contributed to more variable lateral channel movement.

Since 1980, the active channel has remained within a more clearly defined corridor in the reservoir (parallel diagonal lined area in Figure 6.6). These areas are the most likely to experience erosional hazards with the existing park conditions, and should not contain any park structures vulnerable to flooding or bank erosion.

Future erosion risks are strongly linked to current channel planform pattern, as well as adjacent management activities. In the upper portion of the reservoir, the channel is braided with rapid changes in the size, shape, and number of midchannel bars. In these relatively unconstrained portions of Arroyo Seco (e.g., along percolation ponds 6 through 9) , the area is mostly depositional with moderate and localized bank erosion risks. However, where the channel is constrained by percolation ponds (e.g., alongside the diversion structure and ponds 1 through 2), channel banks are extremely steep and erosion risks are very high. Some sort of channel stabilization should be implemented in these areas. In the lower portion of the reservoir, the channel is meandering, forming a sinuous single-thread incised into past stream deposits. The largest erosional risks are associated with the outside of meander bends in the lower portion of the basin. If erosion along the meander bends poses significant design constraints to the park, bank stabilization techniques along



Map units are superimposed on 1995 topography (Source: LACDPW).

NOTE: The zones of fluvial erosion shown here do not include area inundated by water stored behind the dam. This map reflects only active channel processes apparent in historical aerial photographs, historical topographic maps, and recent field observations. It should also be noted that significant gravel mining activities have taken place over the years in the reservoir area and have significantly affected the geomorphology of the area.

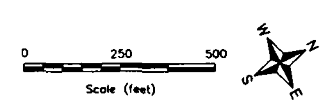


figure 6.6

Geomorphic Map of Historical Channel Movement

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the concave outer bends could be considered. However, wherever possible, due to the dynamic nature of the Arroyo Seco channel in the park area, it should be allowed to migrate and move within the floodplain. Throughout the Hahamongna Watershed Park, vulnerable structures should be located away from channel banks threatened by erosion.

While the limits of the active channel have remained similar throughout the last three decades, if reservoir operation and/or sediment management activities dramatically change, the channel may adjust in new ways that will be difficult to predict based on the observed historically active fluvial environment. In addition, the limited historical record available through aerial photographs and maps does not illustrate the potential erosional hazards posed by extreme flood events (>50-year recurrence interval).

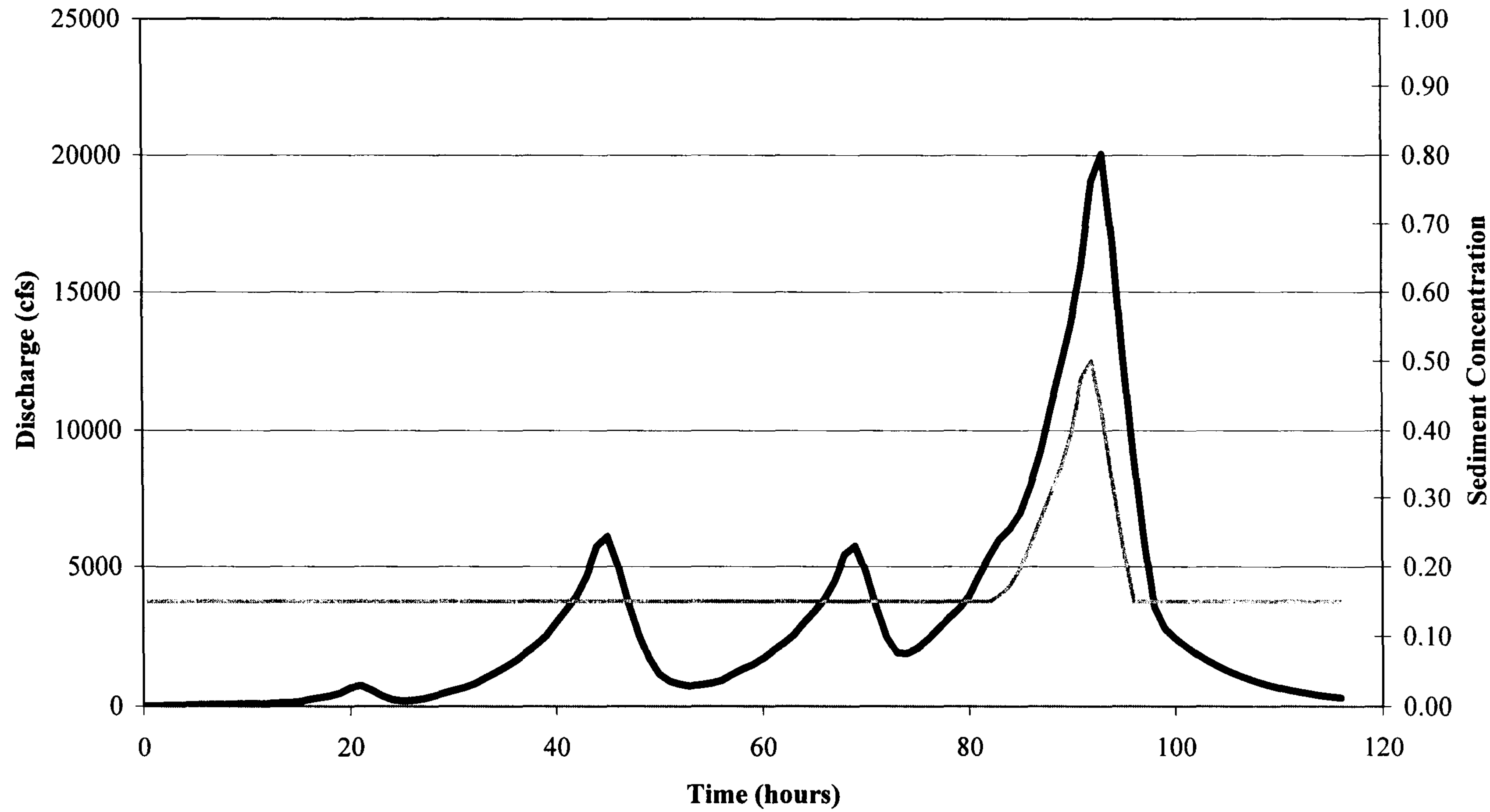
6.3 DEBRIS FLOW HAZARDS

6.3.1 Mud and Debris Flow Modeling

Since very little is known about debris flows in the Hahamongna Watershed Park, specific recurrence interval debris events cannot be determined. At best we can estimate a debris hazard zone where hazardous debris flows are expected periodically. Depending on antecedent watershed conditions (as outlined in Sections 4.9.3 through 4.9.5), a range of debris events may result after rain events (ranging from no significant mud and debris to catastrophic mud and debris events). Because a close correlation has not been found or proven between peak flood events and amount of debris mobilized, our approach is to look at a range of peak flows, coupled with a "worst case" sediment concentration inflow. In order to model the extremely complex physical processes that govern the transport of mud and debris, the FLO-2D two-dimensional debris model was utilized.

6.3.1.1 *FLO-2D Model Formulation*

Three inflow conditions were modeled: the 2-, 10-, and 50-year flood events (see Section 6.1.2.1) were coupled with an assumed "worst case" hyper concentrated sediment influx. The "worst case" influx is based on information provided by the FLO-2D manual (FLO Engineering, 1998). The influx sediment concentration time series has a baseline of 0.15 by volume, and reaches a maximum one hour before the flood peak with a concentration of 0.5 sediment by volume (see Figure 6.7). The sediment inflow represents a conventional water flood with a concentration of 0.15, transitioning to a mud flood, and peaking as a mudflow at a concentration of 0.5. Table 6.3 describes flow characteristics for various sediment concentrations.



Bulked & Burned Inflow Hydrograph

"Worst Case" Sediment Influx

Notes:
1) Bulked and burned inflow is from LADPW Capital Flood Inflow Hydrograph to Devil's Gate Dam
2) Sediment timing was shifted back by 1 hour.

figure ***6.7***

LACDPW Capital Flood With "Worst Case" Sediment Concentration Hydrograph

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TABLE 6.3 Mudflow Behavior as a Function of Sediment Concentration

Flow Description	Sediment Concentration by Volume	Sediment Concentration by Weight	Flow Characteristics
Landslide	0.65-0.08	0.83-0.91	Will not flow; failure by block sliding
	0.55-0.65	0.76-0.83	Block sliding failure with internal deformation during the slide; slow creep prior to failure
Mudflow	0.48-0.55	0.72-0.76	Flow evident; slow creek sustained mudflow; plastic deformation under its own weight; cohesive; will not spread on level surface
	0.45-0.48	0.69-0.72	Flow spreading on level surface; cohesive flow; some mixing
Mud Flood	0.40-0.45	0.65-0.69	Flow mixes easily; shows fluid properties in deformation; spreads on horizontal surface but maintains an inclined fluid surface; large particle (boulder) setting; waves appear but dissipate rapidly
	0.35-0.40	0.59-0.65	Marked setting of gravels and cobbles; spreading nearly complete on horizontal surface; liquid surface with two fluid phases appears; waves travel on surface
	0.30-0.35	0.54-0.59	Separation of water on surface; waves travel easily; most sand and gravel has settled out and moves as bedload
	0.20-0.30	0.41-0.54	Distinct wave action; fluid surface; all particles resting on bed in quiescent fluid condition
Water Flood	<0.20	<0.41	Water flood with conventional suspended load and bedload

Source: FLO-2D Manual, FLO Engineering, 1998.

Using the November 1995 topographic survey, a 30-meter bathymetric grid was prepared as input to the 2-dimensional FLO-2D model. The FLO-2D model grid, as shown in Figure 6.8, extends from 110 meters upstream of the JPL Bridge down to the Dam face and includes the lower portion of Flint Wash.

6.3.1.2 *FLO-2D Results & Discussion*

The FLO-2D model estimates a sediment concentration at each grid cell for every time step for the duration of the modeled events. These results were filtered in order to find the maximum sediment concentration for each grid cell for the different events. Based on a concentration threshold of 0.20 by volume, a debris hazard zone was estimated (see Figure 6.9). All computational nodes reflecting a concentration of 0.20 or higher

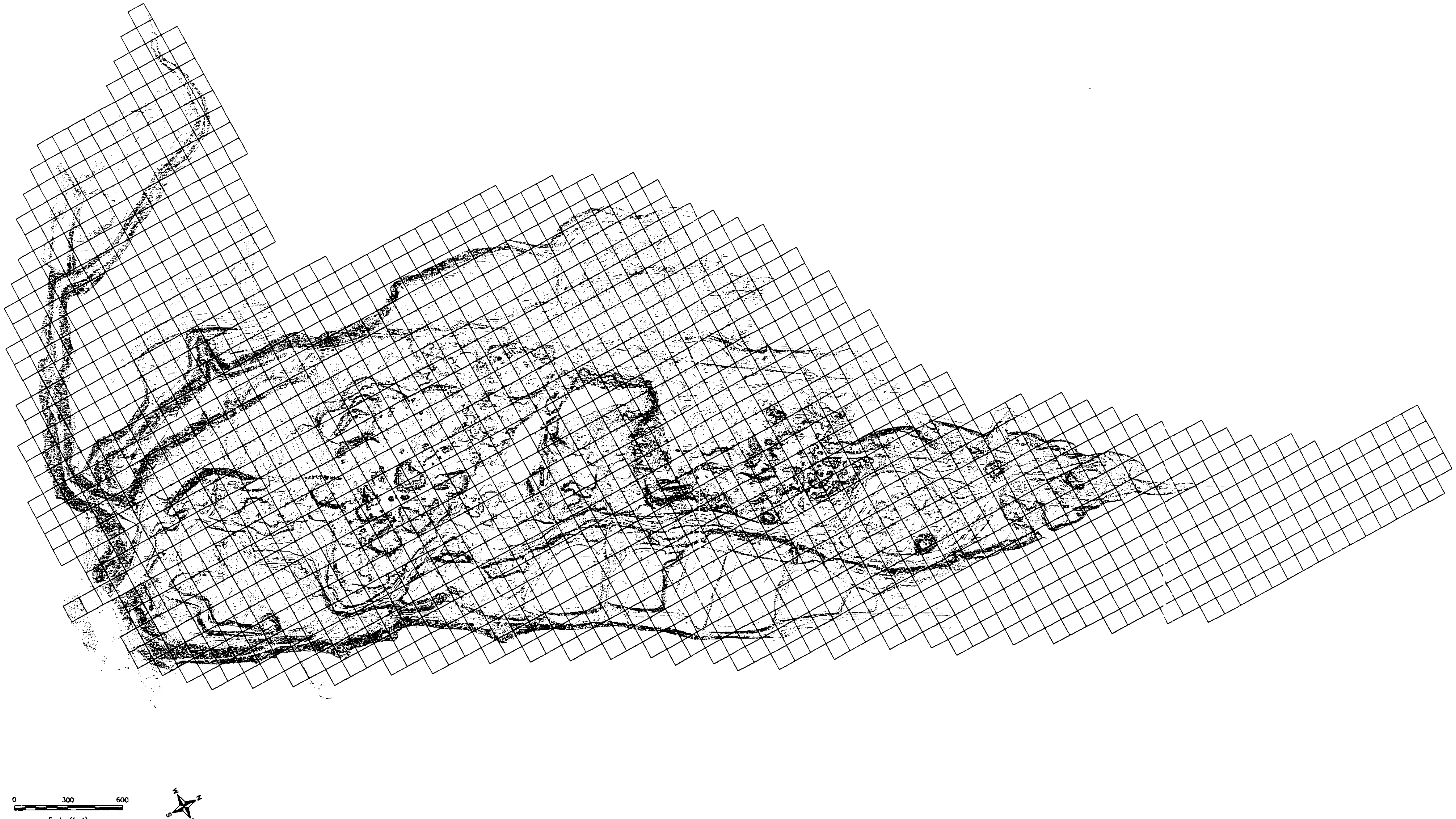
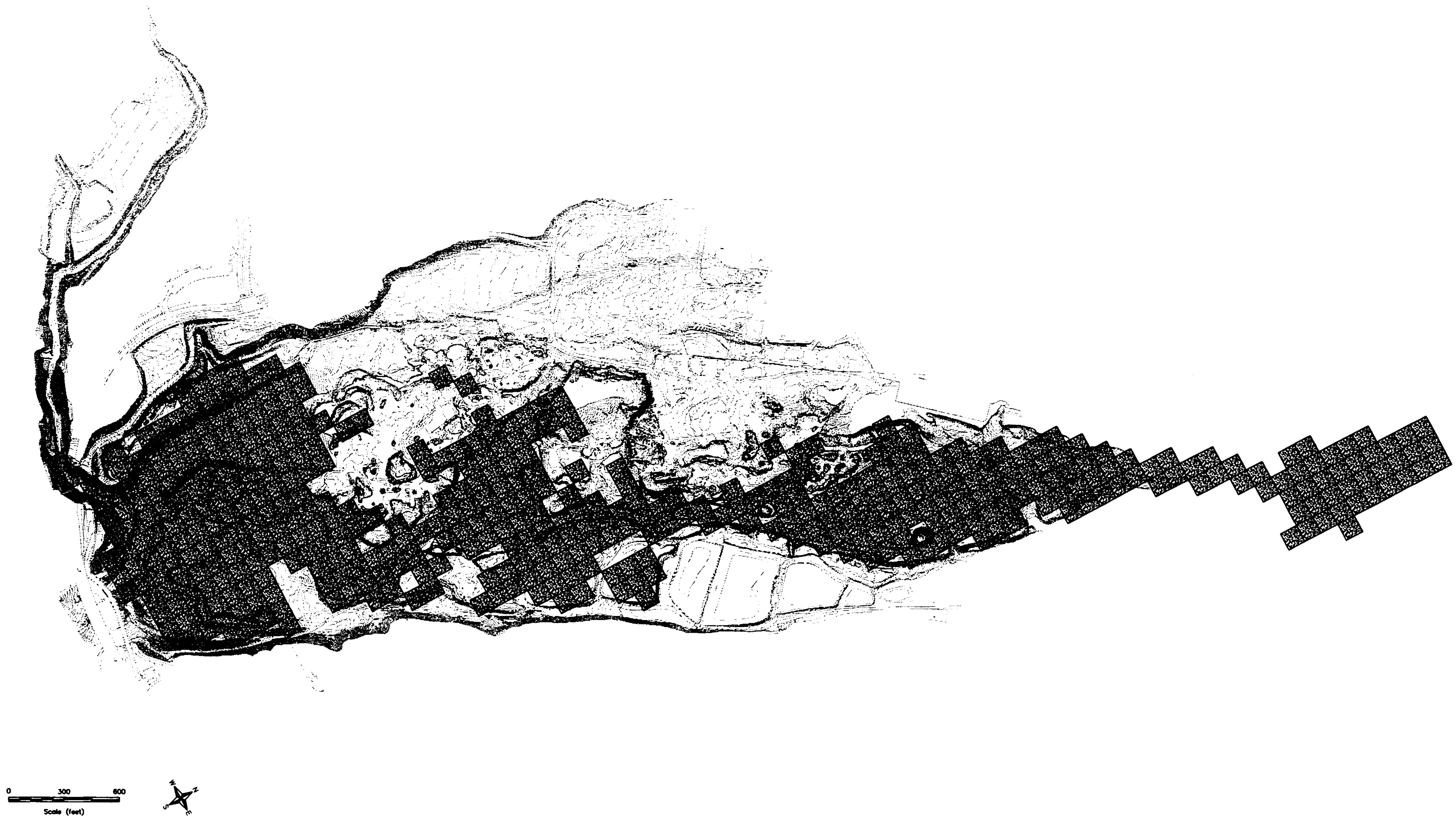


figure 6.8

FLO-2D Model Grid
Hahamongna Watershed Park



Note:
The debris hazard zone represents the zone for which a flow sediment concentration of 0.20 by volume or greater was calculated using FLO-2D for either the 2-year, 10-year, or Capital Storm events.

figure 6.9

Estimated Debris Hazard Zone
Hahamongna Watershed Park

in the 2-year, 10-year, and Capital flood events were designated as within the debris zone. A 0.20 concentration by volume represents the transition between a water flood and a mud flood. It is believed that concentrations in the 0.20 range and above are potentially hazardous to structures. It is possible that debris flows will surge outside of the mapped debris hazard zone due to the turbulent nature of debris flows. Any structures, or other objects in or proximal to the debris hazard zone are potentially subject to the extremely large forces experienced in debris events. Therefore, we do not recommend building structures within the mapped debris hazard zones. The only exceptions are at grade roads and paths which may cross through the debris hazard zones with the understanding that they will require maintenance after significant events.

6.4 DEVIL'S GATE DAM OPERATIONS

Changes to the Devil's Gate Dam operation procedures were qualitatively considered in this study. From the perspective of flood hazards in the Hahamongna Watershed Park the most beneficial operating strategy for the dam would be to keep all gates open under all conditions and simply allow water to flow through as quickly as possible. This kind of passive operation strategy would provide some flood protection to downstream areas while minimizing the extent of inundation in the park. However, this operation strategy is not feasible from the perspective of sediment management. A completely passive system would likely promote excessive sediment accumulation in front of some of the lower dam outlets during large flood events. This accumulation could clog the gates causing increasing maintenance problems (sediment issues, as they pertain to dam operations, are discussed further in Section 7.3.2). Therefore, PWA's judgement is that current dam operations minimize flood elevations in the park as much as possible, given the constraints on dam operation (Section 5.2.3).

7. SEDIMENT MANAGEMENT PLAN

7.1 HISTORIC SEDIMENTATION

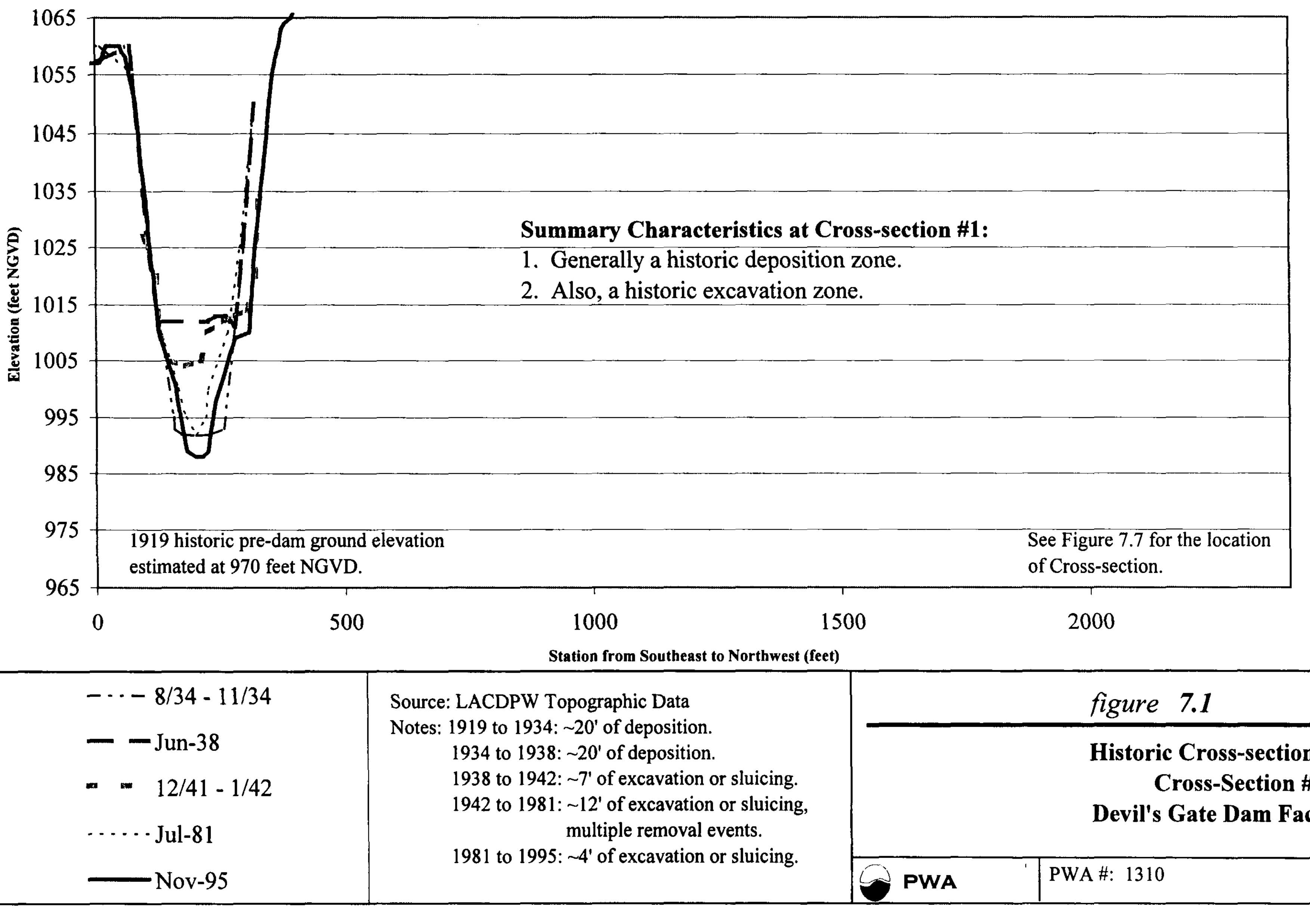
7.1.1 Cross-sections

Historic LACDPW maps were used to assess rates of sedimentation through changes in basin topography over time along selected cross-sectional transects. PWA selected six cross-sections through the basin approximately perpendicular to the Arroyo Seco channel in the basin (see Figure 6.1).

Three of the six cross-sections (cross-sections 2, 4, and 6) were surveyed during PWA's field visit in January 1999. Surveying was conducted using a total station and prism rod, following standard topographic surveying techniques. The positions of the field-surveyed cross-sections were identified in three-dimensional space by tying the survey into local County benchmarks within the park. The cross-sections were surveyed for three primary reasons: 1) for use in PWA's hydraulic modeling of the basin, 2) to provide baseline data in the event of a storm event, after which a second survey could be done to detect channel changes, and 3) for comparison with cross-sections available from the 1995 CAD map.

Figures 7.1 through 7.6 show topography along the six cross-sections for each available year and have notes describing the approximate geomorphic history of each cross-section. Cross-sections include data from 1919, 1934, 1938, 1941-42, 1981, 1995, and 1999. The 1919 estimates of channel bottom (or thalweg) elevation are based on a linear interpolation between an actual 1919 pre-dam channel elevation at the current location of the dam and the 1981 channel thalweg elevation at the JPL bridge, the earliest available data for this location. 1919 thalweg elevations should therefore be viewed as rough estimates only.

The cross-sections indicate a few major topographic trends. First, cross-sections 1 through 5 indicate broad sedimentation across the basin between 1919 and 1942. This is reflected by an increase in channel bed elevation of about 10 feet in the upper reservoir to over 40 feet at the dam face. Following 1942, the cross-sections document significant channel bed and reservoir surface lowering. This is a net result of extensive and persistent sediment management techniques (sluicing and excavation distributed across the reservoir) that essentially began in 1942, and fluvial reworking of past deposits. In the middle zone of the reservoir, cross-sections 3, 4, and 5 show net deposition within the low flow channel of approximately 5 feet between 1981 and 1995. At the upper and lower extent of the reservoir, cross-sections 1, 2, and 6 instead indicate net lowering of the channel bed (via downcutting, excavation, or some combination thereof). Unfortunately



Source: LACDPW Topographic Data

Notes: 1919 to 1934: ~20' of deposition.
1934 to 1938: ~20' of deposition.
1938 to 1942: ~7' of excavation or sluicing.
1942 to 1981: ~12' of excavation or sluicing, multiple removal events.
1981 to 1995: ~4' of excavation or sluicing.

figure 7.1

Historic Cross-sections
Cross-Section #1
Devil's Gate Dam Face

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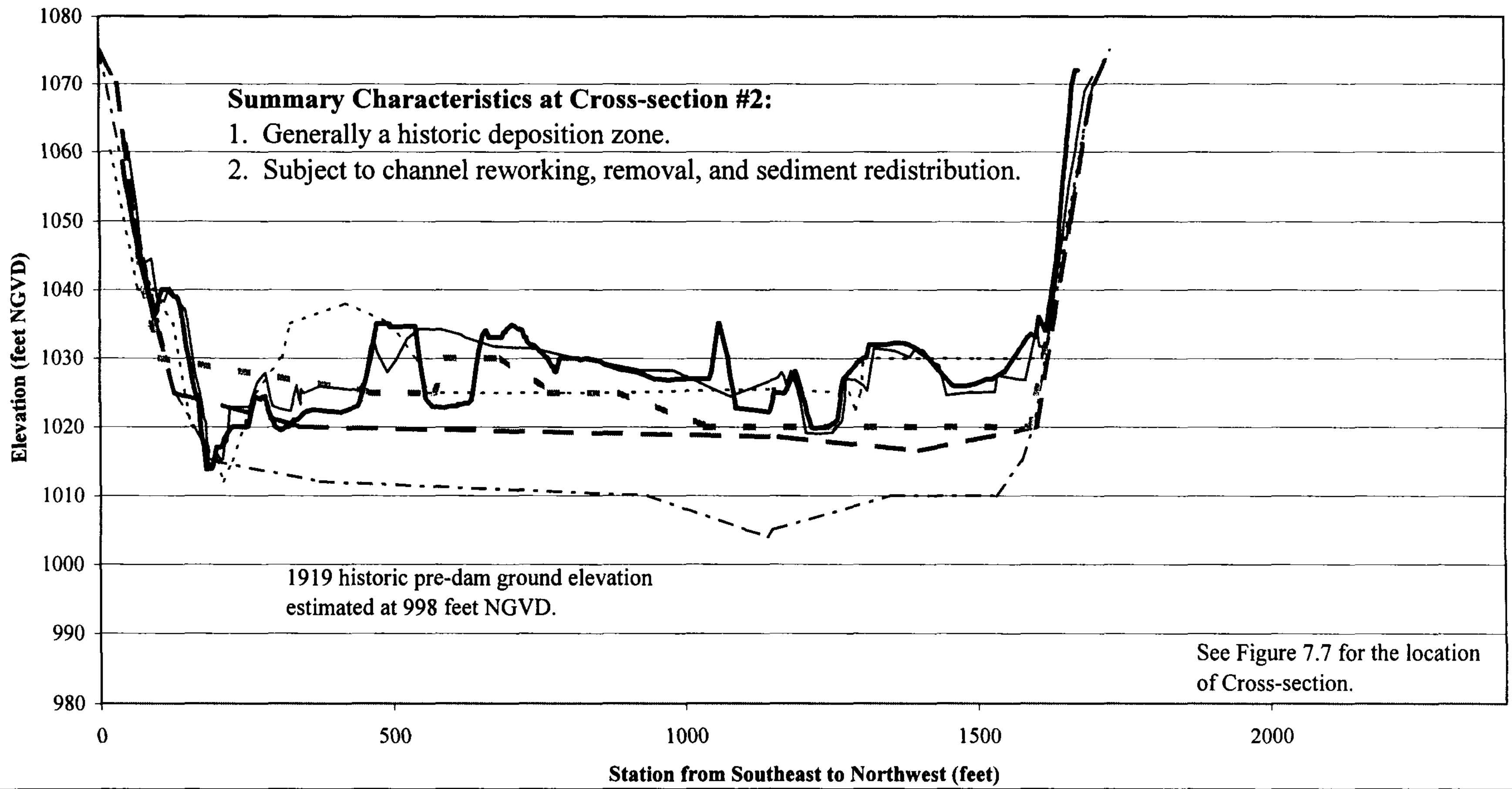

Source: LACDPW Topographic Data.

Notes: 1919 to 1934: ~5' of deposition.
1934 to 1938: ~10' of deposition.
1938 to 1942: ~5' of deposition with sediment removal and relocation.
1942 to 1981: ~5' of deposition with sediment removal and dumping.
1981 to 1995: Irregular removal & dumping.
1995 to 1999: Minor channel reworking & sediment redistribution.

figure 7.2

Historic Cross-sections
Cross-Section #2

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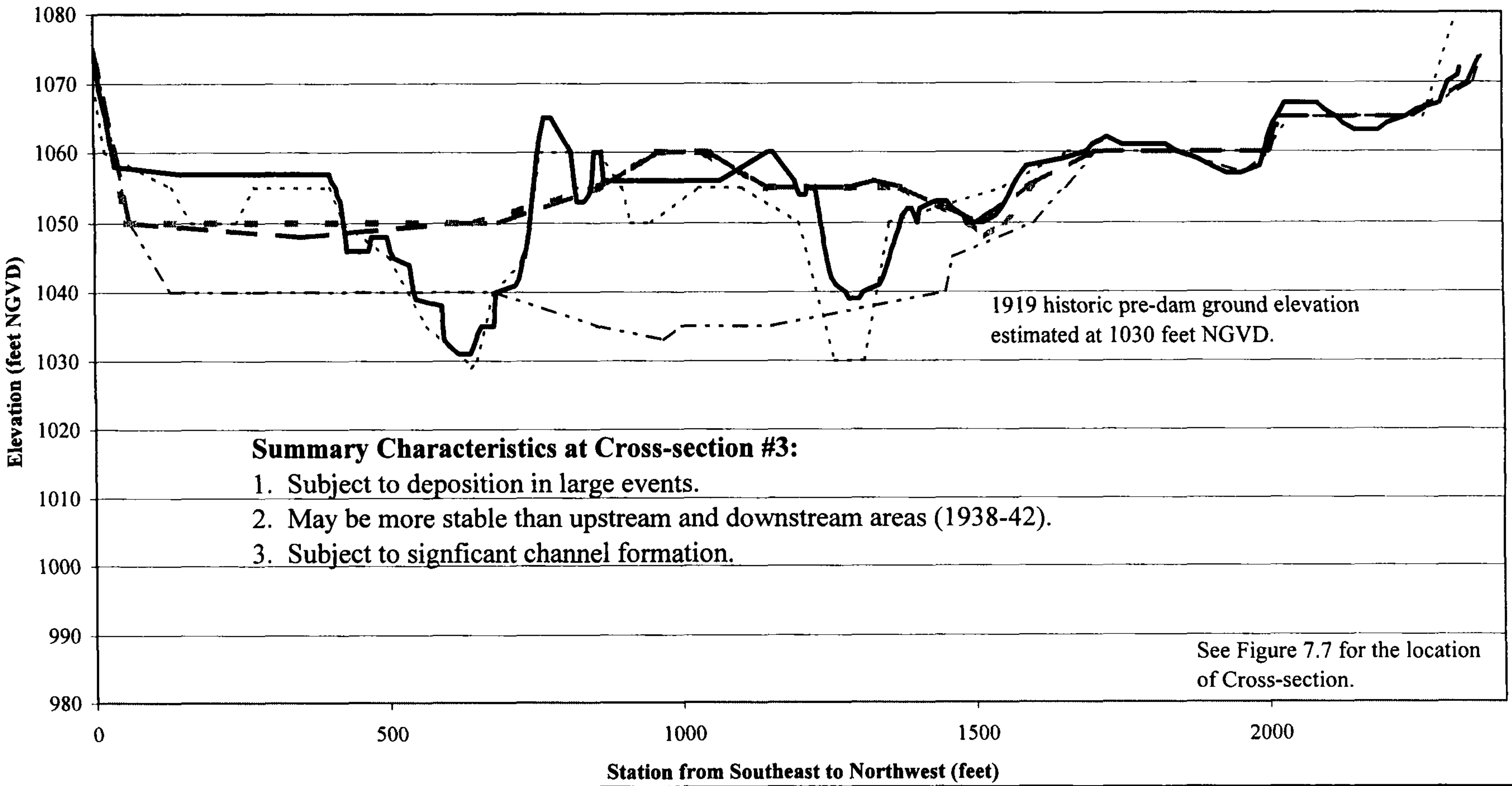


8/34 - 11/34
Jun-38
12/41 - 1/42
Jul-81
Nov-95

Source: LACDPW Topographic Data

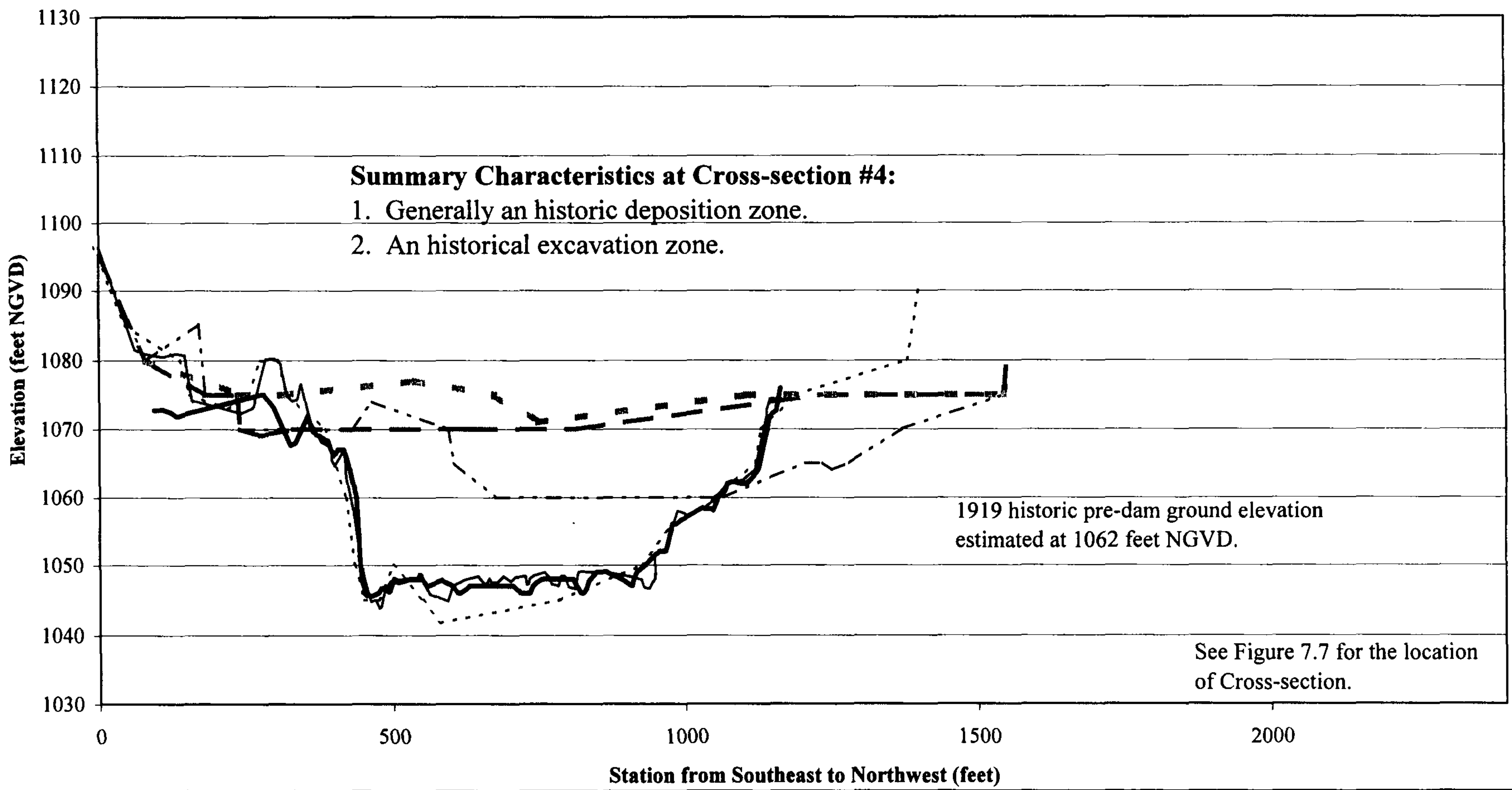
Notes: 1919 to 1934: minor deposition (uncertain).
1934 to 1938: 10'-20' of deposition in low areas, sediment redistribution.
1938 to 1942: No change.
1942 to 1981: Channel incision, sediment redistribution/dumping.
1981 to 1995: Channel aggradation.

figure 7.3

Historic Cross-sections
Cross-Section #3

PWA

PWA #: 1310



8/34 - 11/34	Jun-38
12/41 - 1/42	Jul-81
Nov-95	Jan-99

Source: LACDPW Topographic Data.

