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ESTIMATES OF THE RISKS ASSOCIATED WITH DAM FAILURE

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PREFACE

This report is one of a series of four prepared by UCLA (and a subcontractor, JRB Associates, where Prof. R.C. Erdmann of UCLA was spending a leave of absence) and issued as UCLA Engineering Reports at the request of and with the support of the U.S. Atomic Energy Commission (under Contract No. AT(04-3)-34 P.A. 205 Mod. 2).

The titles and numbers of the four reports are as follows:

1. "Estimates of the Risks Associated with Dam Failure," by P. Ayyaswamy, B. Hauss, T. Hsieh, A. Moscati, T.E. Hicks and D. Okrent, UCLA-ENG-7423.
2. "Airplane Crash Risk to Ground Population," by K.A. Solomon, T.E. Hicks and D. Okrent, UCLA-ENG-7424.
3. "The Risk of Catastrophic Spills of Toxic Chemicals," by J.A. Simmons, R.C. Erdmann and B.N. Noft, UCLA-ENG-7425.
4. "Estimate of the Hazards to a Nuclear Reactor from the Random Impact of Meteorites," by K.A. Solomon, R.C. Erdmann, T.E. Hicks and D. Okrent, UCLA-ENG-7426.

These reports represent an effort to evaluate the probabilities and consequences associated with some unlikely, potential accidents, most of which a certain degree of intuition regarding risk exists. These estimates represent preliminary, early results that need to be continued, expanded and refined over the next few years so as to better understand technological risks. The mortality and property damage estimates are made to provide some perspective on the current risks of modern technologies. For this purpose, preliminary estimates of probabilities and consequences can be of value, even if there exists considerable uncertainty in the results.

1. 2. 3.

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1. SUMMARY

The probabilities and potential consequences of dam failure in California, primarily due to large earthquakes, was estimated, taking as examples eleven dams having a relatively large population downstream.

Mortalities in the event of dam failure range from 11,000 to 260,000, while damage to property may be as high as \$720 million. It was assumed that an intensity IX or X earthquake (on the Modified Mercalli Scale) would be sufficient to completely fail earthen dams. Predictions of dam failure were based on the recurrence times of such earthquakes. For the dams studied, the recurrence intervals for an intensity IX earthquake varied between 20 and 800 years; for an intensity X between 50 and 30,000 years. For the Lake Chabot and San Pablo dams (respectively 20, 30 year recurrent earthquake times for an intensity X) the associated consequences are: 34,000 (Lake Chabot) and 30,000 (San Pablo) people killed; damage \$140 million and \$77 million. Evacuation was found to ameliorate the consequences slightly in most cases because of the short time available. Calculations are based on demography, and assume 10 foot floodwaters will drown all in their path and destroy all one-unit homes in the flood area. Damage estimates reflect losses incurred by structural damage to buildings and do not include loss of income. Hence the economic impact is probably understated.

2. INTRODUCTION

Large man-made structures, such as dams and bridges, impress most people with a sense of "permanence". Any dangers associated with such structures therefore tend to be psychologically minimized. However, the failure or demolition of a high dam (for example) that impounds large volumes of water may create major flooding conditions, causing severe damage to life and property downstream.

The extent of flooding and resulting damage depend on many factors. One must consider the size and shape of the dam, the portion of the dam breached, the volume of water impounded before the break, the height, width and length of the reservoir behind the dam, and conditions relating to the flow of water in and out of the dam. These factors determine the time it would take (in the event of dam failure) for all or most of the impounded water to be released. Following dam failure, the extent of flooding will depend on the topography of the land downstream of the dam. Damage to property and life will depend on zoning conditions (residential versus commercial, single unit versus high rise structure, etc.), time of occurrence of the flooding, and demography in the flooded area.

This section investigates the probability of failure of eleven earthen and concrete dams in California, and the extent of consequent downstream flooding and damage to life and property. Only one failure mode has been considered: complete and instantaneous dam failure, with total release of the impounded water. It has been assumed that such failure would result in the most severe possible consequences. Sites in California have been chosen because of the great seismic activity of the state. It has been assumed in this study that a severe earthquake (IX or X on the Modified Mercalli earthquake intensity scale) would cause the sudden and complete failure of an

earthen dam in the zone of most intense shock. Hence, dam failure is equated with the probability of an intensity IX or X earthquake in the dam area.

Earthquake probabilities are linked to the movement of geologic faults. The locations of major faults in California have been accurately mapped, and their movements have been recorded for several decades. Theories have been developed to account for stress relief via fault movement, and the theories have been tested against empirical data and historical records to develop predictive methods for earthquake analysis. In this study, a computer model has been developed to predict earthquake probabilities at various dam sites in California. This model follows closely a program developed for planning water resource systems in California.

A number of problems arise in deriving estimates on earthquake probability, dam failure and associated consequences. Earthquake prediction is a young art. Historical records of earthquakes are a meaningful data base only if large areas are covered and the time frame is adequately long. The current state of the art in earthquake prediction involves large uncertainties.¹

In addition, although sophisticated mathematical techniques can be applied to the mechanics of dam failure, it is not possible to predict with certainty what magnitude earthquake will lead to failure of a particular dam. The best available estimates are the opinions of qualified engineers and dam experts. In a previous study, for example,² only two out of 18 dams (world wide) studied actually completely failed due to earthquakes (both MM VIII). Dam construction techniques have changed through the years, so dam failure rates may be highly variable and no single set of conditions may apply to all of them.

Abundant instrumental data on earthquakes have only been available in the 20th century. This relatively short record of seismicity in the U.S.

may be too short to allow meaningful predictions of secular seismicity. In this study it was found that the intensity scale predicted by the computer should be lowered about one order of magnitude to conform to historical records. The uncertainty involves inadequate knowledge of the soil conditions in the quake-stricken area.

It is therefore clear that the predictions derived in this study can only be considered a first approximation to the true values. For some of the dam sites earthquakes are forecast to be more frequent than historical records support. Hence we must conclude that although the computer model used in this study is a convenient and useful tool for providing rough estimates of earthquake probabilities at a given site, it is only one of many possible models. The present model must be further refined to more closely reflect the data that do exist on earthquakes.

Each year more sophisticated techniques are developed for evaluating earthquake causes, and for predicting future seismicity. Additions to historical data provide a steadily more reliable data base for evaluating earth movements in California. Additional study is required to produce a reliable computer model for earthquake prediction, and better techniques are needed for evaluating the behavior of dams during seismic events. Such studies will reduce the uncertainties associated with current estimates of dam failure, mortality and property consequences.

Appendices to this report contain details of some of the mathematical methods used to calculate the movement of flood waters in a channel. For the dam sites selected, simplifying assumptions have been made regarding the shape of the channel below the dam (these assumptions are listed) and the flow characteristics in the channel. Each dam has been analyzed independently. A separate appendix for each dam contains details of the analysis, graphs, diagrams, etc.

The techniques for calculating flow characteristics in the channels downstream from the dams studied are fairly reliable, although some simplifying assumptions have been made. It is worth noting that the calculations of consequences of dam failure have been compared in one case to the actual observations of flood waters resulting from dam collapse. The St. Francis dam failed in 1928. A wall of water 125 feet high was reported traveling down the canyon beneath the dam. The water wall was 78 feet high 10 miles downstream from the dam. Calculations of the water height at the dam and at 7 miles downstream, using the methodology developed in this study, gave values of 130 feet and 85 feet, respectively. This agreement with observations is considered excellent, and provides support for the calculation technique used.

The consequences of dam failure, in terms of mortalities and property damage, are therefore forecast with an implicit error band. These calculated consequences are predicted on the assumptions made regarding earthquake frequency and intensity, and dam behavior. The conclusions should therefore be regarded as mainly illustrative and very tentative.

3. METHODOLOGY

The analysis for each dam consists of an evaluation of the amount of water that leaves the dam in the event of complete dam failure. This leads to an evaluation of the dimensions of the flood wave, and finally an assessment of the flow of water in the channel below the dam. The water height can then be estimated at various distances downstream, and thus the extent of flooding and damage to life and property can be calculated.

The analysis initially determines the amount of water impounded by a given dam at any given time. This has been done using simple geometric models (see A-1) because graphs for water storage levels versus dam height were not available for all the dams studied. Cross checks of the models were made against the few "storage versus height" graphs that were available. The models are considered accurate to within 5%, which is sufficiently accurate for the purposes of this report.

Assuming that the dam fails instantaneously and completely, the rate of water volume released is described in two ways, termed $Q_{\max}$ and Q_{ave} (see A-1). Flow of water in the channel below the dam is evaluated through the use of a hydrological equation known as Manning's Formula (see A-2). The calculation leads to an estimate of the height of water traveling in the channel at the dam site and at various distances downstream. In some cases, such as the St. Francis Dam, the calculated values agree well with observations of actual results of failure of the dam.

The shape and actual dimensions of the channel depend on the topography in the vicinity of a given dam. Regardless of the slope of the channel, size of channel or depth of flow, a constant friction coefficient n (Manning's n) is used in the calculation of the Q_{av} through the use of the Manning's equation. The value for n itself is determined in each case from a knowledge of

the downstream flow area. A range of n (0.05 - 0.11) suitable for particular flow paths are used. This approach is called the Normal Depth Method. There are alternative methods available, but it is felt that the procedures used herein lead to realistic results for the failure mode assumed in the analysis.

Certain special cases that have been considered and evaluated include:

- (a) The presence of obstructions (bumps) in the flow channel. A calculation was made to assess the ability of the water to flow over the bump, thus checking for a subcritical flow.
- (b) The failure of an upstream dam, leading to the failure of a dam further downstream along the flow channel. In these cases, Weir equations governing flows over rectangular, broad-crested dams are used in order to assess the extent of flooding.

Approximations have been used where necessary to determine the extent of flooding and flood damage on large flood plains.

It is recognized that the earthquake probabilities calculated in this study contain large associated errors, but it is believed that the results (to a first approximation) are indicative of the potential risks associated with California dams.

4. RESULTS AND DISCUSSION

The assumption of complete and instantaneous dam failure reflects the worst conditions that are theoretically conceivable. However, both experience and theory indicate that such failure, resulting from earthquake action, is a real possibility. With this in mind, estimates have been made of the probabilities of earthquakes in areas containing the California dams studied in this analysis. These dams are listed in Table 1, along with estimates of probabilities of earthquakes with Modified Mercalli intensities of VIII, IX and X for the dam environs. It is assumed[†] that an earthquake of MM intensity VIII results in a significant probability of dam failure; an earthquake of MM intensity IX results in a substantial probability⁶ and an earthquake of MM intensity X results in a high probability. The probabilities listed for earthquakes are those generated by computer analysis adjusted as in Section 1.5 (on computer estimates). The predictions for MM X have the greatest uncertainty, since little empirical basis exists.

Table 2 lists estimates of expected property damage and mortalities due to total and instantaneous failure of the dams investigated (assuming that the dams are filled to capacity when the failure occurs). Mortalities range from as low as 11,000 to as high as 260,000; property damage ranges from \$50 million to \$720 million.

Table 3 gives estimated flood heights at various reaches downstream of the dams investigated (a reach is a section of the dam channel which is a convenient hydraulic measure taken in order to analyze the downstream flooding. The length of the measure itself may change depending on the particular topography. For example, in the case of the Van Norman Dam three reaches of 12,000 feet each have been selected to investigate the downstream flooding.

[†]See Section 7 of this report for the rationale of this assumption.

The first reach is from dam site to a point 12,000 feet downstream; the second reach is 12,000 feet to 24,000 feet, etc.). Floodheights, for the cases investigated, range in the primary reach from 10 feet to 147 feet. The total flood length ranges between 10,000 feet (Lake Chabot Dam) and 40,000 feet (Stone Canyon Dam).

TABLE 1

The results of the investigation could be summarized as follows:

Prediction of Earthquake in California by Fault Theory, Using a Computer

Name of Dam	Estimated Probability of MM VIII per Year*	Estimated Probability of MM IX per Year**	Estimated Probability of MM X per Year***
St. Francis [†]			
Van Norman	0.014	0.0049	.00003
San Andreas	0.044	0.014	.003
Lower Crystal Springs [†]			
Stone Canyon	0.012	0.0014	.0003
Encino	0.012	0.0013	.0002
San Pablo	0.076	0.032	.011
Folsom [†]			
Shasta [†]			
Chatsworth	0.013	0.0028	.00003
Mulholland [†]			
Upper San Leandro	0.12	0.063	.023
Lake Chabot	0.12	0.057	.021

* Significant Probability of Failure

** Substantial Probability of Failure

*** High Probability of Failure

NOTE: Values for significant and substantial probabilities indicated in the above table correspond to the values at an order of magnitude higher as reflected in the computer output. Please see explanation in the text.

[†] These are Gravity type (concrete dams) for which the failure basis of Reference 2 is not applicable. Gast²² has provided an estimate of 10^{-4} per year for world-wide concrete dams due to all causes.

TABLE 2
Estimated Effects of Total and Instantaneous
Failure of Dam Filled to Capacity

Name of Dam	<u>MORTALITIES</u> [†]		DAMAGE ASSESSED IN U.S. DOLLARS
	Day	Night	
St Francis	- - - - Not calculated - - - -		
Van Norman	72,000	123,000	3×10^8
San Andreas	21,000	33,000	1.1×10^8
Lower Crystal Springs			
Stone Canyon	125,000	207,000	5.3×10^8
Encino	11,000	18,000	5×10^7
San Pablo	24,000	36,000	7.7×10^7
Folsom	260,000	260,000	6.7×10^8
Chatsworth	14,000	22,000	6×10^7
Mulholland	180,000	180,000	7.2×10^8
Upper San Leandro	36,000	55,000	1.5×10^8
Lake Chabot			
Shasta	34,000	34,000	1.4×10^8

[†]No allowance for evacuation. See Section 8 which indicates modest reductions for the two dams considered.

TABLE 3

Estimated Flood Heights at Various Reaches Downstream of the Dam

Name of Dam	Number of Reaches Used for Analysis	<u>Estimated Flood Height</u> <u>in Feet</u>							Total Length of Flood in Feet
		Reach Number							
		1	2	3	4	5	6	7	
St Francis	3	130	110	83					36,000
Van Norman	3	38	20	15					36,000
San Andreas	}	2	147	23					20,000
Lower Crystal Springs									
Stone Canyon	5	123	20	16	15	17			40,000
Encino	2	36	23						14,000
San Pablo	2	45	11						32,000
Folsom	3	103	100	85					
Chatsworth	3	10	8	6					18,000
Mulholland	7	122	37	26	20	17	16	21	28,000
Upper San Leandro	}	4 ^{(Area} A	56	35	27	22			20,000
Lake Chabot		2 ^{(Area} B	29	22					10,000

The remainder of the text and appendices provide detailed calculations of these estimates.

5. FEASIBILITY OF COMPLETE AND INSTANTANEOUS DAM FAILURE

The assumption of complete and instantaneous dam failure represents the worst conditions that are theoretically possible. Such conditions may include an actual movement of the dam on its foundation due to shear failure and/or piping in the foundation. A foundation failure could probably release the whole reservoir behind the dam in a very short time. It is felt that in such cases the assumptions made in this study are not far from reality.

It must be noted, however, that the possibility of a shear failure in a dam foundation that would cause the entire dam to move downstream is very low, unless caused by earthquake action. In the latter case, complete and rapid failure is a very real possibility. J. L. Sherard²³ explains the movement and consequent failure as caused by an earthquake as follows:

"The normal shear slide is caused by gravity forces, and the sliding mass simply slips downhill when the component of its own weight exceeds the shearing resistance which can be generated on the sliding surface. The earthquake slide, on the other hand, will be generated primarily by acceleration forces with major horizontal components, and the material can slide on a much flatter failure surface. Large differences in the mechanics of the shearing failure which might develop under the two actions are to be expected. Further, it should not be surprising that the slide most likely to occur during an earthquake involves a much larger mass of the dam than the slide most probable without an earthquake."

According to Sherard, failure by a shear slide of the dam on its foundation offers a more serious and statistically likely hazard than any other possibility of damage from earthquake action. It should also be realized that while the damage to the dam from earthquake action could be severe and drastic,

it is difficult to evaluate reliably. Sherard has, from simple analytical considerations, shown that the margin of safety against shearing movement of any dam on its foundation is greatly reduced by earthquake shaking. These calculations indicate that shearing stresses acting in the foundation during the earthquake may exceed the design strength many times.

With regard to the possibility of piping accounting for the catastrophic failure of earth dams, it should be noted that two interdependent types of detrimental action associated with the typical piping failure may be discerned. These are:

- (1) progressive backward erosion of a concentrated leak, and
- (2) progressive backward sloughing or "ravelling" of the saturated downstream slope.

Both are commonly combined. The former is usually dominant but in some cases the latter can apparently be a major factor and cause a more dangerous type of failure. Although the first mechanism above does not usually result in complete and instantaneous dam failure, the progressive sloughing could result in complete failure. A quantitative estimate of the probability of failure due to soil liquefaction is not currently available.

6. PROCEDURE FOR ESTIMATING THE PROBABLE NUMBER OF DEATHS AND THE AMOUNT OF DAMAGE DUE TO POSTULATED DAM FAILURES

6.1 Probable Number of Deaths

Given a rough flood plain below a dam an estimate of the probable number of deaths to be expected due to the flooding can be readily made. A direct and fairly accurate estimate of the downstream population density may be done using information published by the United States Bureau of the Census on the 19th Census of the Population (1970). This data may be used to approximate to within a few percent the entire resident population as well as the working portion of that population so that daytime/nighttime estimates of the expected number of deaths may be made.

Large metropolitan areas such as the Los Angeles 5-county area, the San Francisco Bay area, Sacramento, etc. are broken down by the Census Bureau into census tracts each containing on the average 4,000 people. The census tracts are designed to be fairly homogeneous with respect to population characteristics, economic status and living conditions. Usually the census tracts are each a few blocks on a side in a populous area.

Once the floodplain has been outlined on a census tract map, the population jeopardized by a possible dam failure may be determined merely by summing the population of the encompassed census tracts. Census tracts which fall only partially into the floodplain may be taken into account by referring to a topographic map of the area to determine what fraction of the tract's total population would be affected. Details down to individual streets and houses are shown on topographic maps available from the United States Geologic Survey (USGS) at the scale of 1:24,000. Characteristics of the population within each tract are taken from the "1970 Census of the Population and Housing - Census Tracts" for the metropolitan areas.

This approach to estimating the downstream population is approximate in that the census data reflects the population distribution of April 1st, 1970, Census Day. However, the estimates should be within a few percent of the present distribution, especially in the more developed areas which are examined.

In totaling the expected casualties from a postulated dam failure it is assumed that flooding to a depth of 10 feet is sufficient to destroy completely the typical one-unit home and drown all in the path. Any exceptions to this assumption are noted where they occur. A discussion of the potential ameliorating effects of evacuation is given in Section 8.

6.2 Amount of Damage

The "1970 Census of the Population and Housing - Census Tracts" also provides information on the number of single-family homes and their median value as well as other housing information. Thus in computing the damage to single-family homes the number of one-unit dwellings destroyed in each census tract is multiplied by the median value of homes in that tract. This yields an estimate reflecting land-plus-improvements values within each tract. Since the value of the land probably holds constant through a flood, except possibly for landscaping and regrading costs, the total damage for each tract is corrected by multiplying by some fraction intended to reflect the inherent value of the land. Bank and real-estate representatives suggested a ratio of house/land value of 40/60 as being typical of plots in established residential areas. The cost of repairing the defaced property through landscaping and regrading was estimated independently as about 10% of the total value of the property. Thus the fraction used to multiply the total damage in each tract was 0.5. Once the discounted damage estimates are obtained for each tract, they are summed over all damaged tracts. It is worth repeating that the

census tracts are designed to be homogeneous with respect to the value of housing among other things.

If commercial property represents a large portion of the flooded region, the dollar value of damage may be much higher.

7. COMPUTER PREDICTION OF EARTHQUAKES IN CALIFORNIA

BY FAULT THEORY

7.1 Introduction

In an attempt to estimate the seismic risk of an earth dam, one has first to estimate the earthquake intensities and their corresponding probabilities at the dam location. Secondly, one also has to decide what minimum intensity of earthquake leads to a complete failure of an average dam or any specific dam. For the latter task, one can apply some sophisticated methods such as finite element analysis to obtain the structural response of a dam, which presumably can predict the load at which the dam will completely fail. However, since the current state of the art of predicting earthquake probability involves a much greater degree of uncertainty,¹ it may not be well justified to do too many elaborate calculations for the second task. It seems to be a reasonable alternative that one can rely on experts and experienced engineers' opinions on the minimum intensity which will lead an average earth dam to a complete failure.

In a paper on "Foundations and Earth Structures in Earthquakes",² Duke indicates that the minimum intensity of shaking to produce damage to the older dams appears to be about VII on the Modified Mercalli scale (see Table 4). However, only two out of eighteen cases which Duke had considered were completely failed. The complete failures resulted in the emptying of the contents of the dam reservoirs (one at intensity VIII, another at IX). In all cases of damaged dams, the dams were constructed without the use of modern compacting control techniques. The minimum intensity for failure could probably be assumed to be somewhere about VIII or even greater.

Table 4

ROSSI-FOREL INTENSITY SCALE (1883)		MODIFIED-MERCALLI INTENSITY SCALE(1930), WOOD AND NAUMANN		GROUND ACCELERATION	
		I		a	
		I	Detected only by sensitive instruments.	$\frac{cm}{sec}$	$\frac{a}{g}$
I	The shock felt only by experienced observer under very favorable conditions.	II	Felt by a few persons at rest, especially on upper floors; delicate suspended objects may swing	-2	-
II	Felt by a few people at rest, recorded by several seismographs.	III	Felt noticeably indoors, but not always recognized as a quake; standing autos rock slightly, vibration like passing truck.	-3	-
III	Felt by several people at rest; strong enough for the duration or direction to be appreciable.			-4	.005g-
				-5	-
				-6	-
IV	Felt by several people in motion; disturbance of movable objects, cracking of floors.	IV	Felt indoors by many, outdoors by a few; at night some awaken; dishes, windows, doors disturbed; motor cars rock noticeably.	-7	-
				-8	.01g-
				-9	-
				-10	-
V	Felt generally by everyone; disturbances of furniture, ringing of some bells.	V	Felt by most people; some breakage of dishes, windows, and plaster; disturbance of tall objects.	-20	-
				-30	-
VI	General awakening of those asleep, ringing of bells, swinging of chandeliers, startled people run outdoors.	VI	Felt by all; many frightened and run outdoors, falling plaster and chimneys; damage small.	-40	.05g-
				-50	-
				-60	-
				-70	-
VII	Overthrow of movable objects, fall of plaster, ringing of bells, panic with great damage to buildings.	VII	Everybody runs outdoors, damage to buildings varies, depending on quality of construction; noticed by drivers of autos.	-80	-
				-90	0.1g-
				-100	-
VIII	Fall of chimneys; cracks in walls of buildings.	VIII	Panel walls thrown out of frames; fall of walls, monuments, chimneys; sand and mud ejected; drivers of autos disturbed.	-200	-
IX	Partial or total destruction of some buildings.			-300	-
X	Great disaster; ruins; disturbance of strata, fissures, rockfalls, landslides, etc.	IX	Buildings shifted off foundations, cracked, thrown out of plumb; ground cracked, underground pipes broken.	-400	.5g-
				-500	-
				-600	-
				-700	-
		X	Most masonry and frame structures destroyed; ground cracked; rails bent, landslides.	-800	-
				-900	1g-
				-1000	-
		XI	New structures remain standing; bridges destroyed; fissures in ground; pipes broken; landslides; rails bent.	-2000	-
				-3000	-
		XII	Damage total; waves seen on ground surface; lines of sight and level distorted; objects thrown up into air.	-4000	5g-
				-5000	-
				-6000	-

To estimate the earthquake probability, two approaches are available. One approach is to assume that all earthquakes occur randomly with equal probability in a certain large area.^{1,3} Historical records of earthquakes that happened in the area are used to describe the mean annual frequency distribution, from which future earthquake magnitude and probability is extrapolated. This approach has one major disadvantage, namely, it is meaningful only if the area covered is sufficiently large and is representatively characteristic in the distribution for the area. We will not consider this approach further simply because of the intrinsic disadvantage and the reasons behind the second approach. The second approach, which can be termed the fault theory, assumes that earthquakes originate from known active faults. This is plausible in an area like California where active faults are scattered throughout the entire state.

Except for smaller shocks which are randomly distributed, it is a general belief that most large earthquakes occur in regions where active faults are recognized.⁴ In a paper on relations between seismicity and geologic structure in the Southern California area,⁵ Allen et. al. pointed out that most large historic earthquakes with magnitude down to 6.0 can be associated with surficial fault displacements and that not all intensely faulted areas have been active at all times--the seismic quiescence may be only temporary and a future large earthquake is yet possible. Numerous investigators have tried to correlate the earthquake magnitude with fault length and/or displacement.^{4,6,7,8} Although data points do scatter to some extent, some general relations (such as, the greater the displacement on the main fault at the ground surface, the greater the magnitude of the associated earthquake, etc.) seem to exist.^{4,6} It also has been observed and reported that the earthquake shaking effects generally are greater in a moderately wide zone along the

fault (this is because earthquake waves usually originate at some miles beneath the surface) and decrease away from the fault.⁴

From these observations, assessment of the seismic risk based on the fault theory seems to be more reasonable than assessment based on the random distribution assumption.

Duke and Jacobsen⁹ had applied the fault theory to the problem of incorporating the hazard of earthquakes into the optimization of the planning of water resource systems in California. A computer program whose primary algorithm follows their methodology closely was developed for the purpose of predicting earthquake probability at any dam location in California. In the code, any location within California state expressed in longitudinal and latitudinal coordinates (in degrees) will specify the problem. Probabilities per unit time and expectations of recurrent times versus intensities are the outputs. The program list is provided at the last section. The computing time for a 15-case problem is only about 4 seconds on the IBM 360/91 machine. The outputs are shown immediately after the program list.

The following section describes briefly the methodology used in the code. For detailed descriptions, the referenced document⁹ should be consulted.

7.2 Methodology of a Computer Program Based on the Fault Theory

Nineteen major faults (or more precisely, 19 major fault segments) in California are considered in the computer program. It is assumed that each fault can be divided into any convenient number of segments with the same characteristics of the fault. It is also assumed that the occurrence distribution of earthquakes on a given fault segment can be represented by the Poisson probability distribution.¹⁰ That is,

$$P_{ij}(n, \tau) = (\mu_{ij}\tau)^n e^{-\mu_{ij}\tau} / n! \quad (1)$$

where i denotes the i th fault ($i \in \{F\}$, $F = 1, 2, \dots, 19$ in the code); j denotes the j th segments of the fault ($j \in \{SG\}$; $SG = 1, 2, 3, \dots, 10$ for each fault in

the code); n denotes the number of occurrence of earthquakes within τ units of time; μ is the occurrence rate; τ is the interval of time. By excluding the zeroth event, the probability of earthquake occurrence for the j^{th} segment on the i^{th} fault is simply

$$P_{ij} [T \leq \tau] = 1 - e^{-\mu_{ij}\tau} \quad (2)$$

where T denotes the time to the first earthquake.

The expectation of recurrent time is

$$E[\tau] = \int_0^{\infty} \tau \mu_{ij} e^{-\mu_{ij}\tau} d\tau = \frac{1}{\mu_{ij}} \quad (3)$$

μ_{ij} is given by

$$\mu_{ij} = \left(\frac{\Delta L_{ij}}{L_i} \right) \mu_i \quad (4)$$

where L_i denotes the total length of the i^{th} fault; ΔL_{ij} denotes the j^{th} incremental segment on the i^{th} fault; μ_i is a characteristic of the i^{th} fault. The assumption made here is that the occurrence of earthquakes at a fault is uniformly distributed among the segments. For a given fault, the characteristic μ_i is to be determined from the seismic record for a given period of time and a given earthquake magnitude range. The activity of the i^{th} fault can be expressed in terms of the a_i and b_i coefficients of Richter's equation,¹¹

$$\log_{10} N_i = a_i - b_i M \quad (5)$$

where N_i is the observed number of earthquakes of magnitude greater than or equal to M per unit time; a_i , b_i are obtained through data-curve fitting and are given in Table 1 (p. 58) of the reference.⁹ Here we use μ_i instead of N_i , hence μ_i is given by

$$\mu_i = N_i = 10^{a_i - b_i M_o} \quad (6)$$

where we have introduced the subscript o to denote the lowest limit of magnitude associated with the i^{th} fault.

Given a magnitude M occurring at the j^{th} segment on the i^{th} fault, the Modified Mercalli intensity s at a location is¹²

$$s = C_1 M - C_2 \log_{10} R_{ij} + C_3 \quad (7)$$

where R_{ij} is the hypocentral distance to the j^{th} segment on the i^{th} fault; C_1 , C_2 , and C_3 are established for a region from the seismic history.

