INVESTIGATION OF THE RESPONSE OF PUDDINGSTONE DAM IN THE WHITTIER NARROWS EARTHQUAKE OF OCTOBER 1, 1987

by

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PREFACE

The California Strong Motion Instrumentation Program (CSMIP) in the Division of Mines and Geology of the California Department of Conservation promotes and facilitates the improvement of seismic codes through the Data Interpretation Project. The objective of the this project is to increase the understanding of earthquake strong ground shaking and its effects on structures through interpretation and analysis studies of CSMIP and other applicable strong motion data. The ultimate goal is to accelerate the process by which lessons learned from earthquake data are incorporated into seismic code provisions and seismic design practices.

The specific objectives of the CSMIP Data Interpretation Project are to:

1. Understand the spatial variation and magnitude dependence of earthquake strong ground motion.

2. Understand the effects of earthquake motions on the response of geologic formations, buildings and lifeline structures.

3. Expedite the incorporation of knowledge of earthquake shaking into revision of seismic codes and practices.

4. Increase awareness within the seismological and earthquake engineering community about the effective usage of strong motion data.

5. Improve instrumentation methods and data processing techniques to maximize the usefulness of SMIP data. Develop data representations to increase the usefulness and the applicability to design engineers.

This report is the fourth in a series of CSMIP data utilization reports designed to transfer recent research findings on strong-motion data to practicing seismic design professionals and earth scientists. CSMIP extends its appreciation to the members of the Strong Motion Instrumentation Advisory Committee and its subcommittees for their recommendations regarding the Data Interpretation Research Project.

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ABSTRACT

The Whittier Narrows Earthquake of October 1, 1987 ($M_L \approx 5.9$) shook Puddingstone Dam, a primarily cohesive, homogeneous section, compacted earth dam which had previously been instrumented as part of the California Strong Motion Instrumentation Program (CSMIP). The resulting maximum (transverse, horizontal) crest acceleration was 0.19 g, and maximum accelerations recorded at abutment stations were on the order of 0.04 to 0.08 g. The resulting recorded accelograms provided a valuable opportunity to investigate and evaluate the accuracy and reliability of conventional geotechnical procedures for evaluation of dynamic response characteristics of earth and rockfill dams. Presented in this report are the results of dynamic analysis studies of the response of Puddingstone Dam to the 1987 Whittier Narrows Earthquake performed using both (a) simple one-dimensional columnar analyses, and (b) two-dimensional (plane strain) dynamic finite element analyses. Nonlinear, strain-dependent dynamic shear moduli and damping characteristics were modelling using the "equivalent linear" method. Nonlinear modulus degradation and damping relationships for the compacted sandy silty clay which comprises a majority of the embankment were modelled based on the relationships proposed by Sun et al. (1988) for clays of low plasticity. The results of the two-dimensional finite element analyses were found to be in good agreement with the observed (recorded) field response, providing good support for these modelling and analysis techniques. The simpler, one-dimensional columnar analyses were found to significantly underestimate the crest response, as a result of their inability to model geometric effects or topographic amplification, but were also found to produce reasonably good agreement between calculated and recorded response at the center of the downstream face.

ACKNOWLEDGEMENTS

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Chapter 1

INTRODUCTION

Much attention has been given over the past twenty years to methods of analyzing the response of earth dams to earthquake shaking. The applicability and reliability of these analytical procedures, however, can only be meaningfully evaluated when the results of analyses are compared with the observed response of full scale prototype earth and rockfill dams during actual earthquakes, or with carefully conducted experimental observations of the response of small-scale models of such dams. An excellent opportunity to check the accuracy and reliability of dynamic analysis procedures for evaluating the seismic response of embankment dams has recently been provided by the excellent response data recorded on the Puddingstone Dam during the Whittier Narrows Earthquake of October 1, 1987 through the State of California Strong Motion Instrumentation Program (CSMIP).

Figure 1-1 shows the location of Puddingstone Dam, which is located in Los Angeles County, California, about 15 miles east of the City of Los Angeles. As shown in Figure 1-2, Puddingstone Dam actually consists of three earth dams and a concrete spillway. The main dam (Dam No. 1) is a rolled earth fill embankment with a maximum height of 148 feet and a crest length of 1,085 feet. Two smaller saddle dams (Dams No. 2 and 3) with heights of 49.5 and 60 feet, respectively, are also rolled earth fill embankments and also serve to retain the reservoir. This study concerns only the main dam (Dam No. 1).

Puddingstone Dam is situated in a seismically active area. As shown in Figure 1-3, the dam is bounded by several major fault systems. The San Andreas Fault Zone is located about 20 miles from the dam site, and the Sierra Madre Fault is located only about 2 miles from Puddingstone Dam. These two faults are considered capable of generating maximum credible earthquakes of magnitude $M_L \approx 8$ to $8\frac{1}{2}$ and magnitude

