R-02-01

HYDRAULIC MODEL STUDY OF THE SAN SEVAINE SIDE-WEIR DIVERSION TO JURUPA BASIN

January 2002

U.S. DEPARTMENT OF THE INTERIOR
Bureau of Reclamation

Technical Service Center
Water Resources Services
Water Resources Research Laboratory
Denver, Colorado

REPORT DOCUMENTATION PAGE			Form Approved OMB No. 0704-0188

1. AGENCY USE ONLY *(Leave Blank)*	2. REPORT DATE	3. REPORT TYPE AND DATES COVERED
	January 2002	Final

4. TITLE AND SUBTITLE

Hydraulic Model Study of the SanSevaine Side-Weir Diversion To Jurupa Basin

5. FUNDING NUMBERS

6. AUTHOR(S)

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7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)

Bureau of Reclamation
Technical Service Center
Water Resources Services
Water Resources Research Laboratory
Denver, Colorado

8. PERFORMING ORGANIZATION REPORT NUMBER

R-02-01

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)

Same

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES

12a. DISTRIBUTION/AVAILABILITY STATEMENT

12b. DISTRIBUTION CODE

13. ABSTRACT (Maximum 200 words)

The Bureau of Reclamation's Water Resources Research Laboratory conducted a 1:30-scale model study of the San Sevaine side-weir diversion structure and the associated Jurupa Basin inlet/energy dissipator. The purpose of the study was to evaluate the diversion performance of the side-weir design under the supercritical flow conditions expected in the prototype. The scope of the study also included an evaluation of the flow conditions in the vicinity of the Jurupa Basin inlet/energy dissipator so that the project designers could assess the scour potential for that portion of the basin. Model testing determined that the original 2800-ft-long side-weir design was more efficient than anticipated and could be shortened to 2440 ft and still meet the diversion goals. Chamfering the top of the side weir further increased its efficiency, allowing an additional reduction in length to 2340 ft. Velocity data was collected on a 30 ft grid pattern in a 240 ft by 180 ft area around the Jurupa Basin inlet/energy dissipator structure for four different inflow/detention pool elevation combinations. This data was used by the design team to assess the scour protection requirements of the basin floor.

14. SUBJECT TERMS

a. DESCRIPTORS–hydraulic model, side weir, diversion, supercritical flow, energy dissipator

b. IDENTIFIERS–San Sevaine, San Bernardino County, Jurupa Basin

15. NUMBER OF PAGES

52

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
UL	UL	UL	UL

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UNITED STATES DEPARTMENT OF THE INTERIOR ★ BUREAU OF RECLAMATION

ACKNOWLEDGMENTS

This hydraulic model study was conducted at the request of the San Bernardino County Flood Control Department. The study was conducted in conjunction with the ongoing efforts of the County's engineering design firm, Boyle Engineering, Inc. Thomas Boyd and Terry Bowen of that firm were instrumental in providing design data for the modeling effort, and integrating the model study results into the design process. Jerry R. Fitzwater and Neal L. Armstrong of the Bureau of Reclamation's Water Resources Research Laboratory were responsible for devising and implementing the overall design concept for the physical model.

Mission Statements

U.S. Department of the Interior

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to tribes.

Bureau of Reclamation

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

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Overview

San Sevaine Creek originates in California's San Gabriel Mountains and flows south through San Bernardino County and into Riverside County where it is tributary to the Santa Ana River. Although San Sevaine Creek is an ephemeral stream, flood flows in the channel can cause damages in both San Bernardino and Riverside Counties. To help minimize potential flood damages along San Sevaine Creek, the County of San Bernardino has undertaken a program of channel and flood-control improvements for San Sevaine Creek. These improvements include lining the channel with concrete and creating or upgrading a series of detention basins to temporarily store excess flood flows to reduce the peak discharge in the stream.

Jurupa Basin is the last San Sevaine Creek detention basin in San Bernardino County, and is located approximately 1 mile upstream of the Riverside County line. To divert excess discharge from San Sevaine Creek into Jurupa Basin, the County's design engineering firm, Boyle Engineering, Inc. (Boyle), recommended a 2800-ft-long, 6-ft-tall side weir. When flow depths in the San Sevaine channel are less than 6 ft, all flow will remain in the main channel and pass downstream towards Riverside County. When flow depths exceed 6 ft, a portion of the main channel discharge will spill over the side weir into a collector channel where it will be directed into Jurupa Basin to be stored and subsequently released back into the main channel after the flood peak has passed. The design flow-split criteria for the side weir requires that for a main channel 100-year peak discharge of 20,400 ft^3/sec, 9,200 ft^3/sec be diverted to Jurupa Basin with 11,200 ft^3/sec remaining in the main channel. The 11,200 ft^3/sec remaining in the main channel will combine with outflows from Jurupa Basin plus local runoff to yield a peak discharge of 12,600 ft^3/sec at the Riverside County line. Since flood discharges in the improved San Sevaine channel are expected to be supercritical and the performance of side weirs in supercritical flow conditions is not well understood, independent reviewers recommended that the side-weir design be physically modeled to verify that it satisfied this flow-split criteria.

At the request of the County of San Bernardino, a model study of the San Sevaine diversion side weir and the inlet structure for Jurupa Basin was conducted at the Bureau of Reclamation's Water Resources Research Laboratory (WRRL) in Denver, Colorado. The primary purpose of the model study was to evaluate the discharge characteristics of the original 2800-ft-long weir design for the supercritical flow conditions expected to occur in the San Sevaine channel. If the flow-split criteria for the design 100-year discharge was not met, alternative weir designs were to be considered. A secondary purpose of the model study was to assess flow velocities in Jurupa Basin in the vicinity of the basin inlet/energy dissipator so that the project designers could better evaluate the potential for scour in the basin and incorporate appropriate protection.

The model study was conducted using a 1:30-scale physical model of the 2800-ft-long side weir and contiguous portions of the main channel, as well as the collector channel and Jurupa Basin inlet structure. The scale of the model was chosen to focus on the design discharge of 20,400 ft^3/sec within the limits of laboratory space while accurately reproducing the frictional resistance effects of the San Sevaine channel. Data collected from the model include discharge measurements using the WRRL's venturi flow-metering system and calibrated sharp-crested weirs, velocity measurements using a Sontek acoustic Doppler velocimeter (ADV), and depth measurements using a point gage.

Prior to model construction, the proposed model material (permaply, a smooth-sided marine plywood) was tested in an adjustable-slope flume to evaluate its effective roughness. These tests indicated that the

Darcy-Weisbach friction factor for the model material tended to follow the "hydraulically smooth" curve of the Moody diagram. For the range of Reynolds numbers expected to be encountered in the model, the roughness of the model channel would then correspond to a Manning's "n" value of approximately 0.0146 in the prototype, slightly larger than the estimated design value of 0.014. Discussions with representatives of San Bernardino County indicated that this was acceptable since the anticipated effects of weathering would tend to increase the roughness, and a larger roughness value was more conservative from a dam safety viewpoint since it would lead to more flow to Jurupa Basin.

Model Testing and Results

Testing of the San Sevaine side weir model began in the fall of 1997. Initial weir performance tests were conducted using a 1-ft-thick quasi-rectangular shape for the weir crest, as specified in the preliminary design information provided by Boyle. The crest shape was then modified to incorporate 3 ½-inch, 45-degree chamfers along the crest edges based on discussions with representatives of Boyle and the County, and the weir performance tests repeated. These tests yielded several important observations and conclusions.

- For the design discharge of 20,400 ft^3/sec, the initial 2,800 ft weir design passed 9,800 ft^3/sec to Jurupa Basin, 6.5 percent more than the required 9,200 ft^3/sec.
- At the design discharge, the upstream end of the weir passed significantly more water than the downstream end (i.e., 50 percent of the required diversion of 9,200 ft^3/sec occurred in the first 19.6 percent (550 ft) of the 2,800 ft weir length).
- For the rectangular crest shape, only the first 2,440 ft of the weir length was necessary to pass the required 9,200 ft^3/sec, resulting in a potential savings of 360 ft of weir construction.
- Chamfering the weir crest increased the efficiency of the weir, allowing the necessary weir length to be reduced by another 100 ft to 2,340 ft. The average coefficient of discharge for the chamfered weir crest was 4.1 for the range of Froude numbers and flow depths evaluated.

These results were obtained for a constant weir height of 6 ft, a main channel and top-of-weir slope of 0.010517 ft/ft, and a linearly varying main channel width starting at 60 ft and decreasing at a rate of 24.5 ft per mile of weir length.

Numerical simulations indicated that the length of the side weir could be further reduced to 1400 ft and still obtain the desired flow split at the design discharge. This would involve varying the height of the side weir along the weir length. This option was not selected because the total diverted flow volume would consequently be increased.

Following the weir performance tests, a series of tests were conducted in the spring of 1998 to assess flow velocities in the vicinity of the Jurupa Basin inlet structure. These tests documented flow velocities observed in the model for four reservoir inflow/pool elevation combinations. The results from these tests demonstrated that if the reservoir was operated to maintain very low pool elevations (e.g., 900 ft) with high inflows (e.g., 8,950 ft^3/sec), then flow velocities as high as 32 ft/sec could occur for distances of 150 ft or greater downstream of the energy dissipator.

PURPOSE

This report documents the results of a physical model study which was conducted for the County of San Bernardino, California. The purpose of the study was to confirm the hydraulic performance of components of a flood-control system which was being designed for diversion of excess San Sevaine Creek flows into Jurupa Basin. Specific objectives of the study included:

- Evaluating the flow-splits between the San Sevaine channel and the diversion side weir for the initial 2800 ft weir design.
- Modifying the side-weir design to achieve the required flow splits if the original design did not meet them.
- Evaluating the performance of the energy dissipator for the Jurupa Basin Inlet Works.

INTRODUCTION

The San Sevaine Creek Water Project is a Bureau of Reclamation "Small Project" under PL 84-984 (SRPA). The project provides environmental enhancements, water conservation, and flood control facilities in the western portion of the San Bernardino Valley in California.

San Sevaine Creek originates in the San Gabriel Mountains and flows south through San Bernardino County and into Riverside County where it is tributary to the Santa Ana River. Throughout much of the year the channel bed is dry, however flood discharges in the San Sevaine channel can cause damages in both counties. In order to minimize flood damages, a series of flood control improvements have been proposed along San Sevaine Creek in San Bernardino County. These improvements include upgrading the channel to a rectangular, concrete-lined shape, and creating or improving a series of detention basins to temporarily store excess runoff and flood flows.

Jurupa Basin

Jurupa Basin is the last detention facility along San Sevaine Creek in San Bernardino County. It is located along the east side of the San Sevaine channel approximately 1 mile upstream of the Riverside County line, near the intersection of Jurupa and Mulberry Avenues in Fontana, California (figure 1). The basin has a designed storage capacity of approximately 1,500 acre-ft. Hydrologic analysis indicates that at a point just upstream of Jurupa Basin the peak discharge in San Sevaine Creek for the 100-year storm is 20,400 ft^3/sec. By agreement, San Bernardino County is seeking to limit the maximum discharge which San Sevaine Creek passes into Riverside County to 12,600 ft^3/sec for the 100-year storm. Thus, excess discharge from San Sevaine Creek must be diverted into Jurupa Basin and stored until after the flood peak has passed.

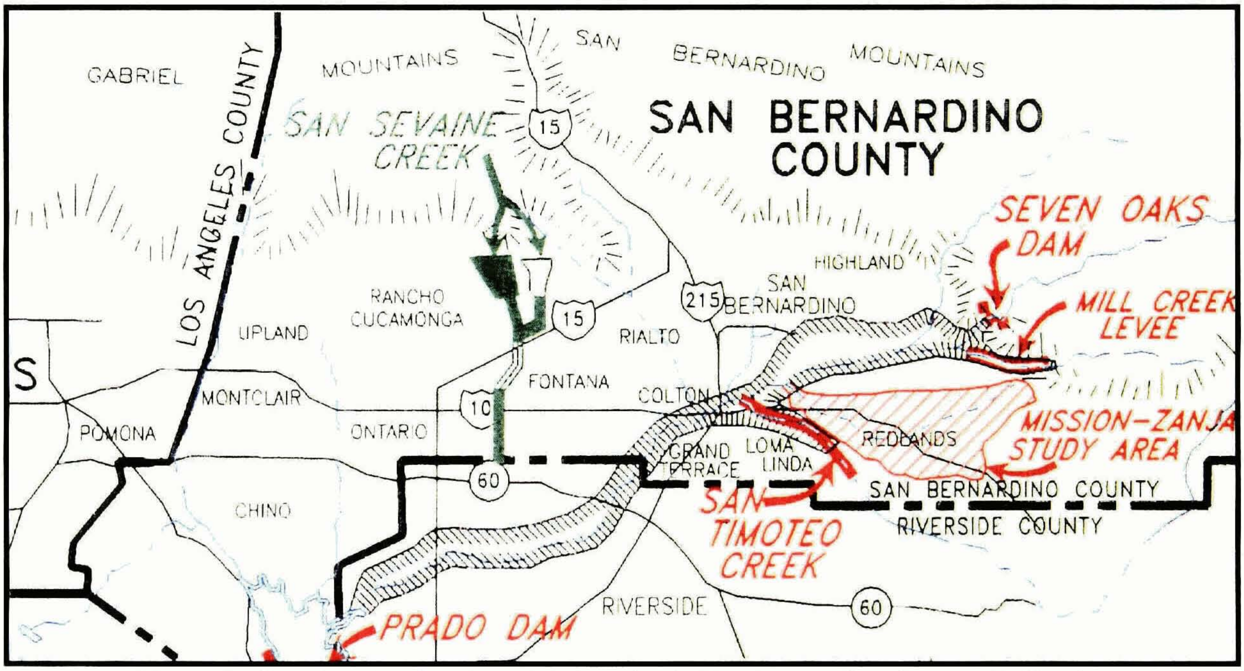

Figure 1.–San Sevaine Creek project location in San Bernardino County, California.

San Sevaine Side Weir

In order to accomplish the diversion of excess flood flows from the San Sevaine channel into Jurupa Basin, the County's design engineering firm, Boyle Engineering, Inc., recommended that a 2800-ft-long, 6-ft-high side weir be constructed along the left side of the San Sevaine channel. The slope of the San Sevaine channel in the vicinity of the weir would be 0.010517 ft/ft. The width of the channel upstream of the weir would be 60 ft, and along the length of the weir the channel width would decrease linearly to 47 ft (figure 2). Using this concept, low flows in the San Sevaine channel (i.e. those with a depth of flow less than 6 ft at the downstream end of the weir) would remain in the channel, and pass directly downstream into Riverside County. When discharge in the channel increased to the point where the weir was overtopped, then some of the discharge in the main channel would spill over the side weir into a 34-ft-wide collector channel with a slope of 0.013729 ft/ft which would conduct the overflow to Jurupa Basin.