Let $G_{ij}(s)$ denote the conditional distribution at a location within the California state, given that an earthquake has occurred at the j^{th} segment on the i^{th} fault, then

$$\begin{aligned} G_{ij}(s) &= P(S_{ij} \leq s) \\ &= P(C_1 M_{ij} - C_2 \log_{10} R_{ij} + C_3 \leq s) \\ &= P\left(M_{ij} \leq \frac{s + C_2 \log_{10} R_{ij} - C_3}{C_1}\right) \\ &= F_{ij}\left(\frac{s + C_2 \log_{10} R_{ij} - C_3}{C_1}\right) \\ &\equiv F_{ij}(m) \end{aligned} \quad (8)$$

where the magnitude probability density function $F_{ij}(m)$ is most widely represented by¹³

$$F_{ij}(m) = P(M_{ij} \leq m) = \begin{cases} 1 - e^{-\beta_{ij}(m-m_0)} & , \quad m \geq m_0 \\ 0 & , \quad \text{otherwise} \end{cases} \quad (9)$$

where β_{ij} is to be determined by¹⁴

$$\beta_{ij} = b_i \ln 10, \quad j = 1, 2, \dots, SG \quad (10)$$

Let $G(s)$ denote the conditional distribution at that particular location, given that earthquakes have occurred at some segments on some faults in California, then

$$G(s) = \frac{\sum_i^f \sum_j^{sg} \mu_{ij} G_{ij}(s)}{\mu}, \quad (f \in F; sg \in SG) \quad (11)$$

where

$$\mu = \sum_i^f \sum_j^{sg} \mu_{ij} \quad , \quad (f \in F; sg \in SG) \quad (12)$$

with the summations extending to those segments and faults which are effective at that particular location. The effectiveness of each segment of any fault is to be determined by equation (7).

The probability of earthquake at any location in California with intensity greater than or equal to s up to time t is

$$P(S \geq s, t) = 1 - e^{-\mu t(1 - G(s))} \quad (13)$$

As analogous to equations (2) and (3), the expectation of recurrent time is

$$E[T; s] = \frac{1}{\mu(1 - G(s))} \quad (14)$$

7.3 Discussions on Some of the Code Outputs

Although the first recorded earthquake in the United States was as early as 1534,¹ abundant instrumental data are only available after 1905.^{5,9} Thus the relatively short historical record of seismicity in the U.S. may not encompass a long enough time period to represent the true secular seismicity. Predictions based on data obtained under such conditions can only provide a relative earthquake risk guideline.

The conditions of the soil which overlays the bedrock at the site play an important role in the severity of shaking on the ground surface.⁹ Soft soil sites usually shake more severely than rock sites. This was demonstrated in the 1906 San Francisco earthquake. Structural damage was consistently greater for buildings erected on filled land, whereas on the higher bedrock hills, damage was much less, even in areas closer to the earthquake center.¹⁵ In general, damage is greater on soft, weak ground, but there are some evidences of an inverse relationship in the case of rigid structures due to soil dynamic

amplification effects.¹⁶ In the computer code, it assumes firm soil throughout California for simplification. The firm soil corresponds approximately with fine to medium grained sand in San Francisco area and with marine terrace deposits in the Los Angeles region.⁹ For an extra-soft soil site, such as artificially filled sites, the probabilities against intensities probably should be modified upward one unit on the Modified Mercalli scale (for example, the probability of intensity X of an extra-soft site may actually correspond to the probability of intensity XI). On the other hand, for extra firm soil such as igneous, metaigneous, granite and metamorphic rock, the probabilities against intensities probably should be modified downward one (or a fraction more) units on the Modified Mercalli scale (i.e., for extra firm site, the probability of intensity X, for instance, becomes probability of intensity IX or a fraction lower).

Some sources on predicting earthquake recurrent times for a few locations in California provide a check against some parameters such as the soil condition used in the computer code.

A. The Borrego Mountain earthquake of April 9, 1968 was believed to be a 200-year event.¹⁷ The ground acceleration was estimated at about 1 g which roughly corresponds to intensity X.¹⁸ The magnitude was 6.4. The computer predictions are

<u>Intensity (MM scale)</u>	<u>Recurrent Time (yr.)</u>
IX	29.11
X	59.06
XI	167.48

This comparison shows that if the prediction of the reference is valid, the intensity scale of the computer predictions should be lowered about one

unit, (i.e., the expectation of the recurrent time for intensity X is about 167.48 years).

B. Reid¹⁹ predicted that about every 100 years an earthquake of April 18, 1906 intensity will repeat at San Francisco. The maximum intensity of the 1906 San Francisco earthquake was XI.²⁰ The corresponding computer predictions are

<u>Intensity (MM scale)</u>	<u>Recurrent time(years)</u>
X	43.68
XI	143.55
XII	893.27

The prediction for intensity XI seems to be in a reasonable agreement with Reid's. However, as discussed above, the most severe shaking occurred at the site of filled land which is about one intensity scale softer than what was assumed in the computer code. In other words, compared with Reid's prediction, the computer code also predicts that 1906 San Francisco earthquake is roughly a 100-year event provided those parameters used in the code can be considered as of soft soil instead of firm soil.

C. The February 9, 1971 San Fernando earthquake is generally believed to be a 200 year event. The intensity at Van Norman Dam was IX. The computer code predictions for Van Norman Dam are

<u>Intensity (MM scale)</u>	<u>Recurrent time(years)</u>
IX	68.58
X	205.49
XI	36861.80

This comparison shows again that if the intensity scale of the computer code is lowered by one unit, the prediction for intensity IX at Van Norman Dam location then seems more reasonable.

D. At the San Onofre nuclear power plant site, the intensity and corresponding expectancy of earthquakes²¹ and the computer predictions are listed below:

<u>INTENSITY (MM SCALE)</u>	<u>RECURRENT TIME (PRED.)</u>	<u>RECURRENT TIME (COMPUTED)</u>
VIII	100 years	49.40 years
IX	600 years	173.96 years
X	-	1081.50 years

Again, the comparison shows that there is about one unit (or slightly more) difference in the Modified Mercalli Intensity Scale between predictions by the computer code and deduction from historical records.

In summary, the computer code based on the fault theory does estimate the relative seismic risks. The parameters assumed for firm soil are more likely associated with a softer soil (about one intensity scale unit difference). The probabilities associated with various earthquake intensities should therefore be modified according to the softness of the soil at the earthquake site. Keeping these observations in mind, we see that the computer code used herein seems to be a convenient and useful tool that provides a general estimate of the earthquake probabilities in various large regions of California.

Table 5 shows the computer predictions for some earthen dam sites in California. Their locations are indicated in Figure 1.

It is believed that the uncertainties in the predicted recurrence intervals are greatest for the largest earthquakes, e.g., MM X, for which little empirical basis exists.

TABLE 5

EARTHQUAKE PREDICTIONS FOR SOME EARTH DAMS IN CALIFORNIA

DAM NAME	INTENSITY*	PROB./YR.	RECURRENCE TIME (YR)
Lake Chabot	VII - - - - -	0.22	4
	VIII - - - - -	0.17	5
	IX - - - - -	0.12	8
	X - - - - -	0.057	2 x 10
	XI - - - - -	0.021	5 x 10
San Andreas Lake	VII - - - - -	0.20	5
	VIII - - - - -	0.11	8
	IX - - - - -	0.044	2 x 10
	X - - - - -	0.014	7 x 10
	XI - - - - -	0.0032	3 x 10 ²
San Pablo	VII - - - - -	0.18	5
	VIII - - - - -	0.12	8
	IX - - - - -	0.076	1 x 10
	X - - - - -	0.032	3 x 10
	XI - - - - -	0.011	9 x 10
Van Norman	VII - - - - -	0.10	9
	VIII - - - - -	0.042	2 x 10
	IX - - - - -	0.014	7 x 10
	X - - - - -	0.0049	2 x 10 ²
	XI - - - - -	0.000027	4 x 10 ⁴
Lower Franklin	VII - - - - -	0.084	1 x 10
	VIII - - - - -	0.036	3 x 10
	IX - - - - -	0.013	8 x 10
	X - - - - -	0.0018	6 x 10 ²
	XI - - - - -	0.00045	2 x 10 ³
Stone Canyon	VII - - - - -	0.084	1 x 10
	VIII - - - - -	0.036	3 x 10
	IX - - - - -	0.012	8 x 10
	X - - - - -	0.0014	7 x 10 ²
	XI - - - - -	0.00033	3 x 10 ³
Chatsworth	VII - - - - -	0.097	1 x 10
	VIII - - - - -	0.039	3 x 10
	IX - - - - -	0.013	7 x 10
	X - - - - -	0.0028	4 x 10 ²
	XI - - - - -	0.000026	4 x 10 ⁴

* Modifications, depending on the softness of the soil at the site, are not included. The dotted lines shown are only to remind the reader that, generally for a firm soil site, the intensity scale is off about one unit. See the discussions in Section III.

TABLE 5 (Continued)

EARTHQUAKE PREDICTIONS FOR SOME EARTH DAMS IN CALIFORNIA

DAM NAME	INTENSITY*	PROB./YR.	RECURRENCE TIME (YR)
Encino	VII - - - - -	0.088	1 x 10
	VIII - - - - -	0.037	3 x 10
	IX - - - - -	0.012	8 x 10
	X - - - - -	0.0013	8 x 10 ²
	XI	0.00016	6 x 10 ³
Mulholland	VII - - - - -	0.089	1 x 10
	VIII - - - - -	0.037	3 x 10
	IX - - - - -	0.014	7 x 10
	X - - - - -	0.0031	3 x 10 ²
	XI	0.00040	3 x 10 ³
San Leandro	VII - - - - -	0.22	4
	VIII - - - - -	0.17	5
	IX - - - - -	0.12	8
	X - - - - -	0.063	2 x 10
	XI	0.023	4 x 10

Duke and Egouchi²⁴ gave the following estimates of MM intensities for four of the dam sites of this study as follows:

<u>Dam Name</u>	<u>Intensity</u>	<u>Prob./Yr.</u>
Van Norman	VIII	>0.1
	IX	0.053
	X	0.0080
Stone Canyon	VIII	0.051
	IX	0.019
	X	0.0086
Chatsworth	VIII	0.064
	IX	0.026
	X	0.011
Encino	VIII	0.092
	IX	0.032
	X	0.014

Some basis for comparison between history and the predictions of Table 5 is available. There are 30 hydraulic filled dams in California, one of which currently contains no water, and one has failed. Many of them are restricted to levels well below the spillway. These dams have operated for a total of 1,794 years. One dam failed due to the 1971 San Fernando earthquake after 50 years of operation. One earth dam, which is not included in either Duke's or Sherard's survey, failed in 1963 after 12 years of operation, not the result of an earthquake but due to settling. This dam was a model of modern design and construction. Four and a half hours of notice was available before this failure and evacuation began two and a half hours before the failure. Five people died with an estimated property damage of approximately 15 million dollars.

Figure 1. Some Earth Dams in California.

8. EFFECTS OF EVACUATION ON MORTALITY PREDICTION

8.1 Introduction

The procedure for estimating the effects of evacuation entails initially assuming a particular shape for the evacuation rate histogram. This assumption of the shape is largely based on experience. Figure 2 indicates such a histogram. Based on this, by a numerical integration, an "Integrated Histogram," which predicts the cumulative percentage of the total population evacuated in each hydraulic reach as a function of time, can be obtained. Figure 3 is the integrated histogram. To compute the percent of people evacuated, it is now only necessary to determine the wave head position as a function of time. Through the use of the equation $V_{av} = \{Q_{av}/(A@y_n)\}$, the average wave velocity in each hydraulic reach can be calculated, and hence the wave head position at any specified time can be determined.

In this study, it will be further assumed that the time in which the people in a given hydraulic reach must evacuate is determined solely by the time taken by the wave head to arrive at the middle of such a reach. This assumption results in a time averaging over the reach lengths.

Finally, with a knowledge of the time taken for the wave head to arrive at the middle of any given hydraulic reach and the percentage of the population that can be evacuated from there in the same length of time, an estimate of the number of people evacuated may be made. This estimate is merely the product of the number of people in the reach and the percentage of population which can be evacuated in the available time. The mortality number can now be obtained by subtracting the above estimate from the actual number of people living in the area defined by the chosen hydraulic reach. Since there exists a possibility that fewer people are present in the area during the day-time as compared with the nighttime, the results for the number of people evacuated between the day and the night may differ.

Figure 2. Assumed Evacuation Rate Histogram.

Figure 3. Integrated Evacuation Rate Histogram.

8.2 Evacuation Effects for Stone Canyon Dam

8.2.1 Determination of Water Velocity Within Each Reach:

The average water velocity, V_{ave} , is given by $V_{ave} = \frac{Q_{av}}{A(@y_n)}$. Q_{av} is given by (SC.5) as $2.37 \times 10^6 \text{ ft}^3/\text{sec.}$

Reach	Length (ft.)	Approximate y_n in ft.	$A \times 10^{-4}$ in ft.^2	V_{ave} (ft./sec.)
1	0 - 8,000	~124	4.0082	59
2	8,000 - 16,000	~ 20	12,2270	19
3	16,000 - 24,000	~ 16	17.9050	13
4	24,000 - 32,000	~ 16	23.5450	10
5	32,000 - 40,000	~ 16	30.8650	8

Schematic Diagram of Wave Front Position

vs. Length of Time After Break:

0 ft.		0 min.
4,000 ft.
8,000 ft.		2.26 min.
12,000 ft.	_____	5.77 min.
16,000 ft.	_____	9.28 min.
20,000 ft.	_____	14.41 min.
24,000 ft.	_____	19.54 min.
28,000 ft.	_____	26.21 min.
32,000 ft.	_____	32.88 min.
36,000 ft.	_____	41.21 min.
40,000 ft.	_____	49.54 min.

8.2.2 Determination of Amount of Population Evacuated:

Using the integrated assumed histogram (Figure 3) we were able to compute the following mortality rates.

Integrated Histogram Results:

Reach #	Time	Expected No. of Deaths w/o Evacuation	No. of People Evacuated	Expected No. of Deaths w/Evacuation
1	Day	No deaths assumed	0	No deaths assumed
	Night	No deaths assumed	0	No deaths assumed
2	Day	14,000	0	14,000
	Night	20,000	0	20,000
3	Day	30,888	0	30,888
	Night	54,960	0	54,960
4	Day	36,126	2,529	33,597
	Night	59,320	4,153	55,167
5	Day	44,457	9,336	35,121
	Night	73,000	15,330	57,670
Total	Day	125,471	11,865	113,606
	Night	207,280	19,483	187,797

8.3 Evacuation Effects for Van Norman Dam

8.3.1 Determination of Water Velocity Within Each Reach

The average water velocity, V_{ave} , is given by $V_{ave} = Q_{av}/A(@yn)$. Q_{av} is given by (VN.5) as $3.502 \times 10^6 \text{ ft}^3/\text{sec.}$

Reach	Length (ft)	Approximate yn in ft.	$A \times 10^{-4}$ in ft^2	V_{ave} . (ft/sec.)
1	0-12,000	37.80	21.08	16.61
2	12,000-24,000	20.0	30.80	11.37
3	24,000-36,000	14.75	37.76	9.27

Schematic Diagram of Wave Front

Position vs. Length of Time After Break:

0 ft.	_____	0 min.
6,000 ft.	_____	6.02 min.
12,000 ft.	_____	12.04 min.
18,000 ft.	_____	20.84 min.
24,000 ft.	_____	29.63 min.
30,000 ft.	_____	40.42 min.
36,000 ft.	_____	51.21 min.

8.3.2 Determination of Amount of Population Evacuated:

Using the integrated assumed histogram we were able to compute the following mortality rates.

Integrated Histogram Results:*

Reach No.	Time	Expected No. of Deaths w/o Evacuation	No. of People Evacuated	Expected No. of Deaths w/Evaluation
1	Day	10,800	0	10,800
	Night	17,500	0	17,500
2	Day	18,500	555	17,945
	Night	31,200	936	30,264
3	Day	42,400	9,328	33,072
	Night	74,700	16,434	58,266
Total	Day	71,700	9,883	61,817
	Night	123,400	17,370	106,030

* Above results are based on the assumption of immediate notification of disaster after the dam failure has occurred.

9. CONCLUSIONS

Mortalities in the event of sudden, complete dam failure ranged from 11,000 to 260,000 for the eleven dams studied. Evacuation produced a 10 to 20% reduction in mortalities for the two dams examined in this regard. Property damage estimates were as high as \$720 million.

It was assumed that an earthquake having an intensity IX or X on the Modified Mercalli Scale would be sufficient to completely fail earthen dams. Recurrence intervals for an MM IX earthquake varied between 20 and 800 years for the dams studied. Recurrence intervals for an MM X varied between 50 and 30,000 years.

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APPENDICES

Appendix A-1

Calculation of $Q_{\max}$ and Q_{av}

Case 1: Steady flow ($Q_{\max}$)

A. Regular Method

(1) Assume infinite water supply (i.e., the height of water does not change with time).

(11) Assume that complete failure of dam can be approximated by an orifice of Length L and height H (corresponding to the water height).

The following discussion is with reference to Figure A1. The ideal velocity of a stream filament or layer issuing from the orifice at a distance ξ below the undisturbed surface is given by

$$V_{\text{ideal}} = \sqrt{2g\xi}$$

layer thickness is $d\xi$ and its width L . The ideal volume rate of flow for an element is

$$\begin{aligned} dQ_{\text{ideal}} &= Ld\xi v \\ &= L\sqrt{2g\xi} d\xi. \end{aligned}$$

This equation when integrated between the limits $\xi = 0$ and $\xi = H$ (where $\xi = 0$ is @ the lower surface), will give the rate of flow for the orifice.

Now,

$$L\sqrt{2g} \int_0^H \xi^{1/2} d\xi = \frac{2}{3} L\sqrt{2g} H^{3/2}$$

Therefore,

$$Q_{\max}^{(1)} = \frac{2}{3} L\sqrt{2g} H^3 \quad (\text{A1-1})$$

where the superscript (1) on $Q_{\max}$ denotes the application of the regular method of calculation.

B. Critical Depth Method

It is customary to calculate $Q_{\max}$ through the critical depth method where the water velocity is evaluated from the energy expression. Based on the principle that nature minimizes energy, it is possible to write

$$\frac{V^2}{2g} = \frac{1}{3} H$$

Therefore,

$$V^2 = \frac{2}{3} gH. \text{ This yields,}$$

$$\begin{aligned} Q_{\max}^{(2)} &= VA \\ &= \frac{2}{3} VLH \\ &= \frac{2}{3} \cdot \frac{\sqrt{2gH}}{\sqrt{3}} LH \\ &= \frac{2}{3} \frac{L}{\sqrt{3}} \sqrt{2g} H^{\frac{3}{2}} \end{aligned} \tag{A1-2}$$

Comparison of this expression with (A1-1) shows that $Q_{\max}^{(2)}$ calculated by the critical depth method and $Q_{\max}^{(1)}$ calculated by the regular method always differ by a factor of $\frac{1}{\sqrt{3}}$.

It is important to note that the values of Q calculated energy considerations gives a more accurate and realistic value for $Q_{\max}$ since the procedure, in a way, accounts for the dip in the water level at the orifice.

Case 2: Unsteady Flow (Q_{av})

(1) Assume finite water supply (i.e., the height of water does change with time).

The following discussion is with reference to Figure A2. In the case of unsteady flow, instead of measuring the distance to the element $d\xi$ from the surface of the water, consider measurements from the bottom surface.

The velocity of the stream filament is given by

$$v_{ideal} = \sqrt{2g(h-\xi)} , \text{ where}$$

$(h-\xi)$ is the distance below the undisturbed surface where the element is located.

From Case 1, we know that

$$dQ_{ideal} = L d\xi v.$$

If we take a small interval of time (dt) about the time (T) , such that $h(t)$ is constant during the interval, then we may write

$$dQ(t)_{ideal} = L \sqrt{2g(h-\xi)} d\xi,$$

and the total volume rate of flow from the time interval (dt) at time (t) is given by

$$Q(t) = \int_0^{h(t)} L \sqrt{2g(h-\xi)} d\xi .$$

The total amount of water leaving the orifice within the time interval (dt) is given by

$$Q(t) dt = \int_0^{h(t)} L \sqrt{2g(h(t)-\xi)} d\xi dt.$$

However, we recognize that this amount must equal the volume decrease of the water in the lake, which is equal to $(-A(h) dh)$.

Therefore,

$$-A(h) dh = \int_0^{h(t)} L \sqrt{2g(h(t)-\xi)} d\xi dt .$$

which yields,

$$dt = \frac{-A(h) dh}{\int_0^{h(t)} L \sqrt{2g(h(t)-\xi)} d\xi}$$

This expression will enable us to find the total amount of time necessary to completely empty the dam. Letting T be this time,

$$T = \int_0^T dt = \int_H^0 - \left\{ \frac{A(h) dh}{\int_0^{h(t)} L \sqrt{2g(h(t)-\xi)} d\xi} \right\} .$$

Integrating with respect to ξ ,

$$T = \int_H^0 - \frac{A(h) dh}{\frac{2}{3} L \sqrt{2g} h^{3/2}} \quad (A1-3)$$

Equation (A1-3) involves the area of the lake expressed as a function of height ($A(h)$). This expression, in reality, will be too complicated and cannot be expressed in general terms. However, in several instances one can hypothesize that the total shape of the immediate upstream topography and the water catchment behind the dam can be equivalently and sufficiently accurately expressed as a solid prismatic wedge. Figures A3(a) and A3(b) show the prismatic wedge model. This hypothesis considerably simplifies the mathematical descriptions and enables us to evaluate (A1-3).

Under the prismatic wedge model assumption, we can represent the area of the lake at any height (h) by a rectangle of constant width (w_o) and a length (L) given by

$$L(h) = \frac{L_o}{H_o} h \quad (A1-4)$$

Since the width of rectangle is constant, we may write the above expression as:

$$A(h) = \frac{A_o}{H_o} h \quad (A1-5)$$

Therefore,

$$\begin{aligned}
 T &= \int_H^0 - \frac{A(h) dh}{\frac{2}{3} L \sqrt{2g} h^{3/2}} \\
 &= \int_H^0 - \frac{A_o h dh}{\frac{2}{3} L H_o \sqrt{2g} h^{3/2}} \\
 &= \frac{3A_o}{L \sqrt{2g} H_o}
 \end{aligned} \tag{A1-6}$$

Defining Q_{av} as the quantity obtained by dividing the total volume of the lake by the time required to empty the lake,

$$Q_{av} = \frac{\text{total Volume}}{T} = \frac{\text{total volume}}{\frac{3A_o}{L \sqrt{2g} H_o}} \tag{A1-7}$$

Also, it is possible to estimate the height of the water at various times by a changing of the limits on the previous equation.

Thus,

$$\begin{aligned}
 \int_0^{t'} dt &= \int_{H_o}^{h'} - \frac{A(h) dh}{\int_0^{h(t)} \sqrt{2g(h(t)-\xi)} d\xi} \\
 &= \frac{-3A_o}{2LH_o \sqrt{2g}} \int_{H_o}^{h'} \frac{dh}{h^{1/2}} .
 \end{aligned}$$

This yields,

$$h' = \left(\sqrt{H_o} - \frac{t' L H_o \sqrt{2g}}{3A_o} \right)^2 \tag{A1-8}$$

Time of first large wave

Time of first large wave,

$$\tilde{t} \approx \sqrt{\frac{H}{g}} \tag{A1-9}$$

If, therefore, it is apparent that a large amount of water is released almost immediately after the dam failure.

Square Cross Sectional Model Theory

Consider (A1-3) which is,

$$T = \int_H^0 - \frac{A(h)dh}{\frac{2}{3} L\sqrt{2g} h^{3/2}}$$

This, as pointed out earlier, involves $A(h)$ which is the area of the lake expressed as a function of height. In some instances it is possible to hypothesize that the total shape of the immediate upstream topography and the water catchment behind the dam can be equivalently and sufficiently accurately expressed by a solid prism of square cross sections. Figure A4 shows the square cross section model. This hypothesis considerably simplifies the mathematical descriptions and enables us to evaluate (A1-3).

By law of similar triangles,

$$\frac{L_o}{\ell'} = \frac{H_o}{h'} \quad (A1-10)$$

Therefore,

$$\frac{L_o^2}{\ell'^2} = \frac{H_o^2}{h'^2}$$

However, since we are dealing with square cross sections,

$$L_o^2 = A_o \text{ and } \ell'^2 = A'$$

Thus,

$$A' = \frac{A_o}{H_o^2} h'^2$$

Total volume,

$$\begin{aligned}
&= \int_0^{H_o} A(h) dh \\
&= \int_0^{H_o} \frac{A_o}{H_o^2} h'^2 dh \\
&= \frac{1}{3} A_o H_o
\end{aligned} \tag{A1-11}$$

Consider now (A1-3) with $A(h) = \frac{A_o}{H_o^2} h^2$

Therefore,

$$\begin{aligned}
T &= \int_{H_o}^0 - \frac{A_o h^2 dh}{\frac{2}{3} \sqrt{2g} L H_o^2 h^{3/2}} \\
&= \frac{A_o}{L \sqrt{2g} H_o}
\end{aligned} \tag{A1-12}$$

Also, it is possible to estimate the height of the water at various times as follows:

Consider,

$$\int_0^{t'} dt' = t' = \int_{H_o}^{h'} - \frac{A(h) dh}{\frac{2}{3} L \sqrt{2g} h^{3/2}} \tag{A1-13}$$

$$= \int_{H_o}^{h'} - \frac{A_o}{H_o^2} \frac{h^2 dh}{\frac{2}{3} \sqrt{2g} L h^{3/2}} \tag{A1-14}$$

$$= \frac{A_o}{H_o^2 \sqrt{2g} L} (H_o^{3/2} - h'^{3/2})$$

Therefore,

$$h' = \left\{ H_o^{3/2} - \frac{t' L H_o^2 \sqrt{2g}}{A_o} \right\}^{2/3} \tag{A1-15}$$

Appendix - A-2

Manning's Formula

The fundamental equation for uniform open-channel flow may be derived readily by equating the equal and opposite force components of gravity and resistance and by the application of some fundamental notions of fluid mechanics. Chezy obtained such a flow rate equation and this can be written

$$Q = CA\sqrt{RS},$$

where

Q = flow rate

$$C = \sqrt{\frac{8g}{f}}$$

f = the friction factor

g = γ/ρ

γ = specific weight

ρ = density

R = A/P = hydraulic radius

A = area of flow cross section

P = wetted perimeter

S = slope of both channel bed and liquid surface

The Chezy coefficient C in the above may be derived from the work of Manning.

Manning's results may be summarized by the empirical relation

$$C = \frac{1.49R^{1/6}}{n}$$

which may be combined with the Chezy equation to give the so-called Chezy-Manning equation

$$Q = \left(\frac{1.49}{n} \right) AR^{2/3} S^{1/2} \quad (A2-1)$$

In this study (A2-1) is referred to simply as the Manning equation. The Manning n is obtained from a descriptive statement of the flow channel roughness and is usually selected from engineering experience.

Figures A-1 and A-2. Steady and Unsteady Flow Descriptions.

Figures A-3(a) and A-3(b). Prismatic Wedge Model.

Figure A-4. Square Cross Sectional Model.

St. Francis Dam

County: Los Angeles

Location	{	Sec 1
		Township 5
		Range 16 W

Type	Concrete
------	----------

Capacity	38000 Acre-feet
----------	-----------------

Height of Dam	208 Ft
---------------	--------

Base Elevation	1630 Ft
----------------	---------

Crest Elevation	1838 Ft
-----------------	---------

Spillway Elevation	1835 Ft
--------------------	---------

Surface Area @

Spillway Elevation	613 Acres
--------------------	-----------

Length of Main Section	700 Feet
------------------------	----------

Calculation of Area @ Various Water Heights:

From the square-cross-section model, we know that $A(h) = \frac{A_o}{H_o^2} h^2$. To see how accurately this computes the area for various, specific water heights, we graphed the area as a function of height (as predicted by the square-cross-section model) on an already existing graph of the Actual Area vs. Height. From graph SFD 1, we see that our model gives an extremely good representation of the area as a function of height.

Determination of Actual Time of Emptying of Dam:

According to the square-cross-section model, the time necessary to empty the Dam is given by the expression:

$$T = \frac{A_o}{LH_o^{1/2} \sqrt{2g}} = \frac{613 \text{ Acres} \times 43560 \text{ ft}^2/\text{Acres}}{700 \times 8 \times (205)^{1/2}}$$
$$= \frac{613 \times 43560}{700 \times 8 \times 14.319} = 333 \text{ seconds} = 5.55 \text{ minutes.}$$

Determination of Q_{ave} :

$$Q_{ave.} = \frac{V_{Tot.}}{T_{Tot.}} = \frac{38,000 \text{ acre-ft.}}{333 \text{ seconds}} = \frac{1655.280 \times 10^6 \text{ ft}^3}{333 \text{ seconds}}$$
$$= 4.971 \times 10^6 \frac{\text{ft}^3}{\text{sec}}$$

(Note: Since the dam did not break instantaneously, as we have assumed in our analysis, the actual $Q_{ave.}$ value should be slightly less than the one calculated above).

