Kennedy/Jenks/Chilton

THOMAS CREEK
DETENTION BASIN STUDY

PREPARED FOR:

TECHNICAL ADVISORY COMMITTEE
WASHOE COUNTY REGIONAL FLOOD CONTROL
MASTER PLAN

DRAFT

K/J/C JOB NO. 897043.02
MAY 1990

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EXECUTIVE SUMMARY

Kennedy/Jenks/Chilton developed specific hydrologic modeling for the Thomas Creek drainage basin and analyzed several storm water detention/debris basin sites within the watershed for the Washoe County Flood Control Master Plan Project. Two detention/debris basins sites and facilities, one on-stream and the other off-stream, were studied. Both facilities greatly reduce the 100-year peak flood flows reaching South Virginia Street. Peak flow rates at South Virginia Street were reduced from between 1400 to 1800 cfs to 515 and 680 cfs with a detention dams in-place.

The On-Stream Detention/Debris Basin offers the greatest attenuation of the 100-year peak flood flows. The estimated construction costs for the On-Stream facility ranges from between $3.1 to $5.6 million while the Off-Stream facility estimated cost is $3.35 million. The primary difference in the costs is the spillway structure required with the On-Stream facility. The range of costs for the On-Stream Detention facility is the result of comparing sub-alternatives involving the construction of a concrete lined spillway (high estimate) compared with an unlined spillway (low estimate).

Both facilities were designed utilizing similar design criteria. It was assumed that on-site materials could be used for construction of homogeneous embankments, embankment slopes would not exceed 3h:1v, cut slopes would be no steeper than 2h:1v and the crest width of the dam would be 20 feet. The design storage for both reservoirs was set at approximately 500 acre-feet. Primary discharge from the facilities would be provided by a low-level outlet works. Emergency spillway capacity for the On-Stream Detention/Debris Basin was provided by a spillway structure built on the left abutment. The Off-Stream Detention/Debris Basin does not require a spillway because the facility is not constructed within the Thomas Creek channel and will not be overtopped during an extreme rainfall event.

Full channelization of Thomas Creek downstream of a detention/debris basin will serve to remove the FEMA alluvial fan designation currently in effect. The construction cost for full channelization is estimated to be $2.3 million.

Construction of partial channelization and diversion works adjacent to South Virginia Street and Holcomb Lane will effectively divert and convey the attenuated 100-year flood flows to South Virginia Street. This alternative will not effect the current FEMA designation, but will greatly reduce the flooding potential along South Virginia Street north of Holcomb Lane. The estimated construction costs for the work is $550,000.

CHAPTER 1

INTRODUCTION

Washoe County contracted with Kennedy/Jenks/Chilton to provide the following engineering studies:

1. Confirm the existence of viable upstream storm water detention and debris basin sites on Thomas Creek as a means of reducing the sizes of the drainage conveyance structures planned for the South Virginia Street widening project and the future extension of Interstate 580.

2. Analyze the potential for reclassifying the current Thomas Creek Federal Emergency Management Association (FEMA) alluvial floodplain designation.

3. Pre-design analysis of the detention dam/debris basins and channel improvements, including estimates of construction and land acquisition costs.

The scope of work entailed analyzing the Thomas Creek watershed and developing HEC-1 hydrologic models for the 100-year, 24 hour and the Probable Maximum Flood (PMF) rainfall events. Based on the developed hydrologic models, detention/debris basin sites were selected and studied in a preliminary manner. These preliminary studies included reservoir sizing, embankment sizing and routing of the 100-year flood event through the detention facility.

Two sites showing favorable detention characteristics were studied in further detail and are presented in this report. Pre-design analyses, construction cost estimates and land ownership and acquisition costs were established for these two facilities.

The scope of work also included the review of the FEMA classification of Thomas Creek. One of the stated goals of this study was to explore the possibility of reclassifying the alluvial floodplain zoning to a less restrictive designation based upon the incorporation of detention/debris basins. To accomplish this goal, Kennedy/Jenks/Chilton analyzed several alternatives and scenarios including the complete channelization of the creek downstream of the detention/debris basin(s) to South Virginia Street; partial channelization and diversion of creek flows adjacent to South Virginia Street; re-mapping of the creek based upon the reduced flood flows; and doing no channel improvements. Conceptual designs for these systems were developed along with construction and land acquisition cost estimates.

The following sections discuss Kennedy/Jenks/Chilton's hydrology studies, conceptual detention/debris basin designs, FEMA classification review, conclusions and recommendations.

CHAPTER 2

DESCRIPTION OF STUDY AREA

GENERAL DESCRIPTION

The Thomas Creek watershed is located in Southern Washoe County, Nevada, near the southern limit of the City of Reno. The Thomas Creek watershed above South Virginia Street (U.S. 395) is 11.5 square miles in size. Normal annual flows and extreme runoff events are conveyed in two directions at South Virginia. A portion of flow proceeds northeast and discharges into Dry Creek and ultimately into Steamboat Creek. The other portion proceeds east into Steamboat Creek. Just downstream of Steamboat Ditch, the natural flow from Thomas Creek is diverted for irrigation purposes into these two primary directions to provide irrigation for agricultural uses in the South Truckee Meadows.

Figure 2-1 is a location map identifying the location of the study area and Thomas Creek watershed.

WATERSHED DESCRIPTION

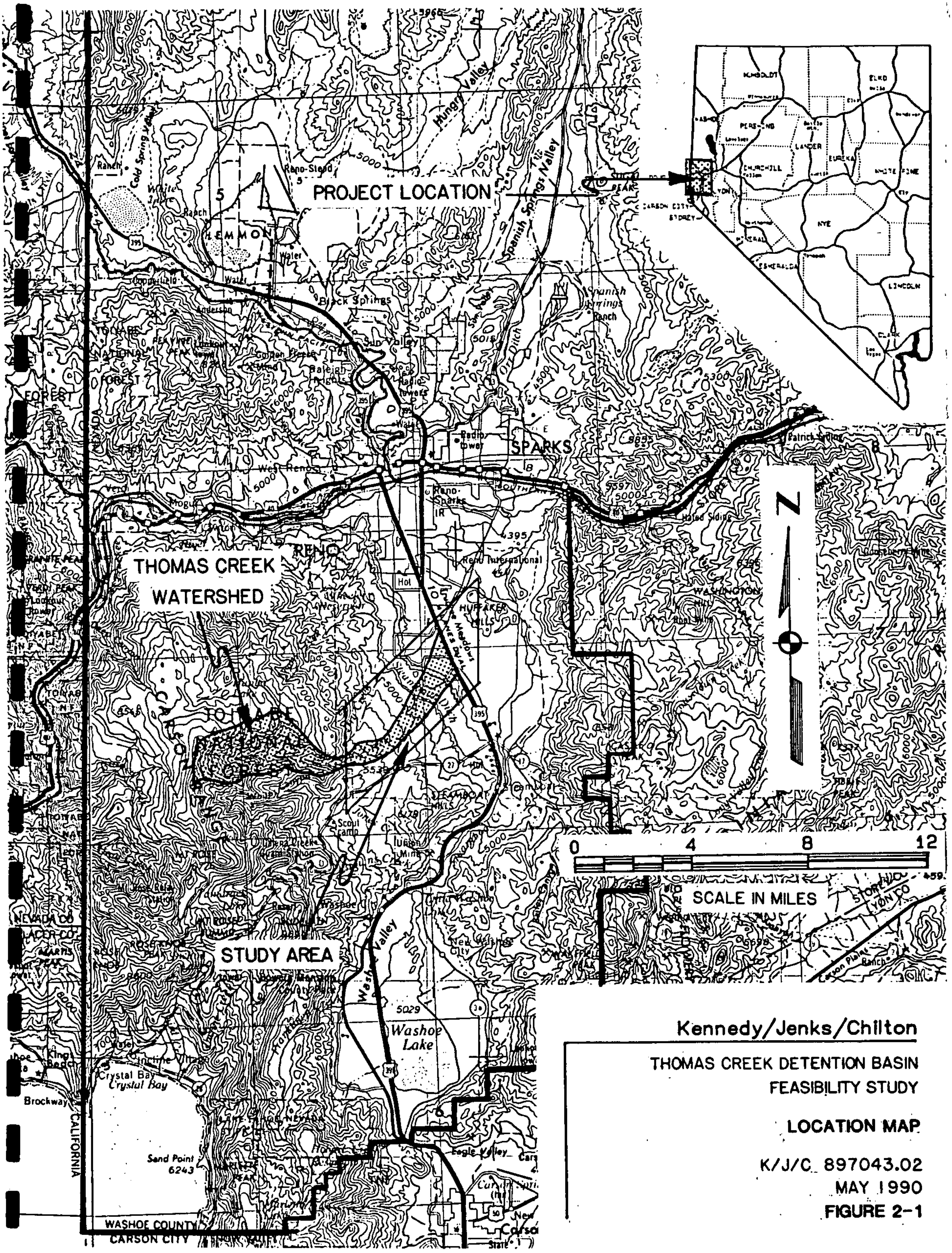
The Thomas Creek watershed originates at the crest of the Carson Range at an elevation of 10,080 feet above mean sea level. The portion of the watershed above the mountain front is approximately 7.3 square miles in size. The lower portion of the watershed, below the mountain front, is approximately 4.2 square miles in size.

The upper portion of the watershed ranges in elevation from 10,080 feet to 6,000 feet. Vegetation in the watershed consists primarily of various varieties of pine and fir, with an understory of manzanita and snowbrush in the upper elevations, and an understory of sagebrush, mountain mahogany and grass in the lower elevations. The central portion of this area includes two large meadows with aspen near the creek. Average annual precipitation in the area ranges from 20 to 50 inches.

The lower portion of the watershed includes a portion of the alluvial and pediment surfaces along the east face of the Carson range. Thomas Creek has formed a well incised channel through this alluvial surface. At an elevation of 4,900 feet, this channel becomes less defined where Thomas Creek enters the agricultural area.

Vegetation in the lower Thomas Creek watershed consists primarily of sagebrush, bitterbrush, and cheatgrass. Average annual precipitation in this region is six to 14 inches.

At the mountain front, Thomas Creek has a minimum average monthly flow of approximately 120 acre feet during the month of August and a maximum average flow of 560 acre feet during the month of May. Between



Kennedy/Jenks/Chilton

THOMAS CREEK DETENTION BASIN
FEASIBILITY STUDY

LOCATION MAP

K/J/C 897043.02
MAY 1990
FIGURE 2-1

Steamboat Ditch and the mountain front, Thomas Creek has formed a well incised channel that varies from 15 to 30 feet deep and 75 to 150 feet wide. The flow from Thomas Creek supports a significant amount of vegetation along the bottom of the Thomas Creek channel. This vegetation includes quaking aspen and willow as well as various other types of vegetation.

Below Steamboat Ditch the incised Thomas Creek channel abruptly ends and the slope of Thomas Creek becomes less steep. The natural channel was braided in this reach. During this last century, irrigation of pasture in this lower portion of Thomas Creek has resulted in modifications to the natural channels to divert Thomas Creek flows for irrigation uses. These channels are well stabilized and appear to transport relatively little sediment during normal flows.

EXISTING DEVELOPMENT

Existing development within the Thomas Creek watershed is primarily limited to the lower region of the watershed that is below the mountain front. The upper region only contains a few homes in Thomas Meadows. The amount of existing development in the upper region of the watershed is not significant with respect to it's impact on downstream hydrology.

The lower region varies in type and amount of existing development. The portion of this region that is above Steamboat Ditch is largely undeveloped with exception of two areas; an area of approximately 0.5 square miles near the mountain front has developed with homes on five to ten acre parcels; and an area of 0.25 square miles at the west end of Zolezzi Lane which is part of a planned development with homes on one half acre lots.

The portion of the lower region below Steamboat Ditch consists primarily of large residential lots. The majority of these lots are maintained as irrigated pasture. Some higher density residential and commercial development also exists in this region near Virginia Street (U.S. 395).

FUTURE DEVELOPMENT

For the purpose of this study, future development was based upon existing zoning. The source used to determine present zoning was the southwest Truckee Meadows and Forest Area Plans adopted by Washoe County Department of Comprehensive Planning. These plans identify existing zoning designations and dwelling unit densities allowed under each designation.

THOMAS CREEK FLOOD ZONE DESIGNATIONS

In 1968 the National Flood Insurance Act was enacted which created the National Flood Insurance Program (NFIP). In 1973 the Flood Disaster Project Act was also enacted. This Act required the purchase of flood insurance "as a condition of receiving any form of federal or federallyrelated financial assistance for acquisition or construction purposes with respect to insurable buildings and mobile homes within an identified special flood, mudslide, or flood-related erosion hazard area that is located within any community participating in the Program."

In order for a property owner to purchase flood insurance from the Federal Insurance Administration (FIA), the community in which the property lies must be participating in the NFIP. Upon initial entry into the NFIP a community typically entered the "Emergency Program".

Upon entry into the Emergency Program, a set of Flood Hazard Boundary Maps (FHBM) are published which identifies approximate "special flood hazard areas." The FHBM's serve as a temporary flood plain management tool until a more detailed study can be conducted.

The Federal Emergency Management Agency (FEMA) then initiates a Flood Insurance Study (FIS). The FIS will include a detailed hydrologic and hydraulic analysis for the primary drainages within the community and approximate studies for important smaller drainages for areas with low development potential. After completion of the FIS and adoption of a Flood Plain Management Ordinance by the community, the Flood Insurance Study and a set of Flood Insurance Rate Maps (FIRM) are published by FEMA and the community enters the regular program of the NFIP.

The City of Reno and Washoe County entered the regular program of the NFIP on July 5, 1983 and February 1, 1984 respectively. On these dates a set of FIRMs and FISs were published by FEMA.

During the original FIS, Thomas Creek and Evans Creek were identified as active alluvial fans. An alluvial fan is typically a cone shaped surface formed where flows from the mountain front exit onto the shallower sloping valley floor. The abrupt change in slope and channel geometry results in the deposition of the streams sediment and debris load. These active alluvial fans are undergoing a constant building process and are being reshaped by the deposition of sediments and the redistribution of those sediments with each significant event. Analysis of flow processes on active alluvial fans is difficult because of the random nature of the flow patterns on alluvial fan surfaces.

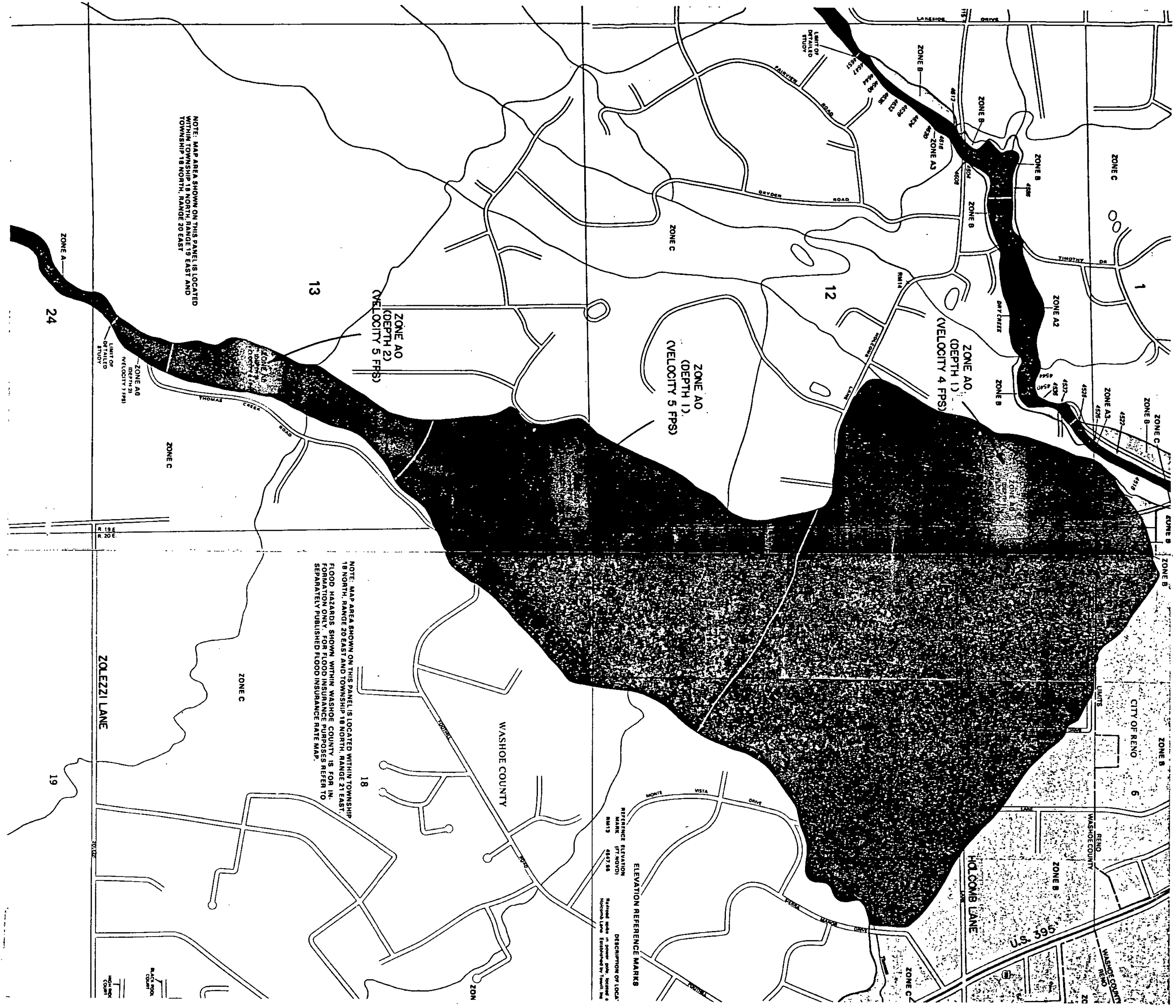
FEMA has developed a method of identifying flooding and flow velocity related hazards on active alluvial fans (FEMA 1985; Dawdy 1979; Dawdy 1986). This method is the stochastic model that relates the potential for a given depth and velocity of flow in each region of the fan surface with the discharge at the alluvial fan apex. During a 100 year event, only a portion of the alluvial fan surface would be flooded. The entire fan surface is mapped as 100 year flood zone since the direction of the flow path on the fan is random and will likely change with each significant event.