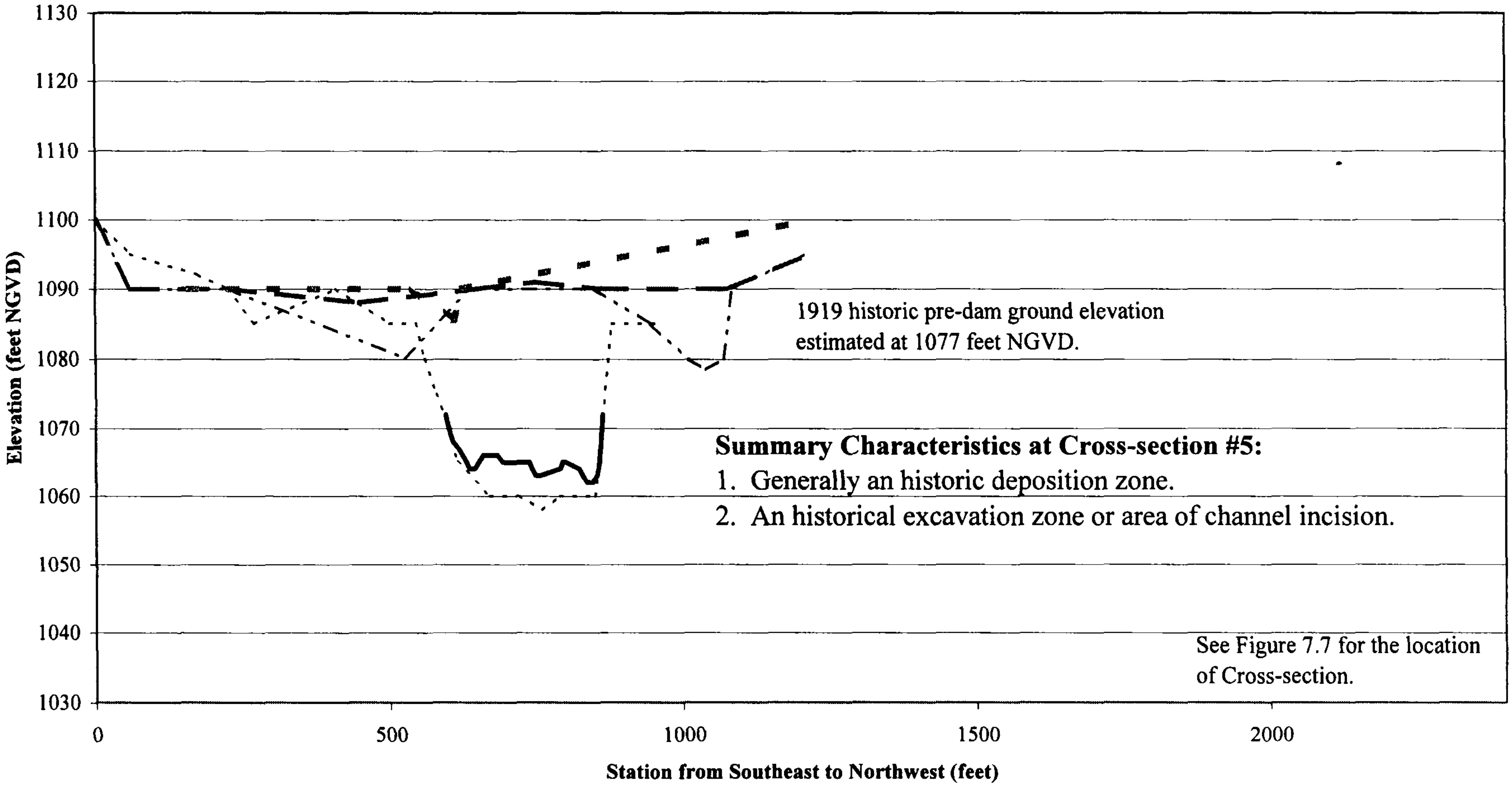
Notes: 1919 to 1934: Minor erosion/removal (uncertain).
1934 to 1938: ~10' of deposition.
1938 to 1942: ~7' of deposition over half of cross-section.
1942 to 1981: ~20' to 30' of excavation.
1981 to 1995: Minor deposition.
1995 to 1999: Minor channel reworking.

figure 7.4

Historic Cross-sections
Cross-Section #4

PWA

PWA #: 1310



8/34 - 11/34
Jun-38
12/41 - 1/42
Jul-81
Nov-95

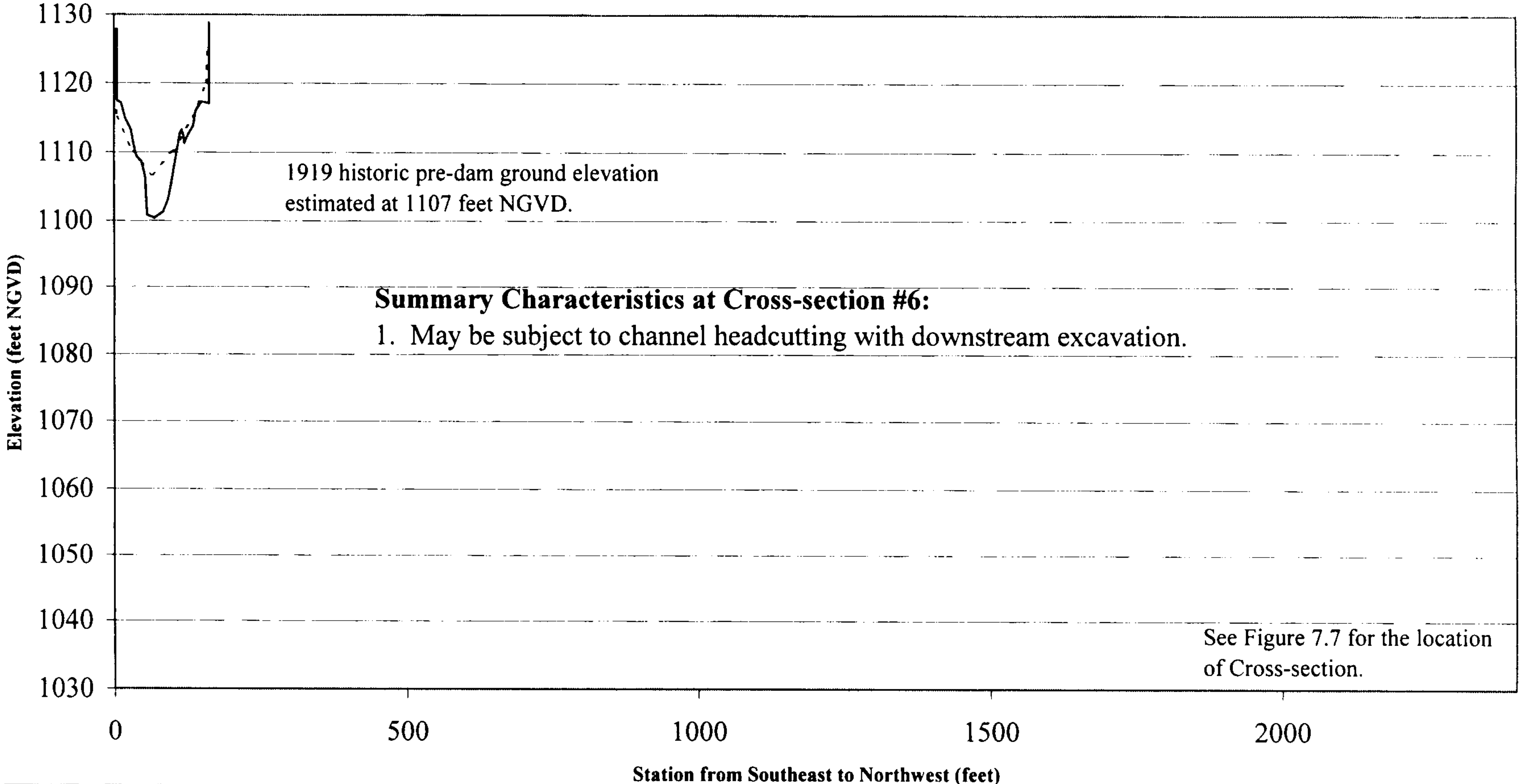
Source: LACDPW Topographic Data.
Notes: 1919 to 1934: ~14' of deposition (uncertain).
1934 to 1938: ~10' of channel deposition.
1938 to 1942: ~5' of deposition on west side.
1942 to 1981: ~30' of excavation or incision.
1981 to 1995: ~5' of deposition.

figure 7.5

Historical Cross-sections
Cross-Section #5

PWA

PWA #: 1310



—— Jan-99 ······ Jul-81	Source: LACDPW Topographic Data, PWA Survey. Notes: 1981 to 1999: ~7' of channel incision, could be caused by channel head-cutting from downstream excavation.	*figure 7.6* **Historic Cross-sections** **Cross-Section #6** **JPL Bridge** PWA PWA #: 1310

historic records of sediment excavation, dumping, and mechanical redistribution within the basin are incomplete so it is impossible to completely understand the causes of geomorphic change within the basin.

7.1.2 Longitudinal Profiles

PWA used the same historic topographic maps to plot longitudinal profiles of the basin over time. This information allowed for yet another evaluation of geomorphic change in the basin. A longitudinal transect was established through the basin, with one pivot point (Figure 7.7). This transect cuts across the historic topographic features of the basin and does not necessarily follow the Arroyo Seco thalweg. Points marking the intersection of elevation contours with the thalweg of the channel were projected back perpendicular to the longitudinal transect to obtain plot points. This ensured that elevations along the thalweg were referenced to the same distance along the longitudinal transect despite changes in the position of the thalweg between years. This plot differs from Figure 4.5 in that it does not follow the thalweg of Arroyo Seco.

Results from these transects are shown in Figure 7.8 with the locations of the reservoir cross-sections indicated by vertical lines. Like the reservoir cross-section measurements, the plot shows that, overall, the bed elevation increased between 1934 and 1942 with sedimentation. Between 1942 and 1981, major changes in the topography of the basin occurred, with an overall net decrease in elevation from about 5 feet near the dam to about 10 feet in the vicinity of cross-sections #4 and #5. The 1995 profile indicates a current low-slope zone in this area, likely created by mechanical excavation and resultant channel down-cutting upstream toward the JPL bridge. It is unknown to what extent the volume of material moved from cross-sections #4 and #5 owes to direct excavation versus transport by fluvial processes.

7.1.3 Sediment Delivery, Hahamongna Watershed Park

7.1.3.1 *Historic Long-term Average Annual Sediment Delivery*

Sediment management records from the LACDPW (Table 4.7) for Devil's Gate Reservoir were used to calculate long-term average annual sediment delivery to Hahamongna Watershed Park. Figure 7.9 shows historic cumulative sediment curves for sediment delivered, sluiced and excavated from the reservoir. As this figure shows, the long-term average annual sediment delivery to the basin since the construction of the dam in 1919 is approximately 90 acre-feet (145,200 cubic yards). This is the best estimate of the amount of sediment that would have to be removed annually from Hahamongna Watershed Park to maintain current storage capacity behind the dam.

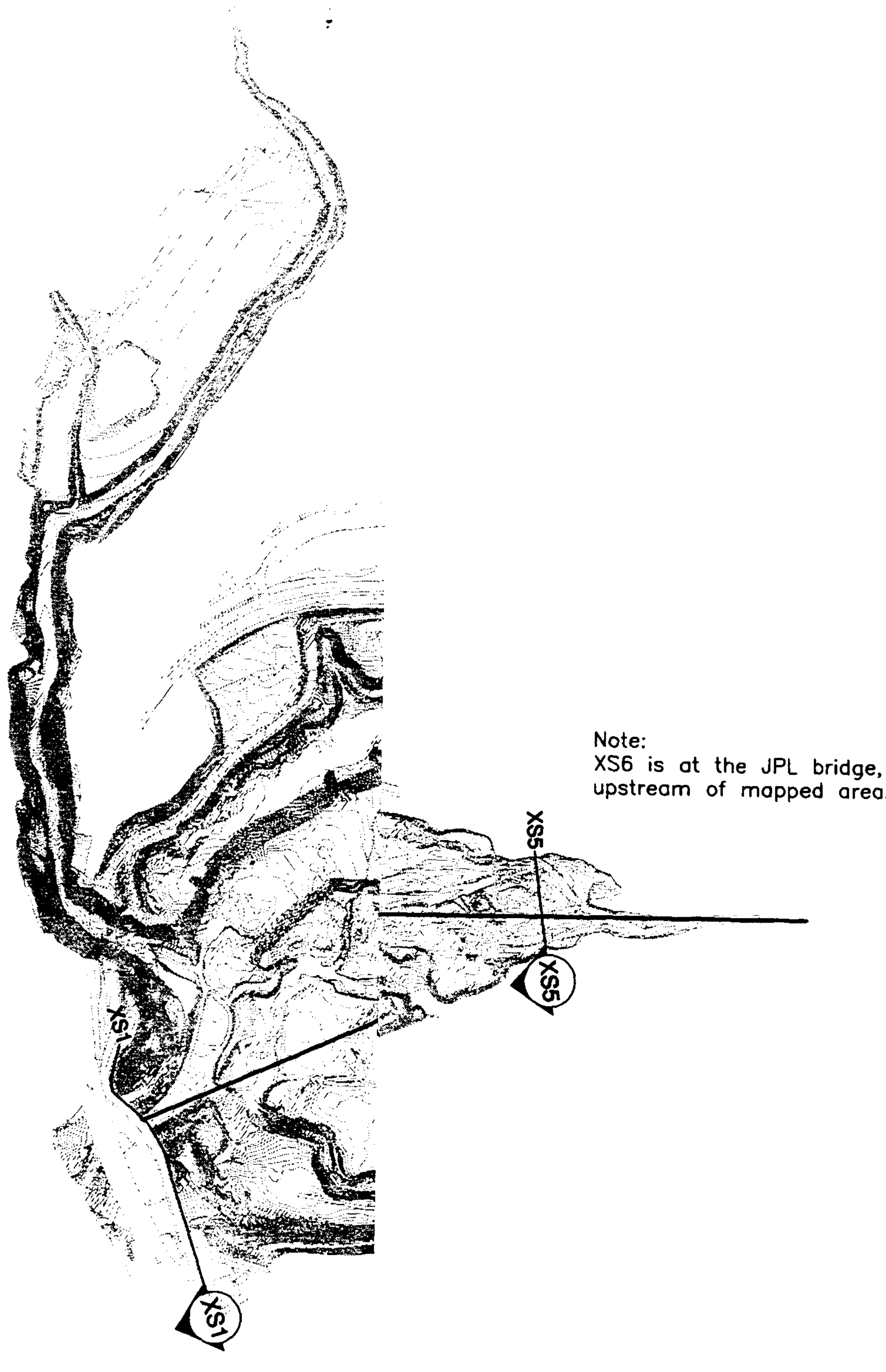
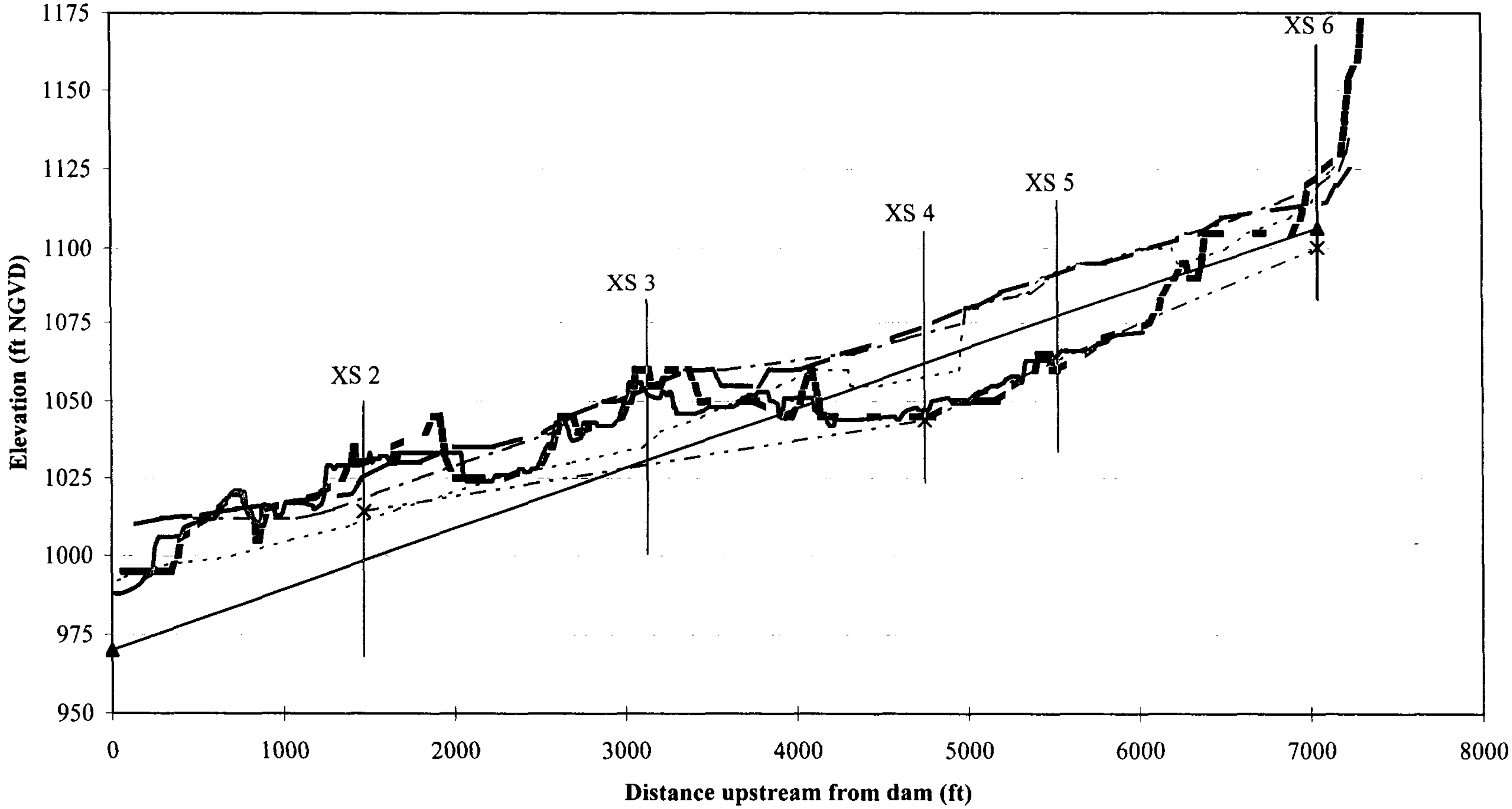


figure 7.7

Longitudinal Profile Transect Hahamongna Watershed Park

D:\projects and CAD\1310 Haha\Master Plan\Long.dwg

PWA



KEY: Date of Topographic Map

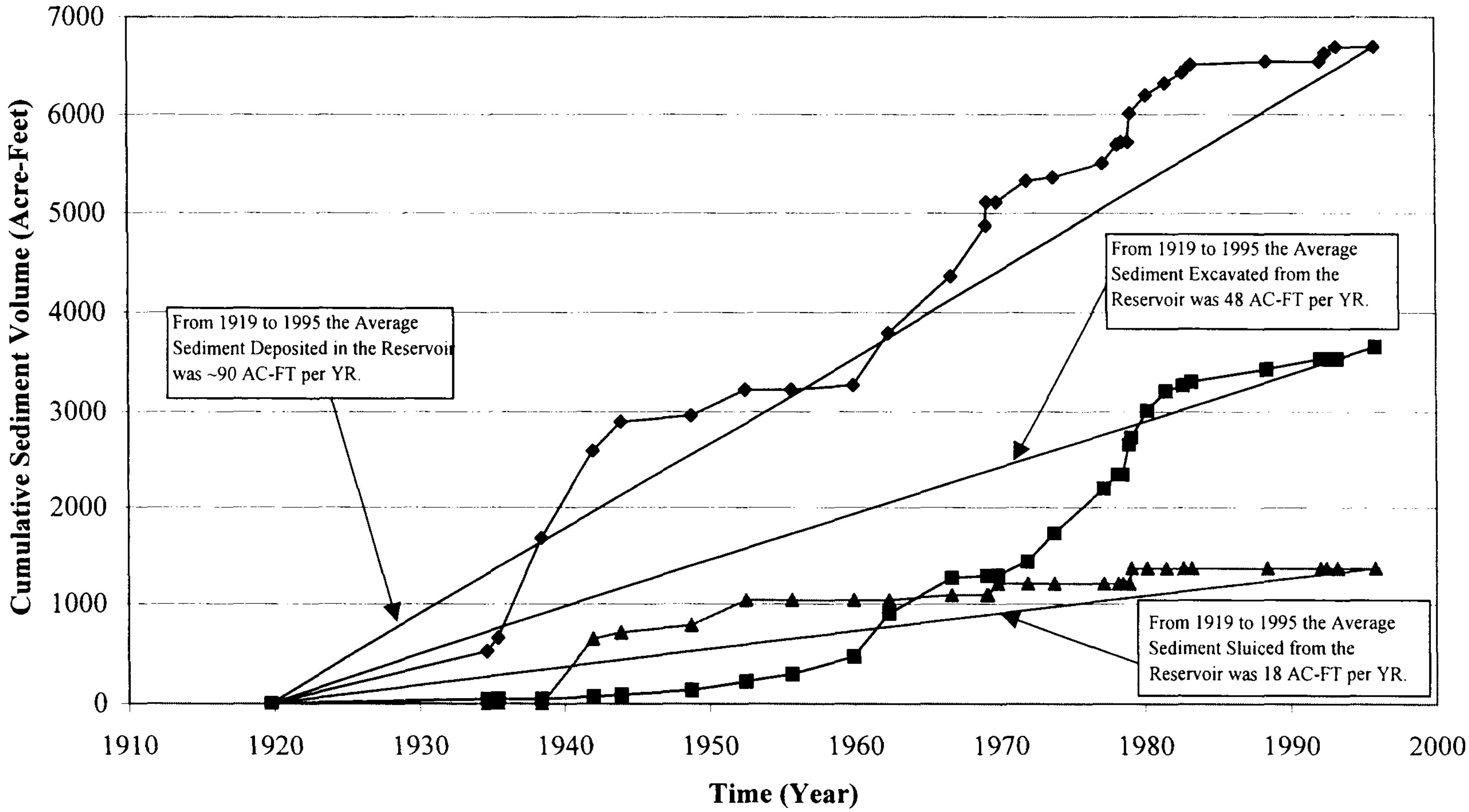
1919	8/34-11/34	June 1938	12/41-1/42
7/24/81	1995	January 1999	

NOTE: Each profile was taken along a linear transect established from the dam and extending upstream to the upper limit of the reservoir basin, rather than along the existing stream channel.
Vertical lines indicate intersections of this profile with selected cross sections (XS).

figure 7.8

**Historic Longitudinal Profiles
Hahamongna Watershed Park**

PWA 1310 Hahamongna Watershed Park



- Cumulative Sediment Deposited
- Average Cumulative Sediment Deposited
- Cumulative Sediment Sluiced
- Average Cumulative Sediment Sluiced
- Cumulative Sediment Excavated
- Average Cumulative Sediment Excavated

Source: LACPWD

Note: Data points indicate discrete survey dates. Curves are not derived from continuous data, therefore, by definition, the curves are smoothed.

figure 7.9

Historic Cumulative Sediment Curves: Deposited, Sluiced, and Excavated

PWA | PWA #: 1310

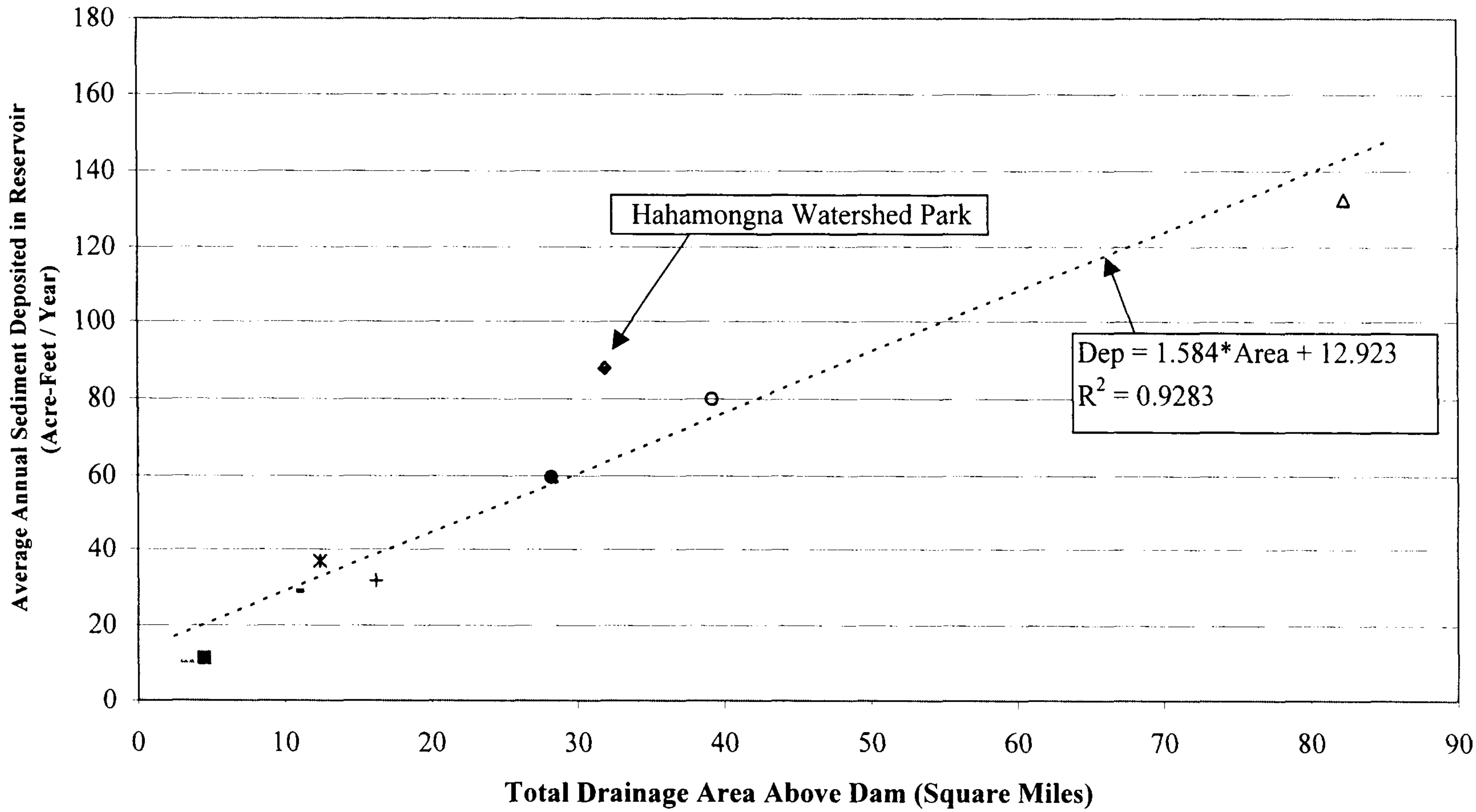
Sediment management records for several other reservoirs in the San Gabriel Mountains region were obtained from LACDPW for comparison with Hahamongna Watershed Park. Long-term average annual sediment deposition was calculated for each reservoir and then plotted against the total drainage area upstream of the dams. A regression relationship was calculated for these data and is shown in Figure 7.10. These data show that, for the size of the watershed area draining to it, the Hahamongna Watershed Park has received a slightly above average amount of sediment annually over the long-term, relative to other reservoirs in the region.

7.1.3.2 *Storm Event Sediment Delivery*

To estimate the volume of material reaching Devil's Gate Reservoir for a variety of storm events, PWA conducted an extensive literature search of available data and publications addressing sediment yield and sediment delivery for the region. Estimates of sediment delivery for the Arroyo Seco basin were important to establish for two main reasons: 1) to compare to sediment delivery quantities calculated for various storm events modeled with the sediment transport model, and 2) to give a general sense of the amount of sediment that could be expected to be delivered to the park for particular un-modeled storm events.

There are many studies that estimate storm event sediment delivery from the San Gabriel mountain region and other nearby ranges. These studies generally emphasize one of two approaches. First, sediment delivery from a particular catchment may be measured for a period of time sufficient to identify the catchment's characteristic sediment delivery behavior. The basis of the majority of these types of studies is records of sediment deposition in and extraction from a reservoir or debris basin. This is the most detailed, accurate, and reliable kind of sediment delivery data, but it is difficult to collect and may require collection over a period of 20 years or more in one particular basin. Therefore, this data is generally limited. Further complicating the evaluation of storm event-based sediment delivery is the fact that in southern California natural conditions (including forest fires) produce extreme temporal and spatial variations in the occurrence of inland sedimentation events. This factor implies that an adequate statistical description of event-based sediment delivery requires a larger-than-average body of field data.

The second approach to evaluating event-based sediment delivery involves the development of a predictive model based on sediment delivery data from multiple basins to estimate an individual catchment sediment delivery. This model would be based on specific physical characteristics of the catchment, including climatic inputs. Catchment sediment delivery is then estimated by measuring the required input variables and applying the model. However, the validity of erosion rates from one watershed for the prediction of sediment delivery in another watershed is difficult to assess. Individual basins can have dramatic differences in geology, vegetation, topography, and fire history that affect the net sediment delivery. Estimates of sediment delivery using this technique are, in general, less accurate than with the first technique because they are



◆ Devil's Gate Reservoir
■ Big Dalton Reservoir
△ Big Tujunga Reservoir
○ Cogswell Reservoir
✶ Eaton Wash Reservoir
● Pacoima Reservoir
\+ San Dimas Reservoir
\- Santa Anita Reservoir
~ Sawpit Reservoir
\- - - - - - Best Linear Fit to Data

Source:
LACDPW Reservoir Data Summaries
Notes:
1) Periods of record between 56 and 76 years, average of 65 years.
2) This is NOT a plot of sediment YIELD (due to trap efficiencies less than 100%).

figure ***7.10***

Long Term Average Reservoir Sediment Deposition by Total Drainage Area for Nine San Gabriel Mtns. Reservoirs

PWA | PWA #: 1310

simplified. The number of physical processes explicitly accounted for in the model is often fewer than the large number of physical processes that are actually important to sediment delivery.

PWA is unaware of any detailed event-based sediment delivery studies specifically addressing the Arroyo Seco watershed. As such, PWA used a combination of the two methods described above to estimate event-based sediment delivery for Arroyo Seco. A logarithmic regression model of sediment delivery versus storm event rainfall volume was formulated using data from both Arroyo Seco and other catchments in the San Gabriel region. PWA obtained sediment management records from the LACDPW indicating the measured volumes of sediment deposited within the Devil's Gate Reservoir (Table 4.7). The time intervals between sediment delivery data recordings and the number of significant flood events between data recordings were noted for the County's records. Sediment delivery volumes that followed a relatively short data recording interval or that could be attributed to one, two or at most three significant storm events were plotted against total rainfall volume for the pertinent storm events. Total storm rainfall depth was calculated by summing consecutive non-zero daily rainfall depths for that storm. Storm rainfall depths were calculated as averages from the three Arroyo Seco basin NWS gages in Table 4.2. The final storm rainfall depth was multiplied by the upstream drainage area to compute a volume of water accumulated throughout the basin during the storm. This volume of storm rainfall was assumed to be responsible for all sediment deposited during the time period. Although actual sediment deposition may have been protracted prior to and following the peak flow event, the two were assumed to be directly correlated. This permitted a worst-case estimate of the maximum sediment volume produced by such a storm event. This portion of the analysis yielded eight data points for volume of sediment delivered per volume of precipitation. Because of the relatively large size of the Arroyo Seco basin and the relatively large storm events that took place over the period of the LACDPW records, the estimated volumes of precipitation for these eight data points are relatively high and range from 6,000 to 26,000 acre-feet.

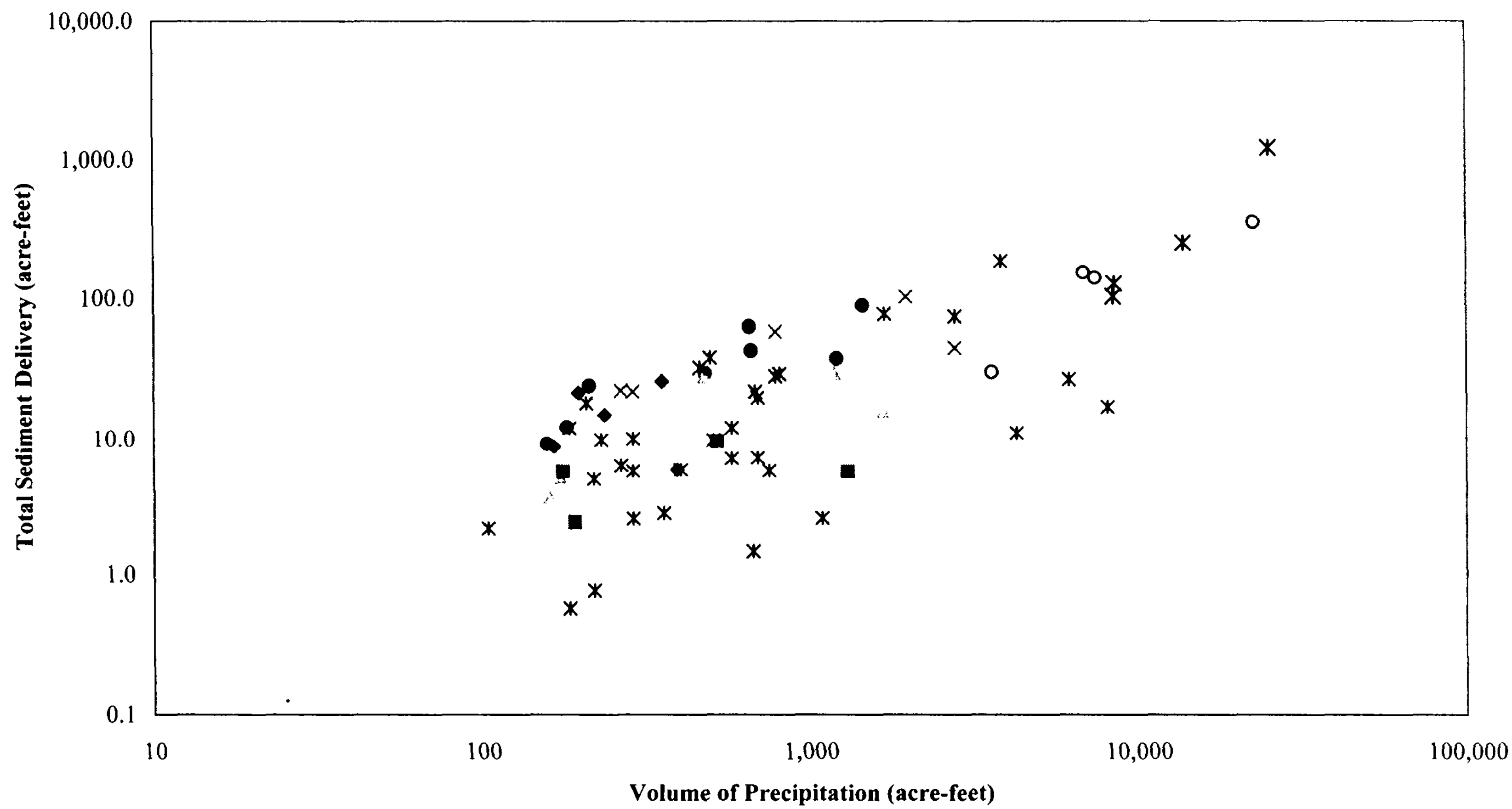
To augment this event-based data set with estimates of sediment delivery for smaller volumes of precipitation, measured sediment delivery data for smaller nearby debris basins were used to approximate likely sediment delivery for smaller storm events in the Arroyo Seco basin. Storm rainfall associated with each deposition event was calculated using the same NWS gages and methodology as described above for the Arroyo Seco data set. Sediment volume data originated primarily from LACDPW debris basin maintenance records and therefore described the amount of sediment left in a debris basin following a storm event, not the actual sediment volume flowing into the debris basin at its upstream end over the course of a storm event. This quantity may be described as total sediment delivery, and is the quantity we were most interested in for modeling purposes. Total sediment delivery would include sediment transported over the basin spillway during the event. To calculate total sediment delivery a basin trapping efficiency of 0.85 was assumed for all data, including Devil's Gate Reservoir data. This trapping efficiency has been cited as a reasonable estimate for debris basins in the San Gabriel Mountains region (Scott and Williams, 1974).

Figure 7.11 shows resulting total sediment delivery data for both Arroyo Seco and other San Gabriel Mountains watersheds.

Previously published regression equations available for the estimation of San Gabriel sediment delivery were not used since results from this generalized data was less accurate than the event-based debris basin data themselves. However, one such regression developed specifically for the Central San Gabriels is shown in Figure 7.11 for comparison (Ferrell, 1979).

Ultimately, Figure 7.12 shows that when sediment delivery data from Arroyo Seco, other debris basins, and the Central San Gabriel regression are plotted together, the overall increase in sediment delivery with increasing precipitation (slope of field of data points) is similar between data sets. However, the range of sediment delivered for a particular volume of precipitation is highly variable, with sediment volumes ranging over two orders of magnitude. Thus, the prediction of sediment delivery, especially from the San Gabriel Mountains, is highly uncertain.

Using this predictive regression model, sediment delivery volumes were estimated for the three storm events modeled in the hydraulic and sediment transport model—the 2-year, 10-year, and Capital storm events—as well as the USGS 50-year flood event. Methods from the LACDPW hydrology manual (LACDPW, 1989) were used to calculate storm rainfall volume since the flood hydrograph distribution used in the hydraulic and sediment transport models was obtained from LACDPW and was derived using these methods. These methods assume a four-day storm with the 24-hour maximum rainfall occurring on the fourth day. The first, second, and third days of this storm receive 10%, 40%, and 35% of this fourth day amount, respectively. We assumed that a rain storm of a certain return interval produces a flood event with the same return interval. In other words, a 2-year rainfall event was assumed to produce a 2-year flood event, a 10-year rainfall event would produce a 10-year flood event, and a 50-year rainfall event would produce a 50-year flood event. While rainfall and runoff frequencies are not necessarily correlated, due to differing statistical methods and variable antecedent watershed conditions—as demonstrated by the difference between the 50-year "Capital" design rainfall event used by LACDPW (20,026 cfs) which assumes saturated antecedent watershed conditions and is statistically based on rainfall data, and the USGS 50-year flood (12,364 cfs) which likely occurred under less-than-saturated watershed conditions and is statistically based on runoff data —the assumption of correlation is reasonable in this case in light of the significant uncertainty associated with the regression equation. Table 7.1 shows estimated storm event sediment delivery to Hahamongna Watershed Park for the four storm events, for both the best-fit regression relationship and the upper bound regression. Since, on average over the long-term, the Arroyo Seco watershed delivers more sediment for its size than other watersheds in the San Gabriel Mountains region (as shown in Figure 7.10) and since LACDPW data for the 1938 flood (peak flow = 12,100 cfs; rainfall volume = 25,200 acre-feet; sediment delivery = 1211 acre-feet (1029/0.85)) falls closer to the upper bound regression than the best fit regression, the upper bound regression may be preferable for describing storm event sediment delivery to the Hahamongna Watershed Park.



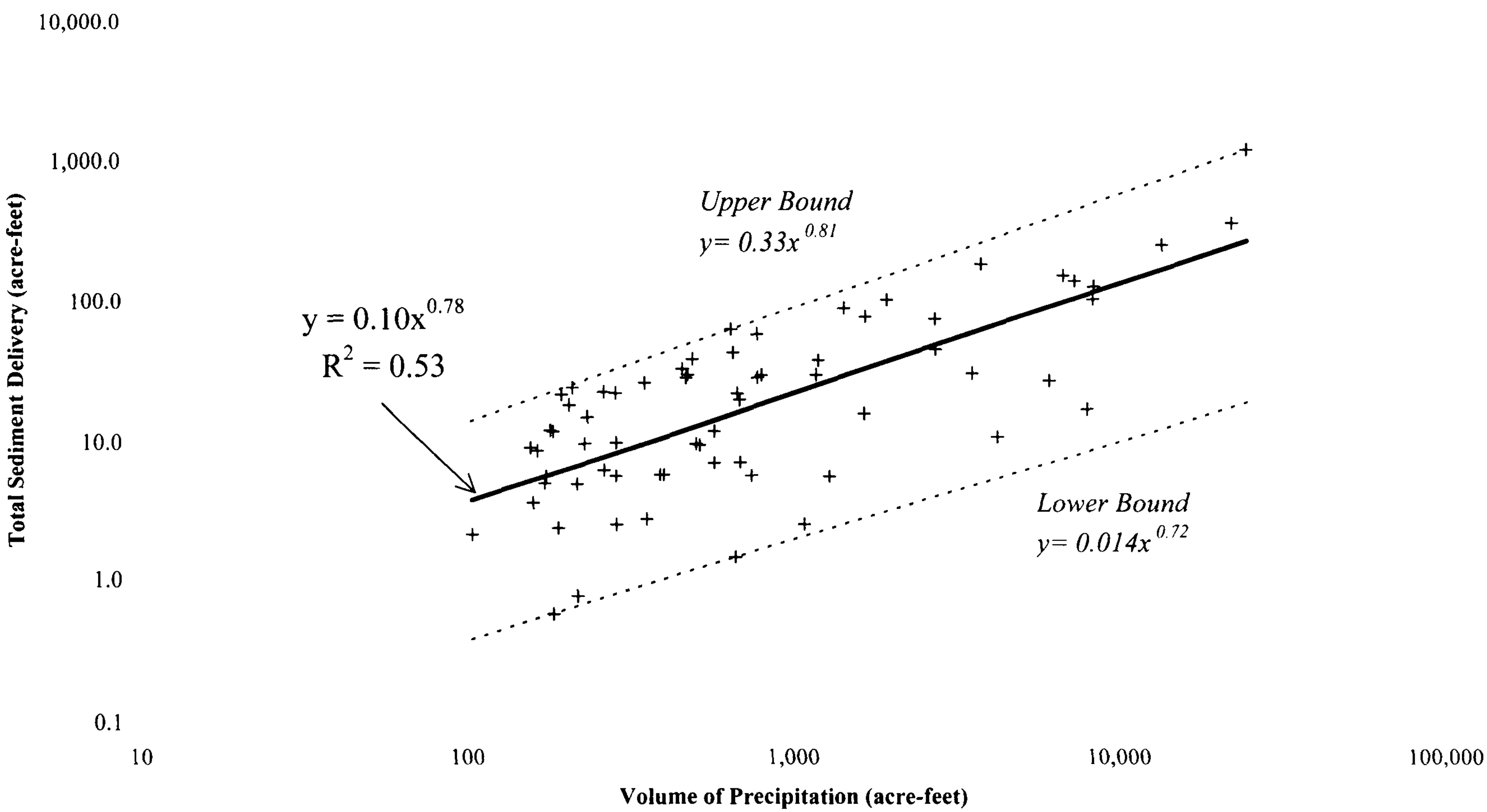
- ● Tatum (1963): 1938 Flood, 5-years Post-Fire
- ◆ Tatum (1963): 3 Years Post-Fire
- × Scott & Williams (1978): 1938 Flood
- Scott & Williams (1978): 1943 Flood
- ■ Scott & Williams (1978): 1952 Flood
- ✱ Scott & Williams (1978): 1969 Flood
- Ferrell (1979): Table G-4 Central San Gabriels
- ○ Arroyo Seco: Single Peak Q
- ✱ Arroyo Seco: 2-3 Event Peak Q

NOTE: Volume of sediment delivery for Arroyo Seco is based on excavation tables provided by LACDPW and an assumed trap coefficient of 0.85 (Scott & Williams, 1974). Other volumes of sediment delivery shown were found in published reports using LACDPW records, and a trap coefficient of 0.85 was also applied to these. Volume of precipitation was calculated by multiplying the measured storm rainfall depth by the upstream drainage area.

figure 7.11

Sediment Delivery by Storm Event, San Gabriel Mtns. Drainage Basins

PWA 1310



+ Sediment Delivery for Arroyo Seco and Nearby Basins (0.85 Trap Efficiency Coefficient)

Regression of Sediment Delivery for Arroyo Seco & Nearby Basins

NOTE: For data from Arroyo Seco and data from published reports, a 0.85 trapping efficiency coefficient was applied.

figure 7.12

Regressions of Sediment Delivery by Storm Event, San Gabriel Mtns. Drainage Basins

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TABLE 7.1 Estimated Storm Event Sediment Delivery, Hahamongna Watershed Park

Storm	Storm Peak Flow (cfs)	Rainfall Volume (acre-feet)	Best-fit Sediment Delivery (acre-feet)	Upper Bound Sediment Delivery (acre-feet)
2-year	912	6,000	140	**620**
10-year	4,920	10,800	230	**1010**
50-year	12,364 (USGS)	14,500	280	**1270**
Capital	20,026	26,000*	450	**2050**

*Based on 15" of rain on the fourth day of the storm (LACDPW, 1989).