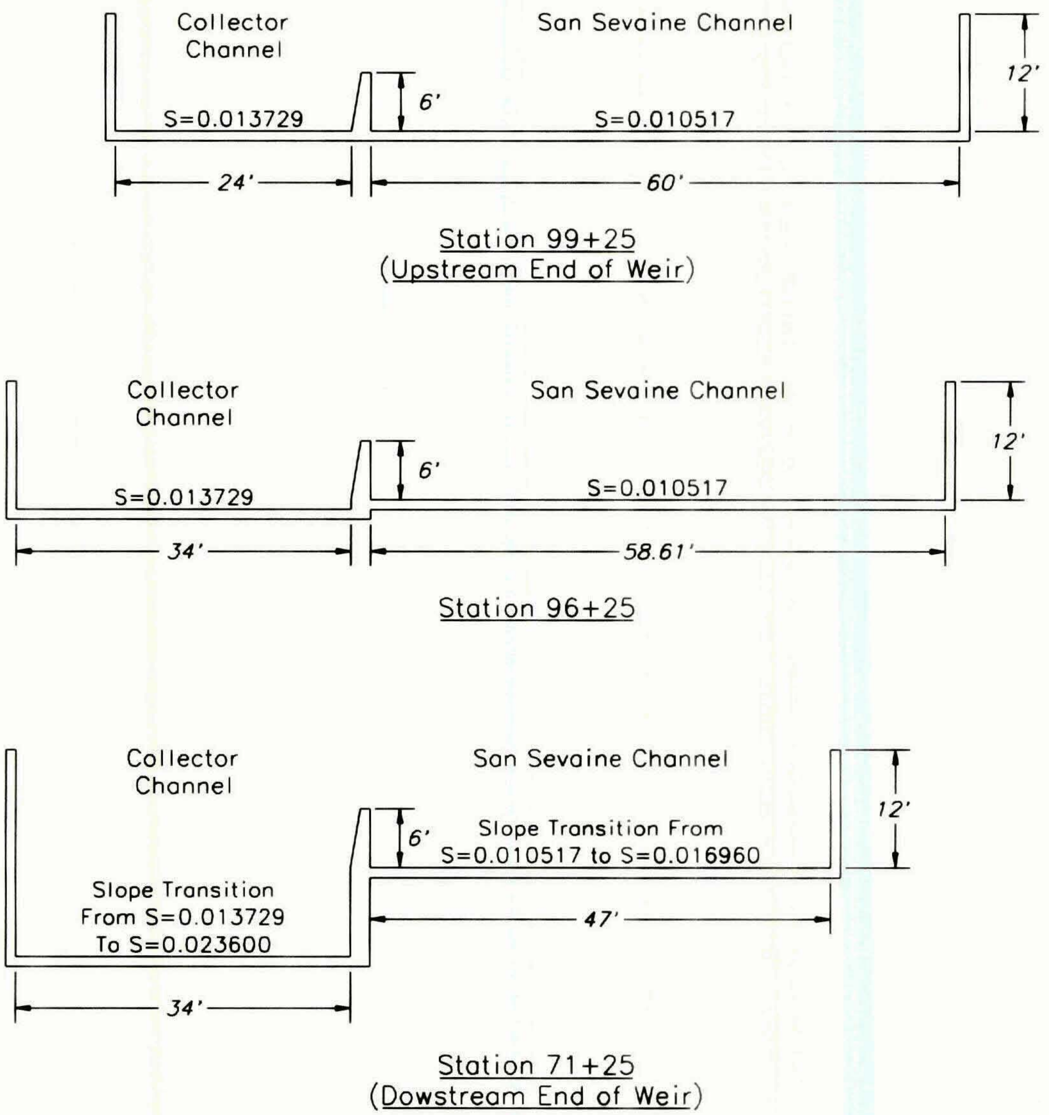

Figure 2.–Section view of the San Sevaine and collector channels at various stations.

The design flow-split requirements for the side weir called for 9,200 ft^3/sec of the 100-year storm's peak discharge of 20,400 ft^3/sec to be diverted over the side weir and into storage at Jurupa Basin. The remaining 11,200 ft^3/sec would pass downstream where contributions from local inflow and Jurupa Basin releases would bring the total discharge up to the allowable maximum of 12,600 ft^3/sec at the Riverside County line.

Due to the slope and concrete lining of the San Sevaine channel, flood discharges in the channel are expected to be supercritical in nature. Hydraulic performance of side weirs under supercritical flow conditions is not well understood. For this reason, independent reviewers of Boyle Engineering's side-weir plans recommended that the side-weir design be physically modeled to confirm the performance of the system.

Jurupa Basin Inlet

As part of the flood control improvements to the San Sevaine system, the existing Jurupa Basin is being enhanced, including the addition of new inlet and outlet works. Flood-flows diverted from San Sevaine channel are designed to enter Jurupa Basin through a planned 35-ft-wide, concrete-lined inlet/energy dissipation structure. The inlet channel will drop into the basin at a slope of 0.1580 and terminate in a plunge pool/slotted roller-bucket-style energy dissipator with a bottom elevation of 872.0 ft and an end sill elevation of 894.5 ft (figure 3). The vertical concrete side-walls of the energy dissipator have a top elevation of 911.5 ft (maximum height 39.5 ft). The maximum design water-surface elevation for Jurupa Basin is 930.0 ft, and the anticipated bed elevation in the vicinity of the energy dissipator is 894.5 ft.

Geotechnical investigations indicate that the Jurupa Basin bed material is composed primarily of noncohesive sands and gravels. In order to evaluate the potential for scour around the proposed energy dissipation structure, the County of San Bernardino, through their engineering design firm, Boyle Engineering, requested that the Bureau of Reclamation's Water Resources Research Laboratory (WRRL) assess flow velocities in Jurupa Basin in the vicinity of the structure.

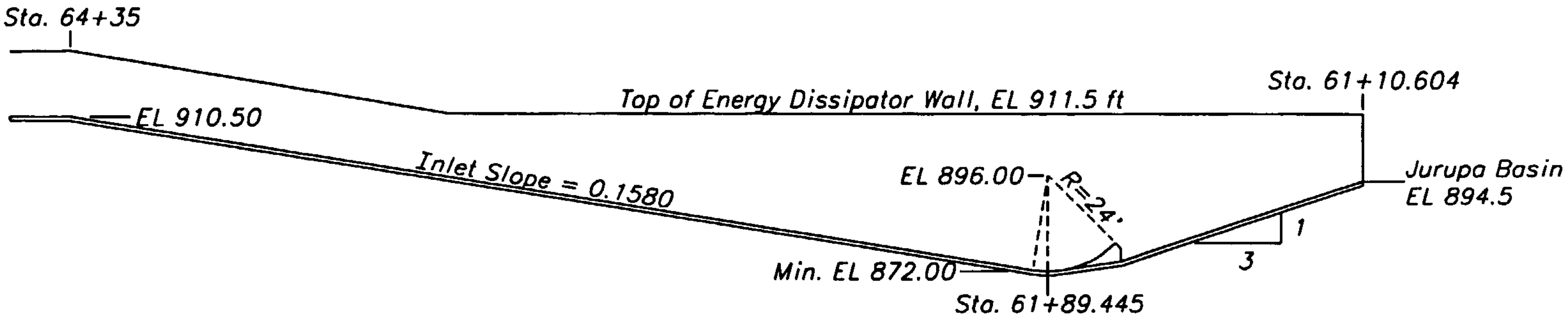

Figure 3.–Profile sketch of Jurupa Basin inlet/energy dissipator.

PHYSICAL MODEL

To study the performance of the San Sevaine side weir and Jurupa Basin inlet, a 1:30-scale physical model was designed and constructed at the Bureau of Reclamation's Water Resources Research Laboratory in Denver, Colorado (figures 4 and 5). The model scale was chosen to focus on the San Sevaine channel design discharge of 20,400 ft^3/sec while conforming to the laboratory space constraints and maintaining similar resistance effects in both model and prototype. The limits of the model were selected to incorporate representations of the 2,800-ft-long side weir and the adjacent San Sevaine channel, the collector channel, and a portion of Jurupa Basin centered around the inlet structure (figure 6).

Similitude and Scaling

Froude similarity was chosen to scale the model parameters due to the dominance of gravitational forces in the open-channel flow processes being investigated. For a 1:30-scale model, scaling according to the Froude criteria produces the following relationships between model and prototype parameters:

$L_r = 1{:}30$ (Length ratio)

$V_r = L_r^{1/2} = 1{:}5.477$ (Velocity ratio)

$Q_r = L_r^{5/2} = 1{:}4{,}929.5$ (Discharge ratio)

$T_r = L_r^{1/2} = 1{:}5.477$ (Time ratio)

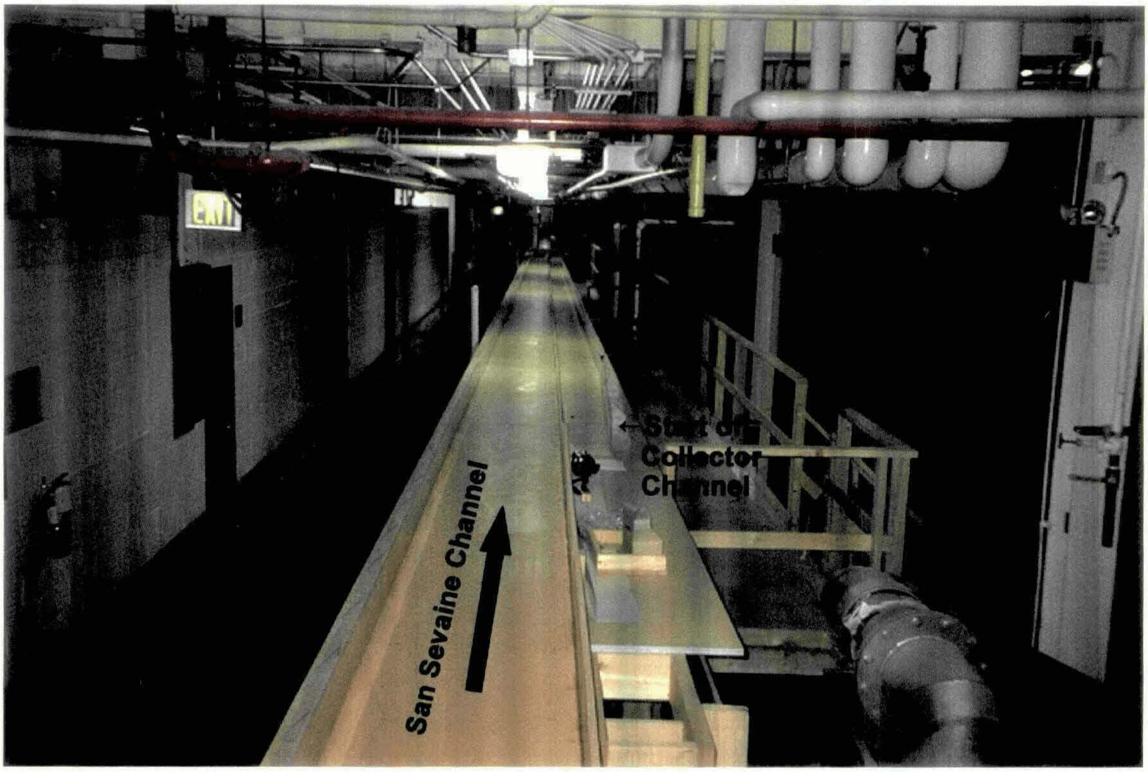

Figure 4.–1:30-scale model of San Sevaine channel and side weir, looking downstream.

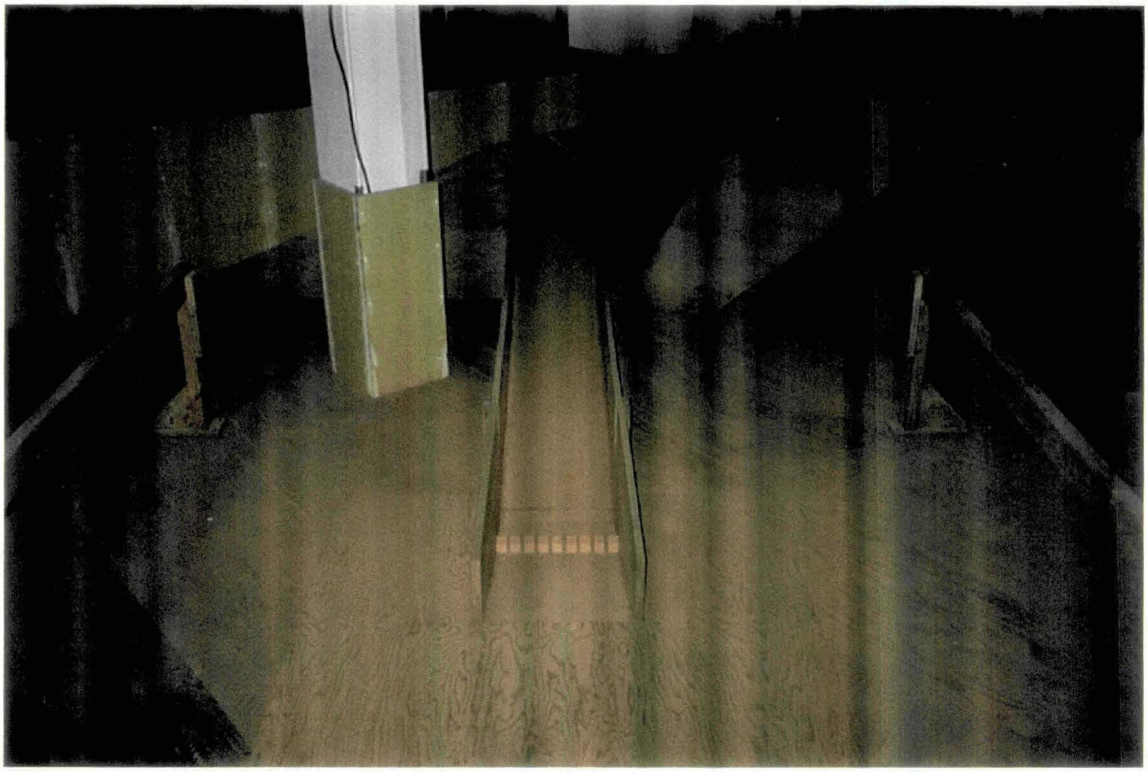
Figure 5.–1:30-scale model of Jurupa Basin inlet/energy dissipator, looking upstream.

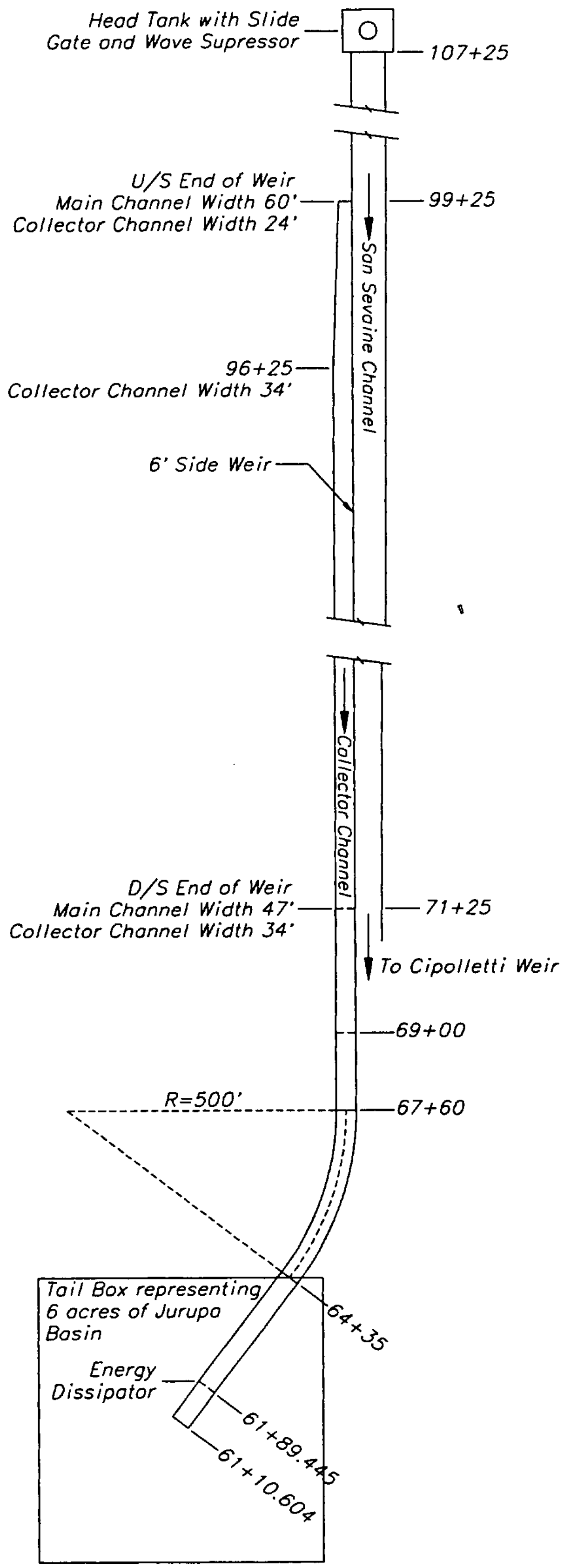

Figure 6.–Plan view of San Sevaine/Jurupa Basin model layout.

Construction

The design layout for the side-weir diversion to Jurupa Basin calls for the 6-ft-high side weir to be located along the left side (when viewed looking downstream) of the San Sevaine channel. Due to space limitations in the laboratory, the model was constructed in mirror-image, with the side-weir and diversion channel located along the right side of the San Sevaine channel. This modification had no physical impact on the hydraulic properties being evaluated in the model.

The San Sevaine and diversion channels were constructed using 3/4 inch perma-ply marine plywood for the floors and 3/8 inch plexiglass for the side walls. The two channels were separated by a polypropylene plastic insert representing the 6-ft-high side weir (figures 7 & 8). Although supported on common leg stands, each of the channels was constructed independently such that the slopes of the channels could be adjusted should the design team elect to test alternative bed slopes for either or both channels.