The 100 year flood zones for alluvial fans are identified as Zone AO with a depth and velocity reported (i.e. Zone AO depth 3, velocity 5). The depth and velocity is a probable depth (in feet) and velocity (in feet/second) that would be experienced in a hypothetical channel which would be formed in that region of the fan as a result of a 100 year discharge occurring at the fan apex.

An active alluvial fan will have several identifiable characteristics. For example, an active alluvial fan will typically have a convex shape to the fan surface as a result of the depositional and reshaping process. The surface soils will be very erodible and there will be many distinguishable shallow channels radiating from the apex. For continuation of the depositional process, the upstream channel must carry significant amounts of sediment and debris during significant flow events. For the FEMA model to be applicable there must also be no improvements or changes on the fan surface that would alter the random flow patterns on the fan surface.

Thomas Creek lacks many of the characteristics typical of an active alluvial fan. For example, the shape of the flood plain is concave rather than convex. The surface of the flood plain is stable and the upstream channel does not appear to have significant amounts of sediment available for transport. The Thomas Creek flood plain would be more appropriately modeled using traditional hydraulic methods.

Figure 2-2 was prepared from a copy of the current FEMA Flood Insurance Rate Maps showing the Thomas Creek 100 year flood zone based upon the alluvial fan methods.



NOTE: MAP AREA SHOWN ON THIS PANEL IS LOCATED WITHIN TOWNSHIP 18 NORTH, RANGE 19 EAST AND TOWNSHIP 18 NORTH, RANGE 20 EAST

NOTE: MAP AREA SHOWN ON THIS PANEL IS LOCATED WITHIN TOWNSHIP 18 NORTH, RANGE 20 EAST AND TOWNSHIP 18 NORTH, RANGE 21 EAST.
FLOOD HAZARDS SHOWN WITHIN WASHOE COUNTY IS FOR INFORMATION ONLY. FOR FLOOD INSURANCE PURPOSES REFER TO SEPARATELY PUBLISHED FLOOD INSURANCE RATE MAP.

N

0 1000 2000 3000

SCALE: 1"=1000'

SOURCE:
FEDERAL EMERGENCY MANAGEMENT AGENCY, MAP PANELS:
320019 1444C
320020 1463D

APRIL 16, 1990

Kennedy/Jenks/Chilton

THOMAS CREEK DETENTION BASIN
FEASIBILITY STUDY

FLOOD PLAIN MAP

K/J/C 897043.02
MAY 1990
FIGURE 2–2

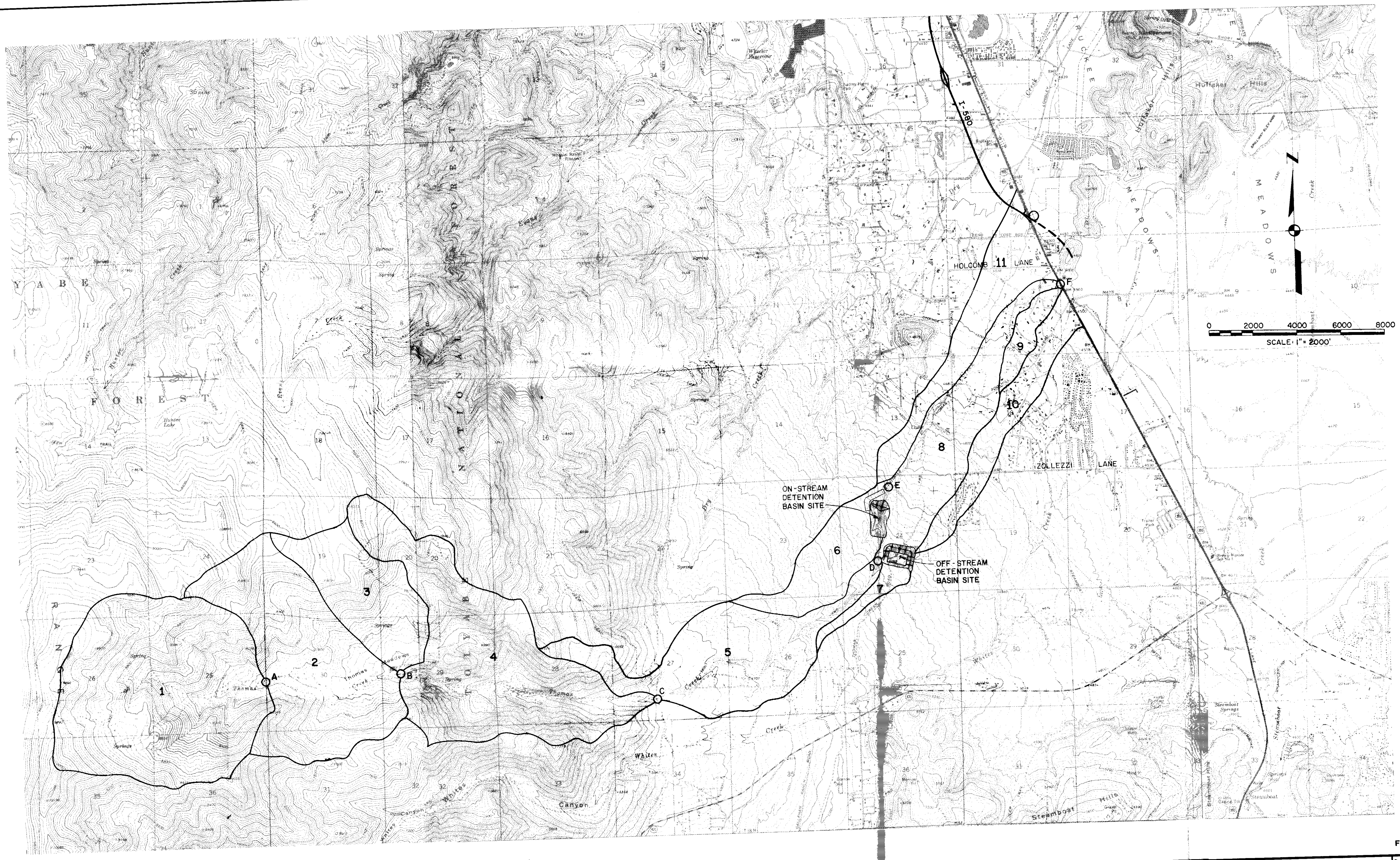


FIGURE 3-1

Rev'd	By	Date	Description

Reference Information and Notes:

Designed

Drafted
SPH

Checked

Date
MAY 1990

Washoe County &
Cities of Reno and Sparks, Nevada

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WASHOE COUNTY REGIONAL
FLOOD CONTROL MASTER PLAN

THOMAS CREEK DETENTION BASIN FEASIBILITY STUDY
WATERSHED MAP

Drawing No.

Job No.
897043.02

Sheet 1 of 1

CHAPTER 3

HYDROLOGIC ANALYSIS

INTRODUCTION

The hydrologic analysis performed as a part of this study is intended to provide the following information:

- Evaluate the effectiveness of various detention basin configurations and locations, to reduce peak discharges in the Thomas Creek flood plain area between the dam site and South Virginia Street.
- Estimate the storage volume required to accomplish the level of protection desired.
- Estimate the probable maximum flood discharge for the purpose of estimating the costs of any required spillway structures.
- Evaluate the impacts of the proposed detention facility on the Federal Emergency Management Agency (FEMA) flood zones for Thomas Creek.

This chapter will provide an overview of the methods and critical assumptions used in development of this information.

PREVIOUS STUDIES

During the initial phase of this project a literature search was performed to identify existing information sources. The following sections describe previous engineering and hydrologic studies which are of relevance to this project.

FEMA Flood Insurance Studies

Thomas Creek was studied by FEMA's study contractor is a part of the original Flood Insurance Study for Washoe County and the City of Reno. This study was performed by Tudor Engineering between 1977 and 1980. Peak discharge estimates for the 10, 50, 100 and 500 year events were estimated using statistical methods. Since flow records were not available for Thomas Creek, these estimates were obtained for Thomas Creek by comparative analysis using stream gage data from gages throughout the study area. These estimates are published in the current Flood Insurance Study as shown in Table 3-1.

TABLE 3-1
FEMA Peak Discharges

Location	Drainage Area (Sq. Mi.)	Peak Discharges 10 YR	50 YR	100 YR	(cfs) 500 YR
Thomas Creek Above U.S. 395	9.3	300	980	1,500	4,100

The discharge values reported in the FIS are the minimum discharge values that FEMA will require the community to use for Flood Plain Management purposes (regulatory discharges).

The FIRMs published for Thomas Creek (City of Reno Map Panel 1463D, Washoe County Map Panels 1444C, 1461C, and 1465C) also identify limits of 100 year flooding for Thomas Creek. These 100 year flooding limits are based upon the alluvial fan mapping methods described by FEMA (FEMA 1985, Dawdy 1979, Dawdy 1986).

A Flood Insurance Re-Study is currently in progress which includes Thomas Creek and Dry Creek. This study will modify the peak discharge data and water surface profiles for portions of these two steams. The results of that study are not yet available.

Corps of Engineers

The Sacramento District office of the U.S. Army Corps of Engineers has prepared several hydrologic studies for the Truckee Meadows during the last 15 to 20 years. The most detailed of these studies was a hydrology report for the Truckee River Basin completed in 1980. This report summarizes the results and input data from a hydrologic modeling study for the Truckee River Basin and summarizes the results of a statistical analysis for several of the watersheds within the Truckee River Basin. This report provides the following results for Thomas Creek:

TABLE 3-2
Corps of Engineers Peak Discharges

Location	Drainage Area (Sq. Mi.)	Peak Discharges 10 YR	20 YR	50 YR	(cfs) 100 YR
Thomas Creek at Steamboat Ditch	11.4	340	620	1,350	2,500

Summit Engineering Corporation

The City of Reno Public Works Department contracted with Summit Engineering Corporation in 1985 to conduct a hydrologic analysis of the City of Reno's major drainage basins. This report was completed in October 1985.

The Summit report utilized the precipitation depth-duration-frequency data and rainfall maps presented in the earlier report prepared for the City of Reno (Winzler and Kelley, 1984).

The peak discharges computed for Thomas Creek are as follows:

TABLE 3-3
Summit Engineering Peak Discharges

Location	Drainage Area (Sq. Mi.)	Peak Discharges 5 YR	25 YR	50 YR	(cfs) 100 YR
Thomas Creek					
at Steamboat Ditch		1,894	3,648	4,949	5,561
at U.S. 395		1,912	3,688	5,005	5,625

In 1988 the City of Reno removed the rainfall isopleth maps that were used in this study from their Public Works Design Manual. The values reported in the Summit study have been considered to be excessively high due to the unrealistic peak rainfall intensities resulting from application of the Winzer and Kelly rainfall data.

Huffaker Hills Detention Basin Feasibility Studies

The Huffaker Hills Detention Facility Feasibility Study (Nimbus Engineers, 1990) presents a hydrologic analysis and design concept alternatives for a detention dam on Steamboat Creek at Short Lane. This report also presents a single design concept drawing for a detention basin on Thomas Creek.

The Sacramento District Corps of Engineers have also been performing a detention structure feasibility study for a similar facility on Steamboat Creek at Short Lane. The Corps has performed hydrologic modeling for several design events including the standard project and probable maximum floods.

The hydrologic models developed for these two studies were utilized in this study with appropriate modifications.

RECURRENCE INTERVAL

The recurrence intervals of primary interest for evaluating a detention facility are the 100 year and probable maximum events. The 100 year event is used as the design event for the detention basin and evaluation of the basins impacts on the downstream flood plain. The probable maximum event is used to evaluate the need for an emergency spillway for the detention basin embankment.

The 100 year event is defined as the event which has a 1% chance of being equalled or exceeded during any given year. In the absence of sufficient data to determine the 100 year peak discharge using statistical methods, it is generally assumed that a 100 year rainfall event will produce a reasonable estimate of the 100 year peak discharge from the watershed. This assumption requires the use of a method or hydrologic model which translates the rainfall depth to a corresponding runoff depth from the watershed and ultimately a peak discharge. Hydrologic models are also used to develop hydrographs for various storm durations and patterns, which are needed to evaluate reservoir operation.

The probable maximum event is defined by the Corps of Engineers as "the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region." Determination of the probable maximum flood (PMF) is usually accomplished by applying the probable maximum precipitation (PMP) event to the watershed with the watershed conditions that would result in the greatest likely runoff.

For the purpose of this study, it was necessary to estimate the hydrograph shape as well as the peak discharges for the events of interest. The 100 year peak discharges computed for this study were developed using the assumption that a 100 year, 24 hour hypothetical storm will produce a reasonable estimate of the 100 year peak discharge. The probable maximum flood model developed by the Corps of Engineers for the Huffaker Dam Feasibility studies, were used to develop estimates of the PMF discharge at the basin locations.

CHOICE OF HYDROLOGIC MODEL

Computation of design peak discharges can be performed using a variety of common methods. The U.S. Army Corps of Engineers Flood Hydrograph Package, HEC-1 was the model selected for use in this study. The basic HEC-1 model used for this study is the model developed by Nimbus Engineers for the FEMA Flood Insurance Re-study currently in progress.

The model for Thomas Creek developed as a part of this study was included into the Huffaker Hills detention basin feasibility study first draft, with some minor adjustments to some of the model parameters. This model was modified by Kennedy/Jenks/Chilton to provide concentration points and parameters applicable to this application.

METHODS OF COMPUTING FLOOD HYDROGRAPH

The methods selected for computing the 100 year flood hydrographs, using the HEC-1 model described above, were based on the SCS methods (Refs. 11 and 12). The SCS curve number method was used to determine loss rates and the resulting rainfall excess. The SCS curve number method computes rainfall excess based upon relative soil moisture, soil type, percentage of the watershed area that is impervious and vegetation type and cover density. The SCS "lag time" and the SCS dimensionless unit hydrograph were used to develop the hydrograph shape.

The models use the following input parameters to compute the runoff hydrographs using these methods:

1. Watershed area (square miles)
2. Curve number - Estimated based upon soil type, relative soil moisture content and vegetation type and cover density (dimensionless)
3. SCS watershed lag time - Defined as the time difference between the center of mass of the rainfall excess and the time of peak discharge (hours)
4. Rainfall depth for period of interest (inches)
5. Time distribution of rainfall - Input as a mass curve or incremental rainfall depths when using a predefined distribution or input as depths for durations of five minutes to the period of interest when the hypothetical storm option is used. The hypothetical storm option was used in this study.
6. Spatial distribution of rainfall - Input as a storm area in square miles when the hypothetical storm option is used. HEC-1 then uses the depth-area reduction factors presented in NOAA Atlas 2 (HEC, 1988).

WATERSHED AREA

The watersheds were delineated on USGS 7.5 minute quadrangle maps (Figure 3-1). The area of each watershed was measured using a planimeter.

CURVE NUMBER

The curve number is based upon soil type, relative soil moisture content, vegetation type and vegetation cover density.

Soil Type - is typically derived from the U.S. Soil Conservation Service soil surveys. The soils identified in the survey are classified into one of four "hydrologic soil groups"; A, B, C and D. Type A soils are soils with a very low runoff potential which are very porous soils such as sandy soils. Type D soils are soils which have a high runoff potential such as very rocky soils or soils with a shallow impervious layer.

The Soil Conservation Service has published a soil survey for southern Washoe County. This soil survey data includes a description of each major association and its characteristics and hydrologic soil group. This information was used to identify soil types for this study.

Relative Soil Moisture Content is described by the SCS using the term "antecedent moisture condition" (AMC). Three different antecedent moisture conditions are identified for estimation of curve number; AMC I, AMC II, and AMC III. AMC I is a condition in which the soil moisture has been depleted by a relatively long period of no rainfall. This condition is assumed to represent the condition in which the soil's infiltration capacity is at the highest point. AMC III is the condition in which soil moisture is high due to recent rainfall or snowmelt. This condition is assumed to be the condition in which the soil infiltration capacity is at its lowest point. AMC II is an average condition and is the condition usually assumed to be present in the watershed for most hydrologic studies of this type. AMC II was the condition used in this study.

Vegetation Type refers to the basic type of plant community which occupies the watershed. The SCS has identified curve numbers for various typical plant communities. The vegetation types which best describe the plant communities encountered in the study area include; "sagebrush with grass understory," pasture, oak-aspen, and woodland.

Vegetation Cover Density refers to the average percentage of the soil surface covered by plant canopy and litter. The plant canopy and litter intercept a portion of the rainfall which reduces the runoff potential. A vegetation type with a thick and absorbent plant canopy would have a lower curve number than a vegetation type with a sparse canopy for the same hydrologic soil group and cover density.

Using the selected soil types, AMC II condition and identified vegetation types, the curve number for each watershed was estimated based on data in the SCS NEH-4 and SCS TR-55 handbook.

Impervious cover estimates were developed for both existing and future conditions. The existing impervious area was estimated using current aerial photos. The future impervious area was based on current zoning data. The amount of existing and future impervious cover was found to be insignificant for the upper watersheds (1-4). Total impervious cover estimates for the lower watersheds were input directly into the model as a separate input from the curve number. HEC-1 computes the losses for the impervious areas separately from the pervious areas and sums the two values. Both methods assume that the impervious areas are evenly distributed throughout the sub-basin.