Source: PWA analysis.

Note: Estimates of sediment delivery are highly uncertain and may vary from actual delivery by over one order of magnitude.

7.2 SEDIMENT TRANSPORT ANALYSIS

MIKE 11, the model used to simulate hydraulic conditions in the Hahamongna Watershed Park during flood events, is also capable of simulating fluvial sediment transport. The MIKE 11 sediment transport model was used to evaluate zones of sedimentation and erosion within the park. This understanding of the spacial distribution of sediment in the park during flood events was important to developing a sediment maintenance strategy.

7.2.1 MIKE 11 Non-cohesive Sediment Transport Model

MIKE 11 is capable of simulating sediment transport processes on river systems such as the Arroyo Seco. Generally the sediments moving in Arroyo Seco are non-cohesive—that is, sediment particles don't have a tendency to adhere to one another—so the non-cohesive sediment transport (NST) module was used in the model. Several types of sediment data are important in formulating a sediment transport model with MIKE 11. Some sort of upstream condition must be assumed regarding the amount of sediment flowing into the system during a flood event. Data describing the size of sediment particles moving in the system is also important. The other crucial choice to make in formulating a model is which sediment transport function should be used to describe sediment movement. There are many mathematical formulas describing the transport of sediment. Each is best suited to a particular set of physical conditions, including channel slope and sediment particle size. Therefore, it is very important to use a sediment transport function in the model that is appropriate to the physical conditions of the system. In addition to these data the information used in a standard hydraulic model (Section 6.1.2) are also required to complete the sediment transport model. Results available from a MIKE 11 sediment transport model include changes in bed-elevation throughout the river system over the course of a flood event, sediment transport rates (in m^3/s or tons/day), and sediment particle-size information.

7.2.2 Sediment Transport Model Formulation

7.2.2.1 *Inflowing Sediment Load*

The best data for characterizing the amount and type of inflowing sediment load in a sediment transport model is actual measurements of sediment transport rates taken during flood events for the particular river system being studied. If unavailable, sediment transport measurements for nearby rivers with similar physical characteristics are also very useful. However, sediment transport data are not available for the Arroyo Seco, nor are they available for any nearby analogous river systems. The reason for this lack is perhaps the violent and unpredictable nature of flood events from the San Gabriel Mountains, which could make measurements difficult during the peak of flood events. The infrequency and often short duration of flood events in this region may also make measurement difficult. PWA recommends that a sediment transport measurement program be established to collect suspended and bedload sediment transport data on the Arroyo Seco at Devil's Gate Dam—to better evaluate flow-assisted sediment transport quantities—and in the upstream areas, either at the JPL bridge or at the USGS gage in the upper watershed—to better evaluate inflowing sediment quantities. Measurements at the USGS gage or the JPL bridge should only be taken when safety is not jeopardized by the magnitude of the flood event. Data collection will likely only be possible in the upstream areas during moderate-sized events. Sediment transport monitoring data would greatly aid future sediment management in the park.

Since sediment transport measurement data was unavailable, an alternative method was used to assess the inflowing sediment load to the Hahamongna Watershed Park during the modeled flood events. Cross-sections in the model were extended upstream of the park boundary at the JPL bridge, creating what is known as a sediment production reach. Rather than directly inputting an expected upstream sediment load for each flood event, flows through this production reach were allowed to scour up sediment from the river bed such that by the time flows reached the upstream boundary of the study area (at the JPL bridge) they carried an appropriate quantity of sediment. The quantity of sediment transported at the JPL bridge over the course of the flood event was checked against the quantity of sediment predicted for the flood event using the event-based sediment delivery regression described in Section 7.1.3.2. A comparison of these inflowing load quantities is shown in Table 7.2. The production reach sediment loads shown in the table are averages of both existing conditions and proposed conditions model runs for the each event size. During the calibration model run the quantity of sediment transported at the JPL bridge was also checked against the LACDPW's reported quantity of sediment delivered to the basin (Section 7.2.3).

TABLE 7.2 Sediment Quantities Generated in Sediment Production Reach Compared to Sediment Quantities Predicted by Sediment Delivery Regression

Storm Event Return Period (Years)	Average Production Reach Sediment Load (acre-feet)	Upper Bound Regression Sediment Load (acre-feet)
2	129	620
10	534	1010
50 (Capital)	1793	2050

Source: PWA analysis.

Given the uncertainties associated with both the regression and the sediment transport modeling these numbers compare quite well. The modeled sediment loads are well within the range of uncertainty inherent in the empirical data, and fall between the upper and lower bound regression lines.

7.2.2.2 *Sediment Transport Function*

Several mathematical functions are available in MIKE 11 to describe sediment transport. These include functions by Ackers and White (1973), Engelund and Fredsoe (1976), Engelund and Hansen (1967), van Rijn (1984a, 1984b), and Smart and Jaeggi (1983). The Smart and Jaeggi function was selected for use in the sediment transport model for several reasons. The first reason is that it was specifically designed to model transport of coarse sediments or sediments with large particle sizes. A large proportion of the sediments transported within the Hahamongna Watershed Park are very coarse. The second reason is that the Smart and Jaeggi function was designed for steep slopes in the range of 1% to 3%. The average slope within the Hahamongna Watershed Park is approximately 2%. A formulation of the Smart and Jaeggi function is shown in Appendix A.

7.2.2.3 *Particle-size Distributions*

In the San Gabriel Mountains, intense stress fracturing continues to reduce granitic and metamorphic bedrock to a broad gradation of fragment sizes. To obtain data for the MIKE 11 sediment transport model, the grain-size distribution of the bed materials within the Arroyo Seco Reservoir were studied in a field reconnaissance of the basin in January 1999. The reconnaissance included a walk through the Hahamongna Watershed Park along the active channel. No observations were made in the Arroyo Seco basin upstream from the park.

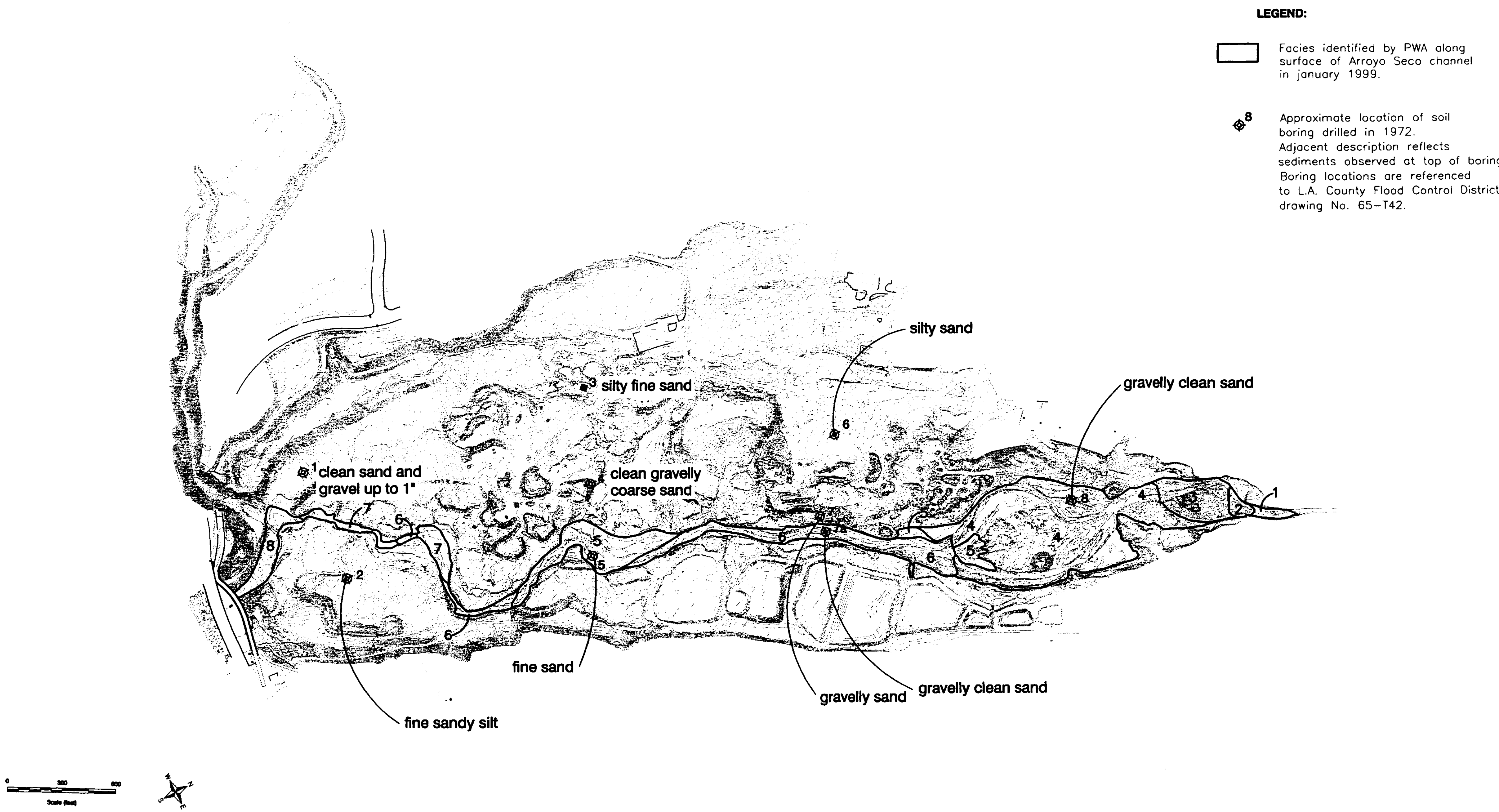
Discrete units of channel bed sediments were mapped on the available 1995 topographic map, based on the sizes and relative volumes of bed material included. Nine distinctive sedimentological zones, here referred to as "facies," were noted along the active channel zone in the watershed park from the uppermost area of the park to the dam itself (Table 7.3). Figure 7.13 illustrates the extent of the nine facies within the current channel.

For those facies dominated by gravel and coarser material, the grain-size distribution was sampled using a "pebble count", a standard surface particle methodology for coarse bed materials along a bed surface (Wolman, 1954). Each pebble count consisted of the by-hand measurement of the intermediate diameter of one hundred particles sampled across the surface of the active channel. The lowermost limit of measurement using this method is 2 millimeters. Particles with intermediate diameters less than 2 millimeters were categorized together as "less than 2 millimeters." For those facies dominated by sand, the Wolman pebble count methodology was not appropriate. Instead, a volume of sediment was bagged and submitted for sieve analysis by a geotechnical laboratory. Sample weights ranged from 305 grams to 1087 grams, depending on the maximum grain size in the sample. There is a small sampling bias between the two applied methodologies. Pebble counts are slightly biased to measure coarser particles, since they are sampled over an area rather than as a volume. To estimate the degree of bias, one facies (Facies #7) was sampled using both the pebble count and sieving methodology.

TABLE 7.3 Description of Sediment Facies Identified During Field Reconnaissance

Facies Number	Description
1	Grayish brown poorly sorted gravel and cobbles with sand and boulders
2	Yellowish brown poorly sorted sand and gravel
3	Grayish brown poorly sorted gravel and cobbles with sand and boulders
4	Grayish brown poorly sorted sand, gravel, and cobbles
5	Tan poorly sorted sand with gravel
6	Yellowish brown poorly sorted gravel and sand
7	Yellow brown poorly sorted sand with gravel
8	Grayish brown silty sand

The measured grain-size distributions are shown in Figure 7.14. The curves for Facies 7 are very similar, except for a small bias toward coarser particles in the upper (>d_{75}, where d_{75} is the diameter for which 75% of the grain-size distribution is finer) portion of the grain-size distribution curve. This bias was deemed insignificant for the purposes of this study.



Map units are superimposed on 1995 topography.

figure 7.13

Map of Existing Bed Material Facies and Past Soil Borings

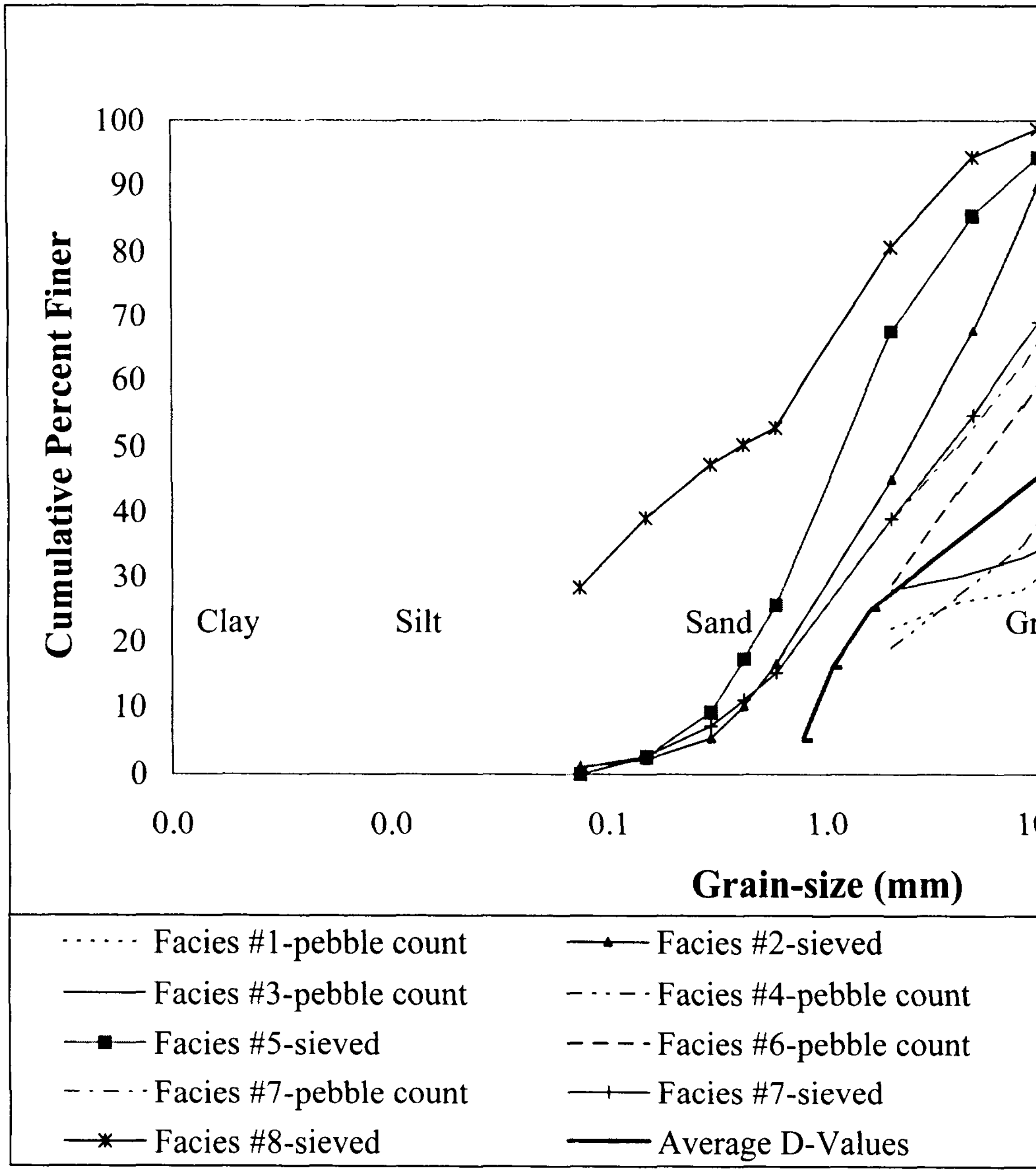


figure 7.14

Grain-size Distributions of Facies, Hahamongna Watershed Park

PWA #: 1310

1310PCs.xls chart 8/20/99

A closer look at Figure 7.13 and Figure 7.14 reveals a gradual progression along the channel from gravel and cobble dominated facies upstream to sand dominated material downstream near the dam. Surface sediments from well borings also illustrate this fining trend (Figure 7.13). This indicates strong selective transport occurring within the reservoir: as channel slope decreases, coarser grains are deposited and progressively finer sediments are transported downstream.

The spatial pattern of grain size is also closely coupled with the planform channel pattern within the reservoir. Channel braiding is most pronounced in Facies 4 material, where large gravels and cobbles form complex combinations of mid-channel bars. In Facies 5 through 7, gravel is dramatically finer and sand comprises a significantly larger proportion of the grain-size distribution. This shift in sediment deposition corresponds with the transition channel reach in which channel meandering is dominant.

The Smart and Jaeggi sediment transport function requires the input of a representative sediment grain-size diameter value and the ratio of D_{90} to D_{30}. A range of both parameters were used in the calibration process (Section 7.2.3.1). However the Smart and Jaeggi sediment transport function requires the ratio that the ration of D_{90} to D_{30} be between 2.0 and 8.5. Using average diameter values for all facies in the park, the ratio of D_{90} to D_{30} would be approximately 50. The average of the ratios for each facies is approximately 34. Clearly the range of values required by the model is lower than these average values and represents a smaller range of particle-sizes than is actually present in the basin. However, several of the sieved facies did yield ratios that are at least in the range of 8.5 (Facies #5 = 10.3, Facies #2 = 9.5, and Facies #7 = 13.8). Overall, the larger ratios calculated from actual basin data indicate that the sediment transport model may be close to its limit of applicability under the extraordinary physical conditions encountered at Hahamongna.

7.2.3 Sediment Transport Model Calibration

The sediment transport model was calibrated using two types of data: topographic information and sediment delivery volumes. The following sections describe these two forms of calibration and the calibration results.

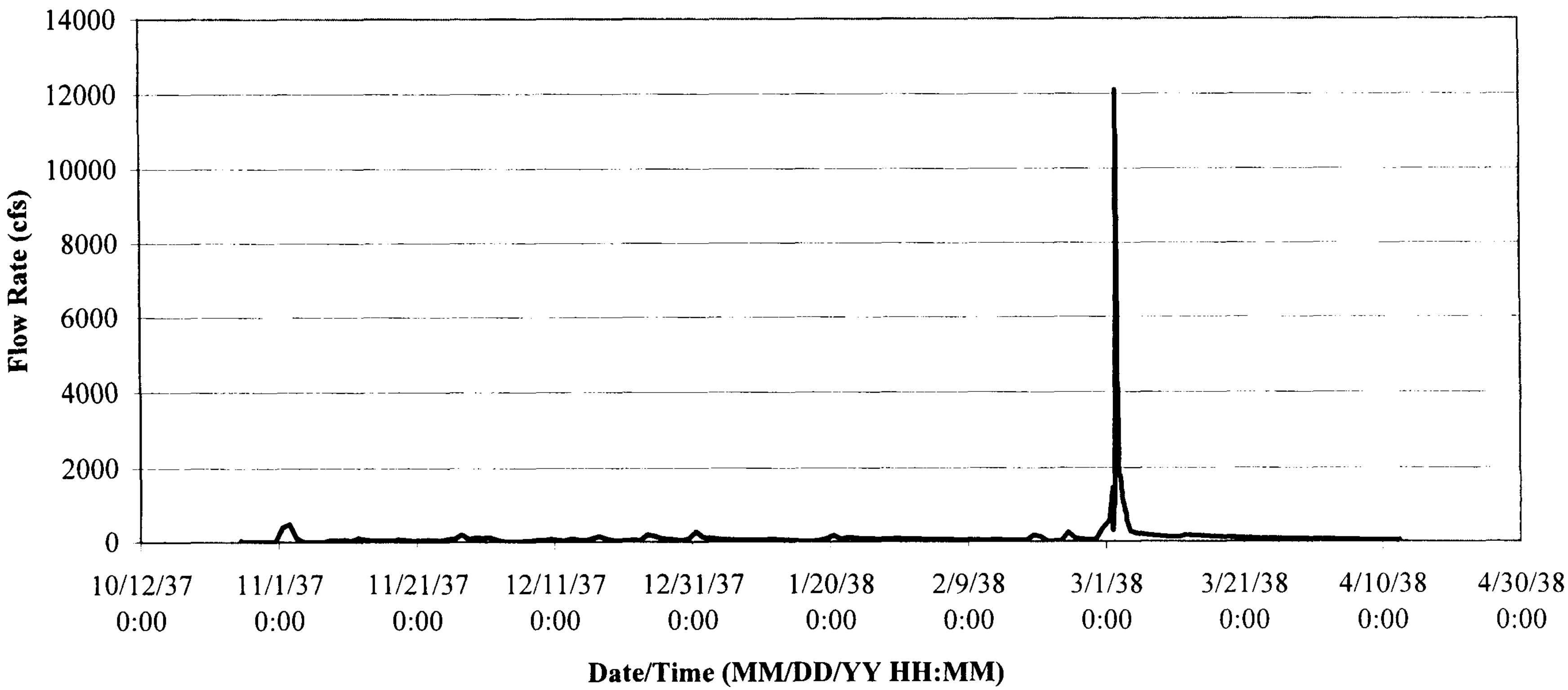
7.2.3.1 *Historic Cross-sections & Topography*

The primary data used to calibrate the model was contained in historic topographic maps of the park area for the years 1934 and 1938. A very large flood occurred in 1938 (12,100 cfs peak flow rate) which transported a lot of sediment into the basin and radically altered the geomorphology of the basin. PWA attempted to replicate these changes in geomorphology using historic flow data in the sediment transport model.

Cross-sections from the 1934 topographic map were assembled and input to the model. Due to the apparent importance of both moderate- and large-scale events in shaping the geomorphology of the basin, a reconstruction of the entire flow record between 1934 and 1938 was used as the upstream hydrologic boundary condition. Using this boundary condition allowed the model to simulate the geomorphic effect of all significant hydrologic events within the four-year calibration time-period. For the months surrounding the large 1938 flood event the LACDPW provided hourly inflow and outflow data for the basin. However, outside of these months the only data PWA had access to were average daily flows from the USGS stream-gage records. These two types of data were combined to produce the calibration flow record. To reduce model running time the four year flow record was shortened by deleting all flows below 35 cfs (1 m^3/s). Flows below this threshold were assumed to move very little sediment and to have virtually no impact on basin geomorphology. The filtered flow record used in the model calibration run was approximately six months long (Figure 7.15). It should be noted that the USGS average daily flows used in the majority of the flow record are likely lower than actual instantaneous flow-rates that occurred in this time period. Therefore, sediment transport rates and the amount of sediment delivery to the basin are likely somewhat low. However, given the lack of data, the use of daily average flow data is justified. Furthermore, since the majority of sediment was moved by the large 1938 event (for which hourly flow rates were used), the USGS daily averages should not have affected calibration results too significantly.

Since a Devil's Gate Dam operation rule curve was unavailable for 1934 to 1938 conditions, for a model downstream boundary condition operations were estimated based on the outflow structures that existed in the dam prior to the 1997 dam renovation. The approximate geometry of these structures was obtained from LACDPW information (LACDPW, 1996) and was input to MIKE 11. The model then internally calculated an appropriate flow-elevation relationship from this geometry. The flow-elevation relationship was smoothed somewhat for stable model computation. Although this downstream boundary condition is less accurate than an actual operation curve for the dam (as used in existing and proposed conditions modeling), it is a reasonable alternative given the lack of data. Furthermore, water-surface elevations at the dam were calibrated for the large 1938 event using actual LACDPW data.

With these input data the calibration model was run several times. Different values of the representative sediment diameter size and the ratio of D_{90} to D_{30} were used in multiple calibration simulations to produce topography within the park resembling that of the LACDPW 1938 survey. A range of representative sediment diameter sizes from 1.0 to 10.0 millimeters and a range of D_{90}:D_{30} from 2.0 to 8.5 were used in calibration simulations. Results of the most successful model calibration simulation are shown in Figures 7.16 through 7.20. The representative sediment diameter size used in this simulation was 5.0 mm, approximately D40 on the average grain-size curve for the basin. The ratio of D_{90} to D_{30} used in this simulation was 8.5, the upper end of the range permissible for the Smart and Jaeggi sediment transport function.



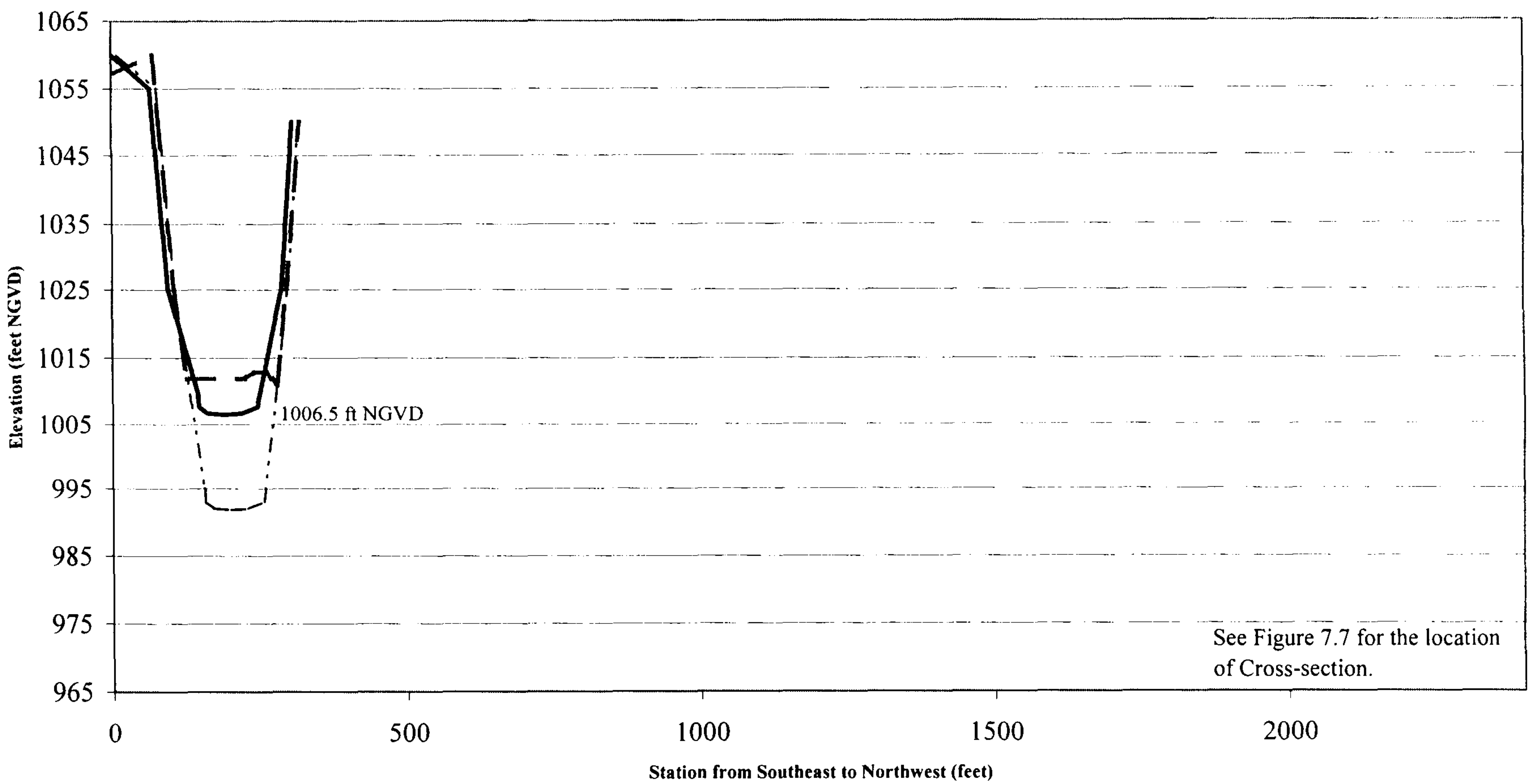
Source: USGS and LACDPW records.
Note: Data represents flows from 1934 to 1938. However, flows below 35 cfs (assumed to be insignificant to sediment transport) were filtered out to reduce simulation time. Therefore, plotted record shows a shorter 6-month series.

figure *7.15*

Filtered Inflow Record, 1934 to 1938
Sediment Transport Model Calibration

PWA

PWA #: 1310



- - · · - 8/34 - 11/34

— —Jun-38

——1938 Model Calibration

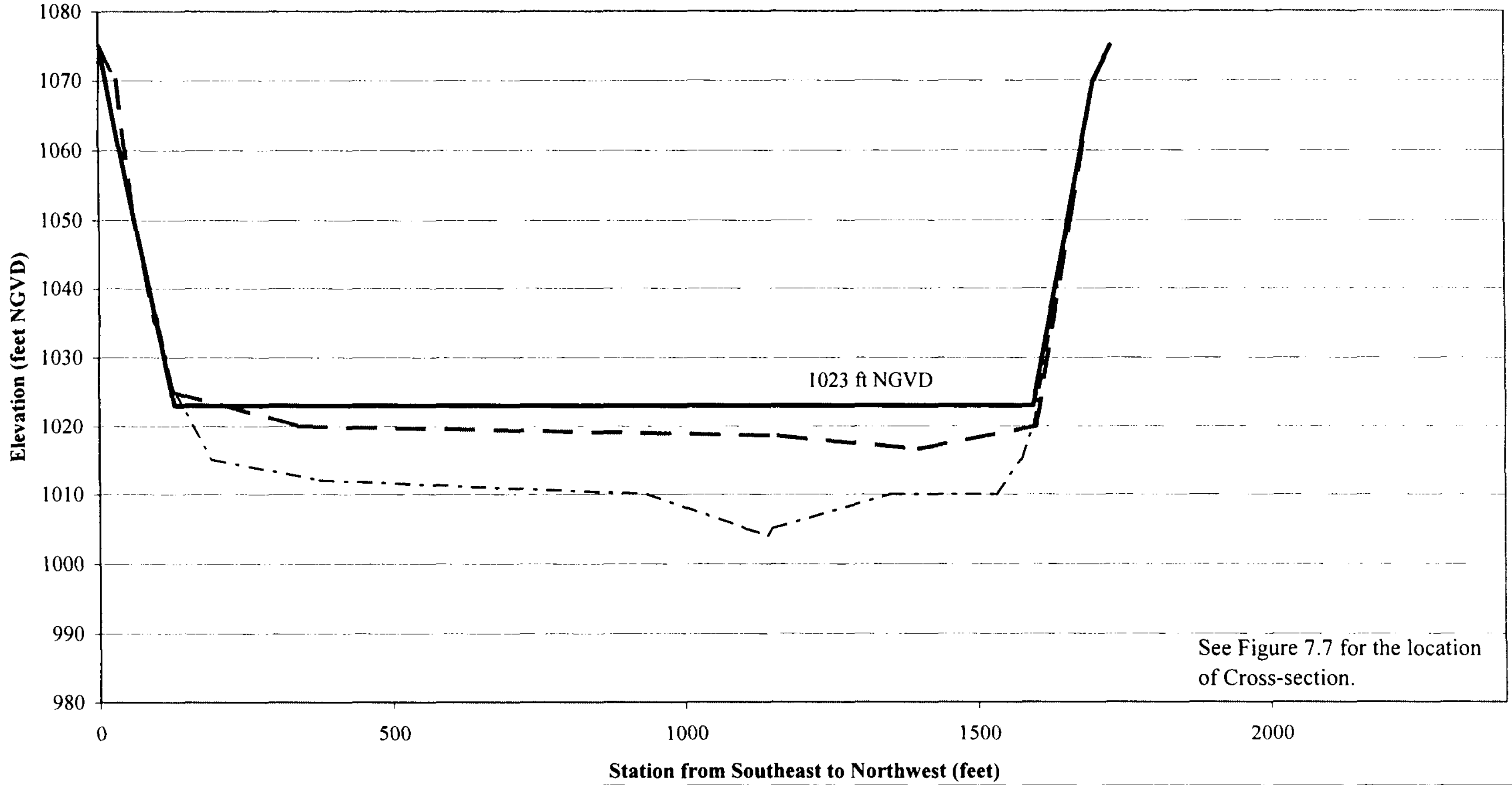
Source: PWA analysis, LACDPW historic data.

figure 7.16

Sediment Transport Model Calibration
Cross-Section #1
Devil's Gate Dam Face

PWA

PWA #: 1310



8/34 - 11/34

Jun-38

1938 Model Calibration

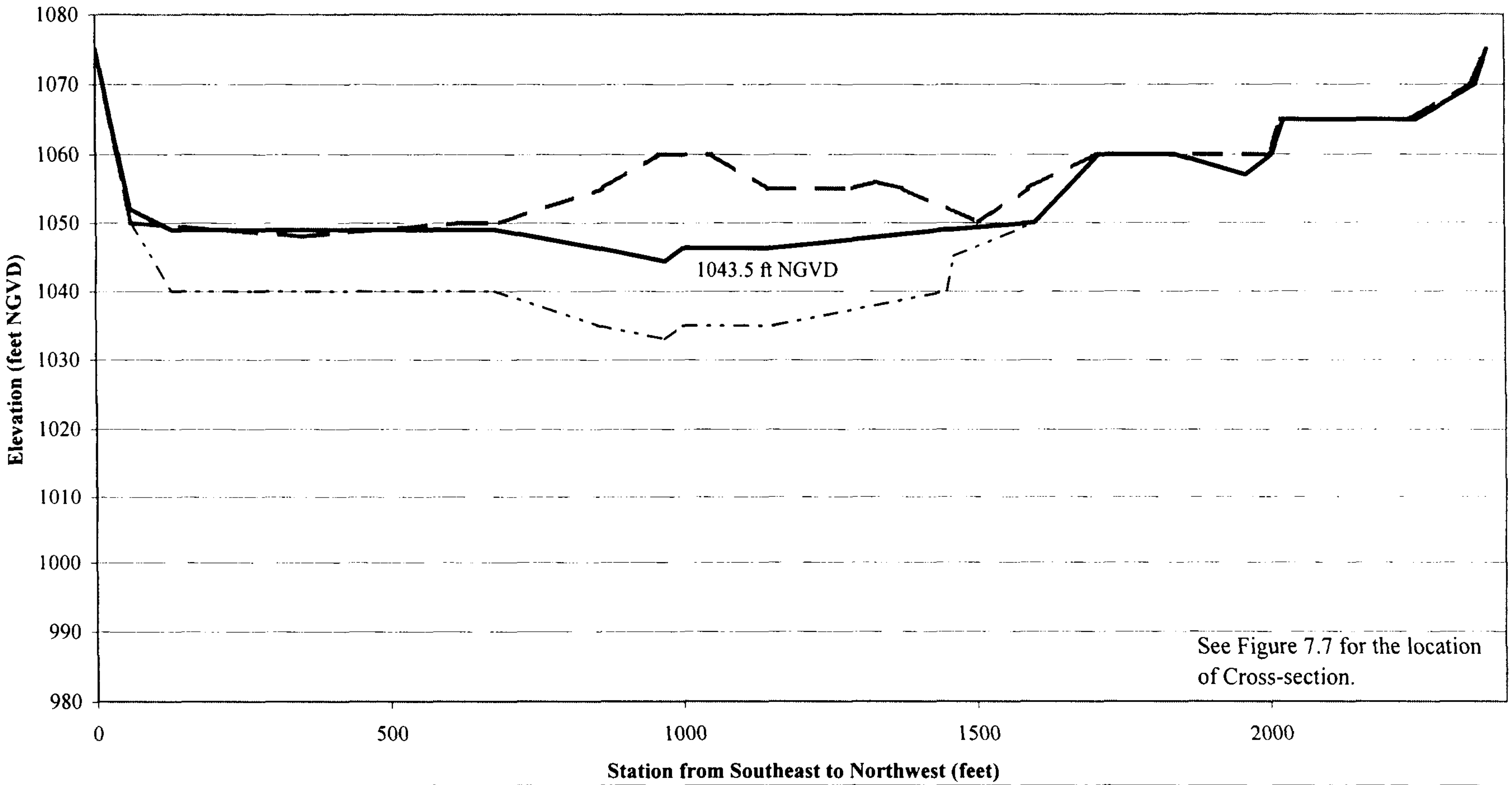
Source: PWA analysis, LACDPW historic data.

figure *7.17*

Sediment Transport Model Calibration Cross-Section #2

PWA

PWA #: 1310



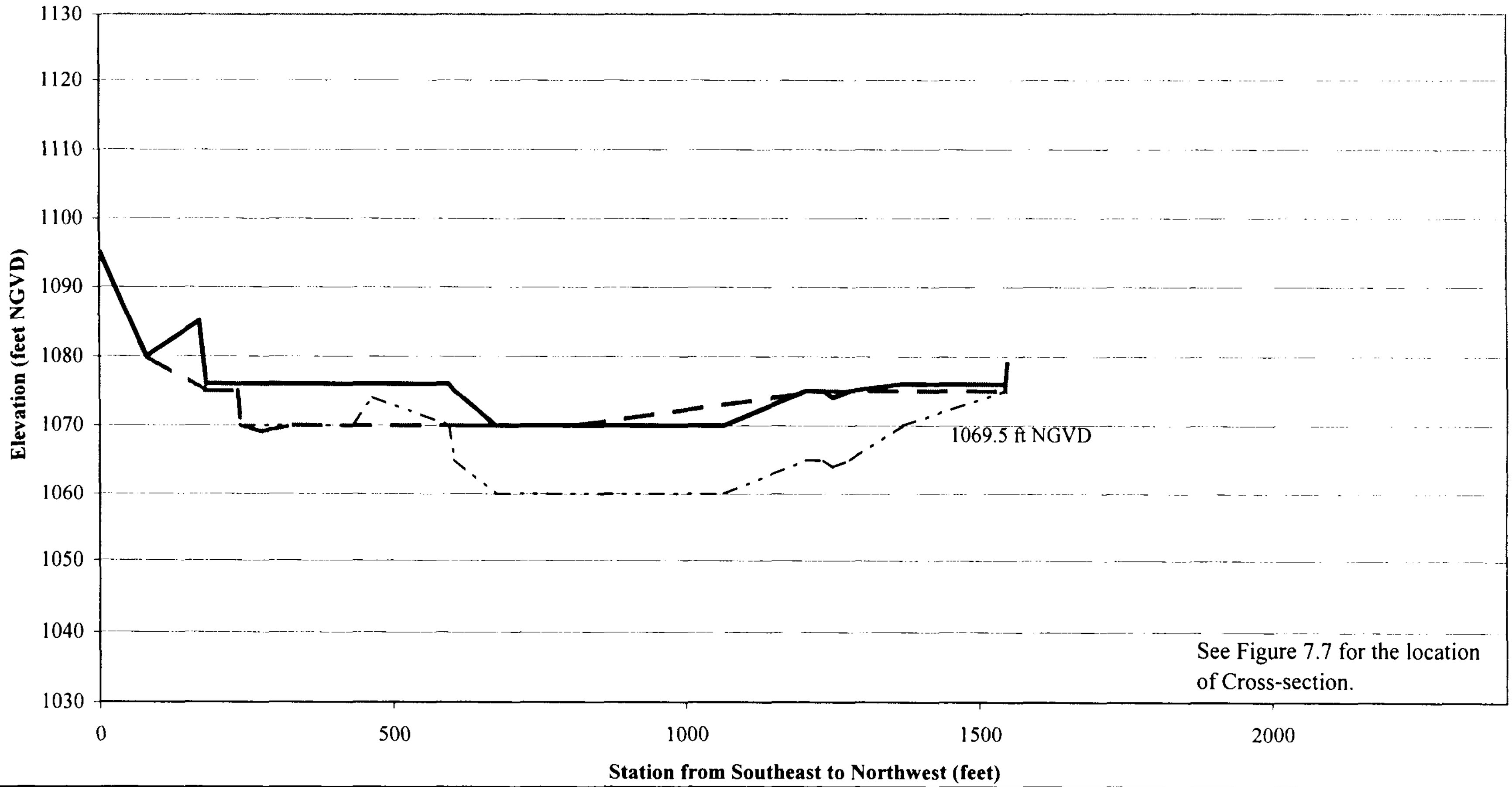
Source: PWA analysis, LACDPW historic data.

figure ***7.18***

Sediment Transport Model Calibration
Cross-Section #3

PWA

PWA #: 1310



8/34 - 11/34

Jun-38

1938 Model Calibration

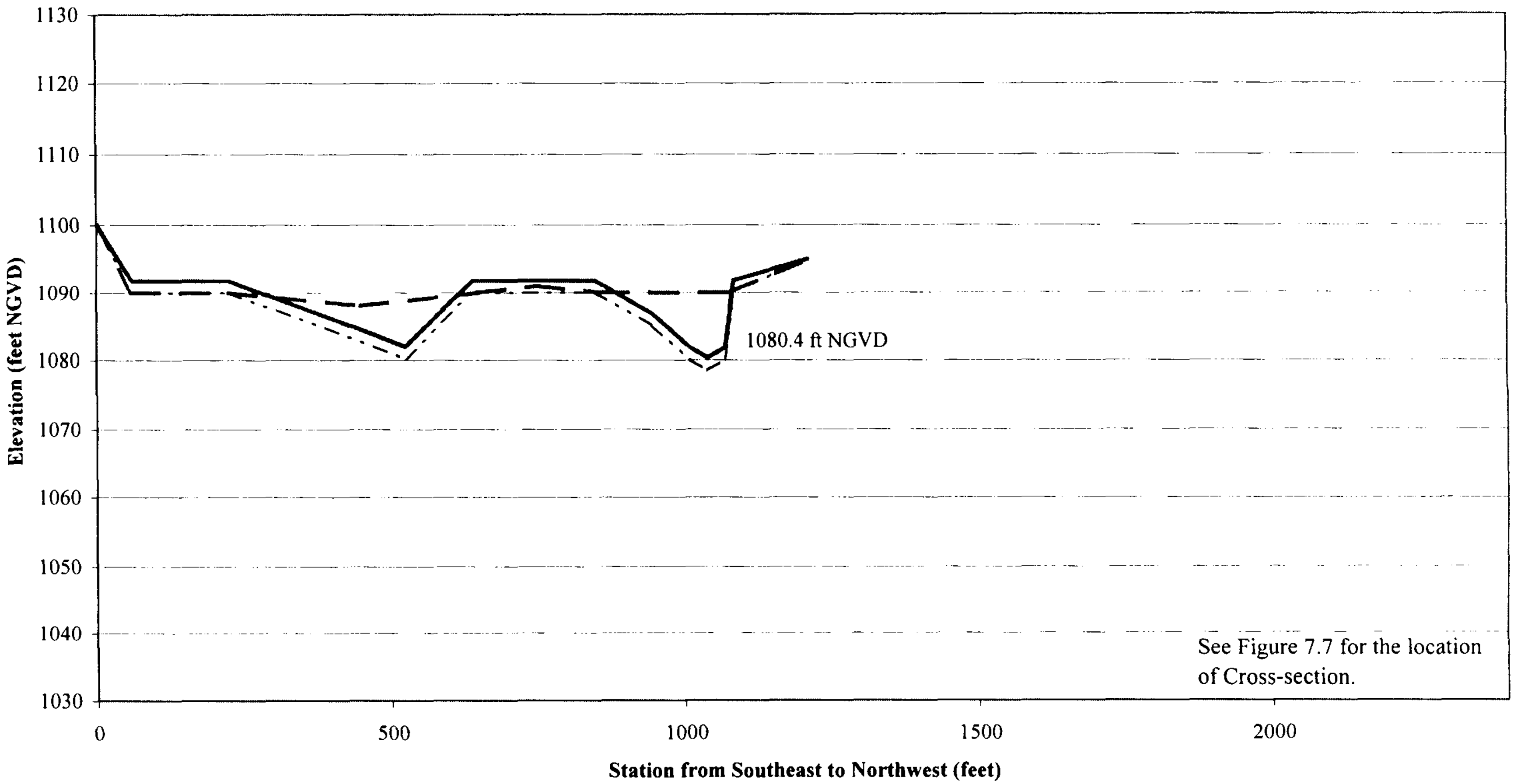
Source: PWA analysis, LACDPW historic data.

figure ***7.19***

Sediment Transport Model Calibration
Cross-Section #4

PWA

PWA #: 1310



- · · – 8/34 - 11/34

– – Jun-38

—— 1938 Model Calibration

Source: PWA analysis, LACDPW historic data.

figure ***7.20***

Sediment Transport Model Calibration
Cross-Section #5

PWA

PWA #: 1310

Figures 7.16 through 7.20 show that calibration results were quite good considering the extremely dynamic sediment transport conditions in the park. Table 7.4 shows modeled bed-elevation changes between 1934 and 1938, measured bed-elevation changes for the same period (based on LACDPW topographic data), and the model error in terms of a percentage of the measured bed-elevation change.