Flow to the model was supplied through the 12-in-diameter laboratory piping. A 3-ft-wide by 2.5-ft-diameter steel head tank was used to transition the flow from the laboratory piping system into the 2-ft-wide (60 ft prototype) San Sevaine approach channel (figure 9). A slide gate on the head tank was used to control the flow depth entering the model in order to achieve the appropriate uniform flow depths at the start of the side weir located 26 ft 8 in (800 ft prototype) downstream of the head tank. An adjustable underpass wave suppressor was constructed in the San Sevaine approach channel immediately downstream of the head tank to moderate the water-surface fluctuations of the flow emanating from the slide gate (figure 10).

The model San Sevaine channel terminated in a free overfall 2 ft (60 ft prototype) downstream of the end of the 93 ft 4 in-long (2,800 ft prototype) side weir (figure 11). The diversion channel entered a 17 ft by 17 ft tail box representing 6 acres of Jurupa Basin and terminated in a slotted-bucket energy dissipator located approximately in the center of the tail box (figure 5). Water levels in the tail box representing various Jurupa Basin pool elevations were controlled with a series of tail-board slats at the outflow of the box (figure 12).

Inflow to the model was measured using a bank of calibrated venturi meters that are built into the laboratory piping system. Undiverted flow remaining in the San Sevaine channel downstream of the side weir dropped into a collection box, passed through a baffle, and was measured with a 2 ft calibrated Cipolletti weir (figure 13). Diversion flows into Jurupa Basin were determined by the difference of these flows.

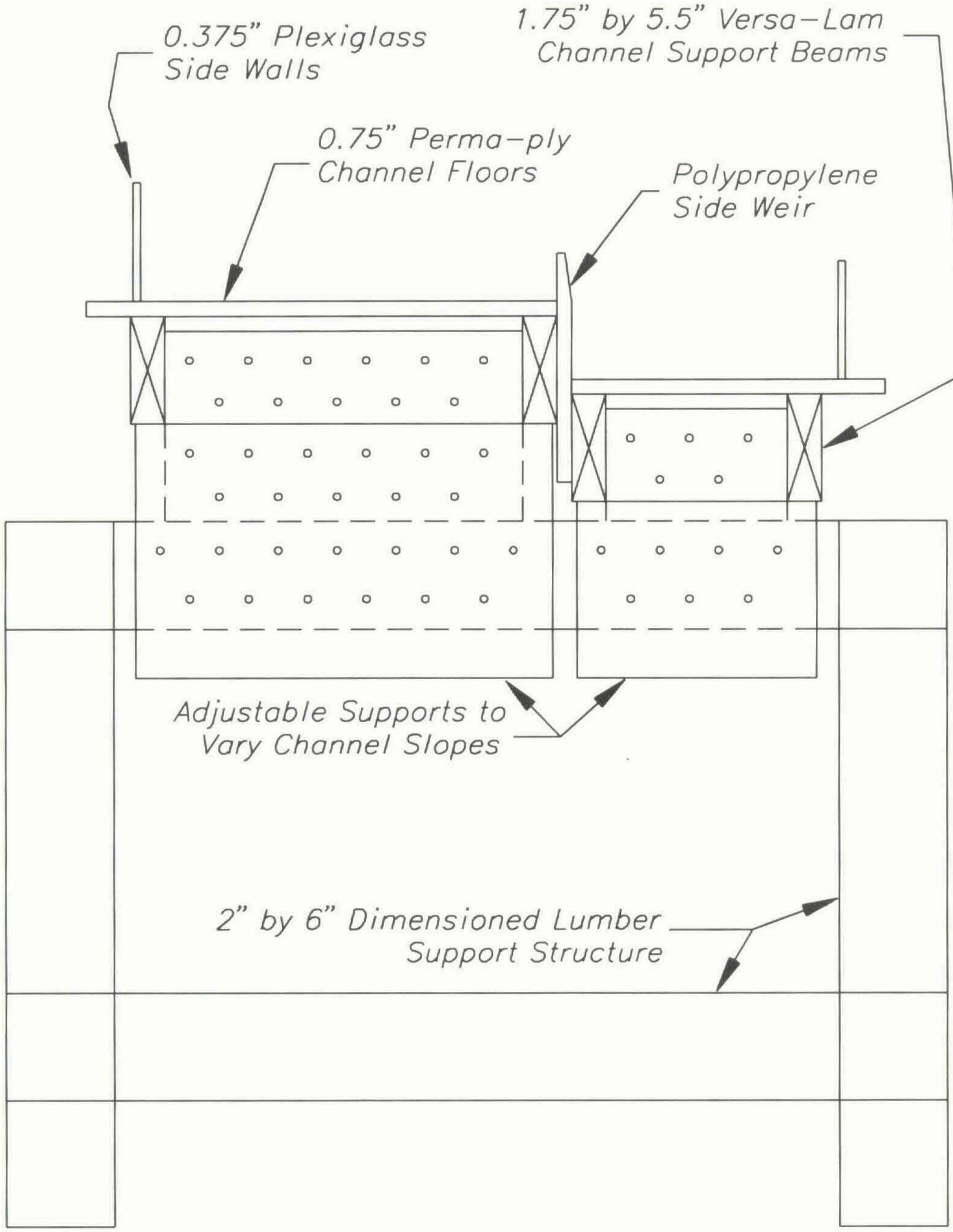

Figure 7.–Section view sketch of model channels, weir, and support structure.

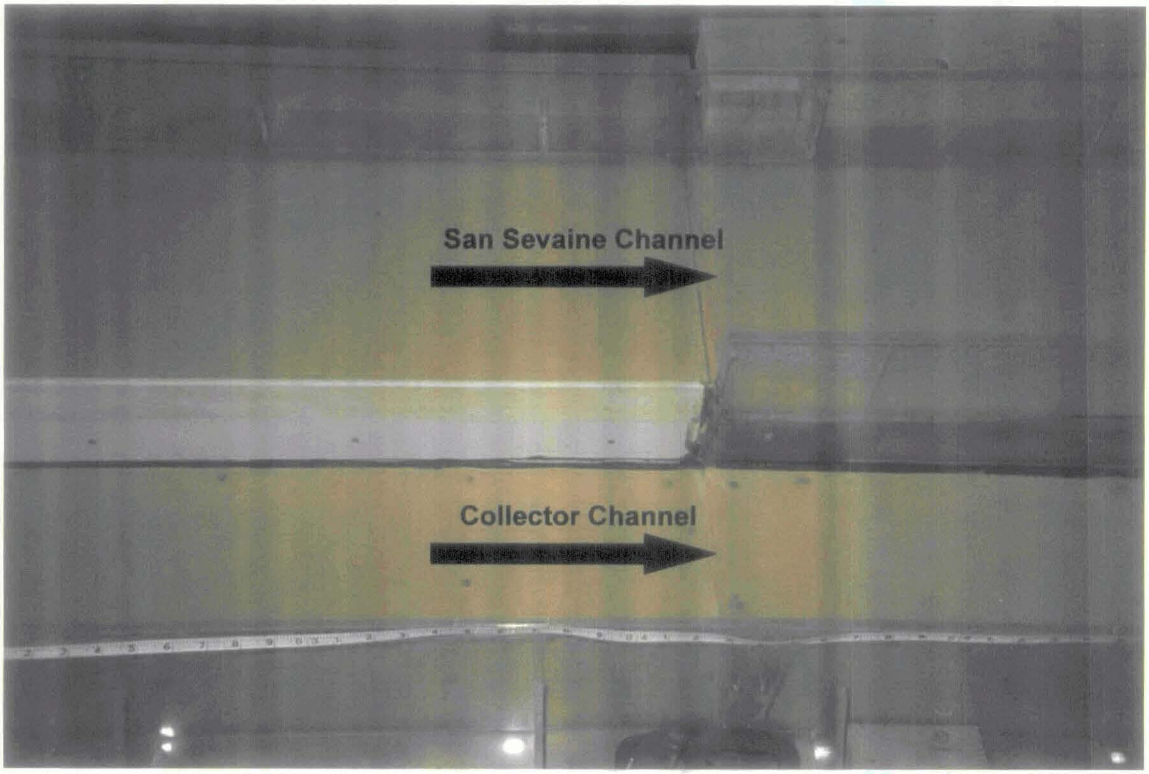

Figure 8.–Model San Sevaine and collector channels at downstream end of side weir.

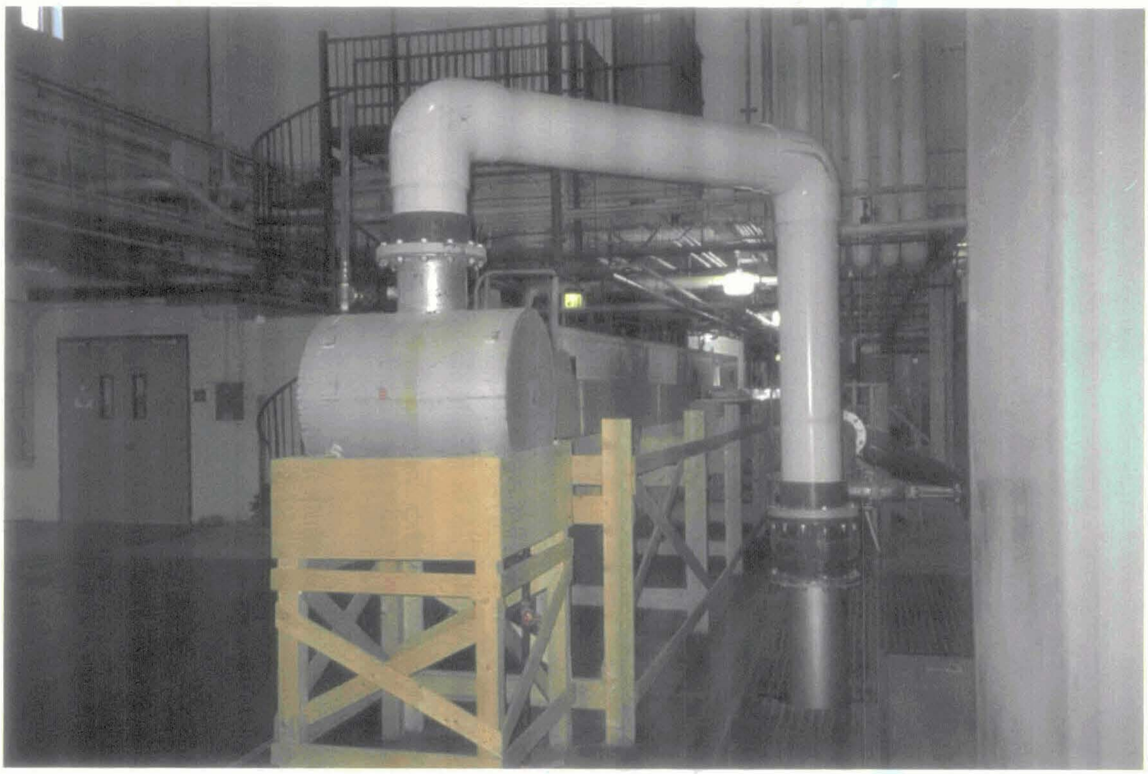
Figure 9.–Model inflow piping and head tank.

Figure 10.–Model head tank with slide gate and wave suppressor.

Figure 11.–Free overfall termination of model San Sevaine channel with collector channel curving into model Jurupa Basin on the right.

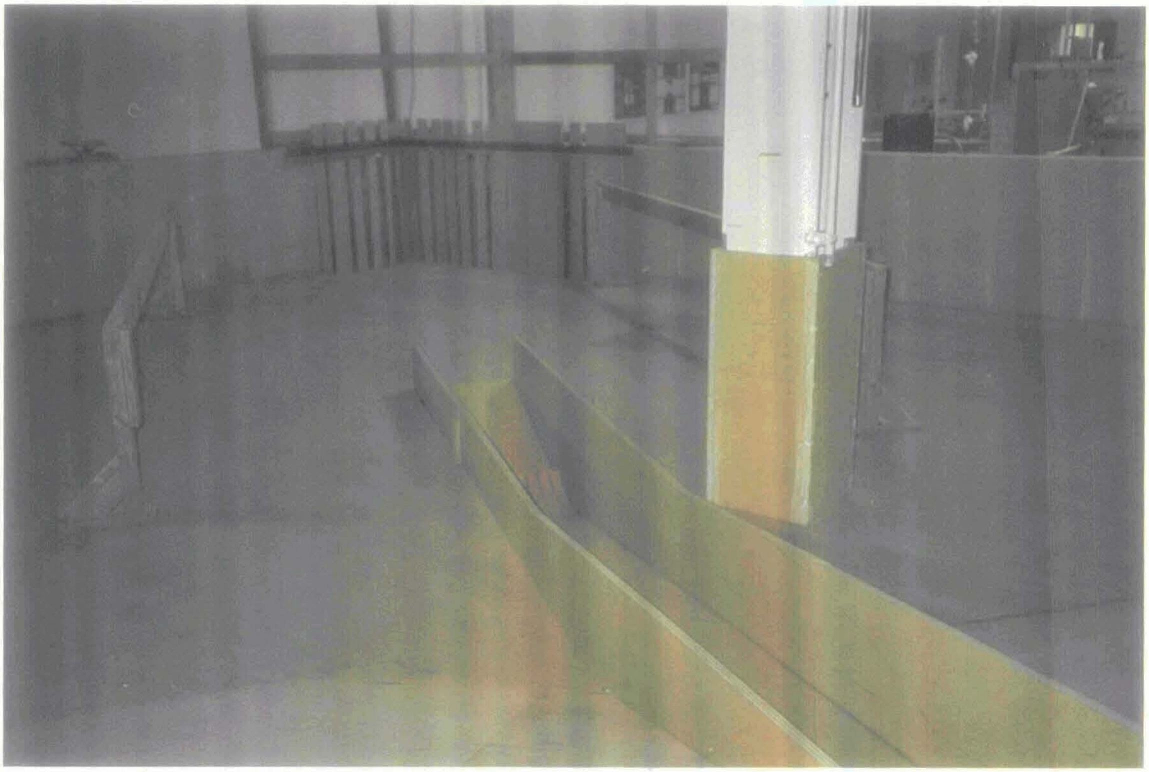

Figure 12.–Model Jurupa Basin with outflow tailboards for water level control.

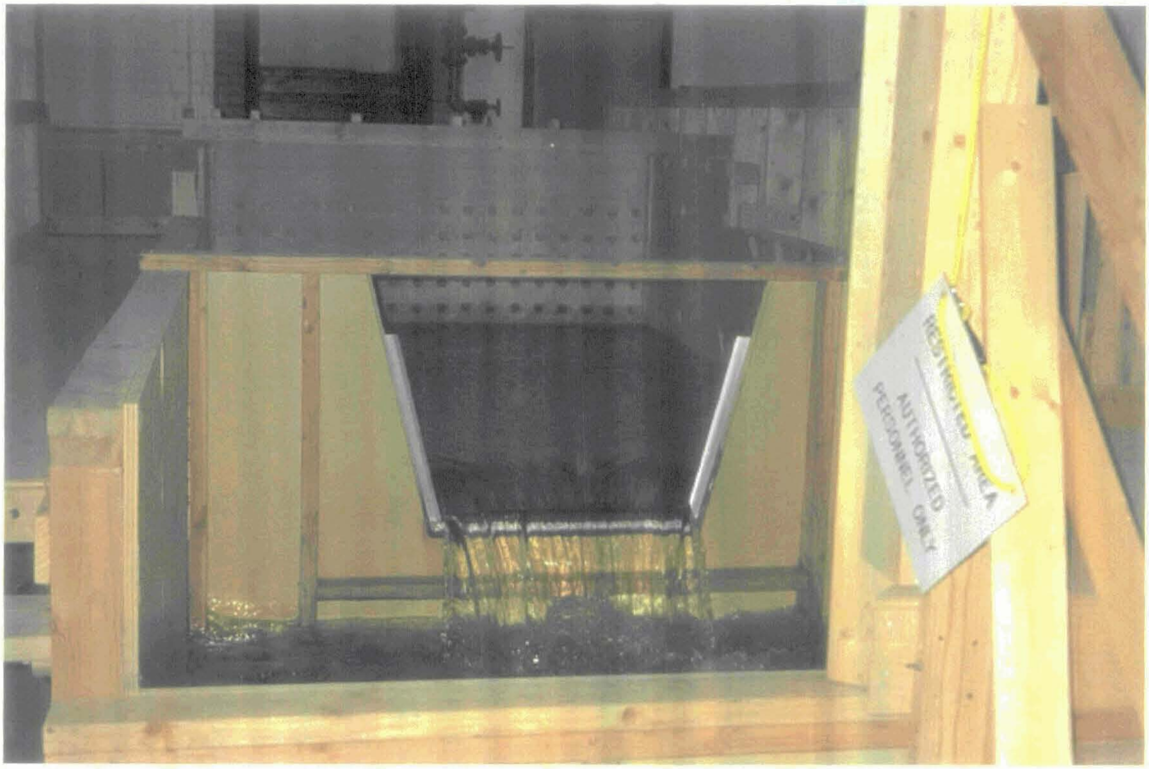

Figure 13.–Calibrated 2 ft Cipolletti weir for measurement of undiverted San Sevaine flows.