Estimation of the Dimensions of Constant Flow Channel: (0-12,000 ft.)

I. Assumptions Made:

1. Since the flow channel is well defined by the natural topography of the area for the first reach (i.e., by high elevations on both sides of the channel), it can be assumed that the effects of divergent outflow of water or

of the lateral sloping above 6000 ft. are insignificant. Any divergent flow will be returned to the channel because of the steep inward slopes of the two sides of the channel (see graphs SFD 2a, 2b and 2c).

2. Assume that the constant flow channel configurations can be approximated by a trapezoid with dimensions shown in Figure 1.

II. Determination of Parameters B_1 , B_2 , C, A, R:

1. B:

$$B_1 = B_2 = \frac{y_n}{\sin 70^\circ}$$

2. C:

$$\begin{aligned} C &= B + 2B_2 \cos 70^\circ \\ &= \{500 + 2y_n \cot 70^\circ\} \end{aligned}$$

3. A:

$$\begin{aligned} A &= \left(\frac{C+B}{2}\right) y_n \\ &= \frac{1}{2} \{(500 + 2y_n \cot 70^\circ) + 500\}y_n \end{aligned}$$

4. P:

$$\begin{aligned} P &= B_1 + B + B_2 \\ &= 2B_1 + B = \left(\frac{2y_n}{\sin 70^\circ}\right) + 500 \end{aligned}$$

5. R:

$$R = \frac{A}{P} = \frac{\frac{1}{2} \{(500 + 2y_n \cot 70^\circ) + 500\}y_n}{\frac{2y_n}{\sin 70^\circ} + 500}$$

III. Values of Parameters for Various Water Heights:

y_n	$A \times 10^{-4}$	R
<u>(Ft)</u>	<u>(Ft²)</u>	<u>(Ft)</u>
4	.2006	3.94
12	.6052	11.52
20	1.0146	18.70

y_n (Ft)	$A \times 10^{-4}$ (Ft ²)	R (Ft)
28	1.428	25.53
36	1.847	32.03
44	2.270	38.25
52	2.698	44.19
60	3.131	49.88
68	3.568	55.35
76	4.010	60.60
84	4.457	65.66
92	4.908	70.54
100	5.364	75.25
108	5.825	79.80
116	6.290	84.21
124	6.760	88.49
132	7.234	92.64

IV. From Manning's Formula given by (A2-1), we can calculate Q-values for various water heights, thus enabling us to make a Q versus Y_n graph. From SFD 3 graph, we see that the normal depth of the first reach, using a roughness coefficient of 0.07, is approximately 130 ft. Also the flood plain for the first reach is approximately 500 ft. wide and 12,000 ft. long.

Estimation of the Dimensions of Constant Flow Channel-Second Reach

(12,000 - 24,000 ft.)

I. Assumptions Made:

1. Since the flow channel is well defined by the natural topography of the area for the second reach (i.e., by high elevations on both sides of the channel), it can be assumed that the effects of divergent outflow of water

or of the lateral sloping above 18,000 ft. are insignificant. Any divergent flow will be returned to the channel because of the steep inward slopes of the two sides of the channel (See Graphs SFD 2c, 2d, and 2e)

2. Assume that the constant flow channel configurations can be approximated by a trapezoid with dimensions show in Figure 2.

Determination of Parameters B_1 , B_2 , C, A, R:

1. B:

$$B_1 = B_2 = \frac{y_n}{\sin 70^\circ}$$

2. C:

$$\begin{aligned} C &= B + 2B_2 \cos 70^\circ \\ &= \{1000 + 2y_n \cot 70^\circ\} \end{aligned}$$

3. A:

$$\begin{aligned} A &= \frac{C+B}{2} y_n \\ &= 1/2 \{(1000 + 2y_n \cot 70^\circ) + 1000\} y_n \end{aligned}$$

4. P:

$$\begin{aligned} P &= B_1 + B + B_2 = 2B_1 + B \\ &= (2y_n/\sin 70^\circ) + 1000 \end{aligned}$$

5. R:

$$R = \frac{A}{P} = \frac{\frac{1}{2}((1000 + 2y_n \cot 70^\circ) + 1000)y_n}{2y_n/\sin 70^\circ + 1000}$$

II. Values of Parameters for Various Water Heights:

y_n	$A \times 10^{-4}$	R
<u>(Ft)</u>	<u>(Ft²)</u>	<u>(Ft)</u>
4	.4006	3.97
12	1.2052	11.75
20	2.0146	19.32
28	2.8285	26.69

y_n	$A \times 10^{-4}$	R
<u>(Ft)</u>	<u>(Ft²)</u>	<u>(Ft)</u>
36	3.6472	33.88
44	4.4705	40.85
52	5.2984	47.71
60	6.1310	54.37
68	6.9683	60.87
76	7.8102	67.23
84	8.6568	73.44
92	9.5081	79.51
100	10.364	85.45
108	11.225	91.27
116	12.090	96.96

III. From Manning's formula given by (A2-1), we can calculate Q - values for various water heights, thus enabling us to make a Q versus y_n graph. From graph SFD 4, we see that the normal depth of the second reach, using a roughness coefficient of 0.07, is approximately 110 ft. Also the flood plain for the second reach is approximately 1000 ft. wide and 12,000 ft. long.

Estimation of Dimensions of Constant Flow Channel

Third Reach (24,000 - 36,000 ft.)

I. Assumptions Made:

1. Since the flow channel is well defined by the natural topography of the area for the third reach (i.e., by high elevations on both sides of the channel), it can be assumed that the effects of divergent outflow of water or of the lateral sloping above 30,000 ft. are insignificant. Any divergent flow will be returned to the channel because of the steep inward slopes of the two sides of the channel (See Graphs SFD 2e, 2f, 2g)

2. Assume that the constant flow channel configurations can be approximated by a trapezoid with dimensions shown in Figure 3.

Determination of Parameters B_1 , B_2 , C, A, R:

1. B:

$$B_1 = B_2 = y_n / \sin 45^\circ$$

2. C:

$$\begin{aligned} C &= B + 2B_2 \cos 70^\circ \\ &= \{1500 + 2y_n \cot 45^\circ\} \end{aligned}$$

3. A:

$$\begin{aligned} A &= \left(\frac{C+B}{2} \right) y_n \\ &= \frac{1}{2} \{ (1500 + 2y_n \cot 45^\circ) + 1500 \} y_n \end{aligned}$$

4. P:

$$\begin{aligned} P &= B_1 + B + B_2 = 2B_1 + B \\ &= (2y_n / \sin 45^\circ) + 1500 \end{aligned}$$

5. R:

$$R = \frac{A}{P} = \frac{\frac{1}{2} \{ (1500 + 2y_n \cot 45^\circ) + 1500 \} y_n}{2y_n / \sin 45^\circ + 1500}$$

II. Values of Parameters for Various Water Heights:

y_n	$A \times 10^{-4}$	R
<u>(Ft)</u>	<u>(Ft²)</u>	<u>(Ft)</u>
4	.6016	3.98
12	1.8144	11.83
20	3.0400	19.53
28	4.2784	27.09
36	5.5296	34.52
44	6.7936	41.82
52	8.0704	49.00

y_n	$A \times 10^{-4}$	R
<u>(Ft)</u>	<u>(Ft²)</u>	<u>(Ft)</u>
60	9.3600	56.06
68	10.662	63.00
76	11.978	69.84
84	13.306	76.58

III. From Manning's formula given by (A2-1), we can calculate Q-values for various water heights, thus enabling us to make a Q versus y_n graph. From graph SFD 5, we see that the normal depth of the third reach, using a roughness coefficient of 0.07, is approximately 83 ft. Also the flood plain for the third reach is approximately 1500 ft. wide and 12,000 ft. long.

Conclusions

It was reported after failure of the St. Francis Dam that a wall of water 125 feet high was traveling down the canyon beneath the dam. At 10 miles from the dam this wall of water was reported to be approximately 78 feet high.

From our calculations we have predicted that the normal depth of water in the canyon immediately beneath the dam would be approximately 130 feet; and, at a distance of approximately seven miles from the dam, we have predicted a normal depth of 83 feet.

Therefore, our results fall very close to those which were reported at the time of failure of the St. Francis Dam. This lends support to our assumptions concerning catchment geometry (i.e., square-cross-section model) and our use of Q_{ave} .

Figures 1, 2 and 3. Trapezoidal Cross Sections: St. Francis Dam Problem.

Graph SFD1. Elevation vs Surface Area and Capacity.

Graph SFD2a. Downstream Topography.

HORIZ. SCALE
1" = 2000 FT.

6000 FT. DOWNSTREAM

1900'

1800'

1700'

1600'

1900'

1800'

1700'

1600'

1500'

ELEVATION

8000 FT. DOWNSTREAM

1900'

1800'

1700'

1600'

1500'

10,000 FT. DOWNSTREAM

5"

4"

3"

2"

1"

1"

Graph SFD2b. Downstream Topography.

Graph SFD2c. Downstream Topography.

Graph SFD2d. Downstream Topography.

Graph SFD2e. Downstream Topography.

HORIZ. SCALE
1" = 2000 FT.

Graph SFD2f. Downstream Topography.

HORIZ. SCALE
1" = 2000 FT.

36,000 FT. DOWNSTREAM

38,000 FT. DOWNSTREAM

40,000 FT. DOWNSTREAM

Graph SFD2g. Downstream Topography.

Graph SFD3. Normal Depth vs. Volume Rate of Flow – 1st Reach.
 Slope = 0.25

Graph SFD4. Normal Depth vs. Volume Rate of Flow – 2nd Reach.
 Slope = .010.

Graph SFD5. Normal Depth vs. Volume Rate of Flow — 3rd Reach.
 Slope = .010.

VAN NORMAN DAM

(Also called the Lower San Fernando Dam)

*Dam Number	:	6-15
Name of Owner	:	City of Los Angeles
County	:	Los Angeles
Stream	:	San Fernando Creek
Location	:	{ Section 5 Township 2N Range 15W Baseline and/or Meridian SB
Type	:	Hydraulic Fill
Storage Capacity	:	892.98 x 10 ⁶ cubic feet
Drainage Area	:	14.20 square miles
Reservoir Area	:	16.626 x 10 ⁶ square feet
Crest Elevation	:	1162 feet
Crest Length	:	2200 feet
Spillway Elevation	:	1125 feet
Ground Elevation { Beneath Dam }	:	1020 feet
Height of Dam	:	142 feet
Height of Water	:	105 feet
Crest Thickness	:	24 feet
Dam Volume	:	2,826,986 cubic yards
Year of Completion	:	1918 A.D.

*As assigned by the Department of Water Resources

Calculation of $Q_{\max}$ and Q_{av} under the assumption of complete and instantaneous failure of Dam

From (A1 - 1),

$$Q_{\max}^{(1)} = \frac{2}{3} L \sqrt{2gH^3}$$

Here,

$$L = 2200 \text{ Ft}$$

$$H = 105 \text{ Ft}$$

Therefore,

$$Q_{\max}^{(1)} = 12.6 \times 10^6 \text{ Ft}^3/\text{Sec} \quad (\text{VN.1})$$

From (A1 - 2),

$$\begin{aligned} Q_{\max}^{(2)} &= \frac{1}{\sqrt{3}} Q_{\max}^{(1)} \\ &= 7.3 \times 10^6 \text{ Ft}^3/\text{Sec} \end{aligned} \quad (\text{VN.2})$$

To calculate Q_{av} through the use of equation (A1 - 7), the following procedure is adopted:

From (A1 - 6),

$$\text{Actual time of emptying} = T = 3A / \left(L \sqrt{2gH} \right)$$

Here,

$$A = 16.62 \times 10^6 \text{ Ft}^2$$

Therefore,

$$T_{\text{actual}} = 276.6 \text{ Secs} \quad (\text{VN.3})$$

Let us compare this with the effective time of emptying. Consider

(A1 - 8) which is,

$$h = \left(\sqrt{H} - \frac{t_{LH} \sqrt{2g}}{3A} \right)^2$$

With this equation we can generate VND 5 which is a graph of the height of water against time. From VND 5, we can make an approximate determination of the effective time necessary to empty all the water from behind the dam.

$$\text{From VND 5, } T_{\text{eff}} = 255 \text{ Secs} \quad (\text{VN.4})$$

Therefore,

$$\text{Also, from (A1 - 7), } Q_{\text{av}} = \frac{V}{T}$$

Here,

$$V = 892.98 \times 10^6 \text{ Ft}^3, T = 255 \text{ Secs}$$

Thus,

$$Q_{\text{av}} = 3.502 \times 10^6 \text{ Ft}^3/\text{Sec} \quad (\text{VN.5})$$

Calculations of the time of first large wave and the amount of instantaneous release of water

From (A1 - 9),

$$\tilde{t} \cong \sqrt{\frac{H}{g}} = 1.811 \text{ Sec} \quad (\text{VN.6})$$

Therefore, it is apparent that a large amount of water is released almost immediately after the dam failure. To find out the amount released, let us first calculate the height of water still behind the orifice at the time $\tilde{t}$.

From (A1 - 8), with $t = \tilde{t}$,

$$h = 103.69 \text{ Ft} \quad (\text{VN.7})$$

Therefore,

$$\Delta h \text{ (from } t = 0 \text{ to } t = 1.811 \text{ Sec)} = 1.31 \text{ Ft}$$

The amount of water which flows out of the orifice in $\tilde{t}$ is given by

$$\Delta V \cong A \Delta h = 22.82 \times 10^6 \text{ Ft}^3 \quad (\text{VN.8})$$

Estimation of Dimensions of Constant Flow Channel

FIRST REACH

Assumptions Made

1. Assume water will be in constant flow channel configuration within 2000 ft. downstream of dam.
2. Assume flow channel configuration does not change as water moves downstream.
3. Neglect lateral slope after water has traveled 2000 ft. downstream.
4. Assume that if there is no divergent water flow or lateral sloping above 2000 ft., then the width of the flow channel will just be the width of the orifice. (See figure 1a)
5. Assume that an allowance of 30° , from the lines normal to the dam, will account for divergent water flow through the orifice. (See figure 1a)
6. Assume that an additional allowance of 30° will account for the lateral slope to the right. (Note: This cancels the allowance for the divergent water flow, on the left end of the dam.) (See figure 1a)
7. Assume that we can approximate the constant flow channel configuration by a trapezoid with dimensions given in figures 1b and 1c.

Determination of Parameters B_1 , B_2 , C, A, R; when B = 5,500 feet and

Normal Depth = y_n : (figure 1c)

1. B_1 & B_2 :

$$B_1 = B_2 = \left(y_n / \sin 26^{\circ} \right)$$

2. C:

$$C = B + 2B_2 \cos 26^\circ \text{ or } C = 5500 + 2 y_n \cot 26^\circ$$

3. A:

$$A = \frac{1}{2} (C + B) y_n \text{ or } A = \frac{1}{2} (5500 + (5500 + 2y_n \cot 26^\circ))y_n$$

4. R:

$$R = A/P \text{ (where } P = \text{wetted perimeter} = B_1 + B + B_2)$$

But

$$B_1 = B_2 = y_n / \sin 26^\circ$$

Therefore,

$$R = A / (2y_n / \sin 26^\circ + B)$$

Or,

$$R = \frac{\frac{1}{2} (5500 + (5500 + 2y_n \cot 26^\circ))y_n}{\frac{2y_n}{\sin 26^\circ} + 5500}$$

Note: If we already know the value of the cross-sectional area, we may just substitute it in the numerator.

Values of Parameters for Various Water Heights:

$$\sin 26^{\circ} = 0.43837$$

$$\cot 26^{\circ} = 2.0503$$

y_n (ft)	B (ft)	B_1 & B_2 (ft)	$C \times 10^{-3}$ (ft)	$A \times 10^{-4}$ (ft ²)	$P \times 10^{-3}$ (ft)	R (ft)
0	5500	-0-	5.500	-0-	5.500	-0-
4		9.12	5.516	2.20	5.518	3.99
8		18.24	5.533	4.41	5.436	7.97
12		27.37	5.549	6.63	5.555	11.93
16		36.49	5.566	8.85	5.573	15.83
20		45.62	5.582	11.08	5.591	19.82
24		54.74	5.598	13.31	5.609	23.74

From Manning's formula given by (A2 - 1), we can calculate Q-values for various water depths, thus enabling us to make a Q vs y_n graph. From Graph VND-6 we see that corresponding to $n = 0.07, 0.09$ and 0.11 values, and for a Q_{av} given by (VN.5),

$$\begin{aligned} y_n^{(0.07)} &= 31.6 \text{ ft} \\ y_n^{(0.09)} &= 37.8 \text{ ft} \\ y_n^{(0.11)} &= 42.4 \text{ ft} \end{aligned} \quad (\text{VN.9})$$

Determination of water velocity and physical dimensions of the sheet of water downstream

The water velocity, v is given by

$$v = \frac{Q_{av}}{A @ 37.8'}$$

A@ 37.8' is $21.08 \times 10^4 \text{ Ft}^2$

With Q_{av} given by (VN.5), $v = 16.61 \frac{\text{Ft}}{\text{Sec}}$ (VN.10)

In T_{eff} (= 255 secs), this amount of water would have traveled a distance,

$$\begin{aligned} d &= 16.6 \times 255 \\ &= 4235.5 \text{ Ft} \end{aligned} \quad \text{(VN.11)}$$

In effect, therefore, we will have a solid sheet of water that is

37.8 ft high, 4235.5 ft long and 5500 ft wide

Normal Water Depth for Second Reach: (12,000 to 24,000 feet)

Estimation of Dimensions of Constant Flow Channel:

Assumptions Made:

1. Assume that we can approximate the constant flow channel configuration for the second reach by a rectangle with dimensions determined at the middle of the second reach (i.e., assume water will be in constant flow channel configuration by the time it has reached the middle of the second reach (18,000 feet)).
2. Assume that water flowing from the dam will be confined on the left-hand side by the natural topography of the area (see figure 2a).
3. Assume that an allowance of 30° , from the line normal to the right-hand end of the dam, will account both for natural divergent water flow and the lateral slope of the topography (see figure 2b for dimensions of flow channel).

Determination of Parameters B_1 , B_2 , A, R: when $B = 15,400$ feet, and

Normal Depth = y_n :

1. B_1 & B_2 :

$$B_1 = B_2 = y_n$$

2. A:

$$A = B y_n = 15,400 y_n \text{ Ft}^2$$

3. R:

$$R = A/P = B y_n / (B + 2 y_n); \text{ but since } y_n \ll B$$

$$R = B y_n / (B + 2 y_n) \approx B y_n / B = y_n$$

Therefore,

$$R \approx y_n$$

Values of Q for Various Water Heights and Roughness Coefficients:

Graph VND 7 shows the values of Q for various water heights and roughness coefficients. From this graph we see that for a roughness coefficient of $n = 0.09$ the average water depth for the second reach is 20 ft and the average width of the second reach is 15,400 ft. (approximately 3 miles).

Normal Water Depth for Third Reach: (24,000 to 36,000 feet) Estimation
of Dimensions of Constant Flow Channel:

Assumptions Made:

1. Assume that we can approximate the constant flow channel configuration for the third reach by a rectangle with dimensions determined at the middle of the reach (i.e., assume water will be in constant flow channel configuration by the time it has reached the middle of the third reach (30,000 feet)).
2. Assume that an allowance of 30° , from the line normal to the right-hand end of the dam, will account both for natural divergent water flow and the lateral slope, to the right, of the topography (see figure 4a).
3. Assume that where the natural confinement, to the left of the dam, ends (approximately 12,000' downstream) an allowance of 30° will account both for natural divergent water flow and the lateral slope, to the left, of the topography (see figure 4a).

Determination of Parameters B_1 , B_2 , A, R; when $B = 25,600$ feet and

Normal Depth = y_n :

1. B_1 & B_2 :

$$B_1 = B_2 = y_n$$

2. A:

$$A = B y_n = 25,600 y_n \text{ Ft}^2$$

3. R:

$$R = A/P = By_n/(B + 2y_n); \text{ but since } y_n \ll B,$$

$$R = By_n/(B + 2y_n) \approx By_n/B = y_n$$

Therefore,

$$R \approx y_n$$

Values of Q for Various Water Heights and Roughness Coefficients:

Graph VND 8 shows the values of Q for various water heights and roughness coefficients. From this graph we see that for a roughness coefficient of $n = 0.09$, the average water depth for the third reach is 14.75 feet and the average width of the third reach is 25,600 feet (approximately five miles).

Van Norman Lakes

The flood plain for the Van Norman Lakes (Lower San Fernando Dam) was generated in three reaches extending South to the Sepulveda Flood Control Basin. The number of expected deaths and the amount of damage to single-family dwellings for each reach is as follows:

1st reach:

Expected number of deaths:	Daytime ~10,800 Nighttime ~17,500
Damage to homes:	4,350 homes \$44,800,000

2nd reach:

Expected number of deaths:	Daytime ~ 18,500 Nighttime ~ 31,200
Damage to homes:	6,243 homes \$84,700,000

3rd reach:

Expected number of deaths:	Daytime ~ 42,400 Nighttime ~ 74,700
Damage to homes:	17,987 \$176,500,000

Totals:

Expected number of deaths:	Daytime ~ 71,600 Nighttime ~ 123,400
Damage to homes:	28,500 \$306,000,000

FRONT VIEW OF FLOW CHANNEL
@ 2000' DOWNSTREAM FROM DAM

ACTUAL DIMENSIONS FOR APPROXIMATION OF
CONSTANT FLOW CHANNEL

NOTE: THIS FLOW CHANNEL APPROXIMATION
BECOMES LESS VALID WHEN THE
NORMAL DEPTH OF THE WATER
EXCEEDS 20 FEET.

Figures 1a, 1b, and 1c.

Figure 2a

NOTE: THIS IS THE AVERAGE FLOW
CHANNEL FOR THE SECOND
REACH

Figure 2b

Figures 3a and 3b.

Figure 4a

Figure 4b. Actual Dimensions for Approximation of Constant Flow Channel.
 Note: This is the Average Flow Channel for the Third Reach.

Graph VND.1. Downstream Topography: First Reach.

Graph VND.2. Downstream Slope of Center Line Normal to Dam.

Graph VND.3. Downstream Slope of Left End Line Normal to Dam.

Graph VND.4. Downstream Slope of Right End Line Normal to Dam.

Graph VND.5. Waterheight vs. Time.

Graph VND.6. Volume Rate of Flow vs. Normal Depth (First Reach).

Graph VND.7. Volume Rate of Flow vs. Normal Depth (Second Reach).

Graph VND.8. Volume Rate of Flow vs. Normal Depth (Third Reach).

A: VNFP-1
Flood Plain Below the Van Norman Lakes

San Andreas Dam

(Details below refer to topographical map)

* Dam Number	10 - 10
Name of Owner	City County San Francisco
County	San Mateo
Stream	San Andreas Creek
Location	{ Section 16 Township 4S Range 5W Baseline and/or Meridian MD
Type	Earth
Storage Capacity	8.0586×10^8 cubic feet
Drainage Area	4.40 square miles
Reservoir Area	2.3958×10^7 square feet
Crest Elevation	456 feet
Crest Length	727 feet
Permanent Spillway	450 feet
Effective Height of Dam (Ground Level @ 375 Feet)	81 feet
Crest Thickness	20 feet
Dam Volume	540,000 cubic yards
Year of Completion	1870

* As assigned by Department of Water Resources

Lower Crystal Springs Dam

(Details below refer to topographical map)

* Dam Number	10 - 6
Name of Owner	City County San Francisco
County	San Mateo
Stream	San Mateo Creek
Location	{ Section 1 Township 5S Range 5W Baseline and/or Meridian MD
Type	Gravity
Storage Capacity	2.35224×10^9 cubic feet
Drainage Area	25,000 square miles
Reservoir Area	6.4991×10^7 square feet
Crest Elevation	289 feet
Crest Length	600 feet
Permanent Spillway	284 feet
Effective Height of Dam (Ground Level @ 175 Ft.)	114 feet
Crest Thickness	40 feet
Dam Volume	157,200 cubic yards
Year of Completion	1888

* As assigned by Department of Water Resources

Calculation $Q_{\max}$ and Q_{av} under the Assumption of Complete and Instantaneous Failure of Dams

In the case of San Andreas Dam, we shall assume that the compacted earth completely erodes immediately after a breach in the concrete occurs, i.e., instantaneous complete dam failure.

From (A1-1),

$$Q_{\max}^{(1)} = \frac{2}{3} L \sqrt{2gh}^3$$

Here,

$$L = 560 \text{ ft.}$$

Therefore,

$$Q_{\max}^{(1)} = 1.94 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \quad (\text{SA.1})$$

The length, L, above does not include the dirt embankment on the right side of dam.

From (A1-2),

$$\begin{aligned} Q_{\max}^{(2)} &= \frac{1}{\sqrt{3}} Q_{\max}^{(1)} \\ &= 1.12 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \end{aligned} \quad (\text{SA.2})$$

To calculate Q_{av} through the use of Equation (A1-7), the following procedure is adopted.

From (A1-6),

$$\text{Actual time of emptying} = T_{\text{actual}} = 3A / (L \sqrt{2gH}).$$

Here,

$$A = 20.49 \times 10^6 \text{ ft}^2$$

Therefore,

$$\begin{aligned} T_{\text{actual}} &= 1584.7 \text{ secs.} \\ &\simeq 26.412 \text{ minutes} \end{aligned} \quad (\text{SA.3})$$

Let us compare this with the effective time of emptying. Consider (A1-8) which is,

$$h = \left(\sqrt{H} - \frac{tLH\sqrt{2g}}{3A} \right)^2$$

With this equation we can generate SAD 4 which is a graph of the height of water against time. From SAD 4, we can make an approximate determination of the effective time necessary to empty all the water from behind the dam.

This, from SAD 4, is

$$T_{\text{eff}} = 26 \text{ minutes} \quad (\text{SA.4})$$

Also, from (A1-7),

$$Q_{\text{av}} = \frac{V}{T}$$

Here,

$$V = 827.5 \times 10^6 \text{ ft}^3, \quad T = 26 \text{ minutes.}$$

Therefore,

$$Q_{\text{av}} = 5.30 \times 10^5 \frac{\text{ft}^3}{\text{sec}} \quad (\text{SA.5})$$

Calculations of the Time of First Large Wave and the Amount of Instantaneous Release of Water.

From (A1-9),

$$\tilde{t} \cong \sqrt{\frac{H}{g}} = 1.531 \text{ sec} \quad (\text{SA.6})$$

Therefore, it is apparent that a large amount of water is released almost immediately after the dam failure. To find out the amount of water released in this time, we must first calculate the height of the water still behind the orifice at the time $\tilde{t}$.

From (A1-8), with $t = \tilde{t}$,

$$h = 74.85 \text{ ft.} \quad (\text{SA.7})$$

Therefore,

$$\Delta h \text{ (from } t = 0 \text{ to } t = 1.531 \text{ sec)} = 0.145 \text{ ft.}$$

The amount of water which flows out of the orifice in $\tilde{t}$ is given by

$$\Delta V \approx A \Delta h = 2.95 \times 10^6 \text{ ft}^3 \quad (\text{SA.8})$$

Estimation of Dimensions of Constant Flow Channel

I. Assumptions Made

1. Since the flow channel is well confined by the natural topography of the area (i.e., by high elevations on both sides of the channel), it can be assumed that the effects of divergent outflow of water or of the lateral sloping above 2,000 ft. are insignificant. Any divergent flow will be returned to the channel because of the steep inward slopes of the two sides of the channel. (See Graphs SAD 2a, SAD 2b and Figure 2.)

2. Assume that the constant flow channel configuration can be approximated by a trapezoid with dimensions shown in Figure 2.

II. Determination of Parameters B_1, B_2, C, A, R

1. B

$$B_1 = B_2 = y_n / \sin 45^\circ$$

2. C

$$\begin{aligned} C &= B + 2B_2 \cos 45^\circ \\ &= 600 + 2y_n \cot 45^\circ \end{aligned}$$

3. A

$$\begin{aligned} A &= \frac{C+B}{2} y_n \\ &= \frac{1}{2} \{600 + (600 + 2y_n \cot 45^\circ)\} y_n \end{aligned}$$

4. P

$$\begin{aligned}
 P &= B_1 + B + B_2 \\
 &= 2B_1 + B \\
 &= \left(\frac{2y_n}{\sin 45^\circ} \right) + 600
 \end{aligned}$$

5. R

$$\begin{aligned}
 R &= (A/P) \\
 &= \frac{1/2\{600 + (600 + 2y_n \cot 45^\circ)\}y_n}{\left(\frac{2y_n}{\sin 45^\circ} \right) + 600}
 \end{aligned}$$

III. Values of the Parameters for Various Water Heights

y_n (ft)	$A \times 10^{-4}$ (ft ²)	R (ft)
0	0	0
4	.2416	3.952
8	.4864	7.812
12	.7344	11.585
16	.9856	15.275
20	1.2400	18.886
24	1.4976	22.423
28	1.7584	25.889
32	2.0224	29.289
36	2.2896	32.624
40	2.5600	35.898
44	2.8336	39.114
48	2.9716	42.274
50	3.2500	43.835

IV. From Manning's formula given by (A2-1), we can calculate Q-values for various water heights, thus enabling us to make a Q vs. y_n graph. From Graph SAD5, we see that the minimum normal depth is

$$y_n^{(0.07)} = 37.8 \text{ ft.} \quad (\text{SA.9})$$

With this minimum normal depth value, we are considering the most conservative case. On the other hand,

$$y_{\text{critical}} = \sqrt[3]{\frac{q^2}{g}}, \text{ where } q = \frac{Q}{B}.$$

B is the average width of the flow channel. A conservative estimate of B at 6000 feet downstream of the dam is 1000 feet.