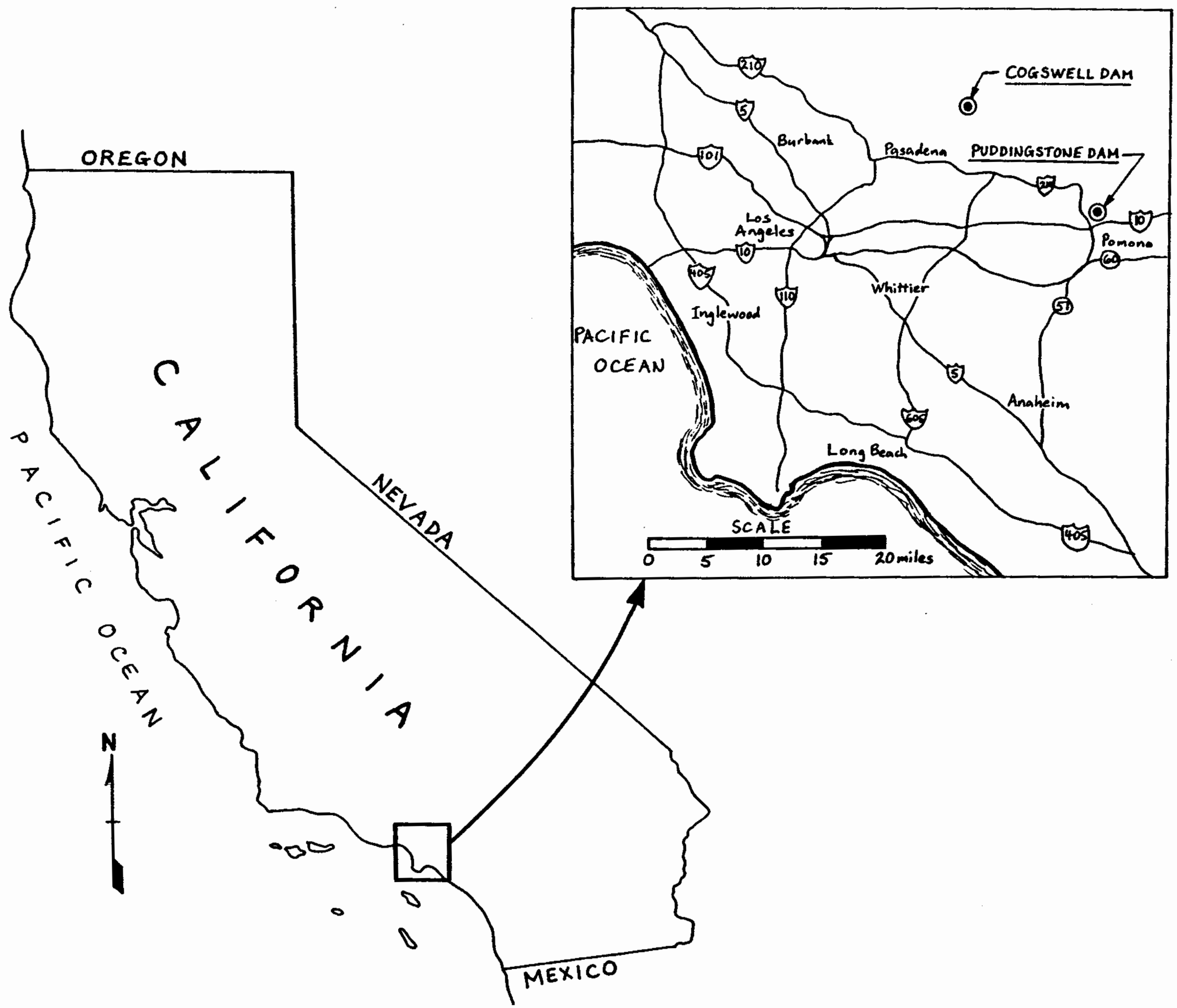


Figure 1-1: LOCATION OF PUDDINGSTONE DAM

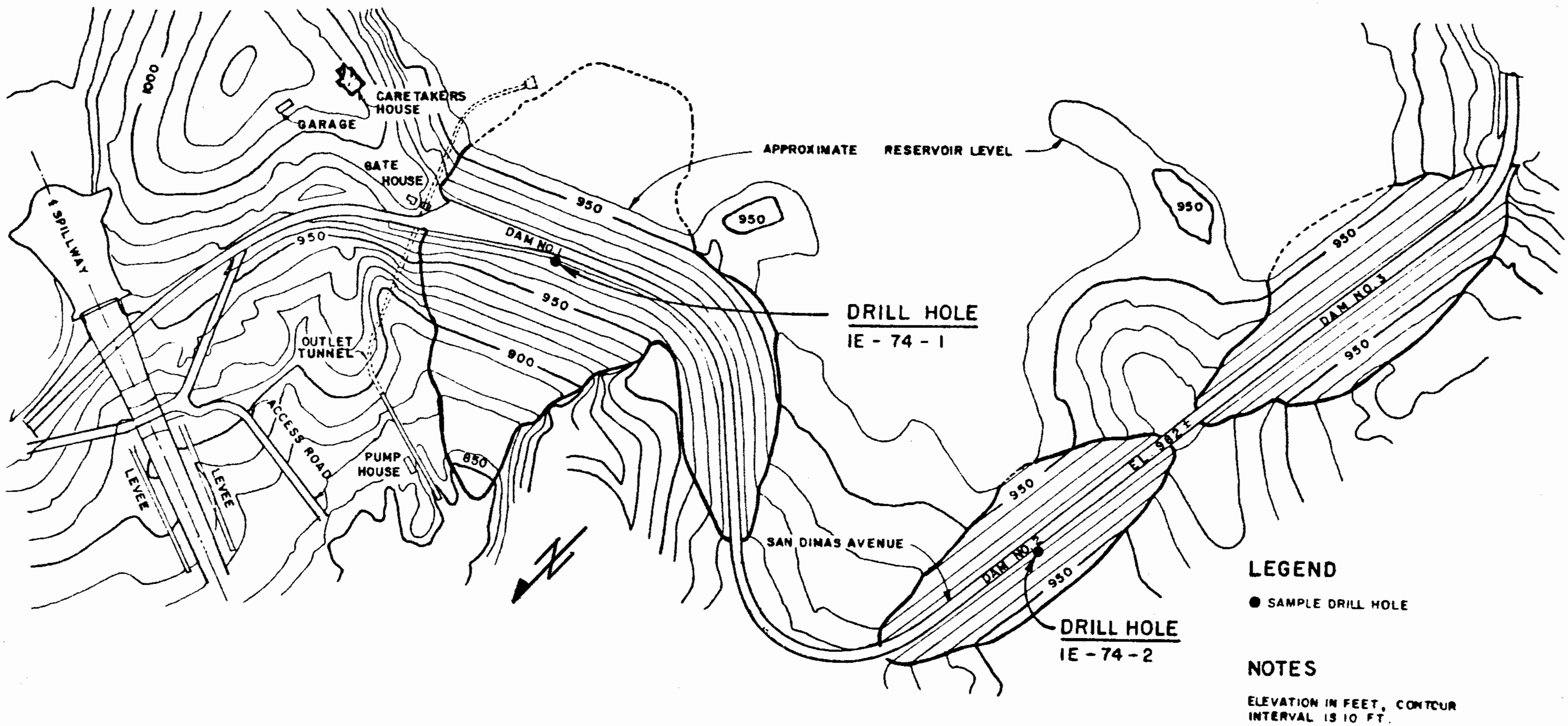


Figure 1-2: PLAN VIEW OF PUDDINGSTONE DAM (After International Engineering Co., 1973)

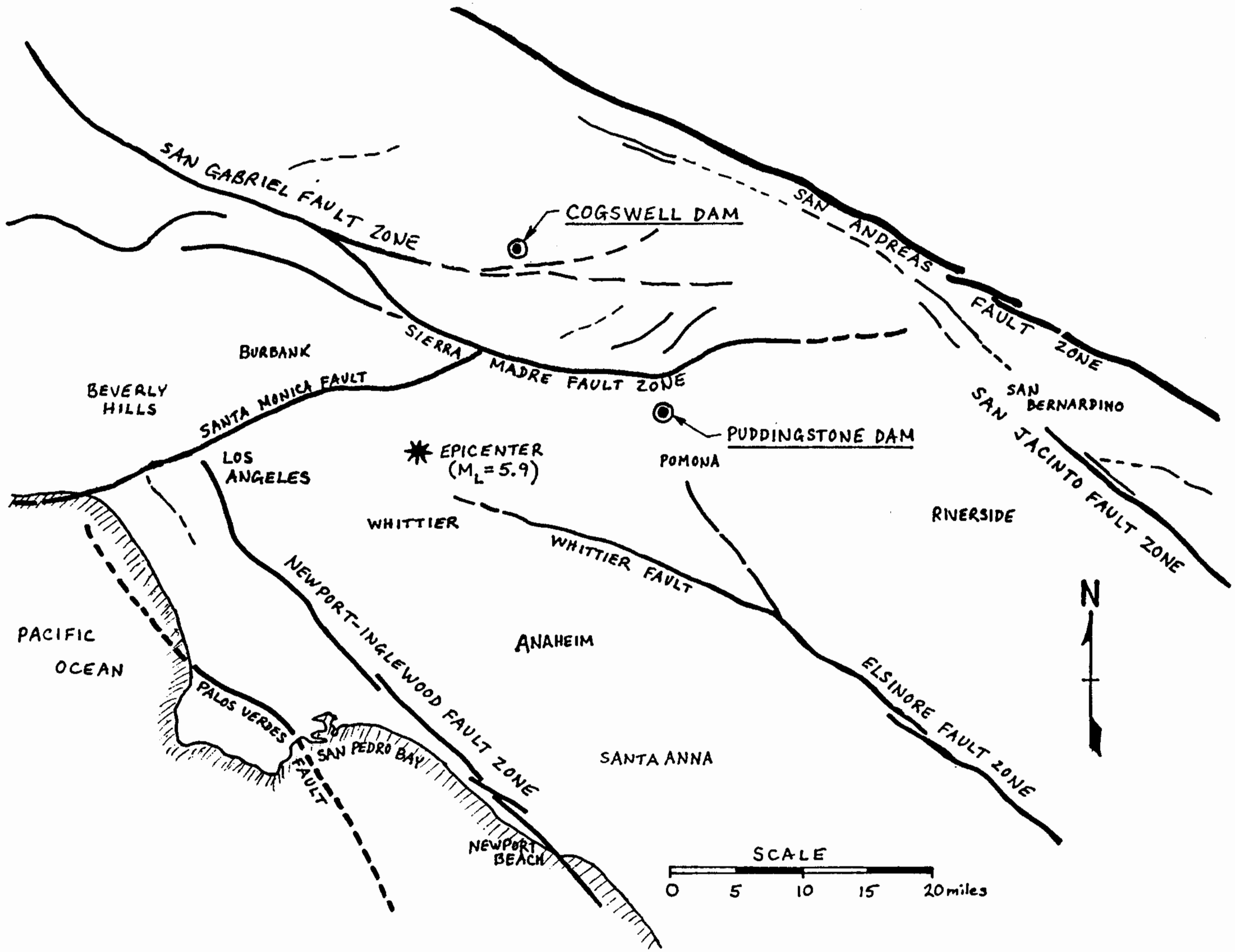


Figure 1-3: FAULTS IN THE VICINITY OF PUDDINGSTONE DAM

$M_L \approx 7$ to $7\frac{1}{2}$, respectively. Because of the area's high seismicity, Puddingstone Dam was selected by the State of California Strong Motion Instrumentation Program for comprehensive instrumentation to investigate the dynamic response of earth dams to strong earthquake shaking.

In this study, empirical relationships and one- and two-dimensional dynamic response analysis methods are used to study and to predict the observed response of Puddingstone Dam to the Whittier Narrows Earthquake of October 1, 1987. This earthquake, with a magnitude of $M_L \approx 5.9$, occurred on a previously unrecognized segment of the Whittier Fault with a epicenter approximately 18 miles west of the dam, as shown in Figure 1-3. This event produced maximum bedrock accelerations of approximately 0.08 g at the dam site, and the maximum acceleration recorded at the crest of the main dam was 0.19 g. Comparison between the observed (recorded) response characteristics and the results of dynamic response analyses provides a basis for improving our understanding of the seismic response of earth dams, and serves to validate the current use of some of these analytical models and analysis techniques. Specifically, this study attempts to evaluate the predominant period of the Puddingstone Dam, to predict the peak acceleration and maximum spectral acceleration at the crest and central downstream face of the embankment, to predict the correct shape of the acceleration response spectra at these two locations, and to estimate the actual dynamic properties of the soil materials comprising the main dam.

Chapter Two presents a description of the Puddingstone Dam and a discussion of the characteristics and engineering properties of the soils in the main embankment dam. Chapter Three presents a description of the instrumentation system installed by CSMIP to record strong motion response data for the dam and abutments, as well as the processed response data obtained during the Whittier Narrows Earthquake of October 1, 1987. Numerical modelling and seismic response analyses are described in Chapter Four, and the results of analyses performed using one-dimensional and two-dimensional response analysis techniques are compared with observed field response data. Chapter Five presents a brief summary of the results of these studies and the principal conclusions drawn from them.

Chapter 2

DESCRIPTION OF PUDDINGSTONE DAM

Puddingstone Dam, as shown in Figure 1-2, consists of three earth embankment dams which are numbered from 1 to 3 from east to west. A concrete spillway is located to the east of the main dam (Dam No. 1). This series of dams retains Puddingstone Reservoir, which has a design storage capacity of 17,190 acre-feet. The dam was completed in 1928, and it is now owned and operated by the Los Angeles County Flood Control District. Although Puddingstone Dam, with its crest elevation of 983.5 feet, was designed to be operated with a normal water surface elevation of 970 feet, the State of California Division of Safety of Dams has restricted the maximum normal reservoir elevation to 945 feet, with temporary storage above Elevation 945 permitted for flood control only.

The principal features and characteristics of the Puddingstone Dam system are summarized in Table 2-1. Of the three embankment dams, Dam No. 1 is the largest and is considered to be the main dam. A cross section through the maximum height section of Dam No. 1 is presented in Figure 2-1. The main dam has a maximum crest height of 148 feet and a crest length of 1,085 feet. Dams No. 2 and No. 3 have crest heights of 49.5 feet and 60.0 feet, respectively, and crest lengths of 785 feet and 823 feet, respectively. Each of these dams is a rolled earth embankment with concrete slope protection on the upstream face. The main dam also has a concrete core wall to cutoff seepage at the contact between the base of the core and the underlying foundation. Dam No. 1 is drained through a triangular toe drain section composed mainly of large boulders. All three dams have crest widths of 25 feet. Dam No. 1 has upstream slopes of 2.5 H:1V in its top 70 feet. The upstream slope reduces to 3 H:1V in the next 25 feet with the remainder of the dam at a slope of 3.5 H:1V. Dam No. 1 has downstream slopes of 2.5 H:1V in the top 97 feet and 3 H:1V below a 35 foot berm which separates

Table 2-1: Puddingstone Dam: Project Description

Summary: Puddingstone Dam is composed of 3 rolled earth fill embankment dams, a spillway, outlet works, and retains the Puddingstone Reservoir. It crosses Walnut Creek to the northeast of Los Angeles, California, approximately 16 miles northeast of Whittier, California.