WATERSHED LAG TIME

Watershed lag time as defined by the SCS as the time difference between the centroid of the rainfall excess and the peak runoff from the watershed. Time of concentration is defined as the time required for a particle of water to travel from the hydraulically most distant point in the watershed to the watershed outlet. The SCS has developed an empirical relationship between the time of concentration and the lag time for a typical watershed. This relationship is:

$$\text{Lag} = 0.6 * T_c$$

where:

T_c = time of concentration in hours
Lag = watershed lag time in hours

The SCS has identified two basic methods for computing time of concentration and lag time; the curve number method and upland method. The curve number method uses average watershed slope, hydraulic length and curve number in an empirical relationship to estimate lag time. The upland method uses the relationship between the slope of the primary flow path and the conditions of the flow path to estimate the velocity of each individual reach of the longest flow path encountered in the watershed. The sum of the travel times for each reach is the estimate of time of concentration. The relationship between slope and velocity is typically taken from a chart contained in the SCS National Engineering Handbook (Figure 15.2 of Ref. 14).

Both of these methods are based upon overland flow being the dominate condition in the watershed. TR-55 recommends that a reach by reach analysis be performed when using the upland method and velocities be based upon approximate or surveyed cross sections when channel flow is encountered (velocity method). McCuen, Wong and Rawls (1984,) also concluded that the velocity method was the most accurate method of estimating time of concentration when the velocities in the channel reaches are estimated based on the channel conditions.

The results obtained from the upland method chart for the watersheds in this study were compared with computed velocities to confirm the accuracy of the results.

RAINFALL DEPTH

Rainfall depths for each duration and recurrence interval were computed using NOAA Atlas 2 (NWS, 1973). Proper application of NOAA Atlas 2 requires reading the approximate values for the six and 24 hour durations and each of the recurrence intervals, from the maps. These values are plotted on a graph contained in NOAA Atlas. A "best fit" line is drawn to obtain the "corrected" values. Then corrected values are applied to a series of equations to estimate the values for the other durations. This method was utilized in this study.

Probable Maximum Precipitation (PMP) data was estimated using HMR 49 (NWS, 1979).

TIME DISTRIBUTION OF RAINFALL

The HEC-1 model also requires the use of a historic or synthetic distribution to distribute the total rainfall depth over the time period of interest. As indicated earlier, the hypothetical storm option in HEC-1 was used to distribute 100 year rainfall depths in this study. Use of this option results in a time distribution of rainfall which has the greatest intensity (five minute depth) in the center of the 24 hour time period. By subtracting the five minute depth from the 15 minute depth, the expected depth for the last 10 minutes of the 15 minute period is obtained. This depth is divided by two and placed on either side of the five minute depth in the distribution. This process is continued for the entire 24 hour period. The resulting rainfall pattern is triangular in shape with the peak intensity at the center of the 24 hour period. Use of this type of distribution is generally conservative, since it assumes that a 100 year, 24 hour event would also include 100 year depths for the shorter durations as well. The short duration, 100 year values would typically be produced only by shorter duration storms.

The time distribution for the PMP was based on the guidelines in HMR 49.

SPATIAL DISTRIBUTION OF RAINFALL

The 100 year rainfall values obtained from NOAA Atlas 2 are statistical point rainfall depths. A storm that would produce this rainfall depth will typically have an elliptical shape and pattern. In mountainous regions the shape and pattern of the rainfall event can be more complex since the pattern may be governed by orographic influences. The depths within a storm pattern will decrease as you move away from the storm center. This spatial distribution of rainfall is usually represented in the form of a reduction factor vs. storm area relationship. The point rainfall value is reduced based on the storm area that is representative for the watershed of interest. In many cases it is assumed that the storm area and watershed area are equivalent. In the case of a long and narrow watershed with a storm pattern that is elliptically shaped and oriented perpendicular to the watershed, the storm area could be significantly greater than the watershed area. Assuming that storm area is equivalent to watershed area, would often be a conservative assumption.

Aerial reduction factors have been developed by NOAA for use in reducing point rainfall values based upon storm area. The first set of curves were developed with NOAA Technical Paper 40 and later included in NOAA Atlas 2. HEC-1 utilizes these factors when the hypothetical storm option is selected. Due to the size of the watersheds in this study, aerial reduction of the rainfall is not very significant.

Aerial reduction for this study was based upon a storm center approach. Watershed 4 was selected at the storm center applying an aerial reduction factor for a two square mile storm. The aerial reduction was increased moving away from the storm center by applying a greater storm area as you move away from the storm center with a maximum storm area of 15 square miles.

METHODS FOR ROUTING HYDROGRAPHS THROUGH CHANNEL REACHES

Routing of hydrographs from one concentration point to another was performed using the Muskingum Method (USCOE, 1960). The Muskingum method uses the following parameters in the routing computations:

1. Muskingum X - Muskingum parameter X is a dimensionless empirical value based upon depth of flow and channel geometry. This value is determined from calibration if observed data is available. In the absence of calibration data, this parameter can be estimated from channel conditions. This value varies from 0 to 0.5 with the value of 0 being representative of a reservoir and 0.5 is representative of a channel with little or no available storage which results in the hydrograph being translated through the reach with no change in shape. The lower the value of X, the more attenuation of the hydrograph occurs.

2. Muskingum K - K is the storage time coefficient for the channel reach. It can be shown that K is approximately equivalent to the travel time of the flood wave through the channel reach (USCOE, 1960). With available data, K would be obtained from calibration. In the absence of calibration data, K can be estimated by estimating the travel time of the flood wave through the routing reach using the velocity method and a factor that relates the velocity of the flood wave to the channel velocity (based on channel geometry). The translation time of the hydrograph peak is related to the value of K.

3. NSTPS - This parameter is the number of steps in the routing reach computations (HEC, 1988). This parameter is a HEC-1 input variable that is obtained from an iterative solution of a set of equations presented in the HEC-1 manual. The solution of NSTPS is based upon X, K and the computational time interval selected in the model. If the time interval, X or K are modified, NSTPS may also need to be modified. The equations presented in the HEC-1 manual will often yield a range of possible values for this parameter. The selected value must be within this range in order to assure computational stability. The greater the number of steps selected, the more attenuation of the hydrograph will occur.

The value of X was estimated based on channel conditions. A value of X near the high end of the range of possible values was selected in all cases. The value of X selected for the channels in this study varied from 0.25 to 0.4.

The value of the parameter K was estimated based on a reach by reach velocity method analysis. NSTPS was computed based upon the equations contained in the HEC-1 manual and the computational time interval of five minutes and the estimated values of K and X. When a range of values for NSTPS was computed.

RESERVOIR ROUTING

Reservoir routing was performed using the modified Puls reservoir routing method. This method is also referred to as the storage indication method. The model inputs include:

1. Stage-Storage Data. The model requires information on the available storage in the reservoir for each elevation from the elevation of the top of the Dam to the lowest point in the reservoir. This can be input as elevation vs. storage data or the user can input elevation vs. area data (area under each contour of interest). The elevation vs. area data is converted by HEC-1 into elevation vs. storage data using the conic method.

2. Stage-Discharge Data. The model requires information on the outflow from the reservoirs low level and spillway outlets for each elevation of interest. The data can be input in this format or the dimensions of the structures can be input to allow HEC-1 to apply the orifice and weir flow equations to compute the needed data.

These data are used by HEC-1 to model outflow from the reservoir using a form of the basic continuity equation; Inflow - Outflow = Change in Storage (I - O = S).

This method of hydrograph routing was applied to each detention basin evaluated with this study. The storage relationship was obtained using computed elevation-storage data. The spillway and low level outlet configurations were input to allow HEC-1 to compute stage vs. discharge data.

MODEL STRUCTURE

100 Year Model

The watershed was subdivided into 11 sub-basins. The upper five sub-basins were subdivided due to the considerable variability in 100 year rainfall depths between the upper and lower portions of the Thomas Creek watershed. The 100 year, 24 hour rainfall depths vary from almost 6.0 inches at the upper drainage divide to 3.0 inches at the lower end of sub-basin #5. When watershed conditions or rainfall depths vary considerably across a watershed, the watershed should be further subdivided into more homogeneous units. Figure 3-2 shows the 100 year depth-duration curves for selected watersheds in order to demonstrate the differences in rainfall depths within the Thomas Creek watershed.

The lower region of the Thomas Creek watershed was subdivided to obtain discharges at key concentration points such as the detention basin locations and key points at South Virginia.

Table 3-4 summarizes the watershed data used in the development of the models.

PMF Model

Two probable maximum flood models were utilized in this study; the General Rain Event Model and Cloudburst Event Model. These models were for evacuating the spillway performance of the On-Stream Detention Basin alternative. The basic models developed by the Sacramento District Corps of Engineers for the Huffaker Detention Basin Feasibility Studies were used for this purpose. The Thomas Creek portion of the Corps model included two sub-basins. The upper portion of the watershed was delineated to Steamboat Ditch. The lower sub-basin is the area between Steamboat Ditch and South Virginia Street. Only the upper portion of the Thomas Creek model was used in the evaluation of the dam performance. This model utilizes a unit hydrograph generated from the Truckee Meadows S-Graph developed by the Corps as a part of the earlier Truckee River Study (COE, 1980).

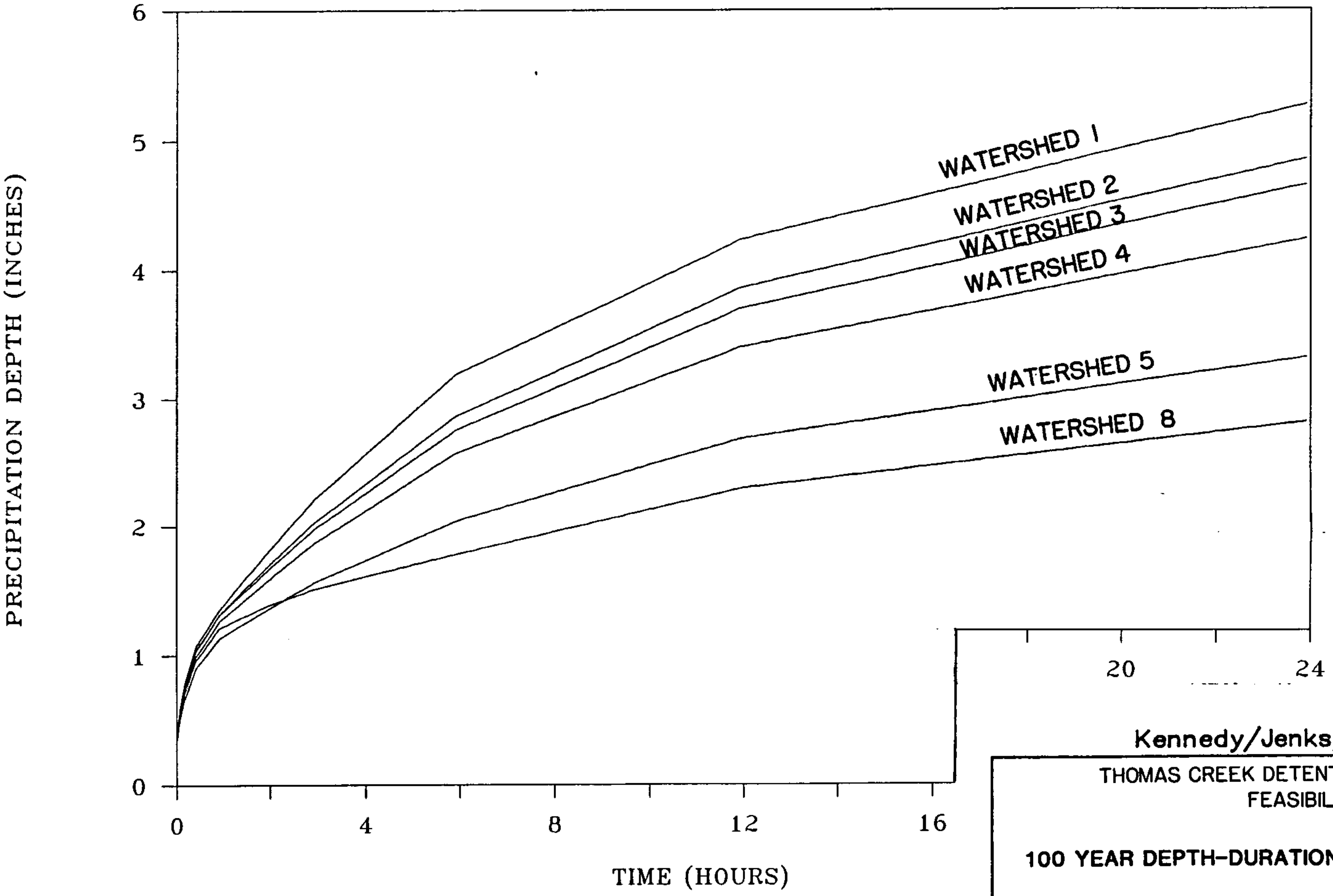
The general rain event is a 72 hour storm that contributes an average rainfall depth of approximately 19 inches. This event produces a peak discharge of 5334 cfs at the Steamboat Ditch (location of dam sites evaluated).

The Cloudburst Model utilizes a three hour rainfall event which contributes 8.06 inches of rainfall on the upper Thomas Creek watershed. The Cloudburst PMF event results in significantly higher peak discharges at Steamboat Ditch, this event produces almost 17,000 cfs at Steamboat Ditch. Since the spillway design criteria utilizes a 1/2 PMF event, the model needed to be adjusted to produce approximately 8,500 cfs. This was accomplished by reducing the total rainfall until this discharge was achieved. The rainfall depth which produces this discharge is 4.4 inches.

Both models were utilized without modification other than to insert the detention basin and reduce the cloudburst rainfall. The rainfall depth for the Cloudburst event was verified using HMR 49 (NOAA, 1977). The basin area used by the Corps at Steamboat Ditch is greater than the contributing area at the selected sites, therefore, the PMF discharges are probably conservative. More detailed PMF studies will need to be performed prior to final design.

100 YEAR
PRECIPITATION DEPTH–DURATION CURVES

FROM NOAA ATLAS



Kennedy/Jenks/Chilton

THOMAS CREEK DETENTION BASIN
FEASIBILITY STUDY

100 YEAR DEPTH–DURATION CURVES

K/J/C 897043.02
MAY 1990
FIGURE 3-2

TABLE 3-4
WATERSHED PARAMETERS

Watershed	Basin Area (Sq. Mi.)	CN	Lag Time (HR) Existing	Lag Time (HR) Future	Impervious % Existing	Impervious % Future
1	2.50	69	0.35	0.35	0	0
2	1.82	58	0.29	0.29	0	0
3	0.82	59	0.30	0.30	0	0
4	2.16	58	0.54	0.54	0	0
5	1.29	75	0.49	0.49	0	9
6	0.54	81	0.35	0.35	0	14
7	0.13	68	0.32	0.32	0	19
8	1.00	66	0.99	0.76	1	20
9	0.17	71	0.36	0.36	5	20
10	0.54	70	0.79	0.60	10	21
11	0.70	71	0.96	0.70	6	27

Table 3-5 summarizes the routing parameter data used in the channel routing reaches. For the purpose of this study, it was assumed that the primary channels between concentration points C and F would not be significantly modified by future development since current zoning indicates relatively low density development for these areas. Confinement of the flow to a narrow channel between concentration points E and F would result in an increased peak discharge at South Virginia Street.

TABLE 3-5
CHANNEL ROUTING PARAMETERS

Reach	NSTP	K (HRS)	X
A to B	1	0.12	.4
B to C	3	0.22	.4
C to D	2	0.19	.4
D to E	1	0.05	.4
E to F	3	0.31	.25
E to F (w/det)	2	0.21	.4

CHAPTER 4

STORM WATER DETENTION DAM AND DEBRIS BASIN ANALYSIS

GENERAL SITE SELECTION

The selection of the detention/debris basin sites for preliminary study were based on several variables including anticipated hydraulic and hydrologic efficiencies of the site, topographic considerations, land valuations and estimated construction costs.

The first step in establishing potential sites for the storm water detention facilities involved determining the upper most point in the watershed in which the 100-year flood flows from the area of the watershed downstream of that point exceed a target discharge rate at South Virginia. A target flow of between 400 and 500 cubic feet second (cfs) was selected at the proposed South Virginia Street conveyance structure. Therefore, a potential detention facility had to be located within the watershed and sized so that the routed outflow combined with the downstream runoff did not exceed the target flows at South Virginia Street.

Kennedy/Jenks/Chilton limited our analysis of potential detention sites to the canyon portion of Thomas Creek upstream of the alluvial fan apex. This limitation came after a cursory look at possible sites within the undeveloped areas of the fan, south of Holcomb Lane and west of South Virginia Street. Because of the nature of the fan, it is likely that the 100-year flood flows below the apex would spread and not be confined to the creek channel. This spreading effect would require a detention facility to be very wide or require diversion structures to capture and convey the flood flows into a detention reservoir. Additionally, the fan location would require a large surface area reservoir because of the relatively flat ground contours. This area appears to very developable property and a detention facility would require the acquisition of a considerable amount of expensive land. For these reasons, Kennedy/Jenks/Chilton excluded further analysis of potential detention sites within the fan area and concentrated our efforts in the canyon upstream of the apex.

Through the HEC-1 hydrologic model established for Thomas Creek, it was determined that the future 100-year runoff contribution from the fan portion of the watershed downstream of the apex of the alluvial fan was approximately 200 to 300 cfs. This set the following criteria:

1. The detention facility had to be sited as closely as possible to the apex in order to meet the target discharge rate at South Virginia Street;
2. The facility needed to be sized so that the routed discharge was in the range of 100 to 200 cfs.