Therefore,

$$q = \frac{3.558 \times 10^5}{10^3} = \underline{\underline{355.8}}$$

Therefore,

$$y_c = \sqrt[3]{\frac{(355.8)^2}{32}} \text{ ft}$$

A comparison of the y_n and y_c values, showing that $y_n > y_c$, indicate that the flow is subcritical. This means that we can break the downstream slope into two straighter slopes and then connect these by smooth, asymptotic transition curves. In other words, no jumps occur.

Determination of Incoming Water Velocity Before the Water Head Reaches the Lower Crystal Springs Lake

Upon the spillage of water from the San Andreas Dam breakage, the flow channel tends to lead the water into the downstream lower Crystal Springs lake. This lake has its own dam which could break and cause disaster.

The water velocity reaching the lower Crystal Springs lake can be calculated as follows:

$$\begin{aligned}
 \text{Velocity} = V &= \frac{Q_{av}}{A(\text{at } 37.8 \text{ ft})} \\
 &= \frac{53.04 \times 10^4 \frac{\text{ft}^3}{\text{sec}}}{24,244 \text{ ft}^2} \\
 &\approx 21.88 \frac{\text{ft}}{\text{sec}}
 \end{aligned}$$

Since the distance from the dam to the lake is approximately 10,000 feet, it will take a time t^* given by,

$$\begin{aligned}
 t^* &= \frac{10,000 \text{ secs}}{21.88} \\
 &= 457.04 \text{ seconds} \\
 &= 7.62 \text{ minutes}
 \end{aligned}$$

for the water from the dam to reach the lake.

Lower Crystal Springs Dam: Preliminary Calculations

Time at which water fills the lower Crystal Springs lake:

$$\text{Area of lower Crystal Springs lake} = 64.99 \times 10^6 \text{ ft}^2$$

$$\text{Elevation of Water Level} = 284 \text{ ft}$$

$$\text{Crest Elevation} = 289 \text{ ft}$$

$$\begin{aligned}
 \text{Volume of Water from Spillway} \\
 \text{Elevation to Top of Dam} &= 5 \times 64.99 \times 10^6 \text{ ft}^3 \quad (\text{LSC.1})
 \end{aligned}$$

This volume calculation assumes that the immediate upstream topography and the water catchment behind the Lower Crystal Springs dam could be sufficiently accurately approximated by a solid prismatic bowl.

On the other hand, the volume of water which must flow from the San Andreas dam breakage to fill up the second dam catchment must be this same volume $324.9 \times 10^6 \text{ ft}^3$. Now consider the following calculations. For the San Andreas Dam we have,

$$H = 75 \text{ ft}$$

$$A = 20.49 \times 10^6 \text{ ft}^2$$

Letting $\Delta V = 324.9 \times 10^6 \text{ ft}^3$, the water head should fall from 75 feet to 56.98 feet to provide a flow volume of this magnitude. The value 56.89 is arrived at from a calculation based on the assumption that the immediate upstream topography and water catchment of the San Andreas Dam can be approximated by a solid prismatic wedge. From the time vs. height graph for the San Andreas lake we see that the time taken for water head to drop from 75 feet to 56.89 feet is approximately 200 seconds. The above approximate calculation shows that in about 3 to 4 minutes time after the breakage of the San Andreas Dam, the issuance of water from the lake behind it would flood the downstream lower Crystal Springs lake.

The instantaneous value of $Q(t)$ at $t = 200$ seconds can now be evaluated.

$$\begin{aligned}
 Q(t) &= \int_0^{h(t)} L \sqrt{2g(h(t) - \zeta)} \, d\zeta \\
 &= \frac{2}{3} L \sqrt{2g} h^{3/2}(t) \\
 &= \frac{2}{3} (560) \sqrt{2(32)} (56.980)^{3/2} \\
 &= 1.285 \times 10^6 \frac{\text{ft}^3}{\text{sec}}
 \end{aligned} \tag{LCS.2}$$

With a known instantaneous value of $Q(t)$ we can calculate the height over the weir, through the use of the following equation:

$$Q(t) = CLH^{3/2} \tag{LCS.3}$$

In (LCS.3), let

$C = 3.25$ (broad crested weir)

$L = 1680$ feet (this value is obtained from the 325 foot contour

which corresponds to an approximate height of 40 feet)

Then,

$$H = 38.118 \text{ feet} \tag{LCS.4}$$

Therefore, with the height of the dam at 114 feet,

$$\left. \begin{array}{l} \text{Effective height of water behind} \\ \text{the lower Crystal Springs Dam} \end{array} \right\} = 114 + 38.118 \\ = 152.118 \text{ feet} \quad (\text{LCS.5})$$

Breakage of the Lower Crystal Springs Dam:

In the case of the Lower Crystal Springs Dam we shall assume that the compacted earth completely erodes immediately after a breach in the concrete occurs, i.e., instantaneous complete dam failure.

From (AI-1),

$$Q_{\max}^{(1)} = \frac{2}{3} L \sqrt{2gH^3}$$

In the above, we shall use an effective length of the orifice which is the weighted average of the length of the trapezoidal weir and the dam length, as shown in Figure 3.

$$\text{Effective length } L_{\text{eff}} = \frac{\sum_{i=1}^M W_i X_i}{\sum_{i=1}^M W_i} \quad (\text{LCS.6})$$

$$= \frac{(38.118)(1135) + (114)(590)}{38.118 + 114} \\ = 726.56 \text{ feet} \quad (\text{LCS.7})$$

Thus, we shall consider the orifice as a rectangle with length 726.56 feet and height 152.118 feet.

Therefore,

$$Q_{\max}^{(1)} = 7.27 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \quad (\text{LCS.8})$$

From (AI-1),

$$Q_{\max}^{(2)} = \frac{1}{\sqrt{3}} Q_{\max}^{(1)} = 4.19 \times 10^6 \frac{\text{ft}^3}{\text{sec}}$$

To calculate Q_{av} through the use of equation (AI-7), the following procedure is adopted.

From (AI-6),

$$\text{Actual time of emptying} = T_{\text{actual}} = \frac{3A}{L\sqrt{2gH}}$$

Here,

$$A = 64.99 \times 10^6 \text{ ft}^2$$

$$H = 152.118 \text{ ft}$$

Therefore,

$$T_{\text{actual}} = 2702.5 \text{ seconds}$$

$$\approx 45 \text{ minutes}$$

Let us compare this with the effective time of emptying. Consider (AI-8) which is,

$$h = \left(\sqrt{H} - \frac{tLH\sqrt{2g}}{3A} \right)^2$$

With this equation we can generate graph LCS3, which is a graph of the height of water against time. From LCS3, we can make an approximate determination of the effective time necessary to empty all the water from behind the dam.

This, from LCS3, is

$$T_{\text{eff}} = 42 \text{ minutes} \quad (\text{LCS.9})$$

Therefore, from (AI-7),

$$Q_{av} = \frac{V}{T}$$

Here,

$$V = \begin{array}{l} \text{Volume of} \\ \text{Lower Crystal} + \text{Flooding volume of} \\ \text{Springs Lake} \quad \text{the weir} \end{array}$$

$$\text{Volume of lake} = 2.35 \times 10^9 \text{ ft}^3 \quad (\text{LCS.10})$$

$$\begin{aligned} \text{Flooding volume} \\ \text{of water over} \\ \text{the weir} &= 64.9 \times 38.11 \times 10^6 \\ &= 2.47 \times 10^9 \text{ ft}^3 \end{aligned} \quad (\text{LCS.11})$$

Therefore,

$$V = 4.82 \times 10^9 \text{ ft}^3, \text{ and}$$

$$Q_{av} = 1.91 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \quad (\text{LCS.12})$$

We note that in the above calculation we have assumed that the height over the weir is the same as considered over the entire surface of the lake.

Calculations of the Time of First Large Wave and the Amount of Instantaneous Release of Water

From (AI-1),

$$\tilde{t} \approx \sqrt{\frac{H}{g}} = 2.18 \text{ sec} \quad (\text{LCS.13})$$

Therefore, it is apparent that a large amount of water is released almost immediately after the dam failure. To find out the amount of water released in this time, we must first calculate the height of water still behind the orifice at the time $\tilde{t}$.

From (AI-8), with $t = \tilde{t}$,

$$h = 151.88 \text{ ft.} \quad (\text{LCS.14})$$

Therefore,

$$\Delta h \text{ (from } t = 0 \text{ to } t = 2.18 \text{ sec)} = 0.24 \text{ ft.}$$

The amount of water which flows out of the orifice in $\tilde{t}$ is given by

$$\begin{aligned} \Delta V &\approx A \Delta h \\ &= 1.598 \times 10^7 \text{ ft}^3 \end{aligned} \quad (\text{LCS.15})$$

Estimations of the Dimensions of Constant Flow Channel

First Reach

I. Assumptions Made

1. Since the flow channel is well defined by the natural topography of the area for the first reach (i.e., by high elevations on both sides of the channel), it can be assumed that the effects of divergent outflow of water or of the lateral sloping above 4000 feet are insignificant. Any divergent flow will be returned to the channel because of the steep inward slopes of the two sides of the channel (see graphs LCD1a, 1b and Figure 4).

2. Assume that the constant flow channel configuration can be approximated by a trapezoid with dimensions shown in Figure 4.

II. Determination of Parameters B_1 , B_2 , C, A, R

1. B

$$B_1 = B_2 = \frac{y_n}{\sin 71^\circ}$$

2. C

$$\begin{aligned} C &= B + 2B_2 \cos 71^\circ \\ &= 250 + 2y_n \cot 71^\circ \end{aligned}$$

3. A

$$\begin{aligned} A &= \frac{C + B}{2} y_n \\ &= \frac{1}{2} \{ (250 + 2y_n \cot 71^\circ) + 250 \} y_n \end{aligned}$$

4. P

$$\begin{aligned} P &= B_1 + B + B_2 \\ &= 2B_1 + B \\ &= \frac{2y_n}{\sin 71^\circ} + 250 \end{aligned}$$

5. R

$$R = \frac{A}{P}$$

$$= \frac{\frac{1}{2} ((250 + 2y_n \cot 71^\circ) + 250) y_n}{\frac{2y_n}{\sin 71^\circ} + 250}$$

III. Values of the Parameters for Various Water Heights

$$\sin 71^\circ = 0.9455 \quad \cot 71^\circ = 0.3443 \quad \sqrt{S} = 0.1118$$

y_n (ft)	$A \times 10^{-4}$ (ft ²)	R. (ft)
0	0	0
4	.1005	3.90
8	.2022	7.57
12	.3049	11.07
16	.4088	14.40
20	.5137	17.57
24	.6198	20.60
28	.7269	23.51
32	.8352	26.29
36	.9446	28.96
40	1.0551	31.53
44	1.1667	34.00
48	1.2793	36.39
52	1.3931	38.69
56	1.5080	40.92
60	1.6239	43.08
64	1.7410	45.17
68	1.8592	47.20
72	1.9780	49.17
76	2.0989	51.09
80	2.2204	52.96
84	2.3429	54.78
88	2.4666	56.55

y_n (ft)	$A \times 10^{-4}$ (ft ²)	R (ft)
92	2.5914	58.28
96	2.7173	59.97
100	2.8443	61.62
104	2.9724	63.24
108	3/1016	64.826
112	3.2319	66.375
116	3.3633	67.894
120	3.4958	69.384
124	3.6294	70.846
128	3.7641	72.281
132	3.8999	73.692
136	4.0368	75.079
140	4.1748	76.443
144	4.3139	77.785
148	4.4542	79.106

IV. From Manning's formula given by (A2-1), we can calculate the Q values for various water heights, thus enabling us to make a Q vs. Y_n graph. From Graph LCS4 we see that the normal depth of the first reach, using a roughness coefficient of 0.07, is approximately 147 feet. Also, the flood plain for the first reach is approximately 1000 feet wide and 10,000 feet long.

Normal Water Depth for Second Reach (10,000 - 20,000 feet)

I. Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the second reach by a rectangle with dimensions determined at the middle of the second reach, i.e., 16,000 feet (see graphs 1b and 1c and Figure 5).

2. Assume that an allowance of 30°, from the lines normal to the small opening at the end of the first reach (10,000 feet) will account for the

divergent flow of water out of the orifice (see graphs LCS1b and 1c and figure 1). Figure 5 shows the actual dimensions of the constant flow channel.

Determinations of Parameters B_1 , B_2 , A , R When $B \approx 8,100$ Feet and Normal

Depth = y_n

1. B

$$B_1 = B_2 = y_n$$

2. A

$$A = B y_n = 8100 y_n$$

3. R

$$R = \frac{A}{P}$$

$$= \frac{B y_n}{B + 2 y_n}$$

But $y_n \ll B$.

Therefore, $R \approx y_n$

We note that the error introduced by using a rectangular flow channel configuration rather than a trapezoidal configuration is small since the base of the flow channel is very much greater than the normal water depth.

Values of the Parameters for Various Water Heights

<u>y_n</u> <u>(ft)</u>	<u>$A \times 10^{-4}$</u> <u>(ft²)</u>	<u>R</u> <u>(ft)</u>
0	0	0
4	3.24	4
8	6.48	8
12	9.72	12
16	12.96	16
20	16.20	20
24	19.44	24

y_n <u>(ft)</u>	$A \times 10^{-4}$ <u>(ft²)</u>	R <u>(ft)</u>
28	22.68	28
32	25.92	32
34	27.54	34

From Manning's formula, given by (A2-1), we can calculate Q values for various water heights, thus enabling us to make a Q vs. y_n graph. From Graph LCS5, we see that the normal depth of the second reach, using a roughness coefficient of 0.09, is approximately 22.6 feet, while the flood plain itself is 10,000 feet long and 8,100 feet wide.

San Andreas Lake and Lower Crystal Springs Reservoir

The flood plain generated, assuming the combined failure of these two reservoirs, extended two reaches before flowing into the San Francisco Bay. No credit was taken for deaths or damage in the first reach since the confined nature of the flow through San Mateo Creek left some doubt as to how large a portion of the population would be affected in the first reach. Estimates of the expected deaths and amount of damage are:

1st reach:

Number of expected deaths:	No deaths assumed
Damage to homes:	No damage assumed

2nd reach:

Expected number of deaths:	Daytime ~ 20,600 Nighttime ~ 32,900
Damage to homes:	\$114,030,000 7,700 homes

Totals:

Expected number of deaths:	Daytime ~ 20,600 Nighttime ~ 32,900
Damage to homes:	7,700 homes \$114,030,000

Figure 1. San Andreas Dam.

Figure 2. Flow Channel Configuration @ 6000 Ft.

Figure 3. Lower Crystal Springs Dam.

Figures 4 and 5. Flow Channel Configurations.

Graph SAD1: Determination of Ground Level.

Graph SAD2a: Downstream Topography.

Graph SAD2b: Downstream Topography.

Graph SAD3: Actual Downstream Slope of Bottom of Flow Channel.

Graph SAD4: Water Height vs. Time.

Graph SAD5: Volume Rate of Flow vs. Normal Depth for Slope of 0.0097.

Graph SAD6: Actual Normal Depth @ Various Places Downstream of Dam.

LOWER CRYSTAL SPRINGS RESERVOIR

HORIZONTAL SCALE:

1" = 2000 FT.

0" or 0' DOWNSTREAM
FROM DAM

1" or 2000' DOWNSTREAM
FROM DAM

2" OR 4000' DOWNSTREAM
FROM DAM

3" or 6000' DOWNSTREAM
FROM DAM

Graph LCS1a: Downstream Topography.

LOWER CRYSTAL SPRINGS RESERVOIR

HORIZ. SCALE 1" = 2000 FT.

4" or 8000' DOWNSTREAM
FROM DAM

5" or 10,000' DOWNSTREAM
FROM DAM

6" or 12,000' DOWNSTREAM
FROM DAM

7" or 14,000' DOWNSTREAM
FROM DAM

Graph LCS1b: Downstream Topography.

Graph LCS1c: Downstream Topography.

Graph LCS2: Downstream Slope of Topography.

Graph LCS3: Water Height vs. Time.

Graph LCS4: Volume Rate of Flow vs. Normal Depth.

Graph LCS5: Volume Rate of Flow vs. Normal Depth.

E: LCSFP-1
Flood Plain Below the San Andreas and the Lower Crystal Springs Reservoirs

LCSFP-2

Census Tract Map – Flood Plain Below the San Andreas and Lower Crystal Springs Reservoirs

Stone Canyon Dam

(Details below refer to topographical map)

* Dam Number	6 - 25
Name of Owner	City of Los Angeles
County	Los Angeles
Stream	Stone Canyon Creek
Location	{ Section 09 Township 01S Range 15W Baseline and/or Meridian SB
Type	Earth
Storage Capacity	4.517172×10^8 cubic feet
Drainage Area	1.36 square miles
Reservoir Area	6.0982×10^6 square feet
Crest Elevation	875 feet
Crest Length	1150 feet
Permanent Spillway	847 feet
Height of Dam	185 feet
Crest Thickness	30 feet
Dam Volume	3,346,402 cubic yards
Year of Completion	1924

* As assigned by Department of Water Resources

Calculation of $Q_{\max}$ and Q_{av} under the Assumption of Complete and Instantaneous Failure of Dam:

In the case of Stone Canyon Dam, we shall assume that the compacted earth completely erodes immediately after a breach in the concrete occurs i.e., instantaneous complete dam failure.

From (A1-1),

$$Q_{\max}^{(1)} = \frac{2}{3} L \sqrt{2gH^3}$$

Here,

$$L = 950 \text{ ft.}$$

$$H = 157 \text{ ft.}$$

Therefore,

$$Q_{\max}^{(1)} = 9.96 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \quad (\text{SC.1})$$

The length, L , in the above calculation is an effective length for the dam since the opening narrows down from the beginning concrete portion of the dam to the end of the compacted earth.

From (A1-2),

$$\begin{aligned} Q_{\max}^{(2)} &= \frac{1}{\sqrt{3}} Q_{\max}^{(1)} \\ &= 5.76 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \end{aligned} \quad (\text{SC.2})$$

To calculate Q_{av} through the use of Equation (A1-7), the following procedure is adopted.

From (A1-6),

$$\text{Actual time of emptying} = T_{\text{actual}} = 3A/L\sqrt{2gH}$$

Here,

$$A = 6.098 \times 10^6 \text{ ft}^2$$

Therefore,

$$\begin{aligned} T_{\text{actual}} &= 192.10 \text{ secs} \\ &\simeq 3.2 \text{ minutes} \end{aligned} \quad (\text{SC.3})$$

Let us compare this with the effective time of emptying. Consider (A1-8) which is,

$$h = \left(\sqrt{H} - \frac{t_{\text{LH}} \sqrt{2g}}{3A} \right)^2$$

With this equation we can generate SCD3 which is a graph of the height of water against time. From SCD3, we can make an approximate determination of the effective time necessary to empty all the water from behind the dam. This, from SCD3, is

$$T_{\text{eff}} = 190 \text{ seconds}, \quad (\text{SC.4})$$

almost the same as the actual time.

Also, from (A1-7),

$$Q_{\text{av}} = \frac{V}{T}$$

Here,

$$V = 451.71 \times 10^6 \text{ ft}^3, \quad T = 190 \text{ secs.}$$

Therefore,

$$Q_{\text{av}} = 2.37 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \quad (\text{SC.5})$$

Calculation of the Time of First Large Wave and the Amount of Instantaneous Release of Water.

From (A1-9),

$$\tilde{t} \approx \sqrt{\frac{H}{g}} = 2.21 \text{ secs.} \quad (\text{SC.6})$$

Therefore, it is apparent that a large amount of water is released almost immediately after the dam failure.

To find out the amount of water released in this time, we must first calculate the height of water still behind the orifice at the time $\tilde{t}$.

From (A1-8), with $t = \tilde{t}$,

$$h = 153.4 \text{ ft.} \quad (\text{SC.7})$$

Therefore,

$$\Delta h(\text{from } t = 0 \text{ to } t = 2.21 \text{ secs}) = 3.6 \text{ ft.}$$

The amount of water which flows out of the orifice in $\tilde{t}$ is given by

$$\Delta v \approx A \Delta h = 21.94 \times 10^6 \text{ ft}^3. \quad (\text{SC.8})$$

Estimation of Dimensions of Constant Flow Channel

First Reach

I. Assumptions Made

1. Since the flow channel is well defined by the natural topography of the area i.e., high elevations on both sides of the channel, it is assumed that the effects of divergent water flow out of the dam or lateral sloping above 8,000 feet are unimportant. Any divergent flow will be returned to the channel because of the steep inward slopes of the two sides of the channel (see Graph SCD1a)

2. It can be assumed that the constant flow channel configuration can be fairly accurately represented by a trapezoid with dimensions shown in Figure 1.

II. Determination of Parameters B_1 , B_2 , C, A, R:

1. B_1 & B_2

$$B_1 = B_2 = (y_n / \sin 63^\circ)$$

2. C

$$\begin{aligned} C &= B + 2B_2 \cos 63^\circ \\ &= 260 + 2y_n \cot 63^\circ \end{aligned}$$

3. A

$$\begin{aligned} A &= \frac{(C + B)}{2} y_n \\ &= \frac{1}{2} \left\{ (260 + 2y_n \cot 63^\circ) + 260 \right\} y_n \end{aligned}$$

4. P

$$\begin{aligned} P &= B_1 + B + B_2 \\ &= 2B_1 + B \\ &= \left(2y_n / \sin 63^\circ \right) + 260 \end{aligned}$$

5. R

$$R = A/P$$

$$= \frac{\frac{1}{2} \left\{ (260 + 2y_n \cot 63^\circ) + 260 \right\} y_n}{\left(\frac{2y_n}{\sin 63^\circ} \right) + 260}$$

III. Values of Parameters for Various Water Heights

y_n (ft)	Slope = 0.026 $A \times 10^{-4}$ (ft ²)	B = 260 ft R (ft)
4	0.10482	3.8968
8	0.21126	7.6006
12	0.31934	11.1290
16	0.42906	14.4990
20	0.54049	17.7240
24	0.65338	20.8170
28	0.76798	23.7880
32	0.88422	26.6470
36	1.0021	29.4040
40	1.1216	32.0650
44	1.2427	34.6390
48	1.3655	37.1320
52	1.4899	39.5490
56	1.6159	41.8960
60	1.7436	44.1780
64	1.8729	46.3980
68	2.0038	48.5620
72	2.1364	50.6720
76	2.2706	52.7310
80	2.4064	54.7440
84	2.5439	56.7130
88	2.6829	58.6400

y_n (ft)	$A \times 10^{-4}$ (ft ²)	R (ft)
92	2.8237	60.5280
96	2.9660	62.3790
100	3.1100	64.1950
104	3.2556	65.9780
108	3.4029	67.7290
112	3.5517	69.4510
116	3.7023	71.1450
120	3.8544	72.8130
124	4.0082	74.4550

IV. From Manning's formula, given by (A2-1), we can calculate Q-values for various water depths, thus enabling us to make a Q vs. y_n graph. From Graph SCD4, we see that corresponding to $n = 0.05, 0.07$ and 0.09 values and for a Q_{av} as given by (SC.5),

$$\left. \begin{aligned} y_n^{(0.05)} &= 100.0 \text{ ft.} \\ y_n^{(0.07)} &= 122.5 \text{ ft.} \\ y_n^{(0.09)} &\approx 143 \text{ ft.} \end{aligned} \right\} \quad (\text{SC.9})$$

where the average base width is approximately 260 feet. The normal depths given in (SC.9) are for the first reach (0-8000 ft.)

Normal Water Depth for Second Reach

(8,000 - 16,000 ft.)

Estimation of Dimensions of Constant Flow Channel

Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the second reach by a trapezoid with dimensions determined at the middle of second reach i.e., at 12,000 ft.

2. Assume that an allowance of 30° , measured from both sides of the flow channel at 8,000 ft. downstream will account for the natural divergent water flow out of the confined channel.

3. Assume that an additional allowance of 15° added on to the right side and subtracted off from the left will account for the lateral slope of 0.016 to the right. (See Graph SCD1b and Figure 2.)

4. Assume that the water is confined by the ridge 1,000 ft. to the right of the center of the dam, from 12,000 to 16,000 ft. (See Graph SCD1b and Figure 2.)

Actual dimensions of trapezoidal flow channel configuration at 12,000 ft. downstream are shown in Figure 2.

Determination of Parameters B_1 , B_2 , C, A, R for the Second Reach

1. B

$$B_1 = B_2 = y_n / \sin 10^\circ$$

2. C

$$C = B + 2B_2 \cos 10^\circ = \{6000 + 2y_n \cot 10^\circ\}$$

3. A

$$A = \frac{C+B}{2} y_n$$

4. P

$$\begin{aligned} P &= B_1 + B + B_2 \\ &= 2B_1 + B \\ &= \frac{2y_n}{\sin 10^\circ} + 6,000 \end{aligned}$$

5. R

$$R = (A/P)$$

Values of Parameters for Various Water Heights

y_n (ft)	Slope = 0.026 $A \times 10^{-4}$ (ft ²)	B = 6,000 ft. R (ft)
0	0	0
4	2.4091	3.9845
8	4.8363	7.9386
12	7.2817	11.8630
16	9.7452	15.7580
20	12.2270	19.6250
24	14.7270	23.4630
28	17.2450	27.2750
32	19.7810	31.0600
36	22.3350	34.8190
40	24.9070	38.5520
44	27.4980	42.2600
48	30.1070	45.9440
50	31.4180	47.7770

Values of Q for Various Water Heights and Roughness Coefficients

Graph SCD5 shows the values of Q for various water heights and roughness coefficients. The results for the second reach (8,000 - 16,000 ft.) are,

$$\left. \begin{array}{l} y_n^{(0.07)} = 17.2 \text{ ft.} \\ y_n^{(0.09)} = 20.0 \text{ ft.} \\ y_n^{(0.11)} = 22.6 \text{ ft.} \end{array} \right\} \quad (\text{SC.10})$$

The average base width for the second reach is approximately 6,000 ft.

Normal Water Depth for Third Reach

(16,000 - 24,000 ft.)

Estimation of Dimensions of Constant Flow Channel

Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the third reach by a trapezoid with dimensions determined at the middle of the third reach (at 20,000 ft.).
2. Assume that an allowance of 15° , measured from the left end of the flow channel at 16,000 ft. downstream, will account for the natural divergent water flow. (See Graph SCD1c and Figure 3).
3. Assume that on the right end of the flow channel the water is confined by a ridge 11,000 ft. to the right of the center of the dam, from 16,000 to 24,000 ft. downstream. (See Graph SCD1c).

Actual dimensions of trapezoidal flow channel configuration at 20,000 ft. downstream are shown in Figure 3.

Determination of Parameters B_1 , B_2 , C, A, R for the Third Reach

1. B

$$B_1 = B_2 = y_n / \sin 10^\circ$$

2. C

$$\begin{aligned} C &= B + 2B_2 \cos 10^\circ \\ &= \left| 11,100 + 2y_n \cot 10^\circ \right| \end{aligned}$$

3. A

$$A = \left(\frac{C+B}{2} \right) y_n$$

4. P

$$\begin{aligned}
 P &= B_1 + B + B_2 \\
 &= 2B_1 + B \\
 &= \left(\frac{2y_n}{\sin 10^\circ} \right) + 11,100
 \end{aligned}$$

5. R

$$R = A/P$$

Values of Parameters for Various Water Heights

y_n (ft)	Slope = 0.0187 $A \times 10^{-4}$ (ft ²)	B = 11,100 ft.	R (ft)
0	0		0
4	4.4491		3.9916
8	8.9163		7.9666
12	13.4020		11.9250
16	17.9050		15.8670
20	22.4270		19.7940
24	26.9670		23.7040
28	31.5250		27.5990
30	33.8100		29.5400

Values of Q for Various Water Heights and Roughness Coefficients for the

Third Reach

Graph SCD6 shows the values of Q for various water heights and roughness coefficients. The results for the third reach (16,000 to 24,000 ft.) are,

$$\left. \begin{aligned}
 y_n^{(0.07)} &= 13.4 \text{ ft.} \\
 y_n^{(0.09)} &= 15.6 \text{ ft.} \\
 y_n^{(0.11)} &= 17.6 \text{ ft.}
 \end{aligned} \right\} \quad (\text{SC.11})$$

The average base width for the third reach is approximately 11,100 ft.

Normal Water Depth for Fourth Reach

(24,000 - 32,000 ft.)