TABLE 7.4 Model Calibration Results: Modeled Versus Measured Bed-Elevation Changes, 1934 to 1938

Cross-section Number	Cross-section Location (Station)	Average Bed-Elevations (Feet NGVD)			Model Error
		Measured		Modeled	
		1934	1938	1938	
1	127+95	992	1012	1006.5	-28%
2	123+50	1010	1020	1023	+30%
3	118+50	1040	1053	1050	-23%
4	113+60	1060	1072	1073	+8%
5	111+14	1085	1090	1087	-60%

Source: PWA analysis.

Calibration results seemed to indicate that both small and large events are important in shaping the geomorphology of the park. Large events tend to move significant material into the park while smaller events play an important role in redistributing sediment within the park, especially downstream toward the dam.

7.2.3.2 *Predicted Sediment Delivery, 1934 to 1938*

As an additional check on the calibrated sediment transport model the amount of sediment delivery predicted by the model for the period between 1934 and 1938 was compared to the LACDPW's measured value of 1029 acre-feet deposited, or 1211 acre-feet total sediment delivery assuming a trapping efficiency of 0.85 (Section 7.1.3.2). The model calculated sediment delivery of approximately 1970 acre-feet over this time period, 62% more than LACDPW's measured value. Given the large amount of uncertainty associated with the sediment transport processes at the project site, this is a reasonable result; both values are of the same order of magnitude.

7.2.4 "Existing Conditions" Sediment Transport Model Results

Using the sediment parameter values from the calibrated sediment transport model, PWA used MIKE 11 to simulate sediment transport under existing park conditions for three different storm events: the 2-year, 10-year, and 50-year (Capital) events. Hydraulic conditions for these simulations were the same as described in Section 6.1, except that a sequence of consecutive storms and a shorter single-peak storm were also run to evaluate the importance of sediment redistribution within the basin during multiple events of different sizes and to evaluate the impact of the reservoir being essentially empty when the peak of a flood arrives. The storm sequence simulated was 16 days in duration, simulating a 10-year event, followed by a 2-year event, followed by a 50-year (Capital) event, with 48 hours between each event. The single-peak event was a 48-hour hydrograph capturing only the largest peak in the 50-year (Capital) flood. The following sections present single-storm and multiple-storm-sequence sediment transport results for existing conditions within the park and a discussion of these results.

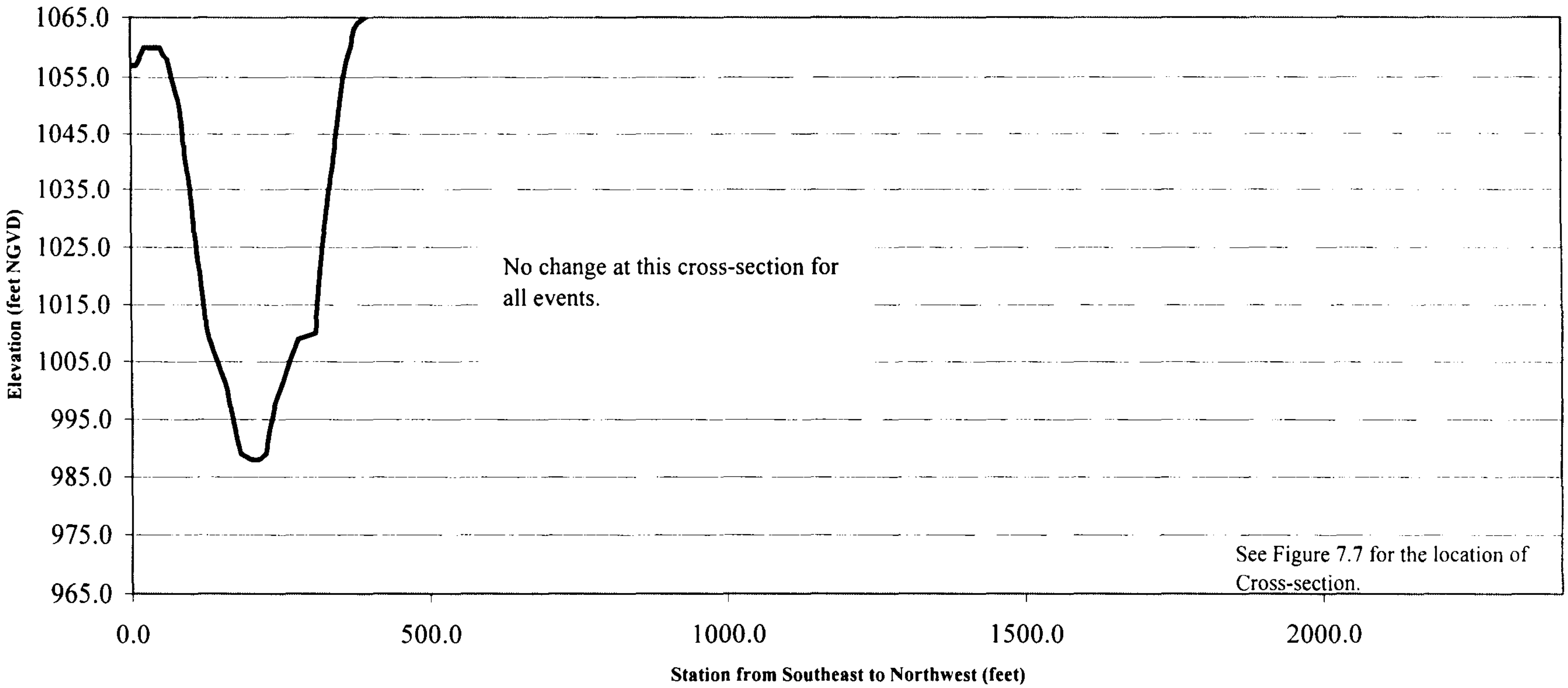
7.2.4.1 *Bed Elevation Changes*

Bed-elevation changes resulting from the modeled design storms and the storm sequence are shown in Figures 7.21 through 7.26. It should be noted that final bed-elevations following a flood do not necessarily reflect the elevations of the bed during the flood. Sediment transport and geomorphic change during flood events are extremely dynamic; over the course of a flood bed-elevations may fluctuate over a greater elevation range than the final bed-elevations represent. In general, existing condition results show that flood events entering the park tend to deposit their sediment loads in the upper portion of the basin. All floods showed deposition at the JPL bridge. Bed elevation changes at each of six cross-section locations for the three flood events and the flood sequence are shown in Table 7.5.

TABLE 7.5 Bed Elevation Changes Under Existing Park Conditions During Modeled Flood Events

Cross-section Number	Cross-section Location (Station)	Bed-Elevation Changes During Modeled Flood Event (feet)				
		2-year Event	10-year Event	50-year (Capital) Event	Flood Sequence	50-Year Single-Peak
1	127+95	0.0	0.0	0.0	0.0	0.0
2	123+50	-0.3	-0.3	0.3	-0.7	-0.7
3	118+50	-0.8	6.2	17.7	21.7	10.2
4	113+60	5.6	17.1	29.9	33.5	27.6
5	111+14	7.2	16.7	26.2	15.6	26.2
6	106+57	9.5	10.8	8.2	13.1	13.1

Source: PWA analysis.



November 1995

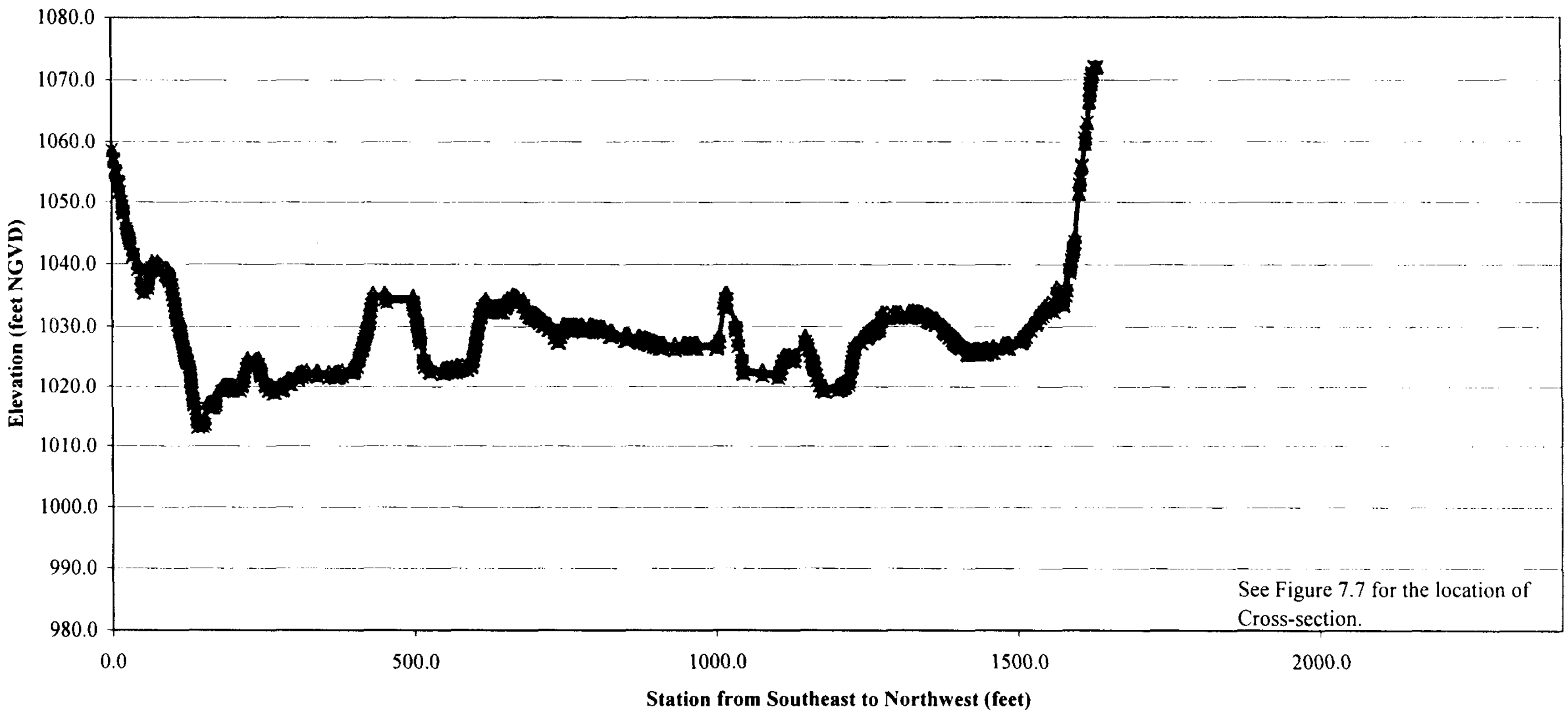
Source: LACDPW Topographic Data, PWA anlysis.

figure 7.21

Existing Conditions Sediment Transport Model Results, Cross-Section #1 Devil's Gate Dam Face

PWA

PWA #: 1310



November 1995
2-year Event
10-year Event
50-year (Capital) Event
Flood Sequence
50-year Single Peak

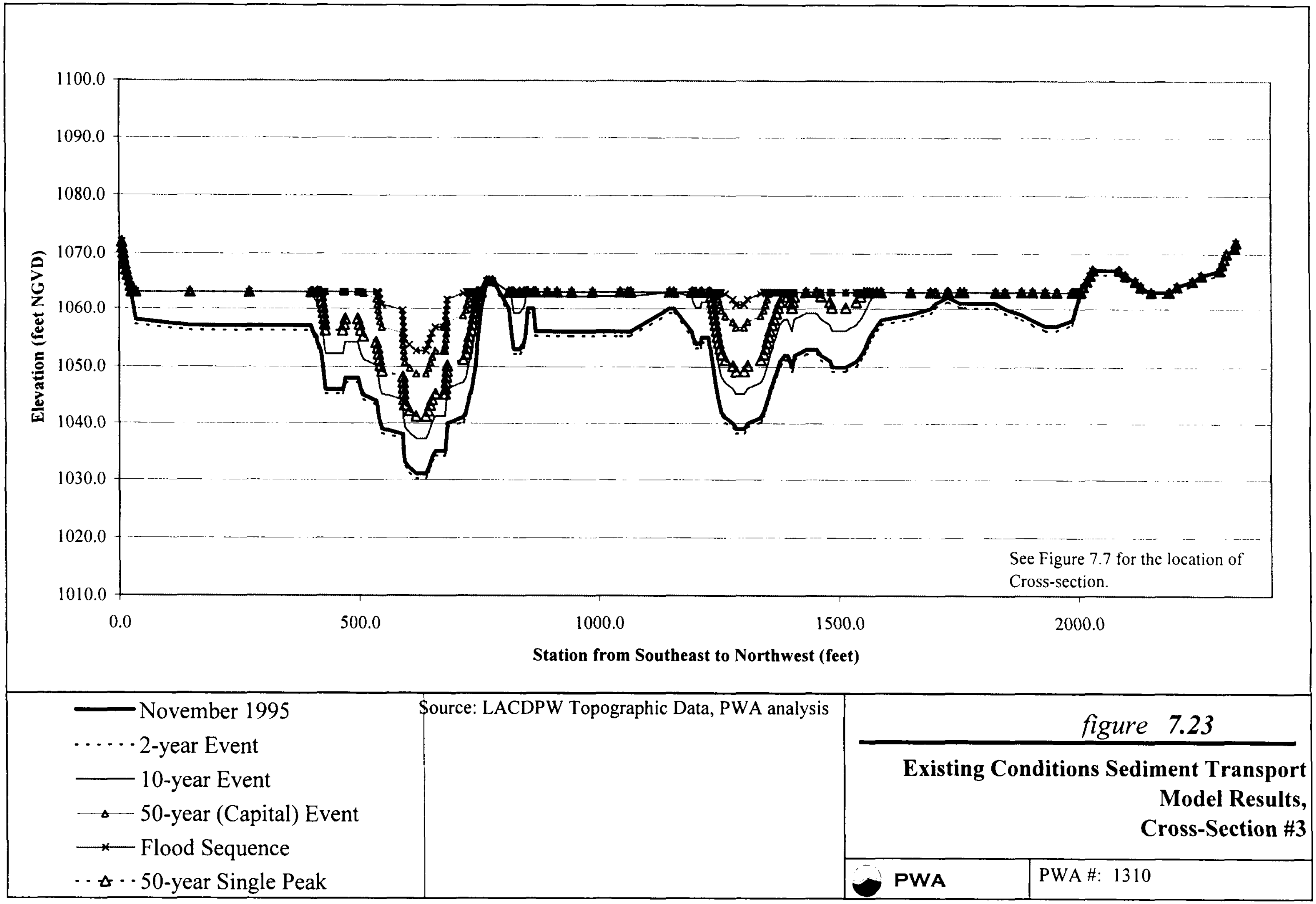
Source: LACDPW Topographic Data, PWA analysis.

figure 7.22

Existing Conditions Sediment Transport Model Results, Cross-Section #2

PWA

PWA #: 1310



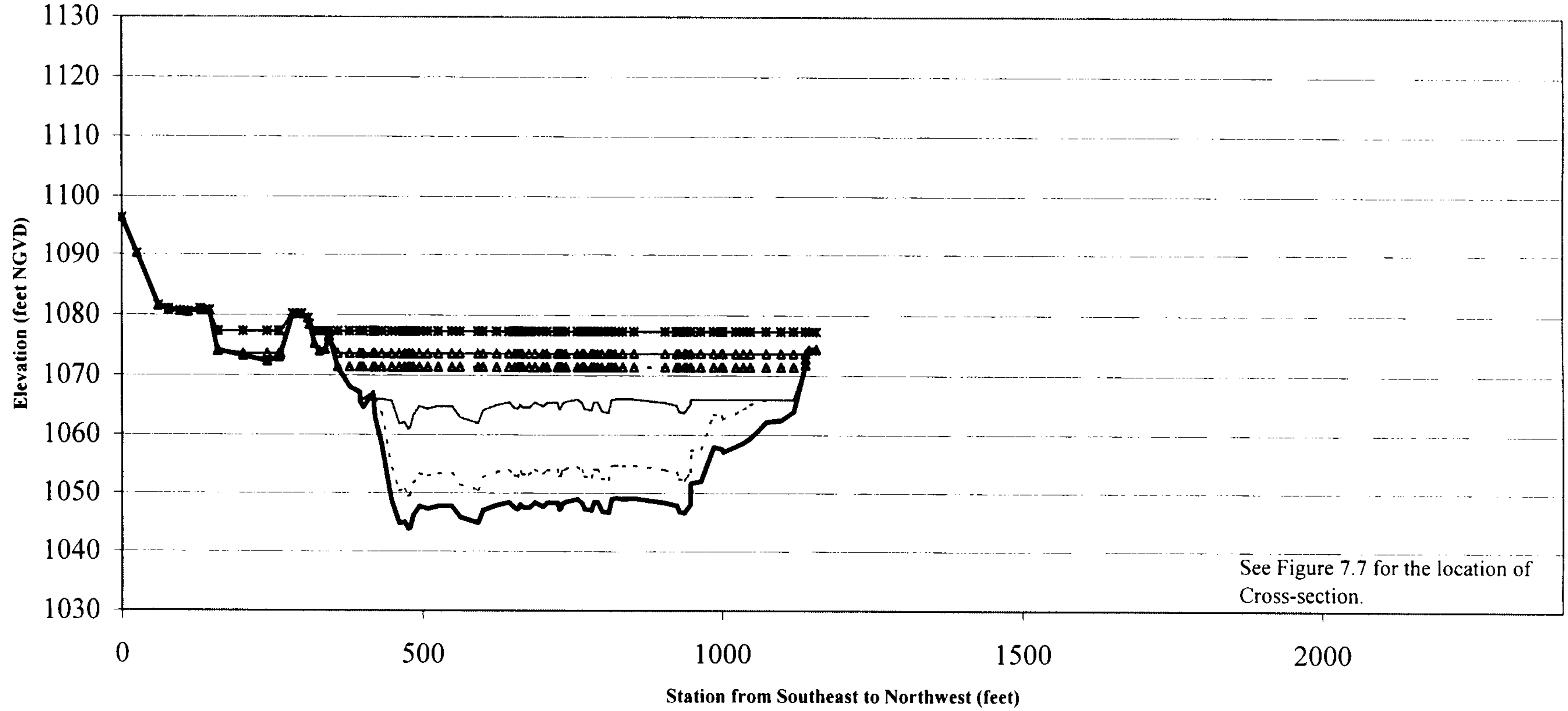
Source: LACDPW Topographic Data, PWA analysis

figure 7.23

Existing Conditions Sediment Transport Model Results, Cross-Section #3

PWA

PWA #: 1310



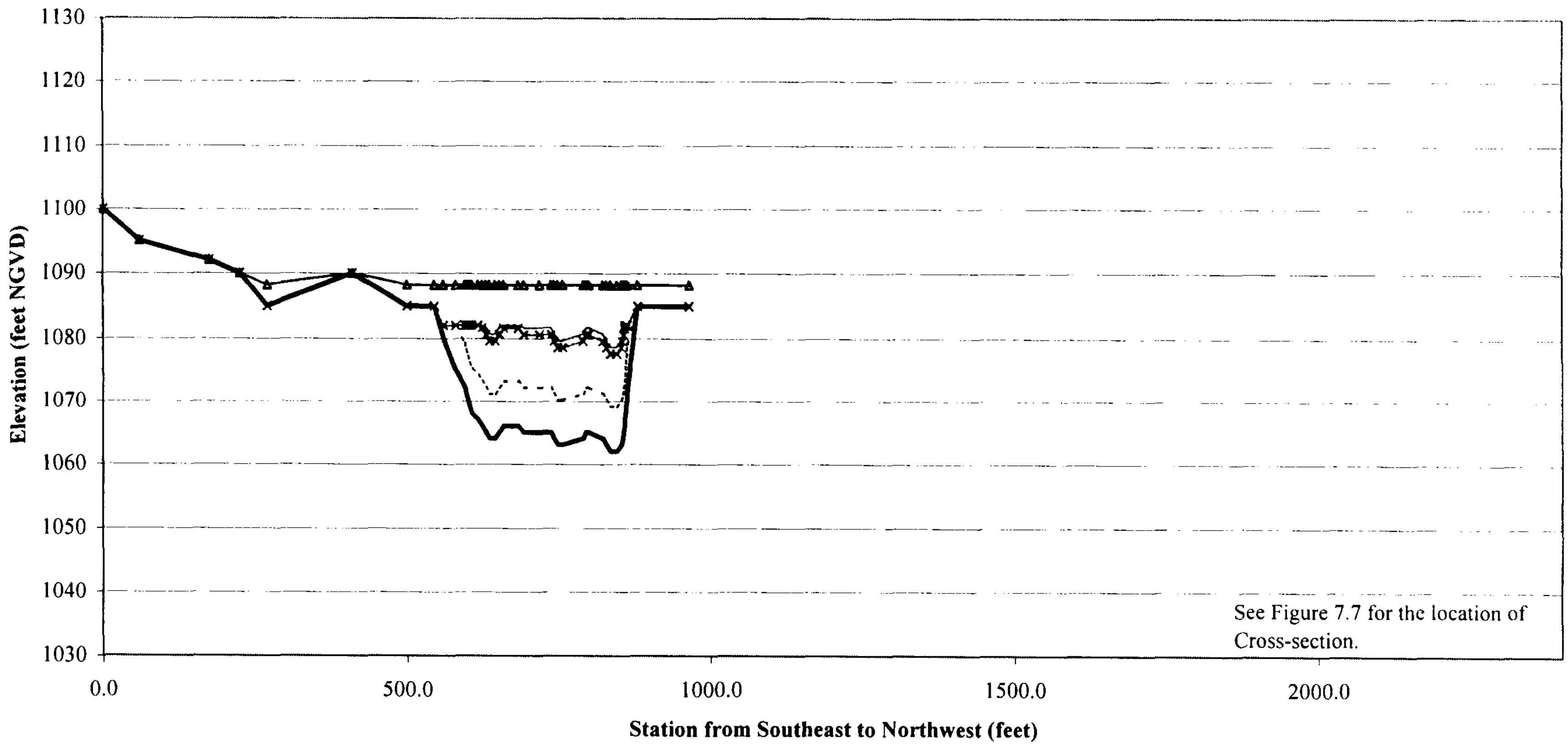
Source: PWA Survey, PWA analysis

figure 7.24

Existing Conditions Sediment Transport Model Results, Cross-Section #4

PWA

PWA #: 1310



Existing Condition ('81 & '95 Survey Combined)
2-year Event
10-year Event
50-year (Capital) Event
Flood Sequence
50-year Single Peak

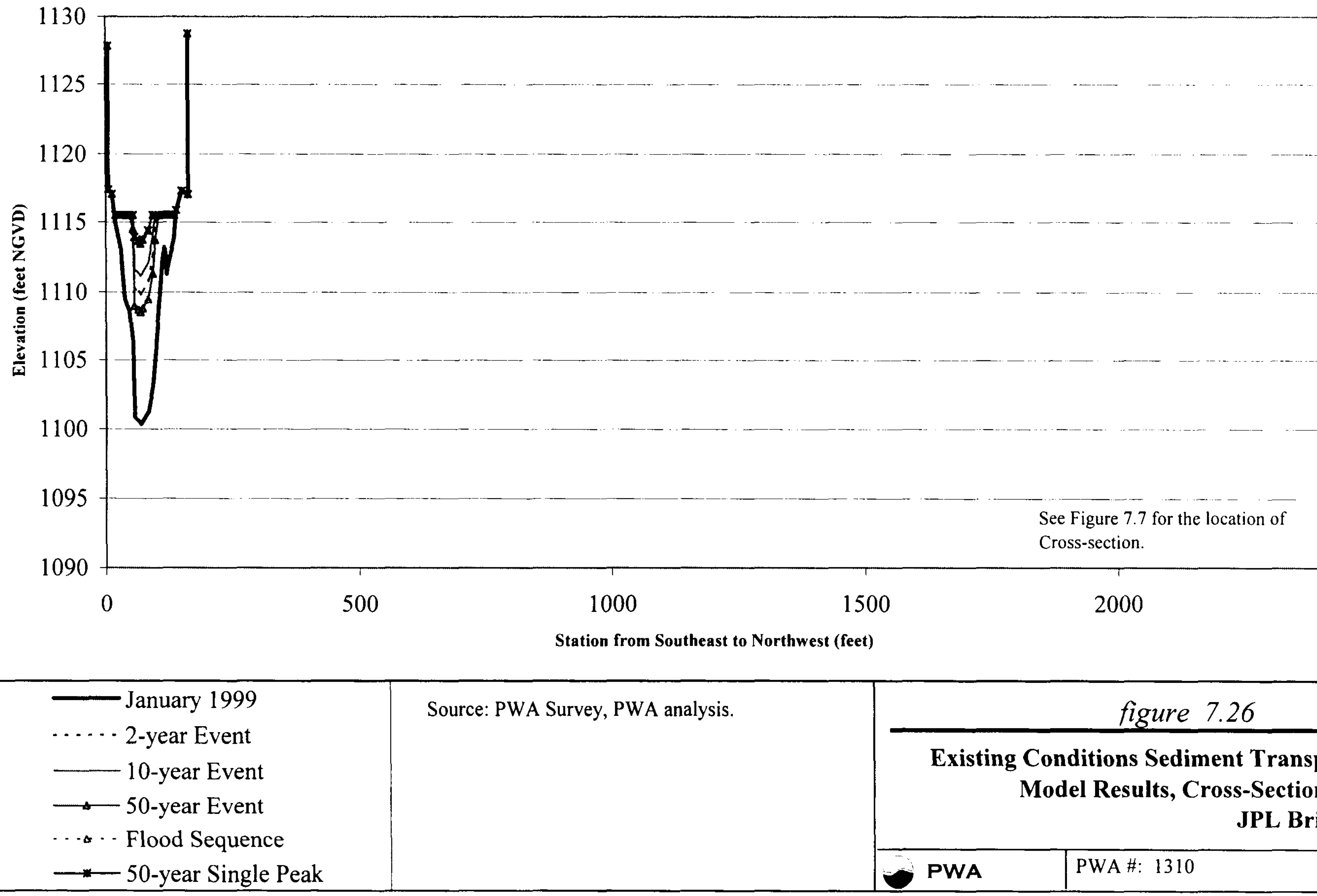
Source: LACDPW Topographic Data, PWA analysis.

figure 7.25

Existing Conditions Sediment Transport Model Results, Cross-Section #5

PWA

PWA #: 1310



Source: PWA Survey, PWA analysis.

figure 7.26

Existing Conditions Sediment Transport Model Results, Cross-Section #6 JPL Bridge

PWA

PWA #: 1310

7.2.4.2 *Sediment Delivery*

For each of the 2-year, 10-year, and 50-year (Capital) flood events the amount of sediment delivered to the park at the JPL bridge was also calculated by the model. These sediment volumes were compared to the sediment volumes predicted from PWA's event-based correlation between storm rainfall volume and sediment delivery (Section 7.1.3.2). As previously shown in Table 7.2 these sediment delivery values compare adequately given the amount of uncertainty involved in both the modeling and the regression.

7.2.4.3 *Results Discussion*

Several important conclusions may be drawn from the existing conditions sediment transport modeling results. First, as expected, the interface between the ponded reservoir area downstream and the fluvial zone upstream is an important deposition zone. At this interface sediments transported in the relatively fast-moving Arroyo meet a pool of water with very little velocity. At this point sediments slow down and are deposited. Although the location of this interface varies over the course of any given flood event, and varies between different sized flood events, the slowing of flow and sediments at the interface remains an important mechanism for deposition in the park in all flood situations.

Secondly, for the smaller events (2-year and 10-year) the channel expansion and topographic depression located just upstream of cross-section #5 slows flow considerably and causes significant deposition in the upper part of the park, near cross-sections #4 and #5. As small-event flow continues toward the dam, downstream of cross-section #4 it has lost much of its sediment load and transport power due to channel braiding. For the 2-year event we observed some erosion at cross-section #3, and for both of the smaller events we observed erosion at cross-section #2. This erosion indicates that flows dropped most of their sediment load upstream and are scouring a new sediment load from the channel. For the smaller events the model showed no sedimentation at cross-sections #2 or #1, near the dam. This lack of sedimentation seems to be due to several factors. The first is that the smaller amount of flow in the smaller events and the relatively low channel slope near the dam create flows that have little stream power to move sediment. The second factor is that the location of the interface between ponded water and channel flows at the peak of these smaller events occurs between cross-sections #2 and #3. Sediment transport beyond this interface is minimal. The third factor is that as a one-dimensional model, MIKE 11 does not account for the three-dimensional phenomenon of bedload transport near the dam due to the open lower release gate. This process would, in reality, be very significant in bringing sediment close to the dam during smaller flood events, and MIKE 11 is unable to simulate it. Therefore, these model results may not be accurate for the cross-sections near the dam and must be tempered with historical sediment accumulation data in making management decisions (Section 7.3.3.1).

Modeling results for the larger floods (50-year and 50-year single-peak events) also show the majority of sediment depositing in the upstream areas of the park, between the JPL bridge (cross-section #6) and cross-section #3. However the physical mechanism for this deposition does not seem to be primarily related to the channel expansion and topographic depression upstream of cross-section #5, as with the smaller events. The primary mechanism for deposition in the upper portion of the park seems to be the slowing of flow as it reaches the ponded water interface. For the larger floods this interface is in the upper reaches of the park (around cross-sections #4 and #5) during the peak of the flood when the most sediment is being moved. The large sediment loads transported during the peak flows are deposited soon after reaching the ponded interface. Thus we see significant sediment deposition in this upper zone following a large flood.

The larger events were able to push more sediment further into the park than were the smaller events; both of the 50-year floods modeled showed significantly more deposition at cross-section #3 than for the 10-year event. Deposition was not observed at cross-section #3 for the 2-year event. This deposition farther downstream in the park seems to be due to the larger stream power and higher sediment transport capacity of the higher flow events. The 50-year Capital event was observed to transport more sediment to cross-section #3 than the 50-year single-peak event. This may be attributed to the fact that in the preliminary stages of the flood the smaller peaks prior to the main peak in the Capital event contributed sediment to the upper park area which was later redistributed downstream by the main peak. The single-peak event does not have these smaller preliminary peaks, and therefore does not produce an initial upstream deposit for the large peak to redistribute downstream. Furthermore, we would generally expect greater deposition from the Capital event as compared to the single-peak event simply because a greater quantity of water flows into the park during the Capital flood, transporting a greater overall sediment load. Neither the Capital flood nor the single-peak flood were observed to transport sediment close to the dam. The upstream location of the ponded interface at the time of the peak flows (as previously described) likely accounts for this. However, again, this may not be an accurate result and model results must be tempered with historical sediment accumulation data in making management decisions (Section 7.3.3.1).

The flood sequence event (10-year flood followed by 2-year flood followed by the Capital flood) also produced significant upstream deposition. This may be attributed to the channel expansion and topographic depression downstream of cross-section #5 for the smaller events (as previously described) and the location of the ponded interface in the upper areas of the park for the larger Capital event (as previously described). Of all the existing conditions flood events simulated, the sequence was able to deposit the most sediment in the downstream area (cross-section #3) of any of the simulations. This suggests the importance of multiple events of different sizes in shaping the geomorphology of the park, as observed during model calibration (Section 7.2.3.1). The 10-year event brings sediment into the upper reaches of the basin; the 2-year event redistributes some of this sediment downstream toward the dam and deposits more in the upper reach; the rising limb of the 50-year (Capital) hydrograph further redistributes already-deposited sediments toward the dam and deposits still more in the upper zone. Additional smaller floods or prolonged residual flows following this flood sequence would likely redistribute sediments further, carrying them closer to the dam. The flood sequence transported the greatest amount of sediment into the park since this event moves the greatest volume of water.

Overall, the results of the existing conditions modeling seem reasonable. The two key mechanisms of deposition in the park, channel expansion/topographic depression downstream of cross-section #5 for smaller events and the slowing of flow at the ponded interface for all flows, are real physical phenomena. The one seemingly incongruent result is the lack of sedimentation near the dam face for all events modeled. Although sedimentation did occur in this zone during model calibration, none was observed during the existing conditions sediment transport modeling. There are physical explanations for this lack, but it seems contrary to historical data. It may simply be that multiple significant events (more than were modeled in the flood sequence simulation) are required to push sediment this far into the basin (as shown in the 1934 to 1938 model calibration simulation), and that the relatively short duration flood records modeled under existing conditions were unable to achieve this. In general, given historical records, we would expect more sedimentation near the dam following flood events under existing conditions than was shown by the model results.

7.3 RECOMMENDED SEDIMENT MAINTENANCE STRATEGY

7.3.1 Upstream Source Control Measures

Often, in cases where large amounts of sedimentation are occurring on a river, the rate of sedimentation can be reduced to some extent by applying source control measures in the upstream watershed. Source control measures are steps that reduce the upstream supply of sediment reaching the downstream area-of-concern. If channel down-cutting in the upper watershed is a significant source of the downstream sedimentation problem, check-dams or gully plugs can be installed to encourage channel aggradation in the upper watershed and reduce the channel erosion problem. Development and grazing in the upper watershed can cause erosion (including channel downcutting), which leads to downstream sedimentation. Another common source control measure is therefore limiting development and grazing in the upper watershed and restoring disturbed parts of the watershed to their natural condition.

Source control measures were considered in developing a sediment maintenance strategy for the Hahamongna Watershed Park. However, due to the naturally highly-erosive character of the upper watershed, and its generally undisturbed existing state, source control measures did not seem like a feasible way to reduce sediment inflow to the park on a long-term basis. Due to the natural vegetation and geologic conditions of the watershed (as described in Section 4), and the character of the San Gabriel Mountains in general, high rates of erosion are common for watershed areas such as the Arroyo Seco, even in a natural state undisturbed by human activities. Sedimentation at the foot of the San Gabriel Mountains—the geographic location of the Hahamongna Watershed Park—is simply a natural, unavoidable part of alluvial fan formation.

Grazing and development are not major land uses in the upstream watershed; much of the watershed remains in a relatively undisturbed condition. Some roads do traverse the upper watershed area and likely increase erosion to a certain degree. However they do not likely increase erosion and sedimentation rates significantly from natural conditions. Therefore, there is not an opportunity to reduce grazing and development to effect reduced sedimentation rates downstream. The one land-use practice that could have a significant impact on downstream erosion rates is fire management. Since catastrophic erosion events in the San Gabriel Mountains have been closely related to hill-slope conditions following large fire events, controlling and reducing large fire events in the watershed could be a long-term way of reducing sediment delivery to the Hahamongna Watershed Park. A regular program of controlled burning, such as that currently being developed by the USFS (Section 4.9.2) would likely bring a long-term decrease in sediment delivery to the basin, reducing potential for catastrophic watershed erosion events.

In the Los Angeles area a common method of preventing damage due to debris events and sedimentation from the San Gabriel Mountains is to construct large debris basins upstream of the vulnerable area. As part of this study PWA considered the potential for constructing such a debris basin upstream of the Hahamongna Watershed Park. The basin would act as the primary sediment deposition and excavation area, greatly reducing sediment delivery to the park below. However, this form of upstream source control was considered infeasible since it would likely come at great environmental expense to the relatively undisturbed upstream riparian zone, and since it would necessitate large maintenance truck traffic through adjacent neighborhoods that are primarily residential. Furthermore, this strategy could require more excavation since sediments that may not have made it to the park would be captured in the basin.

7.3.2 Flow-Assisted Sediment Transport

Allowing sediment mobilized by stream flow to pass through the lower gates of the dam is a process known as flow-assisted sediment transport. This mode of operation should be practiced as frequently as possible at Devil's Gate Dam because it will reduce the amount of sediment that must be excavated mechanically from the reservoir, thus reducing maintenance costs therein.