DESCRIPTION OF ALTERNATIVE SITES

In all, Kennedy/Jenks/Chilton analyzed four potential sites within the canyon reach of Thomas Creek. The upper most site was located just upstream of the recently constructed Washoe County water storage tank in the center of the N¼ of Section 26, T18N, R19E. Two sites were explored in the vicinity of the existing water storage tank (approximately in the center of Section 24, T18N, R19E) at the west end of Zolezzi Lane, approximately 1500 to 2500 feet upstream of alluvial fan apex. These have been designated as the middle sites. The lower, fourth site studied was located at approximately the apex of the fan (adjacent to the quarter corner between Sections 13 and 24, T18N, R19E), very near the detention site indicated in the study prepared by Nimbus Engineers dated February 1990. The upper site, lower sites and one of the middle sites were on-stream facilities. The other middle site was situated off-stream. In accordance with scope of work, two of the above sites were selected for further, more detailed study. The two middle sites, one on-stream and the other off-stream, were selected for further study.

GENERAL DESIGN CRITERIA

The dam embankment, reservoir, low-level outlet works and spillway designs developed for this study are based upon accepted, conservative criteria. Multiple alternative analyses of various spillway and outlet works configurations were not undertaken. Kennedy/Jenks/Chilton's approach on this study was to develop reasonable, workable conceptual designs in which relative cost magnitudes could be determined and sites compared in an objective manner. The analysis of design alternatives for specific structures is more appropriate at a later date.

The dams were designed assuming the construction of compacted homogeneous embankments using on-site, native materials. Embankment slopes were held to a maximum of 3 horizontal to 1 vertical (3h:1v). Cut slopes were designed based on slopes of 2 horizontal to 1 vertical (2h:1v). These slopes are generally considered stable for most soil types, but may have to be modified based on actual geotechnical findings.

The detention dam designs did not consider extensive foundation preparation, foundation grouting or the inclusion of impervious zoning within the embankments. It was assumed that the detention dam reservoirs would be dry and would not impound water except during flood events. Normal, base flows would be carried through the facilities by the low-level outlet works without a significant headwater depth.

The reservoirs were analyzed based on the routed 100-year flood event volumes based on future development conditions. At the four sites studied, the reservoir storage required to route the 100-year flood while limiting discharge to between 100 and 200 cfs was in the range of 400 to 500 acre-feet. All reservoirs were therefore designed to provide storage in the 400 to 500 acre-feet range. Included in the storage volumes are allowances made for sediment deposition and debris.

Each facility required extensive excavation in the reservoir area to provide for the necessary storage volumes. The canyon is rather narrow and steep and does not offer a great amount of natural storage potential. In most places, the top width is only approximately 150 to 200 feet across. The channel flowline gradient approaches 7 percent. To limit earthwork quantities, the dam heights and the reservoir sizes were manipulated to achieve a somewhat optimum balance between cut and fill volumes.

The low-level outlet works was designed to allow for unregulated, metered discharge. The metering effect was provided by an appropriately sized orifice opening in the intake structure. As stated above, the inlet configuration limited the maximum discharge at the routed 100-year reservoir pool depth to between 100 and 200 cfs.

The intake structure was designed to allow for the passage of normal base flows while limiting the maximum discharge. The inlet elevations were set such that an allowance was made for debris and sediment storage. The intake structure was designed to have full trash rack protection, including sufficient size to accommodate normal clogging. The conveyance pipe was sized to flow at less than full depth and thus under no pressure. The low-level pipe was terminated downstream into impact stilling structure to dissipate the energy.

The spillway was designed to accommodate the ½ Probable Maximum Flood (PMF) event flows, which are approximately 8500 cfs. High hazard, high risk dams which pose significant threats to downstream persons or property are often required to provide spillway capacity for the full PMF event. However, the dams that were analyzed as part of this study do not pose significant downstream risks in our opinion. Given the small storage volume (500 acre-feet ±) of these facilities, a breach of a dam embankment during a PMF event would not increase the peak PMF flows substantially. Therefore, it is our assumption that the spillway design utilizing the ½ PMF event is valid and appropriate. This assumption must be confirmed by a dam break analysis and is beyond the scope of this project. The State Engineer of Nevada and possibly the Corps of Engineers will make the final determination of the design flood event for spillway design purposes. Based upon these determinations, the final design criteria may be different than was assumed for this study.

The spillways were designed to discharge the ½ PMF flows with appropriate allowances made for freeboard.

Kennedy/Jenks/Chilton analyzed two spillway alternative scenarios. The first scenario provided for a design that included full reinforced concrete lining, with an efficient ogee crest inlet, rectangular shaped chute and a standard USBR Type II stilling basin. This type of design was explored because of the alluvial nature of the watershed. Kennedy/Jenks/Chilton assumed that the characteristics of the subsurface soils would also be alluvial. Alluvial soils are known to contain coarse, unconsolidated materials which tend to be erodible. This fact can only be confirmed through extensive geotechnical sampling and testing and was not included in this scope of work.

The concrete lined spillway structure design provides the highest degree of safety to the dam embankment during extreme flood events. The flood flows would be contained within the spillway structure and be directed downstream, away from the dam embankment.

The second scenario studied was a spillway design which carried the flood flows around the dam and a sufficient distance away from the dam embankment prior to discharging onto native ground. The flood flows would be conveyed in an unlined channel excavated in the sidehill around one abutment to a broadcrested weir control structure and then directed and discharged down the original ground slope to the creek channel below. The discharge would be contained by training dikes which would direct the flows away from the dam embankment. Erosion and other damage to the existing ground and training dikes downstream of the control structure can be expected with this design. However, damage to the dam embankment would be unlikely.

DETAILED DESCRIPTION OF TWO DETENTION AND DEBRIS BASIN SITES

As indicated above, two sites were selected for greater analysis and study. They were the two middle sites, one located on-stream and the other located off-stream at the west end of Zolezzi Lane adjacent to the water storage tank. Both sites are approximately 1500 to 2500 feet upstream of the apex of the alluvial fan and located within Section 24, T18N, R19E, MDM. Figures 4-1, 4-2, 4-3 and 4-4 show the plans and details developed for the on-stream and off-stream facilities. These figures are contained in map pockets at the end of this Chapter.

These sites were selected for further study because both sites could be located on unimproved, unsubdivided land; they were close to the apex of the alluvial fan and thus would not require unreasonable storage volumes to maintain the targeted downstream discharge of between 400 and 500 cfs; and the ground topography was suitable for the construction of the detention/debris dam facilities.

Besides the obvious difference of on-stream (see Figure 4-1) versus off-stream (see Figure 4-4) locations, the two sites differ because the off-stream site will not require a primary spillway to convey the ½ PMF flows. The off-stream detention facility is configured such that the ½ PMF flows will remain in the Thomas Creek canyon rather than being routed through the detention reservoir. Sufficient freeboard on the dam embankment was provided to accommodate the depth of the full PMF flows at the point of diversion in the creek. The on-stream facility completely dams Thomas Creek and therefore must be designed to pass the ½ PMF flows without damage to the dam embankment. In this case, the on-stream detention facility requires that a spillway structure be incorporated into the design. Other differences between the two sites are that the on-stream facility required somewhat less earthwork than the off-stream facility because the natural canyon made up a significant portion of the reservoir storage volume and the embankment was less substantial even though it was approximately 40 feet higher because of the narrow canyon. The off-stream detention facility required extensive excavation over a large surface area coupled with a long embankment structure to provide for the necessary reservoir volume. The off-stream facility also differed in that an open channel diversion structure had to be constructed in Thomas Creek to divert the 100-year flood flows into the off-stream reservoir.

Both facilities had very similar low-level outlet works. The main differences were in the lengths of the low-level pipe conduit and the size of the pipe. The intakes, low-level outlet works and outlet structures were assumed to be the same in the analysis of the two detention facilities. The intake structures were designed utilizing Los Angeles County Flood Control District guidelines and standard details. The intake invert elevations are set to allow for debris and sediment storage. The intakes also make allowances for clogging of the portal openings.

The low-level outlet works for the On-Stream facility was designed using a 36 inch diameter reinforced concrete pipe, encased in 18 inches of reinforced concrete. The Off-Stream facility pipe size was 48 inches in diameter. The pipe sizes were selected to carry the peak 100-year routed discharge at less than full pipe conditions, thus under no pressure. This is the safest condition for conveyance of flows under the dam embankment.

The outlet structures were developed using a standard United States Bureau of Reclamation impact stilling basin design. The flow velocities exiting the outlet conduits are high, in the range of between 15 to 25 feet per second. The impact stilling basins provide the necessary energy dissipation of the high velocity flows. Figure 4-2 shows the various details of the outlet works structure.

As indicated above, the Off-Stream Detention/Debris Basin does not require a primary spillway. The Off-Stream facility will be constructed in such a manner that the main channel of Thomas Creek is not dammed except for the channel works to divert the 100-year flood flows. The ½ PMF flows will generally remain in the main Thomas Creek channel, largely by-passing the Off-Stream detention basin. It is planned that a major flood in excess of the 100-year event would overtop the diversion works and likely breach the structure.

During the ½ PMF flood, the depth of flow in the channel adjacent to the diversion and the reservoir pool would be approximately at the same elevation. Water from the reservoir will flow back into the creek channel to join the ½ PMF flows. The portion of the dam embankment in this location must be protected from scour and erosion to prevent undercutting and the potential failure of the embankment structure. The design of the Off-Stream Detention Dam includes slope protection in this vulnerable location.

Two alternative spillway structures were analyzed for the On-Stream Detention/Debris Basin. One alternative incorporated a fully lined, reinforced concrete spillway structure which was designed to convey the ½ PMF flows without damage to the dam embankment or the spillway structure itself. This alternative would convey the flows through a flat, unlined trapezoidal channel 125 feet wide around the left abutment (as looking downstream) to the control structure. From this point on, the spillway is fully reinforced concrete lined. A hydraulically efficient ogee weir control structure 75 feet in width discharges into the chute section, which has also been designed at a 75 foot width. The chute carries the discharge into a United States Bureau of Reclamation Type II design stilling basin which is 75 feet wide by 100 feet long with 25 foot high walls. The walls of the chute are generally 5 feet high. The total length of the spillway from the inlet to the end of the stilling basin is 1350 feet, with 800 feet being concrete lined. Figure 4-1 shows the plan of the reinforced concrete spillway alternative and Figure 4-2 contains an profile and miscellaneous details.

The second alternative was designed to convey the ½ PMF flows through a largely unlined channel. The inlet portion to the control structure of this spillway alternative generally is the same as the lined spillway alternative described above. However, the control structure location is a little further downstream. The control structure was designed as a broad-crested weir, 100 feet wide. The inlet channel immediately upstream of the weir and the weir structure will be constructed with reinforced concrete. A concrete cutoff will be installed under the downstream edge of the weir control structure to prevent head cuts in the spillway channel. The broad-crested weir control structure will discharge the flood flows onto the existing ground of the sloping canyon sidehill of Thomas Creek, utilizing a small natural drainage channel to carry (or direct) the lower flows coming over the spillway. Training dikes will be constructed on each side the existing, small drainage course to contain and direct the ½ PMF flows. It was assumed that ½ PMF flows would erode the original ground along the alignment of the existing channel, gradually widening to accommodate the full flows. The training dikes will serve to prevent erosion from expanding upstream to a point that could undermine the dam embankment. The downstream dike will help channel the flood flows directly back into Thomas Creek and prevent the flows from running downstream along the sidehill and impacting adjacent properties. Figure 4-3 shows the plan of the "unlined" spillway alternative.

No erosion protection for the existing ground downstream of the control structure or the training dikes will be provided and any resultant damage will require repair.

Table 4-1 below lists the hydrologic, hydraulic and physical comparisons between the two detention/debris basins studied.

TABLE 4-1

COMPARISONS OF ON-STREAM AND OFF-STREAM SITES

Description	On-Stream Detention/ Debris Basin		Off-Stream Detention/ Debris Basin
Thomas Creek			
100-Year Flood Flows[1], cubic feet per second	2,570		2,460
1/2 PMF Flows, cubic feet per second	8,500		8,500
Reservoir			
Routed 100-Year Storage Volume, acre-feet	428		470
Routed 1/2 PMF Storage Volume, acre-feet	627		730[2]
100-Year Pool Depth (from intake to water surface), feet	59.6		30.4
1/2 PMF Pool Depth (from intake to water surface), feet	70.6		43[2]
Freeboard, feet	15.4		17.7
Freeboard, feet	4.4		5.0[2]
Total Project Surface Area, including Dam, Reservoir, Spillway, etc. acres	80		70
Excavation Quantity, cubic yards	350,000		600,000
Dam			
Crest Height (at centerline), feet	95		53
Crest Length, feet	1,900		2,800
Crest Width, feet	20		20
Embankment Side Slopes, upstream/downstream	3:1/3:1		3:1/3:1
Embankment Fill Quantity, cubic yards	400,000		525,000
Spillway	On-Stream		Off-Stream
Alternate	"A"	"B"	
Routed 1/2 PMF Discharge, cubic feet per second	7,900	7,900	8,500[3]
Inlet Width, feet	125	125	N/A
Control Structure Width, feet	75	100	N/A
Chute Length, feet	650	N/A	N/A
Chute Width, feet	75	N/A	N/A
Chute Wall Height, feet	5	N/A	N/A
Stilling Basin Length, feet	100	N/A	N/A
Stilling Basin Width, feet	75	N/A	N/A
Stilling Basin Wall Height, feet	25	N/A	N/A
Spillway Excavation and Backfill, cubic yards	240,000	100,000	N/A
Outlet Works			
Routed 100-Year Discharge, cubic feet per second	204		145
Intake Orifice Size, square feet	5.06		5.06
Conduit Diameter, inches	36		48
Conduit Length	850		550

Notes:

1. All 100-year flood flows reported are based on Future Buildout Conditions of the tributary area.
2. The dam crest elevation of the off-stream facility was set to contain the pool depth resulting from the full PMF (17,000 ± cfs) flows in Thomas Creek. The storage volume pool depth and freeboard values reported above for the off-stream facility relate to the resultant pool elevation in the detention reservoir due to the full PMF depth of flow in the main channel of Thomas Creek.
3. Carried in the main Thomas Creek channel.

VISUAL IMPACTS

Both of the selected sites will be visible from the South Truckee Meadows lying below. The off-stream detention/debris basin will be more prominent than the on-stream site. With the 53 foot high embankment laid on the existing ground surface, the face of the off-stream embankment will be readily seen from most of the valley. Additionally, the cut slope to the south of the reservoir, especially above the top embankment elevation, will also be visible.

The on-stream facility will also be visible, but to a lesser degree. This is because the dam embankment is mostly located within the creek canyon. However, the embankment slopes, reservoir cut-slopes and the spillway structure will be seen to some degree from the valley below.

LAND OWNERSHIPS AND VALUATIONS

Washoe County provided Kennedy/Jenks/Chilton with assessor parcel maps and assessed values of the properties potentially affected by the detention/debris basins. To derive approximate market values for the properties, Kennedy/Jenks/Chilton multiplied the assessed value by 2.86. This figure is based on the assumption that assessed values set by the County Assessor represents approximately 0.35 of the market value. 2.86 is the reciprocal of 0.35. No other factors were used in the determination of land values for this study. A much more detailed analysis of the cost of the land will have to be done if the project moves forward beyond this stage. Land acquisition could play a major role in the feasibility of this project.

The on-stream detention/debris dam lies entirely within the Washoe County Assessor's Parcel Number (APN) 49-010-10 (see Figure 4-1). This parcel is currently owned by the Redfield Trust. The total on-stream project site requires approximately 80 acres to provide a 100 foot envelope around the site. Based on the assessed valuation of this parcel of $175.00 per acre and a market value of $500.00 per acre ($175 x 2.86 = $500.00), the total cost of the required land for the on-stream site is estimated to be $40,000.00.

The off-stream detention/debris dam lies on two parcels, Washoe County APN's 49-010-09 and 49-010-10 (see Figure 4-4). APN 49-010-09 is currently owned by the U.S. Government. As indicated above, APN 49-010-10 is owned by the Redfield Trust. The total off-stream project site requires approximately 70 acres of land, with 40 acres on the Redfield land and 30 acres on Government land. The assessed valuation of the U.S. Government land is $1,750.00 per acre and the market value is estimated to be $5,005.00 per acre. The Redfield land has the valuation as shown above. Therefore, based on the market valuations derived herein, the estimated cost of the land acquisition for the off-stream detention/-debris basin is $170,150.00.

COST ESTIMATES

Tables 4-2, 4-3 and 4-4 below list Kennedy/Jenks/Chilton's estimated project costs for the On-Stream detention/debris basin facility, together with Spillway Alternatives "A" and "B" and the Off-Stream detention/debris basin facility studied on Thomas Creek. These estimates are based on 1990 construction dollars and do not include facility operation and maintenance costs or engineering/project management costs for the design and construction of the facilities.