Dam No. 1

Crest Elevation		983.5 ft
Freeboard		7.1 ft
Height Above Streambed		148.0 ft
Crest Length		1,085.0 ft
Crest Width		25.0 ft
Upstream Slope	(Elev. 982 to 912)	2.5 H:1V
	(Elev. 912 to 887)	3 H:1V
	(Elev. 887 & below)	3.5 H:1V
Downstream Slope	(Elev. 982 to 885)	2.5 H:1V
	(Elev. 885 & below)	3 H:1V
Material		CH-MH

Dams No. 2 & No. 3

	No. 2	No. 3
Crest Elevation	983.5 ft	983.5 ft
Height Above Streambed	49.5 ft	60.0 ft
Crest Length	785.0 ft	828.0 ft
Crest Width	25 ft	
Upstream Slope	2.5 H:1V	
Downstream Slope	2.25 H:1V	
Material	CH-MH	

Reservoir

Design Normal Water Surface Elevation	970 ft
Restricted Storage Elevation	945 ft
Design Storage	17,190 acre-feet
Design Water Surface Area	490 acres
Spillway Design Flood Pool Elevation	982 ft
Spillway Design Flood Pool Area	625 acres
Spillway Design Flood Pool Volume	23,000 acre-feet

Spillway

Type	Ungated Ogee Section
Crest Elevation	970 ft
Capacity at Spillway Design Flood Pool	30,000 cfs
Spillway Width at Crest	140 ft
Spillway Length	775 ft
Design Flood Surcharge	12 ft
Energy Dissipation	None

Outlet Works

Type	Concrete Lined Tunnel
Inlet Elevation at Trashrack sill	883.7 ft
Inlet & Outlet tunnel section	5 ft x 6 ft
Length of Inlet Tunnel	300 ft
Length of Outlet Tunnel	374 ft
Control gates, 2-slide gates each	2 ft x 5 ft
Outlet grade Elevation	882.1 ft
Energy Dissipator	15 ft long stilling basin
Maximum Discharge	1,000 cfs

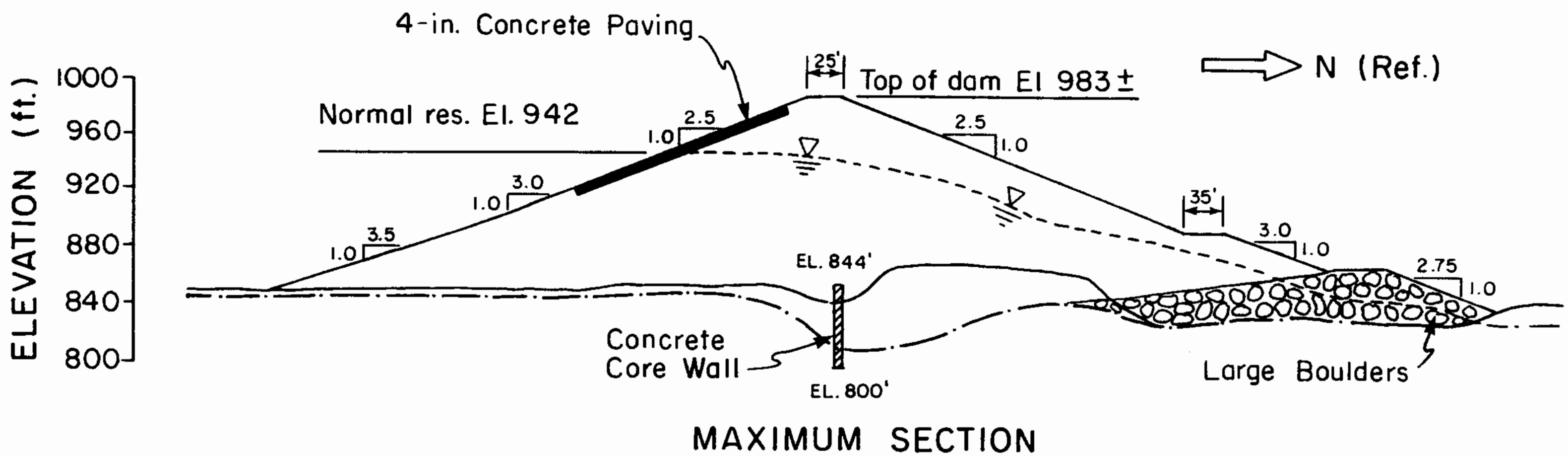


Figure 2-1: CROSS SECTION THROUGH THE MAXIMUM HEIGHT SECTION OF THE MAIN PUDDINGSTONE DAM (DAM NO. 1)

the top section from the bottom section. The upstream slopes of Dams No. 2 and 3 are 2.5 H:1V, and their downstream slopes are 2.25 H:1V.

Puddingstone Dam has been designed with a spillway and outlet works. The spillway is an ungated ogee concrete section with a crest width of 140 feet, a length of 775 feet, and a maximum design flow capacity at flood stage of 30,000 cubic feet per second. The outlet works consists of a slide-gated 5 foot x 6 foot wide, 674 foot long concrete lined tunnel with a maximum design discharge rate of 1000 cubic feet per second. Puddingstone Dam was constructed of locally available weathered shale bedrock. The resulting compacted material, which comprises the homogeneous main sections of all three earth embankment dams, is a sandy silty clay (CH-MH) with weathered shale fragments. Typically the soil is composed of 30% to 45% clay-sized particles, 30% to 45% silt-sized particles and 10% to 40% sand and gravel sized particles, though the sand and gravel sized fraction is as high as 60% in some zones. It is a brown to gray-brown soil with medium to high plasticity.

Before describing the dam materials in further detail, it is useful to briefly explore the history of Puddingstone Dam. Construction of the dam was initiated in 1926, and documentation of the dam's construction is meager. For example, not much is known about the treatment of the dam's foundation. The only documentation available is a memorandum dated July 7, 1926, where a statement is made that "Work is also in progress on excavating on the axis of the dam to bedrock, or shale, and unwatering the lower portion, after which fill on the axis will be started." It appears that the weathered bedrock material was excavated until fairly intact rock was found. This may have required on the order of 5 feet to 30 feet of excavation.

A work crew from the Los Angeles County Flood Control District initiated construction of the main Puddingstone Dam embankment in the summer of 1926 using horses with dump wagons, tractors with sheepsfoot rollers, horses with graders, and steam shovels. Later in the project (around December of 1926), trucks were substituted for the horses. This is of interest primarily because the organic content of the placed fill material decreased somewhat upon removal of the horses. Project reports noted that the construction rates varied from 2,000 to 6,000 cubic yards of compacted earth fill placed per day.

Reports and memoranda indicate that the quality of work varied throughout the course of the construction of the earth dams. A September 23, 1926 report described the fill to be excellent, being throughly rolled in thin layers. The November 16, 1926 report stated that compaction was inferior to the work previously examined. In December, 1926, a report found that the fill was being sufficiently compacted with four sheepsfoot rollers. This December 22, 1926 report also noted, however, that the line of contact with the rock and natural earth walls of the abutment was a "weakness" in the dam. It added, "To improve bond, these sections are being puddled." Memoranda dated August 22, 1927 and October 26, 1927 reported satisfactory progress with the fill material being placed and compacted in an "excellent" manner. The last field construction memorandum indicated that the contractor expected to finish the work in November 1927, if the weather permitted. In fact, construction was completed in early 1928.

A report which investigated the design of Puddingstone Dam found that the original design of the dam was "poor" and had to be modified as the job progressed. Although this poor rating is not described in detail, it appears that the design changes resulted from an inadequate knowledge of the materials in the borrow sites. Design changes are noted in the inspection reports. There are no "As Built" drawings of the final constructed design available.

An instrumentation and monitoring program was developed for Puddingstone Dam by the Los Angeles County Flood Control District, and this program is described in Table 2-2. No significant unexpected seepage, movements or displacements have been identified over the past 20 years.

Table 2-2. Instrumentation and Monitoring of Puddingstone Dam

General: The types, methods and frequencies of monitoring at Puddingstone Dam are shown below. The monitoring data is tabulated, in most cases plotted, and then analyzed by the Los Angeles County Flood Control District. Most of the information is submitted to the Division of Safety of Dams annually.

Surveillance Summary

Monitoring System	Method	Frequency
1. Movement of dam and foundation	(a) Precise survey of all local points on Dam 1	Semiannual
	(b) Precise survey of all local points of Dams 2 and 3	Annual
	(c) Expansion joints and cracks	Weekly
2. Seismic	(a) Accelerograph (left abutment)	Per occurrence
	(b) Seismoscope (2) (left abutment and downstream face of dam	Per occurrence
	(c) Slope indicator	Annual or more frequent, as needed
3. Hydrologic and Seepage	(a) Reservoir water surface	Elevation recorder (punch tape)
	(b) Leakage points (drains, springs, and leaks)	Weekly
	(c) Piezometers	Weekly (1 recorder)
4. Foundation Solution, Erosion	(a) Electroconductivity	Semiannual
	(b) Turbidimeter	Continuous (alarm)
	(c) Outflow alarm	Continuous (alarm)
5. Rainfall	Rain gauge	5-minute intervals

After the State of California imposed an operational restriction limiting the height of storage in the reservoir to Elevation 945 in August of 1966, the Los Angeles County Flood Control District initiated stability studies to evaluate the safety of dam and, if warranted, to justify eliminating the State imposed water level restriction. The District's revised study, dated January 1966, did not include seismic considerations nor did it include detailed analysis of the dam at reservoir levels above Elevation 940. Hence, the storage restriction remained, although it was modified to allow temporary flood storage above Elevation 945.