Estimation of Dimensions of Constant Flow Channel

Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the fourth reach by a trapezoid with dimensions determined at the middle of the fourth reach i.e., 28,000 ft.
2. Assume that we may continue the 15° allowance, measured from the left end of the flow channel at 14,000 ft. downstream, through the fourth reach to account for natural divergent water flow. (See Graph SCD1d and Figure 4.)
3. Assume that on the right end of the flow channel, water is confined by a ridge 11,000 ft. to the right of the center of the dam until 26,000 ft. downstream. From here the channel curves to the right at roughly a 45° angle until 28,000 ft. We shall assume a 15° allowance for the natural divergence of water from this point, as shown in Figure 4. (See Graph SCD1d.)

Actual dimensions of trapezoidal flow channel configuration at 28,000 ft. downstream are shown in Figure 4.

Determination of Parameters B_1, B_2, C, A, R for the Fourth Reach

1. B

$$B_1 = B_2 = y_n / \sin 10^\circ$$

2. C

$$\begin{aligned} C &= B + 2B_2 \cos 10^\circ \\ &= 14,625 + 2y_n \cot 10^\circ \end{aligned}$$

3. P

$$\begin{aligned} P &= B_1 + B + B_2 \\ &= 2B_1 + B \\ &= \left(\frac{2y_n}{\sin 10^\circ} \right) + 14,625 \end{aligned}$$

4. A

$$A = \left(\frac{C+B}{2} \right) y_n$$

5. R

$$R = (A/P)$$

Values of Parameters for Various Water Heights

y_n (ft)	Slope = 0.0114 $A \times 10^{-4}$ (ft ²)	B = 14,625 ft. R (ft)
0	0	0
4	5.8591	3.9936
8	11.7360	7.9746
12	17.6320	11.9430
16	23.5450	15.8990
20	29.4770	19.8430
24	35.4270	23.7740
28	41.3950	27.6930
30	44.3850	29.6470

Values of Q for Various Water Heights and Roughness Coefficients

Graph SCD7 shows the value of Q for various heights and roughness coefficients. The results for the fourth reach (24,000 - 32,000 ft.) are,

$$\left. \begin{array}{l} y_n^{(0.07)} = 13.1 \text{ ft.} \\ y_n^{(0.09)} = 15.2 \text{ ft.} \\ y_n^{(0.11)} = 17.1 \text{ ft.} \end{array} \right\} \quad (\text{SC.12})$$

The average base width for the fourth reach is approximately 14,625 ft.

Normal Water Depth for the Fifth Reach

(32,000 - 40,000 ft.)

Estimation of Dimensions of Constant Flow Channel

Assumption Made

1. Assume that we can approximate the constant flow channel configurations for the fifth reach by a trapezoid with dimensions determined at the middle of the fifth reach i.e., 36,000 ft.

2. Assume that we may continue the 15° allowance, measured from the left end of the flow channel at 16,000 ft. downstream, through the fifth reach to account for natural divergent water flow. See Graphs SCD 1e and 1f and Figure 5.

3. Assume that we may continue the 15° allowance, measured from the right end of the flow channel at 28,000 ft. downstream, through the fifth reach to account for the natural divergent water flow. See Graphs SCD 1e, SCD 1f and Figure 5.

Actual dimensions of trapezoidal flow channel configurations at 36,000 ft. downstream are shown in Figure 5.

Determination of Parameters B_1, B_2, C, A, R

1. B

$$B_1 = B_2 = y_n / \sin 10^\circ$$

2. C

$$\begin{aligned} C &= B + 2B_2 \cos 10^\circ \\ &= 19,200 + 2y_n \cot 10^\circ \end{aligned}$$

3. P

$$\begin{aligned} P &= B_1 + B + B_2 \\ &= 2B_1 + B \\ &= \left(\frac{2y_n}{\sin 10^\circ} \right) + 19,200 \end{aligned}$$

4. A

$$A = \left(\frac{C+B}{2} \right) y_n$$

5. R

$$R = (A/P)$$

Values of Parameters for Various Water Heights

Slope = 0.0042		B = 19,200 ft.
y_n (ft)	$A \times 10^{-4}$ (ft ²)	R (ft)
0	0	0
4	7.6891	3.9957
8	15.3960	7.9806
12	23.1220	11.9560
16	30.8650	15.9230
20	38.6270	19.8800
24	46.4070	23.8270
28	54.2050	27.7650
30	58.1100	29.7310

Value of Q for Various Water Heights and Roughness Coefficients

Graph SCD8 shows the values of Q for various water heights and roughness coefficients. The results for the fifth reach (32,000 - 40,000 ft.) are,

$$\left. \begin{array}{l} y_n^{(0.07)} = 15.0 \text{ ft.} \\ y_n^{(0.09)} = 17.4 \text{ ft.} \\ y_n^{(0.11)} = 19.6 \text{ ft.} \end{array} \right\} \quad (\text{SC.13})$$

The average base width for the fifth reach is approximately 19,200 ft.

Expected Number of Deaths and Damage from a Failure of the Stone Canyon Dam.

Computation of the flood plain below the Stone Canyon Reservoir extended for five reaches to the hills just southeast of Marina del Rey. The flood plain is plotted through Westwood, West Los Angeles, Culver City, Santa Monica and the Marina area. The downstream effects of such a flood are detailed below.

1st reach: Narrow canyon area--No deaths assumed

2nd reach:

Expected number of deaths

Daytime 14,000

Nighttime: 20,000

Amount of damage

Single-unit homes destroyed: 2,663

Value: \$62,508,800

3rd reach:

Expected number of deaths

Daytime: 30,888

Nighttime: 54,960

Amount of damage

Single-unit homes destroyed: 8,351

Value: \$133,103,550

4th reach:

Expected number of deaths

Daytime: 36,126

Nighttime: 59,320

Amount of damage

Single-unit homes destroyed: 13,084

Value: \$194,786,400

5th reach:

Expected number of deaths

Daytime: 44,457

Nighttime: 73,000

Amount of damage

Single-unit homes destroyed: 12,022

Value: \$139,418,100

Total:

Expected number of deaths

Daytime: 125,471

Nighttime: 207,280

Amount of damage

Single-unit homes destroyed: 36,120

Value: \$529,816,850

Figure 1. Flow Channel Configurations @ 4,000 Ft. Downstream.

Figure 2. Actual Dimensions of Trapezoid:
Flow Channel Configuration @ 12,000 Ft. Downstream.

Figure 3. Actual Dimensions of Trapezoid:
Flow Channel Configurations @ 20,000 Ft. Downstream.

Figure 4. Actual Dimensions of Trapezoid:
Flow Channel Configuration @ 28,000 Ft. Downstream.

Figure 5. Actual Dimensions of Trapezoid:
Flow Channel Configuration @ 36,000 Ft. Downstream.

Graph SCD1a: Downstream Topography.

HORIZONTAL SCALE: 1" = 2000 FT.
VERTICAL SCALE: ELEVATION IN FEET

Graph SCD1b: Downstream Topography.

HORIZONTAL SCALE: 1" = 2000 FT.
VERTICAL SCALE: ELEVATION IN FEET

Graph SCD1C: Downstream Topography.

HORIZONTAL SCALE: 1" = 2000 FT.
VERTICAL SCALE: ELEVATION IN FEET

Graph SCD1d: Downstream Topography.

HORIZONTAL SCALE: 1" = 2000 FT.
VERTICAL SCALE: ELEVATION IN FEET

Graph SCD1e: Downstream Topography.

Graph SCD1f: Downstream Topography.

Graph SCD2: Graph of Downstream Slope.

Graph SCD3: Water Height vs. Time.

Graph SCD4: Volume Rate of Flow vs. Normal Depth.

Graph SCD5: Volume Rate of Flow vs. Normal Depth.

Graph SCD6: Volume Rate of Flow vs. Normal Depth.

Graph SCD7: Volume Rate of Flow vs. Normal Depth.

Graph SCD8: Volume Rate of Flow vs. Normal Depth.

SCFP-1
Flood Plain Below the Stone Canyon Reservoir

SCFP-2
Census Tract Map – Flood Plain Below the Stone Canyon Reservoir

Encino Dam

(Details below refer to topographical map)

* Dam Number	6 - 7
Name of Owner	City of Los Angeles
County	Los Angeles
Stream	Encino Creek
Location	{ Section 24 Township 01N Range 16W Baseline and/or Meridian SB
Type	Earth
Storage Capacity	4.48668×10^8 cubic feet
Drainage Area	1.40 square miles
Reservoir Area	7.10×10^6 square feet
Crest Elevation	1088 feet
Crest Length	1850 feet
Permanent Spillway	1075 feet
Height of Dam	168 feet
Crest Thickness	30 feet
Dam Volume	3,158,406 cubic yards
Year of Completion	1924

* As assigned by Department of Water Resources

Calculation of $Q_{\max}$ and Q_{av} Under the Assumption of Complete and Instantaneous Failure of Dam

In the case of Encino Dam, we shall assume that the compacted earth completely erodes immediately after a breach in the concrete occurs, i.e., instantaneous complete dam failure.

From (A1-1),

$$Q_{\max}^{(1)} = \frac{2}{3} L \sqrt{2gH^3}$$

Here,

$$L = 1800 \text{ feet}$$

$$H = 155 \text{ feet}$$

Therefore,

$$Q_{\max}^{(1)} = 18.52 \times 10^6 \frac{\text{ft}^2}{\text{sec}} \quad (\text{EN.1})$$

From (A1-2),

$$Q_{\max}^{(2)} = \frac{1}{\sqrt{3}} Q_{\max}^{(1)} = 10.69 \times 10^6 \frac{\text{ft}^2}{\text{sec}} \quad (\text{EN.2})$$

To calculate Q_{av} through the use of equation (A1-7), the following procedure is adopted:

From (A1-6),

$$\text{Actual time of emptying} = T_{\text{actual}} = 3A / (L \sqrt{2gH})$$

Here,

$$A = 7.1 \times 10^6 \text{ ft}^2$$

Therefore,

$$\begin{aligned} T_{\text{actual}} &= 118.8 \text{ secs} \\ &\approx 1.980 \text{ minutes} \end{aligned} \quad (\text{EN.3})$$

Let us compare this with the effective time of emptying. Consider (A1-8) which is

$$h = \left(\sqrt{H} - \frac{tLH\sqrt{2g}}{3A} \right)^2$$

With this equation we can generate ED 3 which is a graph of the height of water against time. From ED 3 we can make an approximate determination of the effective time necessary to empty all the water from behind the dam.

This, from ED 3, is

$$\begin{aligned} T_{\text{eff}} &= 115 \text{ secs} \\ &\approx 2 \text{ minutes} \end{aligned} \quad (\text{EN.4})$$

Also, from (A1-7),

$$Q_{\text{av}} = \frac{V}{T}$$

Here,

$$V = 449.23 \times 10^6 \text{ ft}^3, \quad T = 115 \text{ secs}$$

Therefore,

$$Q_{\text{av}} = 3.906 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \quad (\text{EN.5})$$

Calculations of the Time of First Large Wave and the Amount of Instantaneous Release of Water

From (A1-9),

$$\tilde{t} \approx \sqrt{\frac{H}{g}} = 2.201 \text{ sec} \quad (\text{EN.6})$$

Therefore, it is apparent that a large amount of water is released almost immediately after the dam failure. To find out the amount of water released in this time, we must first calculate the height of water still behind the orifice at the time $\tilde{t}$.

From (A1-8), with $t = \tilde{t}$,

$$h = 149.31 \text{ ft} \quad (\text{EN.7})$$

Therefore,

$$\Delta h \text{ (from } t = 0 \text{ to } t = 2.01 \text{ sec)} = 5.69 \text{ ft}$$

Under the assumption that the immediate upstream topography and the water catchment can be geometrically and equivalently approximated by a

solid prismatic bowl, the amount of water which flows out of the orifice in $\tilde{t}$ is given by

$$\Delta V \approx A\Delta h = 4.04 \times 10^7 \text{ ft}^3 \quad (\text{EN.8})$$

Estimation of the Dimensions of Constant Flow Channel for the First and Second Reaches of the Encino Dam Downstream

An arbitrary choice of two reaches - 0 to 8000 feet, 8000 to 14,000 feet - has been made. The constant flow channel dimensions for these two reaches are set using the details at 5000 feet and 11,000 feet, respectively. It is thought that this representation is a sufficiently good approximation for the purposes of the flood plain calculations. With reference to graphs ED 1a, ED 1b and the flood plain map, we could explain the flow channel as follows:

As the water emerges from the dam opening, its flow is confined on the right and left sides by high ridges. This confinement lasts only until the water has moved some 2000 feet downstream. From this point, the water is virtually unconfined. From 2000 feet to 4000 feet, equal lateral slopes on both sides of the flow center line are accounted for by allowing a 30° divergence from both ends of the orifice starting at 2000 feet. This allowance accounts for the normal divergence of the flow as well. From 4000 feet downstream until the end of the flood plain, the only lateral sloping is to the left as viewed from facing the dam. This sloping is accounted for by making the right end of the flood plain converge by 15° while holding the left end constant. This condition prevails until the end of the flow channel (14,000 feet). Figure EN 1a and EN 1b show these above configurations.

Values of Q for Various Water Heights and Roughness Coefficients

Graphs ED 4a and 4b show the values of Q for various water heights, roughness coefficients, and reaches. From these graphs, for a roughness coefficient of $n = 0.09$, the results can be described as follows:

First reach:

Width = 4625 feet

Average water depth = 35.5 feet

Second reach:

Width = 9750 feet

Average water depth = 23.1 feet

Expected Number of Deaths and Damage from a failure of the Encino Dam.

Computation of the flood plain below the Encino Reservoir extended for two 7,000 foot reaches into the Sepulveda Flood Control Basin. Due to the fairly great height of the water at the end of the second reach, 23.1 feet, the flow is assumed to travel up a slightly adverse slope for about 2,000 feet beyond the Sepulveda Basin to Vanowen Street. The consequences for each reach are as follows:

1st reach: No deaths assumed in area immediately before dam

Expected number of deaths

Daytime: 2,900

Nighttime: 4,450

Amount of damage

Single-unit homes destroyed: 865

Value: \$17,738,000

2nd reach:

Expected number of deaths

Daytime: 8,330

Nighttime: 13,620

Amount of damage

Single-unit homes destroyed: 3,428

Value: \$40,352,000

Total:

Expected number of deaths

Daytime: 11,230

Nighttime: 18,070

Amount of damage

Single-unit homes destroyed: 4,300

Value: \$50,909,000

TRAPEZOID @ 5000 FT

TRAPEZOID @ 11,000 FT

Figures EN1a and 1b: Flow Channel Configurations.

HORIZONTAL SCALE: 1" = 2000 FT.
 VERTICAL SCALE: ELEVATION IN FEET

Graph ED1a: Downstream Topography.

HORIZONTAL SCALE: 1" = 2000 FT.
 VERTICAL SCALE: ELEVATION IN FEET

Graph ED1b: Downstream Topography.

Graph ED2: Slopes Downstream from Encino Dam.

Graph ED3: Water Height vs. Time.

Graph ED4a: Volume Rate of Flow vs. Normal Depth.

Graph ED4b: Volume Rate of Flow vs. Normal Depth.

B: EFP-1
Flood Plain Below the Encino Reservoir

San Pablo Dam

(Details below refer to topographical map)

* Dam Number	31 - 6
Name of Owner	East Bay M U District
County	Contra Costa
Stream	San Pablo Creek
Location	{ Section 12 Township 1N Range 4W Baseline and/or Meridian MD
Type	Hydraulic fill
Storage Capacity	1.881487×10^9 cubic feet
Drainage Area	32.000 square miles
Reservoir Area	37.2×10^6 square feet
Crest Elevation	329 feet
Crest Length	1250 feet
Permanent Spillway	315 feet
Effective Height of Dam (Ground Level @ 200 Feet)	129 feet
Crest Thickness	50 feet
Dam Volume	2,200,200 cubic yards
Year of Completion	1920

* As assigned by Department of Water Resources

Calculation of $Q_{\max}$ and Q_{av} under the assumption of complete and instantaneous failure of Dam:

In the case of San Pablo Dam, we shall assume that the compacted earth completely erodes immediately after a breach in the concrete occurs, i.e., instantaneous complete dam failure.

From (A1-1),

$$Q_{\max}^{(1)} = \frac{2}{3} L \sqrt{2 g H^3}$$

Here,

$$L = 1100 \text{ Ft}$$

$$H = 115 \text{ Ft}$$

Therefore,

$$Q_{\max}^{(1)} = 7.235 \times 10^6 \frac{\text{Ft}^3}{\text{Sec}} \quad (\text{SP.1})$$

From (A1-2),

$$\begin{aligned} Q_{\max}^{(2)} &= \frac{1}{\sqrt{3}} Q_{\max}^{(1)} \\ &= 4.177 \times 10^6 \frac{\text{Ft}^3}{\text{Sec}} \end{aligned} \quad (\text{SP.2})$$

To calculate Q_{av} through the use of equation (A1-7), the following procedure is adopted.

From (A1-6),

$$\text{Actual time of emptying} = T_{\text{actual}} = 3A / (L \sqrt{2gH})$$

Here,

$$A = 31.05 \times 10^6 \text{ ft}^2$$

Therefore,

$$T_{\text{actual}} = 987.07 \text{ secs.} \quad (\text{SP.3})$$

Let us compare this with the effective time of emptying. Consider (A1-8) which is,

$$H = \left(\sqrt{H} - \frac{t_{LH} \sqrt{2g}}{3A} \right)^2$$

With this equation, we can generate SPD 3, which is a graph of the height of water against time. From SPD3, we can make an approximate determination of the effective time necessary to empty all the water from behind the dam.

This, from SPD 3, is

$$T_{\text{eff}} = 987.07 \text{ secs.} \quad (\text{SP.4})$$

The T_{eff} turns out to be the same as T_{actual} in this case.

Also, from (A1-7),

$$Q_{\text{av}} = \frac{V}{T}$$

Here,

$$V = 1881.48 \times 10^6 \text{ ft}^3, \quad T = 987.07 \text{ secs}$$

Therefore,

$$Q_{\text{av}} = 1.906 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \quad (\text{SP.5})$$

Calculations of the time of first large wave and the amount of instantaneous release of water

From (A1-9),

$$\tilde{t} \approx \sqrt{\frac{H}{g}} = 1.896 \text{ sec} \quad (\text{SP.6})$$

Therefore, it is apparent that a large amount of water is released almost immediately after the dam failure.

To find out the amount of water released in this time, we must first calculate the height of the water still behind the orifice at the time $\tilde{t}$.

From (A1-8), with $t = \tilde{t}$,

$$h = 114.51 \text{ ft} \quad (\text{SP.7})$$

Therefore,

$$\Delta h \text{ (from } t = 0 \text{ to } t = 1.896 \text{ sec)} = 0.48 \text{ Ft}$$

The amount of water which flows out of the orifice in $\tilde{t}$ is given by

$$\Delta V \approx A \Delta h = 15.16 \times 10^6 \text{ ft}^3 \quad (\text{SP.8})$$

II. Calculations to determine if water head has sufficient energy to flow over bump in flow channel:

The following discussion is with reference to graph SPD 2. We will assume, for the moment, that an isosceles triangle can approximate the constant flow channel configuration downstream of the dam. Although this detracts somewhat from the accuracy of the calculations, it will make the mathematics of the problem tractable. (For the actual determination of the normal depth y_n , however, we shall return to a trapezoidal flow channel configuration.) Figures 1A and 1B show the isosceles triangle approximation for the constant flow channel configuration. With reference to these figures, we can determine the

parameters B_1 , B_2 , C , A and R when the normal depth = y_n

1. B_1 & B_2 :

$$B_1 = B_2 = y_n / \sin 36^\circ$$

2. C :

$$\frac{1}{2} C / B_1 = \cos 36^\circ$$

Therefore,

$$\frac{1}{2} C = B_1 \cos 36^\circ = y_n \cot 36^\circ.$$

$$C = 2 y_n \cot 36^\circ$$

3. A :

$$\text{Area} = 2 \left(\frac{1}{2} \right) \left(\frac{1}{2} \right) (C) y_n = \frac{1}{2} C y_n$$

$$= \left(y_n \cot 36^\circ \right) y_n = y_n^2 \cot 36^\circ$$

We note that Area (1) = $\left(\frac{1}{2} \right) \left(\frac{1}{2} \right) C y_n$ and $\text{Area}_{\text{Tot}} = 2 \text{Area (1)}$.

4. R :

$$R = A/P \left(\text{where } P = B_1 + B_2 = 2B_1 = 2 \frac{y_n}{\sin 36^\circ} \right)$$

Therefore,

$$R = \frac{1}{2} y_n \cos 36^\circ.$$

Now, from Manning's Equation we have

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

By setting $Q = Q_{ave}$ in the above equation, we can calculate y_n (the normal depth, as shown in Figure 2). The slope S immediately downstream of the dam, from Figure 2, is 0.0275.

Since $R = A/P$, we have

$$\begin{aligned} Q &= \frac{1.486}{n} A \left(\frac{A}{P} \right)^{2/3} S^{1/2} \\ &= \frac{1.486}{n} \frac{(y_n^2 \cot 36^\circ)^{5/3}}{(2y_n / \sin 36^\circ)^{2/3}} S^{1/2} \\ &= \frac{1.486}{n} y_n^{8/3} \frac{(\cot 36^\circ)^{5/3}}{(2/\sin 36^\circ)^{2/3}} S^{1/2} \end{aligned}$$

Therefore,

$$y_n = \left(\frac{Qn}{1.486 S^{1/2}} \right)^{3/8} \frac{(2/\sin 36^\circ)^{1/4}}{(\cot 36^\circ)^{5/8}}.$$

With an assumed roughness coefficient of $n = 0.09$, and with $Q = Q_{av}$, we get

$$\begin{aligned} y_n &= \left(\frac{1.906 \times 10^6 \times 0.09}{1.486 \times 0.16583} \right)^{3/8} \frac{(2/0.58779)^{1/4}}{(1.3764)^{5/8}} \\ &= 172.64 \text{ Ft} \end{aligned} \quad (\text{SP.9})$$

As the next step towards calculating the total head H , let us compare the vertical and normal depths. With reference to Figure 3, let h be

the distance from a chosen datum to a point x above the slope.

Now,

$$S = (\Delta y / \Delta x) = 0.0275$$

$$(\Delta y / \Delta x) = \cot \alpha = 0.0275$$

Therefore,

$$\alpha = 88^{\circ}.425 \text{ and } \beta = 1^{\circ}.575$$

The projection of the normal depth in the vertical direction is given by

$$(172.64) (x) = \cos 1^{\circ}.575$$

$$x = 172.57 \text{ Ft.} \quad (\text{SP.10})$$

The vertical and normal depths may therefore be considered the same.

Thus,

$$h = 172.64 + 105 = 277.64 \text{ Ft} \quad (\text{SP.11})$$

Letting B represent the average width of the flow channel just below the dam,

$$\text{Total head } H = \frac{Q^2}{2gy_n^2 B^2} + h \quad (\text{SP.12})$$

With datum at 100 Ft., and B = 1400 Ft. from topography,

$$\begin{aligned} H &= \frac{(1.906 \times 10^6)^2}{(2)(32)(172.64)^2 (1400)^2} + 277.64 \\ &= 278.611 \text{ Ft.} \end{aligned} \quad (\text{SP.13})$$

If we now take our datum at 205', then

$$h = 172.64, \text{ and}$$

$$H = 278.611 - 105$$

$$= 173.611 \text{ Ft.} \quad (\text{SP.14})$$

Therefore, at the point of inflexion, the energy head

$$\begin{aligned} H_{\text{hump}} &= 50 + 173.611 \\ &= 223.611 \text{ Ft.} \end{aligned} \quad (\text{SP.15})$$

Now, the minimum energy needed to pass the water head over the hump is given by

$$H_{\text{critical}} = \frac{2}{3} y_{\text{critical}} + \frac{q^2}{2gy_{\text{critical}}^2} \quad (\text{SP.16})$$

where,

$$y_{\text{critical}} = \sqrt[3]{\frac{q^2}{g}} \quad (\text{SP.17})$$

and,

$$q = \frac{Q}{B_{\text{hump}}} \quad (\text{SP.18})$$

B_{hump} , here is the average channel width at the hump. At 12000 Ft.

where the hump occurs, the average channel width is $\approx 1600 \text{ Ft.}$

Therefore,

$$\begin{aligned} q &= \frac{Q}{B_{\text{hump}}} = \frac{1.906 \times 10^6}{1600} \\ &= 1191.25 \frac{\text{Ft}^2}{\text{Sec}} \end{aligned} \quad (\text{SP.19})$$

Also,

$$y_{\text{critical}} = \sqrt[3]{\frac{(1191.25)^2}{32}} = 35.3 \text{ Ft.} \quad (\text{SP.20})$$

Then, from (SP.16),

$$H_{\text{critical}} = \left\{ \frac{2}{3} (35.3) + \frac{(1191.25)^2}{(2)(32)(35.3)^2} \right\}$$
$$= 41.32 \text{ Ft.} \quad (\text{SP.21})$$

At the hump we have an elevation of 127 ft.

Therefore,

$$\left. \begin{array}{l} \text{minimum energy} \\ \text{needed to pass} \\ \text{the water over} \\ \text{the hump} \end{array} \right\} \begin{array}{l} = 127 + 41.32 \\ = 168.32 \text{ Ft.} \end{array} \quad (\text{SP.22})$$

Since the actual energy line is greater than the minimum energy required, we see that the water does have enough energy to flow over the hump.

III. Estimation of the dimensions of constant flow channel:

Assumptions made:

1. Since the flow channel is well defined by the natural topography of the area (i.e., high elevations on both sides of the channel), we need not consider the effects of divergent water flow out of the dam or lateral sloping above 2000 Ft.
(See Graph SPD 1a @ 2000 Ft.)
2. Assume that we may approximate the constant flow channel configuration by a trapezoid with dimensions as shown in Figure 1a.

Determination of parameters B_1 , B_2 , C, A, R:

$$1. \quad B_1 = B_2 = y_n / \sin 42^\circ$$

$$2. \quad C = B + 2B_2 \cos 42^\circ$$

$$= 1000 + 2y_n \cot 42^\circ$$

$$3. \quad A = \frac{1}{2} (C + B) y_n$$

$$= \frac{1}{2} \{1000 + (1000 + 2y_n \cot 42^\circ)\} y_n$$

$$4. \quad P = B_1 + B + B_2 = 2B_1 + B = \frac{2y_n}{\sin 42^\circ} + 1000$$

$$5. \quad R = A/P$$

$$= \frac{\frac{1}{2} \{1000 + (1000 + 2y_n \cot 42^\circ)\} y_n}{\frac{2y_n}{\sin 42^\circ} + 1000}$$

Determination of y_n and y_{critical}

In order to ascertain whether the flow is supercritical or subcritical, we need to evaluate y_n and y_{critical} . From Manning's Equation:

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

Let $n = 0.07$, and set the slope $S = 0.0275$, that occurs just beneath the dam. Then consider the following cases in turn.

Case a): $y_n = 20 \text{ ft}$

$$Q = \frac{1.486}{0.07} (20,444.24) (19.29)^{2/3} (0.0275)^{1/2}$$

$$= .576 \times 10^6 \quad (\text{SP.23})$$

Case b): $y_n = 40 \text{ ft}$

$$\begin{aligned} Q &= \frac{1.486}{0.07} (41,776.96) (37.31)^{2/3} (.0275)^{1/2} \\ &= 1.64 \times 10^6 \end{aligned} \quad (\text{SP.24})$$

Case c): $y_n = 30 \text{ ft}$

$$\begin{aligned} Q &= \frac{1.486}{0.07} (30,999.54) (28.449)^{2/3} (0.0275)^{1/2} \\ &= 1.018 \times 10^6 \end{aligned} \quad (\text{SP.25})$$

From Figure 2 for the value of $Q_{av} = 1.906 \times 10^6$, the normal depth $y_n = 44.5 \text{ ft}$. This, then, is the minimum normal depth.

Consider next,

$$y_{\text{critical}} = \sqrt[3]{\frac{q^2}{g}} \quad \text{where } q = \frac{Q}{B}.$$

A conservative estimate of B @ 6000 ft downstream is 1400 ft.

Therefore,

$$q = \frac{1.906 \times 10^6}{1400} = 1361.43 \frac{\text{ft}^3}{\text{sec}} \quad (\text{SP.26})$$

$$y_{\text{critical}} = \sqrt[3]{\frac{(1361.43)^2}{32}} = 38.6 \text{ ft.} \quad (\text{SP.27})$$

Therefore,

$$y_n > y_c \quad \text{and}$$

hence, the flow is subcritical.

This means that we can break the downstream slope into many straighter slopes and then connect these by smooth transition curves. In other-words, no jumps occur. Therefore, we must now find the normal depths (y_n) for the slopes of 0.0275, 0.0125 and 0.005 and a Q_{av} of $1.906 \times 10^6 \text{ ft}^2/\text{sec}$.