Roughness Scale Effects

Flow over a weir is largely dependent upon the depth of flow upstream of the weir relative to the weir crest. To properly simulate flows over a side weir in a model, it is necessary to properly simulate both the velocity and the depth of flow in the main channel. This requires that the roughness effects in a scale model properly reproduce the roughness effects of the prototype. To achieve this goal requires that the Darcy-Weisbach friction factor (f) be the same in both model and prototype.

The Darcy-Weisbach friction factor is a function of both the relative roughness of the boundary surface material and the Reynolds number of the flow. Since flow in a Froude-scale model typically has a lower Reynolds number than flow in the prototype, the model boundary roughness must consequently be smoother to achieve the same friction factor (unless the model is operated with a high enough Reynolds number to be in the fully rough zone, where similar relative roughness values will yield the same friction factor regardless of the difference in Reynolds numbers).

For the prototype San Sevaine channel the design Manning's n value of 0.014 would yield a uniform flow depth of 8.7 ft in the 60-ft-wide channel upstream of the side weir for the 100-year design discharge of 20,400 ft^3/sec. This corresponds to a Reynolds number of $8.8x10^7$ and a friction factor of 0.012. To determine the friction factor of the model material (a resin-coated plywood referred to as perma-ply), a series of preliminary tests were conducted in the laboratory using an adjustable-slope flume with perma-ply affixed to the floor of the flume. In these tests, flow depths were measured at 10 stations along a 50 ft length of the flume (figure 14) for several different flow rates and bed slopes. These tests confirmed that the model bed material was "hydraulically smooth." At the Reynolds number of the model design flow ($5.3x10^5$) a hydraulically smooth boundary material would yield a friction factor of 0.013, slightly rougher than the prototype. In terms of Manning's n, this meant that the model material would simulate roughness effects similar to a prototype Manning's n of 0.0146.

The roughness scale effects were discussed with San Bernardino County representatives during a meeting on April 9, 1997, prior to construction of the model. Slightly increasing the model bed slope to use additional gravitational impetus to offset the extra resistance effects of the model was considered at that time. Ultimately the County personnel decided to use the model with the design bed slope, arguing that this would provide more conservative results from a dam safety perspective since it would result in slightly deeper channel flows and thus more water diverted to the detention basin where storage volume was critical. Thus the model was constructed with the understanding that the model would more nearly simulate a Manning's n value of 0.0146 rather than the preliminary design value of 0.014.

Figure 14.–Preliminary model material roughness tests in laboratory adjustable-slope flume.

MODEL TESTING

Overview

Tests were conducted using the physical model to achieve two primary goals. The first goal was to evaluate the hydraulic performance of the side-weir design. Hydrologic analysis indicated that the peak San Sevaine channel discharge resulting from the 100-year storm was expected to be 20,400 ft^3/sec at the Jurupa Basin diversion site. To stay below the maximum allowable discharge of 12,600 ft^3/sec at the Riverside County line, design specifications called for a diversion structure capable of removing 9,200 ft^3/sec of the peak discharge from the San Sevaine channel, leaving 11,200 ft^3/sec in the channel to which local inflows as well as Jurupa Basin outflows would be added before reaching the county line. Should the preliminary side-weir design fail to meet this flow-split requirement, alternative weir configurations would be evaluated until an acceptable design was found.

The second goal of the testing program was to evaluate the performance of the energy dissipator for the Jurupa Basin Inlet Works. Geotechnical investigations indicated that the Jurupa Basin bed material was composed primarily of noncohesive sand and gravel material. Due to the potential for scour in this material, it was requested that flow velocities in the vicinity of the Jurupa Basin energy dissipator be evaluated in the model. This information would be used by the project designers as part of their assessment of scour potential and appropriate scour protection.

Side-Weir Tests

The hydraulic performance of the San Sevaine side weir was evaluated using two different series of tests. The first series of tests involved the development of a rating curve of side-weir discharges versus San Sevaine channel flows. This would allow the project designers to evaluate the side-weir performance throughout the range of flows which would be encountered during an hydrograph of unsteady channel inflows. To accomplish this goal, a series of steady discharges were passed down the model San Sevaine channel and for each discharge the amount of flow remaining in the channel downstream of the side weir was measured using a calibrated 2-ft Cipolletti weir. The corresponding amount of flow passed over the side weir was then determined by difference. The lowest discharge in the series was the largest channel flow which could be contained in the channel without overtopping the side weir. This became the point of zero overflow for the side-weir rating. The remaining inflow discharges used to develop the rating curve corresponded to San Sevaine channel flows of 10,000, 12,000, 14,000, 16,000, 18,000, 20,000, and 20,400 ft^3/sec. For each discharge tested, the settings for the slide gate and wave suppressor at the inflow to the model were carefully calibrated to yield the appropriate normal depth of flow at the upstream end of the side weir.

The second series of tests involved an incremental assessment of side-weir discharges along the length of the side weir for the design peak channel discharge of 20,400 ft^3/sec. This would allow an estimate to be made of how much the original weir design could be lengthened or shortened to meet the design flow-split criteria. The procedure for this series of tests was to block off the entire length of the side weir with lengths of sheet metal, and then incrementally remove measured lengths of the sheet metal beginning at the upstream end of the weir (figure 15). As each successive length of sheet metal was removed the amount of flow passing through the open portion of the side weir was measured. In this manner a relationship for cumulative side-weir discharge versus distance along the weir crest was developed.

Initially, all weir performance tests were conducted using the 1-ft thick, quasi-rectangular crest shape (figure 16) specified in the preliminary design. Experience with other weir performance studies suggested that rounding the weir crest might increase the effective coefficient of discharge for the weir and thus the overall weir efficiency. This would allow a shorter side weir to be used in the final design. Discussions of this possibility with the project designers yielded general agreement with the concept, however concerns were raised regarding the constructability of a rounded weir crest. It was decided that a more realistic and constructable alternative would be to chamfer the crest of the side weir using a 3.5 inch chamfer such as would result from placing a 4-inch by 4-inch piece of lumber ripped diagonally in the corners of the concrete forms. Based on this discussion, it was determined to repeat the series of weir performance tests with a chamfered weir crest (figure 16).

Rectangular-Crest Weir

The testing program for the quasi-rectangular weir crest began with tests to establish the side-weir discharge versus channel discharge rating relationship for the original 2,800-ft long weir design. Through a series of successive trials, it was determined that the highest discharge which could be passed down the San Sevaine channel without overtopping the weir was approximately 7,800 ft^3/sec. This became the zero point on the rating curve. Due to the decreasing width of the San Sevaine channel along the side weir (tapering from 60-ft wide at the upstream end to 47-ft wide at the downstream end) the side weir first overtopped at the downstream end where the unit discharge was highest. As the San Sevaine channel flow increased, the length of weir overtopped increased in the upstream direction. Figures 17, 18, and 19 show longitudinal profiles of depth of flow point gage data taken along the centerline of the San Sevaine channel for cases where the weir was partially overtopped (channel flow of 10,000 ft^3/sec), fully overtopped (channel flow of 12,000 ft^3/sec), and operating under peak discharge conditions (channel flow of 20,400 ft^3/sec), respectively.

Initial rating data indicated that for the design peak discharge of 20,400 ft^3/sec, the preliminary 2,800-ft long side-weir design would pass 9,800 ft^3/sec to Jurupa Basin, 600 ft^3/sec more than the required 9,200 ft^3/sec. Cumulative side-weir discharge versus weir length data (figure 20) were then collected to determine how much the original 2,800 ft weir length could be shortened and still achieve the required flow split. This data indicated that only the first 2,440 ft of weir length were necessary in order to pass 9,200 ft^3/sec to Jurupa Basin for the design peak discharge, a potential savings of 360 ft of side weir and collector channel construction. Side-weir discharge versus San Sevaine channel discharge data was then collected for this shortened weir and plotted along with the original 2,800 ft weir data in figure 21.

Chamfered-Crest Weir

Following the tests on the original weir crest shape, the weir crest was chamfered as previously described and the testing program was repeated with the chamfered crest shape. As expected, the chamfered weir proved to be more efficient, passing 10,000 ft^3/sec over its 2,800 ft length for the peak discharge of 20,400 ft^3/sec, or 2 percent more than the unchamfered weir. Analysis of the cumulative discharge versus weir length data (figure 20) for the chamfered weir indicated that the chamfered weir could be shortened to 2,340 ft in length and still achieve the required flow splits, a potential savings of an additional 100 ft over the unchamfered weir. Side weir discharge versus San Sevaine channel discharge data were collected for both the 2,800- and 2,340-ft long chamfered weirs and plotted alongside the unchamfered weir data in figure 21. The cumulative discharge and flow-split data used to develop figures 20 and 21 are presented in Appendices A and B, respectively.

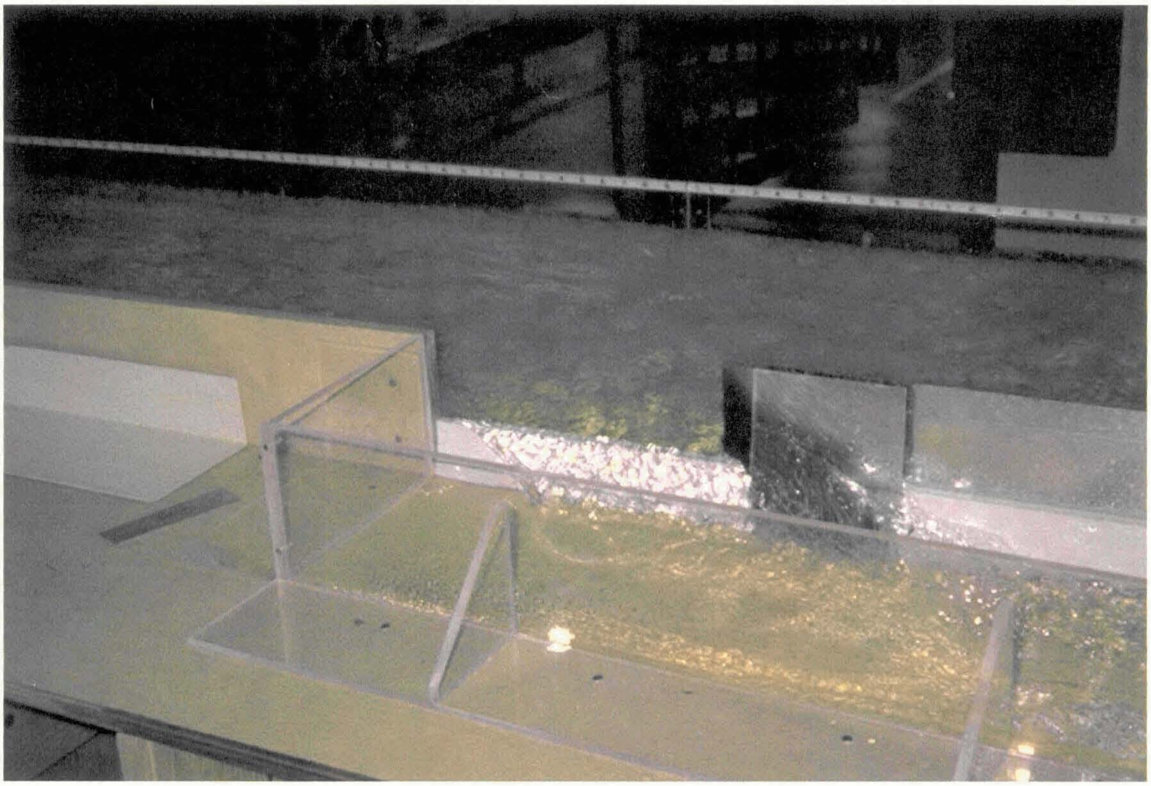

Figure 15.–Cumulative side-weir discharge test configuration.

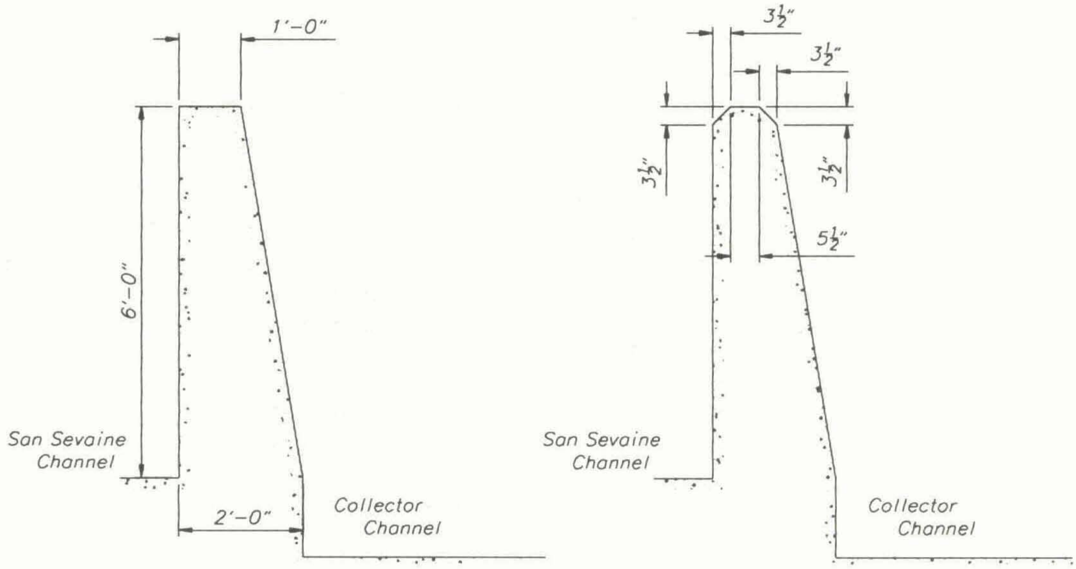

Figure 16.–Rectangular and chamfered side-weir crest shapes evaluated.

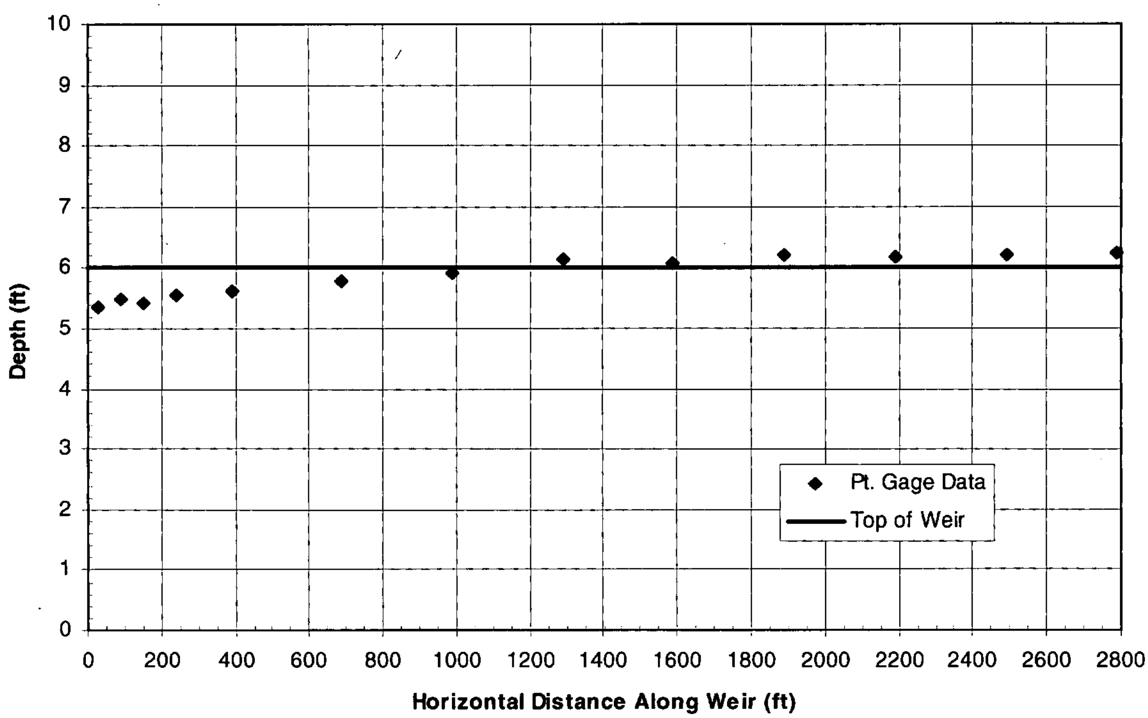

Figure 17.–San Sevaine channel centerline water-surface profile for an inflow of 10,000 ft³/sec.