TABLE 4-2

ON-STREAM DETENTION/DEBRIS BASIN COST ESTIMATE WITH SPILLWAY ALTERNATIVE "A"

		Item Description	Unit	Quantity	Unit Price	Total Cost
1.		Main Dam				
	a.	Excavation & Embankment	Cubic Yards	400,000	$4	$1,600,000
2.		Spillway				
	a.	Excavation	Cubic Yards	240,000	$4	$ 960,000
	b.	Reinforced Concrete				
		Inlet		400		
		Chute		4,655		
		Stilling Basin		1,350		
		Total	Cubic Yards	6,405	$400	$2,562,000
3.		Outlet Works				
	a.	Excavation & Backfill	Cubic Yards	10,000	$4	$ 40,000
	b.	Inlet Structure	Lump Sum	Lump Sum	$20,000	$ 20,000
	c.	Pipe Conduit, 36"Ø with 18" Reinforced Concrete Encasement	Lineal Feet	850	$365	$ 310,250
	d.	Impact Stilling Basin	Lump Sum	Lump Sum	$20,000	$ 20,000
4.		Land Acquisition	Acres	80	$500	$ 40,000
						$5,552,250
			TOTAL ESTIMATE FOR ON-STREAM FACILITY		Use	$5,600,000

TABLE 4-3

ON-STREAM DETENTION/DEBRIS BASIN COST ESTIMATE WITH SPILLWAY ALTERNATIVE "B'

Item Description		Unit	Quantity	Unit Price	Total Cost
1.	Main Dam				
a.	Excavation & Embankment	Cubic Yards	400,000	$4	$1,600,000
2.	Spillway				
a.	Excavation	Cubic Yards	95,000	$4	$ 380,000
b.	Reinforced Concrete Inlet	Cubic Yards	1,600	$400	$ 640,000
3.	Outlet Works				
a.	Excavation & Backfill	Cubic Yards	10,000	$4	$ 40,000
b.	Inlet Structure	Lump Sum	Lump Sum	$20,000	$ 20,000
c.	Pipe Condudit, 36"0 with 18" Reinforced Concrete Encasement	Lineal Feet	850	$365	$ 310,250
d.	Impact Stilling Basin	Lump Sum	Lump Sum	$20,000	$ 20,000
4.	Land Acquisition	Acres	80	$500	$ 40,000
					$3,050,250
		TOTAL ESTIMATE FOR ON-STREAM FACILITY		USE	$3,100,000

TABLE 4-4

OFF-STREAM
DETENTION/DEBRIS BASIN COST ESTIMATE

	Item Description	Unit	Quantity	Unit Price	Total Cost
1.	Main Dam				
	a. Excavation & Embankment	Cubic Yards	600,000	$4	$2,400,000
2.	Diversion				
	a. Embankment	Cubic Yards	4,500	$4	$ 18,000
	b. Concrete Lined Diversion Channel	Lineal Feet	1350	$250	$ 337,500
	c. Channel Excavation	Cubic Yards	30,000	$4	$ 120,000
3.	Outlet Works				
	a. Excavation & Backfill	Cubic Yards	15,000	$4	$ 60,000
	b. Inlet Structure	Lump Sum	Lump Sum	$20,000	$ 20,000
	c. Pipe Conduit, 48"0 with 18" Reinforced Concrete Encasement	Lineal Feet	470	$380	$ 178,600
	d. Impact Stilling Basin	Lump Sum	Lump Sum	$20,000	$ 20,000
4.	Land Acquisition	Acres	70	$2430.71	$ 170,150
					$3,324,250
		TOTAL ESTIMATE FOR OFF-STREAM FACILITY		Use	$3,350,000

CHAPTER 5

THOMAS CREEK CHANNEL IMPROVEMENTS

INTRODUCTION

Several alternatives are available for conveyance of flows from the detention basin site to South Virginia Street. The primary alternatives include:

- No improvements downstream of detention basin
- Trapezoidal or landscaped channel from outlet of detention basin to Virginia Street
- Interceptor channel near Virginia Street

No Improvements Downstream of Detention Basin

This option would allow flows from the detention basin to be transported in the existing channel and drainage system. The majority of the flow from the basin combined with watersheds 8 and 11 would concentrate at the new I-580/Virginia Street Interchange.

The 100 year peak discharge at this location under future land use conditions with the on-stream or off-stream detention basins in place are 515 and 680 cfs respectively.

Since there are not adequate drainage facilities to intercept and convey this amount of flow safely to Dry Creek, this option is not considered to be adequate. This rate of flow is considerably less than the 1400 or 1800 cfs that would currently arrive at this location, but it is not sufficiently reduced to a rate that would prevent damage or safety hazards.

Channel Improvements

Channel improvements from the outlet of the detention basin to Virginia Street would provide a completion of the conveyance system. The options for cross section geometry and construction materials would primarily depend upon their adequacy to withstand potentially erosive velocities.

The channels designed for this option would need to convey the discharges summarized in Table 5-1.

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WASHOE COUNTY REGIONAL FLOOD CONTROL MASTER PLAN

THOMAS CREEK DETENTION BASIN STUDY

ON-STREAM DETENTION/DEBRIS DAM AND SPILLWAY ALTERNATIVE "A" PLAN

JOB NO. 897043.02 MAY 1990 FIGURE 4-1

Table 5-1
Peak Discharge Estimates
Downstream of Detention Basins

Facility	D/S of Facility (cfs)	U/S of South Virginia Street (cfs)	At South Virginia Street (cfs)
On-Stream	204	328	355
Off-Stream	145	493	537

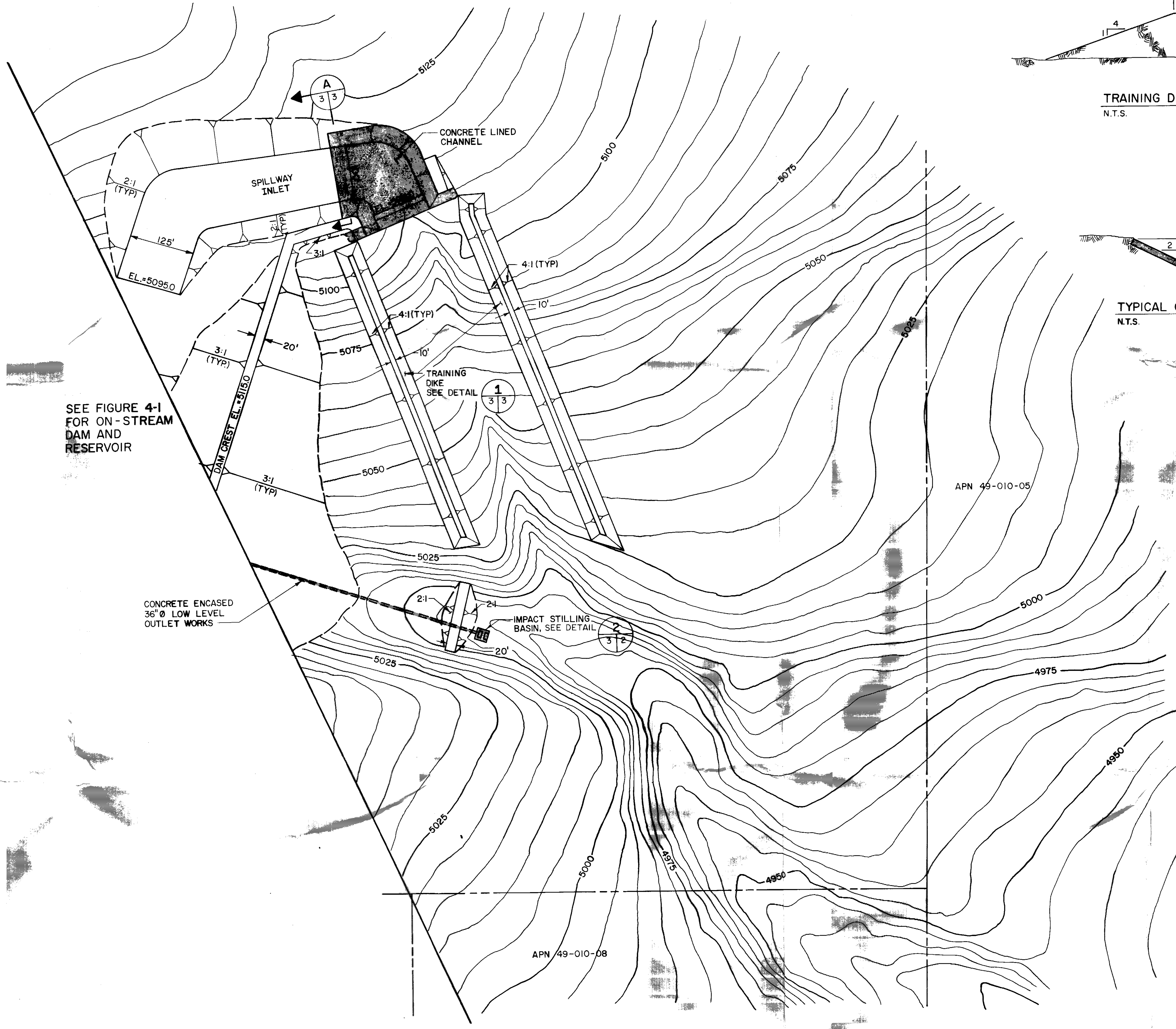
Interceptor Channel

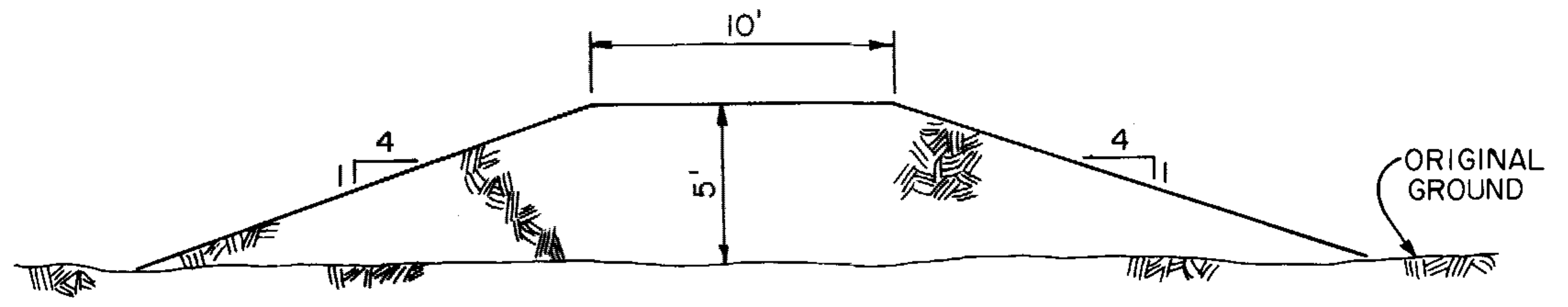
An interceptor channel would capture flows just upstream of the proposed box culvert at South Virginia. This option would reduce the costs in comparison to the previous option. Under this option, flows exiting the basin would be conveyed by the existing Thomas Creek channel for a distance of 11,000 to 14,000 feet before being captured by an interceptor channel. As development occurs along Thomas Creek, developers may need to provide improvements to Thomas Creek to provide adequate conveyance capacity within individual reaches of the stream.

FEMA FLOOD LIMITS

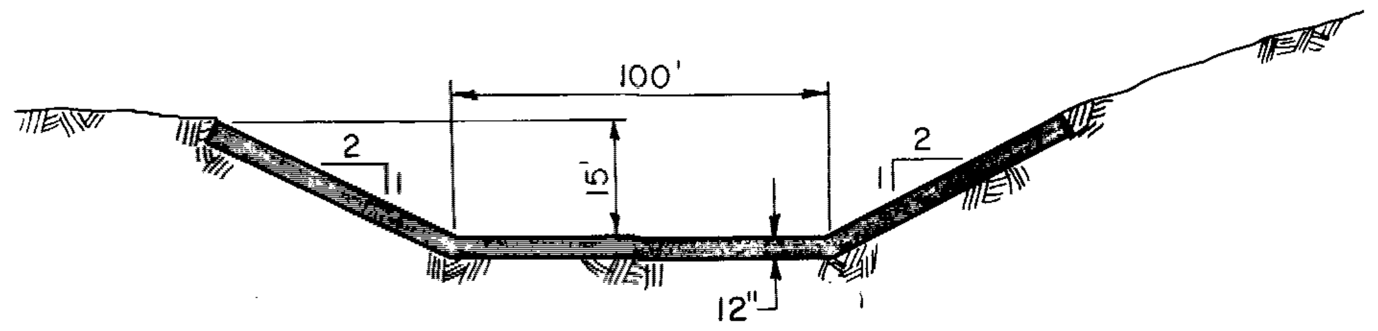
As discussed in Chapter 3, the flood limits for Thomas Creek are based upon the assumption that the lower portion of Thomas Creek is an active alluvial fan. In the past, FEMA has been resistant to allowing a watercourse to be re-classified from an alluvial fan to a standard riverine system once this designation has been established, despite the weight of the evidence to the contrary.

FEMA has traditionally required the construction of a debris basin and conveyance system before the alluvial fan flood limits can be removed. If the flow is substantially reduced by detention, and the existing downstream channel has adequate capacity to convey the reduced discharge, FEMA may allow reclassification of the system. If alluvial fan methods are still required downstream of the detention basin, mapping of the fan would be performed using the reduced discharges. This approach would result in a small Zone AO and a very large Zone B from the downstream end of the Zone AO to the top of the alluvial fan. Due to the low discharge released from the basin, there may be no Zone AO designation using these methods since the 100 year discharge may be lower than the discharge that produces the depth 1 zone boundary.





TRAINING DIKE DETAIL ① 3|3
N.T.S.



TYPICAL CHANNEL LINING SECTION Ⓐ 3|3
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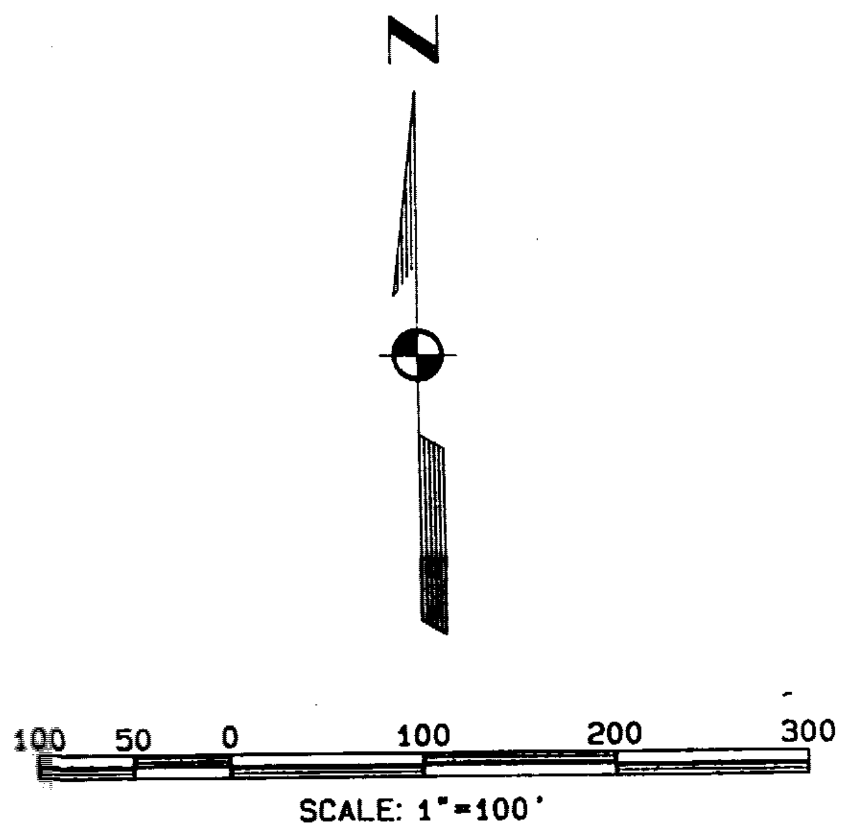

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WASHOE COUNTY REGIONAL FLOOD CONTROL
MASTER PLAN

THOMAS CREEK DETENTION BASIN STUDY

ON-STREAM SPILLWAY ALTERNATIVE "B"
PLAN

JOB NO. 897043.02 MAY 1990 FIGURE 4-3

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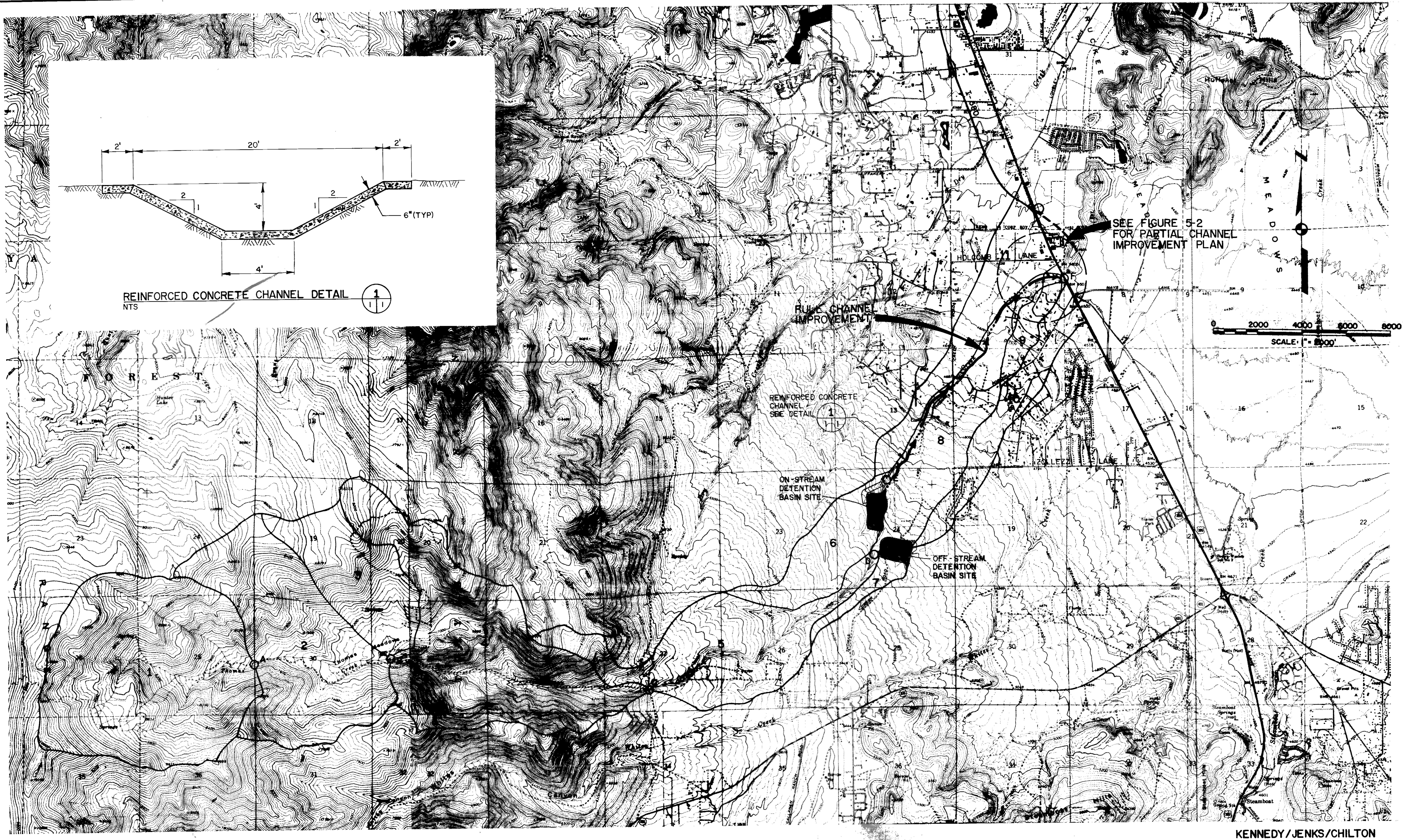
WASHOE COUNTY REGIONAL FLOOD CONTROL MASTER PLAN