Flow-assisted sediment transport is closely related to dam operations during flood events. Current dam operations seem to provide a flow regime that is beneficial to transporting sediment out of the reservoir. In fact, maximizing sediment transport out of the reservoir is one of the stated goals of LACDPW's operating strategy at Devil's Gate Dam. This mode of operation has been in use by the LACDPW at the Devil's Gate facility for over 20 years and is also used by the U.S. Army Corps of Engineers at its flood control dams in the Los Angeles County area. During low base-flow conditions on Arroyo Seco the lowest release gate on the dam is left open. This allows any small sediment-mobilizing flow events to convey sediment past the dam and out of the park area. During larger flood events, where reservoir water-levels rise significantly, the release gate is kept open until water levels reach 1010 feet NGVD. At this point the release gate is closed and outflow continues through the other higher-elevation gates and spillway. LACDPW has found that this strategy of passing the peak flood flows through the higher elevation outlets reduces the potential for sediment clogging at the lower release gate. If the release gate were left open during the high-flow period of the storm peak volumes of bedload sediment would be drawn toward the gate (situated approximately at the reservoir bed elevation), increasing the potential for clogging. With the lower gate closed sediment tends to drop out of suspension near the interface between the reservoir pool and the flowing stream, farther away from the dam and gate. After the peak of the flood, when water-surface elevations drop below elevation 1010 feet NGVD, the lower release gate is reopened and normal low-flow operations are resumed. Overall, LACDPW's operations strategy seems to balance well the desire to manage flood elevations in the reservoir and downstream, to maximize flow-assisted sediment transport, and to minimize the risk of sediment blockage at the dam's outlets.

In the event that the lower release gate does become blocked during a large flood event, it may be beneficial to immediately excavate the area directly adjacent to the lower gate and allow residual flood flows to erode a channel upstream from the initial excavation, transporting sediment as it does so. This method seems to have been practiced historically at Devil's Gate Dam. Since it could increase quantities of sediment passed through the dam it merits further study by the LACDPW. While accurately quantifying flow-assisted sediment transport potential at Devil's Gate Dam was beyond the scope of this study, historical numbers indicate that approximately 20% of sediment delivered to the park could be transported through the dam if dam operations are oriented to this end (Section 4.10.4). This quantity is probably a reasonable conservative estimate of flow-assisted transport potential. PWA recommends that a sediment transport monitoring plan be implemented for flows through all dam outlets so that a better understanding of vertical sediment distribution in the water column, and ultimately transported sediment quantities, can be gained.

7.3.3 Sediment Excavation

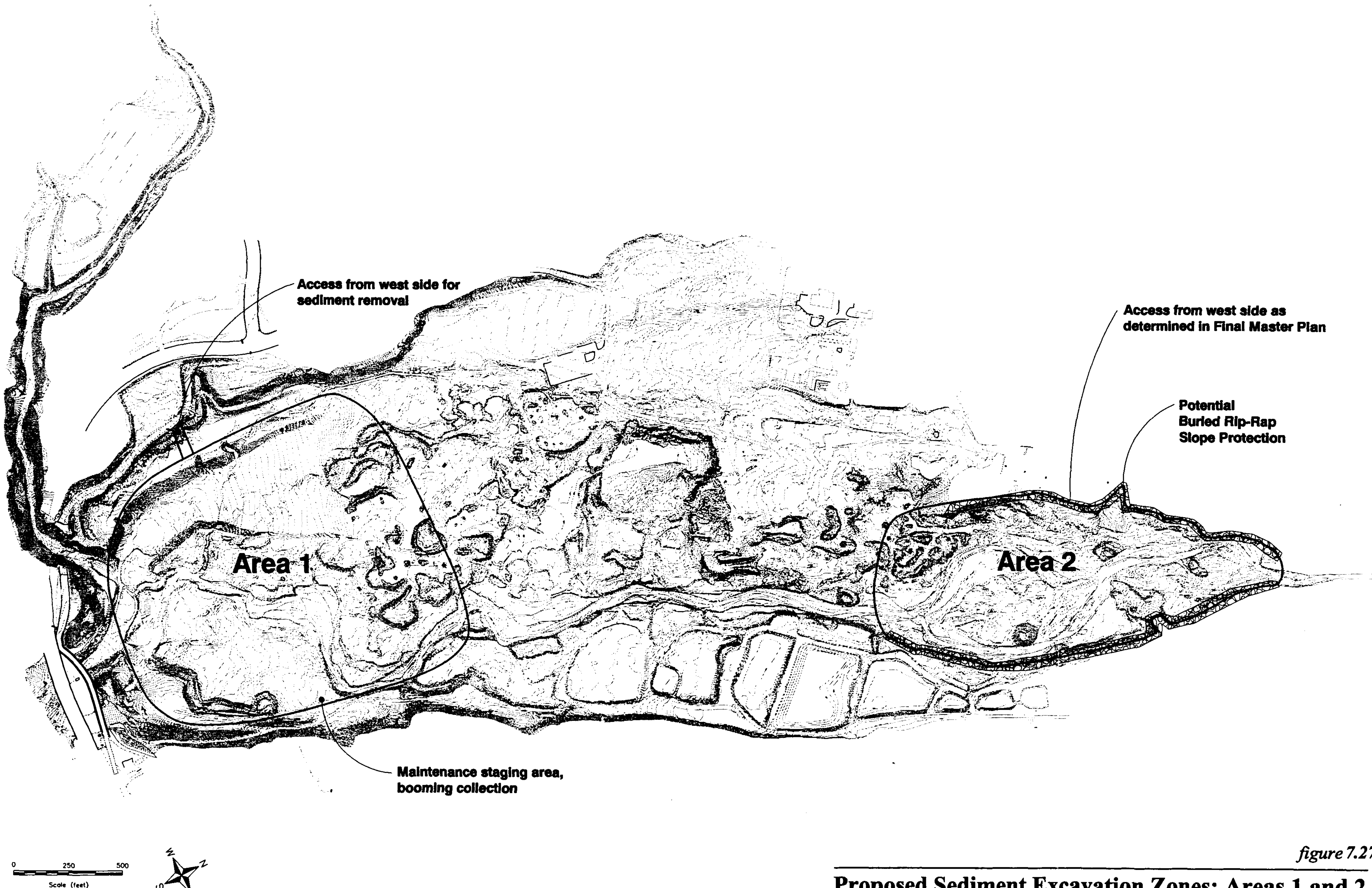
7.3.3.1 *Excavation Locations*

Historic data (maps, photos, cross-sections, and profiles), modeling results, and current Arroyo Seco morphology show that there are two main areas of sediment deposition within the Hahamongna Watershed Park. The first area, Area 1, is directly upstream of the dam face. The second area, Area 2, is adjacent to spreading ponds 3 through 10, immediately downstream of the narrow, confined section of channel in the upper portion of the park. Both areas are shown in Figure 7.27. The limits of Areas 1 and 2 shown in Figure 7.27 are conceptual and are therefore subject to further refinement and change during future design phases.

PWA recommends that, under ordinary circumstances, sediment maintenance excavation zones correspond to these primary deposition areas. In this way sediment excavation will be most efficient and mainly localized to two controlled sites, allowing wildlife habitat to remain undisturbed for longer periods of time in other parts of the park. If it proves feasible from an economic and regulatory perspective, Area 1—the largest of the two depositional areas—could be sub-divided into two or three sub-areas. Excavation from each of the sub-areas could be alternated to give habitat in Area 1 a longer life-span.

Equipment and truck access is an important element of planning for maintenance excavation and sediment removal from the park area. For Area 1 a haul road should be implemented on the west side of the park, connecting the excavation area with the main road planned to circum-navigate the park. A western haul road will create a much shorter distance to the freeway than a haul road on the east side of the park. Landings and staging areas should be constructed as required by LACDPW. An equipment landing near the southeast corner of Area 1 may be useful to facilitate floating debris collection following large storm events. According to representatives of the City of Pasadena, predominant wind patterns tend to direct floating debris to this area of the reservoir (pers. comm. John Cox). For Area 2 a short haul road should be constructed on the west side of the excavation site, connecting it to the main park circulation road. Figure 7.27 roughly shows proposed access roads and the location of the southeastern equipment landing. Locations of access points are conceptual and are therefore subject to refinement and change during future design phases.

Although our findings show that the main deposition zones in the park are Areas 1 and 2 and that these areas should be the primary excavation areas, it is crucial to note that these are simply areas where sedimentation is concentrated. During large flood events significant sedimentation may occur in areas outside of Areas 1 and 2, including the main wildlife habitat zones. Due to the uncertainties associated with sedimentation processes in an active reservoir it is impossible to predict with complete accuracy the patterns of sedimentation that will occur in the park during flood events. Periodic maintenance excavation may be necessary in park areas outside of the designated excavation zones, such as the proposed riparian zone



Note:
Excavation areas are schematic and conceptual.
Limits are subject to refinement and change.

Map units are superimposed on 1995 topography.

figure 7.27

Proposed Sediment Excavation Zones: Areas 1 and 2
Hahamongna Watershed Park

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between Areas 1 and 2. Any park structures located outside Areas 1 and 2 but within the LACDPW flood control and water conservation easement may require maintenance or restoration by the City after significant events and LACDPW remediation activities.

7.3.3.2 *Performance-based Management*

PWA proposes that all sediment maintenance excavation within the park be conducted on a performance basis. Since large sediment transport events occur infrequently and unpredictably, rather than prescribe set annual or biennial excavation quantities, sediment should be excavated from the park on an as-needed basis. A preferred geometry should be set and then a trigger elevation above that preferred geometry. Excavation zones should be initially configured to the preferred geometry. When sediments accumulate to the trigger elevation the areas should be excavated back down to the preferred geometry. Cross-sections of the excavation areas should be surveyed following flood events to determine if, on average, the trigger elevation has been reached at that location.

If performance-based management is infeasible for regulatory reasons, prescribed annual or biennial excavation maintenance could be implemented. Under this alternative strategy, maintenance zones would be returned to their preferred geometries on a prescribed annual or biennial basis. Regulatory concerns that could necessitate this alternative prescribed maintenance strategy are discussed in Section 7.3.4.

7.3.3.3 *Proposed Excavation Zone Geometry*

For each of the proposed excavation zones, Areas 1 and 2, PWA developed a preferred design geometry. Several important criteria guided the development of this geometry. The first criterion was that proposed geometry should not reduce the overall capacity behind Devil's Gate Dam at elevation 1040.5 (the elevation of the spillway) from 1995 levels. LACDPW expressed a desire to maintain the elevation-storage relationship established in 1995. Also, LACDPW requires that sediment excavation zones have a total debris capacity of 1200 acre-feet to accommodate an extreme sediment deposition event.

The second criterion was to not significantly increase flood hazards to the spreading grounds operated and maintained by the City of Pasadena Water and Power Department. The City established a requirement of at least five feet of freeboard between the maximum allowable ground-surface elevation and flooding elevations for the spreading grounds. This five feet should provide adequate channel capacity to convey flood-waters without inundating the spreading grounds. This criterion was relevant only for Area 2, adjacent to the spreading grounds.

The third criterion was to minimize impacts to existing sensitive habitat. Currently, a significant zone of Coastal Sage Scrub—a valuable diminishing habitat in Southern California—exists in most of Area 2. Team biologists advised PWA that this area should not be excavated from its existing condition, but that excavation could take place in Area 2 following a large flood event that buried the Coastal Sage Scrub with at least 5 feet of sediment (James Eckert, pers. comm.). Therefore, Area 2 should be allowed to aggrade somewhat before excavating any material.

The fourth important criterion for establishing preferred geometries in the two excavation zones was that the geomorphology of the zone should be as stable as possible. This is achieved primarily by establishing a slope that will not initiate significant channel erosion or down-cutting. PWA chose a slope of 0.02 or 2% as the preferred slope for the two excavation zones. This slope was chose because historic longitudinal profiles seem to indicate that this is the slope at which sediments tend to naturally settle out in the upper (fluvial) portion of the park following a large flood event. This is the slope at which sediments settled in the upper part of the basin following the 1938 flood event. Therefore, a 2% slope was assumed to approximate a preferred or equilibrium channel slope for the excavation zones.

Based on these criteria PWA established preferred geometries for the two excavation zones, Areas 1 and 2. Figures 7.28 through 7.34 show these preferred geometries in cross-section and longitudinal profile, along with proposed excavation trigger elevations, superimposed on existing conditions topography. For Area 2 the City's flood control elevations are included. Excavation would occur in Area 1 after approximately 10 feet of sedimentation and in Area 2 after approximately 5 feet of sedimentation. Figure 7.30 shows the preferred riparian reach cross-section proposed by Takata Associates. Although regular LACDPW maintenance would not be prescribed for this reach, because sedimentation may occur periodically in this reach the City should establish an excavation trigger elevation that is beneficial to maintaining their preferred habitat. One possibility would be to excavate after approximately 5 feet of sedimentation in the riparian reach.

Areas 1 and 2 should be graded wide and flat with gentle side-slopes of 4:1 or 5:1. Buried slope protection should be implemented in the excavation zones adjacent to any important structures near the edge of Area 2, as shown in Figure 7.27. This will protect structures from natural channel migration that will likely occur in the broad flat zone of Area 2. The advantage of burying the slope protection is that it will only be exposed if erosion occurs and the protection is needed. It should also be noted that localized over-excavation below the proposed preferred geometry is not recommended as it could cause head-cutting and erosion problems upstream of the excavation.

If the preferred geometry shown in Figures 7.28 through 7.34 is implemented, Area 1 will have a debris event sediment capacity of approximately 390 acre-feet below elevation 1020 feet NGVD, and approximately 1300 acre-feet below elevation 1040.5 feet NGVD (See section 7.3.3.5 for further discussion of quantities). Including the approximately 90 acre-feet of debris capacity in Area 2 (the volume between the preferred and trigger elevations in that area) the reservoir will have approximately 1390 acre-feet of debris capacity in the designated excavation areas. This debris capacity exceeds LACDPW's single debris-event capacity requirement of 1200 acre-feet by approximately 190 acre-feet.

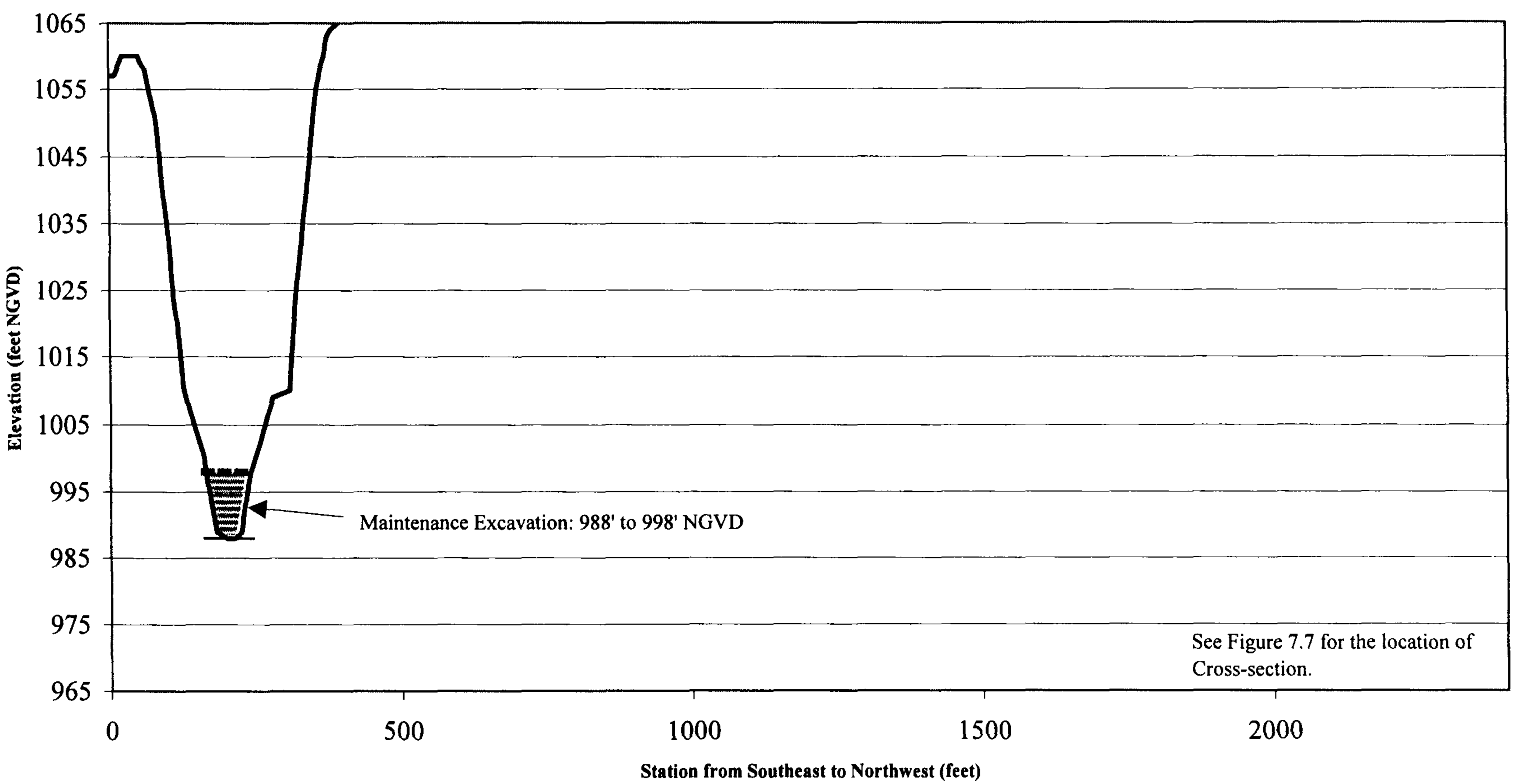
Furthermore, if the preferred geometry shown in Figures 7.28 through 7.34 is implemented, flood storage capacity in the reservoir below elevation 1040.5 feet NGVD will be initially increased. Although implementing the preferred geometry would allow some sediment accumulation in Area 2 to raise it to the recommended elevations, this accumulation would not significantly affect reservoir flood storage capacity since virtually all of Area 2 is already above elevation 1040.5 feet NGVD under existing conditions. The implementation of Area 1 (all of which is below elevation 1040.5 NGVD) as recommended will require an initial excavation from existing conditions of approximately 310 acre-feet (see Section 7.3.3.5). Therefore, reservoir flood storage capacity will be increased from existing conditions by approximately 310 acre-feet if the preferred geometry is implemented. During the next phase of design, as detailed grading alternatives are formulated, the volume of any fill features proposed in Area 1 as part of the park master plan should be quantified to ensure that LACDPW reservoir capacity criteria are met.

The proposed geometry should keep spreading grounds flood hazards at levels that are acceptable to the City. Furthermore, the existing area of Coastal Sage Scrub habitat would not be initially disturbed and the slopes through Areas 1 and 2 would be approximately 2%.

7.3.3.4 *"Proposed Conditions" Sediment Transport Results*

PWA used MIKE 11 to simulate sediment transport under proposed park conditions for three different storm events: the 2-year, 10-year, and 50-year (Capital) events. Hydraulic conditions for these simulations were the same as described in Section 6.1. Two geometries were modeled for proposed conditions. The first may be called the true "proposed" condition, the condition of the park after the initial excavation of Area 1, after the regrading of the proposed riparian reach, and prior to significant deposition in Area 2. This is essentially the park geometry immediately following construction. The second geometry modeled may be called the "preferred" condition, when topography in the park is at the preferred elevations outlined in Section 7.3.3.3. This geometry includes initial excavations and expected deposition in Area 2 and upstream.

Figures 7.35 through 7.40 show bed-elevation changes following each flood event at the six cross-section locations for proposed conditions. Figure 7.41 shows longitudinal thalweg-elevation profiles following the modeled flood events. Modeled bed-elevation changes for proposed conditions geometry are also shown in Table 7.6.



Nov-95

Preferred Geometry

Excavation Trigger Geometry

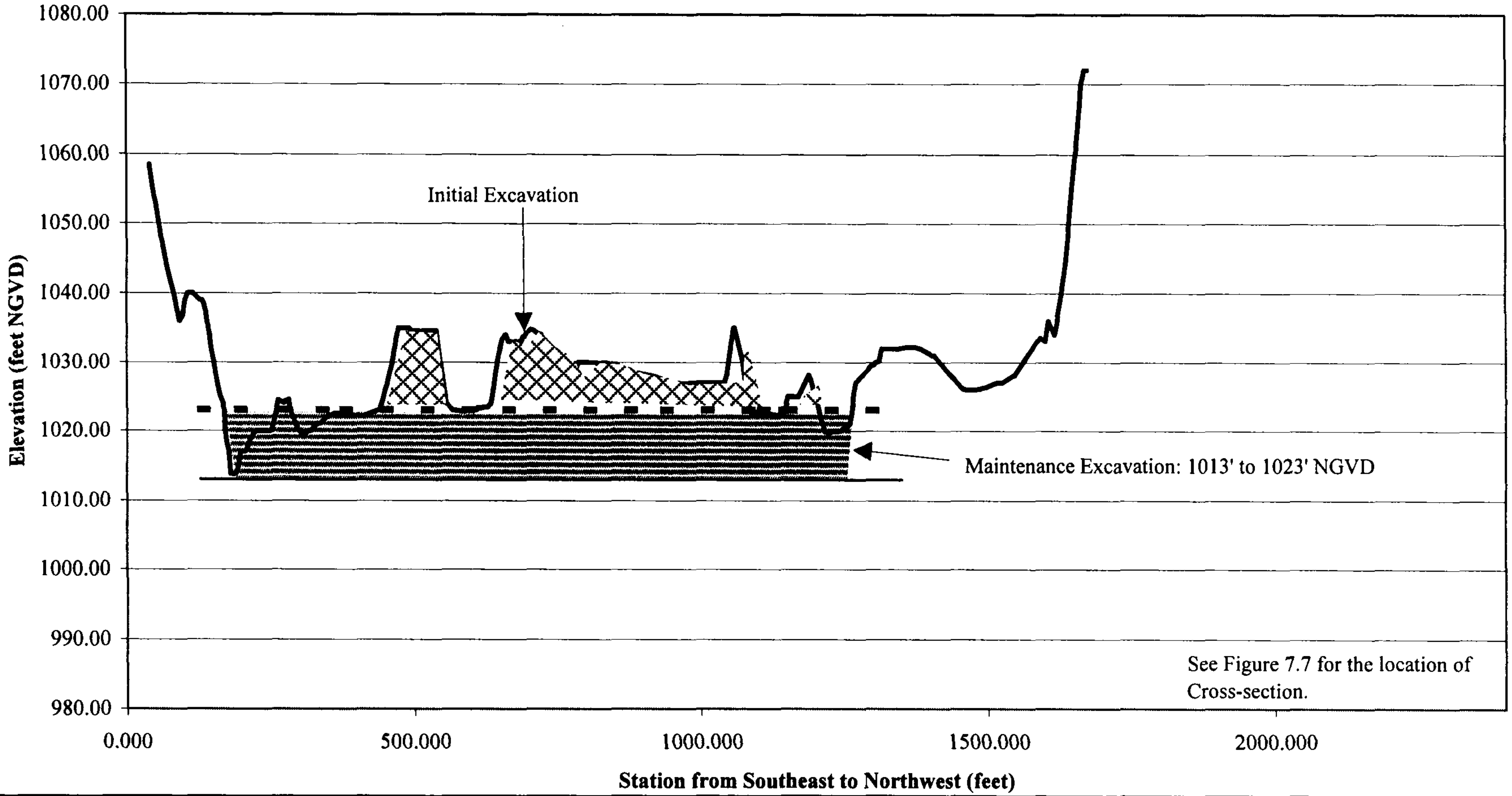
Source: LACDPW Topographic Data
Notes: Existing thalweg elevation (988 feet NGVD) is an appropriate preferred geometry at this cross-section. Excavation trigger elevation set at 998 feet NGVD, 10 feet above preferred geometry.

figure 7.28

Proposed Geometry for Excavation Zone 1
Cross-Section #1
Devil's Gate Dam Face

PWA

PWA #: 1310



Nov-95

Preferred Geometry

Excavation Trigger Geometry

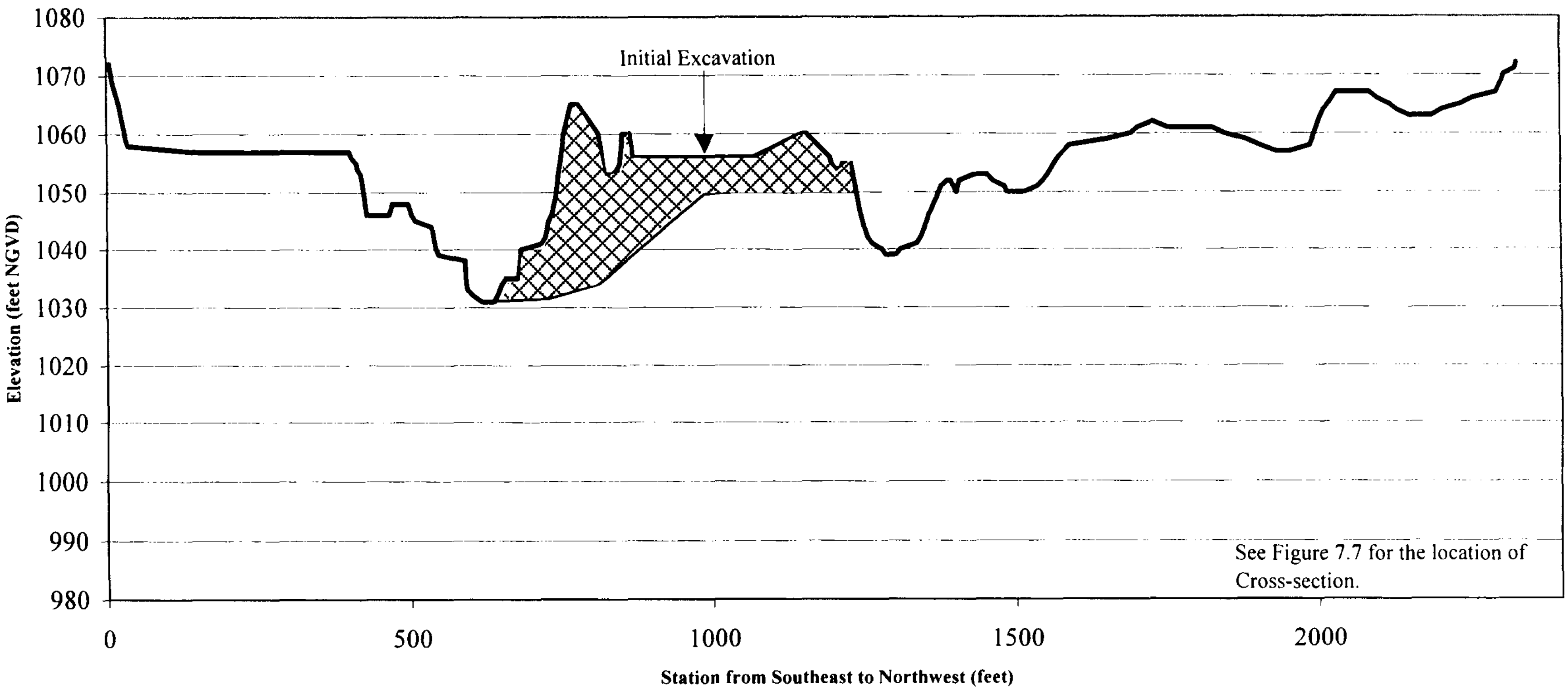
Source: LACDPW Topographic Data.
Notes: Preferred geometry is elevation 1013 feet NGVD, excavation trigger elevation set at 1023 feet NGVD, 10 feet above preferred geometry.

figure 7.29

Proposed Geometry for Excavation Zone 1 Cross-Section #2

PWA

PWA #: 1310



Source: LACDPW Topographic Data

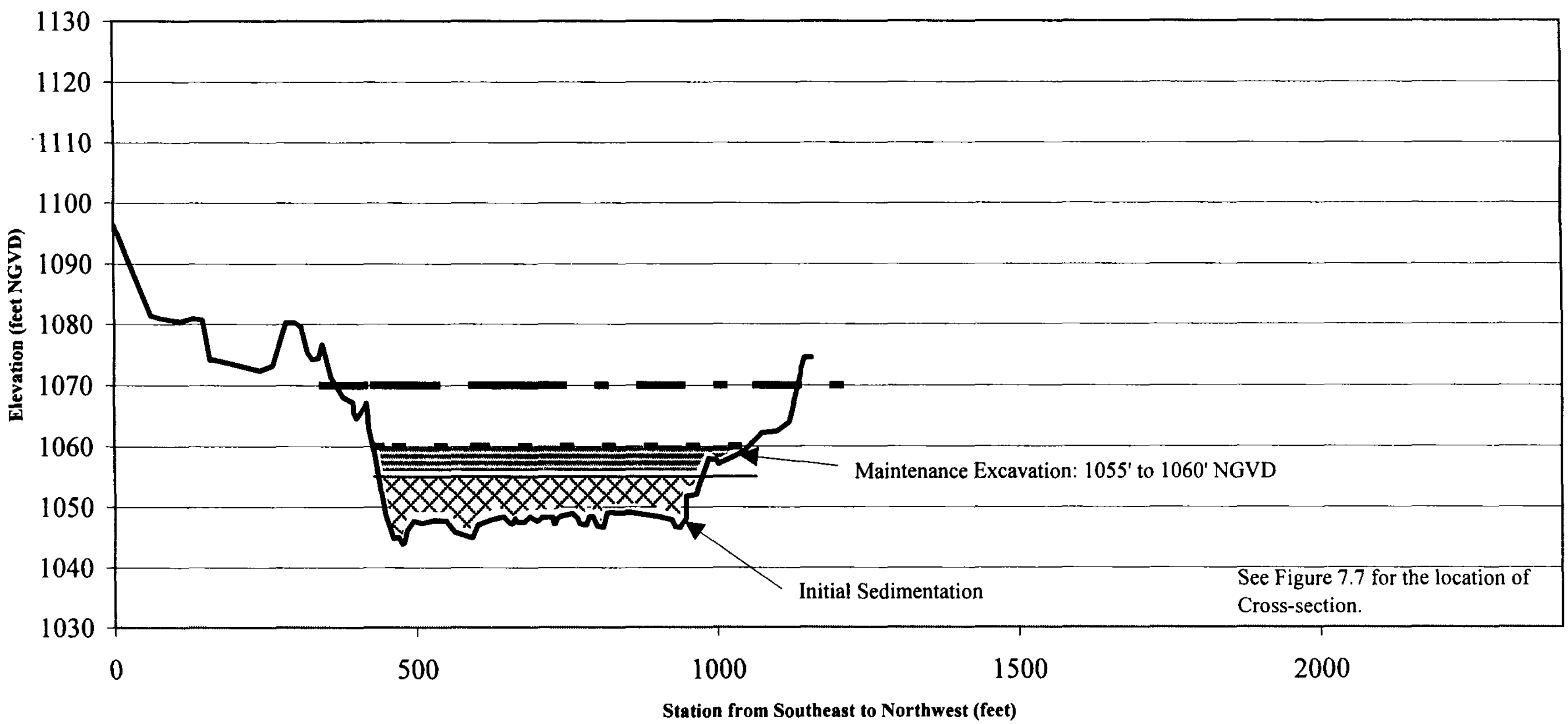
Notes: Proposed cross-section geometry from Takata Associates.

figure ***7.30***

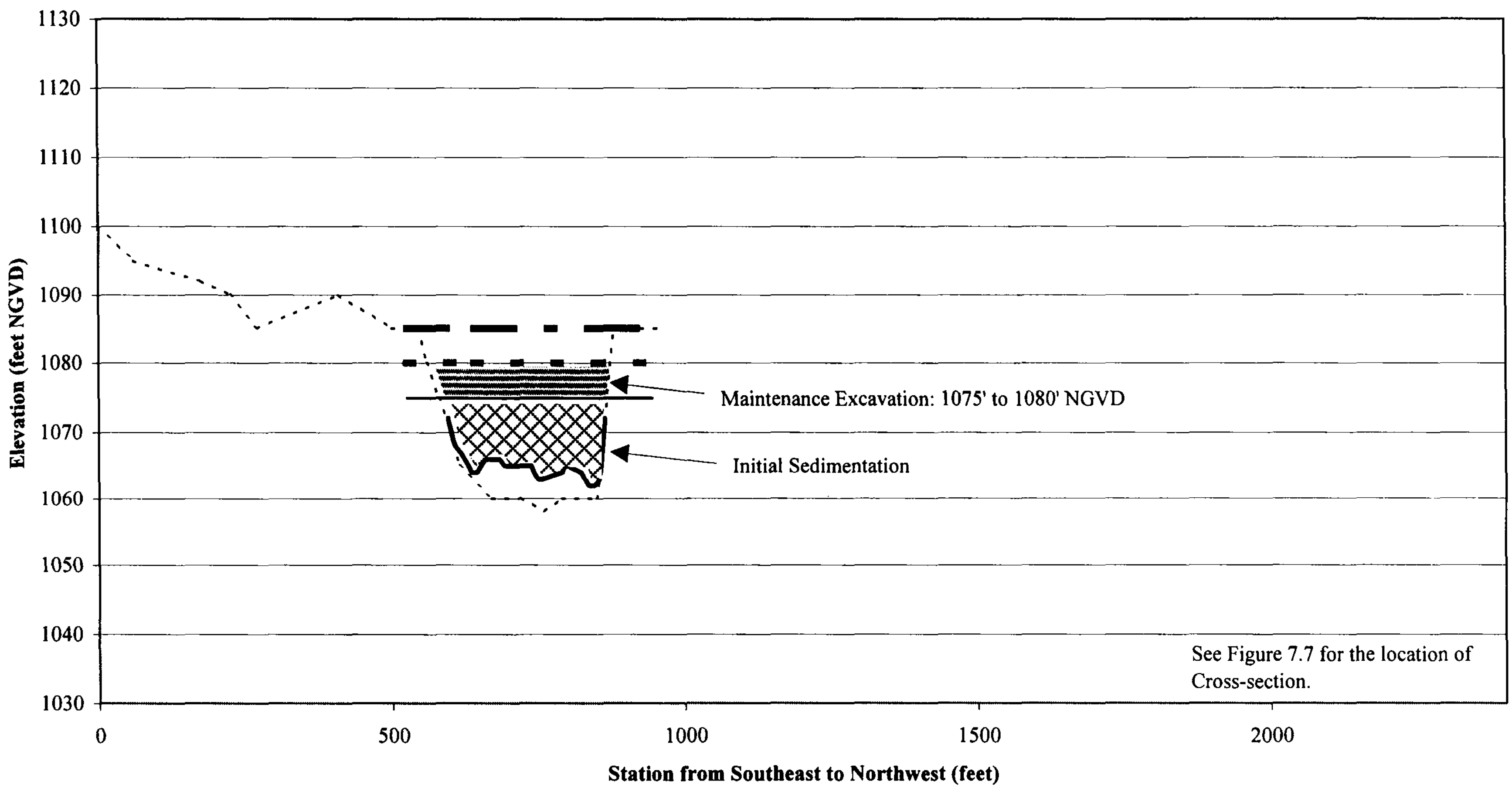
Proposed Geometry for Riparian Reach Cross-Section #3

PWA

PWA #: 1310



Jan-99 Preferred Geometry Excavation Trigger Geometry City of Pasadena Flood Criterion	Source: PWA Survey. Notes: Preferred geometry is 1055 feet NGVD, excavation trigger elevation set at 1060 feet NGVD, 5 feet above preferred geometry. City of Pasadena Water & Power Department flood control criterion is 1070 feet NGVD.	*figure 7.31* **Proposed Geometry for Excavation Zone 2 Cross-Section #4** PWA — PWA #: 1310



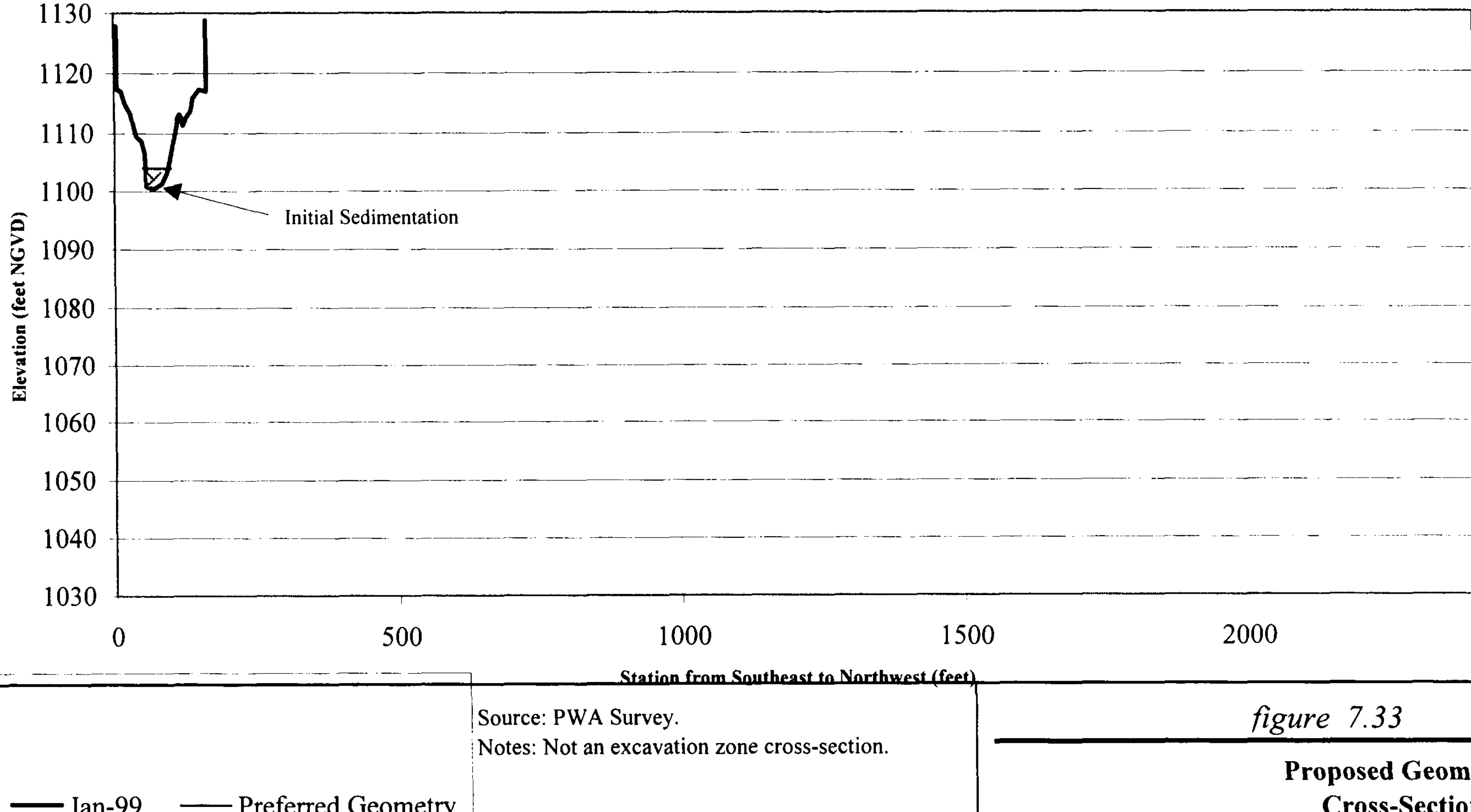
- Jul-81
- Nov-95
- Preferred Geometry
- Excavation Trigger Geometry
- City of Pasadena Flood Criterion

Source: LACDPW Topographic Data.
Notes: Preferred geometry is 1075 feet NGVD, excavation trigger elevation ste at 1080 feet NGVD, 5 feet above preferred geometry. City of Pasadena Water & Power Department flood control criterion is 1085 feet NGVD. 1981 geometry was plotted since 1995 data is limited in this area.