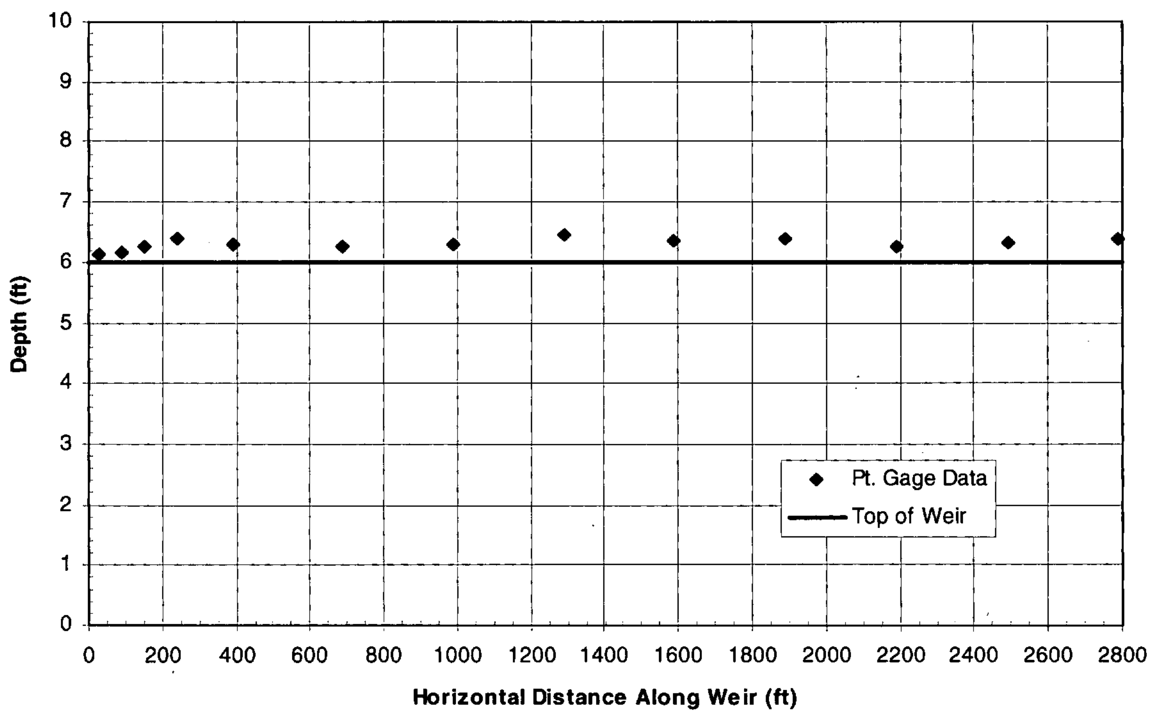

Figure 18.–San Sevaine channel centerline water-surface profile for an inflow of 12,000 ft³/sec.

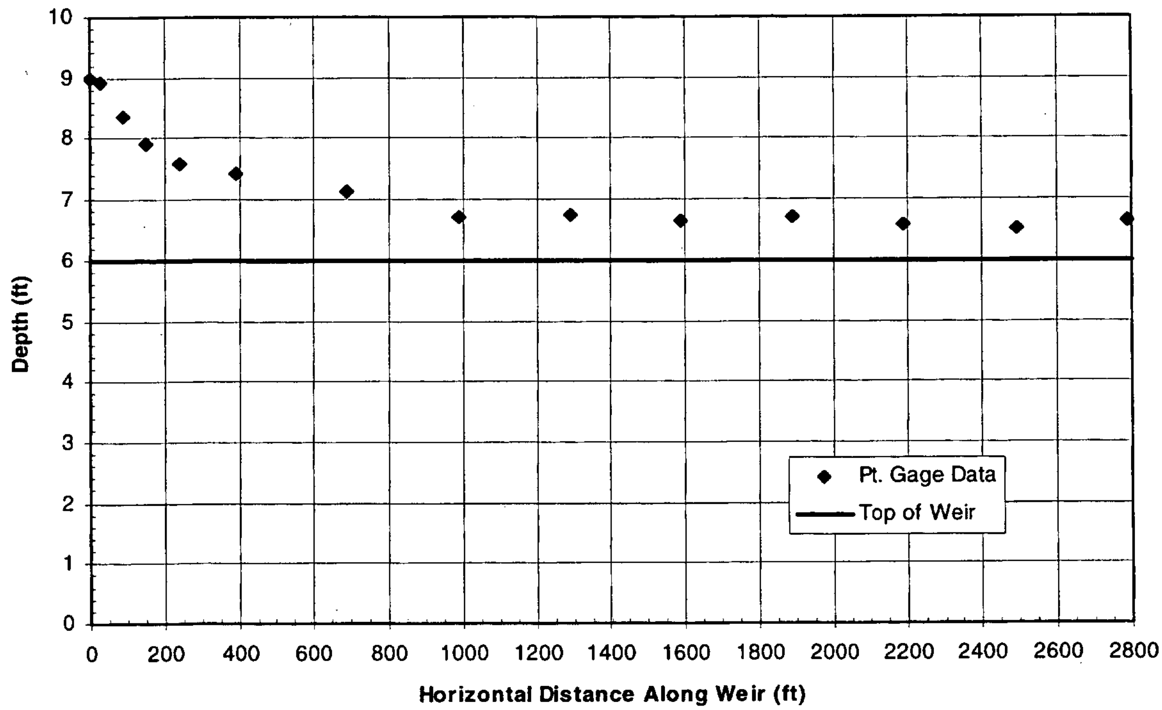

Figure 19.–San Sevaine channel centerline water-surface profile for an inflow of 20,400 ft^3/sec.

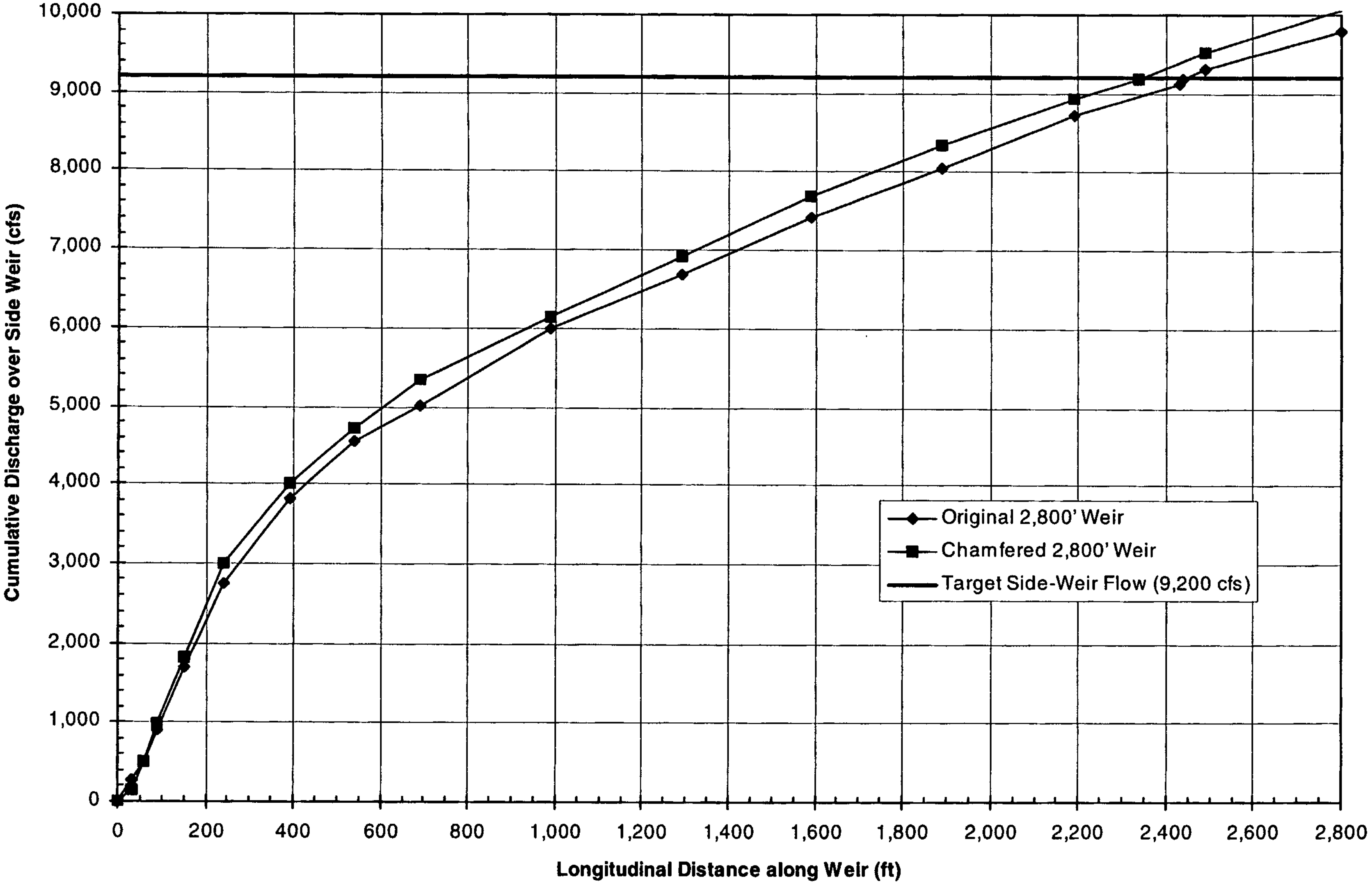

Figure 20.–Cumulative side-weir discharge versus distance along weir for rectangular and chamfered weir crests with an inflow of 20,400 ft^3/sec.

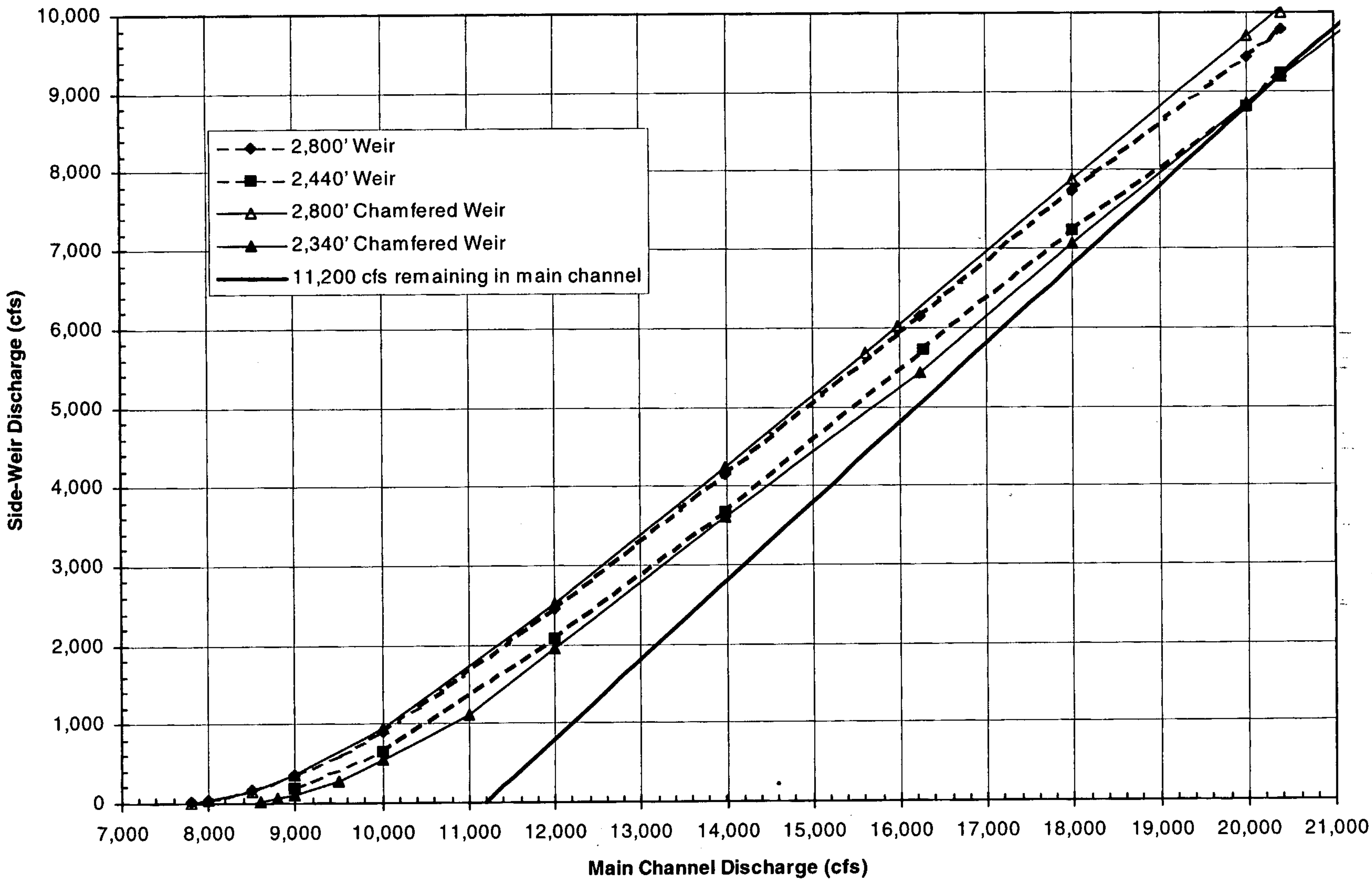

Figure 21.–Inflow discharge versus side-weir discharge ratings for several weir options.

Jurupa Inlet Velocity Tests

Following the side-weir performance tests, a series of tests was conducted to evaluate flow velocities in Jurupa Basin in the vicinity of the basin inlet/energy dissipator. The purpose of these tests was to provide the project designers with information which would aid them in their assessment of scour potential in the vicinity of the inlet structure. To accomplish this goal, velocity measurements were made in the model for four simulated reservoir inflow/pool elevation combinations.

Measurement Methods

Flow velocities in Jurupa Basin were evaluated using a 1:30-scale physical model of the basin inlet/energy dissipation structure and approximately six acres of the surrounding basin. Horizontal (x-y) water velocity components were measured in the model for a variety of flow conditions using a 2D "side-looking" Acoustic Doppler Velocimeter (ADV) manufactured by Sontek. Vertical (z) velocity components were not measured. Velocity data was collected at a rate of 25 Hz for a time period of 60 seconds at each measurement location, resulting in 1500 samples for each position. Each set of 1500 samples was then filtered to remove samples with low correlation coefficients or excessive spiking, resulting in a filtered data set for each location containing some fraction (ranging from 10 percent to 100 percent) of the original 1500 samples. The filtered data set was then used to determine average x and y velocity components for each measurement position. The measured velocities were then adjusted to prototype scale based on Froude similarity (i.e. $v_p = v_m 30^{1/2}$ where v_p is prototype velocity and v_m is model velocity).

Velocity data was collected on a grid composed of 1-ft squares in the model, corresponding to 30-ft squares in the prototype. The origin of the grid was the center of the energy dissipator's end sill, and the grid extended up to 150 ft (prototype distance) downstream of the end sill, 90 ft (prototype) upstream of the end sill, and 90 ft (prototype) left and right of the inlet centerline (see figures 22a-22h). Data collection was limited to points along the centerline of the energy dissipation structure and to one side of the centerline. Velocity values for locations on the other side of the centerline were determined assuming symmetry about the centerline.

Four basin pool elevation and inflow combinations were simulated in the model, as directed by Boyle Engineering. These combinations were as follows:

1. Pool Elevation = 896 ft; Inflow = 2,000 ft^3/sec.

2. Pool Elevation = 900 ft; Inflow = 8,950 ft^3/sec.

3. Pool Elevation = 919 ft; Inflow = 8,950 ft^3/sec.

4. Pool Elevation = 930 ft; Inflow = 11,825 ft^3/sec.

For each of these cases, velocity measurements were made near the bed. For the deeper reservoir pool conditions (cases 3 and 4) velocities were also measured at locations near mid-depth and near the surface of the pool.