THOMAS CREEK DETENTION BASIN STUDY

OFF-STREAM DETENTION/DEBRIS DAM PLAN

JOB NO. 897043.02 MAY 1990 FIGURE 4-4



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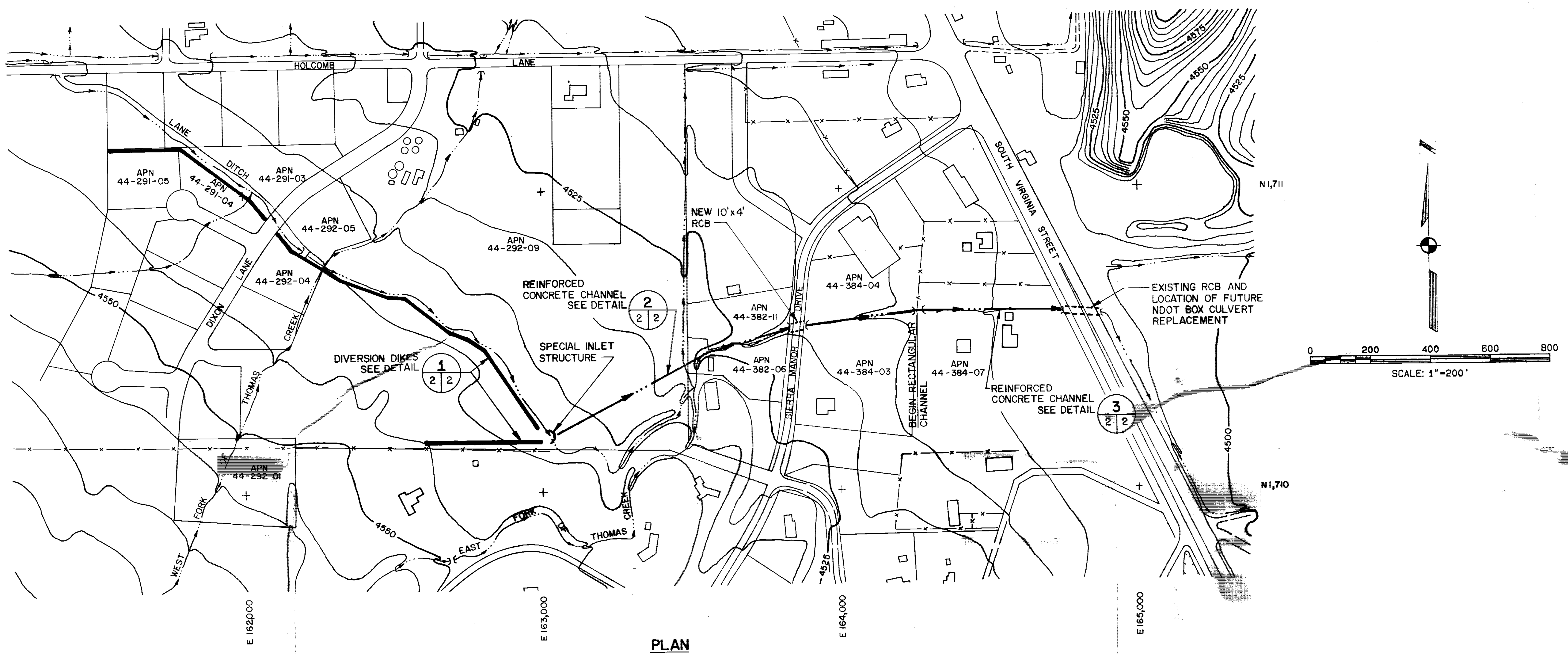
WASHOE COUNTY REGIONAL FLOOD CONTROL
MASTER PLAN

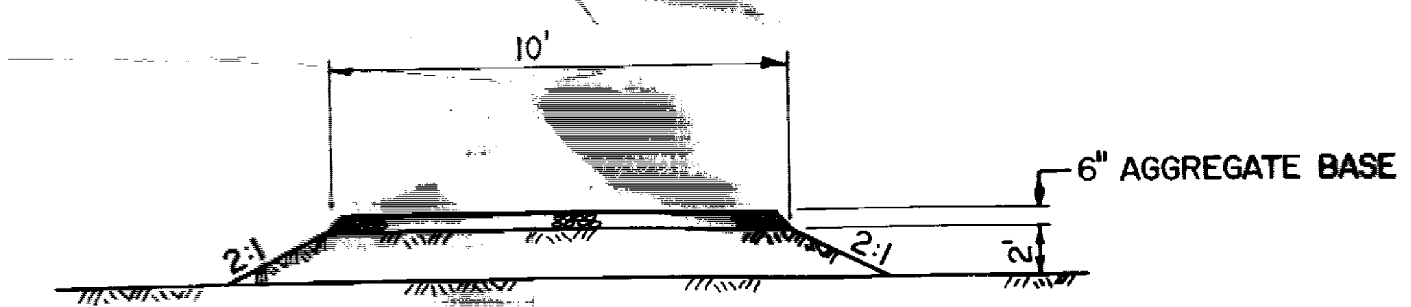
THOMAS CREEK DETENTION BASIN STUDY

FULL CHANNEL IMPROVEMENT ALTERNATIVE

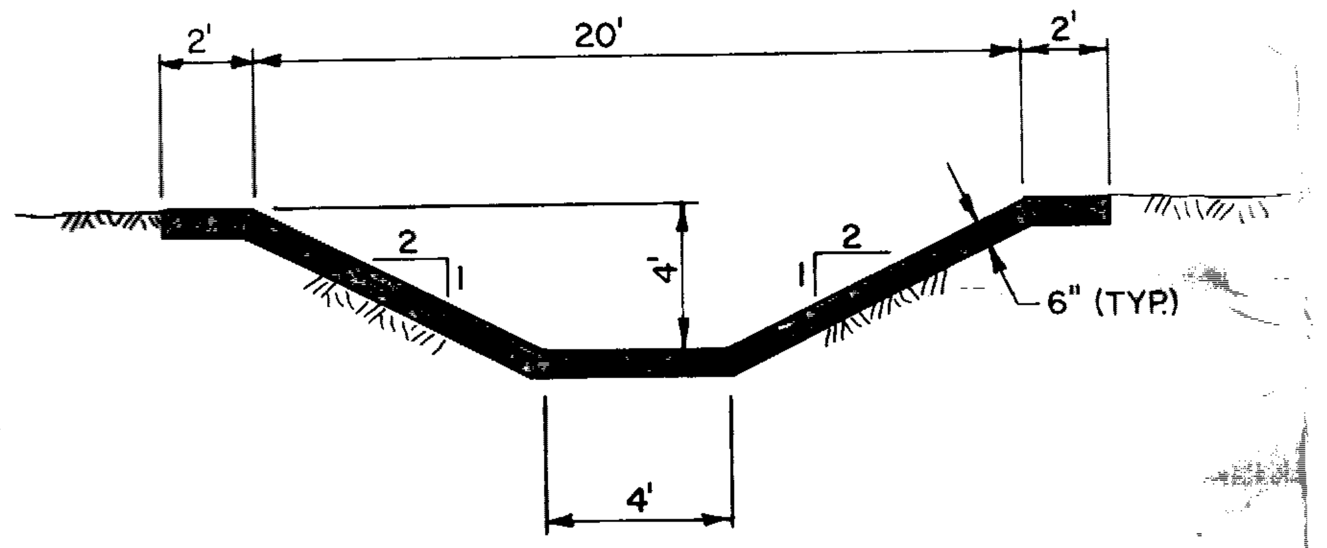
JOB NO. 897043.02 MAY 1990 FIGURE 5-1



PLAN



DIVERSION DIKE DETAIL ①
NTS



REINFORCED CONCRETE CHANNEL DETAIL ②
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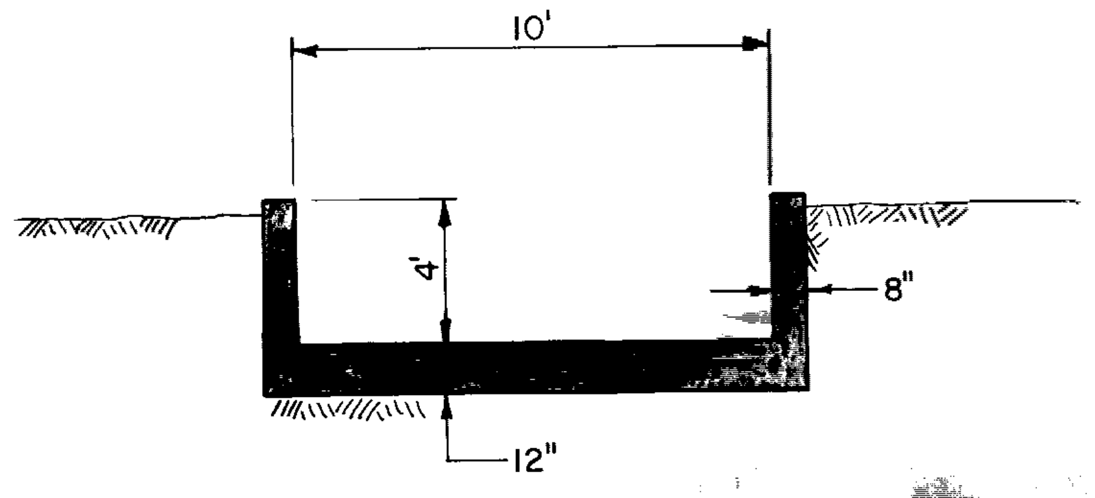