Finally, a geotechnical Investigation and stability analysis of Puddingstone Dam was completed by the International Engineering Company (IECO) in April of 1976. This study (IECO, 1970 and 1976) found that calculated safety factors for high reservoir levels (Elev. 970) and postulated strong shaking from an earthquake of magnitude 7¼ on the nearby Sierra Madre Fault, producing a maximum ground acceleration of 0.70 g at the dam site, were lower than levels considered to be "acceptable", though the dynamic stability of the dam was found to be satisfactory under these seismic loading conditions with a reduced reservoir level of Elevation 945 ft. The safety factor was defined as the ratio of the cyclic shear stress required to cause 10% strain in 10 cycles to the equivalent uniform cyclic shear stress developed in 10 cycles during an earthquake. The first 16, 35, and 60 seconds of an adjusted Taft 1952 accelerogram were used as base input motions in eight dynamic analyses. These input motions were used to model earthquakes which ranged from a Magnitude 8¼ event on the San Andreas Fault (at a distance of 20 miles) producing a maximum input bedrock acceleration of 0.42 g's to a Magnitude 7 to 7¼ event on the more local Sierra Madre Fault (at a distance of 2 miles) producing a maximum input bedrock acceleration of 0.70 g's. Various reservoir levels and phreatic surfaces were employed, and both effective stress and total (undrained) stress analyses were performed. The total stress analysis which applied the local $M \approx 7\frac{1}{4}$ earthquake to the No. 1 embankment dam when the reservoir level was at Elevation

970 was found to produce the most critical conditions, and the safety of the existing Puddingstone Dam was judged to be unacceptable at this level of storage. In June 1976, the Los Angeles County Flood Control District decided not to rehabilitate the dam and to continue operation under the current State restricted maximum normal water surface elevation of 945 ft.

The International Engineering Company's geotechnical investigation and stability studies of the Puddingstone Dam also served to produce data useful in evaluating the engineering properties of the embankment materials for the studies reported herein. In particular, two borings were performed in Dams 1 and 2 at the locations indicated in Figure 1-2. Boring 1 was a 135 foot boring through the maximum section of the main dam, and Boring 2 was a 50 foot boring through the maximum central section of Dam No. 2. These borings, along with a complimentary laboratory testing program, produced these findings (IECO, 1970 and 1976):

1. "The majority of the tests indicated low in-place densities, with the moisture content in some cases very close to saturation. Dry density values of the silty-sandy clays in the dam average 89 pounds per cubic-foot and the moisture content averages 35 percent."
2. The soil's specific gravity was found to be about 2.64.
3. The material in the dam varies in gradation from 60% sand and gravel sized particles and 40% silt and clay size particles, to 20% sand and gravel sized particles and 80% silt and clay size particles.
4. The average coefficient of permeability was found to be less than 1×10^{-7} centimeters per second.
5. Plasticity Index and Liquid Limit data for the minus No. 40 sieve fraction of a number of samples extracted from the two embankments explored are presented on the Plasticity Chart shown in Figure 2-2. The material is

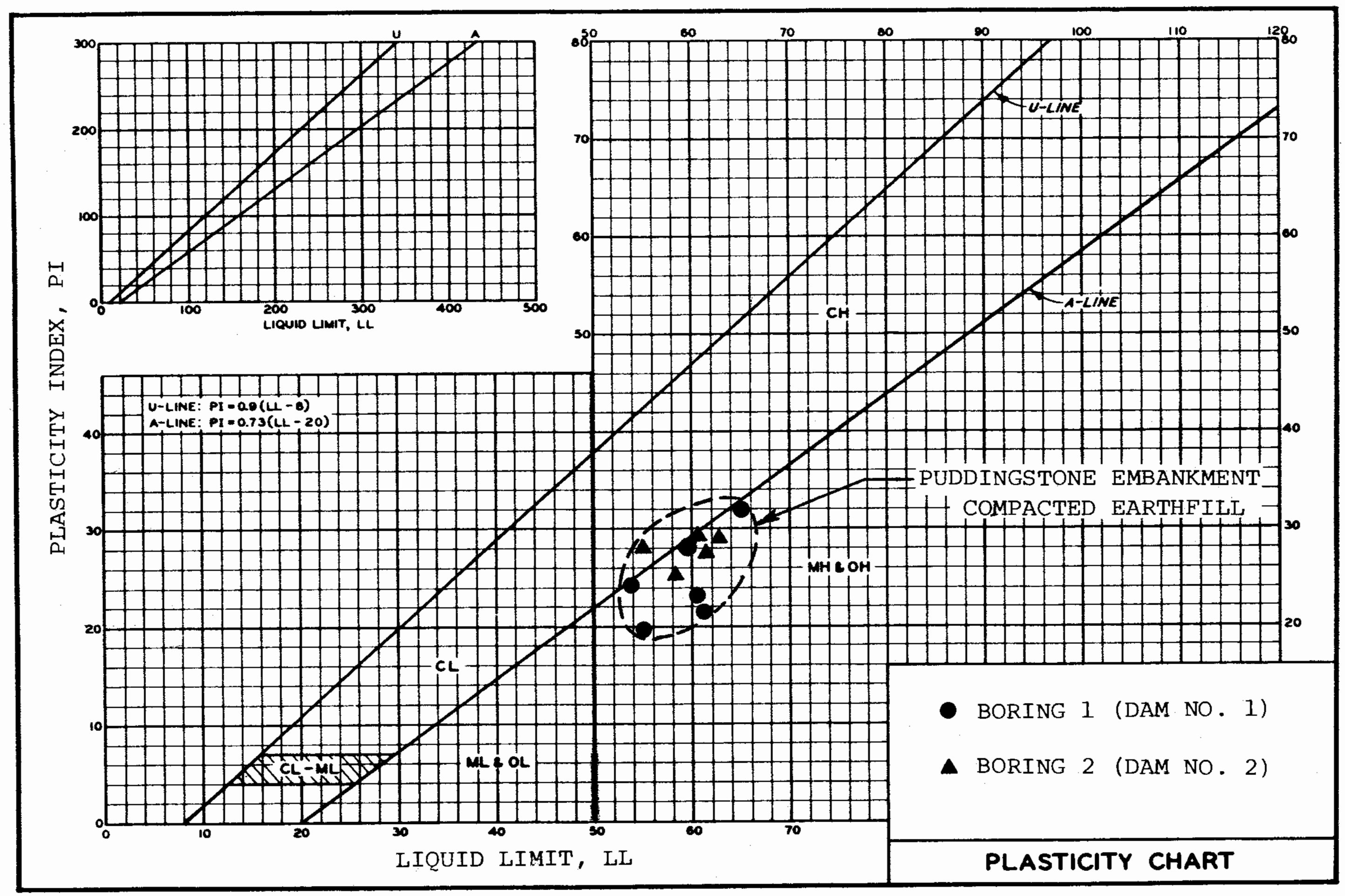


Figure 2-2: ATTERBERG LIMITS TEST RESULTS FOR SAMPLES OBTAINED BY IECO (1973,1976) FROM PUDDINGSTONE DAMS NO. 1 AND 2

classified as CH-MH and possesses medium to high plasticity. Its Plasticity Index ranges from 20 to 32 and its Liquid Limit ranges from 53 to 65.

6. Soil strength parameters were developed as:

   a. $S = 1000 + T_n \cdot \tan(17^\circ)$ [PSF] "Total Stress"

   b. $S' = 900 + T_n \cdot \tan(25^\circ)$ [PSF] "Effective Stress"

   where S denotes soil shear strength and T_n denotes the normal stress on the plane in which S acts.

Finally, laboratory cyclic triaxial tests on "undisturbed" samples of the compacted embankment fill produced data regarding dynamic shear modulus values and damping ratio values at various levels of cyclic shear strain. These results are reproduced in Figures 2-3 and 2-4. At the time of these investigations, the resulting data was judged to be fairly consistent with "typical" curves for "clayey soils" presented by Seed and Idriss (1970), as indicated in Figures 2-3 and 2-4. Alternate modelling hypotheses will, however, be employed in modelling the shear-strain-dependent dynamic shear modulus and damping behavior of the embankment soils in the response analysis studies described later in Chapter 4.