Values of Q for various water heights, slopes and roughness coefficients:

Graphs SPD 4, SPD 5 and SPD 6 show the values of Q for various water heights and S values of 0.0275, 0.0125 and 0.005, respectively. The height of water in the flow channel for any distance downstream of the dam can be directly obtained from Graph SPD 7.

Maximum velocity of water in the flow channel

The maximum velocity occurs at the point of minimum water height.

Therefore, at $y_n = 43.6 \text{ ft}$

$$\begin{aligned} V_{\max} &= \frac{Q_{av}}{A @ 43.6 \text{ ft}} \\ &= \frac{1.906 \times 10^6}{46,150.12} \\ &\approx 41.29 \text{ ft/sec} \quad . \quad \quad \quad (\text{SP.28}) \end{aligned}$$

Normal Water Depth for Second Reach (16,000 to 32,000 Feet): Estimation of Dimensions of Constant Flow Channel

I. Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the second reach by a rectangle with dimensions determined at the middle of the second reach (24,000 feet).

2. Assume that water flowing from the dam will be confined on the left hand side by the natural topography of the area (see Graph SPD 8).

3. Assume that an allowance of 45° , measured from the left end of the flow channel @ 16,000 feet downstream, will account for the natural divergent water flow out of the confined channel, and also for the lateral slope of the topography. (Note: We are making an allowance of 45° , instead of the usual 30° , because the lateral slope in this case is approximately equal to the downstream slope (see Graphs SPD 8 and 9)).

4. Assume an additional allowance of 15° to account for the shifting of the flow channel topography to the right @ 16,000 feet (see SPD 8 and Graphs SPD 1a and 1b as well).

Determination of Parameters B_1, B_2, A, R : When $B \approx 13,000$ Feet and Normal

Depth = Y_n .

1. B_1 and B_2 :

$$B_1 = B_2 = Y_n$$

2. A :

$$A = BY_n = 13,000 Y_n \text{ Ft}^2$$

3. R :

$$R = \frac{A}{P} = \frac{BY_n}{B + 2Y_n}$$

But

$$Y_n \ll B$$

Therefore,

$$R \approx Y_n$$

We recognize that the error introduced by using a rectangular flow channel configuration rather than a trapezoidal configuration is extremely small since the base of the flow channel is much greater than the normal water depth.

Values of Q for Various Water Heights and Roughness Coefficients:

Graph SPD 10 shows the values of Q for various water heights and roughness coefficients. From this graph, we see that for a roughness coefficient of $n = 0.07$, the average water depth for the second reach is 10.5 feet and the average width of the second reach is 13,000 feet.

San Pablo Reservoir

The flood plain resulting from a failure of the San Pablo dam was generated in two reaches extending to San Pablo Bay. The number of expected deaths and the amount of damage to single-unit dwellings for each reach is as follows:

1st reach:

Expected number of deaths:	Daytime ~ 700
	Nighttime ~ 1,100
Damage to homes:	287 homes
	\$3,903,000

2nd reach:

Expected number of deaths:	Daytime ~ 23,100
	Nighttime ~ 34,800
Damage to homes:	8,272 homes
	\$71,590,000

Totals:

Expected number of deaths:	Daytime ~ 23,800
	Nighttime ~ 35,900
Damage to homes:	\$77,493,000

**FLOW CHANNEL CONFIGURATION
@ 3" OR 6000 FT DOWNSTREAM**

Figure 1a. Flow Channel Configuration.

Figure 1b. Flow Channel Configuration for Energy Calculation.

Figure 2. Downstream Slope: Energy Head.

Horizontal Scale 1" = 2000'

Vertical Scale: Elevation in Feet

Graph SPD1a. Downstream Topography.

Horizontal Scale 1" = 2000'
Vertical Scale: Elevation in Feet

Graph SPD1b: Downstream Topography.

Graph SPD2: Actual Downstream Slope of Bottom Point of Flow Channel.

Graph SPD3: Water Height Versus Time.

Graph SPD4: Volume Rate of Flow Versus Normal Depth.

Graph SPD5: Volume Rate of Flow Versus Normal Depth.

Graph SPD6: Volume Rate of Flow Versus Normal Depth.

Graph SPD8: Downstream Topography.

Graph SPD9: Downstream Slope of Bottom Point of Flow Channel.

Graph SPD10: Volume Rate of Flow Versus Normal Depth.

F: SPFP-1
Flood Plain Below the San Pablo Reservoir

SPFP-2
Census Tract Map – Flood Plain Below the San Pablo Reservoir

Folsom Dam

(Details below refer to topographical map)

Name of Owner	Bureau of Reclamation
County	Sacramento
Stream	American River
Location	{ Section 24 Township 10N Range 7E Baseline and/or Meridian MD
Type	Gravity
Storage Capacity	4.39956×10^{10} cubic feet
Drainage Area	1875 square miles
Reservoir Area	4.98762×10^8 square feet
Crest Elevation	480 feet
Crest Length	1400 feet
Permanent Spillway	466 feet
Height of Dam	280 feet
Crest Thickness	36 feet
Dam Volume	12,830,000 cubic yards
Year of Completion	1956

Failure of Dam

From (AI-1),

$$Q_{\max}^{(1)} = \frac{2}{3} L \sqrt{2gH^3}$$

Here,

$$L \approx 1400 \text{ feet}$$

$$H = 266 \text{ feet}$$

Therefore,

$$Q_{\max}^{(1)} = 32.39 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \quad (\text{F.1})$$

From (AI-2),

$$\begin{aligned} Q_{\max}^{(2)} &= \frac{1}{\sqrt{3}} Q_{\max}^{(1)} \\ &= 18.703 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \end{aligned} \quad (\text{F.2})$$

To calculate Q_{av} through the use of equation (AI-7), the following procedure is adopted:

From (AI-12),

$$\text{Actual time of emptying} = T_{\text{actual}} = \frac{A}{\sqrt{2gH} L}$$

Here,

$$A = 11,450 \text{ acres} \times 43,560 \frac{\text{ft}^2}{\text{acre}}$$

Therefore,

$$T_{\text{actual}} \approx 2730.369 \text{ seconds} = 45.5 \text{ minutes} \quad (\text{F.3})$$

Let us compare this with the effective time of emptying. Consider (AI-15) which is,

$$h = \left[H^{\frac{3}{2}} - \frac{tH^2L\sqrt{2g}}{A} \right]^{\frac{2}{3}}$$

An approximate determination of the effective time necessary to empty all the water from behind the dam gives:

$$T_{\text{eff}} \approx 45.5 \text{ minutes} \quad (\text{F.4})$$

Thus T_{eff} and T_{actual} are essentially the same in this case.

Then, from (AI-7),

$$Q_{\text{av}} = \frac{V}{T}$$

Here,

$$V = 4.39 \times 10^{10} \text{ ft}^3$$

$$T = 2730.36 \text{ seconds}$$

Thus,

$$Q_{\text{av}} = 1.611 \times 10^7 \frac{\text{ft}^3}{\text{sec}} \quad (\text{F.5})$$

Calculation of the Time of First Large Wave and the Amount of Instantaneous Release of Water

From (AI-9),

$$\tilde{t} \approx \sqrt{\frac{H}{g}} = 2.88 \text{ seconds} \quad (\text{F.6})$$

Therefore, it is apparent that a large amount of water is released almost immediately after the dam failure. To find out the amount released, let us first calculate the height of water still behind the orifice at the time $\tilde{t}$.

From (AI-15), with $t = \tilde{t}$,

$$h = 265.8 \text{ feet} \quad (\text{F.7})$$

Therefore,

$$\Delta h \text{ (from } t = 0 \text{ to } t = 2.88 \text{ seconds)} = 0.179 \text{ feet}$$

The amount of water which flows out of the orifice in $\tilde{t}$ is given by

$$\begin{aligned}\Delta V &\approx A\Delta h \\ &= 89.27 \times 10^6 \text{ ft}^3\end{aligned}\tag{F.8}$$

With the constant flow channel dimensions and reaches as described in the addendum for the Folsom Dam breakage calculations, we can proceed to make Q vs. y_n graphs for the various reaches and roughness coefficients. The following is a summary of the relevant results from such calculations.

First Reach:

Length

Width = 8,000 feet

$$y_n^{(0.09)} = 103 \text{ feet}$$

Second Reach:

Length

Width = 11,000 feet

$$y_n^{(0.09)} = 100.5 \text{ feet}$$

Third Reach:

Length

Width = 24,000 feet

$$y_n^{(0.09)} = 84.5 \text{ feet}\tag{F.9}$$

Expected Number of Deaths and Damage from a Failure of the Folsom Dam.

The flood plain below the Folsom Dam is divided by the natural topography into 4 reaches ending in the city of Sacramento after crossing the junction of the American and Sacramento Rivers. Due to the narrow, confined channel afforded the flood in the 1st reach, no credit in deaths or damage is taken there. The consequences, enumerated for each reach are as follows:

1st reach: No deaths assumed

2nd reach:

Expected number of deaths: 22,410

Amount of damage:

Single-unit homes destroyed: 5,065

Value: \$47,776,000

3rd reach:

Expected number of deaths: 52,020

Amount of damage:

Single-unit homes destroyed: 13,100

Value: \$156,176,000

4th reach:

Expected number of deaths: 181,430

Amount of damage:

Single-unit homes destroyed: 54,810

Value: \$462,267,000

Total:

Expected number of deaths: 255,860

Amount of damage:

Single-unit homes destroyed: 72,975

Value: \$666,220,000

Since such a large portion of this metropolitan area is destroyed, both residential and commercial centers alike, it is not expected that there would be a significant difference in the number of casualties occurring in the daytime and in the nighttime.

Graph FD1: Volume Rate of Flow vs. Normal Depth.

G: FFP-1
Flood Plain Below the Folsom Lake

Chatsworth Dam

(Details below refer to topographical map)

* Dam Number	6 - 4
Name of Owner	City of Los Angeles
County	Los Angeles
Stream	Tributary, Los Angeles River
Location	{ Section 25 Township 2N Range 17W Baseline and/or Meridian SB
Type	Hydraulic fill
Storage Capacity	4.3063×10^8 cubic feet
Drainage Area	5.4 square miles
Reservoir Area	2.644×10^7 square feet
Crest Elevation	900 feet
Crest Length	2700 feet
Permanent Spillway	889 feet
Height of Dam	45 feet
Crest Thickness	21 feet
Dam Volume	739,304 cubic yards
Year of Completion	1918

* As assigned by Department of Water Resources

Calculation of $Q_{\max}$ and Q_{av} Under Assumption of Complete and Instantaneous Failure of Dam:

In the case of Chatsworth Dam, we shall assume that the compacted earth completely erodes immediately after a breach in the concrete occurs, i.e., instantaneous complete dam failure.

From (A1-1),

$$Q_{\max}^{(1)} = \frac{2}{3} L \sqrt{2gH^3}$$

Here,

$$L \approx 610 \text{ ft}$$

$$H = 34 \text{ ft}$$

Therefore,

$$Q_{\max}^{(1)} = 6.45 \times 10^5 \frac{\text{ft}^3}{\text{sec}} \quad (\text{CH.1})$$

The value for L in the above calculation does not include the dirt embankment on the right side of the dam.

From (A1-2),

$$\begin{aligned} Q_{\max}^{(2)} &= \frac{1}{\sqrt{3}} Q_{\max}^{(1)} \\ &= 3.72 \times 10^5 \frac{\text{ft}^3}{\text{sec}} \end{aligned} \quad (\text{CH.2})$$

To calculate Q_{av} through the use of the equation (A1-7), the following procedure is adopted.

From (A1-6),

$$\text{Actual time of emptying} = T_{\text{actual}} = 3A / (L\sqrt{2gH})$$

Here,

$$A = 26.44 \times 10^6 \text{ ft}^2$$

Therefore,

$$T_{\text{actual}} = 46.5 \text{ minutes} \quad (\text{CH.3})$$

Let us compare this with the effective time of emptying. Consider (A1-8) which is

$$h = \left(\sqrt{H} - \frac{tLH\sqrt{2g}}{3A} \right)^2$$

With this equation, we can generate CD3 which is a graph of the height of water against time. From CD3, we can make an approximate determination of the effective time necessary to empty all the water from behind the dam.

This, from CD3, is

$$T_{\text{eff}} = 42 \text{ minutes} \quad (\text{CH.4})$$

We shall use this value for calculating Q_{av} .

From (A1-7),

$$Q_{\text{av}} = \frac{V}{T}$$

Here,

$$V = 430.63 \times 10^6 \text{ ft}^3, T = 42 \text{ minutes}$$

Therefore,

$$Q_{\text{av}} = 1.708 \times 10^5 \frac{\text{ft}^3}{\text{sec}} \quad (\text{CH.5})$$

Calculations of the Time of First Large Wave and the Amount of Instantaneous Release of Water

From (A1-9),

$$\tilde{t} \approx \sqrt{\frac{H}{g}} = 1.031 \text{ sec.} \quad (\text{CH.6})$$

Therefore, it is apparent that a large amount of water is released almost immediately after the dam failure.

To find out the amount of water released in this time we must first calculate the height of water still behind the orifice at the time $\tilde{t}$.

From, (A1-8), with $t = \tilde{t}$,

$$h = 33.97 \text{ ft} \quad (\text{CH.7})$$

Therefore,

$$\Delta h \text{ (from } t = 0 \text{ to } t = 1.031 \text{ sec)} = 0.026 \text{ ft}$$

Under the assumption that the immediate upstream topography and the water catchment can be geometrically and equivalently approximated by a solid prismatic bowl, the amount of water which flows out of the orifice in $\tilde{t}$ is given by

$$\Delta V \cong A \Delta h = 6.87 \times 10^5 \text{ ft}^3 \quad (\text{CH.8})$$

Estimation of the Dimensions of Constant Flow Channel for the First, Second and Third Reaches of the Chatsworth Dam Downstream

An arbitrary choice of three reaches - 0 to 6000 feet, 6000 to 12,000 feet, 12,000 to 18,000 feet, has been made. The constant flow channel dimensions for these three reaches are set using the details at the centers of the reaches, i.e., the configuration details at 3000 feet, 9000 feet and 15,000 feet, serve for the dimensions for the three reaches respectively. It is thought this representation is a sufficiently good approximation for the purposes of flood plain calculations.

With reference to Graphs CD 1a - 1h and the flood plain map, we could explain the flow channel as follows: As the water emerges from the dam opening, it is immediately turned to the right by high ridges on both sides of the flow channel. This confinement of the flow, however, lasts only until the water has traveled some 2,000 feet to the right of the center of the dam. From this point, the flow is virtually unconfined. Also, the topography reveals a slight downstream slope towards the right of the dam from 2,000 feet to 7,500 feet. The natural divergence of flow and the added effect of this particular slope are accounted for by allowing the left and right sides of the flow channel to diverge by 30° and 15° respectively. The remainder of the flow channel length, as reflected by the topography, shows the channel to slope slightly inwards, towards the center of the flow channel itself. This sloping has been

accounted for by letting the left side of the flow channel converge by 15° while holding the right side constant. This condition is maintained until the end of the flow channel (18,000 feet).

As pointed out earlier, using the width of the flow channel at the middle of each reach as the base of the cross-sectional trapezoid and allowing the sides of the trapezoid to be inclined at about 5° from the horizontal, the dimensions of the constant flow channel for each reach are determined. Figures CH1a, CH1b and CH1c show these configurations.

Values of Q for Various Water Heights and Roughness Coefficients

Graphs CD 4a, 4b and 4c show the values of Q for various water heights, roughness coefficients, and reaches. From these graphs, for a roughness coefficient of $n = 0.09$, the results may be described as follows:

First reach:

Width = 2625 feet

Average water depth = 10.2 feet

Second reach:

Width = 6375 feet

Average water depth = 7.5 feet

Third reach:

Width = 9000 feet

Average water depth = 6.0 feet

Expected Number of Deaths and Damage from a Failure of the Chatsworth #2 Dam.

Computation of the flood plain below the Chatsworth Reservoir extended for three 6,000 foot reaches to just past Canoga Park. As the height of the water is the least of any failure examined, the estimates of downstream damage, must be appropriately qualified. Assuming that the height of water listed for each reach is sufficient to destroy single-unit dwellings and to drown the inhabitants the estimates are:

1st reach: Average height of water: 10.2 feet

Expected number of deaths

Daytime: 3,614

Nighttime: 5,870

Amount of damage

Single-unit homes destroyed: 1,443

Value: \$21,170,000

2nd reach: Average height of water: 7.5 feet

Expected number of deaths

Daytime: 4,950

Nighttime: 7,466

Amount of damage:

Single-unit homes destroyed: 1,731

Value: \$24,210,000

3rd reach: Average height of water: 6.0 feet

Expected number of deaths

Daytime: 5,156

Nighttime: 8,238

Amount of damage

Single-unit homes destroyed: 1,626

Value: \$19,215,000

Total:

- A) Assuming that an average water height of 10 feet is necessary to destroy all single-unit homes and to drown the occupants the consequences are the estimates for reach 1 above:

Expected number of deaths

Daytime: 3,614

Nighttime: 5,870

Amount of damage

Single-unit homes destroyed: 1,443

Value: \$21,170,000

- B) Assuming that an average water height of 7.5 feet will accomplish the destruction of single family dwellings and the drownings of all occupants, the consequences are the totals for reaches 1 and 2:

Expected number of deaths

Daytime: 8,564

Nighttime: 13,336

Amount of damage

Single-unit homes destroyed: 3.174

Value: \$45,380,000

- C) Assuming that an average water height of 6.0 feet is sufficient to accomplish the destruction of single-family dwellings and the drowning of all the occupants, the three reach totals are:

Expected number of deaths

Daytime: 13,720

Nighttime: 21,574

Amount of damage

Single-unit homes destroyed: 4,800

Value: \$64,595,000

TRAPEZOID @ 3000 FT

TRAPEZOID @ 9000 FT

TRAPEZOID @ 15,000 FT

Figures CH1a, CH1b, and CH1c: Actual Dimensions of Trapezoids.

Graph CD1a: Downstream Topography.

Graph CD1b: Downstream Topography.

Graph CD1c: Downstream Topography.

Graph CD1d: Downstream Topography.

Graph CD1e: Downstream Topography.

Graph CD1f: Downstream Topography.

Graph CD1g: Downstream Topography.

Graph CD1h: Downstream Topography.

Graph CD2: Downstream Slope for Chatsworth Dam.

Graph CD3: Water Height vs. Time.

Graph CD4a. Volume Rate of Flow vs. Normal Depth.

Graph CD4b. Volume Rate of Flow vs. Normal Depth.

Graph CD4c: Volume Rate of Flow vs. Normal Depth.

CFP-1
Flood Plain Below the Chatsworth Reservoir

CFP-2

Census Tract Map – Flood Plain Below the Chatsworth Reservoir

Shasta Dam

(Details below refer to topographical map)

Name of Owner	Bureau of Reclamation
County	Shasta
Stream	Sacramento River
Location	{ Section 15 Township 33N Range 5W Baseline and/or Meridian MD
Type	Gravity
Storage Capacity	1.9602×10^{11} cubic feet
Drainage Area	666,500 square miles
Reservoir Area	1.28502×10^8 square feet
Crest Elevation	1078 feet
Crest Length	3460 feet
Permanent Spillway	1065 feet
Height of Dam	487 feet
Crest Thickness	30 feet
Dam Volume	6,541,000 cubic yards
Year of Completion	1949

A Preliminary Study of Shasta Dam Breakage

With reference to Figure 1, water will flow through the orifice until the water height reduces to the lowest point of the orifice, i.e. 587 feet.

$$\text{Area of Shasta Lake} = 1.285 \times 10^9 \text{ ft}^2$$

$$\text{Volume of Shasta Lake} = 1.960 \times 10^{11} \text{ ft}^3$$

Determination of Flood Plain

I. Assumptions Made

1. Assume that each 7.5 minute quadrangle in the topographical map represents one reach of the flow channel.
2. Assume that the first three reaches have normal water depths of 125 feet, 100 feet and 75 feet, as measured from the bottom of the flow channel, respectively. We note that since the river is at the lowest elevation, we can take this as the base of the flow channel, and then add the normal water depth to find the contour line at which the water will be contained.
3. Assume that, if the contour line found as described above is too far from the base of the flow channel, then a slanting degree allowance will enable the calculation of the immediately flooded area.
4. Assume that at the start of the second reach an allowance of 45° must be made to account for the water flow. This assumption is based on the fact that the topography indicates a sharp turn to the right which combined with a steep downward slope imparts approximately equal momentum in horizontal and vertical directions.
5. Assume that at the midpoint of the second reach, a 30° allowance is sufficient because the downstream slope is much steeper than the lateral slope.
6. Assume that the area of the flood plain can be approximated by smooth curves connecting the contour points.

Measurement of Flood Plain Area

$$\text{Area of First Reach} = 7.6 \times 10^6 \text{ ft}^2$$

$$\begin{array}{l} \text{Area of Second and} \\ \text{Third Reaches} \end{array} = 1.779 \times 10^9 \text{ ft}^2$$

Therefore,

$$\text{Total Area of Flood Plain} = 1.78 \times 10^9 \text{ ft}^2$$

$$\underline{\text{Height of Water in Parallelepiped of Plan Area Equal to } 1.78 \times 10^9 \text{ Ft}^2}$$

The height of water sheet, under the assumption of a rectangular parallelepiped model, can be calculated as follows:

$$\begin{array}{l} \text{Volume of Water} \\ \text{of Shasta Lake} \end{array} = 1.960 \times 10^{11} \text{ ft}^3$$

$$\begin{array}{l} \text{Height of Water} \\ \text{in Parallelepiped} \end{array} = \frac{1.960 \times 10^{11} \text{ ft}^3}{1.78 \times 10^9 \text{ ft}^2}$$

$$= 109.68 \text{ feet}$$

Downstream Consequences of a Failure of the Shasta Dam

Employing the rough analysis of the hydrology group a first approach may be made to estimating the approximate number of deaths and the damage to be expected downstream. An estimate must be made of the point at which evacuation measures may significantly change the number of people drowned.

The approximate distance in river miles from the damsite to the town of Anderson is 22 miles. It is then 30 miles further down river to Red Bluff, the next town. (The town of Cottonwood seems to be located far enough away from the main flood to avoid inundation.)

Assuming what is probably a fairly modest velocity for the flood of 10 miles per hour, it is clear that from the moment the dam fails the towns of Anderson and Red Bluff have about 2 hours and 5 hours, respectively, in which to effect evacuation.

Assuming further that the municipal authorities of each town were notified at the exact time that Shasta Dam failed it might be possible to evacuate some people from Anderson (pop.: 5500) and it is certainly possible that everyone in Red Bluff (pop.: 8000) could be moved to safety. It is not known how soon after the dam's failure word would be sent out. Since it is not thought that a very large portion of the population of Anderson could be evacuated in the two hours or less allotted, it is assumed that the entire populace will be lost and the town destroyed. One must keep in mind that the velocity of the flood will be higher (perhaps 2 to 2.5 times as high) than the 10 mph we assumed during the first 5-10 miles past the damsite. Thus Anderson may have much less than 2 hours to evacuate.

Red Bluff, on the other hand, is at the base of a tortuous set of turns which the flood must make. Therefore, it is expected that Red Bluff will have almost the full 5 hours or so in which to evacuate. Although the

populace is assumed to escape, the town of Red Bluff will be engulfed and destroyed. It is further assumed that this pattern continues for about another 35 miles downstream from Red Bluff to Hamilton City.

Thus, the towns and cities within the first 25 miles of the damsite suffer both death and destruction while those towns in the next 30-35 miles along the river were assumed to suffer only destruction of buildings, although this may be optimistic. The consequences are outlined below.

The towns which are expected to be engulfed before evacuation of the populace can be effected are: Redding, Enterprise, Girvan and Anderson.

The total number of expected deaths is then about 34,000 as broken down here:

<u>Town</u>	<u>Population</u>	<u>Approximate Value of Homes Destroyed</u>
Redding	16,659	
Enterprise	11,486	
Girvan	~300-400	
Anderson	<u>5,492</u>	
	~34,000	\$142,800,000

The towns which are expected to suffer the destruction of buildings are: Red Bluff, Proberta, Gerber, Tahoma, Los Molinos, Vina, and Hamilton City. The total loss in homes is expected to be about \$48,000,000.

Thus the damage expected from the failure of Shasta Dam as outlined above would be about \$190,000,000 and a cost in lives of about 34,000 people.

Figure 1. Front View of Dam Orifice.

H: SFP-1
Flood Plain Below the Shasta Lake

Mulholland Dam

(Details below refer to topographical map)

* Dam Number	6 - 17
Name of Owner	City of Los Angeles
County	Los Angeles
Stream	Weid Canyon
Location	{ Section 3 Township 1S Range 14W Baseline and/or Meridian SB
Type	Gravity
Storage Capacity	1.75808×10^8 cubic feet
Drainage Area	1.00 square miles
Reservoir Area	3.57192×10^6 square feet
Crest Elevation	756 feet
Crest Length	933 feet
Permanent Spillway	721 feet
Height of Dam	195 feet
Crest Thickness	16 feet
Dam Volume	303,000 cubic yards
Year of Completion	1924

* As assigned by Department of Water Resources

Failure of Dam

In the case of Mulholland Dam, we shall assume that the compacted earth completely erodes immediately after a breach in the concrete occurs, i.e., instantaneous complete dam failure. From (AI-1),

$$Q_{\max}^{(1)} = \frac{2}{3} L \sqrt{2gH^3}$$

Here,

$$L = 780 \text{ feet}$$

$$H = 160 \text{ feet}$$

Therefore,

$$Q_{\max}^{(1)} = 8.41 \times 10^6 \frac{\text{A}^3}{\text{sec}} \quad (\text{MH.1})$$

The length, L , above is the effective length for the dam since the opening is shorter than the actual curved dam and we are only interested in the length of the orifice itself.

From (AI-2),

$$\begin{aligned} Q_{\max}^{(2)} &= \frac{1}{\sqrt{3}} Q_{\max}^{(1)} \\ &= 4.86 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \quad (\text{MH.2}) \end{aligned}$$

To calculate Q_{av} through the use of equations (AI-7) the following procedure is adopted. From (AI-12),

$$\text{Actual time of emptying} = T_{\text{actual}} = A / (L\sqrt{2gH})$$

$$\text{Here, } A = 3.57 \times 10^6 \text{ ft}^2$$

Therefore,

$$T_{\text{actual}} = 45.25 \text{ seconds}$$

$$\approx 0.75 \text{ minutes}$$

(MH.3)

Let us compare this with the effective time of emptying. Consider (AI-15) which is

$$h = \left(H^{\frac{3}{2}} - \frac{\sqrt{2g} LH^2}{A} t \right)^{2/3}$$

With this equation, we can generate MD3 which is a graph of the height of water against time. From MD3, we can make an approximate determination of the effective time necessary to empty all the water from behind the dam.

This, from MD3, is

$$T_{\text{eff}} = 45.2 \text{ seconds} \quad (\text{MH.4})$$

The T_{eff} turns out to be the same as T_{actual} in this case.

Therefore, from (AI-7),

$$Q_{\text{av}} = \frac{V}{T}$$

Here,

$$V = 1.75 \times 10^9 \text{ ft}^3, T = 45.2 \text{ seconds}$$

Therefore,

$$Q_{\text{av}} = 3.89 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \quad (\text{MH.5})$$

Calculations of the Time of First Large Wave and the Amount of Instantaneous Release of Water

From (AI-9),

$$\tilde{t} = \sqrt{\frac{H}{g}} = 2.236 \text{ seconds} \quad (\text{MH.6})$$

Therefore, it is apparent that a large amount of water is released almost immediately after the dam failure.

To find out the amount released in this time, we must first calculate the height of water still behind the orifice at the time $\tilde{t}$.

From (AI-15), with $t = \tilde{t}$,

$$h = 154.67 \text{ feet} \quad (\text{MH.7})$$

Therefore,

$$\Delta h \text{ (from } t = 0 \text{ to } t = 2.236 \text{ seconds)} = 5.32 \text{ feet}$$

The amount of water which flows out of the orifice in $\tilde{t}$ is given by

$$\Delta V \approx A \Delta h = 18.99 \times 10^6 \text{ ft}^3 \quad (\text{MH.8})$$

Estimation of the Dimensions of Constant Flow Channel

First Reach (0 to 4000 feet)

I. Assumptions Made

1. Since the flow channel is well defined by the natural topography of the area (i.e., by high elevations on both sides of the channel), the effects of divergent water flow out of the channel or of the lateral sloping can be considered insignificant. Any divergent flow of water will return to the channel because of the steep inward slopes of the two sides (see Graph MD1a and Figure 1).

2. Assume that we can approximate the constant flow channel configuration by a trapezoid with the dimensions shown in Figure 1.