figure 7.32

Proposed Geometry for Excavation Zone 2 Cross-Section #5

PWA | PWA #: 1310



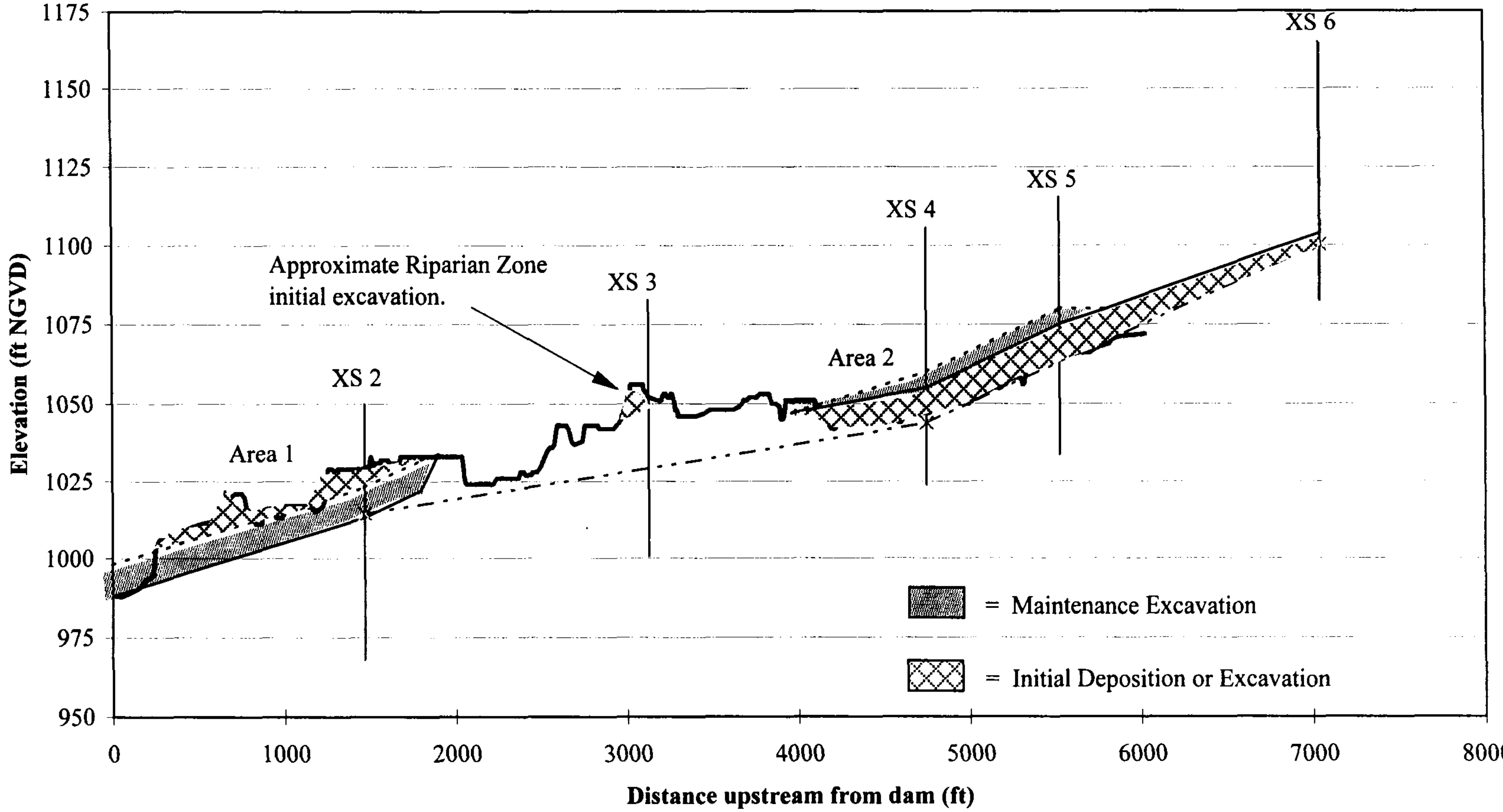
Source: PWA Survey.
Notes: Not an excavation zone cross-section.

figure 7.33

Proposed Geometry
Cross-Section #6
JPL Bridge

PWA

PWA #: 1310



KEY: Date of Topographic Map

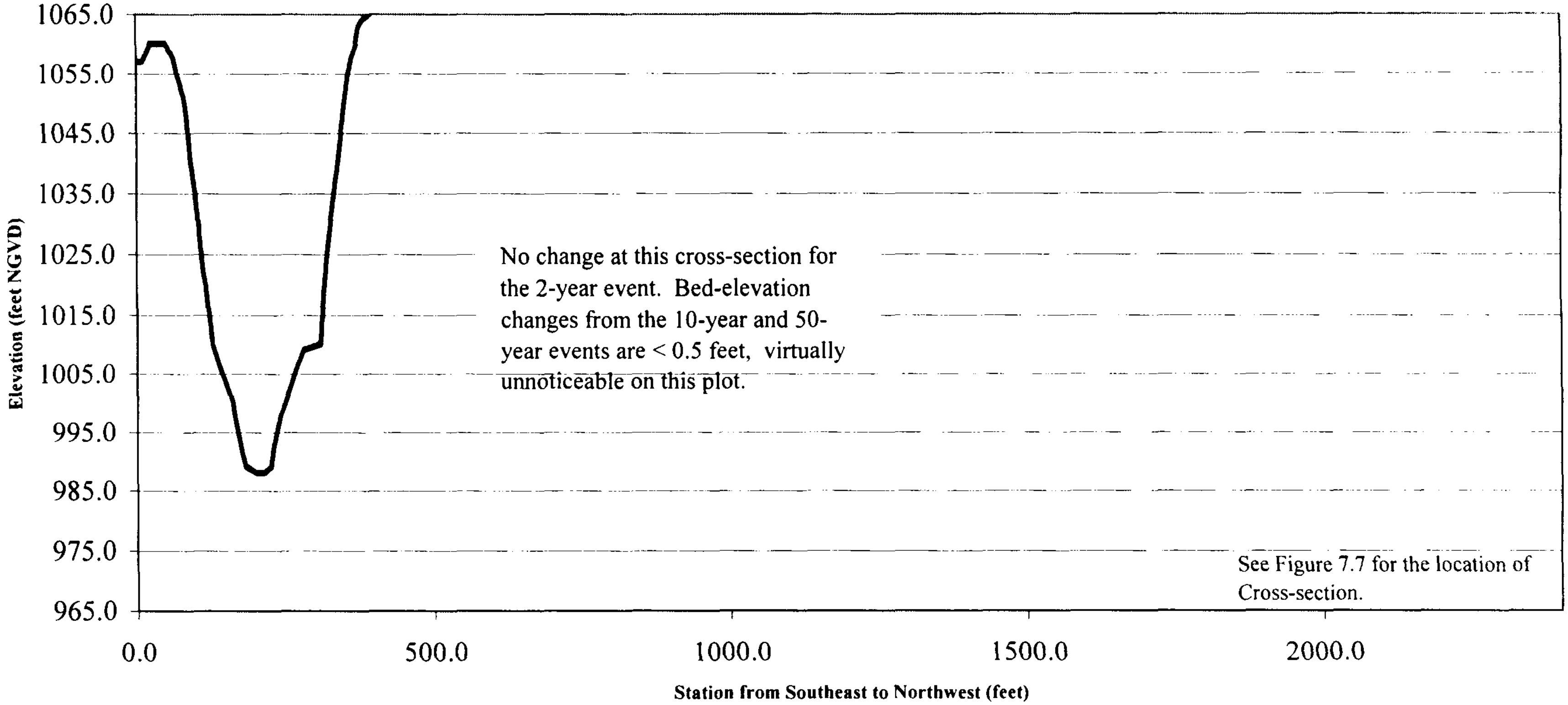
1995	January 1999
Area 1 Preferred	Area 2 Preferred
Area 1 Excavation Trigger	Area 2 Excavation Trigger

NOTE: Profile was taken along a linear transect established from the dam and extending upstream to the upper limit of the reservoir basin, rather than along the existing stream channel.
Vertical lines indicate intersections of this profile with selected cross sections (XS).

figure 7.34

Proposed Geometry
Longitudinal Profiles
Hahamongna Watershed Park

PWA 1310 Hahamongna Watershed Park



Proposed Geometry

10-year Event

50-year (Capital) Event

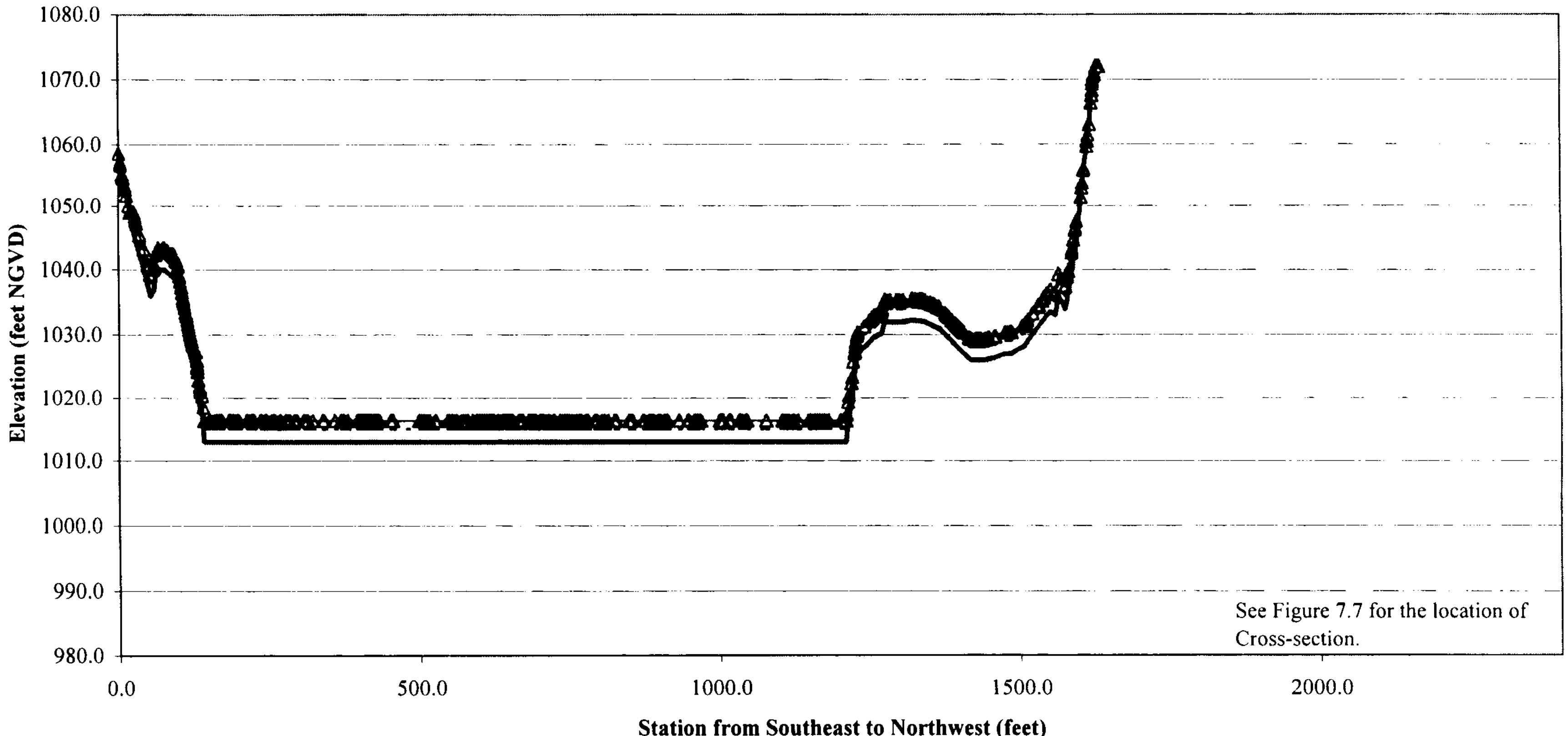
Source: LACDPW Topographic Data, PWA anlysis.

figure 7.35

Proposed Conditions Sediment Transport Model Results, Cross-Section #1 Devil's Gate Dam Face

PWA

PWA #: 1310



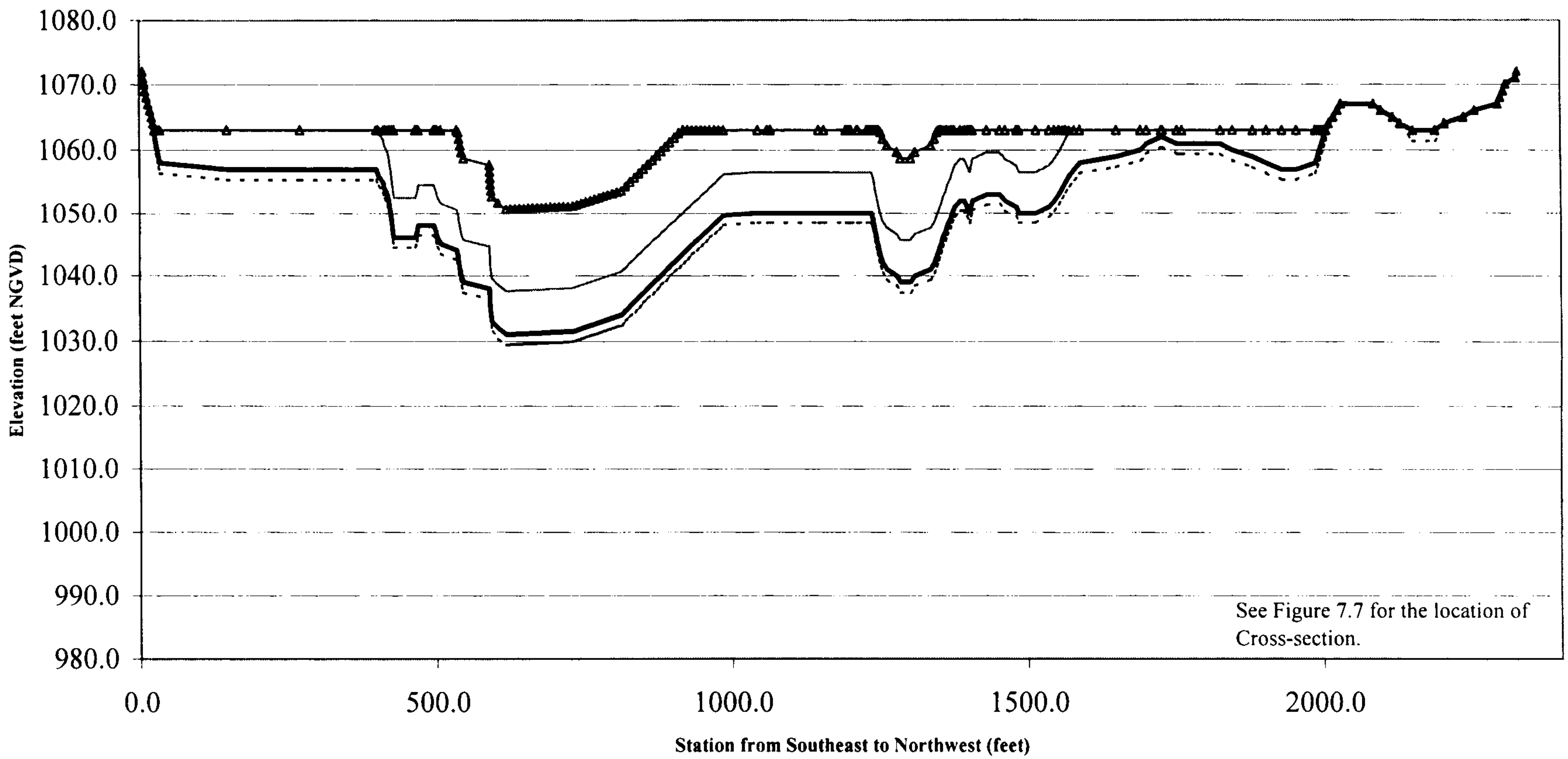
Source: LACDPW Topographic Data, PWA analysis.

figure 7.36

Proposed Conditions Sediment Transport Model Results, Cross-Section #2

PWA

PWA #: 1310



Proposed Geometry

2-year Event

10-year Event

50-year (Capital) Event

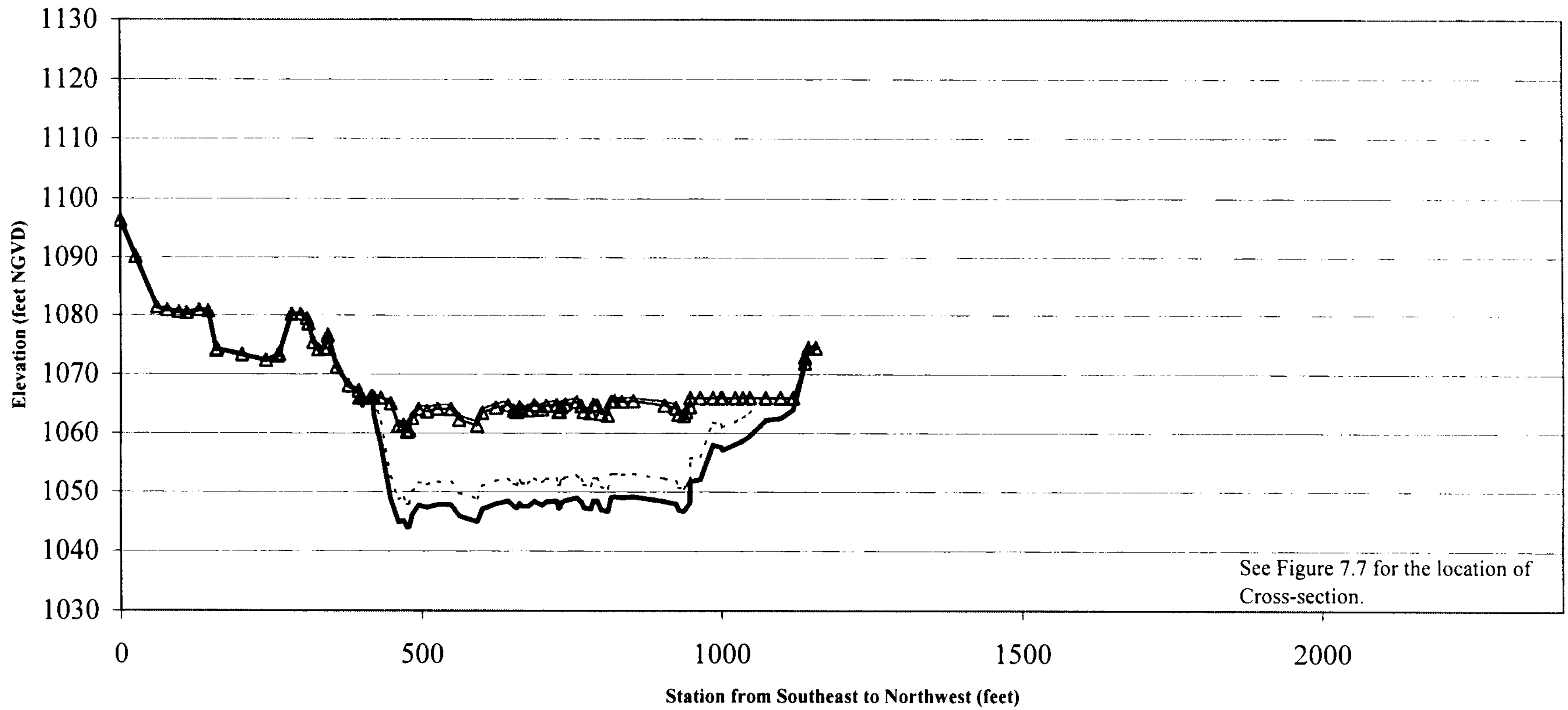
Source: LACDPW Topographic Data
Notes: Proposed cross-section geometry from Takata Associates.

figure 7.37

Proposed Conditions Sediment Transport Model Results, Cross-Section #3

PWA

PWA #: 1310



Proposed Geometry
2-year Event
10-year Event
50-year (Capital) Event

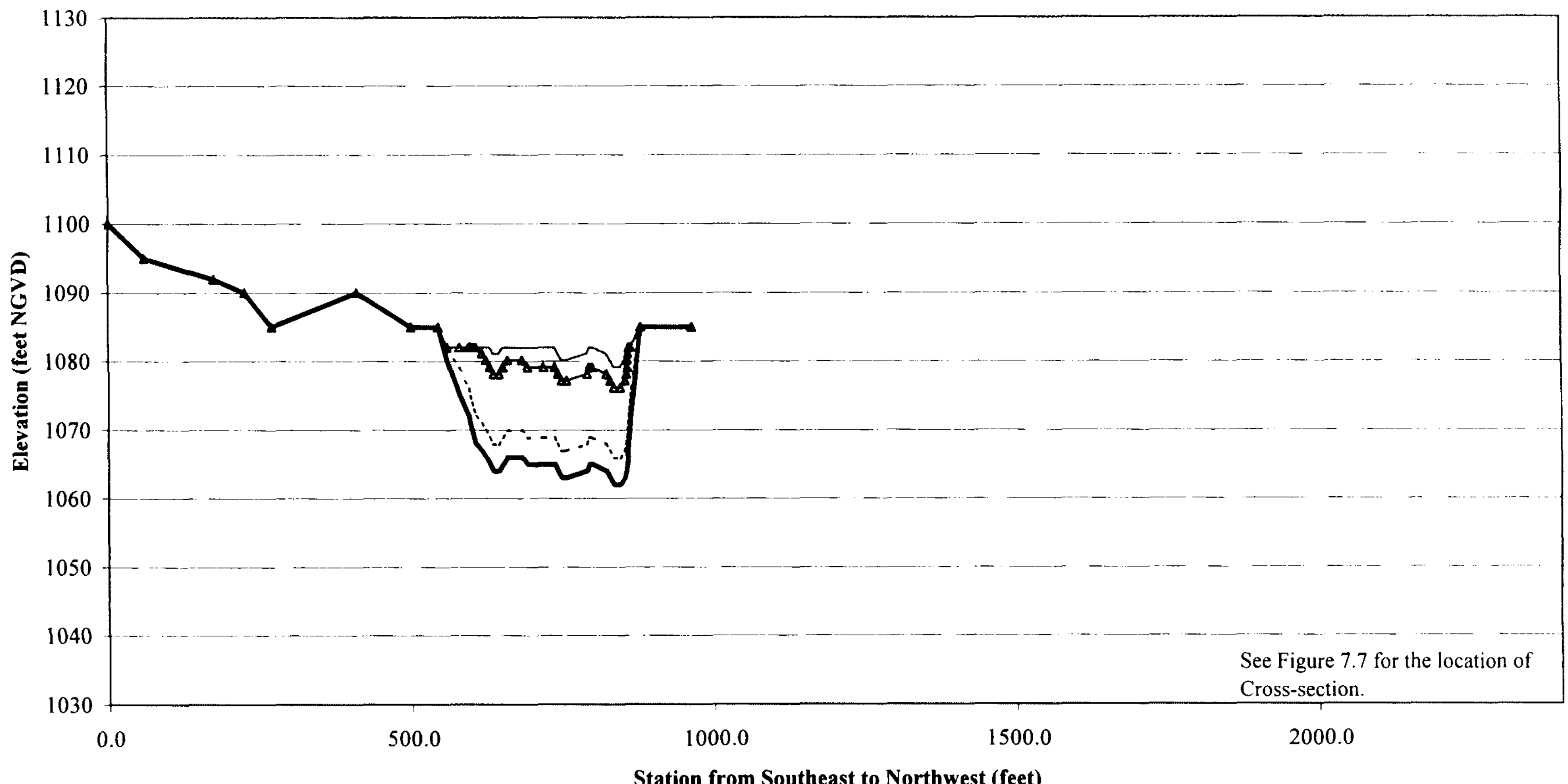
Source: PWA Survey, PWA analysis

figure 7.38

Proposed Conditions Sediment Transport Model Results, Cross-Section #4

PWA

PWA #: 1310



Proposed Geometry

2-year Event

10-year Event

50-year (Capital) Event

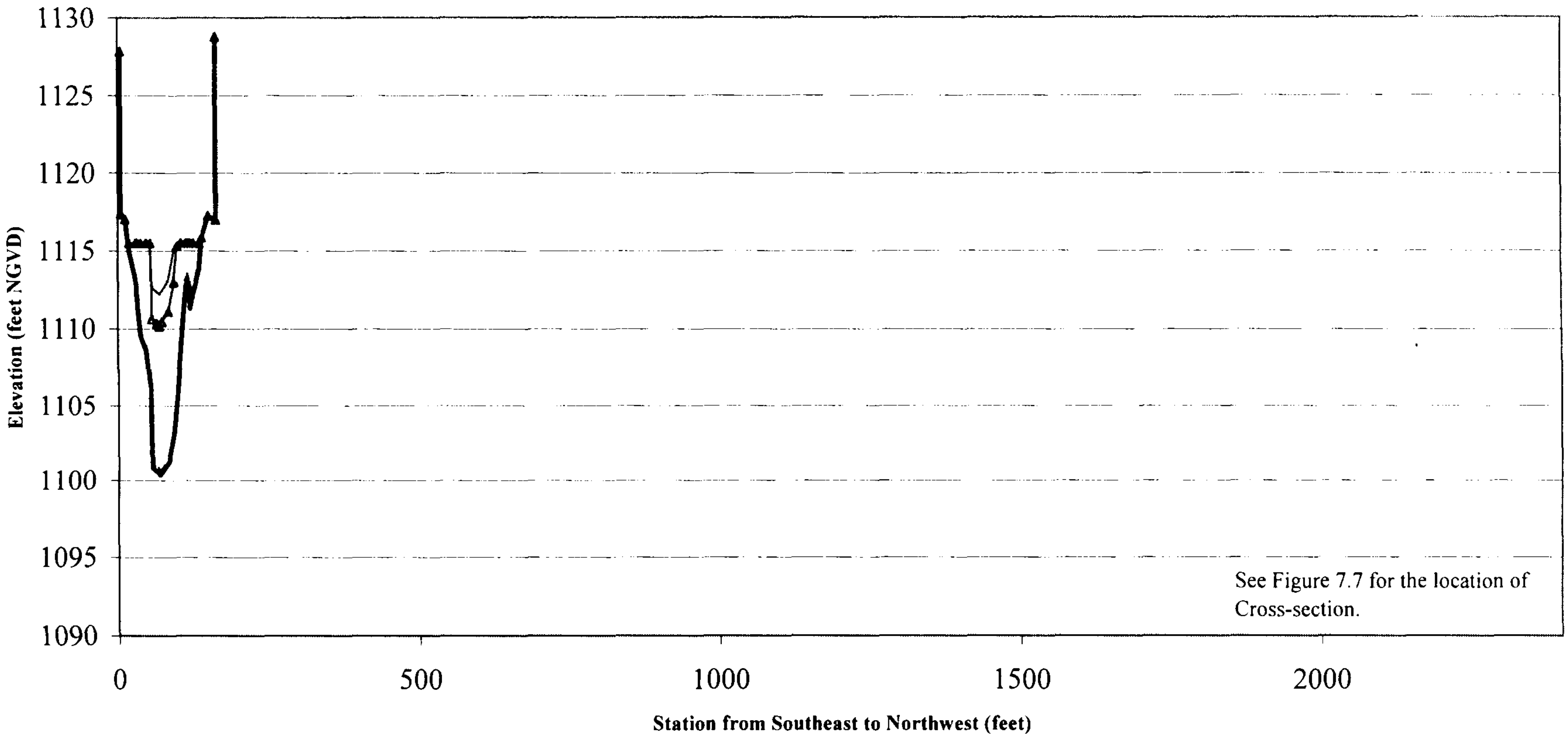
Source: LACDPW Topographic Data, PWA analysis.

figure 7.39

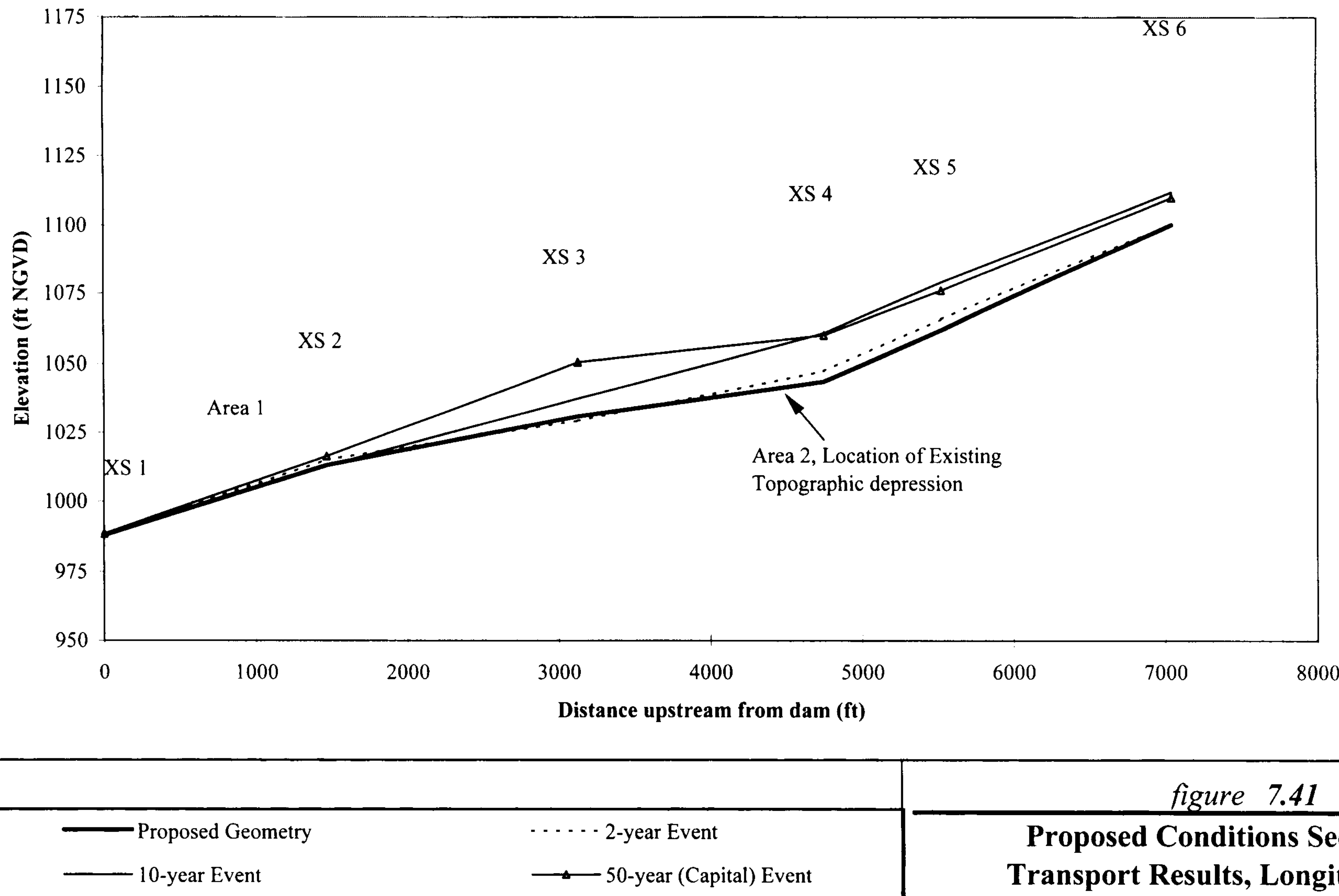
Proposed Conditions Sediment Transport Model Results, Cross-Section #5

PWA

PWA #: 1310



Proposed Geometry 2-year Event 10-year Event 50-year (Capital) Event	Source: PWA Survey, PWA analysis.	*figure 7.40* **Proposed Conditions Sediment Transport Model Results, Cross-Section #6 JPL Bridge** PWA — PWA #: 1310



NOTE: Profile is along the thalweg (low-point) of each cross-section.

figure 7.41

Proposed Conditions Sediment Transport Results, Longitudinal Profiles Hahamongna Watershed Park

PWA 1310 Hahamongna Watershed Park

TABLE 7.6 **Bed Elevation Changes Following Modeled Flood Events, Proposed Park Conditions**

Cross-section Number	Cross-section Location (Station)	Bed-Elevation Changes During Modeled Flood Event (feet)		
		2-year Event	10-year Event	50-year (Capital) Event
1	127+95	0.0	0.3	0.5
2	123+50	2.0	-0.3	3.3
3	118+50	-1.6	6.6	19.7
4	113+60	3.9	17.1	16.4
5	111+14	3.9	17.1	14.1
6	106+57	0.3	11.8	9.8

Source: PWA analysis.

Results of the proposed conditions sediment transport modeling show trends in bed-elevation changes that are similar to existing conditions simulations. The majority of deposition occurs in the upper area of the park, from the JPL bridge (cross-section #6) to cross-section #3. For the smaller events (2-year and 10-year) this deposition may be attributed to rapid channel expansion and the topographic depression in Area 2 (see the 1995 profile, near cross-sections 4 and 5, in Figure 7.8). For the 50-year (Capital) event deposition in the upper reach may be attributed to the location of the ponded interface in this zone during peak flows. As in the existing conditions simulations, some erosion was observed at cross-section #3 for the 2-year event, and at cross-section #2 for the 10-year event, attributable to the upstream loss of sediment load in Area 2. The one significant difference between existing conditions and proposed conditions results is that more sedimentation occurred close to the dam under proposed conditions. Especially for the 2-year and 50-year (Capital) events there seems to be a general shift of sedimentation from upstream to downstream. Both the 10-year and the 50-year (Capital) events showed a small amount of deposition right at the dam face, while the 2-year and 50-year (Capital) events showed 2.0 to 3.0 feet of deposition at cross-section #2. For all three events modeled, this deposition likely occurred due to an increased channel slope in the vicinity of the dam. The proposed initial excavation in Area 1 would increase the channel slope at cross-sections #1 and #2 to approximately 2% from its flatter existing condition. This increased slope would cause smaller events, such as the 2-year and 10-year events, to flow with more stream power in this zone, thus transporting more sediment. While the 50-year (Capital) flood still does not move sediment close to the dam during peak flows (due to an upstream ponded interface), the increased channel slope near the dam allows the smaller preliminary peak flows in this event to transport sediment. These smaller peaks prior to the main peak redistribute sediment near the dam. Increased deposition near the dam may also be partially attributable to increased storage capacity near the dam, which causes water levels and the ponded interface to rise more slowly in the park. Therefore, especially for the smaller events and the preliminary peaks in the 50-year (Capital) event, we would expect this interface to be located farther downstream than under existing conditions, allowing sediment transport farther downstream. The effect of this increased capacity is likely insignificant during the higher flows of the 50-year (Capital) event.

Modeled bed-elevation changes for preferred conditions geometry are shown in Table 7.7.

TABLE 7.7 Bed Elevation Changes Following Modeled Flood Events, Preferred Park Conditions

Cross-section Number	Cross-section Location (Station)	Bed-Elevation Changes During Modeled Flood Event (feet)		
		2-year Event	10-year Event	50-year (Capital) Event
1	127+95	0.0	0.0	0.0
2	123+50	0.0	0.0	0.0
3	118+50	0.3	3.6	18.0
4	113+60	6.2	15.1	25.6
5	111+14	2.6	9.0	20.7
6	106+57	8.9	3.3	15.7

Source: PWA analysis.

Results of the preferred conditions sediment transport modeling show trends in bed-elevation changes that are consistent with the other simulations conducted in this study. Significant deposition continues to occur in the upper portion of the park, for previously described reasons. Many of the physical phenomena described for the proposed conditions modeling are also true for the preferred conditions model. However, there are some important differences between the two simulations. The first difference is that there seems to be a damping of bed-elevation changes exhibited in the preferred conditions model for the smaller floods. Similar trends are evident in both proposed and preferred simulations, but the preferred simulation displays a more muted version of the trends for the 2-year and 10-year floods. This is likely attributable to the more uniform channel slope of approximately 2% in the preferred conditions modeling. This slope may be closer to an equilibrium channel slope for Arroyo Seco (as described in Section 7.3.3.3) and may therefore be more geomorphically stable for smaller events than the channel slope modeled under proposed conditions. This is especially true for the upper areas of the park, near cross-sections #4, #5, and #6, where the two geometries are significantly different.

The second difference is that the 50-year (Capital) event is shown to deposit significantly more sediment at cross-sections #4 through #6 under preferred conditions than under proposed conditions. This may be attributed to the reduced slope, and reduced stream power, under preferred conditions in this reach of the channel. The third difference is that near the dam, the preferred conditions simulation does not show any deposition adjacent to the dam, at cross-sections #1 or #2, as proposed conditions modeling did. The difference is actually quite minimal since only 0.5 feet of deposition or less were observed at the dam under proposed conditions, and is likely attributable to the overall muted sediment transport regime of the preferred condition simulations, and consequently lower sediment loads transported from upstream.

Overall, proposed and preferred conditions model results suggest that the proposed excavation zones (Areas 1 and 2) and the proposed riparian reach between them would function generally as desired. Initially, Area 2 and upstream to the JPL bridge (in the vicinity of cross-sections #4, #5, and #6), would likely experience sedimentation during significant flood events. Over time, deposition in Area 2 would likely decrease during smaller flood events as channel slope is brought closer to a potential equilibrium condition of 2% with rising local bed elevations. However, Area 2 will remain a significant area of deposition and will require maintenance as specified, especially during large events.

Results indicate that the proposed riparian reach downstream of Area 2 may see some erosion and downcutting during smaller events. Results indicate that the riparian reach may not be as significant an area for deposition during small to medium sized events; riparian habitat in this reach would therefore likely have longer periods without disturbance than in other areas of the park. However, during large flood events, such as the 50-year (Capital) flood, the riparian reach will experience significant deposition.

Near the dam, in Area 1, results showed less than 1.0 feet of deposition occurring during all events simulated. In reality, more sedimentation may occur near the dam than proposed conditions modeling indicates. The 1934 to 1938 calibration simulation indicates that this may be especially true as multiple smaller flood events redistribute sediments from the upstream areas downstream to the dam. Historical topographic data strongly supports Area 1 as a primary excavation zone.

7.3.3.5 *Excavation Quantities & Frequencies*

To achieve the preferred design elevations recommended in Section 7.3.3.3 for Area 1 will require an initial excavation of approximately 310 acre-feet (500,000 cubic yards) of material from within Area 1. Area 2 is currently below the preferred grade recommended in Section 7.3.3.3 and should therefore be allowed to aggrade to preferred geometry.

Table 7.8 reports estimates of two different sets of volumes for Areas 1 and 2. The first set of volumes is the storage capacities below elevations 1020 feet NGVD and 1040.5 feet NGVD for each area. The second set of volumes it reports is the potential maintenance excavation volumes for the two areas—that is the volumes contained between the excavation trigger elevations and the preferred geometry elevations. Volumes were approximated based on rough average-end-area-method calculations.

TABLE 7.8 Storage Capacities and Potential Maintenance Excavation Volumes for Areas 1 and 2

Quantity Description	Volume (acre-feet)	
	Area 1	Area 2
Storage Capacity (Below 1020 feet NGVD)	390	0*
Storage Capacity (Below 1040.5 feet NGVD)	1300	0*
Potential Maintenance Excavation Volumes	260	90

*Note: Virtually all of Area 2 is currently above elevation 1040.5 feet NGVD and would remain so under preferred geometry conditions.

Source: PWA analysis.

As discussed in Section 7.3.3.3, under preferred geometry conditions the excavation areas would provide adequate debris event storage capacity (exceeding LACDPW's requirement of 1200 acre-feet), and reservoir flood storage capacity below 1040.5 feet NGVD would be increased by the volume of the initial excavation from Area 1, approximately 310 acre-feet. Therefore, LACDPW's desire to maintain reservoir flood storage capacity at 1995 levels would generally be fulfilled by the proposed sediment management plan.

However, there is one possible situation where overall reservoir flood storage capacity (below elevation 1040.5 feet NGVD) could be less than the 1995 level with the proposed plan in effect. Because the preferred geometries of Areas 1 and 2 have a combined storage capacity of approximately 1300 acre-feet below elevation 1040.5 feet NGVD—which is less than the overall 1995 reservoir capacity of 1424 acre-feet below elevation 1040.5 feet NGVD by approximately 124 acre-feet—and because the proposed plan would limit ordinary LACDPW maintenance excavation to Areas 1 and 2, it is conceivable that a situation could arise where sediment had accumulated in areas outside of Areas 1 and 2 (e.g., in the proposed riparian zone) to the point where reservoir capacity dipped below the 1995 level. This could occur even if Areas 1 and 2 were graded to their preferred geometry. This situation would arise only after approximately 310 acre-feet of sedimentation occurred in areas that are both outside of the maintenance areas and lower than 1040.5 feet NGVD under existing conditions. Essentially this amounts to the point when 310 acre-feet of sedimentation has occurred in the proposed riparian reach. This quantity of sedimentation is equivalent to the initial excavation quantity for Area 1. Initially excavating 310 acre-feet from Area 1 and then allowing 310 acre-feet of sedimentation in the riparian zone would bring the overall reservoir capacity to its existing level (assumed to approximate 1995 levels). Any sedimentation in the riparian zone after this point would impinge upon LACDPW's capacity requirements. At this point excavation in the riparian zone would likely be required to maintain overall flood storage capacity in the reservoir.

Although this situation is certainly possible it seems highly unlikely that it would occur given that 310 acre-feet of sediment spread over the approximately 30 acres of the riparian zone amounts to approximately 10 feet of sedimentation. It is likely that the City of Pasadena would require excavation in the riparian zone for habitat reasons before an accumulation of 10 feet of sediment had occurred. If a single event deposited 10 feet of sediment in the riparian zone (a distinct possibility during a large storm event) both the City and LACDPW would likely agree that maintenance excavation would be required in this area. However, this decision would depend on the excavation trigger depth chosen by the City (discussed in Section 7.3.3.3). The responsibility for excavation costs and the conditions under which excavation would occur in areas outside of Areas 1 and 2 should be negotiated by the City and LACDPW during later design phases if they decide to proceed with this maintenance plan.

Although the goal in developing this sediment maintenance plan was not to prescribe the frequency of maintenance excavation but to propose excavation on an performance basis, it is useful to have a general sense of potential excavation frequencies for each of the proposed excavation zones, Areas 1 and 2, and also for the riparian reach (assuming conditions noted above). A detailed calculation of sediment delivery probabilities and resulting excavation frequencies for the three areas is beyond the scope of this study and is likely unmerited given the amount of uncertainty associated with sediment delivery to the park. However, some useful general information can be derived by looking more closely at the volumes of the two excavation zones and the long-term average annual sediment deposition within the park.

Long-term average annual sediment delivery to the park was calculated to be approximately 90 acre-feet (Section 7.1.3.1). Assuming that 20% of the sediment delivered to the park is transported through Devil's Gate Dam, an historically reasonable estimate (Sections 7.3.2 and 4.10.4), and that 10% of deposited sediment is deposited outside the excavation zones (in the riparian reach), long-term average annual deposition in the excavation areas would be 65 acre-feet, and long-term average annual deposition in the riparian reach would be 7 acre-feet.