Velocity Results

The results of the velocity measurements, adjusted to prototype scale, are presented in figures 22a-22h. The figures depict the spatial variation in velocity vectors, plotted in relative size, determined from the model measurements. Each figure represents the results from a separate simulated flow condition and/or vertical measurement location within the pool (i.e. near-bed, mid-depth, near-surface).

As expected, the lower reservoir conditions result in the highest flow velocities near the energy dissipator (figures 22a and 22b). In particular, case 2 (pool elevation = 900 ft, inflow = 8,950 cfs) yielded extremely high velocities in the area downstream from the structure, with average prototype velocities reaching as high as 32 ft/sec. In both case 1 and case 2, the water surface elevation of the flow exiting the energy dissipator was higher than the ambient pool elevation in the reservoir. Thus, the water surface of the jet emanating from the energy dissipator was forced to draw down to match the reservoir pool elevation. As the depth of the jet decreased, the velocity in the jet increased, resulting in higher velocity flow downstream from the energy dissipator. Surprisingly, the jet of high velocity flow downstream of the energy dissipator did not appear to spread laterally a great deal. While high velocity flow was measured 30 ft off of the centerline, the flow velocities at 60 ft off of the centerline were considerably slower.

For cases 3 and 4, the reservoir pool elevations were high enough to submerge the energy dissipation structure, overtopping the energy dissipator walls (top elevation 911.5 ft) and forcing the hydraulic jump to occur further upstream in the inlet channel. For example, in case 2 (inflow = 8,950 cfs) with an unsubmerged energy dissipator, the toe of the hydraulic jump in the inlet channel occurred at prototype Station 163+15. For case 3, with the same inflow but a submerged energy dissipator, the toe of the hydraulic jump moved upstream to prototype Station 163+85. Not only did the hydraulic jump move further upstream for the submerged cases, but the inflowing water began to spill over the side-walls of the energy dissipator long before reaching the downstream end of the structure, resulting in less flow exiting at the downstream end, and lower near-bed velocities. The effect of energy dissipator submergence on the flow velocities is demonstrated in figures 22c-22h. Figures 22c and 22f show the average near-bed velocities for cases 3 and 4, respectively. In comparison with the unsubmerged data from cases 1 and 2 (figures 22a and 22b), these velocities are much lower and the velocities decrease significantly as the flow moves away from the energy dissipator. At mid-depth (figures 22d and 22g) the velocity field begins to show the effects of the flow spilling over the side-walls of the energy dissipator, with generally higher flow velocities throughout the flow field. Finally, near the water surface (figures 22e and 22h) the velocity field shows the full effect of flow spilling over the side-walls. Not only are the velocities generally higher than those recorded at the lower depths, but the dispersion of the inflow about the energy dissipation structure is clearly demonstrated.

It should be noted that, for all of the flow and submergence conditions tested, there was a strong current at the end of the energy dissipation structure in the outward direction (figures 22a, 22b, 22c, and 22f). Reverse currents at the end of an energy dissipation structure can carry rocks upstream into the structure and cause abrasion damage. The tested design did not show a tendency for such reverse currents to occur.

Note: Velocities measured 1 ft above bed at elevation 895.5 ft.

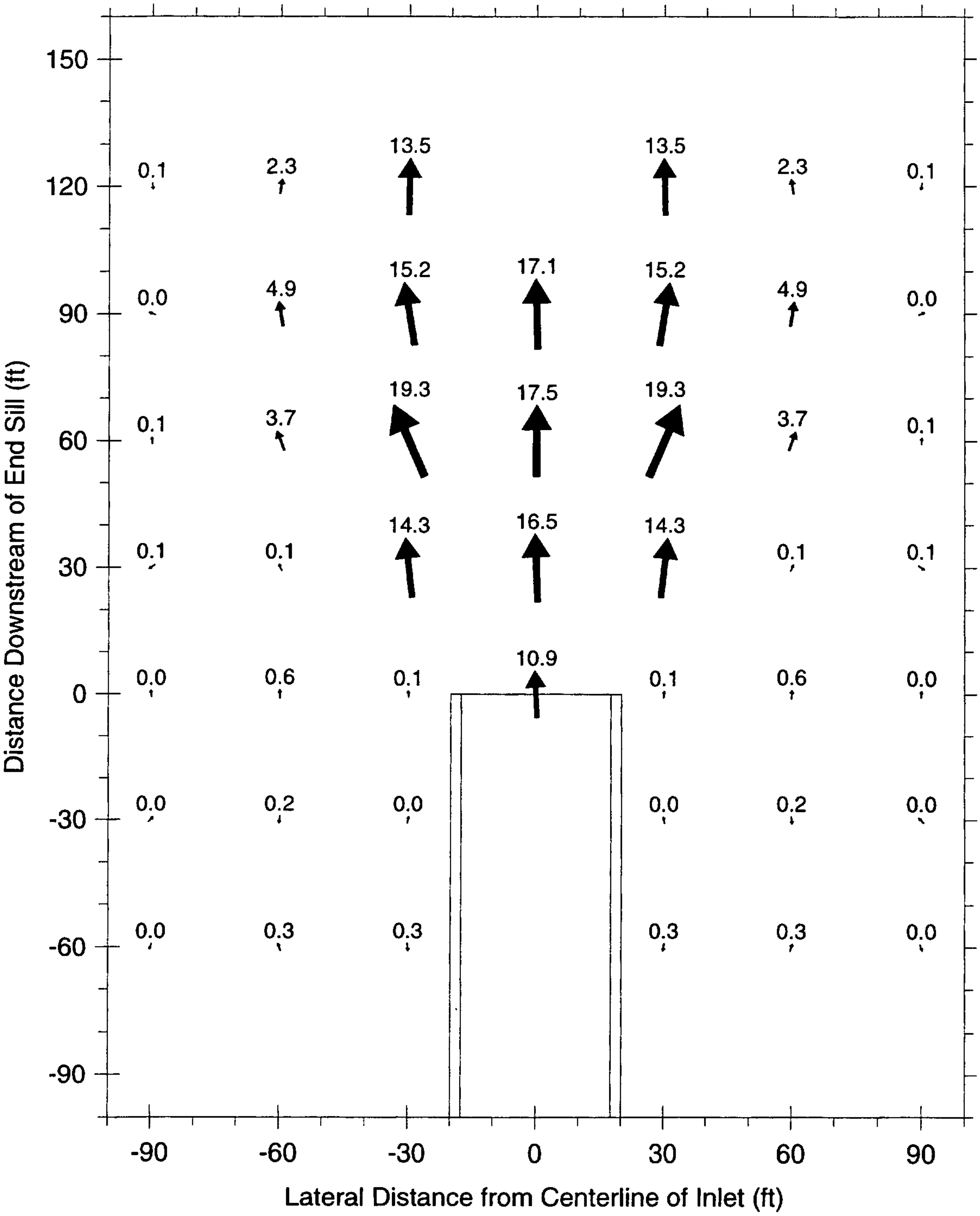

Figure 22a.–Jurupa Basin inlet average near-bed velocities for inflow=2,000 ft^3/sec, pool=896 ft.

Note: Velocities measured 1 ft above bed at elevation 895.5 ft.

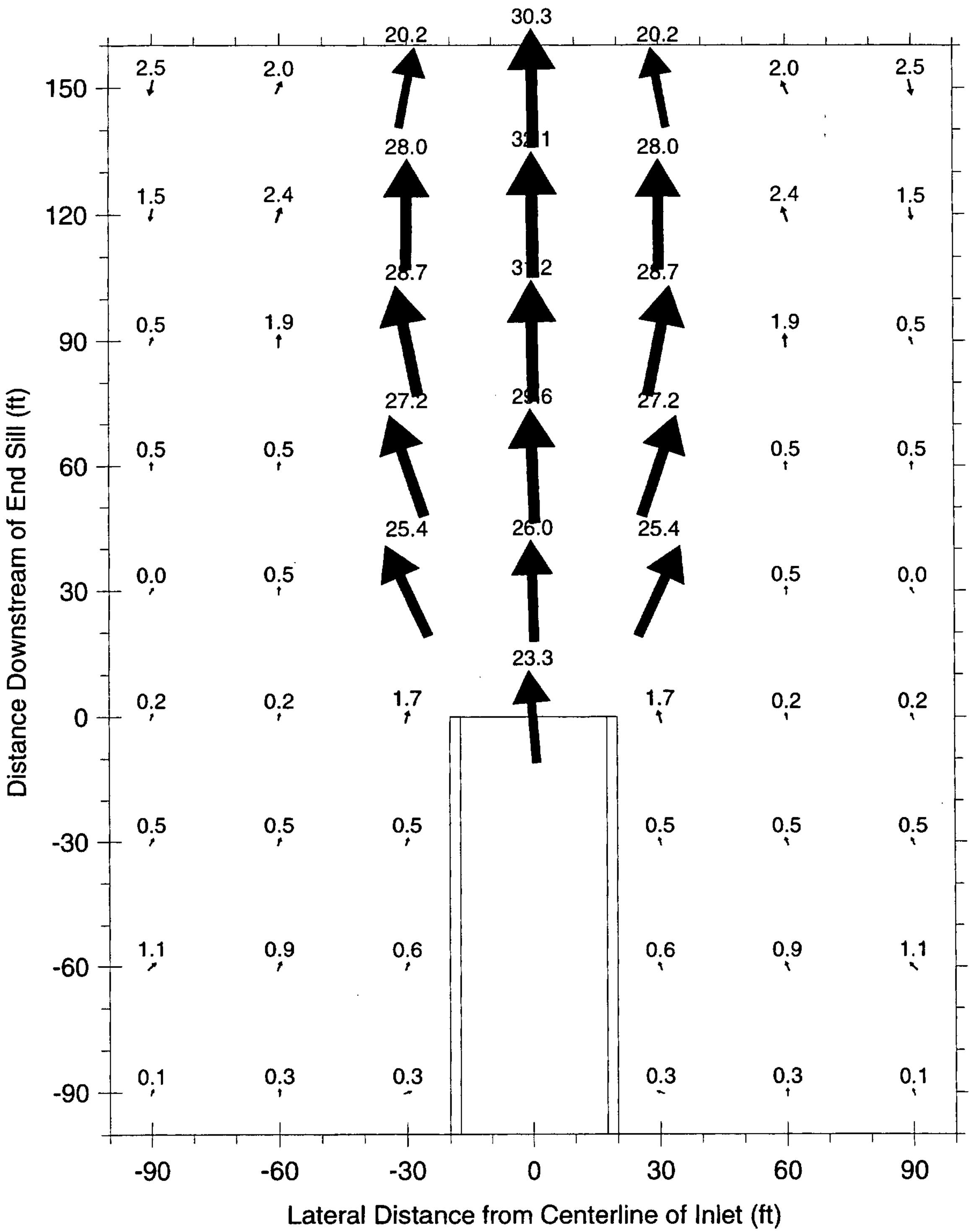

Figure 22b.–Jurupa Basin inlet average near-bed velocities for inflow=8,950 ft³/sec, pool=900 ft.

Note: Velocities measured 1 ft above bed at elevation 895.5 ft.

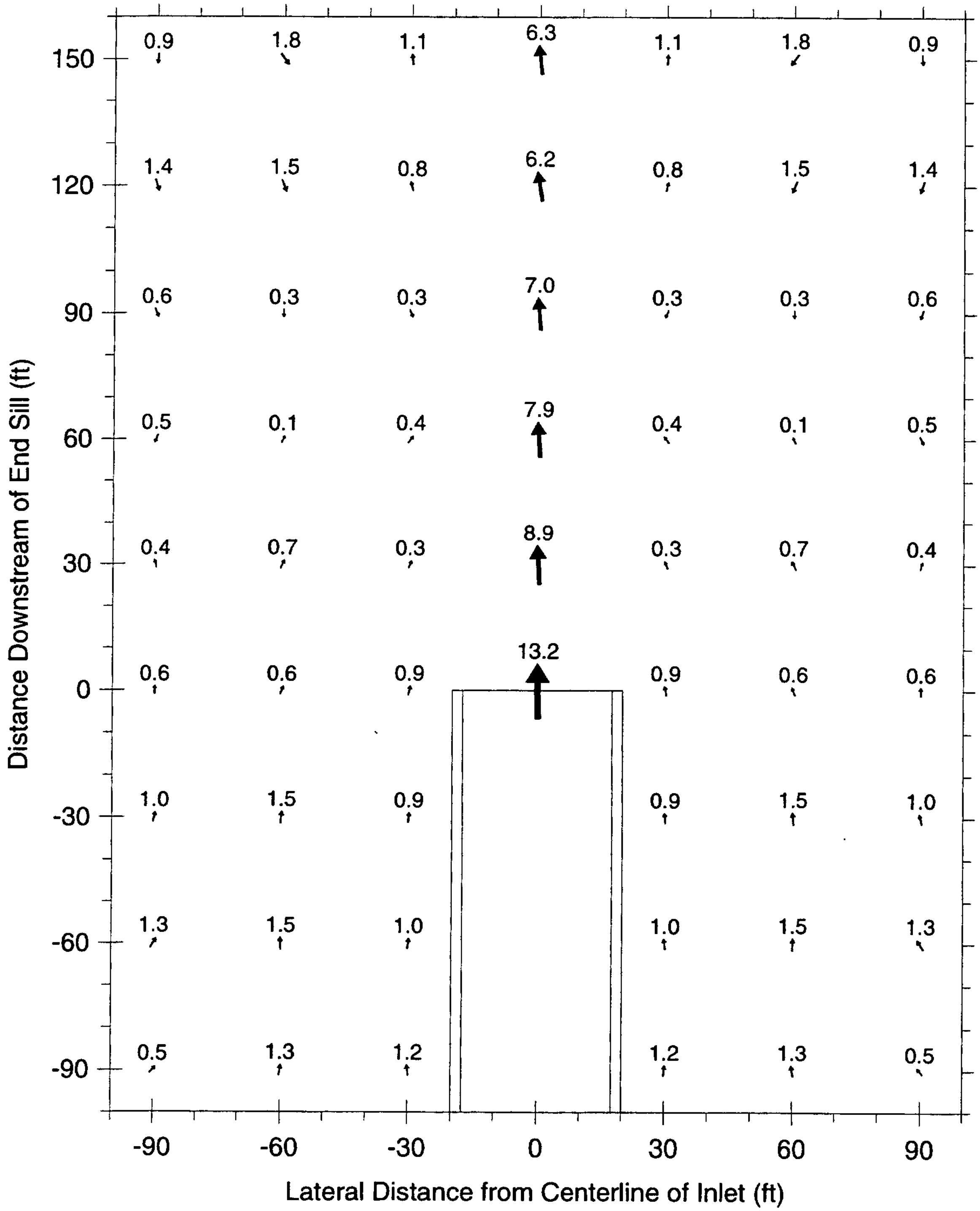

Figure 22c.–Jurupa Basin inlet average near-bed velocities for inflow=8,950 ft^3/sec, pool=919 ft.

Note: Velocities measured 13 ft above bed at elevation 907.5 ft.