In summary, although the construction of the sixty-three year old Puddingstone Dam is not well documented, sufficient records are available to provide a feel for the general conditions of the dam and the materials which comprise it. Additionally, subsequent studies of Puddingstone Dam (IECO, 1970 and 1976) provide more detailed information regarding the "as-built" composition of the embankments and the engineering properties of the predominant embankment material, the locally available sandy silty clay (CH-MH). Moreover, the dam's geometry and the local topography are reasonably well defined. The field soil borings indicate some variability in the soil of the main dam, which agrees well with reports of inconsistent quality control of the earth placement process during the dam's construction. Yet, relative to the highly variable

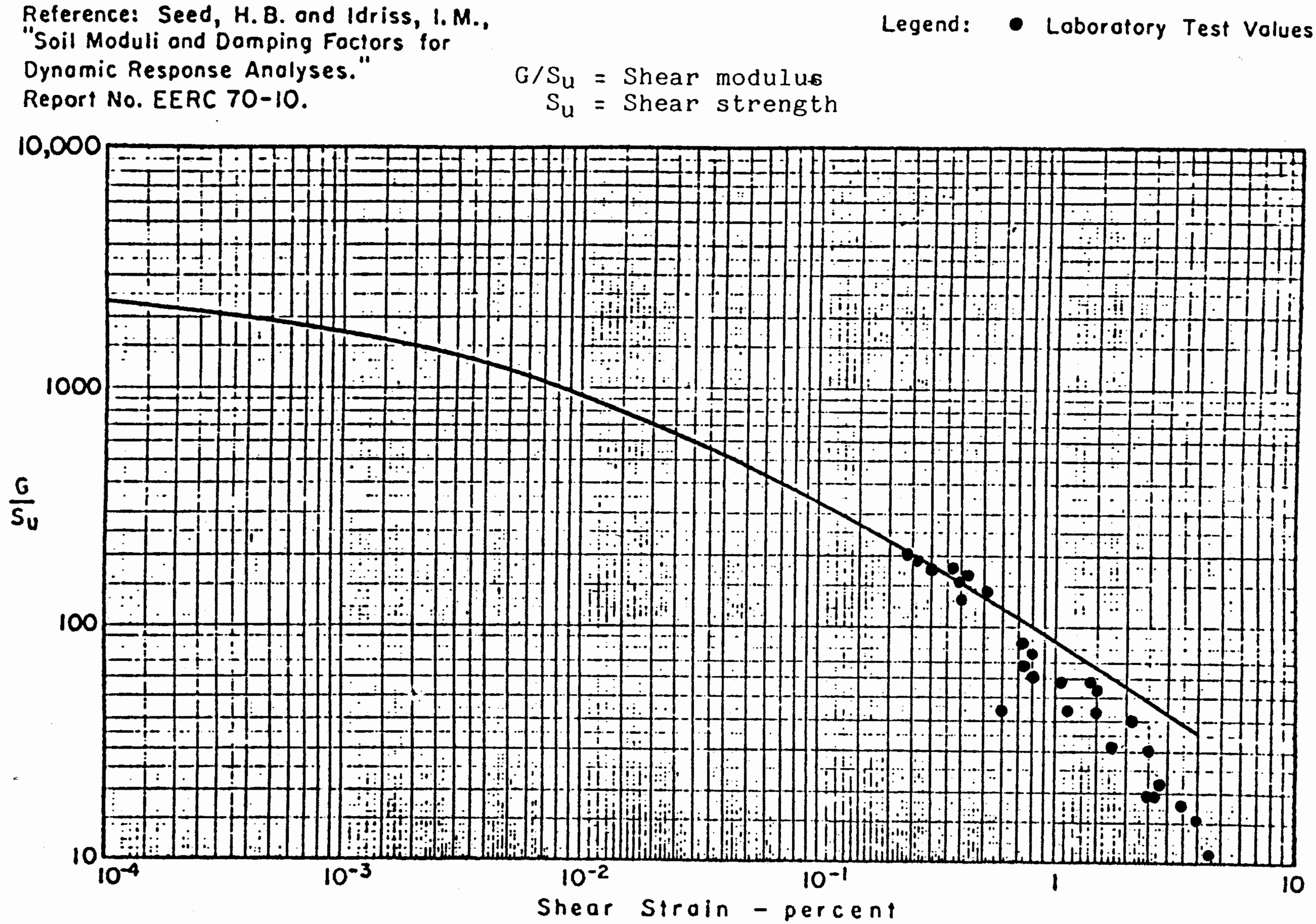
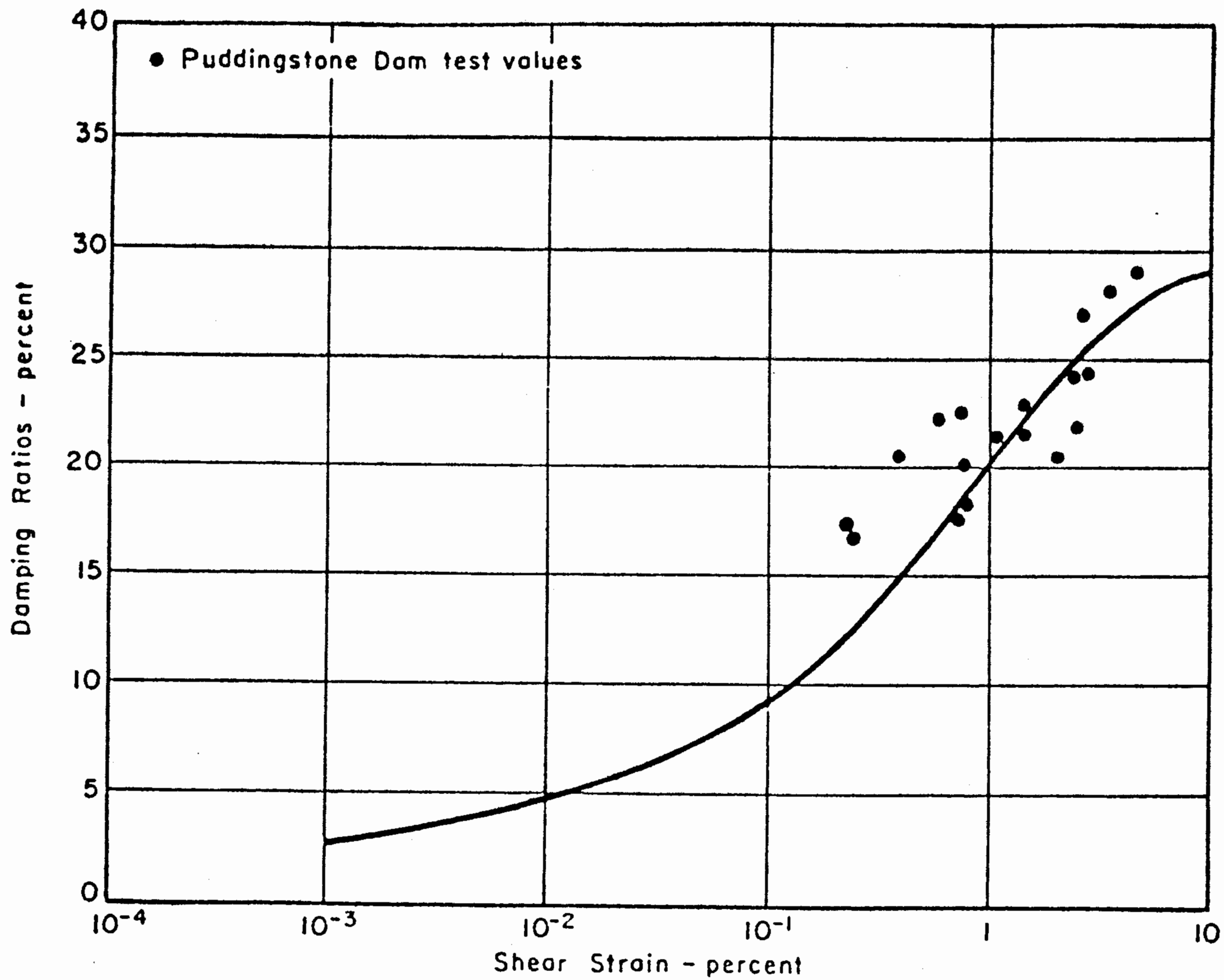


Figure 2-3: DYNAMIC SHEAR MODULI FROM 1973 LABORATORY TESTS OF PUDDINGSTONE DAM SILTY SANDY CLAY (After International Engineering Co., 1976)



DAMPING RATIOS FOR SATURATED CLAYS

Figure 2-4: DAMPING RATIOS FROM 1973 LABORATORY TESTS (After International Engineering Co., 1976) soil conditions which often exist within natural deposits, this "controlled" earth filled structure may be regarded as relatively homogeneous. In summary, sufficient information and data is available to provide a reasonable engineering basis for performing response analyses of the Puddingstone Dam.

Chapter 3

SEISMIC INSTRUMENTATION AND THE RECORDED GROUND MOTIONS

Puddingstone Dam is situated in a seismically active area, as shown in Figure 1-3. The San Andreas Fault Zone, located about 20 miles northeast of Puddingstone Dam, is capable of generating a maximum credible earthquake of magnitude $M_L \approx 8$ to $8\frac{1}{2}$. Closer to the dam, at a distance of only 2 miles, the Sierra Madre Fault is capable of generating a maximum earthquake of magnitude $M \approx 7$ to $7\frac{1}{2}$. Typical ground acceleration attenuation relationships (e.g. Campbell, 1981; Joyner and Boore, 1981; Bolt and Abrahamson, 1982) suggest that the San Andreas Fault and the Sierra Madre Fault could produce peak horizontal ground accelerations of approximately 0.35 g and 0.65 g, respectively, at the dam site.

Because of the relatively high seismicity of the area, Puddingstone Dam was selected by the State of California Strong Motion Instrumentation Program (CSMIP) for comprehensive instrumentation to investigate the dynamic response of dams to strong earthquake shaking. Accordingly, a total of 18 strong motion accelerographs were installed at six locations on and near the main dam. The locations of these accelerographs are shown in plan view in Figure 3-1. These locations were chosen to investigate the spatial variations of motions across the valley as well as from bedrock to the crest of the embankment, including the possible effects of topographic irregularities and the different stiffness characteristics of the materials comprising the embankment and the walls of the valley. At most locations, motions were recorded in three orthogonal directions: vertically, longitudinally (parallel to the main dam axis) and transverse to the main dam axis. This paper will concentrate on the transverse motions, as these are the motions of primary engineering interest. Acceleration sensors 4-12 were installed in ground vaults, and sensors 1-3 and 13-18 were placed in housings already constructed.