Determination of Parameters B_1 , B_2 , C , A , R

1. B

$$B_1 = B_2 = \frac{y_n}{\sin 65^\circ}$$

2. C

$$\begin{aligned} C &= B + 2B_2 \cos 65^\circ \\ &= 400 + 2y_n \cot 65^\circ \end{aligned}$$

3. A

$$A = \frac{C + B}{2} Y_n$$

4. P

$$\begin{aligned} P &= B_1 + B + B_2 \\ &= 2B_1 + B \\ &= \frac{2y_n}{\sin 65^\circ} + 400 \end{aligned}$$

5. R

$$R = \frac{A}{P}$$

Values of Parameters of Various Water Heights

Slope = 0.03025 B = 400 feet

y_n (ft)	$A \times 10^{-4}$ (ft ²)	R (ft)
0	0	0
4	0.1607	3.9319
8	0.3229	7.7333
12	0.4867	11.4120
16	0.6519	14.9760
20	0.8186	18.4330
24	0.9868	21.7870
28	1.1566	25.0450
32	1.3278	28.2130
36	1.5004	31.2950
40	1.6746	34.2970
44	1.8503	37.2220
48	2.0274	40.0740
52	2.2061	42.8570
56	2.3862	45.5760
60	2.5679	48.2320
64	2.7510	50.8280

y_n (ft)	$A \times 10^{-4}$ (ft ²)	R (ft)
68	2.9356	53.3690
72	3.1217	55.8560
76	3.3093	58.2920
80	3.4984	60.6800
84	3.6890	63.0210
88	3.8811	65.3170
92	4.0747	67.5710
96	4.2698	69.7850
100	4.4663	71.9890
104	4.6644	74.0960
108	4.8639	76.1970
112	5.0649	78.2650
116	5.2675	80.2990
120	5.4715	82.3010
124	5.6770	84.2740
128	5.8840	86.2170
132	6.0925	88.1320
136	6.3025	90.0200
140	6.5140	91.8830
144	6.7269	93.7200
148	6.9414	95.5330
150	7.0492	96.4310

From Manning's formula given by (A2-1), we can calculate Q values for various water depths, thus enabling us to make a Q vs. y_n graph. From Graph MD4, we see that corresponding to $n = 0.05, 0.07$ and 0.09 values, the y_n are

$$\left. \begin{aligned} y_n^{(0.05)} &= 98.05 \text{ feet} \\ y_n^{(0.07)} &= 121.60 \text{ feet} \\ y_n^{(0.09)} &= 142.20 \text{ feet} \end{aligned} \right\} \quad (\text{MH.9})$$

The length of the first reach is 4,000 feet and the average breadth is 400 feet.

Normal Water Depth for Second Reach (4000 - 8000 Feet)

Estimation of Dimensions of Constant Flow Channel

I. Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the second reach by a trapezoid with dimensions determined at the middle of the second reach, i.e., 6000 feet

2. Assume that an allowance of 30° , measured from the left and right of the confined flow channel at 4,000 feet downstream, will account for the natural divergent water flow out of the confined channel. Since there is no lateral sloping above 10,000 feet (Graph MD1a), no further allowance is necessary (see flood plain map). Figure 2 shows the actual dimensions of the flow channel.

Determination of Parameters B_1 , B_2 , C, A, R

1. B

$$B_1 = B_2 = \frac{y_n}{\sin 10^\circ}$$

2. C

$$\begin{aligned} C &= B + 2B_2 \cos 10^\circ \\ &= \{3640 + 2y_n \cot 10^\circ\} \end{aligned}$$

3. A

$$A = \frac{(C + B)}{2} y_n$$

4. P

$$\begin{aligned} P &= B_1 + B + B_2 \\ &= 2B_1 + B \\ &= \frac{2y_n}{\sin 10^\circ} + 3640 \end{aligned}$$

5. R

$$R = \frac{A}{P}$$

Values of Parameters for Various Water Heights

Slope = 0.0235		Base = 3640 feet
y_n (ft)	$A \times 10^{-4}$ (ft ²)	R (ft)
0	0	0
4	1.4651	3.9746
8	2.9483	7.8997
12	4.4497	11.7770
16	5.9692	15.6090
20	7.5068	19.3960
24	9.0626	23.1400
28	10.6370	26.8430
32	12.2290	30.5070
36	13.8390	34.1310
40	15.4670	37.7190
44	17.1140	41.2700
48	18.7790	44.7870
50	19.6180	46.5330

From Manning's formula, given by (A2-1), we can calculate Q values for various water depths, thus enabling us to make a Q vs. y_n graph. From graph MD5, we see that corresponding to $n = 0.07, 0.09$ and 0.11 values the y are

$$\left. \begin{array}{l} y_n^{(0.07)} = 32.2 \text{ feet} \\ y_n^{(0.09)} = 37.3 \text{ feet} \\ y_n^{(0.11)} = 42.0 \text{ feet} \end{array} \right\} \quad (\text{MH.10})$$

The length of the second reach is 4,000 feet, and the average breadth is 3,640 feet.

Normal Water Depth for Third Reach (8000 - 12,000 Feet)

Estimation of Dimensions of Constant Flow Channel

I. Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the third reach by a trapezoid with dimensions determined at the middle of the third reach, i.e., 10,000 feet.

2. Assume that we may continue the 30° allowance from the first reach, for natural divergent water flow, until 10,000 feet downstream from the dam. (See Graph MD1b and Figure 3).

3. Assume that an additional allowance of 15° , added on to the left and subtracted from the right end of the flow channel at 10,000 feet downstream, will account for the lateral sloping to the left (see Graph MD1b and Figure 3). Figure 3 shows the actual dimensions for approximation of constant flow channel at 10,000 feet downstream.

Determination of Parameters B_1 , B_2 , C, A, R

1. B

$$B_1 = B_2 = \frac{y_n}{\sin 10^\circ}$$

2. C

$$\begin{aligned} C &= B + 2B_2 + \cos 10^\circ \\ &= (8250 + 2y_n \cot 10^\circ) \end{aligned}$$

3. A

$$A = \frac{C + B}{2} y_n$$

4. P

$$\begin{aligned}
 P &= B_1 + B + B_2 \\
 &= 2B_1 + B \\
 &= \frac{2y_n}{\sin 10^\circ} + 8250
 \end{aligned}$$

5. R

$$R = \frac{A}{P}$$

Values of Parameters for Various Water Heights

Slope = 0.01625 Base = 8250 feet

<u>y_n</u> <u>(ft)</u>	<u>A x 10⁻⁴</u> <u>(ft²)</u>	<u>R</u> <u>(ft)</u>
0	0	0
4	3.3091	3.9887
8	6.6363	7.9551
12	9.9817	11.9000
16	13.3450	15.8230
20	16.7270	19.7240
24	20.1270	23.6050
28	23.5450	27.4650
32	26.9810	31.3050
36	30.4350	35.1260
40	33.9070	38.9260

From Manning's formula, given by (A2-1), we can calculate Q values for various water heights, thus enabling us to make a Q vs. y_n graph. From Graph MD6, we see that corresponding to n = 0.07, 0.09 and 0.11 values, the y_n are

$$\left. \begin{array}{l} y_n^{(0.07)} = 22.2 \text{ feet} \\ y_n^{(0.09)} = 25.7 \text{ feet} \\ y_n^{(0.11)} = 29.2 \text{ feet} \end{array} \right\} \quad (\text{MH.11})$$

The length of the third reach is 4,000 feet, and the average breadth is 8,250 feet.

Normal Water Depth for the Fourth Reach (12,000 - 16,000 Feet)

Estimation of the Dimensions of Constant Flow Channel

I. Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the fourth reach by a trapezoid with dimensions determined at the middle of the fourth reach, i.e., 14,000 feet.

2. Assume that we may continue the 45° allowance (30° for original natural divergence of water, and 15° for lateral sloping to left, measured at 10,000 feet downstream, until the end of the reach (see Graph MD1b and Figure 4).

3. Assume that we may continue the 15° allowance (30° for original natural divergence of water flow, from which we subtract 15° to account for the lateral sloping to the left), measured from the right end of the flow channel at 10,000 feet downstream, until the end of the reach. (See Graph MD1b and Figure 4). Figure 4 shows the actual dimensions for approximation of constant flow channel at 14,000 feet downstream.

Determination of Parameters B_1 , B_2 , C, A, R

1. B

$$B_1 = B_2 = \frac{y_n}{\sin 10^\circ}$$

2. C

$$C = B + 2B_2 \cos 10^\circ$$

$$= \{13,375 + 2y_n \cot 10^\circ\}$$

3. A

$$A = \frac{C + B}{2} y_n$$

4. P

$$P = B_1 + B + B_2$$

$$= 2B_1 + B$$

$$= \frac{2y_n}{\sin 10^\circ} + 13,375$$

5. R

$$R = \frac{A}{P}$$

Values of Parameters for Various Water Heights

Slope = 0.015 Base = 13,375 Feet

<u>y_n</u> <u>(ft)</u>	<u>A x 10⁻⁴</u> <u>(ft²)</u>	<u>R</u> <u>(ft)</u>
0	0	0
4	5.3591	3.9930
8	10.7360	7.9722
12	16.1320	11.9380
16	21.5450	15.8900
20	26.9770	19.8280
24	32.4270	23.7530
28	37.8950	27.6650
30	40.6350	29.6170

From Manning's formula, given by (A2-1), we can calculate Q values for various water heights, thus enabling us to make a Q vs. y_n graph. From Graph

MD7, we see that corresponding to $n = 0.07, 0.09$ and 0.11 values, the y_n are

$$\left. \begin{aligned} y_n^{(0.07)} &= 17.0 \text{ feet} \\ y_n^{(0.09)} &= 19.6 \text{ feet} \\ y_n^{(0.11)} &= 22.3 \text{ feet} \end{aligned} \right\} \quad (\text{MH.12})$$

The length of the fourth reach is 4,000 feet, and the average breadth is 13,375 feet.

Normal Water Depth for the Fifth Reach (16,000 - 20,000 Feet)

Estimations of the Dimensions of Constant Flow Channel

I. Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the fifth reach by a trapezoid with dimensions determined at the middle of the fifth reach, i.e., at 18,000 feet.

2. Assume that we may continue the 45° allowance, measured from the left end of the flow channel at 10,000 feet downstream, until the end of the reach (see Graph MD1b and 1c).

3. Assume that we may continue the 15° allowance, measured from the right end of the flow channel at 10,000 feet downstream, until the end of the reach (see Graphs MD1b and 1c, and also the flood plain map). Figure 5 shows the actual dimensions for the approximation of constant flow channel at 18,000 feet downstream.

Determination of Parameters B_1, B_2, C, A, R

1. B

$$B_1 = B_2 = \frac{y_n}{\sin 10^\circ}$$

2. C

$$C = B + 2B_2 \cos 10^\circ$$

$$= \{18,500 + 2y_n \cot 10^\circ\}$$

3. A

$$A = \frac{C + B}{2} y_n$$

4. P

$$P = B_1 + B + B_2$$

$$= 2B_1 + B$$

$$= \frac{2y_n}{\sin 10^\circ} + 18,500$$

5. R

$$R = \frac{A}{P}$$

Values of Parameters for Various Water Heights

Slope = 0.0125

Base = 18,500 feet

<u>y_n</u> <u>(ft)</u>	<u>A x 10⁻⁴</u> <u>(ft²)</u>	<u>R</u> <u>(ft)</u>
0	0	0
4	7.4091	3.9950
8	14.8360	7.9799
12	22.2820	11.9550
16	29.7450	15.9200
20	37.2270	19.8750
24	44.7270	23.8210
28	52.2450	27.7560
30	56.0100	29.7210

From Manning's formula, given by (A2-1), we can calculate Q values for various water heights, thus enabling us to make a Q vs. y_n graph. From

Graph MD8, we see that corresponding to $n = 0.07, 0.09$ and 0.11 values, the y_n are

$$\left. \begin{array}{l} y_n^{(0.07)} = 14.8 \text{ feet} \\ y_n^{(0.09)} = 17.2 \text{ feet} \\ y_n^{(0.11)} = 19.4 \text{ feet} \end{array} \right\} \quad (\text{MH.13})$$

The length of the fifth reach is 4,000 feet, and the average breadth is 18,500 feet.

Normal Water Depth for the Sixth Reach (20,000 - 24,000 Feet)

Estimation of the Dimensions of Constant Flow Channel

I. Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the sixth reach by a trapezoid with dimensions determined at the middle of the sixth reach, i.e., at 22,000 feet.

2. Assume that we may continue the 45° allowance, measured from the left end of the flow channel at 10,000 feet downstream, until 21,000 feet downstream. From this point until the end of the reach, the water is confined by high elevations of the topography (see Graphs MD1c, 1d).

3. Assume that we may continue the 15° allowance, measured from the right end of the flow channel at 10,000 feet downstream, until the end of the reach (see Graphs MD1c, 1d and also see the flood plain). Figure 6 shows the actual dimensions for the approximation of constant flow channel at 22,000 feet downstream.

Determination of Parameters B_1, B_2, C, A, R

1. B

$$B_1 = B_2 = \frac{y_n}{\sin 10^\circ}$$

2. C

$$C = B + 2B_2 \cos 10^\circ$$

$$= \{22,800 + 2y_n \cot 10^\circ\}$$

3. A

$$A = \frac{C + B}{2} y_n$$

4. P

$$P = B_1 + B + B_2$$

$$= \frac{2y_n}{\sin 10^\circ} + 22,800$$

5. R

$$R = \frac{A}{P}$$

Values of Parameters for Various Water Heights

Slope = 0.01 Base = 22,800 feet

<u>y_n</u> <u>(ft)</u>	<u>A x 10⁻⁴</u> <u>(ft²)</u>	<u>R</u> <u>(ft)</u>
0	0	0
4	9.1291	3.9959
8	18.2760	7.9837
12	27.4420	11.9630
16	36.6250	15.9350
20	45.8270	19.8980
24	55.0470	23.8540
28	64.2850	27.8020
30	68.9100	29.7730

From Manning's formula, given by (A2-1), we can calculate Q values for various water heights, thus enabling us to make a Q vs. y_n graph. From Graph MD9, we see that corresponding to n = 0.07, 0.09 and 0.11 value, the y_n are

$$\left. \begin{array}{l} y_n^{(0.07)} = 14.0 \text{ feet} \\ y_n^{(0.09)} = 16.2 \text{ feet} \\ y_n^{(0.11)} = 18.2 \text{ feet} \end{array} \right\} \quad (\text{MH.14})$$

The length of the sixth reach is 4,000 feet, and the average breadth is 22,800 feet.

Normal Water Depth for the Seventh Reach (24,000 - 28,000 Feet)

Estimation of the Dimensions of Constant Flow Channel

I. Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the seventh reach by a trapezoid with dimensions determined at the middle of the seventh reach, i.e., at 26,000 feet.

2. Assume that an allowance of 15° , measured from the left end of the flow channel at 24,000 feet, will account for natural divergence and small lateral sloping to the left. However, the water is confined at 26,000 feet downstream by high elevations of topography. (See Graphs MD1c,1d.)

3. Assume that we may continue the 15° allowance, measured from the right end of the flow channel at 10,000 feet downstream, until the end of the reach. (See Graphs MD1c, d and also the flood plain map.) Figure 7 shows the actual dimensions for the approximation of constant flow channel at 26,000 feet downstream.

Determination of Parameters B_1 , B_2 , C , A , R

1. B

$$B_1 = B_2 = \frac{y_n}{\sin 10^\circ}$$

2. C

$$\begin{aligned} C &= B_1 + 2B_2 \cos 10^\circ \\ &= \{25,000 + 2y_n \cot 10^\circ\} \end{aligned}$$

3. A

$$A = \frac{C + B}{2} y_n$$

4. P

$$\begin{aligned} P &= B_1 + B + B_2 \\ &= 2B_1 + B \\ &= \frac{2y_n}{\sin 10^\circ} + 25,000 \end{aligned}$$

5. R

$$R = \frac{A}{P}$$

Values of Parameters for Various Water Heights

Slope = 0.00375 Base = 25,000 feet

y_n (ft)	$A \times 10^{-4}$ (ft ²)	R (ft)
0	0	0
4	10.009	3.9963
8	20.036	7.9851
12	30.082	11.9670
16	40.145	15.9410
20	50.227	19.9070

y_n (ft)	$A \times 10^{-4}$ (ft ²)	R (ft)
24	60.327	23.8670
28	70.445	70.4450
30	75.510	75.5100

From Manning's formula, given by (A2-1), we can calculate Q values for various water heights, thus enabling us to make a Q vs. y_n graph. From Graph MD10, we see that corresponding to $n = 0.07, 0.09$ and 0.11 values, the y_n are

$$\left. \begin{array}{l} y_n^{(0.07)} = 17.8 \text{ feet} \\ y_n^{(0.09)} = 20.6 \text{ feet} \\ y_n^{(0.11)} = 23.3 \text{ feet} \end{array} \right\} \quad (\text{MH.15})$$

The length of the seventh reach is 4,000 feet and the average breadth is 25,000 feet.

Mulholland Reservoir

The flood plain for the Mulholland Reservoir (Hollywood Reservoir) was generated in eight reaches extending southerly to the Santa Monica Freeway. The number of expected deaths and the amount of damage (in dollars) to single-family dwellings for each reach is as follows:

1st reach:

expected # of deaths = ~ 3,130

damage to homes = ~ \$7,500,000

2nd reach:

expected # of deaths = ~ 6,090

damage to homes = ~ \$15,000,000

3rd reach:

expected # of deaths = ~ 16,660

damage to homes = ~ \$56,000,000

4th reach:

expected # of deaths = ~ 13,840

damage to homes = ~ \$85,700,000

5th reach:

expected # of deaths = ~ 22,890

damage to homes = ~ \$142,200,000

6th reach:

expected # of deaths = ~ 38,720

damage to homes = ~ \$151,700,000

7th reach:

expected # of deaths = ~ 59,290

damage to homes = ~ \$183,900,000

8th reach:

expected # of deaths = ~ 21,690

damage to homes = ~ \$80,400,000

Totals:

Expected # of deaths = ~ 182,310

damage to homes = ~ \$722,400,000

Figure 1. Constant Flow Channel Configuration @ 1" or 2000 Ft. Downstream.

Figures 2 and 3. Actual Dimensions for Constant Flow Channel Approximation:
Second and Third Reaches.

Figures 4 and 5. Actual Dimensions for Constant Flow Channel Approximation: Fourth and Fifth Reaches.

Figures 6 and 7. Actual Dimensions for Constant Flow Channel Approximation:
Sixth and Seventh Reaches.

Graph MD1a: Downstream Topography.

HORIZONTAL SCALE: 1" = 2000 FT.
VERTICAL SCALE: ELEVATION IN FEET

Graph MD1b: Downstream Topography.

HORIZONTAL SCALE: 1" = 2000 FT.
VERTICAL SCALE: ELEVATION IN FEET

Graph MD1c: Downstream Topography.

HORIZONTAL SCALE: 1" = 2000 FT.
VERTICAL SCALE: ELEVATION IN FEET

Graph MD1d: Downstream Topography.

BASE 1 = 937.5 FT.
 BASE 2 = 3640 FT.
 BASE 3 = 8250 FT.
 BASE 4 = 13375 FT.
 BASE 5 = 18500 FT.
 BASE 6 = 22800 FT.
 BASE 7 = 25000 FT.

Graph MD2. Downstream Topography.

Graph MD3. Height vs. Time Graph.

Graph MD4. Volume Rate of Flow vs. Normal Depth for First Reach.

Graph MD5. Volume Rate of Flow vs. Normal Depth for Second Reach.

Graph MD6. Volume Rate of Flow vs. Normal Depth for Third Reach.

Graph MD7. Volume Rate of Flow vs. Normal Depth for Fourth Reach.

Graph MD8. Volume Rate of Flow vs. Normal Depth for Fifth Reach.

Graph MD9. Volume Rate of Flow vs. Normal Depth for Sixth Reach.

Graph MD10. Volume Rate of Flow vs. Normal Depth for Seventh Reach.

J. MFP-1
Flood Plain Below the Mulholland Dam (Hollywood Reservoir)

MFP-2

Census Tract Map — Flood Plain Below Mulholland Dam (Hollywood Reservoir)

Upper San Leandro Dam

(Details below refer to topographical map)

* Dam Number	31 - 7
Name of Owner	East Bay M U District
County	Alameda
Stream	San Leandro Creek
Location	{ Section 16 Township 2S Range 2W Baseline and/or Meridian MD
Type	Hydraulic Fill
Storage Capacity	1.804958×10^9 cubic feet
Drainage Area	31.00 square miles
Reservoir Area	3.432528×10^7 square feet
Crest Elevation	475 feet
Crest Length	660 feet
Permanent Spillway	460 feet
Height of Dam	190 feet
Crest Thickness	75 feet
Dam Volume	1,248,000 cubic yards
Year of Completion	1926

* As assigned by Department of Water Resources

Chabot Dam

(Details below refer to topographical map)

* Dam Number	31 - 5
Name of Owner	East Bay M U District
County	Alameda
Stream	San Leandro Creek
Location	{ Section 30 Township 2S Range 2W Baseline and/or Meridian MD
Type	Earth
Storage Capacity	5.48856×10^8 cubic feet
Drainage Area	43.00 square miles
Reservoir Area	1.37214×10^7 square feet
Crest Elevation	245 feet
Crest Length	450 feet
Permanent Spillway	227 feet
Height of Dam	135 feet
Crest Thickness	35 feet
Dam Volume	622,000 cubic yards
Year of Completion	1892

* As assigned by Department of Water Resources

Calculation of $Q_{\max}$ and Q_{av} Under the Assumption of Complete and Instantaneous

Failure of Dam

In the case of Upper San Leandro Dam, we shall assume that the compacted earth completely erodes immediately after a breach in the concrete occurs, i.e., instantaneous complete dam failure.

From (AI-1),

$$Q_{\max}^{(1)} = \frac{2}{3} \sqrt{2gH^3}$$

Here,

$$H = 175 \text{ feet}$$

$$L = 600 \text{ feet}$$

Therefore,

$$Q_{\max}^{(1)} = 7.40 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \quad (\text{USL.1})$$

From (AI-2),

$$\begin{aligned} Q_{\max}^{(2)} &= \frac{1}{\sqrt{3}} Q_{\max}^{(1)} \\ &= 4.27 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \quad (\text{USL.2}) \end{aligned}$$

Unsteady Flow

Determination of Q at Time of Fill-Up of Chabot Dam Catchment

From Graphs USLD 1a and 1b, we see that all the water released from Upper San Leandro reservoir is funnelled into Lake Chabot. The most critical case occurs when the Lake Chabot Dam breaks immediately after the catchment reservoir has been filled to the brim.

Time at Which Water Fills the Chabot Reservoir

$$\text{Surface Area of Lake Chabot} = 13.72 \times 10^6 \text{ ft}^2$$

$$\text{Elevation of Water Height} = 227 \text{ feet}$$

$$\text{Elevation of Dam} = 245 \text{ feet}$$

$$\begin{aligned} \text{Volume of Water from} \\ \text{Spillway Elevation to} \\ \text{Top of Dam} &= 18 \times 13.72 \times 10^6 \text{ ft}^3 \\ &= 2.46 \times 10^8 \text{ ft}^3 \end{aligned} \quad (\text{USL.3})$$

On the other hand, if we consider this amount as the volume of water flow from the San Leandro reservoir required to fill up the Chabot Lake, then,

$$\begin{aligned} \text{Height of water at San Leandro} \\ \text{Dam corresponding to the time} \\ \text{of fill-up of Lake Chabot} &= h = 167.8 \text{ feet} \end{aligned} \quad (\text{USL.4})$$

This height has been calculated by the assumption that the volume of the immediate upstream topography and the water catchment of the San Leandro Dam can be approximated by a solid prismatic bowl.

We must now find the $Q(t)$ value, corresponding to the time of fill up of the Lake Chabot, which corresponds to the time at which the height behind the San Leandro Dam is approximately 168 feet.

We recognize that the instantaneous value of $Q(t)$, for an infinitesimal time interval dt , is given by

$$\begin{aligned} Q(t) &= \int_0^{h(t)} L \sqrt{2g(h(t) - \zeta)} \, d\zeta \\ &= \frac{2}{3} L \sqrt{2g} h^{3/2}(t) \end{aligned} \quad (\text{USL.5})$$

Here,

$$L = 600 \text{ feet}$$

$$h \approx 168 \text{ feet}$$

Therefore,

$$Q(t) = 6.96 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \quad (\text{USL.6})$$

With this known value of $Q(t)$, we shall attempt to apply the weir equation to the situation. First an approximate application, followed by an accurate one, will be attempted. The weir equation is

$$Q(t) = CLH^{\frac{3}{2}} \quad (\text{USL.7})$$

Assume

$$C \approx 3$$

Then, with $L = 450$ feet,

$$H^{\frac{3}{2}} = \frac{6.96 \times 10^6 \frac{\text{ft}^3}{\text{sec}}}{3 \times 450 \text{ ft}}$$

Therefore,

$$H = 298.6 \text{ ft} \quad (\text{USL.8})$$

This height over the weir is so high that it will overflow many of the lower elevations which currently confine the water. Therefore,

- a) the effective area of the lake will be increased and,
- b) the weir length will be increased.

However, an increased weir length assumption greatly diminishes the height over the weir, and it is a good approximation to ignore the effects of the area increase of the lake. On this basis, let us assume a weir length obtained from the 300 foot contour of the topography. This effective weir length is approximately 1120 feet and corresponds to a height over weir of 55 feet.

But from the weir equation,

$$\begin{aligned} H &= \left[\frac{6.96 \times 10^6}{3 \times 1120} \right]^{\frac{2}{3}} \\ &= 162.6 \text{ feet} \end{aligned} \quad (\text{USL.9})$$

This value of H does not correlate with our choice of a 55 foot height over the weir.

A second line of reasoning would be to examine the use of the critical depth volume rate of flow, given by

$$\begin{aligned}
 Q_{\max}^{(2)} &= \frac{1}{\sqrt{3}} Q_{\max}^{(1)} \\
 &= \frac{6.96 \times 10^6 \frac{\text{ft}^3}{\text{sec}}}{1.732} \\
 &= 4.02 \times 10^6 \frac{\text{ft}^3}{\text{sec}}
 \end{aligned}
 \tag{USL.10}$$

Then,

$$H = \{4.02 \times 10^6 / (3 \times 1120)\}^{2/3} = 112.76
 \tag{USL.11}$$

Again, there is no correlation with the choice of the 55 feet over the weir. It is thought, therefore, that a height over the weir exceeding this above value must be assumed. Such an assumption implies that the incoming water will overflow the natural boundary at the right end of the lake and flow down into the Castro Valley. This flow causes a double orifice flow problem.

Let, then, the height over the weir be 85 feet, implying that the water goes up to the 330 foot contour line. Further, we shall assume that the area of the lake remains the same. Although this assumption is highly questionable, the calculations are more tractable and largely indicate possible situations.

Since there are two weir openings over which the water may flow, we are faced with the case of dealing with two different Q values, one for each weir. Although the Q values may be different, still the heights over the weir must be the same. It is possible to obtain these Q values by a weighting procedure as shown below.

$$Q_{\text{dam as weir}}^* = \frac{1460 \times 6.96 \times 10^6}{1460 + 1360} \approx 3.6 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \quad (\text{USL.12})$$

$$Q_{\text{natural opening}}^* = \frac{1360 \times 6.96 \times 10^6}{1460 + 1360} \approx 3.36 \times 10^6 \frac{\text{ft}^3}{\text{sec}} \quad (\text{USL.13})$$

Therefore,

$$H_{\text{height over dam as weir}} = \left[\frac{3.6 \times 10^6}{3 \times 1460} \right]^{2/3} = 87.87 \text{ feet} \quad (\text{USL.14})$$

and,

$$H_{\text{natural opening with weir height above 245 foot elevation}} = \left[\frac{3.36 \times 10^6}{3 \times 1360} \right]^{2/3} = 87.85 \text{ feet} \quad (\text{USL.15})$$

In the above calculations, we have taken the orifice for the natural opening at 1,600 feet downstream from the lake, since this is the point which controls the flow of the water head, as can be seen from the sketch of downstream topography shown in Figure 2.

Since the assumed height over weir (85 feet) and the calculated height over weir (≈ 88 feet) are not very different, we may assume that the actual height over weir is about 87 feet, relative to the 245 foot elevation. Thus, the effective height of water behind the dam opening is

$$135 \text{ ft} + 87 \text{ ft} = 222 \text{ feet} \quad (\text{USL.16})$$

and the effective height of water behind the natural opening is

$$[(245 + 87) - 280] = 52 \text{ feet} \quad (\text{USL.17})$$

In the above, 245 feet is the crest elevation, 280 feet is the highest elevation of natural opening, at approximately 1,600 feet downstream, and 52 feet is the height of water above the 280 foot elevation.