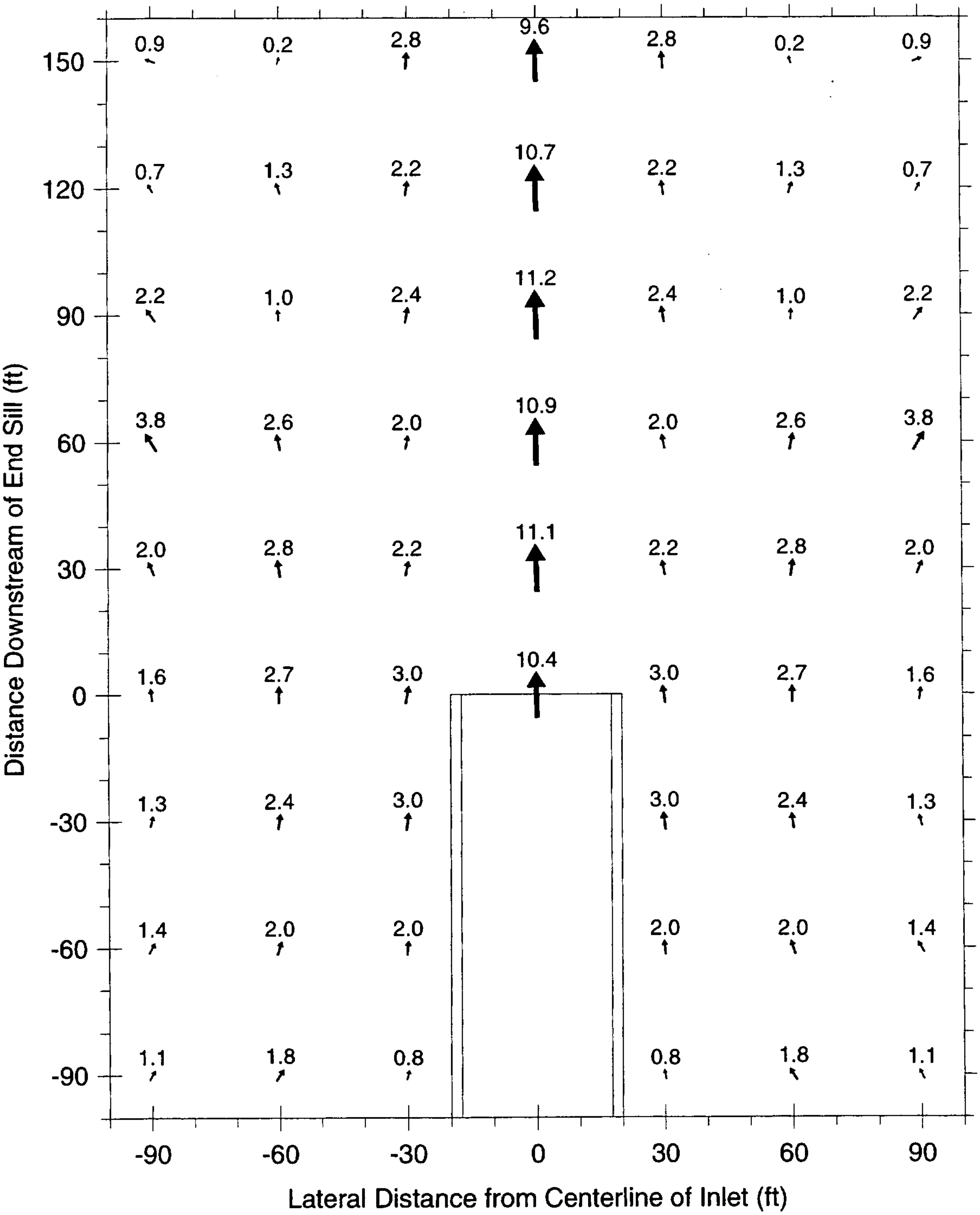

Figure 22d.–Jurupa Basin inlet average mid-depth velocities for inflow=8,950 ft^3/sec, pool=919 ft.

Note: Velocities measured 21 ft above bed at elevation 915.5 ft.

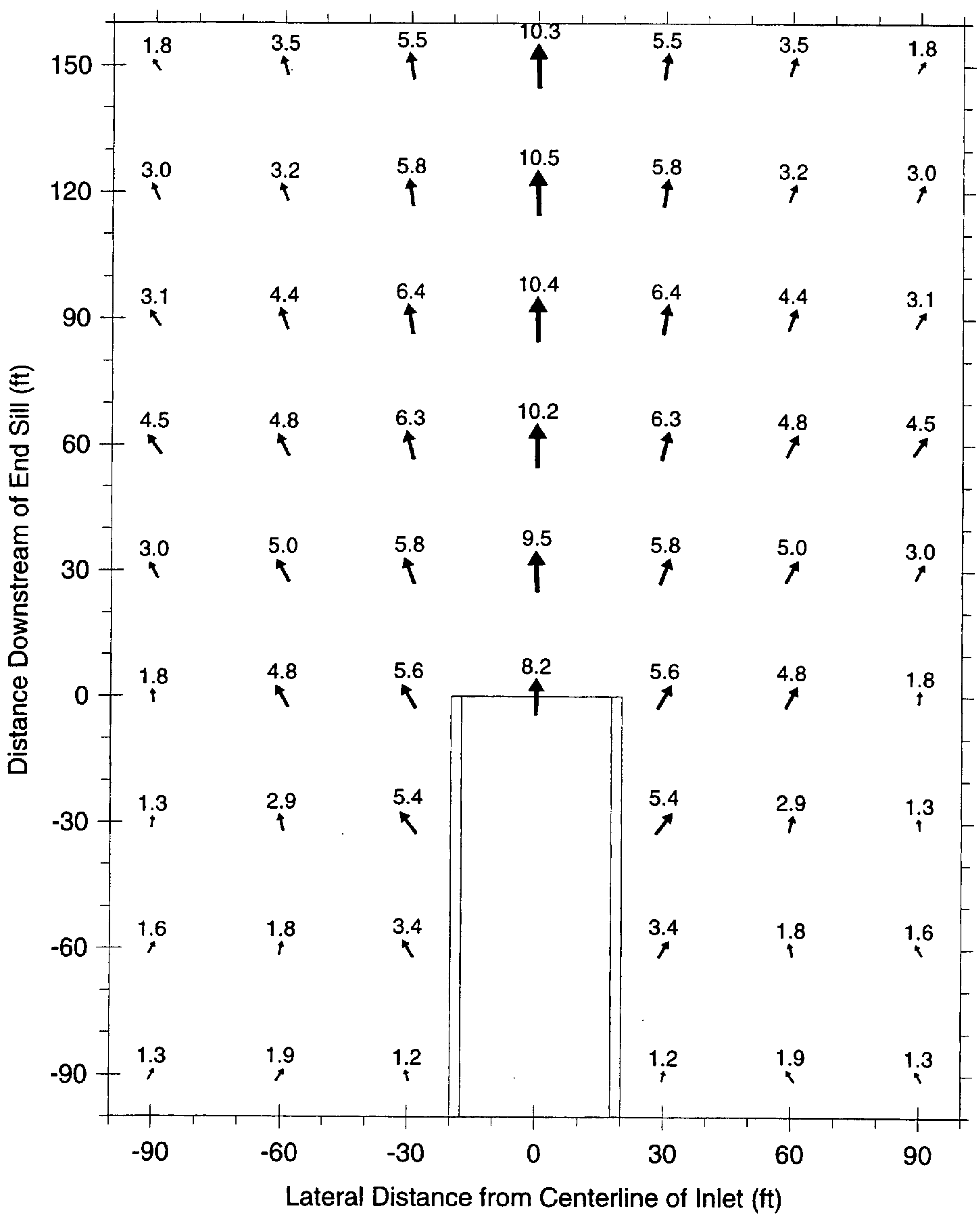

Figure 22e.–Jurupa Basin inlet average near-surface velocities for inflow=8,950 ft³/sec, pool=919 ft.

Note: Velocities measured 1 ft above bed at elevation 895.5 ft.

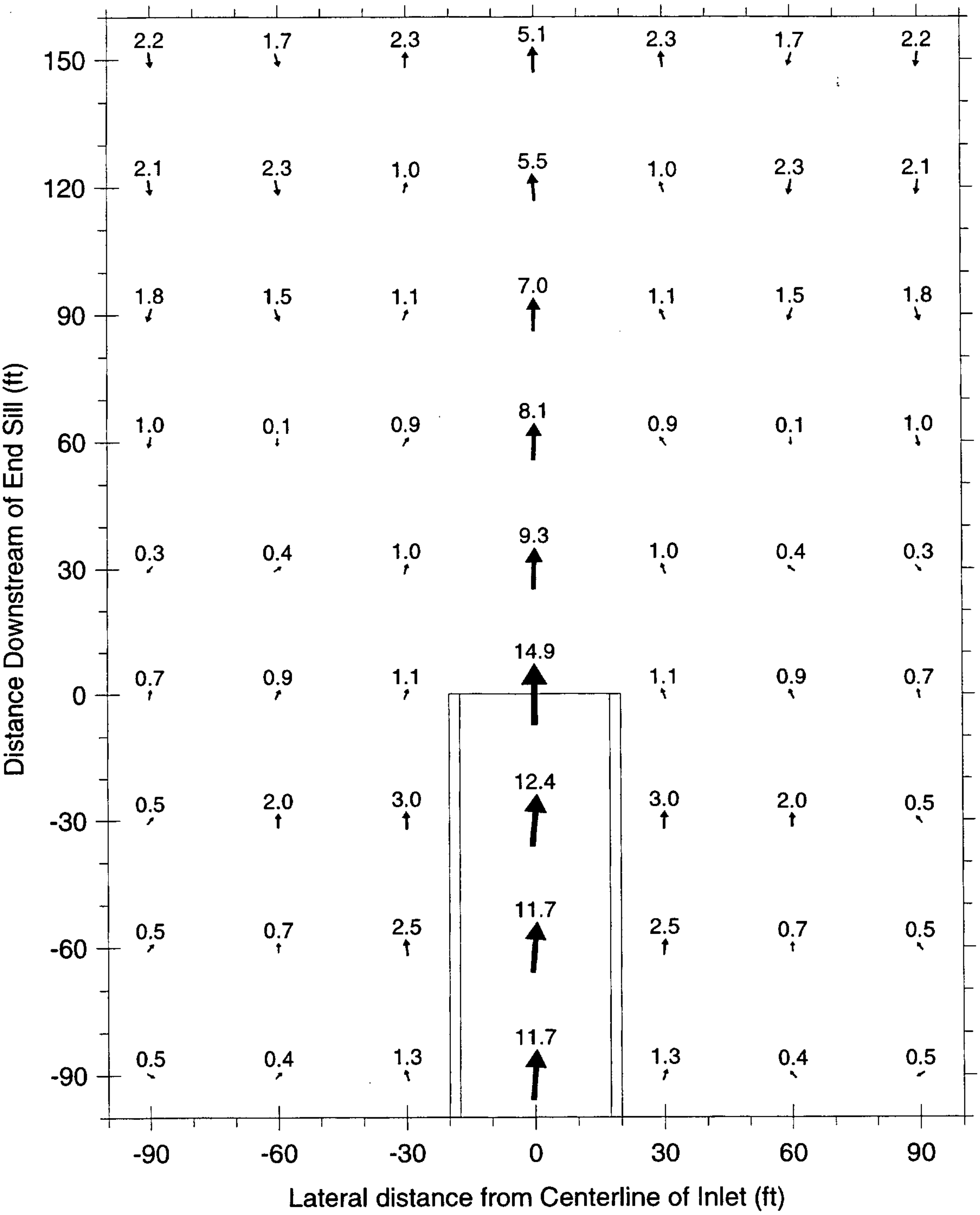

Figure 22f.–Jurupa Basin inlet average near-bed velocities for inflow=11,825 ft³/sec, pool=930 ft.

Note: Velocities measured 17.5 ft above bed at elevation 912 ft.

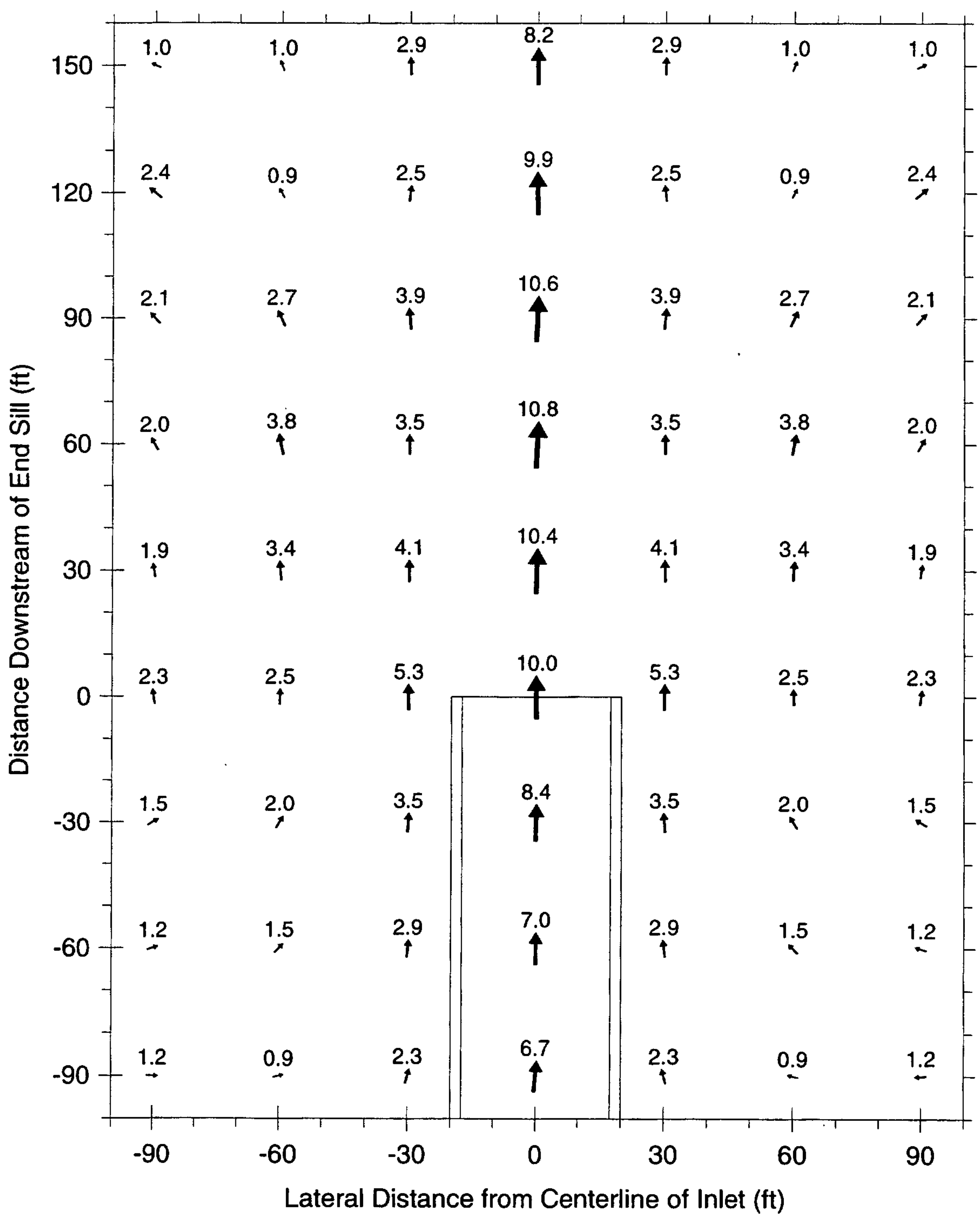

Figure 22g.–Jurupa Basin inlet average mid-depth velocities for inflow=11,825 ft³/sec, pool=930 ft.

Note: Velocities measured 32.5 ft above bed at elevation 927 ft.

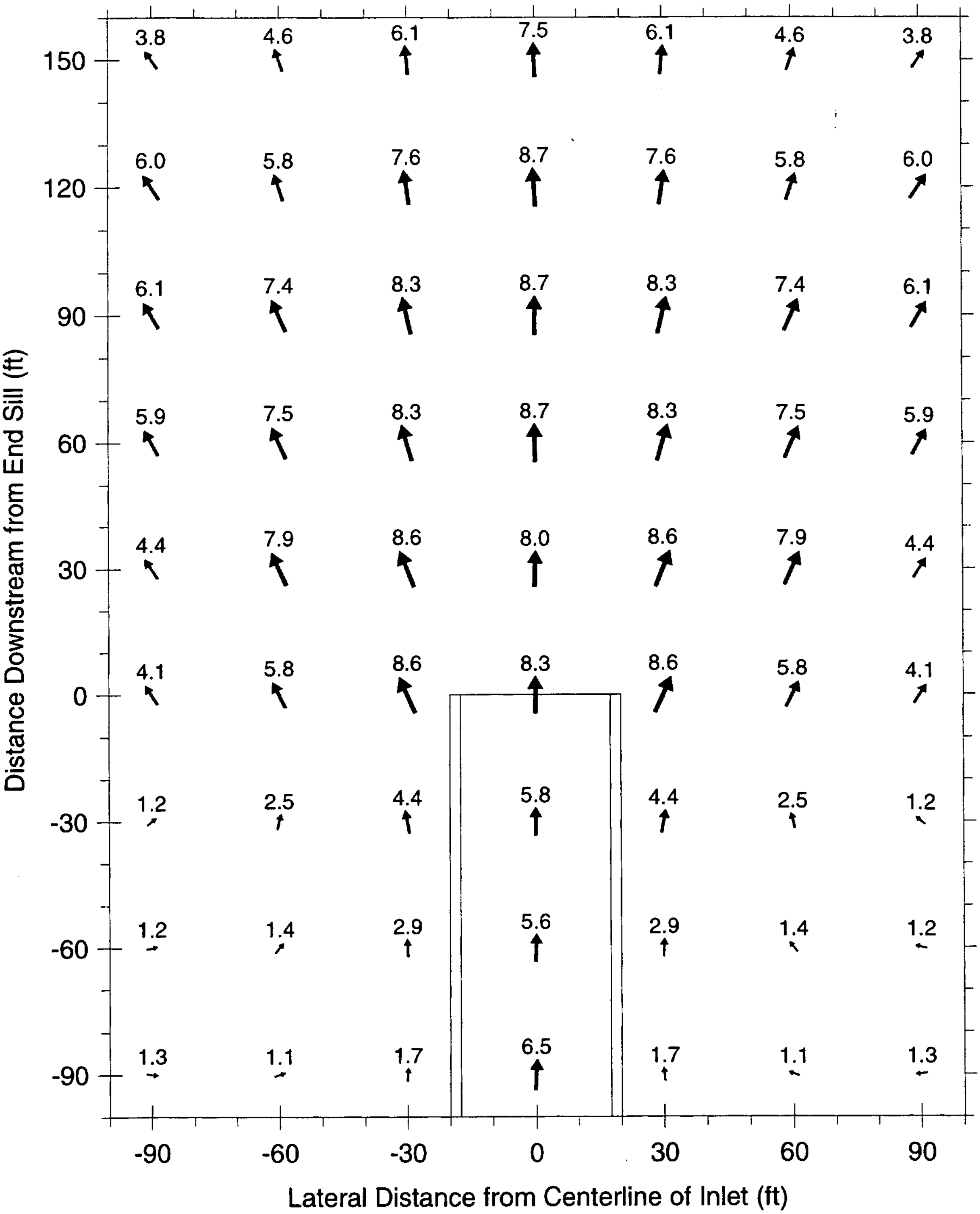

Figure 22h.–Jurupa Basin inlet average near-surface velocities for inflow=11,825 ft³/sec, pool=930 ft.