REINFORCED CONCRETE CHANNEL DETAIL ③
NTS

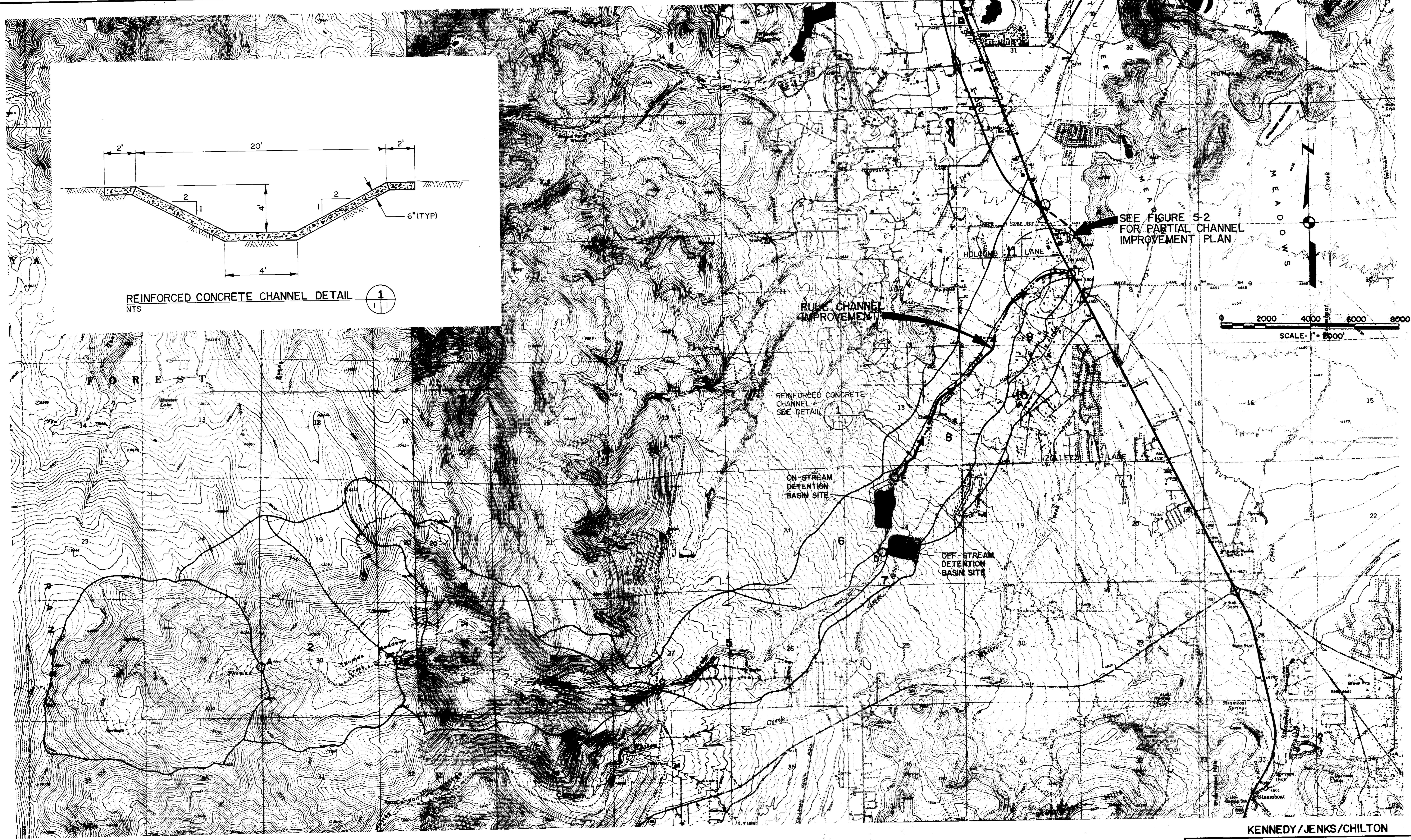
KENNEDY/JENKS/CHILTON

WASHOE COUNTY REGIONAL FLOOD CONTROL
MASTER PLAN

THOMAS CREEK DETENTION BASIN STUDY

PARTIAL CHANNEL IMPROVEMENT ALTERNATIVE

JOB NO. 897043.02 MAY 1990 FIGURE 5-2



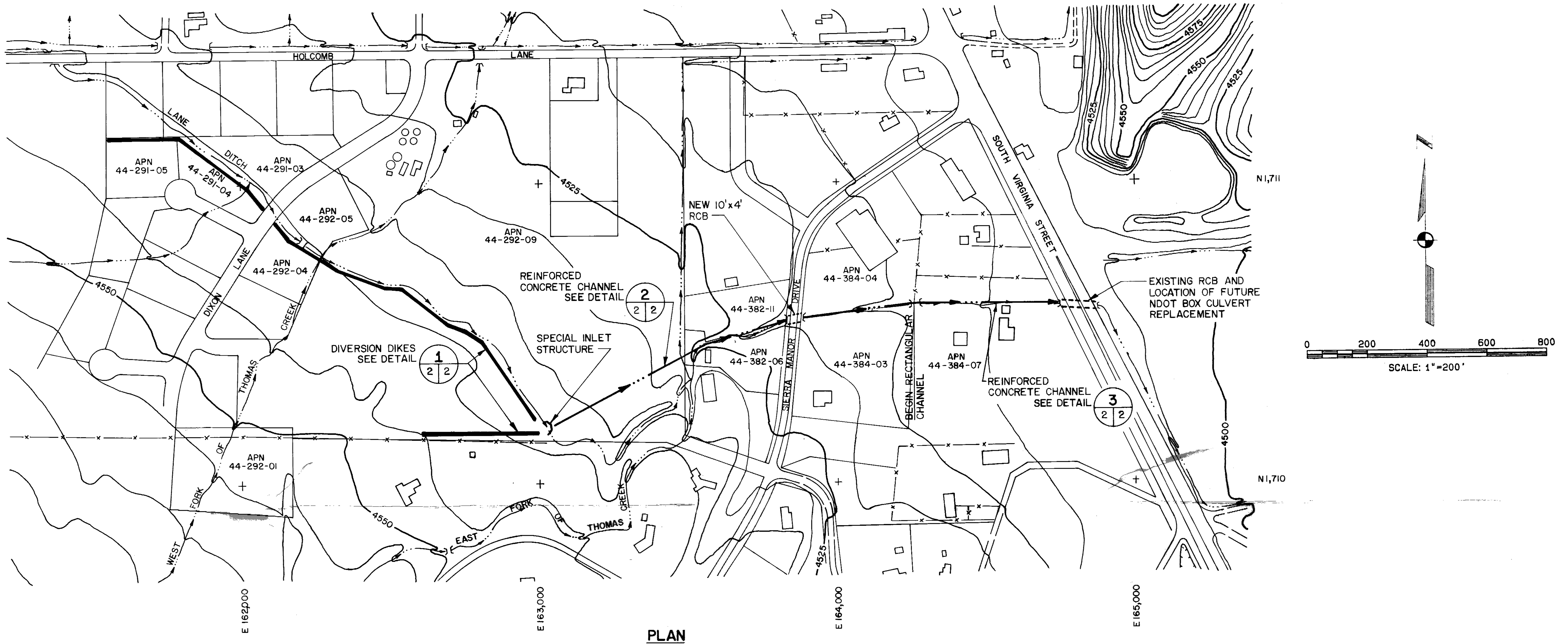
KENNEDY/JENKS/CHILTON

WASHOE COUNTY REGIONAL FLOOD CONTROL
MASTER PLAN

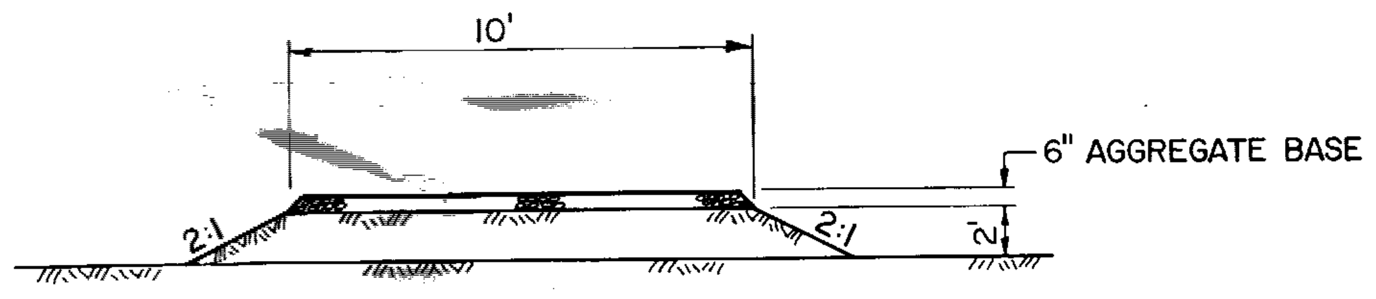
THOMAS CREEK DETENTION BASIN STUDY

FULL CHANNEL IMPROVEMENT ALTERNATIVE

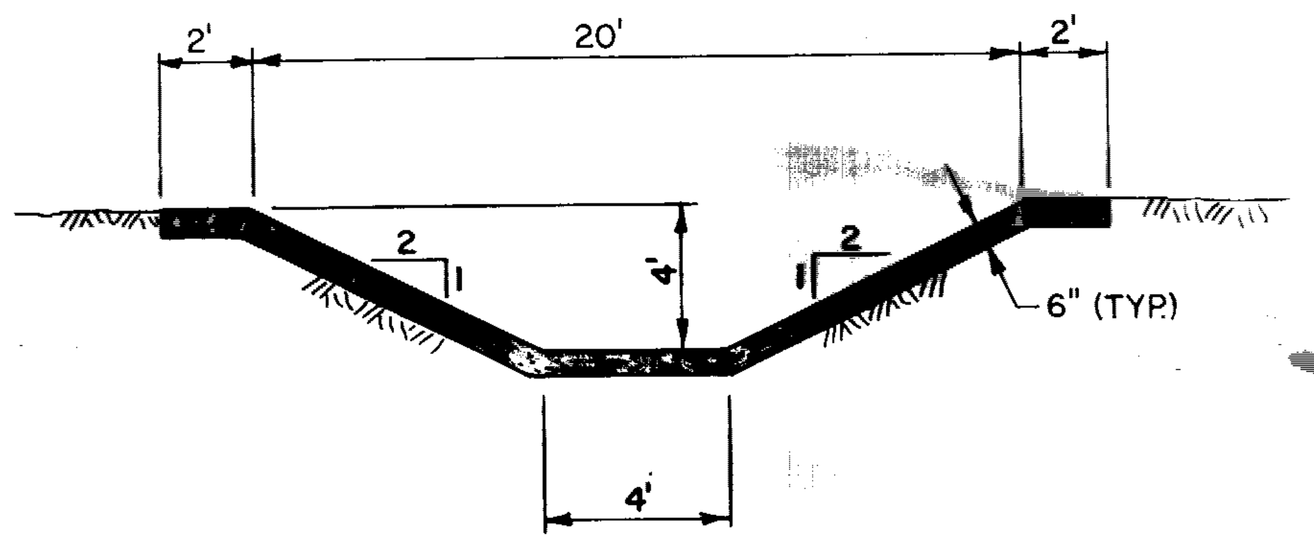
JOB NO. 897043.02 MAY 1990 FIGURE 5-1



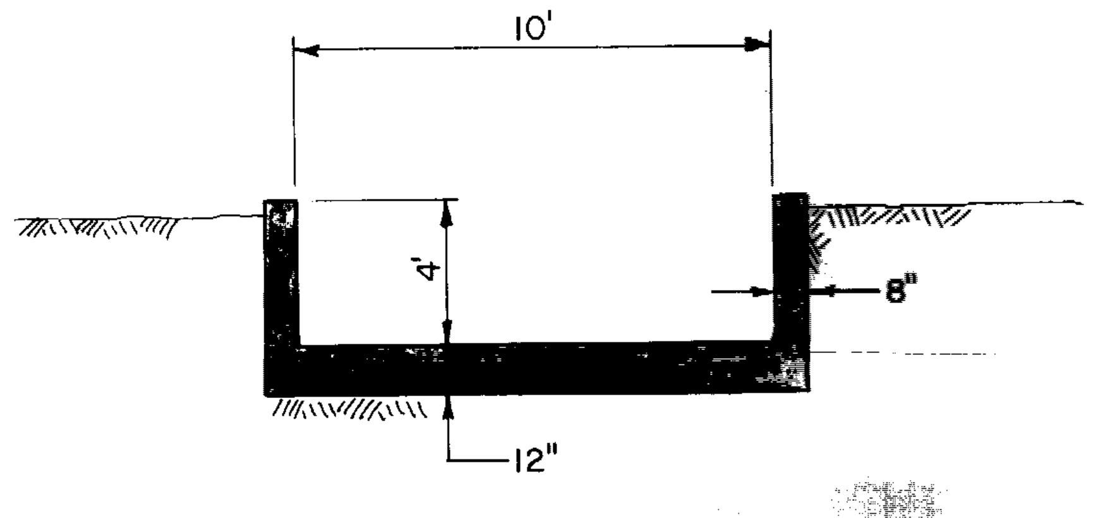
PLAN



DIVERSION DIKE DETAIL 1 2|2
NTS



REINFORCED CONCRETE CHANNEL DETAIL 2 2|2
NTS



REINFORCED CONCRETE CHANNEL DETAIL 3 2|2
NTS

KENNEDY/JENKS/CHILTON

WASHOE COUNTY REGIONAL FLOOD CONTROL
MASTER PLAN

THOMAS CREEK DETENTION BASIN STUDY

PARTIAL CHANNEL IMPROVEMENT ALTERNATIVE

JOB NO. 897043.02 MAY 1990 FIGURE 5-2

Channelization from the outlet of the detention basin to Virginia Street would provide the conveyance system normally required by FEMA, therefore, the alluvial fan designation could be removed. If alternatives to channelization are desirable, the design concept, analysis of downstream impacts and recommended methods of flood plain analysis could be presented to FEMA as a Conditional Letter of Map Revision (CLOMR) prior to final design. The CLOMR process was designed to allow a community or private party to obtain approval from FEMA for a design concept prior to final design or construction.

CREEK CHANNELIZATION ALTERNATIVES

Based on the goals established in the preceding section of this report, Kennedy/Jenks/Chilton studied several alternative means of conveying Thomas Creek 100-year flood flows from the proposed detention/debris basins to the future South Virginia Street conveyance structure. Presented herein are two alternatives, one providing for full channelization and conveyance from the proposed basin(s) (see Figure 5-1) and the other providing for partial channelization and diversion adjacent to the Holcomb Lane/South Virginia Street area (see Figure 5-2). Figures 5-1 and 5-2 are located in map pockets at the end of this Chapter.

The full channel improvement alternative basically provided for an engineered conveyance structure starting from the discharge point out of the detention/debris basin(s), running along the existing creek channel to the location of the new South Virginia Street culvert planned for constructed by the Nevada Department of Transportation as part of the roadway widening project. This alternative assumed that the new channel improvements would be constructed within the existing creek bed and banks and that all existing irrigation diversions would be maintained. In the area below the split of the Thomas Creek flows (near the Last Chance Ditch), the existing channel alignment would be utilized up to the residential properties to the east, but would be realigned to the north around these residences and re-joined to the existing channel west of the Sierra Manor Drive culvert crossing.

Kennedy/Jenks/Chilton based the analysis of the full improvement alternative on a reinforced concrete open channel conveyance structure. Other conveyance systems are possible and should be explored. However, it was concluded that the concrete channel alternative was a good choice to set the magnitude of costs for improvements on Thomas Creek.

The existing Thomas Creek channel slopes vary as follows:

Upper Reach from Detention/Debris Sites to Steamboat Ditch	6-7%
Middle Reach from Steamboat Ditch to Last Chance Ditch	3-5%
Lower Reach from Last Chance Ditch to South Virginia Street	1-3%

Kennedy/Jenks/Chilton assumed the discharge, ("Q") carried by the fully improved channel to be 400 cfs. This equates to the routed reservoir flows combined with the downstream tributary runoff. At 400 cfs and the above slopes, the velocities become too high for unprotected channels and somewhat questionable for rock riprapped channels. As an example, assuming an unlined trapezoidal channel at a slope of 3%, with bottom width of between 6 feet to 20 feet wide, 2h:1v sideslopes and a Mannings "n" of 0.025, the velocities vary between 9.1 feet per second (fps) at the 20 foot width to 11.6 fps at the 6 foot width. Therefore, because of the velocity considerations, the alternative analysis was confined to looking at reinforced concrete channels only. Other alternatives include a channel system with multiple check structures, rip-rap lined channel and a gabion lined channel.

A trapezoidal channel was selected for costing purposes. The same channel dimensional and geometric configuration was used for all improvement reaches, with the exception of immediately adjacent to South Virginia Street. In this area, the channel was analyzed assuming a rectangular section. The selected trapezoidal channel dimensions and geometry were as follows:

Bottom Width	4 feet
Sideslopes	2 horizontal to 1 vertical
Depth	4 feet

The rectangular channel section studied for conveying flows immediately adjacent to the future South Virginia Street culvert had the following dimensions:

Bottom Width	10 feet
Depth	4 feet

The 4 foot depth provides for approximately 1.2 feet of freeboard in the trapezoidal channel at the flatter slopes and approximately 1.2 feet in the rectangular channel section.

For cost estimating purposes, the same full improvement channel length and discharge value were assumed from both detention/debris dam sites studied.

As mentioned above, the design "Q" was 400 cfs. The channel length from the detention basin sites to South Virginia Street is approximately 14,000 lineal feet. It was also assumed that land would not have to be acquired where channel improvement was constructed within the existing Thomas Creek section. Furthermore, it was assumed that construction easements would be at no cost because of the benefit to the affected landowners. However, in the area requiring re-routing, including the run from adjacent to the Sierra Manor Drive culvert crossing to South Virginia Street, land acquisition costs were determined. The flows being carried through this section are greater than under existing conditions and the benefits of channelization to the adjacent landowners are questionable. While the landowners may grant easements for these improvements, the conservative approach dictates that the costing be based on the purchase of the required property or easements. Land valuations were determined in the same manner as detailed in Chapter 4 of this report for the detention/debris basin sites. The assessor parcel numbers of the affected lands assumed to require purchase are shown on Figure 5-2.

Table 5-2 below lists Kennedy/Jenks/Chilton's estimated project costs for the Full-Improvement Channelization of Thomas Creek from the detention/debris basin sites to South Virginia Street. These estimates are based on 1990 construction dollars and do not include facility operation and maintenance costs or engineering/project management costs for the design and construction of the facilities.

TABLE 5-2
Full-Improvement Channelization Alternative
Cost Estimate

	Description	Unit	Quantity	Unit Price	Total Cost
1.	Trapezoidal Channel Section				
	Excavation & Grading	c.y.	25,000	$ 5.00	$ 125,000
	Concrete	c.y.	5,125	$ 300.00	$1,537,500
	Bedding & Backfill	c.y.	5,125	$ 15.00	$ 76,875
2.	Rectangular Channel Section				
	Excavation & Grading	c.y.	4,200	$ 5.00	$ 21,000
	Concrete	c.y.	855	$ 300.00	$ 256,500
	Bedding & Backfill	c.y.	1,350	$ 15.00	$ 20,250
3.	Land Acquisition	acres	3.75	$58,666.67	$ 220,000
				TOTAL COST	$2,257,125
				USE	$2,300,000

The partial channel improvement and diversion alternative encompassed the study of a conveyance system constructed in the lower Thomas Creek reach in the vicinity of the Holcomb Lane/South Virginia Street area. The purpose of this alternative was to pick up the overbank flows below where Thomas Creek splits downstream of the Last Chance irrigation ditch and to convey the runoff to the South Virginia Street culvert. As developed in previous sections of this report, flood flows in this reach of Thomas Creek tend to spread out below the split and inundate large areas along South Virginia Street between Foothill Road to the south and the interchange of Interstate 580 and South Virginia Street to the north. The flooding potential to South Virginia Street north of Holcomb Lane should be greatly reduced by the construction of this alternative.

This alternative assumed that the natural Thomas Creek channel remained largely unimproved downstream of the proposed detention/debris basins. It was also assumed that the natural carrying capacity of Thomas Creek was not sufficient to confine the flood flows and that overbank conditions would exist during extreme rain events.

The diversion would be provided by low earthen dikes constructed across the Thomas Creek drainage path in the Dixon Lane area south of Holcomb Lane. The existing Lake Ditch irrigation ditch runs essentially perpendicular to the Thomas Creek drainage path in this area. Actually, the Lake Ditch crosses both forks of Thomas Creek after it splits below the Last Chance Ditch. The Lake Ditch crosses the east fork of Thomas Creek adjacent to the culvert structure on Sierra Manor Drive. The earthen dikes would be constructed alongside the Lake Ditch, parallel to the ditch alignment to the area of the east fork of Thomas Creek. Dikes across this area will intercept most of the overland runoff and effectively divert the flood flows towards South Virginia Street and away from Holcomb Lane.

At the point where the Lake Ditch crosses the east fork of Thomas Creek, an improved drainage channel would be constructed to convey the diverted flood flows to South Virginia Street and the future culvert structure. The improved channel would follow the existing alignment of the east fork of Thomas Creek to South Virginia Street. It was assumed that the existing east fork of Thomas Creek in this vicinity does not have sufficient capacity to carry the flood flows. Except for the inlet, the improved channel for this reach was assumed to be similar to the full improvement channel discussed above, that is, constructed of reinforced concrete with the same dimensions and geometry.

The discharge "Q" was also assumed to be the same. A special inlet design would be required to force the overland flow into the trapezoidal channel section.

Land acquisition costs were determined for the area required to construct the earthen dikes and for the construction of the improved channel. These costs were derived in the same manner as detailed in Chapter 4. The assessor parcel numbers are indicated on Figure 5-2.

Table 5-3 below lists Kennedy/Jenks/Chilton's estimated project costs for the Partial Improvement Channelization and Diversion of Thomas Creek in the vicinity of Holcomb Lane and South Virginia Street. These estimates are based on 1990 construction dollars and do not include facility operation and maintenance costs or engineering/project management costs for the design and construction of the facilities.

TABLE 5-3
Partial-Improvement Channelization and Diversion Alternative Cost Estimate

	Description	Unit	Quantity	Unit Price	Total Cost
1.	Rectangular Channel Section				
	Excavation & Grading	c.y.	4,200	$ 5.00	$ 21,000
	Concrete	c.y.	855	$ 300.00	$ 256,500
	Bedding	c.y.	1,350	$ 15.00	$ 20,250
2.	Diversion Dikes				
	Import Embankment Material	c.y.	2,420	$ 8.00	$ 19,360
	Aggregate Base	c.y.	440	$ 15.00	$ 6,660
3.	Land Acquisition	acres	3.50	$60,000.00	$ 210,000
				TOTAL COST	$ 533,770
				USE	$ 550,000

CHAPTER 6

CONCLUSIONS

Based on the results of this study, Kennedy/Jenks/Chilton concludes the following:

- Construction of a detention/debris facility on Thomas Creek in the vicinity of the two sites studied will dramatically attenuate the 100-year peak runoff flows reaching South Virginia Street.

- Detention sites downstream of the alluvial fan apex are not economically feasible. Detention sites upstream of the two middle sites studied are too high in the watershed to effectively reduce the routed 100-year peak flows reaching South Virginia Street.

- Construction of the Full-Improvement Channelization Alternative in conjunction with a detention/debris basin would serve to remove the FEMA alluvial fan designation on Thomas Creek. The alluvial fan designation may be removable without channelization based upon the reduced flow from the detention basin.

- Construction of the Partial-Improvement Channelization and Diversion Alternative effectively diverts and conveys the attenuated 100-year flood flows at South Virginia Street and will reduce the flooding potential north of Holcomb Lane along South Virginia Street.