If, for simplicity, we assume an equal probability of sediment deposition for each of the two excavation zones the time period between required excavations would be approximately 8 years for Area 1 and 3 years for Area 2. Since the probability of deposition for each excavation zone is difficult to calculate and is not likely the same for both areas, it may be more reasonable to calculate the period between excavations for the total sediment storage in both excavation zones. This time would be approximately 5 years. The approximate frequency of excavation for the riparian zone would be 44 years, assuming that it would only be excavated after 310 acre-feet of sedimentation had occurred.

While these numbers may be useful to gain a general long-term understanding of excavation frequencies and quantities, it should be noted that actual sediment accumulation and resulting excavation requirements will vary widely from these calculations. Sediment deposition within the basin tends to occur during large infrequent events rather than in steady annual amounts.

7.3.3.6 *Excavated Sediment Composition*

PWA's MIKE 11 sediment transport model formulation did not calculate the movement of particular sediment grain-sizes and therefore did not yield information on the distribution of various grain-sizes throughout the park during flood events. However, our field sampling gives a clear picture of the types of sediments that can be expected to deposit in various parts of the park (Section 7.2.2.3). In general we observed a trend from larger grain-sizes deposited in the upper areas of the park to smaller, finer grain-sizes deposited in the lower areas of the park. From our field data we can also estimate the approximate grain-size characteristics of sediments that will tend to deposit in the two excavation zones. In Area 1 we would expect fine sediments with a D_{50} (median diameter) of approximately 0.5 mm to 6.0 mm. In Area 2 we would expect coarser sediments with a D_{50} of approximately 15 mm to 50 mm. The smallest grain-sizes in the park appear to be in the silt size category. PWA found no evidence of clay in the park.

7.3.4 Sediment Maintenance Strategy Implementation

7.3.4.1 *Summary of Potential Environmental Impacts*

If it is possible to achieve regulatory permits that will allow un-mitigated excavation within the excavation areas proposed (Areas 1 and 2) habitat removal impacts should be limited to the rare case when maintenance is required outside of Areas 1 and 2 in vegetated zones (Section 7.3.3.5). Excavation should be primarily limited to areas that are permitted for regular disturbance, leaving the primary riparian habitat zones within the park untouched. Therefore, riparian habitat destruction should not be a major environmental issue.

However, if permits allowing this type of excavation practice within Areas 1 and 2 are not achieved, environmental impacts resulting from the proposed sediment maintenance strategy may be significant. The most significant impacts could be due to the initial implementation of the proposed excavation zones, particularly Area 1. If Area 1 is implemented it will require significant removal of existing vegetation. It is likely that mitigation for this removed vegetation will be required by the regulating agencies. The implementation of Area 2—that is, conforming ground elevations to the preferred geometry—would not have significant environmental impacts. Although this area currently has sensitive Coastal Sage Scrub habitat in it, under the proposed plan approximately 5 to 7 feet of sedimentation would be allowed to occur to bring the elevations of the area up to the preferred geometry. Since this type of "implementation" would occur naturally, no human impact would be made to this habitat.

Once implemented, environmental impacts should be minimal during routine maintenance excavation of Area 1. Any vegetation removal required in Area 1 should be fully permitted by the regulatory agencies (see Section 7.3.4.2). If routine vegetation removal from Area 1 is not permitted a prescribed annual or biennial maintenance schedule would likely be pursued (Section 7.3.3.2) that would preclude vegetation growth, thereby eliminating this kind of environmental impacts.

The first routine maintenance excavation in Area 2 could have significant environmental impacts associated with it depending on how the 5 to 7 feet of sedimentation occurred in the area to bring elevations up to the preferred and ultimately the excavation trigger elevations. This sedimentation could potentially occur two ways, each of which would result in a different level of environmental impact for the first routine clean-out. The first way sedimentation could occur is during one or two storm events with return intervals of between 2 and 10 years (as shown by the proposed conditions sediment transport modeling results in Figure 7.38). If sedimentation occurred as a result of one or two events of this size, sensitive habitat would likely be buried and not survive. Thus the sensitive habitat zone would no longer be present at the time of excavation and maintenance would have no environmental impact to the habitat. If the proposed sedimentation occurs in Area 2 as a result of multiple smaller events over a longer period of time vegetation could grow and adjust to the rising ground elevation. In this case, the sensitive habitat could still be present once the trigger elevations were achieved in Area 2. In this case the first routine maintenance excavation in Area 2 could require mitigation for removal of this sensitive habitat. Following this first maintenance excavation in Area 2, agencies would either permit future un-mitigated habitat removal or prescribed annual or biennial excavation would be practiced, precluding future vegetation and therefore habitat impacts.

If excavation is limited to the slopes and depths prescribed in the proposed sediment maintenance strategy, excavation should not effect significant geomorphic impacts to the stream channel. Regular excavation will increase groundwater recharge rates in the designated areas, an overall benefit to the groundwater hydrology of the region.

If flow-assisted sediment transport is greatly increased from current practice there could be increased sediment transport in Arroyo Seco downstream of Devil's Gate Dam. This could lead to flood channel sedimentation in some cases. Since the majority of the flood conveyance system is concrete channel, sedimentation could be a significant environmental impact if flow-assisted transport increases greatly. There may be some limited noise impacts during excavation periods, however this should not be a significant change from the status quo. There will be increased traffic on the roads and highways used by the excavation equipment and sediment removal trucks. However, haul roads and freeway access would be such that traffic increases in residential areas should be limited.

7.3.4.2 *Regulatory Issues*

As discussed in Section 7.3.4.1, the key regulatory issue associated with the sediment maintenance strategy will likely be vegetation removal during excavation. It is crucial that permits be negotiated with the regulatory agencies to achieve the best agreement for the City and LACDPW. Ideally, permits would allow un-mitigated implementation and maintenance excavation within Areas 1 and 2 with the understanding that the overall implementation of the Hahamongna Watershed Park will provide significant net habitat benefits outside of those two excavation zones, and that habitat could be allowed to regenerate in the excavation zones between maintenance events.

While mitigation for initial vegetation removal during implementation may be unavoidable (Section 7.3.4.1), if regulatory agencies do not permit un-mitigated maintenance of Areas 1 and 2, prescribed annual or biennial maintenance could be practiced, which would preclude vegetation growth between maintenance cycles in the excavation zones. The potential for this type of maintenance program, where no vegetation would ever be present in the excavation zones, could be used as leverage in negotiations with regulatory agencies. Their choice would be between un-mitigated maintenance excavation that allows some vegetation to be present in Areas 1 and 2 between maintenance cycles, or a mitigation requirement that would essentially preclude vegetation growth in Areas 1 and 2 at all times.

In addition to addressing these habitat issues, it is very important that traffic and access impacts are clearly explained up-front in the planning process and that agreement is reached with regulatory agencies and local stake-holders over these issues. Permit documents should clearly establish the right to equipment access and traffic increases agreed upon in the planning phase of the project.

7.3.4.3 *Estimated Costs*

There are two main costs associate with implementing and operating the proposed sediment maintenance strategy. Both are excavation costs. The first is the cost of initial excavation to preferred elevations in Area 1. The second is the cost of ongoing excavation according to the proposed strategy. The quantity of the initial excavation from Area 1 would be approximately 310 acre-feet (675,000 tons). Over the long-term, the average annual excavation quantity should approximate the average annual sediment deposited in the basin, approximately 90 acre-feet (197,000 tons). Applying a unit cost of $5.42/ton—the equivalent 1999 value of the average bid for excavation from Devil's Gate Reservoir in 1993 by the LACDPW—the total initial cost of excavation may be estimated at $3,700,000 and the long-term annual excavation costs may be estimated at $1,100,000.

8. WATER FEATURE FEASIBILITY

During the planning process for the Hahamongna Watershed Park the desire for a permanent or seasonal water feature within the park has been repeatedly expressed by stake-holders. As part of this study PWA qualitatively assessed the feasibility of creating and maintaining a water feature in the park. This section describes several of the alternatives PWA examined and our assessment of the feasibility of each. This was a reconnaissance level assessment only; further detailed analysis and design would be required if a water feature were to be implemented as part of the park design.

8.1 TEMPORARY SEASONAL WATER FEATURE NEAR DEVIL'S GATE DAM

One alternative considered in PWA's feasibility analysis was a water feature located directly upstream of Devil's Gate Dam. The concept is essentially that the City of Pasadena would halt water diversions from the Arroyo Seco watershed and allow streamflow into the park through the creek channel. When adequate runoff was available seasonal temporary ponding would be pursued at Devil's Gate Dam. This pool could serve as an enlarged groundwater recharge/percolation pond, potentially enhancing groundwater recharge in the park. The pool area could be drained as needed to also serve as a primary excavation site in the basin, allowing LACDPW to maintain active storage volume in the reservoir and also maintaining sediment hydraulic conductivity for percolation. Current dam operations could be modified slightly to allow this ponding, perhaps holding water after a significant flood event when ponded water is present. However, it is crucial that any minor modifications to dam operations do not compromise the primary flood control purpose of the facility. This water resource management strategy could potentially render the Arroyo Seco Spreading Grounds obsolete, opening the potential for additional natural habitat and recreational areas within the park. The following sections describe PWA's feasibility assessment of such a water feature in more detail.

8.1.1 Percolation

PWA did a preliminary evaluation of the amount of percolation that could be achieved by restoring natural flows to the Arroyo Seco channel downstream of the JPL bridge and seasonally allowing water to pool behind Devil's Gate Dam.

To evaluate stream-bed percolation rates PWA used data from a report on percolation along Tahquitz Creek near Palm Springs, California (Bookman and Edmonston, 1971). The authors of the report compiled data relating channel percolation rates in cfs/mile to channel discharge rates in cfs. Data was gathered for many different Southern California rivers including Big Santa Anita Wash, Big Tujunga Wash, Pacoima Wash, San Antonio Wash, and the San Gabriel and Santa Ana Rivers. The authors plotted the data to give an indication of the maximum percolation rate for various river discharges. This plot is shown in Appendix B and is summarized in Table 8.1.

TABLE 8.1 Maximum and Minimum Observed Channel Percolation Rates for Southern California Streams

Channel Discharge (cfs)	Channel Percolation Rate (cfs/mile)	
	Maximum Observed	Minimum Observed
10	7	5
20	12	7
25	13	8
40	18	9
60	24	10
80	29	11
100	33	12
200	50	15
400	70	19
600	80	20
1,000	100	22

Source: Bookman and Edmonston, 1971.

PWA also estimated stream-bed percolation rate at Arroyo Seco for flow observed on 18 January 1999 during a field visit. Average daily flow at the upstream USGS gage for this date was reported to be approximately 3.0 cfs (provisional data, subject to revision by USGS). Using a scaling ratio of watershed areas, discharge at the JPL bridge was estimated at 3.7 cfs. The City of Pasadena diverted all of this water to the existing percolation ponds. However, as described in Section 5.1.1, PWA observed leakage from one of the upstream-most percolation ponds back into the channel. This rate of leakage was quite high. Water leaking back into the channel continued to flow downstream. At a point approximately 2,000 feet downstream of the leak flow disappeared, indicating that all flow had percolated into the substrate at that point. Assuming that the rate of leakage equaled half of the diversion rate of 3.7 cfs, the percolation rate along this reach of the Arroyo Seco channel would be approximately 5 cfs/mile. This generally agrees with the data compiled by Bookman and Edmonston, although the river discharge in Arroyo Seco for January 18, 1999 was lower than the range of discharges plotted.

The Green-Ampt infiltration method was used to estimate percolation of water held behind the dam. This method assumes that water percolates under steady-state saturated conditions. Since hydraulic conductivities are relatively high for the alluvial sediments in the reservoir, and since we would expect water to be ponded for durations greater than 1 to 5 days, this assumption of steady-state saturated infiltration is reasonable. Using this assumption generally causes infiltration estimates to be lower than if the unsaturated transient flow period is accounted for since infiltration rates are generally higher during the initial unsaturated transient period. Using the Green-Ampt method is therefore conservative for this situation. Neglecting the critical pressure-head (another conservative assumption since capillary rise is relatively small for alluvial substrate), the Green-Ampt equation is (Bouwer, 1978, p. 253):

$$Q = K * A * (H_w + L_f) / L_f$$

where Q is infiltration rate in cubic feet per second (cfs), K is saturated hydraulic conductivity in feet per second (fps), A is the ponded area in square feet (ft^2), H_w is the average depth of ponded water above the ground surface in feet, and L_f is the depth of the saturated substrate above the water table in feet.

To calculate values of L_f an average seasonal groundwater elevation of 950 feet NGVD was assumed, although infiltration results were not particularly sensitive to this parameter. Since the depth of the ponded area would vary between the maximum depth at the low point in front of the dam and a minimum of zero depth at the edges of the pond the midpoint between these values was used to estimate H_w. Ponded area, A, was estimated from the most recent LACPWD storage table.

Saturated hydraulic conductivity, K, was calibrated using an infiltration rate of 18 cfs for a ponded area of 15.1 acres at the Arroyo Seco Spreading Grounds (1.22 cfs/acre). These numbers were reported by LACPWD (LACPWD, 1998). For the Arroyo Seco Spreading Grounds, the calibrated K-value was calculated to be approximately $2.75x10^{-5}$ fps, or 44% of actual measurements of K for the spreading grounds, $6.2x10^{-5}$ fps (40 gpd/ft^2) (Converse Consultants West, 1995). Calibrated K-values were therefore assumed to be approximately 44% of measured values. Based on this calibration PWA calculated percolation rates for three different pond scenarios at the dam: 1) K equal to that of the spreading grounds, $2.75x10^{-5}$ fps; 2) K equal to twice the spreading grounds value, $5.50x10^{-5}$ fps; 3) K equal to 44% of the lowest measured value from well-tests conducted in the basin, $5.54x10^{-4}$ fps (the lowest measured K-value was assumed to be $1.25x10^{-3}$ fps or 806 gpd/ft $)^2$(Converse Consultants West, 1995). Results of the water infiltration calculations for water held behind the dam are shown in Table 8.2.

TABLE 8.2 Estimated Percolation Potential for Ponded Water Behind Devil's Gate Dam, Green-Ampt Method

Ponded Water Surface Elevation (feet NGVD)	Ponded Area (acres)	Ponded Water Volume (Storage) (acre-feet)	$K_1 = 2.75x10^{-5}$ fps (spreading grounds calibration)		$K_2 = 5.50x10^{-5}$ fps (2 x K_1)		$K_3 = 5.54x10^{-4}$ fps (lowest well-test calibration)	
			Q (cfs)	Q/A (cfs/acre)	Q (cfs)	Q/A (cfs/acre)	Q (cfs)	Q/A (cfs/acre)
990	0.2	0.2	0.2	1.2	0.4	2.5	4.0	24.8
995	1.5	2.9	1.9	1.3	3.8	2.6	38.2	26.2
1000	3.8	15.2	5.2	1.4	10.3	2.7	104	27.4
1005	6.5	41.0	9.2	1.4	18.4	2.8	186	28.5
1010	11.0	83.6	16.1	1.5	32.2	2.9	325	29.6
1015	18.7	157.1	28.2	1.5	56.4	3.0	568	30.5
1020	27.7	270.5	43.0	1.6	86.0	3.1	866	31.3
1025	41.6	438.9	66.2	1.6	132.4	3.2	1334	32.0
1030	56.9	685.1	92.4	1.6	184.7	3.3	1861	32.7
1035	74.0	1013.1	122.5	1.7	244.9	3.3	2467	33.4
1040	91.8	1423.9	154.7	1.7	309.4	3.4	3116	33.9

The results of this analysis of stream-bed percolation and percolation of water held behind Devil's Gate Dam indicate that significant increases in groundwater recharge might be realized if natural flows are restored to Arroyo Seco below the JPL bridge and if water is held behind Devil's Gate Dam. For a relatively low-flow situation of 25 cfs in Arroyo Seco, and assuming water is held behind Devil's Gate Dam at an elevation of 1010 feet NGVD, with a hydraulic conductivity of K_1, a combined percolation rate of between 24 cfs and 29 cfs might be realized for the basin. This indicates that all flow would percolate into the groundwater. This potential percolation rate, equivalent to between 133% and 161% of current City percolation rates, should be seen as a minimum. Higher flows in Arroyo Seco, higher elevations for water held behind Devil's Gate Dam, and higher hydraulic conductivities would all cause increases in percolation potential within the basin.

The idea that percolation rates within the basin can be greatly increased by holding water behind Devil's Gate Dam is supported by data from several recent groundwater studies. In CH2MHill's recent report, "Phase 2, First Technical Assessment, Devil's Gate Multi-use Project, Raymond Basin, California," (CH2MHill, 1992) it was observed that groundwater levels rose approximately 100 feet between 1965 and 1968 in the Monk Hill Groundwater Basin area, near Devil's Gate Dam. California Department of Water Resources (DWR) records showed that approximately 5,100 acre-feet were spread at the Arroyo Seco Spreading Grounds during that period, an inadequate volume to account for the 100-foot rise in groundwater table. In fact, modeling showed that an additional 30,000 acre-feet of infiltration would be required to elevate the groundwater table by 100 feet over that period. This additional infiltration equates to approximately 600% of DWR-recorded spreading. It was also observed that significant rainfall (>30 inches) occurred in 1965, 1966, and 1968. The study concluded that the additional 30,000 acre-feet of groundwater recharge likely resulted from water held behind Devil's Gate Dam. This implies that percolation potential for water held at Devil's Gate Dam may be approximately 600% of current City percolation rates.

A second study, "Hydrogeologic Investigation: Devil's Gate Water Collection Tunnel, Pasadena, CA," (Converse Consultants West, 1995) shows a strong correlation between periods when water is held in the reservoir and high percolation rates in the water-supply tunnels beneath Devil's Gate. Furthermore, this report hypothesizes that "if a lake is allowed to accumulate behind the dam, lake water...has a relatively long term effect on groundwater elevations below the Altadena Fan." (Converse Consultants West, 1995) This is further evidence that water held behind the dam may significantly increase percolation rates within the basin.

From their experience managing other reservoirs and deep pit spreading basins, the LACDPW has raised questions as to whether or not ponding behind Devil's Gate Dam would actually increase percolation rates to groundwater over the long-term. They have noted that percolation rates may be very high initially at such facilities but tend to decrease if water is stored for extended periods. This decrease is due to siltation sealing off infiltration paths through the bottom of the reservoir. They have also noted that while it is possible to control the level of sediment entering the existing Arroyo Seco Spreading Grounds by only diverting during times of relatively sediment-free flow, there is no way to control the level of sediment carried by flows that eventually pond at the dam. Therefore, infiltration rates tend to decrease more rapidly in situations such as ponding at Devil's Gate Dam.

In general, PWA agrees with these observations. It is unlikely that potentially high groundwater percolation rates could be maintained with ponding over extended periods of time. Therefore, temporarily holding water behind the dam following significant flood events may be the most reasonable method of increasing groundwater percolation in the basin. If, during wet season, water is held for periods of up to two weeks following a significant flood event, the benefits of initially high groundwater infiltration rates would likely be realized without the long-term clogging of pore-spaces associated with extended ponding. Furthermore, since sediment removal maintenance will likely be required from the area in front of the dam following wet seasons where significant sediment is deposited there (Section 7), the pore-clogging potential of this deposited sediment will be substantially reduced. Coupled with LACDPW's regular maintenance excavation activities in the area near the dam, PWA believes that a program of seasonal temporary ponding at Devil's Gate Dam could significantly increase the quantity of groundwater recharge to the Monk Hill aquifer.

Furthermore, it should be emphasized that if natural low flows were restored to the Arroyo Seco channel through Hahamongna Watershed Park—specifically flows up to 25 cfs that are currently diverted and spread by the City at the Arroyo Seco Spreading Grounds—data suggests that virtually all of these flows would naturally percolate into the groundwater along the riparian corridor without significant ponding or flow-through at the dam. This suggests that, together, restoring natural flows to the Arroyo Seco channel through the park and holding water temporarily behind the dam following flood events could provide groundwater recharge quantities that exceed current amounts, without the use of the Arroyo Seco Spreading Grounds.

8.1.2 Water Supply

If natural flows were restored to the Arroyo Seco channel through the Hahamongna Watershed Park and a temporary seasonal water feature were implemented behind Devil's Gate Dam, the main implication for water supply in the basin is that the City would likely be able to increase the amount of groundwater recharge credit they obtain from the Raymond Basin Management Board (RBMB). The RBMB strongly supports the concept of holding water at the dam more frequently (pers. comm. Ron Palmer, RBMB) as they believe this management strategy would bring increases to groundwater recharge in the Monk Hill aquifer. If the City obtained increased groundwater recharge credit, they would be allowed to pump more groundwater for municipal water supply than they currently do. Since groundwater is currently the most cost-effective source of water supply for the City, this could reduce City water supply costs somewhat.

The second implication for water supply is that the existing Arroyo Seco Spreading Grounds would no longer receive inflow and would be essentially obsolete. Therefore, the City would also likely need to establish a new method of accounting for percolation credit with the RBMB. A simple system incorporating flow-measurement and a calibrated rainfall-runoff model of areas draining directly to the reservoir could be put in place to more carefully track percolation within the basin. Since the RBMB is currently in the process of reevaluating their accounting procedure for percolation in the basin (due to the recent transfer of operation and maintenance responsibility for the spreading grounds from the LACPWD to the City Department of Water and Power), and since they desire to make this accounting procedure more scientifically rigorous, this may be an excellent time to suggest an alternative way of tracking City percolation credit.

8.1.3 Riparian Habitat

Restoring natural flows to the Arroyo Seco channel through the Hahamongna Watershed Park and allowing temporary seasonal ponding behind Devil's Gate Dam would have an effect on riparian habitat within the reservoir. Restoring natural flows to Arroyo Seco downstream of the JPL bridge would increase the amount of stream-flow and near-surface groundwater available for vegetation, enhancing the quality of riparian habitat adjacent to the river. A temporary seasonal water feature at Devil's Gate Dam would also raise groundwater elevations throughout the basin, further enhancing water availability for riparian vegetation. The area dedicated to natural riparian habitat could also be expanded greatly if the Arroyo Seco Spreading Grounds were eliminated.

The City has expressed concern that improved riparian habitat along the Arroyo resulting from flow restoration would preclude the City's option of using their water right to divert water from the stream again in the future. Once riparian habitat is established, re-institution of an upstream diversion would likely cause a significant adverse impact on this habitat. Therefore, regulatory agencies such as the U.S. Army Corps of Engineers would likely prevent the City from future flow diversion once flow restoration and riparian habitat was established. While this is likely true, if the City could obtain appropriate groundwater recharge credit from the RBMB for the in-stream use of their water right, there would not likely be a reason to re-institute their diversion in the future.

Under the proposed scenario, downstream riparian habitat would likely remain approximately unchanged from the existing condition. Temporarily holding water at the dam after significant flood events would delay outflow from the dam to downstream areas by the length of time that water is held and could reduce downstream flows by approximately the amount that is percolated into the groundwater. However, on balance, this would likely be a very minimal impact to downstream flows and riparian habitat. During most low-flow periods flows that would have percolated in the Arroyo Seco Spreading Grounds would percolate along the Arroyo Seco riparian zone through the Hahamongna Watershed Park with no effect on downstream habitat.

8.1.4 Recreation

Restoring natural flows to the Arroyo Seco channel through the Hahamongna Watershed Park and allowing temporary seasonal ponding behind Devil's Gate Dam would enhance recreational opportunities along a revitalized riparian corridor through the park. Riparian habitat could eventually provide a shady river-side area similar to that upstream of the JPL bridge. Recreational opportunities associated with a temporary seasonal water feature at the dam would be minimal due to LACPWD's concerns about liability and safety

(Section 8.1.1.6). However, the water feature at the dam would have some aesthetic value when seen from various locations around the basin. Due to the temporary seasonal nature of the ponding at the dam (taking place over perhaps two weeks following a significant flood event) recreational interests should understand that such a water feature is not permanent. Therefore, draining the water feature after two weeks should not inspire nearly the opposition from recreational interests that draining a more permanent water feature might.

8.1.5 Maintenance

If this alternative is implemented, the area in front of the dam where ponding would occur would also be a primary excavation area to remove excess sediment accumulation and maintain active flood storage in the reservoir (Section 7). Excavation would take place in the dry season and would be unaffected by the temporary seasonal ponding. In addition to maintaining flood storage, excavation would maintain the hydraulic conductivity of the substrate near the dam by removing the accumulation of fines at the ground-surface. This would help keep percolation rates high near the dam.

This alternative would not require additional infrastructure such as pumps or pipelines that might require intensive operation and maintenance efforts in the future. Furthermore, removing the existing Arroyo Seco Spreading Grounds would eliminate the need to maintain the structures associated with them.

The temporary seasonal ponding associated with this alternative would not affect flow-assisted sediment transport through Devil's Gate Dam since ponding would occur following storm events. The majority of flow-assisted sediment transport occurs during the preliminary stages of a flood event, when reservoir operations would be unchanged from existing conditions. Flow-assisted sediment transport associated with the draining of the reservoir following a flood event would simply be delayed approximately two weeks when the temporary pond was drained.

8.1.6 Flooding

Generally, the temporary seasonal ponding at the dam associated with this alternative would not impact the flood control function of the reservoir. Water would generally not be present behind the dam at the beginning of a significant flood event since water would only be held for approximately two weeks (the suggested temporary ponding time), after the passing of a flood event. The only exception to this would be if two large flood events occurred in series within the two-week holding period. If a second flood event occurred during the two-week holding period following a first flood event, flood control capacity in the reservoir could be compromised. Therefore, if this alternative is chosen PWA recommends that a conservative policy be implemented whereby the temporary seasonal water feature is drained at the first sign of significant rainfall during the two-week holding period. With this policy, flood control functions should be adequately maintained in the reservoir.

8.1.7 Liability and Safety

There would be very limited liability and safety risks associated with this alternative since the water feature at Devil's Gate Dam would be seasonal and temporary. The water feature would have no recreational benefits nor any of the liability and safety issues associated with recreational water features.

8.1.8 Recommended Further Study

This preliminary analysis shows that some significant benefits could be realized by restoring natural flows to the Arroyo Seco channel through the Hahamongna Watershed Park and allowing a seasonal temporary water feature at Devil's Gate Dam for approximately two weeks following significant flood events. However, the City's concerns over the establishment of riparian vegetation along the Arroyo due to restoring natural flows, and the potential for future flow diversion to be precluded as a result, likely mean that this alternative will be infeasible. If, in the future, this alternative becomes feasible, more detailed analysis of actual percolation potential should be pursued. This analysis should include a more detailed water balance of the basin. Using flow data from three of the past 20 years (an average rainfall year, a wet year, and a dry year), total annual percolation under the management regime suggested in this alternative should be modeled and compared with the actual amount of percolation achieved in each of those years at the Arroyo Seco Spreading Grounds. Surface water inflow and outflow from the basin, evapotranspiration, and reservoir storage changes would be used to model percolation rates for these three years.

A further step could be taken to analyze the feasibility of this alternative by actually testing the strategy without removing the Arroyo Seco Spreading Grounds. City diversions would be ceased and natural flows would be restored to the Arroyo downstream of the JPL bridge. The spreading grounds would be left dry for a year and water would be allowed to flow toward Devil's Gate Dam, percolating along the riparian corridor. Water would be held for approximately two weeks following any significant flood events. A detailed monitoring program could be put in place to observe actual percolation under this alternative. Results of this more detailed feasibility assessment would allow greater certainty in making a decision about this alternative.

8.2 ADDITIONAL WATER FEATURE ALTERNATIVES

In addition to the restoration of natural flows to the Arroyo Seco channel through the park coupled with a seasonal temporary water feature at Devil's Gate Dam, the feasibility of several other water feature alternatives was evaluated. Benefits and drawbacks to these alternatives are briefly discussed in this section. Implementation of any of these water feature alternatives would require further in-depth, quantitative analysis and design.

8.2.1 Recreational Water Features in Upstream Portions of the Park

The alternative of implementing several smaller recreational water features in the upstream portion of the park has been discussed during the design process. These water features would be more permanent and would accommodate recreational uses such as fishing and picnicking. This type of water feature would be beneficial in several ways. It would be relatively small and single-purpose and would therefore not be subject to significant liability issues. Beneficial aquatic habitat could also be established in a water feature like this. This habitat would not be subject to the hydrologic stress of frequently fluctuating water levels since the water feature would be relatively permanent. There are several drawbacks to this type of water feature. It would be somewhat artificial since this type of permanent pond is not part of the natural ecosystem of the Arroyo Seco. It would be relatively expensive to implement, requiring grading, construction of a pond liner, and some sort of water supply structure or system for filling and flushing. At a minimum the water feature would have to be located near the outlet of one of the existing spreading basins; potentially this type of water feature could require a new water conveyance system including a pump and pipeline. Maintenance costs would also likely be high for this type of water feature since it will require structures. Furthermore, this type of permanent water feature would not enhance groundwater percolation within the basin. Overall, this type of water feature is certainly feasible but it may only provide a limited range of benefits at a relatively high cost.

8.2.2 Expansion of the Arroyo Seco Spreading Grounds

PWA also evaluated the alternative of expanding the Arroyo Seco Spreading Grounds to achieve the City's desired increase in groundwater recharge. Several locations have been proposed including north of the existing ponds and on the west side of the park. The City has proposed expanding the spreading grounds from the 13 acres currently in operation to approximately 21 acres of wetted spreading area. The main benefit of additional spreading ponds within the park would be increased groundwater percolation, which would bring increased water supply revenue to the City. This would certainly be a substantial benefit to the City. However, there are several drawbacks to implementing additional spreading ponds. Additional ponds would reduce area available for achieving the recreational and natural habitat goals of the project. The habitat value of the ponds would be limited, as in the case of the existing ponds, since they would be intensively managed structures. City maintenance costs would increase since the ponds would require the same annual maintenance as the existing ponds. Although the percolation ponds would increase the amount of groundwater recharge credit the City could achieve with the RBMB, as discussed in Section 5.1.1 the Arroyo Seco Spreading Grounds may not be the most efficient means of recharging groundwater in the park area. The City could perhaps achieve the same or a greater recharge benefit by restoring natural flows to the Arroyo Seco channel in the park and supporting a policy of seasonally and temporarily holding water at Devil's Gate Dam, as discussed in Section 8.1.1.

8.2.3 Pumped-Back Water Management

A variation on the management of a seasonal temporary water feature at the dam, known as pumped-back water management, was also evaluated as part of this analysis. During the temporary period following significant flood events, when water was being held at the dam, portions of this held water would be pumped back to the spreading basins using a new conveyance system. This would have the advantage of spreading water in the Arroyo Seco Spreading Grounds that would otherwise be lost downstream when the reservoir was drained. Sediment-laden storm-water, currently not diverted to the spreading ponds for reasons of increased maintenance, could be allowed to drop its sediment load at the dam and then be recycled for percolation. Water would likely be pumped to the most northern spreading basin to maximize the amount of percolation achieved by this strategy. The primary drawback of this alternative is that significant new infrastructure would be required to convey water from Devil's Gate Dam to the north end of the percolation ponds, an approximate distance of one mile. This infrastructure would be expensive, and would require regular maintenance. This alternative would add infrastructure to an already highly-engineered area, contrary to the park goal of increasing natural habitat. Furthermore, the increased groundwater recharge benefit realized through pumping this water back could likely also be realized if the City could gain credit for groundwater infiltration occurring during temporary ponding periods at the dam, without the need for the extra infrastructure.

9. GLOSSARY OF TECHNICAL TERMS

Alluvial fan is an outspread, gently sloping mass of alluvium deposited by a stream, especially in an arid or semiarid region where a stream issues from a narrow canyon onto a plain or valley floor. Viewed from above, it has the shape of an open fan, the apex being at the valley mouth.

Bankfull stage is the elevation of the water surface of a stream flowing at channel capacity.

Bedload is the material moving on or near the streambed by rolling, sliding, and sometimes making brief excursions into the flow a few diameters above the bed.

Braided channel is a stream that divides into an interlacing network of branching and reuniting shallow channels separated from each other by islands or channel bars, resembling in plan the strands of a complex braid; especially an overloaded and aggrading stream flowing in a wide channel on a floodplain.

Cross-section is a diagram showing the topographic features transected by a vertical plane oriented perpendicular to a stream channel.

Debris Flow is a moving mass of rock fragments, soil, and mud, more than half of the particles being larger than sand size.

Denudation is the sum of the processes that result in the wearing away or the progressive lowering of the earth's surface by weathering, mass wasting, and transportation.

Down-cutting is the process by which a channel becomes deeper through erosion of its bed and banks.

Dry ravel is the downslope movement by gravity of individual grains or aggregates of soil along a hillslope.

Dynamic flood routing is a technique used in hydraulic modeling whereby the water-surface profile may be changing over time in response to variable flow rates. This technique is more physically realistic than "steady-state" flood routing, in which the water surface profile is constant over time and only one flow rate is modeled.

Evapotranspiration describes the total water removed from an area by transpiration (plant use) and by evaporation from soil, snow, and water surfaces.

Facies is a discrete bed material unit that includes bed material of similar characteristics (size, shape, etc.) over an area of the channel bed, used to differentiate it from adjacent or associated facies units.

Groundwater is that part of the subsurface water that is in the zone of saturation, including underground streams.

Hydrodynamics are the physical processes associated with the movement of water.

Knickpoint is any interruption or break in the slope of a channel; especially a point of abrupt change or inflection in the longitudinal profile of a stream or its valley.

Longitudinal profile is the profile of a stream or valley, drawn along its length from source to mouth; it is the straightened-out, upper edge of a vertical section that follows the winding stream or valley.

Longitudinal section is a diagram drawn on a vertical plane and parallel to the length of a valley.

Meandering stream is a mature stream winding freely on a broad floodplain.

Mudflow is a general term for a mass-movement process characterized by a flowing mass of fine-grained earth material with a high degree of fluidity. Water content may range up to 60%. A mudflow can be distinguished from a debris flow as grain size decreases and mud content exceeds 10 percent of the dry weight of the contained sediment.

NGVD is an abbreviation for the National Geodetic Vertical Datum, an elevation datum set in 1929 that corresponds approximately to mean sea-level for coastal areas bordering the U.S. It is commonly used as a baseline for referencing elevations in engineering-design projects.

Particle-size distribution is the percentage, usually by weight (sieving) or by number (Wolman pebble count), of particles in each size fraction into which a disaggregated sample of a soil, sediment, or rock has been classified, such as the percentage of sand retained on each sieve in a given size range.

Percolation ponds are ponds constructed for the purpose of capturing runoff water and allowing it to infiltrate into the groundwater reservoir.

Percolation rate is the rate at which water infiltrates to the groundwater reservoir, generally from a percolation pond, and is expressed as a volume per time (e.g., cubic feet per second).

Recharge is the processes involved in the addition of water to the zone of saturation; also, the amount of water added.

Recharge efficiency characterizes the overall effectiveness of some action in terms of its ability to allow percolation (or recharge) of surface water to the groundwater reservoir.

Riparian is pertaining to or situated on the bank of a body of water, especially of a river.

Sediment concentration is the mass of dry solids divided by the volume of water and usually is expressed in milligrams per liter.

Sediment yield is the quantity of sediment, total or suspended, that is transported from or produced per unit area. Sediment yield usually is expressed as a mass or volume per unit area and time (for example, tons per square mile per year).

Suspended sediment is sediment that is moved in suspension in water and is maintained in suspension by the upward components of turbulent currents or by colloidal suspension.

Thalweg is the point of lowest elevation in a water-conveying channel.

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APPENDIX A

[illegible] Model Component Equations and Numerical Solution Schemes (Hydraulics and Sediment Transport)

Expressions are now sought for:

$$\frac{\partial Q}{\partial h} = \frac{\partial Q}{\partial A}\frac{\partial A}{\partial h} + \frac{\partial Q}{\partial R}\frac{\partial R}{\partial h} + \frac{\partial Q}{\partial S}\frac{\partial S}{\partial h} \approx \frac{\partial Q}{\partial A}\frac{\partial A}{\partial h} + \frac{\partial Q}{\partial S}\frac{\partial S}{\partial h} \tag{3}$$

From the basic equation we have:

$$\frac{\partial Q}{\partial A} = CR^{1/2}S^{1/2} \quad and \quad \frac{\partial A}{\partial h} = b$$
$$\frac{\partial Q}{\partial S} = \frac{1}{2}AC\sqrt{\frac{R}{S}} \quad and \quad \frac{\partial S}{\partial h} = \frac{1}{\Delta x} \tag{4}$$

Inserting this in the above linearized expression gives:

$$Q^{n+1} = Q^{n+1/2} + \left(C\sqrt{RS}\,b + \frac{1}{2\Delta x}AC\sqrt{\frac{R}{S}}\right)\left(h^{n+1} - h^{n+1/2}\right) \tag{5}$$

giving:

$$Q^{n+1} = Q^{n+1/2} + \left(\frac{b\Delta x S}{A} + \frac{1}{2}\right)\frac{AC}{\Delta x}\sqrt{\frac{R}{S}}\left(h^{n+1} - h^{n+1/2}\right) \tag{6}$$

A.7 Saint Venant Equations

MIKE 11 applied with the fully dynamic descriptions solves the vertically integrated equations of conservation of volume and momentum (the 'Saint Venant' equations), which are derived on the basis of the following assumptions:

- the water is incompressible and homogeneous, ie without significant variations in density
- the bottom slope is small
- the wave lengths are large compared to the water-depth. This ensures that the flow everywhere can be regarded as having a direction parallel to the bottom, i.e. vertical accelerations can be neglected and a hydrostatic pressure variation along the vertical can be assumed
- the flow is sub-critical

For a rectangular cross-section with a horizontal bottom and a constant width, the conservation of mass and momentum can be expressed as follows (in the first instance neglecting friction and lateral inflows):

Conservation of mass:

$$\frac{\partial(\rho Hb)}{\partial t} = -\frac{\partial(\rho Hb\bar{u})}{\partial x} \tag{1}$$

Conservation of momentum:

$$\frac{\partial(\rho Hb\bar{u})}{\partial t} = -\frac{\partial(\alpha'\rho Hb\bar{u}^2 + \frac{1}{2}\rho gbH^2)}{\partial x} \tag{2}$$

where, ρ is the density, H the depth, b the width, $\bar{u}$ the average velocity along the vertical and α' the vertical velocity distribution coefficient.

Introducing the bottom slope, I_b, and allowing for the channel width to vary will give rise to two more terms in the momentum equation. These terms describe the projections in the flow direction of the reactions of the bottom and side-walls to the hydrostatic pressure.

The momentum equation now becomes:

$$\frac{\partial(\rho Hb\bar{u})}{\partial t} =$$

$$-\frac{\partial\left(\alpha'\rho Hb\overline{u^2} + \frac{1}{2}\rho gbH^2\right)}{\partial x} + \frac{\partial b}{\partial x}\frac{\rho gH^2}{2} - \rho gHbI_b \tag{3}$$

$$= -\frac{\partial(\alpha'\rho Hb\overline{u^2})}{\partial x} - b\frac{\partial\left(\frac{1}{2}\rho gH^2\right)}{\partial x} - \rho gHbI_b$$

When the water level, h, is introduced into the relationship instead of water depth:

$$\frac{\partial h}{\partial x} = I_b + \frac{\partial H}{\partial x} \tag{4}$$

and the equations are divided by ρ, the conservation laws of mass and momentum become:

$$\frac{\partial(Hb)}{\partial t} = -\frac{\partial(Hb\bar{u})}{\partial x} \tag{5}$$

$$\frac{\partial(Hb\bar{u})}{\partial t} = -\frac{\partial(\alpha Hb\overline{u^2})}{\partial x} - Hbg\frac{\partial h}{\partial x} \tag{6}$$

These equations can be integrated to describe the flow through cross-sections of any shape when divided up into a series of rectangular cross sections as shown in Fig. A.7.1.:

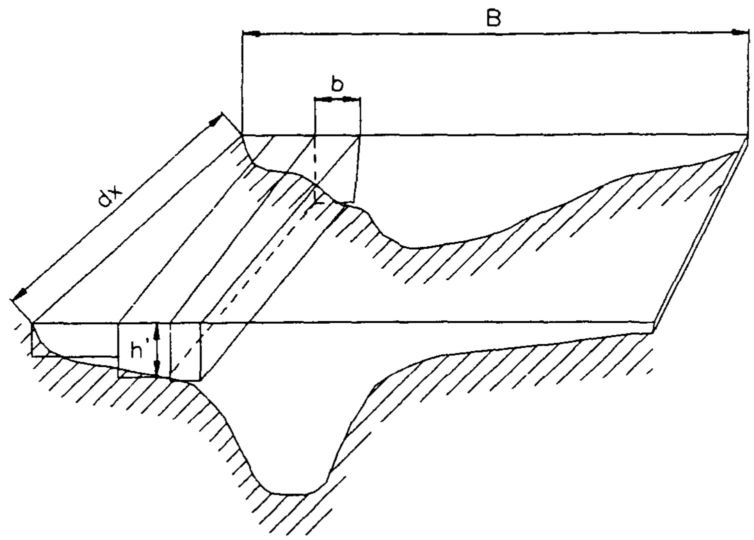

Fig. A.7.1 *Cross-section divided into a series of rectangular channels*

According to the previous assumptions, ∂h/∂x is constant across the channel and no exchange of momentum occurs between the subchannels. If the integrated cross sectional area is called A and the integrated discharge Q, and B is the full width of the channel, then:

$$A = \int_0^B H\,db \tag{7}$$

$$Q = \int_0^B H\bar{u}\,db = \bar{u}A \tag{8}$$

Integrating the mass and momentum conservation equations and introducing Equations (7) and (8) yields:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \qquad (9)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial\left(\alpha \frac{Q^2}{A}\right)}{\partial x} + gA\frac{\partial h}{\partial x} = 0 \qquad (10)$$

Including the hydraulic resistance, eg using the Chezy description and the lateral inflow; 'a' into these equations leads to the basic equations used in MIKE 11:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \qquad (11)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial\left(\alpha \frac{Q^2}{A}\right)}{\partial x} + gA\frac{\partial h}{\partial x} + \frac{gQ|Q|}{C^2AR} = 0 \qquad (12)$$

A.8 Solution Scheme

The solution to the combined system of equations at each time step is performed according to the procedure outlined below. The solution method is the same for each model level (kinematic, diffusive, dynamic).