NUMERICAL WEIR SIMULATIONS

Physical modeling of the San Sevaine side-weir diversion system demonstrated that the original 2,800-ft-long weir design could be significantly shortened while still meeting the design flow-split criteria, resulting in substantial construction cost savings. The physical modeling program did not, however, encompass optimization testing which would have varied a number of weir design parameters in an effort to find the most economical weir design meeting the flow-split criteria. For example, while the physical modeling effort varied the weir crest shape and the longitudinal length of the weir, the rate of the main channel width contraction and the weir height were held constant at 24.51 ft/mile and 6 ft, respectively. Variations in either or both of these parameters would significantly impact the performance of the side weir.

To assess the potential for further optimizing and shortening the side-weir design, it was decided to numerically simulate a variety of weir configurations. Data from the physical modeling effort would be used to calibrate the numerical model. Alternative weir designs that were identified by the model as meeting the design requirements while offering significant additional construction cost savings would be reviewed by the County and its engineering design firm. Should a particular alternative appear both viable and desirable, it would then be physically modeled to confirm the performance of the design.

Analysis Method

A computer program was developed at the Water Resources Research Laboratory to perform the numerical modeling of alternative side-weir configurations. The program was written in Fortran and was similar to many step-backwater computational programs. The numerical routine computed flow depths and discharges along the side-weir in a step-wise fashion using an energy-balance approach. Due to the supercritical flow conditions being evaluated in the San Sevaine channel, the computations were carried out from upstream to downstream, starting with a normal-depth assumption at the upstream end of the side weir for a main channel discharge determined by the user. Side weir configuration data input by the user included length of weir, main channel width and weir height at both upstream and downstream ends of the weir, and main channel slope and roughness.

Discharge over the side weir was determined incrementally using a simple head-discharge weir relationship:

$$q = C_d \, h^{(3/2)}$$

where q is the unit discharge over the side weir (ft^3/sec/ft), C_d is a coefficient of discharge, and h is the head on the weir crest (ft). The value of C_d was calibrated using data available from tests of the 1-ft-thick chamfered weir. Based on this data, an initial value of 4.10 was adopted for C_d, with a reduced value of C_d used at the upstream end of the weir to account for the transition effects at the weir entrance.

After initial simulations and analyses were completed with the program, the program was enhanced to expand its capabilities. The enhanced version of the program could develop a side-weir discharge versus main-channel discharge rating curve for a given weir design and compute the side-weir overflow hydrograph for a given main-channel inflow hydrograph. The simplified coefficient of discharge, C_d, was also enhanced to account for the fact that the coefficient of discharge was not constant, but varied with both the Froude number of the flow and the head on the weir crest. Since the laboratory data did not cover a broad enough range of Froude numbers to evaluate its impact on C_d, the enhanced C_d was expressed simply as a function of h, ranging linearly from a minimum value of 3.8 for h equal to 1 ft or less, to a maximum value of 4.3 for h greater than or equal to 2.5 ft.

Weir Options

Numerical analysis was used to evaluate numerous variations in both weir height configurations (including sloping weirs) and main channel contraction rates. All weir configurations were evaluated assuming a main channel slope of 0.010517 and a Manning's n of 0.0146. Initially over 1,000 weir configuration options were simulated using the constant C_d value of 4.1. For each weir simulated, the side-weir discharge was determined for an inflow of 20,400 ft^3/sec, and the total weir overflow volume was computed for an inflow hydrograph supplied by Boyle Engineering. From this initial set of weir configurations, a short list of 35 configurations was identified which met, or nearly met, a relaxed side-weir performance criteria of side-weir discharge greater than or equal to 8,500 ft^3/sec for an inflow of 20,400 ft^3/sec, and total weir overflow volume less than or equal to 2,100 acre-ft for the design inflow hydrograph.

The short list of 35 possible alternative weir configurations was then evaluated by Boyle Engineering using their own side-weir numerical analysis program, JURUPA, a variation of their initial side-weir design program, SWEIR95. The side-weir performance indicators predicted by the program JURUPA agreed within 1 percent or less with the values predicted by the Water Resources Research Laboratory analysis, providing an independent verification of the general numerical procedure being used.

Based on the potential for further cost savings in the side-weir design as demonstrated by the numerical modeling, Boyle Engineering, acting on behalf of the County of San Bernardino, requested that the Water Resources Research Laboratory perform additional side-weir numerical modeling evaluations. The goal of these evaluations was to determine a final list of approximately five side-weir options which would meet the design criteria for the project, including a slightly revised side-weir discharge requirement of 9,079 ft^3/sec for a peak main-channel inflow of 20,300 ft^3/sec. These final options would then be evaluated by the County and Boyle Engineering using hydrologic routing techniques to assess the impact of each design option with regard to the performance of Jurupa Basin. Due to the limited amount of storage available in Jurupa Basin, it was important that the side-weir discharge hydrograph resulting from diversions off of the San Sevaine channel design-flood hydrograph be carefully evaluated for each side-weir option under consideration. Should a particular side-weir option meet the project flow-split requirements and prove satisfactory with regard to Jurupa Basin storage requirements, it would be considered for additional physical modeling and performance verification.

Using the enhanced version of the side-weir analysis program, including the variable coefficient of discharge, a list of seven possible alternative weir designs was provided to Boyle Engineering and the County of San Bernardino for their consideration. These alternative weir designs are summarized in Table 1, and include weir configurations ranging from 2,340 ft in length (the shortest side weir physically modeled) down to 1,400 ft in length. After considerable analysis with regard to the Jurupa Basin operation and storage issues, the County decided to adopt the physically-modeled 2,340 ft chamfered weir as the final design configuration. Thus, no additional physical modeling or verification of the numerically-simulated weir options was pursued. However, the optimized length evaluations and methodology may be applicable to future side-weir designs.

Table 1.—Possible alternative side-weir configurations selected by numerical analysis

	Option 1*	Option 2	Option 3	Option 4	Option 5	Option 6	Option 7
Configuration Data							
Weir Length (ft)	2,340	2,200	2,000	1,800	1,800	1,600	1,400
U/S Weir Ht. (ft)	6	5.6	6	5.6	5.4	5.2	5.6
D/S Weir Ht. (ft)	6	6.8	7.2	7.2	6.6	7.2	6.8
U/S Channel Width (ft)	60	60	60	60	60	60	60
D/S Channel Width (ft)	49.14	44	40	40	44	40	40
Minimum Flow Rating							
Max. Flow before Overtopping Weir (ft^3/sec)	8,379.1	8,620.9	8,095.6	8,010.7	8,133.3	7,911.1	7,189.5
U/S Normal Depth (ft)	5.04	5.14	4.94	4.90	4.95	4.86	4.58
D/S Computed Depth (ft)	6.00	6.80	7.20	7.20	6.60	7.20	6.80
Peak Flow Rating							
Side-weir discharge for inflow of 20,400 ft^3/sec	9,201.1	9,172.6	9,120.9	9,150.0	9,190.4	9,155.6	9,187.6
U/S Normal Depth (ft)	8.98	8.98	8.98	8.98	8.98	8.98	8.98
D/S Computed Depth (ft)	6.61	7.37	7.96	7.95	7.26	7.94	7.76
Design Hydrograph							
Side-weir Overflow Hydrograph Vol. (acre-ft)	2,185.1	2,184.5	2,236.4	2,321.4	2,350.1	2,417.1	2,595.1

* This option physically modeled prior to the numerical analysis.

CONCLUSIONS

Side-Weir Performance

Evaluation of the initial 2,800-ft-long, 6-ft-high side-weir configuration indicated that the side-weir was more efficient at diverting flow than anticipated. Testing demonstrated that the side-weir could be shortened by 360 ft (13 percent) and still achieve the required diversion flow of 9,200 ft^3/sec for the peak San Sevaine inflow of 20,400 ft^3/sec. Chamfering the weir crest further increased the efficiency of the structure, allowing the side-weir to be shortened an additional 100 ft (4 percent) to a final length of 2,340 ft, while still achieving the required flow splits.

A side-weir discharge versus San Sevaine channel discharge rating was developed for four different side-weir configurations (figure 21). Measurements were also made of cumulative side-weir discharge versus longitudinal length along the weir for the design San Sevaine channel inflow of 20,400 ft^3/sec (figure 20). This data clearly indicated that at the design discharge most of the side-weir flow passed over the initial portions of the weir, with increasing weir lengths offering decreasing benefits with regard to total diversion capacity. The reduction in main channel width along the length of the side weir was not enough to offset the rapid loss of flow over the upstream portion of the weir, thus the unit discharge in the main channel rapidly decreased with an associated loss of flow depth. In general, for side weirs operating in supercritical flow conditions, the decreasing unit discharge in the main channel resulting from lateral outflows will yield decreasing flow depths and lower head acting on the weir. Thus, long weirs with fixed crest heights will offer diminished diversion performance in the downstream direction.

Jurupa Inlet Velocities

Flow conditions where the Jurupa Basin energy dissipation structure is unsubmerged (i.e. the pool elevation in Jurupa Basin is less than the water surface elevation in the energy dissipator) resulted in the most extreme velocities downstream of the structure, with average velocities as high as 32 ft/sec. When the energy dissipation structure was submerged, the flow velocities exiting the structure tended to be significantly less, and decreased as the flow moved away from the structure. In particular, when the pool elevation overtopped the energy dissipator side-walls there was significant dispersion of the inflowing jet before the end of the energy dissipation structure was reached.

The lateral spread of high-velocity flow away from the energy dissipator centerline was much less than expected for both the submerged and unsubmerged cases. For the unsubmerged cases, high velocity flow was observed at 30 ft on either side of the centerline, but at 60 ft off of the centerline the velocities had decreased dramatically. For the submerged cases, the near-bed velocities exhibited very little lateral dispersion, but higher in the water column the lateral dispersion of the flow became evident, particularly near the water surface. This was due to the effects of flow spilling over the side-walls of the energy dissipation structure all along the length of the energy dissipator.

The measured flow velocities, adjusted to prototype scale, are presented as vector plots in figures 22a-22h. It is important to note that these velocities represent average flow velocities. Significant turbulent velocity fluctuations about the reported averages were observed in the model, and can be expected to occur in the prototype as well.

Alternative Weir Designs

A numerical model was developed to evaluate the potential for achieving further savings in weir length and construction costs. This model was based on a step-wise energy-balance approach moving from upstream to downstream along the side weir. The model used a simplified weir head-discharge relationship to compute flow over the side weir incrementally. The coefficient of discharge for this relationship was calibrated using laboratory data for the chamfered weir crest. Based on this calibration, the coefficient of discharge was observed to vary with the head on the weir crest. Good agreement between the numerical model predictions and the laboratory data was achieved using a coefficient of discharge which varied linearly from a minimum value of 3.8 for h equal to 1 ft or less, to a maximum value of 4.3 for h greater than or equal to 2.5 ft. This variable coefficient of discharge was used for all final numerical predictions, although a constant average value of 4.1 for the coefficient of discharge was also found to provide reasonable agreement with the laboratory data.

The model was used to analyze numerous alternative weir configurations, varying the length of the weir, the height and slope of the weir crest, and the rate of contraction of the main channel width. Several alternative weir configurations were identified for further consideration by the County of San Bernardino and their designated engineering design consultants. Ultimately, the County selected the 2,340-ft-long, 6-ft-high, chamfered side weir tested in the physical model as the final design configuration. This decision was driven, in part, by concerns related to the operation and storage capacity of Jurupa Basin. The need to minimize total diversions to the Basin, while still reducing the peak discharge in the San Sevaine channel, limited the amount by which the side weir could be shortened.

REFERENCES

Hager, W.H., 1987, Lateral Outflow Over Side Weirs, *Journal of Hydraulic Engineering*, ASCE, 113(4).

Henderson, F.M., 1966, Section 7.6: Lateral inflow and outflow, *Open Channel Flow*, MacMillan Publishing Co., New York, New York.

Subramanya, K., and S.C. Awasthy, 1972, Spatially Varied Flow Over Side Weirs, *Journal of the Hydraulic Division*, ASCE, 98(HY1).

Ranga Raju, K.G., B. Prasad, and S.K. Gupta, 1979, Side Weirs in Rectangular Channels, *Journal of the Hydraulic Division*, ASCE, 105(HY5).

Swamee, P.K., S.K. Pathak, and M.S. Ali, 1994, Side Weir Analysis Using Elementary Discharge Coefficient, *Journal of Irrigation and Drainage Engineering*, ASCE, 120(4).

Tetra Tech, 1986, Day Creek Peak Reduction Facility Model Studies, TC-3289, Pasadena, California.

APPENDIX A

Cumulative side-weir discharge data used to develop Figure 20. The data is presented in prototype scale and was collected for a simulated San Sevaine channel inflow of 20,400 ft^3/sec.

Original 2,800' Weir	
Dist. Along Weir (ft)	Side-Weir Q (cfs)
0.00	0
30.00	272
60.00	499
90.00	902
149.99	1,699
239.99	2,742
389.98	3,806
539.97	4,559
689.96	5,016
989.95	5,995
1,289.93	6,678
1,589.91	7,426
1,889.90	8,046
2,189.88	8,722
2,429.87	9,123
2,439.77	9,193
2,489.86	9,300
2,800.00	9,801

Chamfered 2,800' Weir	
Dist. Along Weir (ft)	Side-Weir Q (cfs)
0.00	0
30.00	137
60.00	499
90.00	991
149.99	1,830
239.99	2,997
389.98	4,014
539.97	4,723
689.96	5,339
989.95	6,153
1,289.93	6,910
1,589.91	7,691
1,889.90	8,343
2,189.88	8,933
2,339.87	9,187
2,489.86	9,508
2,800.00	10,073

APPENDIX B

Flow-split discharge data used to develop Figure 21. The data is presented in prototype scale.

2,800' Rectangular Weir

Inflow Q (cfs)	Side-Weir Q (cfs)
7,798	31
8,001	48
8,503	175
9,001	355
10,002	905
12,008	2,452
14,000	4,145
16,228	6,139
17,998	7,740
19,999	9,435
20,398	9,801

2,440' Rectangular Weir

Inflow Q (cfs)	Side-Weir Q (cfs)
9,001	195
10,002	644
11,998	2,077
14,000	3,674
16,282	5,719
17,998	7,229
19,999	8,815
20,398	9,214

2,800' Chamfered Weir

Inflow Q (cfs)	Side-Weir Q (cfs)
7,798	0
8,001	48
8,503	143
9,001	355
10,002	937
11,998	2,509
14,000	4,245
15,602	5,680
15,981	6,027
17,998	7,875
19,999	9,707
20,398	10,073
20,398	10,005

2,340' Chamfered Weir

Inflow Q (cfs)	Side-Weir Q (cfs)
8,597	15
8,799	57
9,001	98
9,499	272
10,002	545
10,998	1,110
11,998	1,943
14,000	3,607
16,243	5,439
17,998	7,056
19,999	8,850
20,398	9,179
21,601	10,207