- Preliminary estimates of construction costs range from $3.1 to $5.6 million for the On-Stream Facility, depending on the spillway alternative selected and approximately $3.35 million for the Off-Stream facility.

- Preliminary estimates of construction costs for the Full-Improvement Channelization Alternative is $2.3 million and $550,000 for the Partial-Improvement Channelization Alternative.

APPENDIX A - BIBLIOGRAPHY

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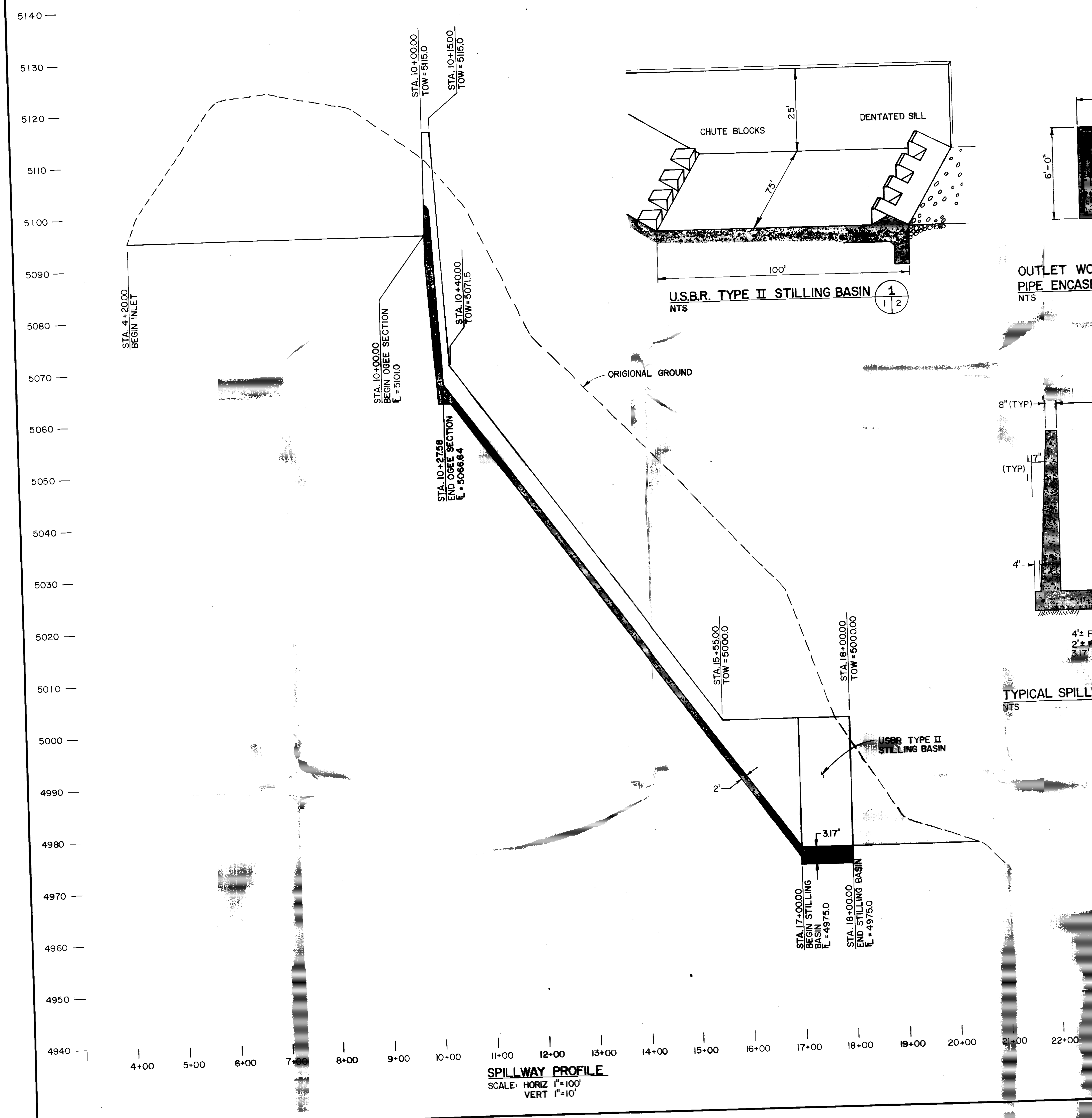
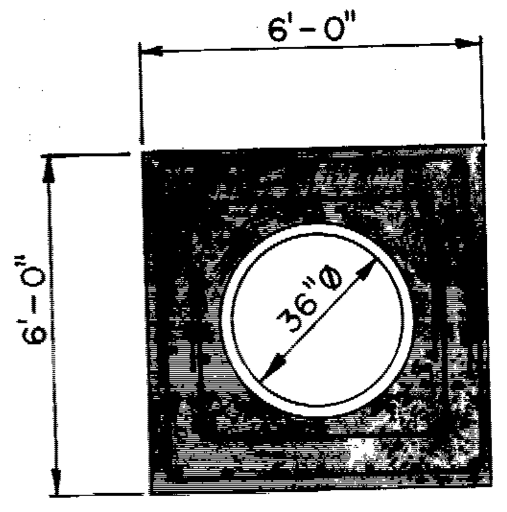
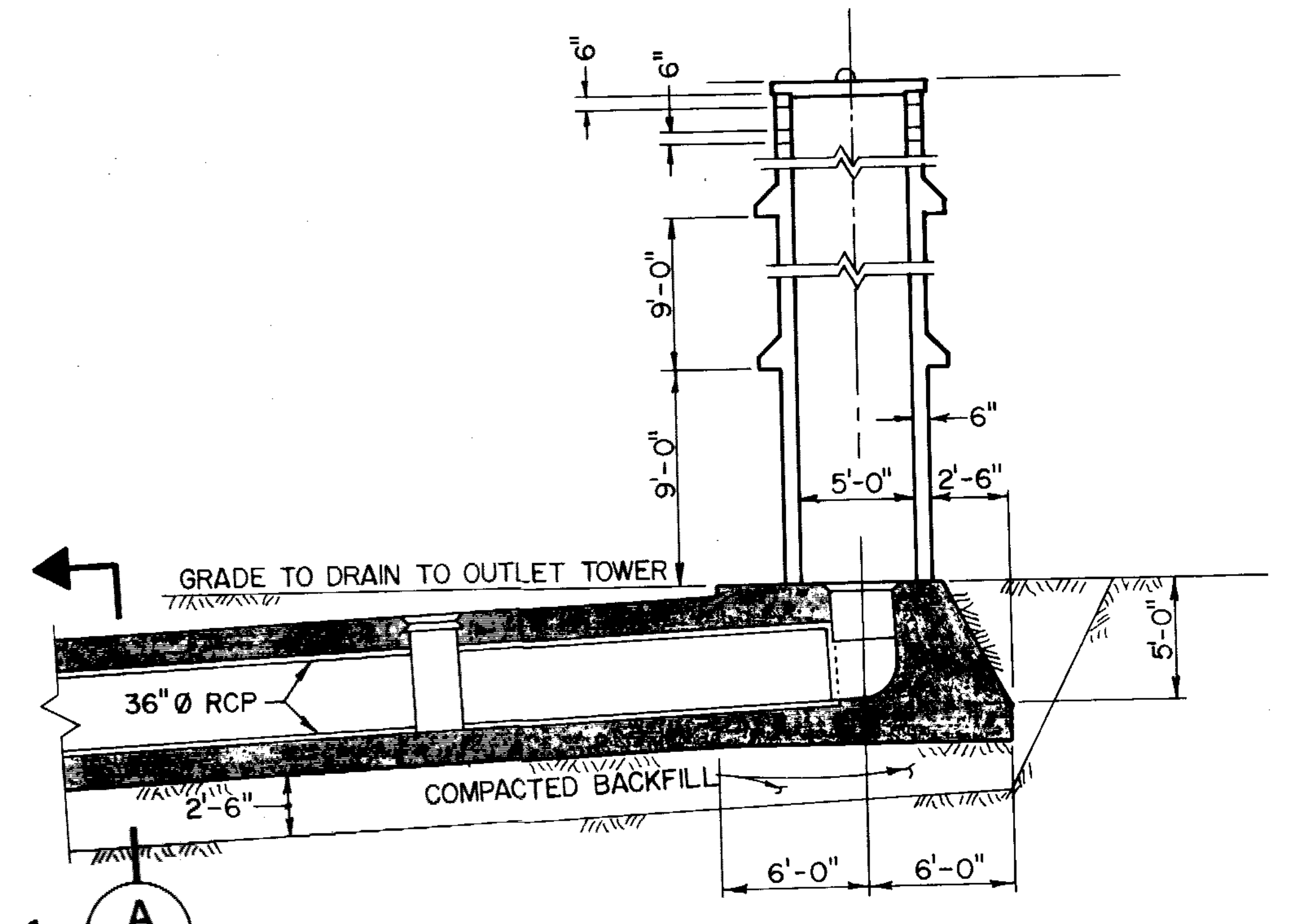


U.S.B.R. TYPE II STILLING BASIN ① 1|2
NTS

SPILLWAY PROFILE
SCALE: HORIZ 1"=100'
VERT 1"=10'



OUTLET WORKS PIPE ENCASEMENT SECTION (A 2|2)
NTS



OUTLET WORKS INTAKE SECTION 4 (B 1|2)
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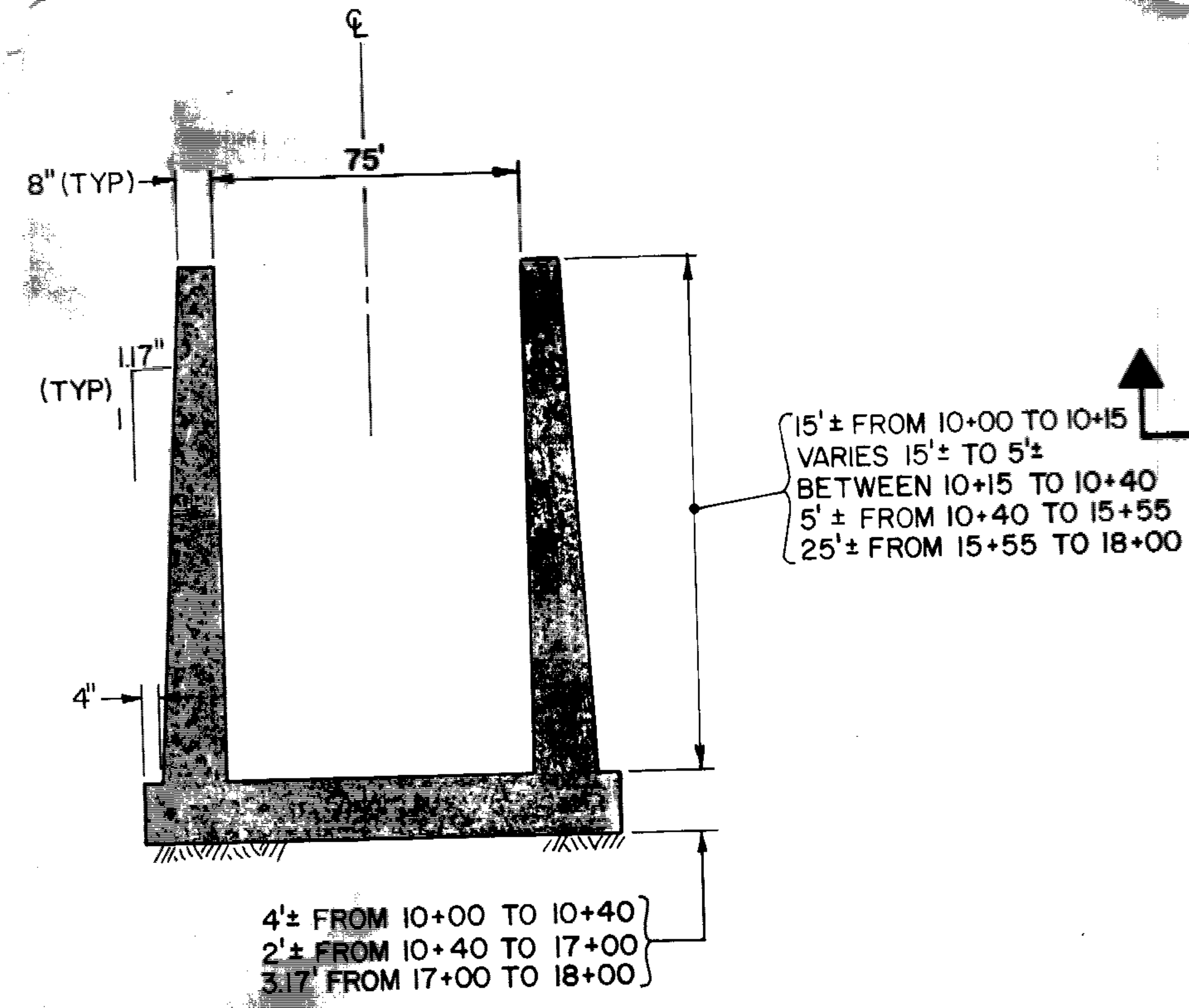

TYPICAL SPILLWAY SECTION (D 1|2)
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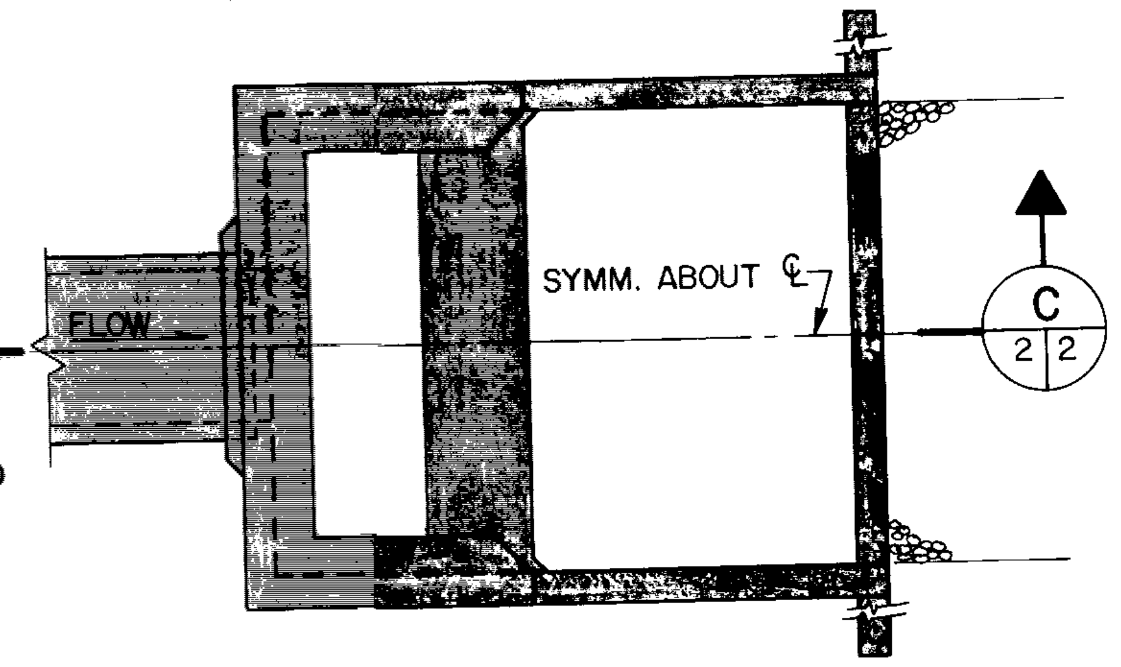

IMPACT STILLING BASIN 4,3 (2 1|2)
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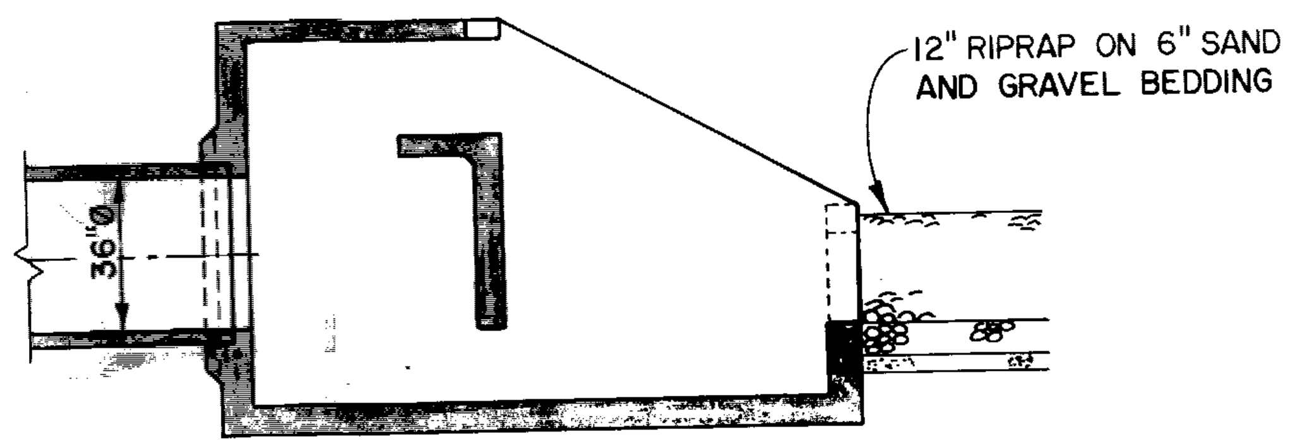

SECTION (C 2|2)
NTS

KENNEDY/JENKS/CHILTON

WASHOE COUNTY REGIONAL FLOOD CONTROL MASTER PLAN

THOMAS CREEK DETENTION BASIN STUDY

ON-STREAM SPILLWAY ALTERNATIVE "A" DETAILS

JOB NO. 897043.02 MAY 1990 FIGURE 4-2