The transformation of Equations (11) and (12) in **Saint Venant Equations**, section A.7 of this Appendix, to a set of implicit finite difference equations is performed in a computational grid consisting of alternating Q- and h-points, ie points where the discharge, Q and water level h, respectively, are computed at each time step, see Fig. A.8.1. The computational grid is generated automatically by the model on the basis of the user requirements. Q-points are always placed midway between neighbouring h-points, while the distance between h-points may differ. The discharge will, as a rule, be defined as positive in the positive x-direction (increasing chainage).

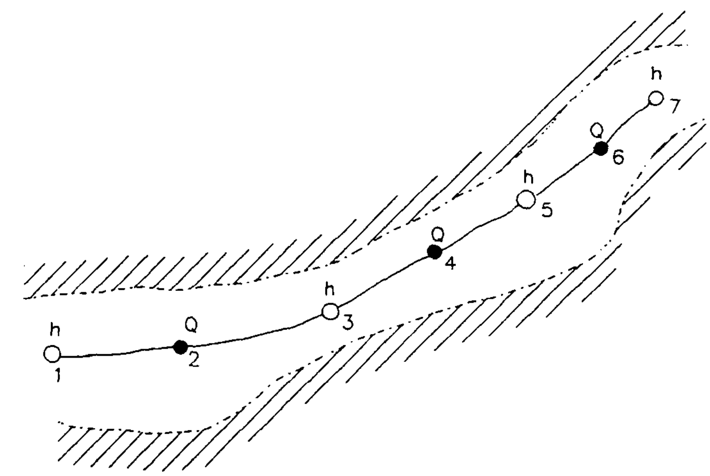

Fig. A.8.1 *Channel section with computational grid*

The adopted numerical scheme is a 6-point Abbott-scheme as shown in Fig. A.8.2.

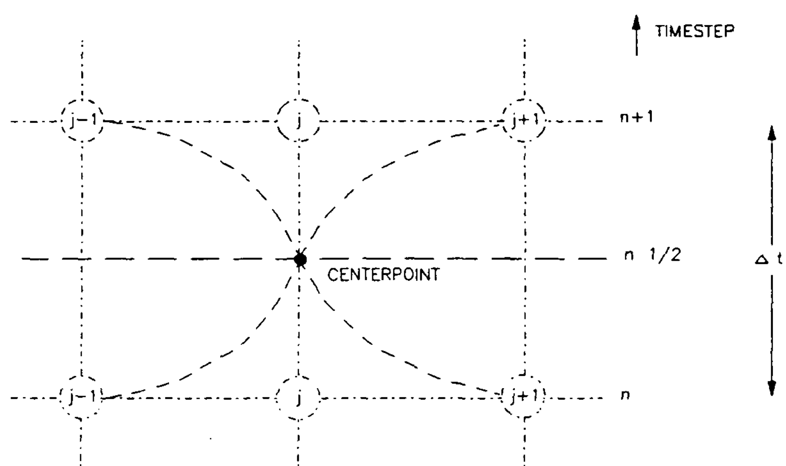

Fig. A.8.2 *Centred 6-point Abbott scheme*

Continuity equation

In the continuity equation the storage width, b_s, is introduced as:

$$\frac{\partial A}{\partial t} = b_s \frac{\partial h}{\partial t} \tag{1}$$

giving:

$$\frac{\partial Q}{\partial x} + b_s \frac{\partial h}{\partial t} = q \tag{2}$$

As only Q has a derivative with respect to x, the equation can easily be centred at a h-point, see Fig. A.8.3.

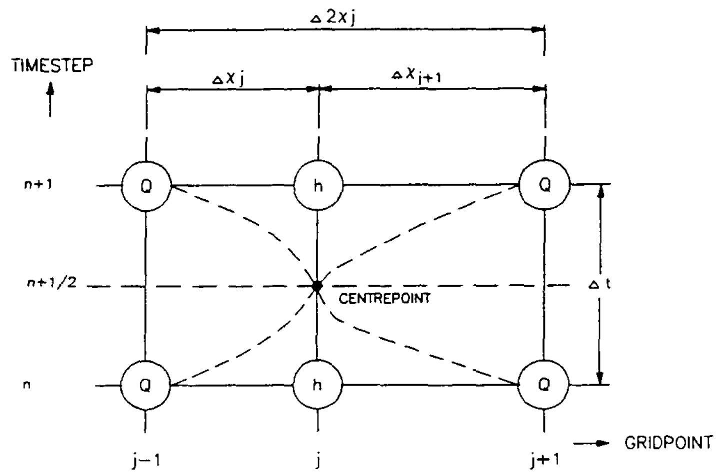

Fig. A.8.3 Centring of continuity equation in 6-point Abbott scheme

The derivatives in Equation (2) are expressed at the time level, n+½, as follows:

$$\frac{\partial Q}{\partial x} \approx \frac{\dfrac{\left(Q_{j+1}^{n+1} + Q_{j+1}^{n}\right)}{2} - \dfrac{\left(Q_{j-1}^{n+1} + Q_{j-1}^{n}\right)}{2}}{\Delta 2x_j} \tag{3}$$

$$\frac{\partial h}{\partial t} \approx \frac{\left(h_j^{n+1} - h_j^{n}\right)}{\Delta t} \tag{4}$$

b_s in Equation (2) is approximated by:

$$b_s = \frac{A_{o,j} + A_{o,j+1}}{\Delta 2x_j} \tag{5}$$

where:

$A_{o,j}$ is the surface area between grid point j-1 and j

$A_{o,j+1}$ is the surface area between grid point j and j+1

$\Delta 2x_j$ is the distance between point j-1 and j+1

Substituting for the derivatives in Equation (2) gives a formulation of the following form:

$$\alpha_j Q_{j-1}^{n+1} + \beta_j h_j^{n+1} + \gamma_j Q_{j+1}^{n+1} = \delta_j \tag{6}$$

where, α, β and γ are functions of b ?? and δ and, moreover, depend on Q and h at time level n and Q on time level n+½.

Momentum equation

The momentum equation is centred at Q-points as illustrated in Fig. A.8.4.

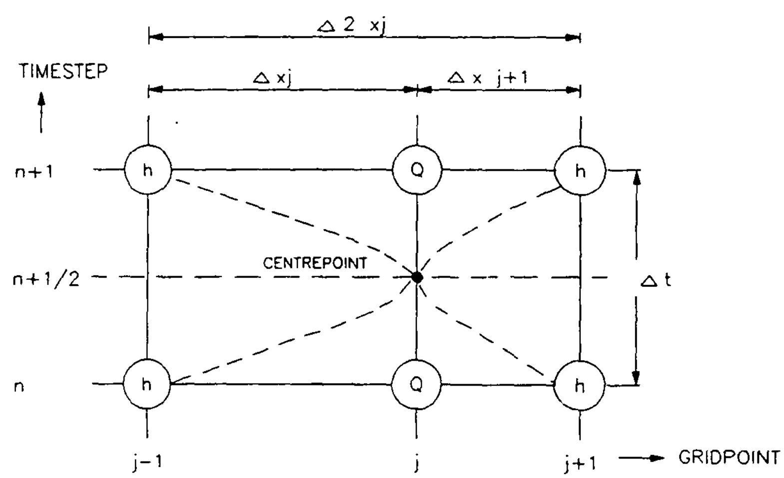

Fig. A.8.4 Centring of momentum equation in 6-point Abbott scheme

The derivatives of Equation (12) in **Saint Venant Equations**, section A.7 of this Appendix, are expressed in the following way:

$$\frac{\partial Q}{\partial t} \approx \frac{\left(Q_j^{n+1} - Q_j^n\right)}{\Delta t} \tag{7}$$

$$\frac{\partial\left(\alpha \dfrac{Q^2}{A}\right)}{\partial x} \approx \frac{\left(\left[\alpha \dfrac{Q^2}{A}\right]_{j+1}^{n+\frac{1}{2}} - \left[\alpha \dfrac{Q^2}{A}\right]_{j-1}^{n+\frac{1}{2}}\right)}{\Delta 2x_j} \tag{8}$$

$$\frac{\partial h}{\partial x} \approx \frac{\dfrac{\left(h_{j+1}^{n+1} + h_{j+1}^{n}\right)}{2} - \dfrac{\left(h_{j-1}^{n+1} + h_{j-1}^{n}\right)}{2}}{\Delta 2x_j} \tag{9}$$

For the quadratic term in (8), a special formulation is used to ensure the correct sign for this term when the flow direction is changing during a time step:

$$Q^2 \approx f Q_j^{n+1} Q_j^n - (f-1) Q_j^n Q_j^n \tag{10}$$

where, f can be specified by the user (THETA coefficient in Menu G.5.5) and by default is set to 1.0.

With all the derivatives substituted, the momentum equation can be written in the following form:

$$\alpha_j h_{j-1}^{n+1} + \beta_j Q_j^{n+1} + \gamma_j h_{j+1}^{n+1} = \delta_j \tag{11}$$

where,

$$\alpha_j = f(A)$$

$$\beta_j = f(Q_j^n, \Delta t, \Delta x, C, A, R)$$

$$\gamma_j = f(A)$$

$$\delta_j = f(A, \Delta x, \Delta t, \alpha, q, v, \phi, h_{j-1}^n, Q_{j-1}^{n+½}, Q_j^n, h_{j+1}^n, Q_{j+1}^{n+½})$$

To obtain a fully centred description of A_{j+1}, these terms should be valid at time level n+½ which can only be fulfilled by using an iteration. For this reason, the equations are solved by default two times at every time step, the first iteration starting from the results of the previous time step, and the second iteration using the centred values from this calculation. The number of iterations can be changed using the NR-ITER coefficient.

A.9 Structures

Some of the structure types described under Structures in the Reference Manual share the same mathematical threatment. These are:

- broadcrested weir, control structures (overflow and non surcharged underflow structures), dambreak (crest flow and breach flow)
- special weir, Q = f(h)
- culverts, user-defined Q-h relations

Their mathematical treatment is described below.

Mathematical Treatment

The structure description combines a wide range of elements including weirs, narrow cross sections, flood plains, etc. It is obtained by replacing the momentum equation with an h-Q-h relationship, an h-Q relationship, or a Q assignment.

The grid used to describe a structure consists of h-points at either side, and a Q-point at the structure, see Fig. A.9.1.

Q_s and h_s will be used in the following to describe the discharge and the water level at the structure, respectively, ie at point j.

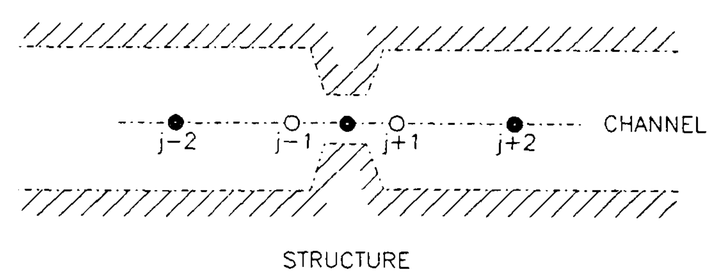

Fig. A.9.1 Channel with structure and grid

A.5 The "Double Sweep" Algorithm

The continuity equation and momentum equation can be formulated in a similar form (compare Equations (6) and (11) under Solution Scheme, section A.8 of this Appendix).

Using, instead of h and Q, the general variable name, Z, the general formulation will be:

$$\alpha_j Z_{j-1}^{n+1} + \beta_j Z_j^{n+1} + \gamma_j Z_{j+1}^{n+1} = \delta_j \qquad (1)$$

Applying a local elimination, the coefficient matrix can, in principle, be transformed as shown in Fig. A.5.1 below. It is thus possible to write any water level or discharge variable within the branch as a function of the water levels in the upstream and downstream nodal points H_1 and H_2, ie

$$h,Q = h,Q(H_1,H_2) \qquad (2)$$

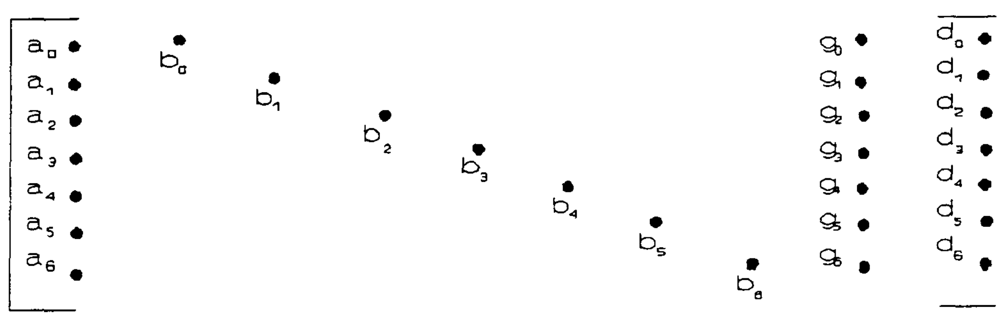

Fig. A.5.1 Branch matrix after local elimination

The continuity equation around a nodal point can in principle be expressed as:

$$a h_{node}^{n+1} + b h_{branch_1}^{n+1} + c Q_{branch_1}^{n+1} + d h_{branch_2}^{n+1} + e Q_{branch_2}^{n+1} + \ldots = z \qquad (3)$$

where, a..z are quasi-constants. If Equation (2) is substituted herein, a global relation can be obtained:

$$AH_1 + BH_2 + ... = Z \tag{4}$$

A,B, ... to Z are quasi constants.

Equation (4) shows that the water level in a nodal point can be described as a function of the water levels in the neighbouring nodal points. It is therefore possible to set up a nodal point matrix at each time step using the coefficients from Equation (4) and the solution to the matrix yields, by backward substitution, the water levels in all nodal points at the next time step.

Fig. A.5.2 shows an example with 8 nodal points and 9 branches.

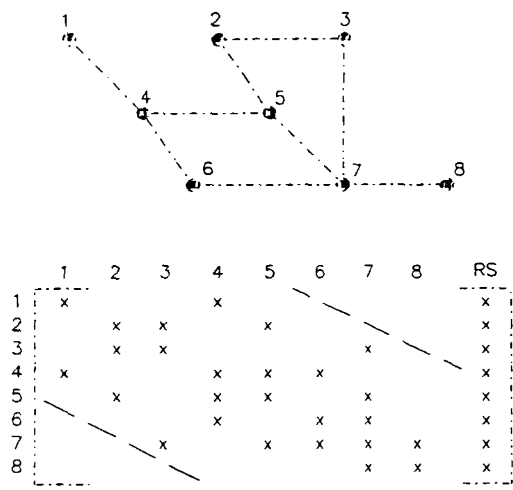

Fig. A.5.2 *Principle of nodal point matrix for a network system with 8 nodal points and 9 branches*
***NOTE** ! A boundary is treated as a nodal point*

The crosses in the matrix symbolise coefficients, meaning that, for instance, the water level in point 4 can be expressed as a function of the water levels in points 1, 5 and 6. When the nodal point matrix has been solved, the solution in the branches is found by backward local elimination.

The band width of the matrix in Fig. A.5.2 as indicated by the dotted lines, is a function of the order in which the nodal points are defined. The band width in Fig. A.5.2 is equal to 5. The total amount of computational time will increase with increasing band width.

In order to minimize the computational work and thus increase the computational speed a routine has been developed which automatically minimizes the band width by internal re-ordering of the nodal points. The band width indicated in Fig. A.5.2 for the 8 nodal points and the 9 branches can be reduced to 4 as shown in the matrix in Fig. A.5.3 below.

	1	4	5	6	7	2	3	8	RS
1	x	x							x
2	x	x	x	x					x
3		x	x		x	x			x
4		x		x	x				x
5			x	x	x		x	x	x
6			x			x	x		x
7					x	x	x		x
8					x			x	x

Fig. A.5.3 Minimised band width

A.6 Momentum Equation with Modified "Diffusive Wave Approximation"

The diffusive wave approximation can be written for example for the case of Chezy roughness as:

$$Q = ACR^{1/2}S^{1/2} \tag{1}$$

where, S is the slope of the water surface. This equation is linearized through the following approximations:

$$Q^{n+1} \approx Q^{n+1/2} + \frac{\partial Q}{\partial h}\left(h^{n+1} - h^{n+1/2}\right) \tag{2}$$

$$\frac{dL}{dt} = \left(\frac{L - L_e}{10\ L_e}\right) \cdot c = \left(\frac{L}{L_e} - 1\right) q_b \cdot \frac{1}{10\ (1-\epsilon)H} \qquad (2)$$

where, ϵ is the sediment porosity.

5.19 Sediment Transport Models

Five sediment transport models are available in MIKE 11. These are described in the following sections: **Ackers and White Model, Engelund and Fredsøe Model, Engelund and Hansen Model, Smart and Jaeggi Model** and the **van Rijn Model.**

The selection of transport model for a particular appliction depends on the nature of the water course under study and on local experience in sediment transport modelling. In the absence of any such knowledge, trial simulations should be carried out with each model to see which gives better agreement with measurements.

Once the most appropriate transport model has been selected, further adjustments can be made to the predicted transport rates during the calibration procedure by the use of the two factors FAC_1 and FAC_2. These are specified in Menu E.5.3 and are used to apply a linear correction factor to the predicted suspended and bedload transport rates, respectively (or to the total load, FAC_1 only).

5.20 Smart and Jaeggi Model

Smart and Jaeggi (1983) presented a sediment transport formula which calculates the transport of coarse sediments in steep channels/rivers. The transport formulae is based on the original **Meyer-Peter Mueller** equation, which was derived from laboratory experiments with non-uniform sediments of various densities and flume-slopes ranging from 0.04% to 2%.

The original **Meyer-Peter Mueller** equation reads:

$$\phi = 8\left[\left(\frac{K_s}{K_r}\right)^{1.5}\theta - 0.047\right]^{1.5} \tag{1}$$

where,
ϕ = dimensionless sediment transport
θ = dimensionless shear stress
(K_s/K_r) = corr.factor for bed form roughness

Revised transport formula

Through comparison of new laboratory experiments and the original Equation (1) it was found that the computed transport is seriously underestimated with slopes steeper than 3%. Smart and Jaeggi therefore proposed a new transport formulation based on Equation (1).

At higher slopes, grains in the bed have a slope induced height advantage of those grains immediately downstream of them and the critical shear stress is less than it would be for a flat bed. Shields parameter is therefore adjusted as a function of the slope to horizontal, Eq.(2):

$$\theta_{cr} = \theta_{0cr}\cdot\cos\alpha\left(1 - \frac{\tan\alpha}{\tan\beta}\right) \tag{2}$$

where,
θ_{0cr} = Shields parameter
α = Slope to horizontal of water surface or bottom level
β = Angle of repose of submerged bed material (33°)

Including non-uniformarity of the sediment as well as the slope induced effect of the transport the revised equation - still based on laboratory experiments - reads:

$$\phi = 4\left[\left(\frac{d_{90}}{d_{30}}\right)^{0.2} I^{0.6}\ c\ \theta^{0.5}\ (\theta - \theta_{cr})\right] \tag{3}$$

$$Q_{tbnd} = \sum_{i=1}^{N} Q_{ibnd} \qquad (15)$$

Graded Sediments Boundary Conditions

In the case where $Q_{tbnd}=0$, ie there is no transport of any fraction across the boundary, all the fraction proportions are automatically set to zero except for the most coarse fraction which is set to 1.

The distribution of sediment at nodal points is determined as described below in section 5.16, Morphological Model.

5.16 Morphological Model

The morphological model updates the bed level due to erosion or deposition using the continuity equation for sediment transport. The model updates the level of the entire cross-section or only a part of it (normally representing the river channel) and leaving the bottom level of the remaining part of the cross-section (normally representing the flood plains) unchanged. The model distinguishes between the river channel and flood plain using a divide level specified in the cross-sectional data base (A.6.5), ie levels in the cross-section below this divide level are assumed to belong to the river channel. Various possibilities for morphological changing of the cross-section shape is avialable. If a divide level has been specified only the river channel is updated, which implies that sediment transport only takes place in the river channel.

The sediment transport is calculated on the basis of flow velocity (u), depth (D) and transporting width (W_t) and is obtained in the following way:

$$u = Q_c / A_c \qquad (1)$$

$$D = R_c \qquad (2)$$

$$W_t = A_c / R_c \qquad (3)$$

where,
Q is discharge
A is cross-sectional area
R is resistance radius

and subscript 'c' refers to the river channel values. If the cross-sections are not divided into river channel and flood plain components the corresponding values for the entire cross-section are used.

The discharge in the river channel is obtained from:

$$Q_c = Q_t \begin{cases} \dfrac{M_c R_c^{2/3} A_c}{M_t R_t^{2/3} A_t} & \textit{Manning No.} \\ \\ \dfrac{C_c \sqrt{R_c}\, A_s}{C_t \sqrt{R_t}\, A_t} & \textit{Chezy No.} \end{cases} \tag{4}$$

where,
M is Manning roughness coefficient
C is Chezy roughness coefficient

and subscript 't' refers to the entire cross-section and 'c' to the river channel part.

The sediment continuity equation is solved at h-points using a **Preissmann Scheme**.

$$(1-\epsilon)\left[(1-\psi)\frac{W\Delta z_j^{n+1}}{\Delta t} + \psi\frac{W\Delta z_{j+1}^{n+1}}{\Delta t}\right] + \theta\,\frac{Qt_{j+1}^{n+1} - Qt_j^{n+1}}{\Delta x} + (1-\theta)\,\frac{Qt_{j+1}^{n} - Qt_j^{n}}{\Delta x} = 0 \tag{5}$$

where,

W	=	width of the river channel or entire cross-section, defined as the width of the cross-section at the calculated water surface
Δz^{n+1}	=	change of bottom level.
Qt^n_j	=	$W_t qt^n_j$.
qt^n_j	=	sediment transport rate per unit width.

ϵ = porosity of sediment.

ψ = space centring coef. $(0.5 \leq \psi \leq 1)$ (PSI in Menu E.5.2).

θ = time centring coef. $(0.5 \leq \theta \leq 1)$ (FI in Menu E5.3).

The transport at t = (n+1)Δt is approximated by:

$$Qt_j^{n+1} = Qt_j^n + \left(\frac{\partial Qt}{\partial u} \frac{\partial u}{\partial z} + \frac{\partial Qt}{\partial D} \frac{\partial D}{\partial z} \right)_j \Delta z_j^{n+1} \tag{6}$$

or introducing the term α

$$Qt_j^{n+1} = Qt_j^n + \alpha \Delta z_j^{n+1} \tag{7}$$

where,
u is flow velocity
D is flow depth

$\partial u/\partial z$ and $\partial D/\partial z$ are, by default, calculated assuming locally steady flow (back water curve).

Further, it is possible to select:

$$\frac{\partial D}{\partial z} = -1 \ and \ \frac{\partial u}{\partial z} \frac{D}{u} = +1$$

(Menu E.5.2), which is an acceptable approximation in strongly unsteady (tidal) flow.

$\partial Qt/\partial u$ and $\partial Qt/\partial D$ are obtained by numerical differentiation, eg:

$$\frac{\partial Qt}{\partial u} \approx \frac{Qt(u+\Delta u,D) - Qt(u,D)}{\Delta u}$$

or

$$\frac{\partial Qt}{\partial u} \approx \frac{Qt\ (u\ fac,D - Qt(u,D)}{u\ fac\ -u}$$

where,

$$fac = \frac{u+\Delta u}{u}$$

and is given in Menu E.5.2.

Updating of Bottom Level

Various assumptions regarding the change in bathymetry of a cross-section during erosion and deposition can be made. In some cases sediment will accumulate in the deepest parts of the cross-section before deposition occurs closer to the bank (eg in a reservoir behind a dam). For tidal estuaries another deposition pattern with pronounced deposition along the banks can occur. Such considerations are necessary when the results from the MIKE 11 morphological model are interpreted, as the model is only one-dimensional. However, the MIKE 11 provides different options, see below, for how to update the bottom level. The appropriate model should be chosen based on engineering judgement.

Instead of solving equation (5) with respect to Δz^{n+1} , the change in cross-sectional area ΔA is calculated, $\Delta A^{n+1} = W \cdot \Delta z^{n+1}$. The cross-section is subsequently updated (ie the processed cross-sectional data tables used in the computation are changed) in one of the following ways, see below and Fig. 5.16.1.

In the following model descriptions subscript 'i' refers to the levels in the processed data table. Subscript 'k' in model 1 refers to the level in the processed data table where the area is larger than the change in cross-section area ΔA.

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Model 1 - Deposition in horizontal layers. Erosion as model 5:

Levels:

$$z_i^{n+1} = z_k^{n+1} + \frac{\Delta A - A_k^n}{w_{k+1} - w_k} \quad for\ z_i^n \le z_k^n$$

$$A_k^n < \Delta A < A_{k+1}^n$$

$$z_i^{n+1} = z_i^n \quad for\ z_i^n > z_k^n$$

(1)

Widths: unchanged

Areas:

$$A_i^{n+1} = 0 \quad for\ z_i^n \le z_k^n$$

$$A_i^{n+1} = A_i^n - \Delta A \quad for\ z_i^n > z_k^n$$

(2)

Hydraulic radii:

$$R_i^{n+1} = 0 \quad for\ z_i^n \le z_k^n$$

$$R_i^{n+1} = \left[\frac{\sum_{j=1}^{i} (z_i^{n+1} - z_j^{n+1})^{\frac{3}{2}} \cdot \Delta W_j}{A_i^{n+1}}\right]^2 \quad for\ z_i^n > z_k^n$$

(3)

Model 2 - Deposition and erosion uniformly distributed <u>below the water surface. No deposition and erosion above:</u>

Levels:

$$z_i^{n+1} = z_i^n + \frac{\Delta A}{w_{surface}} \quad for\ z_i^n \le h$$

$$W_{surface} = width\ of\ cross\text{-}section\ at\ watersurfacelevel\ h$$

$$z_i^{n+1} = z_i^n \quad for\ z_i^n > h$$

(4)

Widths: unchanged

Areas:

$$A_i^{n+1} = A_i^n \quad for\ z_i^n \le h$$

$$A_i^{n+1} = A_i^n - \Delta A \quad for\ z_i^n > h$$

(5)

Hydraulic radii:

$$R_i^{n+1} = R_i^n \quad for\ z_i^n \le h$$

$$R_i^{n+1} = \left[\frac{\sum_{j=1}^{i} (z_i^{n+1} - z_j^{n+1})^{\frac{3}{2}} \cdot \Delta W_j}{A_i^{n+1}}\right]^2 \quad for\ z_i^n > h$$

(6)

Model 3 - Deposition and erosion proportional with depth below water surface. No deposition and erosion above:

Levels:

$$\alpha = \frac{\Delta A}{A^n_{surface}}$$

$$z_i^{n+1} = z_i^n + \alpha \cdot (h - z_i^n) \qquad for\ z_i^n \leq h$$

$$A_{surface} = area\ of\ cross\text{-}section\ at\ watersurface\ level\ h$$

$$z_i^{n+1} = z_i^n \qquad for\ z_i^n > h$$

(7)

Widths: unchanged

Areas:

$$A_i^{n+1} = (1-\alpha) \cdot A_i^n \qquad for\ z_i^n \leq h$$

$$A_i^{n+1} = A_i^n - \Delta A \qquad for\ z_i^n > h$$

(8)

Hydraulic radii:

$$R_i^{n+1} = (1-\alpha) \cdot R_i^n \qquad for\ z_i^n \leq h$$

$$R_i^{n+1} = \left[\frac{\sum_{j=1}^{i} (z_i^{n+1} - z_j^{n+1})^{\frac{3}{2}} \cdot \Delta W_j}{A_i^{n+1}}\right]^2 \qquad for\ z_i^n > h$$

(9)

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Model 4 - Deposition and erosion uniformly distributed over the whole cross-section (ie below the banklevel):

Levels:

$$z_i^{n+1} = z_i^n + \frac{\Delta A}{w_{top}}$$

$$W_{top} = \textit{width of cross-section at the toplevel } z_{\max} \tag{10}$$

Widths: unchanged

Areas:

$$A_i^{n+1} = A_i^n \tag{11}$$

Hydraulic radii:

$$R_i^{n+1} = R_i^n \tag{12}$$

Model 5 - Deposition and erosion proportional with depth below banklevel (top level):

Levels:

$$\alpha = \frac{\Delta A}{A_{top}^n}$$

$$z_i^{n+1} = z_i^n + \alpha \cdot (h - z_i^n)$$

$$A_{top} = \textit{area of cross-section at toplevel } z_{\max} \tag{13}$$

Widths: unchanged

Areas:

$$A_i^{n+1} = (1-\alpha)\cdot A_i^n \tag{14}$$

Hydraulic radii:

$$R_i^{n+1} = (1-\alpha)\cdot R_i^n \tag{15}$$

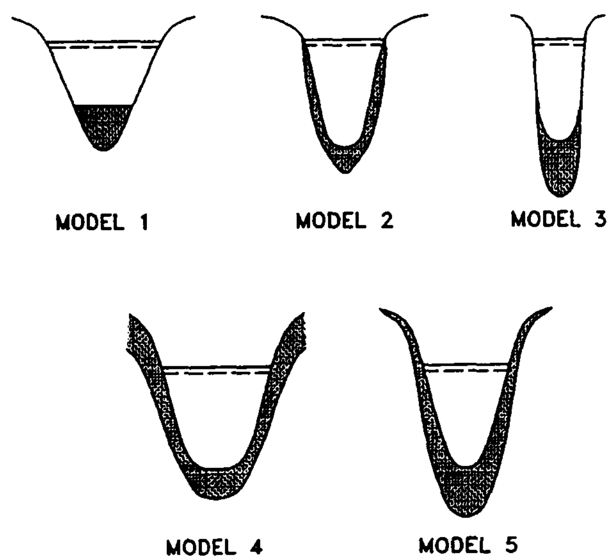


Fig. 5.16.1 Update of bed level in MIKE 11 morphological model

Boundary Conditions

The unknown variable in the finite difference scheme is Z. This becomes evident by substituting Equation (2) into Equation (1). Boundary conditions should therefore be given in terms of bed level variation, although a sediment transport boundary condition can also be given. In this case Equation (7) is normally used to determine ΔZ with Q_t^{n+1} replaced by the boundary sediment transport. However, when there is a serious imbalance at the boundary point, ie when alpha is small and $Qt_{bnd} >> Qt^n$ this formulation may give rise to numerical stability problems. In this case the difference between the transport specified at the boundary and the calculated transport at the in-flow point (first or last gridpoint depending on the flow direction) is assumed to erode/deposit at the inflow point, (one point continuity), ie:

$$Qt_{bnd}/W - \left(Qt_j^n + \alpha \Delta z_j^{n+1}\right) = \frac{\Delta x}{\Delta t}(1-\epsilon)\ \Delta z_j^{n+1} \qquad (8)$$

which is solved for Δz_j^{n+1}.

Boundary conditions should be given at all inflow boundaries. In tidal models, boundary conditions should also be given at the tidal boundary (alternating in- and outflow).

At nodal points the sediment is distributed according to the flow discharge, Q, (ie with coefficients K_1 to K_4 and exponents n_1 to n_4 equal to 1) or according to a user specified distribution (ie with K_1 to K_4 and/or n_1 to n_4 different from 1). For example, in Fig. 5.16.2 the transport rate in branch 3 is expressed as:

$$Q_{t_3}^{n+1} = \frac{K_3 \cdot Q_3^{n3}}{K_3 Q_3^{n3} + K_4 Q_4^{n4}} \left(Qt_1^{n+1} + Qt_2^{n+1}\right)$$

and similarly for branch 4. The first grid points in branches 3 and 4 are treated in the same way as external boundaries with Qt_{bnd} replaced by Qt_3^{n+1} and Q_t^{n+1}, respectively.

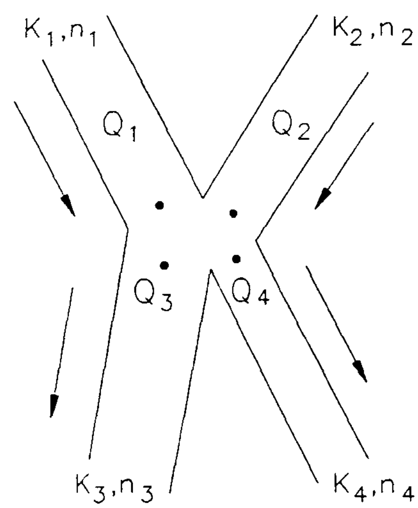


Fig. 5.16.2 Distribution of sediment according to discharge

In nodal points sediment can be transported into a branch which has been specified as passive, but no sediment can be transported out of a passive branch.

Low Flow Correction

When the water level becomes less than a certain threshold value (delh, given in Menu G.5.5) the hydrodynamic model introduces an artificial slot, see Chapter 2, HD Reference Manual. In this case the calculated velocity may have a rather arbitrary value, which may give rise to unrealistically large sediment transport rates. This problem has been solved by multiplying the calculated transport rate with a reduction factor 'f' given by:

$$f = \begin{cases} 0 & D < delh \\ (D-delh)/D & delh < D < 2\ delh \\ 1 & D > 2\ delh \end{cases}$$

Hints

The **Preissmann scheme** is well suited to the solution of hyperbolic problems with only one characteristic. However, it has a drawback. At small Courant numbers it may generate short wave oscillations (wave length 2Δx). The oscillations can be damped by space foreward centring the scheme (ie by using a large value of ψ). This, however, will give rise to numerical dispersion at high Courant numbers. At a later stage an option for automatic selection of the optimum value of ψ will be incorporated, ie the value which just prevents the short wave oscillations.

The factor 'fac' used in the numerical differentiation of the sediment transport formula should not be made too small because this will give rise to 'overshoot' phenomena in the numerical solution. A value of about 1.5 will be on the safe side in most cases unless very strong gradients occur.

The sediment transport Courant number can be approximated by

$$C_r = 5 \frac{\Delta t}{\Delta x} \frac{q_t}{D} \frac{1}{1-\epsilon}$$

This implies that the morphological model can run with a much larger time step than the dynamic flow model and that a quasi-steady flow simulation will be sufficiently accurate, (see also **de Vries, 1981**). Often the time step will be limited only by the ability to resolve the boundary conditions.

5.17 Non-Equilibrium Dune Height

The rate of change of the dune height has been analysed by **Fredsøe (1979)**. It can be described by the equations used to calculate the equilibrium dune height.

The rate of change of the bed level 'z' near the dune top is expressed as:

$$\frac{\partial z}{\partial t} = -c \cdot \frac{\partial z}{\partial x} + \frac{dz}{dt} \tag{1}$$

where the celerity of the dune front is

$$c = \frac{q_{bt}}{(1-\epsilon)H} \tag{2}$$

where, q_{bt} is the transport rate at the dune top, ϵ is the sediment porosity and $\partial z/\partial t$ is found from the equation of continuity:

$$\frac{\partial z}{\partial t} = \frac{-1}{1-\epsilon}\left.\frac{\partial\,(q_b + q_s)}{\partial x}\right|_{top} = -\frac{1}{1-\epsilon}\left.\frac{\partial(q_b + q_s)}{\partial\theta}\right|_{top}\frac{2\theta_{top}}{1 - H/D}\cdot\frac{\partial z}{\partial x} \tag{3}$$

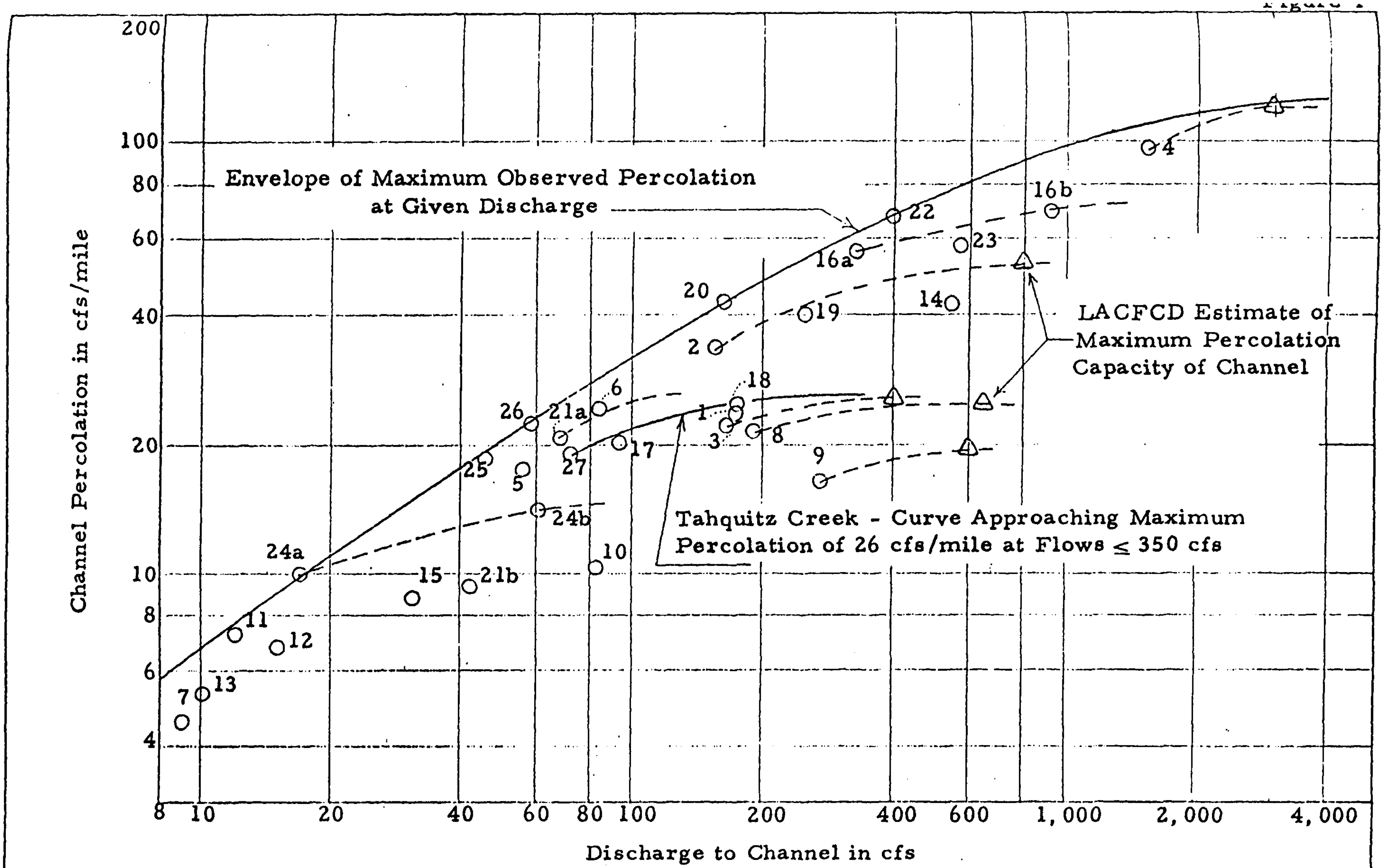
Equations (1), (2) and (3) yield:

$$\frac{dH}{dt} = \frac{1}{1-\epsilon}\,\frac{\partial z}{\partial x}\,\frac{\Phi_b}{H} - \left.\frac{\partial(\Phi_b + \Phi_s)}{\partial\theta}\right|_{top}\frac{2\theta_{top}}{D-H}\sqrt{(s-1)gd^3} \tag{4}$$

APPENDIX B

Stream Channel Percolation Data

from [illegible] and Edmonston, 1971



RELATIONSHIP BETWEEN DISCHARGE AND MAXIMUM OBSERVED
CHANNEL PERCOLATION IN SOUTHERN CALIFORNIA STREAMS