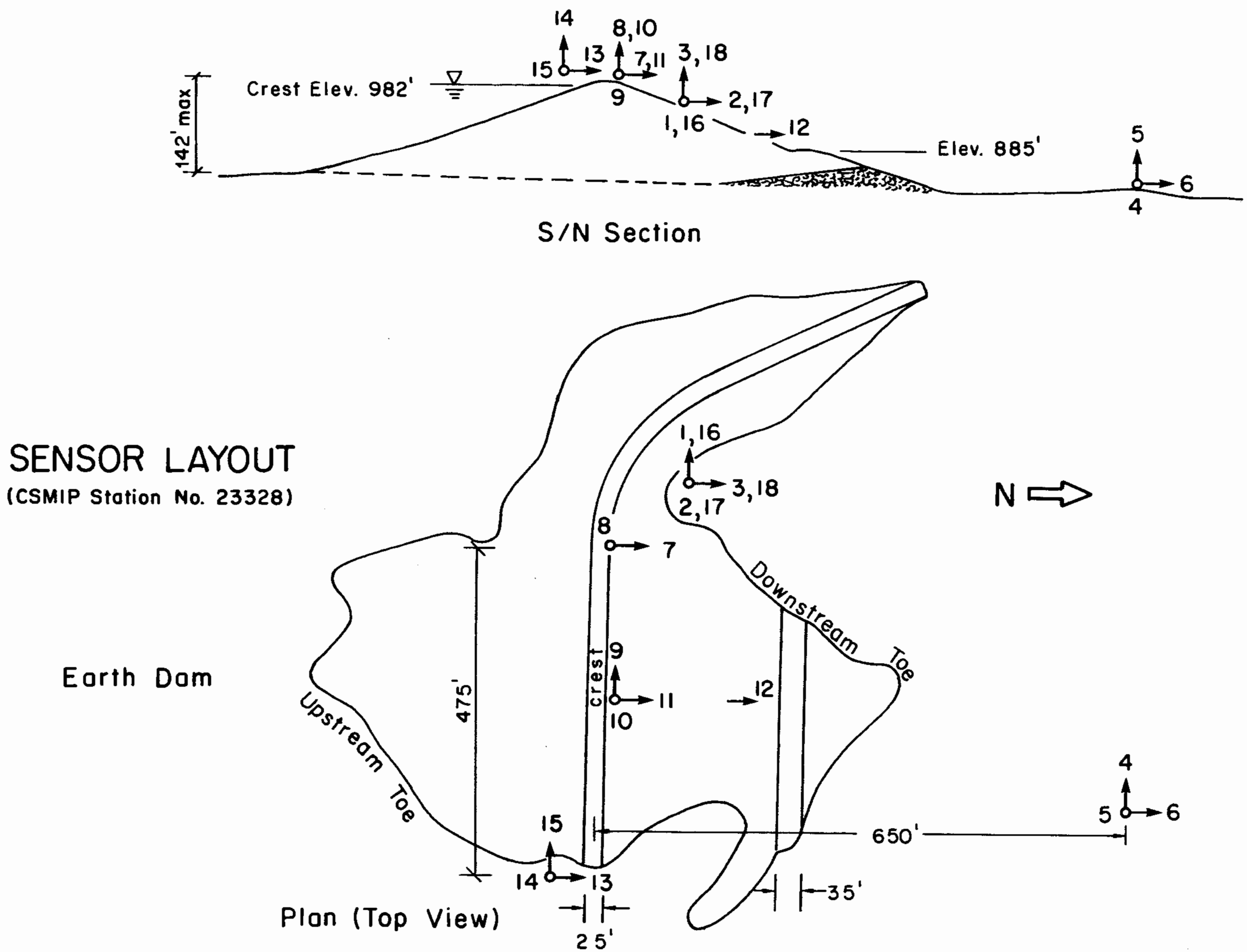


Figure 3-1: PLAN VIEW OF PUDDINGSTONE MAIN DAM SHOWING SENSOR LOCATIONS

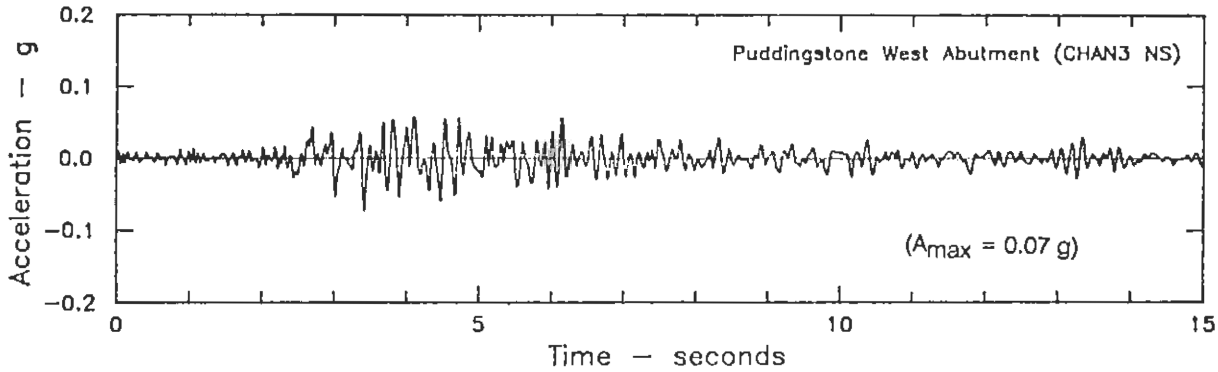
Sensors 1-6 and 13-18 were sited to record bedrock motions. However, these sensors were actually installed on shallow, stiff soil deposits (Sensors 4-6) or on low-grade weathered rock (Sensors 1-3 and 13-18) so they do not quite represent recordings of "true rock" motions. They will be referred to hereafter as "near rock" sites. Sensors 16-18 were co-located with sensors 1-3, and responded very similarly. Sensors 7-11 were sited near the main dam's crest. Of these sensors, Sensors 9-11 were located near the mid-point of the highest section of the dam, and sensors 7 and 8 were located near a rocky saddle separating the main dam section from a smaller, western extension of the embankment fill. Sensor 12 was located near the center of the downstream face of the main embankment section.

The Whittier Narrows Earthquake of October 1, 1987 provided an excellent set of records of the seismic response and performance of Puddingstone Dam. This event, with magnitude M_L = 5.9, occurred on a newly discovered segment of the Whittier Fault, with an epicenter located approximately 18 miles from the dam site as shown in Figure 1-3. This event triggered the accelerographs, producing strong motion recordings. Recorded peak ground accelerations of the near rock sites (Sensors 1-6 and 13-15) ranged from 0.04 g to 0.08 g. Unfortunately, one of the most strategically placed sensors (Station 7) did not operate, and as a result the variation of strong motions in the transverse direction along the crest of the dam cannot be studied. On the other hand, the recordings obtained for Sensors 11 and 12, in conjunction with the recorded "near rock" motions, provide an excellent opportunity to study the variation of strong motions transverse to the dam at the crest and at the mid-height downstream slope of the dam at its maximum cross section, and thus to study the dam's response characteristics of principal engineering interest, as well as the ability of various dynamic analysis techniques to correctly model and predict these motions. Finally, it should be noted that Puddingstone Dam suffered no significant damage as a result of the earthquake shaking.

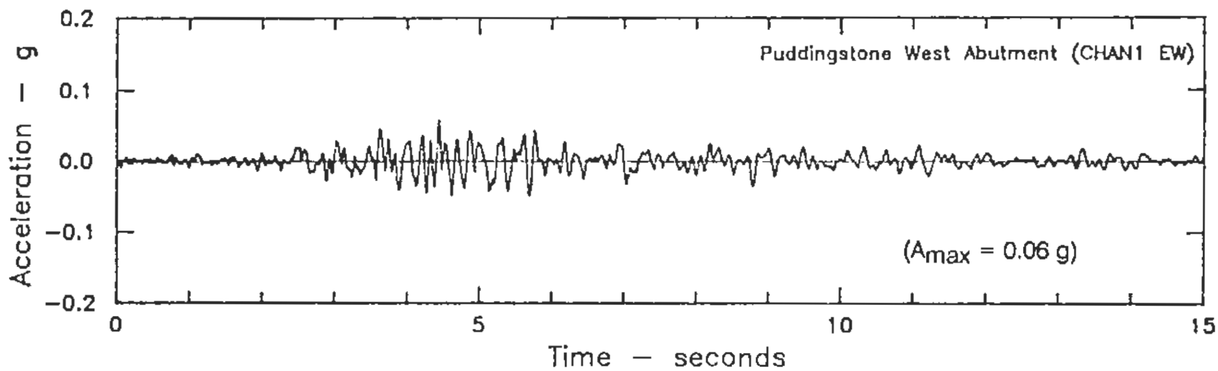
The recorded horizontal ground motions have been processed by CSMIP to produce instrument and baseline corrected acceleration time histories, and these have been re-plotted and are presented in Figures 3-2 through 3-7. Figures 3-8 through 3-11 present the corresponding computed response spectra, with 5% material damping, for the transverse horizontal motions at selected stations.

The observed duration of strong shaking was typically on the order of 7 or 8 seconds at most "near rock" sensor locations, and somewhat longer at sensor locations on the earthfill embankment. Examining Figures 3-2 through 3-4, which illustrate the horizontal accelerations recorded at "near rock" sites, one can see that these motions are fairly consistent with each other. In particular, the transverse ground motions (oriented in the NS direction) are similar in terms of peak ground accelerations and frequency content. The maximum "near rock" accelerations recorded at Stations 1-6 and 13-15 were on the order of 0.04 g to 0.08 g. The transverse components of the recorded earthquake motions are the motions of primary engineering interest in analyzing the response of an earth dam. As shown in Figures 3-2 through 3-4, the maximum transverse "near rock" accelerations recorded were on the order of 0.06 g to 0.08 g. It is interesting to compare these with the maximum transverse accelerations recorded at the crest and at the center of the downstream face. As shown in Figures 3-5(a) and 3-6, these maximum accelerations were both approximately 0.19 g, representing peak response amplification by a factor of approximately two to three from the recorded "near rock" maximum transverse accelerations.