Accurate Application of the Weir Equation

A fairly good estimate of the constant in the weir equation can be made since we know that we are dealing with a broad-crested weir, of trapezoidal cross-section and notch, having free discharge. Letting

$$C = 3.25,$$

and with,

$$Q(t) = CLH^{3/2}$$

$$\begin{aligned} H_{\text{height over dam as weir}} &= \left[\frac{3.6 \times 10^6}{3.25 \times 1460} \right]^{2/3} \\ &= 83.3 \text{ feet} \end{aligned} \quad (\text{USL.18})$$

and

$$\begin{aligned} H_{\text{height over natural opening as weir}} &= \left[\frac{3.36 \times 10^6}{3.25 \times 1360} \right]^{2/3} \\ &= 83.29 \text{ feet} \end{aligned} \quad (\text{USL.19})$$

These values suggest the effective height of water behind the dam opening is

$$135 \text{ ft} + 84 \text{ ft} = 219 \text{ ft} \quad (\text{USL.20})$$

while the effective height of water behind the natural opening is

$$245 \text{ ft} + 84 \text{ ft} - 280 \text{ ft} = 49 \text{ ft} \quad (\text{USL.21})$$

This 49 feet corresponds to the water height over the 280 foot elevation.

Consider next the Lake Chabot Dam. It was shown previously that the weir height, above the 245 foot crest elevation, is approximately 84 feet. The next step in the calculation procedure is to determine the average volume rate of flow (Q) out of each opening. This calculation will enable us to determine the normal depths for the flood plains. The following discussion is with reference to Figures 3, 4, 5.

For an infinitesimal time interval dt and infinitesimal volume element $d\zeta$, we may write,

$$dQ_t^{(1)} = L' \sqrt{2g(h'(t) - \zeta')} \quad (\text{USL.22})$$

and

$$dQ_t^{(2)} = L'' \sqrt{2g(h''(t) - \zeta'')} \quad (\text{USL.23})$$

Integrating over all the volume elements gives:

$$Q^{(1)}(t) = \int_0^{h'(t)} L' \sqrt{2g(h'(t) - \zeta')} d\zeta' \quad (\text{USL.24})$$

and

$$Q^{(2)}(t) = \int_0^{h''(t)} L'' \sqrt{2g(h''(t) - \zeta'')} d\zeta'' \quad (\text{USL.25})$$

The total volume rate of flow out of the reservoir for infinitesimal time interval dt at t is

$$Q(t) = Q_1(t) + Q_2(t) \quad (\text{USL.26})$$

$$= \left\{ \begin{array}{l} \int_0^{h'(t)} L' \sqrt{2g(h'(t) - \zeta')} d\zeta' + \\ \int_0^{h''(t)} L'' \sqrt{2g(h''(t) - \zeta'')} d\zeta'' \end{array} \right\} \quad (\text{USL.27})$$

$$= \frac{2}{3} L' \sqrt{2gh'^3(t)} + \frac{2}{3} L'' \sqrt{2gh''^3(t)} \quad (\text{USL.28})$$

But $h''(t) = h'(t) - 170$

Therefore,

$$Q(t) = \frac{2}{3} L' \sqrt{2gh'^3(t)} + \frac{2}{3} L'' \sqrt{2g(h'(t) - 170)^3} \quad (\text{USL.29})$$

The total amount of water leaving the lake for infinitesimal time interval dt , at t , is

$$Q(t) dt = \left\{ \frac{2}{3} L' \sqrt{2gh'^3(t)} + \frac{2}{3} L'' \sqrt{2g(h'(t) - 170)^3} \right\} dt$$

This must equal the volume decrease of the lake.

Therefore,

$$\begin{aligned} Q(t)dt &= -A(h'(t))dh'(t) \\ &= \left\{ \frac{2}{3} L' \sqrt{2gh'^3(t)} + \frac{2}{3} L'' \sqrt{2g(h'(t) - 170)^3} \right\} dt \end{aligned} \quad (\text{USL.30})$$

Rewriting this expression, we have,

$$dt = \frac{-A(h'(t))dh'(t)}{\frac{2}{3} L' \sqrt{2gh'^3(t)} + \frac{2}{3} L'' \sqrt{2g(h'(t) - 170)^3}} \quad (\text{USL.31})$$

The time T required to lower the water head from $H_0 = 219$ feet to $h' = 170$ feet, i.e., from 329 feet to 280 feet relative to the datum at zero elevation, is given by

$$\begin{aligned} T &= \int_0^T dt = \int_{H_0=219}^{h'=170} - \frac{A(h'(t))dh'(t)}{\left\{ \frac{2}{3} L' \sqrt{2gh'^3(t)} + \frac{2}{3} L'' \sqrt{2g(h'(t) - 170)^3} \right\}} \\ &= \int_{170}^{219} \frac{A(h'(t))dh'(t)}{\left\{ \frac{2}{3} L' \sqrt{2gh'^3(t)} + \frac{2}{3} L'' \sqrt{2g(h'(t) - 170)^3} \right\}} \end{aligned} \quad (\text{USL.32})$$

This, then, is the time necessary to lower the water head to the 280 foot elevation. Below this elevation, the single orifice problem must be solved. Equation USL.32 has been integrated numerically, and the value of T obtained from such an integration is approximately 69 seconds.

Figure 5 shows the details involved in the determination of the effective orifice width.

Determination of Q_{av} for the Natural Opening

Although we now know that the total time it takes to lower the water to the 280 foot elevation, we still have to calculate the volume of water flowing out of each orifice.

Assuming that the head over the weir (219 ft - 170 ft = 49 ft) is a constant over the entire lake, we have

$$\begin{aligned}
 V_{219 \text{ ft to } 170 \text{ ft}} &= 49 \text{ ft} \times 315 \text{ acres} \\
 &\text{(i.e., above 280} \\
 &\text{ft elevation)} \\
 &= 6.72 \times 10^8 \text{ ft}^3
 \end{aligned}$$

To determine how much of this volume is released through each orifice, we will consider the following weighting procedure:

$$\begin{aligned}
 V_{\text{dam opening}} &= \frac{Q_{\text{max dam opening}}^{(1)}}{Q_{\text{max dam opening}}^{(1)} + Q_{\text{max natural opening}}^{(1)}} V_{219 \text{ ft to } 170 \text{ ft}} \\
 &\text{219 ft} \\
 &\text{to 170 ft}
 \end{aligned} \tag{USL.33}$$

and

$$\begin{aligned}
 V_{\text{natural dam opening}} &= \frac{Q_{\text{max natural opening}}^{(1)}}{Q_{\text{max dam opening}}^{(1)} + Q_{\text{max natural opening}}^{(1)}} V_{219 \text{ ft to } 170 \text{ ft}} \\
 &\text{219 ft} \\
 &\text{to 170 ft}
 \end{aligned} \tag{USL.34}$$

where

$$\begin{aligned}
 Q_{\text{max dam opening}}^{(1)} &= \frac{2}{3} L'_{\text{eff}} \sqrt{2gh'}^3(t) \\
 &= \frac{2}{3} (644) \sqrt{(2)(32)(219)}^3 \\
 &= 1.131 \times 10^7 \frac{\text{ft}^3}{\text{sec}}
 \end{aligned} \tag{USL.35}$$

and

$$\begin{aligned}
\textcircled{1} \quad Q_{\text{max natural opening}} &= \frac{2}{3} L''_{\text{eff}} \sqrt{2gh''^3(t)} \\
&= \frac{2}{3} (960) \sqrt{(2)(32)(49)^3} \\
&= 1.756 \times 10^6 \frac{\text{ft}^3}{\text{sec}}
\end{aligned}
\tag{USL.36}$$

Using these equations, we find

$$\begin{aligned}
V_{\text{dam opening 219 ft to 170 ft}} &= \left(\frac{1.131 \times 10^7}{1.288 \times 10^7} \right) \cdot (6.72 \times 10^8) \\
&= 5.80 \times 10^8 \text{ ft}^3
\end{aligned}
\tag{USL.37}$$

and

$$\begin{aligned}
V_{\text{natural opening 219 ft to 170 ft}} &= \frac{1.75 \times 10^6}{1.28 \times 10^7} (6.72 \times 10^8) \\
&= 9.161 \times 10^7 \text{ ft}^3
\end{aligned}
\tag{USL.38}$$

From these calculations,

$$\begin{aligned}
Q_{\text{av natural opening}} &= \frac{9.161 \times 10^7 \text{ ft}^3}{69 \text{ secs}} \\
&= 1.328 \times 10^6 \frac{\text{ft}^3}{\text{sec}}
\end{aligned}
\tag{USL.39}$$

Determination of Time Necessary to Lower Water Head from 280 Foot Elevation to 110 Foot Elevation

We realize that this is the same as reducing the water head height from 170 feet to zero since we consider 110 feet as ground level for the dam opening.

Since the head controlling flow is below the 280 foot elevation, we have the simple, single orifice problem.. A fairly accurate representation of the situation can be seen in Figure 5. From the appendix, we have

$$\begin{aligned}
 T &= \int_0^T dt = \int_H^0 \frac{-A(h) dh}{\int_0^{h(t)} L \sqrt{2g(h(t) - \zeta)} d\zeta} \\
 &= \int_{h''}^H \frac{A(h) dh}{\int_0^{h(t)} L \sqrt{2g(h(t) - \zeta)} d\zeta} + \int_0^{h''} \frac{A(h) dh}{\int_0^{h(t)} L \sqrt{2g(h(t) - \zeta)} d\zeta}
 \end{aligned}
 \tag{USL.40}$$

$$\begin{aligned}
 T &= \int_{h''}^H \frac{A_o dh}{\int_0^{h(t)} L \sqrt{2g(h(t) - \zeta)} d\zeta} + \int_0^{h''} \frac{A_o h^2(t) dh}{\int_0^{h(t)} h''^2 L \sqrt{2g(h(t) - \zeta)} d\zeta}
 \end{aligned}
 \tag{USL.41}$$

Here,

$$\left. \begin{aligned}
 h'' &= 117 \text{ feet} \\
 L &= 644 \text{ feet} \\
 A_o &= 13.72 \times 10^6 \text{ ft}^2 \\
 H &= 170 \text{ feet}
 \end{aligned} \right\}
 \tag{USL.42}$$

We note that the first part of this above sum is obtained from a solid prismatic bowl model assumption while the latter is a consequence of the square cross-sectional model theory.

Therefore,

$$T = 372.03 \text{ seconds}
 \tag{USL.43}$$

Determination of Q_{av} for the Dam Opening

We now know that it takes 69 seconds to lower the driving head from a 329 foot to a 280 foot elevation, during which time $5.80 \times 10^8 \text{ ft}^3$ of water

empties out of the dam orifice. Also, we know that it takes an additional 372 seconds to lower the driving head from a 280 foot elevation to ground level (110 foot elevation), during which time,

$$(727.23 \times 10^6 \text{ ft}^3 + 535.11 \times 10^6 \text{ ft}^3) = 1262.35 \times 10^6 \text{ ft}^3 \text{ of water empties out of the dam opening.}$$

Therefore, the total amount of water which empties out of the dam orifice is

$$(1.262 \times 10^9 + 5.80 \times 10^8) \text{ ft}^3 = 1.84 \times 10^9 \text{ ft}^3 \quad (\text{USL.44})$$

The total time needed to empty this water is 441 seconds.

Thus,

$$Q_{av} = \frac{V_{total}}{T} = \frac{1.84 \times 10^9}{441} = 4.18 \times 10^6 \frac{\text{ft}^3}{\text{sec}}$$

Estimation of Dimensions of Constant Flow Channel for Dam Opening

First Reach (0 - 8000 Feet)

The flood plain calculations are started at 4000 feet even though the length of the first reach extends from 0 to 8000 feet. This is because water is channelized in a narrow cross sectional flow until about the first 4000 feet (see Graph LCD 1a and flood plain map).

I. Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the first reach by a trapezoid with dimensions determined at the middle of that reach, i.e., at 6000 feet.

2. Assume that an allowance of 30° , measured from the lines normal to both the left and right ends of the flow channel at 4,000 feet to the left of the dam, will account for natural divergent flow of water out of the confined channel.

3. Assume an additional 15° allowance, added on to the right side and subtracted from the left side of the flow channel, will account for the slightly dominant slope to the right side. (See Graph LCD 1b and flood plain map.)

Figure 6 shows the actual dimensions for the approximation of constant flow channel of dam opening, over the first reach.

Determination of Parameters B_1 , B_2 , C , A , R When $B = 4000$ Feet and Normal

Depth = y_n

1. B

$$B_1 = B_2 = \frac{y_n}{\sin 10^\circ}$$

2. C

$$\begin{aligned} C &= B + 2B_2 \cos 10^\circ \\ &= 4000 + 2y_n \cot 10^\circ \end{aligned}$$

3. A

$$A = \frac{C + B}{2} y_n$$

4. P

$$\begin{aligned} P &= B_1 + B + B_2 \\ &= 2B_1 + B \\ &= \frac{2y_n}{\sin 10^\circ} + 4000 \end{aligned}$$

5. R

$$R = \frac{A}{P}$$

Values of Parameters for Water Heights

Slope = 0.00566

Base = 4000 Feet

y_n (ft)	$A \times 10^{-4}$ (ft ²)	R (ft)
0	0	0
4	0.8022	3.7969
8	3.2363	7.9086
12	4.8817	9.8579
16	6.5452	15.6420
20	8.2268	19.4470
24	9.9266	23.2130
28	11.6450	26.7400
32	13.3810	30.6300
36	15.1350	34.2840
40	16.9070	37.9030
44	18.6980	41.4890
48	20.5070	45.0410
52	22.3330	48.5630
56	24.1780	52.0530
60	26.0420	55.5130
64	27.9230	58.9450
68	29.8220	62.3480
70	30.7790	64.0400

From Manning's formula (A2-1), we can calculate Q values for various water depths, thus enabling us to make a Q vs. y_n graph. From LCD5, we conclude that:

Length of first reach: 8,000 feet

$$\left. \begin{array}{l} y_n^{(0.07)} = 48.3 \text{ ft} \\ y_n^{(0.09)} = 56.2 \text{ ft} \\ y_n^{(0.11)} = 63.2 \text{ ft} \end{array} \right\} \quad (\text{USL.45})$$

Base width = 4000 ft

Estimation of Dimensions of Constant Flow Channel for Dam Opening

Second Reach (8000 - 12,000 Feet)

I. Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the second reach by a trapezoid with dimensions determined at the middle of that reach (i.e., at 10,000 feet).

2. Assume that we may continue the net allowances of 15° , on the left side, and 45° , on the right side of flow channel, allowed for the natural divergent flow of water as well as the lateral sloping of the topography. (See Graphs LCD 1b and 1c and also the flood plain map.) Figure 7 shows the actual dimensions for approximation of the constant flow channel of dam opening for the second reach.

Determination of Parameters B_1 , B_2 , C, A, R When $B = 9000$ Feet and Normal

Depth y_n

1. B

$$B_1 = B_2 = \frac{y_n}{\sin 10^{\circ}}$$

2. C

$$\begin{aligned} C &= B + 2B_2 \cos 10^{\circ} \\ &= 9000 + 2y_n \cot 10^{\circ} \end{aligned}$$

3. A

$$A = \frac{C + B}{2} y_n$$

4. P

$$\begin{aligned}
 P &= B_1 + B + B_2 \\
 &= 2B_1 + B \\
 &= \frac{2y_n}{\sin 10^\circ} + 9000
 \end{aligned}$$

5. R

$$R = \frac{A}{P}$$

Values of Parameters for Various Water Heights

Slope = 0.0056 Base = 9000 Feet

<u>y_n</u> <u>(ft)</u>	<u>A x 10⁻⁴</u> <u>(ft)</u>	<u>R</u> <u>(ft)</u>
0	0	0
4	3.6091	3.9897
8	7.2363	7.9588
12	10.8820	11.9080
16	14.5450	15.8370
20	18.2270	19.7470
24	21.9270	23.6370
28	25.6450	27.5080
32	29.3870	31.3610
36	35.0190	35.1950
40	36.9070	39.0110

From Manning's formula (A2-1), we can calculate Q values for various water depths, thus enabling us to make a Q vs. y_n graph. From LCD6, we conclude that:

Length of second reach: 4,000 feet

$$y_n^{(0.07)} = 30.0 \text{ ft}$$

$$y_n^{(0.09)} = 34.9 \text{ ft}$$

$$y_n^{(0.11)} = 39.0 \text{ ft}$$

(USL.46)

Base Width = 9000 ft

Estimation of Dimensions of Constant Flow Channel for Dam Opening

Third Reach (12,000 - 16,000 Feet)

I. Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the third reach by a trapezoid with dimensions determined at the middle of that reach, i.e., at 14,000 feet.

2. Assume that we may continue the net allowances of 15° , on the left side, and 45° , on the right side of the flow channel, allotted for the natural divergent flow of water and the lateral sloping of the topography.

Figure 8 shows the actual dimensions of the constant flow channel of dam opening for the third reach.

Determination of Parameters B_1 , B_2 , C, A, R When $B = 14,000$ Feet and Normal

Depth = y_n

1. B

$$B_1 = B_2 = \frac{y_n}{\sin 10^\circ}$$

2. C

$$\begin{aligned} C &= B + 2B_2 \cos 10^\circ \\ &= 14,000 + 2y_n \cot 10^\circ \end{aligned}$$

3. A

$$A = \frac{C + B}{2} y_n$$

4. P

$$\begin{aligned}
 P &= B_1 + B + B_2 \\
 &= 2B_1 + B \\
 &= \frac{2y_n}{\sin 10^\circ} + 14,000
 \end{aligned}$$

5. R

$$R = \frac{A}{P}$$

Values of Parameters for Various Water Heights

Slope = 0.00566		Base = 14,000 Feet	
y_n (ft)	$A \times 10^{-4}$ (ft ²)	R (ft)	
0	0	0	
4	5.6091	3.9933	
8	11.2360	7.9734	
12	16.8820	11.9400	
16	22.5450	15.8940	
20	28.2270	19.8360	
24	33.9270	23.7640	
28	39.6450	39.6450	
32	45.3810	45.3810	
36	51.1350	51.1350	
40	56.9070	56.9070	

From Manning's formula (AI-2), we can calculate Q values for various water heights, thus enabling us to make a Q vs. y_n graph. From LCD7, we conclude that:

Length of Third Reach: 4,000 Feet

$$\left. \begin{aligned}
 y_n^{(0.07)} &= 23.0 \text{ ft} \\
 y_n^{(0.09)} &= 26.8 \text{ ft} \\
 y_n^{(0.11)} &= 30.3 \text{ ft}
 \end{aligned} \right\} \text{Base Width} = 14,000 \text{ Feet} \quad (\text{USL.47})$$

Estimation of Dimensions of Constant Flow Channel for Dam Opening

Fourth Reach (16,000 - 20,000 Feet)

I. Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the fourth reach by a trapezoid with dimensions determined at the middle of the reach, i.e., at 18,000 feet.

2. Assume that we may continue the net allowance of 15° , on the left side, and 45° , on the right side of the flow channel, allotted for the natural divergent flow of water and lateral sloping of the topography. (See Graph LCD 1c and also the flood plain map.)

Figure 9 shows the dimensions for approximation of the constant flow channel of dam opening for the fourth reach.

Determination of Parameters B_1 , B_2 , C, A, R

1. B

$$B_1 = B_2 = \frac{y_n}{\sin 10^\circ}$$

2. C

$$\begin{aligned} C &= B + 2B_2 \cos 10^\circ \\ &= 19,000 + 2y_n \cot 10^\circ \end{aligned}$$

3. A

$$A = \frac{C + B}{2} y_n$$

4. P

$$P = \frac{2y_n}{\sin 10^\circ} + 19,000$$

5. R

$$R = \frac{A}{P}$$

Values of Parameters for Various Water Heights

Slope = 0.00566 Base = 19,000 Feet

y_n (ft)	$A \times 10^{-4}$ (ft ²)	R (ft)
0	0	0
4	7.6091	3.9951
8	15.2360	7.9804
12	22.8820	11.9560
16	30.5450	15.9220
20	38.2270	19.8780
24	45.9270	23.8250
28	53.6450	27.7630
32	61.3810	31.6910
36	69.1350	35.6100
40	76.9070	39.5190
44	84.6980	43.4200
48	92.5070	47.3110
50	96.4180	49.2530

From Manning's formula (A2-1), we can calculate Q values for various water heights, thus enabling us to make a Q vs. y_n graph. From LCD 8, we conclude that

Length of Fourth Reach: 4,000 Feet

$$\left. \begin{array}{l} y_n^{(0.07)} = 19.2 \text{ ft} \\ y_n^{(0.09)} = 22.0 \text{ ft} \\ y_n^{(0.11)} = 25.2 \text{ ft} \end{array} \right\} \quad (\text{USL.48})$$

Base Width = 19,000 Feet

Estimation of Dimensions of Constant Flow Channel for the Natural Opening

First Reach (0 - 6000 Feet)

We note that since the effective orifice has been taken at 1600 feet downstream of the natural opening, it is enough to consider the first reach as starting at 2000 feet, realizing that all areas prior to this length will be completely flooded.

I. Assumptions Made

1. Assume that we can approximate the constant flow channel for the first reach by a trapezoid with dimensions determined at the middle of that reach, i.e., at 4000 feet.

2. Assume that water is confined within a relatively narrow channel until just above 4000 feet, where the confinement on the right side must be expanded by another 2000 feet. (Note: Since Graph LCD 2a gives the topography only at 2000 feet and 4000 feet, we should look at the flood plain to see the confinement of water between 2000 and 4000 feet.)

3. Assume that an allowance of 30° , added on to both left and right ends of the flow channel just above 4000 feet, will account for the natural divergent flow of water out of the confined channel.

4. Assume that between 4000 and 5000 feet downstream, water is again confined on the right. (See Graph LCD 2a and flood plain map.)

Figure 11 shows the actual dimensions for the constant flow channel of natural opening for the first reach.

Determination of Parameters B_1 , B_2 , C, A, R When $B = 1860$ Feet and Normal

Depth = y_n

1. B

$$B_1 = B_2 = \frac{y_n}{\sin 45^\circ}$$

2. C

$$\begin{aligned} C &= B + 2B_2 \cos 45^\circ \\ &= 1860 + 2y_n \cot 45^\circ \end{aligned}$$

3. A

$$A = \frac{C + B}{2} y_n$$

4. P

$$P = \frac{2y_n}{\sin 45^\circ} + 1860$$

5. R

$$R = \frac{A}{P}$$

Values of Parameters for Various Water Heights

Slope = 0.026 Base = 1860 Feet

<u>y_n</u> <u>(ft)</u>	<u>A x 10⁻⁴</u> <u>(ft²)</u>	<u>R</u> <u>(ft)</u>
0	0	0
4	0.7456	3.9840
8	1.4944	7.9380
12	2.2464	11.8610
16	3.0016	15.7540
20	3.7600	19.6180
24	4.5216	23.4540
28	5.2864	27.2610
32	6.0544	31.0400
36	6.8256	34.7920
40	7.6000	38.5170

From Manning's formula (A2-1), we can calculate Q values for various water heights, thus enabling us to make a Q vs. y_n graph. From LCD 9, we conclude that:

Length of First Reach for Natural Opening: 6,000 Feet

$$\left. \begin{array}{l} y_n^{(0.07)} = 24.7 \text{ ft} \\ y_n^{(0.09)} = 28.7 \text{ ft} \\ y_n^{(0.11)} = 32.4 \text{ ft} \end{array} \right\} \quad (\text{USL.49})$$

Base Width = 1860 Feet

Estimation of Dimensions of Constant Flow Channel for Natural Opening

Second Reach (6,000 - 10,000 Feet)

I. Assumptions Made

1. Assume that we can approximate the constant flow channel configuration for the second reach by a trapezoid with dimensions determined at the middle of the second reach, i.e., at 8000 feet.

2. Assume that we may continue the 30° allowance on the left side, allotted for the natural divergent flow of water out of the confined channel (see Graphs LCD 2a and b and the flood plain map as well).

3. Assume that the water is still confined on the right side (see Graphs LCD 2a and b and flood plain map).

Figure 12 shows the actual dimensions for the approximation of the constant flow channel for the second reach of the natural opening.

Determination of Parameters B_1 , B_2 , C, A, R When $B = 5460$ Feet and Normal

Depth = y_n

1. B

$$B_1 = B_2 = \frac{y_n}{\sin 10^\circ}$$

2. C

$$\begin{aligned} C &= B + 2B_2 \cos 10^\circ \\ &= 5460 + 2y_n \cot 10^\circ \end{aligned}$$

3. A

$$A = \frac{C + B}{2} y_n$$

4. P

$$P = \frac{2y_n}{\sin 10^\circ} + 5460$$

5. R

$$R = \frac{A}{P}$$

Values of Parameters for Various Water Heights

Slope = 0.0075

Base = 5460 Feet

<u>y_n</u> <u>(ft)</u>	<u>A x 10⁻⁴</u> <u>(ft²)</u>	<u>R</u> <u>(ft)</u>
0	0	0
4	2.1931	3.9830
8	4.4043	7.9320
12	6.6337	11.8500
16	8.8812	15.7350
20	11.1470	19.5890
24	13.4310	23.4130
28	15.7330	27.2070
30	16.8900	29.0940

From Manning's formula (A2-1), we can calculate Q values for various water heights, thus enabling us to make a Q vs. y_n graph. From LCD 10, we conclude that:

Length of Second Reach for Natural Opening: 4,000 Feet

$$\left. \begin{array}{l} y_n^{(0.07)} = 18.7 \text{ ft} \\ y_n^{(0.09)} = 21.8 \text{ ft} \\ y_n^{(0.11)} = 24.5 \text{ ft} \end{array} \right\} \quad (\text{USL}, 50)$$

Base Width = 5460 Feet

Expected Number of Deaths and Damage from a Coupled Failure of the Upper San
Leandro and Lake Chabot Dams

The postulated coupled failure of these two dams results in a situation unique among the dams studied thus far. Two flood plains are actually created due to the overtopping of a ridge at one end of Chabot Reservoir and subsequent flooding of the Castro Valley.

Designating the flood plain below the Chabot dam face as area A and the flood plain in the Castro Valley as area B, we note the following consequences:

Area A

1st Reach

Expected number of deaths

Daytime: 2500

Nighttime: 3870

Amount of damage

Single unit homes destroyed: 1369

Value: \$10,152,000

2nd Reach

Expected number of deaths

Daytime: 5700

Nighttime: 8790

Amount of damage

Single unit homes destroyed: 2243

Value: \$28,000,000

3rd Reach

Expected number of deaths

Daytime: 14,145

Nighttime: 20,630

Amount of damage

Single-unit homes destroyed: 4860

Value: \$47,737,000

4th Reach

Expected number of deaths

Daytime: 7835

Nighttime: 11,800

Amount of damage

Single-unit homes destroyed: 2828

Value: \$119,900,000

Total for Area A

Expected number of deaths

Daytime: 30,190

Nighttime: 45,090

Area B

1st Reach

Expected number of deaths

Daytime: 2230

Nighttime: 3815

Amount of damage

Single-unit homes destroyed: 1116

Value: \$15,656,000

2nd Reach

Expected number of deaths

Daytime: 3460

Nighttime: 5740

Amount of damage

Single-unit homes destroyed: 1323

Value: \$16,614,000

Total for Area B

Expected number of deaths

Daytime: 5690

Nighttime: 9560

Amount of damage

Single-unit homes destroyed: 2439

Value: \$32,270,000

Totals for areas A and B

Expected number of deaths

Daytime: 35,880

Nighttime: 54,640

Amount of damage

Single-unit homes destroyed: 13,747

Value: \$152,170,000

Figures 1 and 2. Upper San Leandro and Lake Chabot Dams.

Figures 3,4 and 5. Determination of Time Necessary to Lower Water Heat to 110 Ft. Elevation Eventually.

DAM OPENING
FLOW CHANNELS

Figures 6 and 7. Actual Dimensions: Constant Flow Channel Approximation for Dam Opening First and Second Reaches.

Figures 8,9 and 10. Actual Dimensions: Constant Flow Channel Approximation for Dam Opening Third and Fourth Reaches, Also, Natural Opening Sketch.

TRAPEZOID @ 4000'

TRAPEZOID @ 8000'

NATURAL OPENING FLOW CHANNELS

Figures 11 and 12. Actual Dimensions for Constant Flow Channel Approximation: Natural Opening. First and Second Reaches.

Graph USLD1a: Downstream Topography.

Graph USLD1b: Downstream Topography.

Graph LCD1a: Topography: Dam Opening.

Graph LCD1b: Downstream Topography: Dam Opening.

Graph LCD1c: Downstream Topography: Dam Opening.

Graph LCD2a: Downstream Topography: Natural Opening.

Graph LCD2b: Downstream Topography: Natural Opening.

Graph LCD3. Downstream Slope: Dam Opening.

Graph LCD4. Downstream Slope: Natural Opening.

Graph LCD5. Volume Rate of Flow vs. Normal Depth: First Reach: Dam Opening.

Graph LCD6. Volume Rate of Flow vs. Normal Depth: Second Reach: Dam Opening.

Graph LCD7. Volume Rate of Flow vs. Normal Depth: Third Reach: Dam Opening.

Graph LCD8. Volume Rate of Flow vs. Normal Depth: Fourth Reach: Dam Opening.

Graph LCD9. Volume Rate of Flow vs. Normal Depth: First Reach: Natural Opening.

Graph LCD10. Volume Rate of Flow vs. Normal Depth: Natural Opening: Second Reach.

D: SLFP-1
Flood Plain Below the Upper San Leandro and Lake Chabot Reservoirs