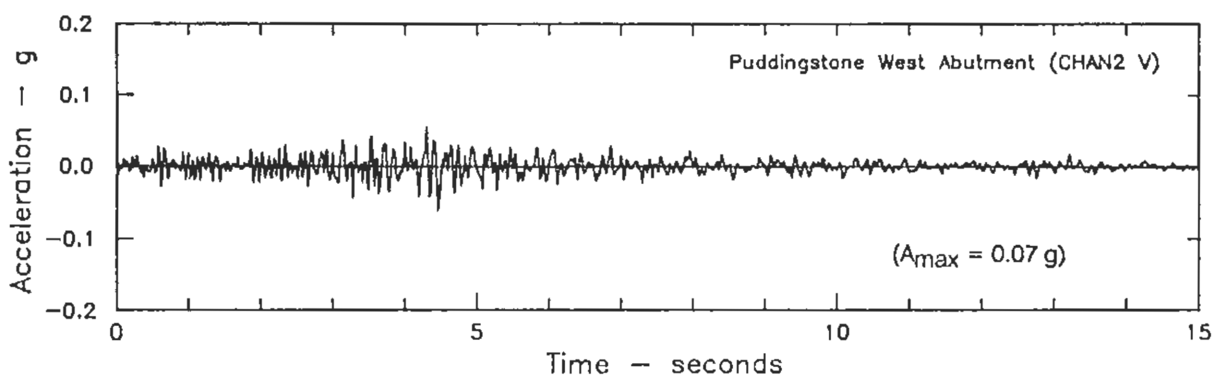
In order to throw more light on the characteristics of the horizontal earthquake motions recorded at Puddingstone Dam during the Whittier Narrows earthquake, acceleration response spectra (with 5% material damping) have been developed and are shown in Figs. 3-8 through 3-11. The first of these figures depict acceleration response spectra of each of the horizontal acceleration sensors on the "near rock" sites. The transverse acceleration response spectra indicate peak ground accelerations of



(a) Horizontal Acceleration: Transverse to the Crest Axis

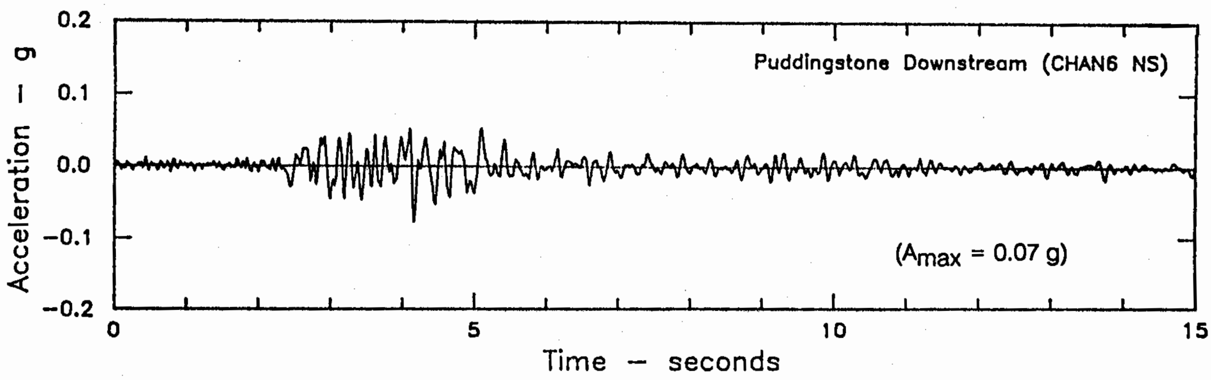


(b) Horizontal Acceleration: Parallel to the Crest Axis

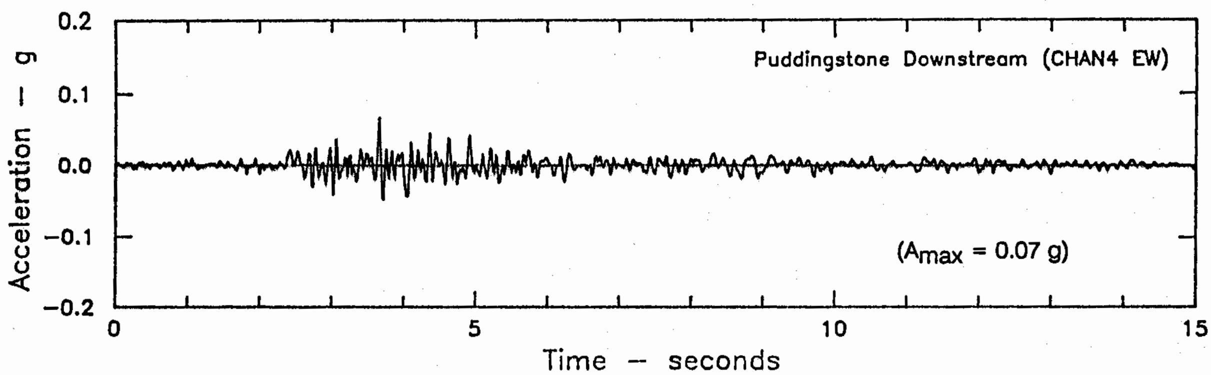


(c) Vertical Acceleration

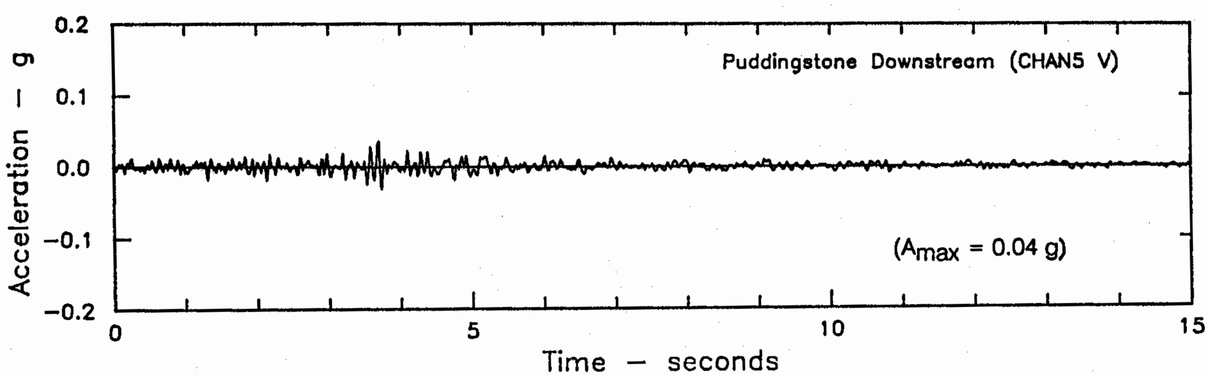
Figure 3-2: RECORDED ACCELERATION TIME HISTORIES AT STATIONS 1, 2 AND 3 (LEFT ABUTMENT, ON WEATHERED ROCK)



(a) Horizontal Acceleration: Transverse to the Crest Axis

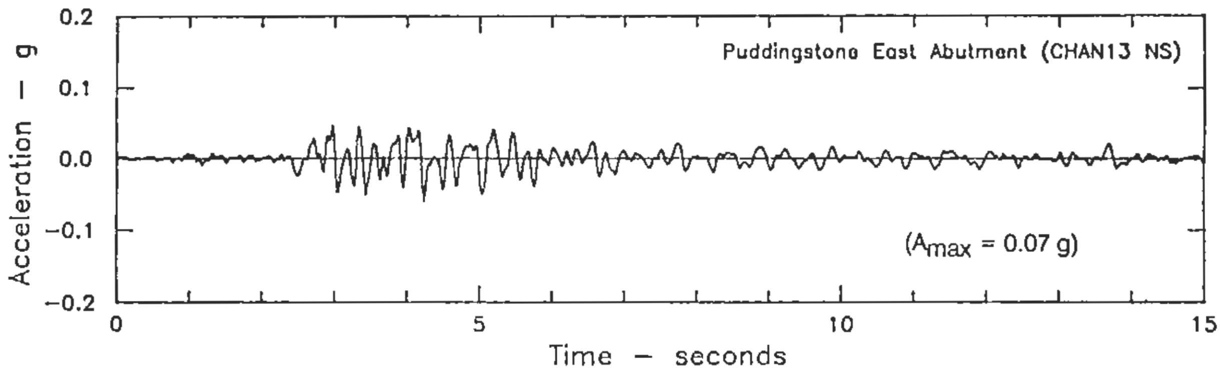


(b) Horizontal Acceleration: Parallel to the Crest Axis

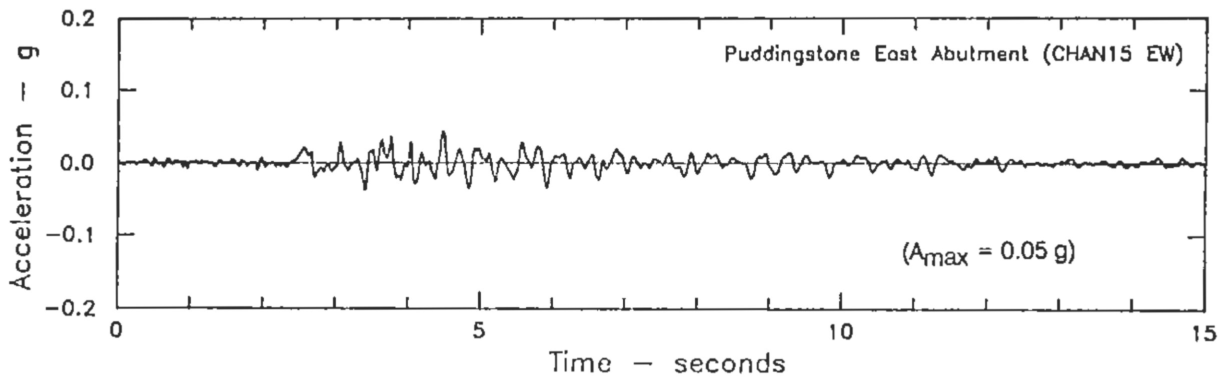


(c) Vertical Acceleration

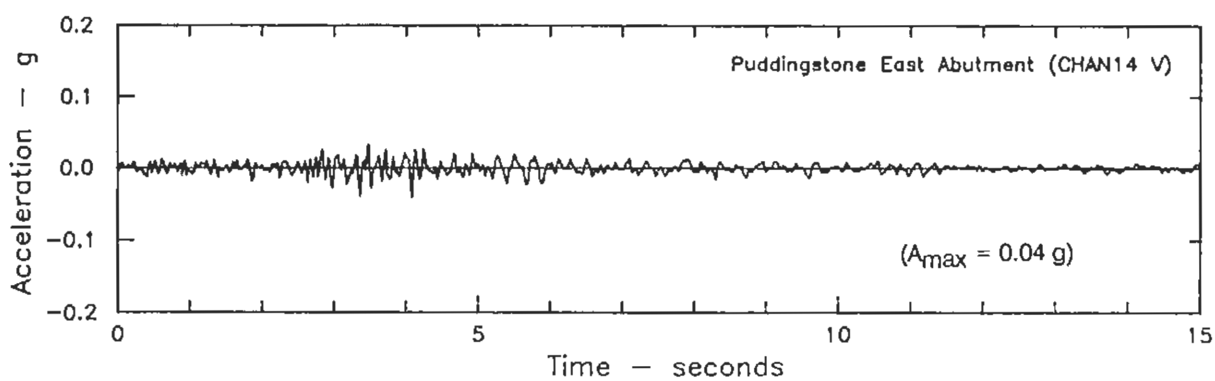
Figure 3-3: RECORDED ACCELERATION TIME HISTORIES AT STATIONS 4, 5 AND 6 (DOWNSTREAM OF THE EMBANKMENT, ON SHALLOW ALLUVIUM)



(a) Horizontal Acceleration: Transverse to the Crest Axis

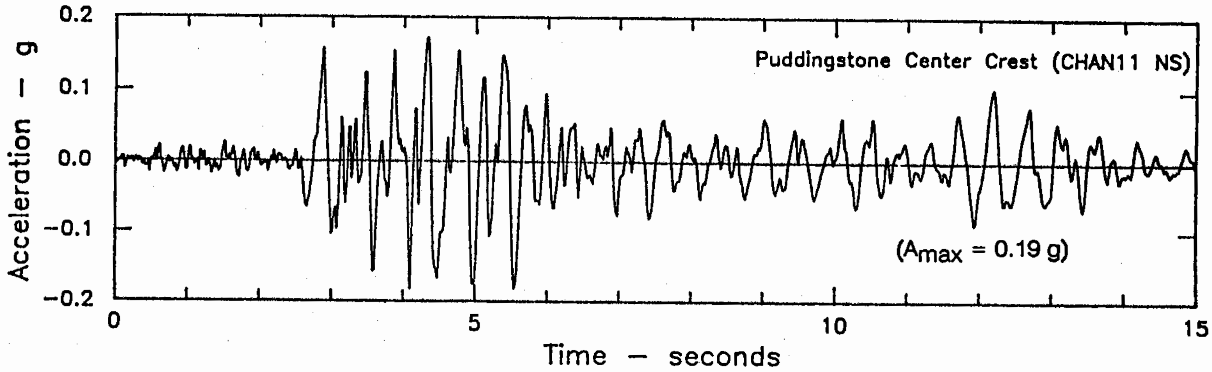


(b) Horizontal Acceleration: Parallel to the Crest Axis

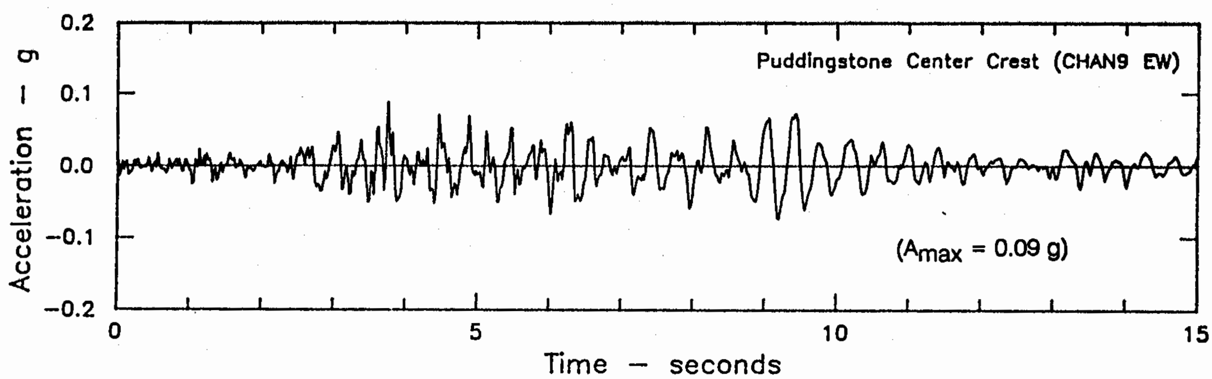


(c) Vertical Acceleration

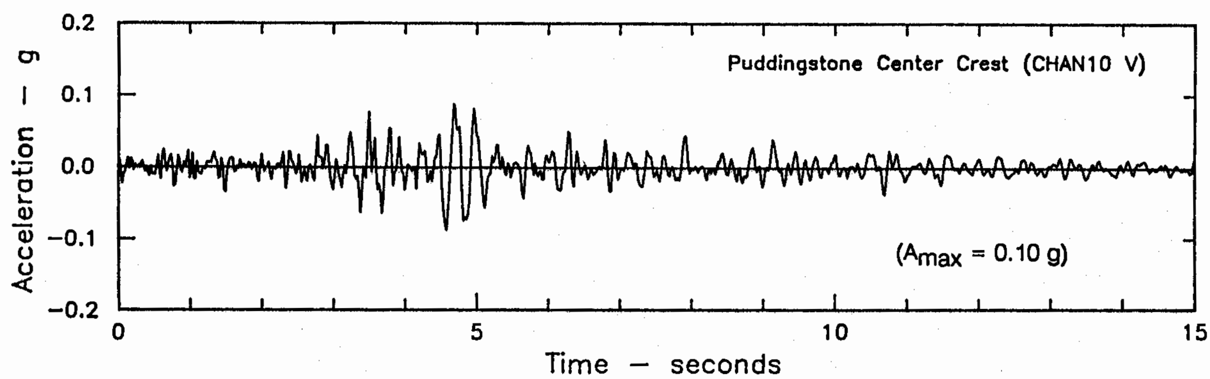
Figure 3-4: RECORDED ACCELERATION TIME HISTORIES AT STATIONS 13, 14 AND 15 (RIGHT ABUTMENT, ON WEAK, WEATHERED ROCK)



(a) Horizontal Acceleration: Transverse to the Crest Axis

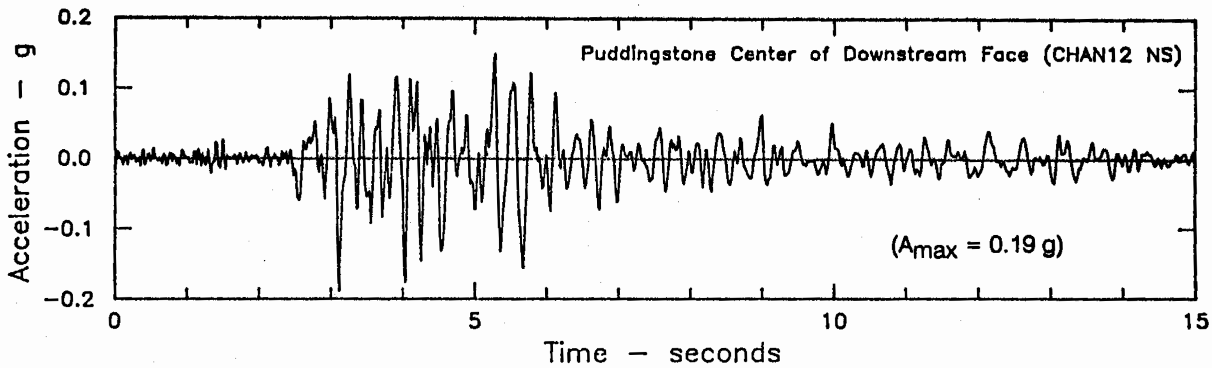


(b) Horizontal Acceleration: Parallel to the Crest Axis



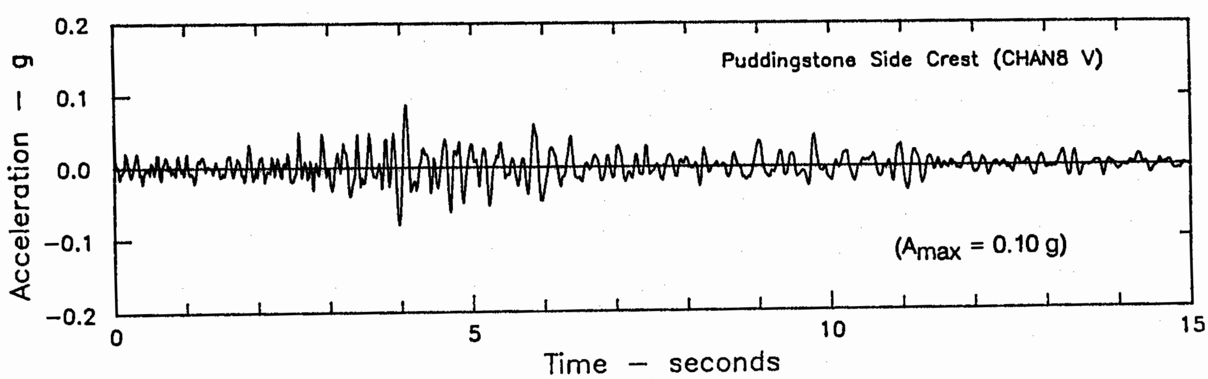
(c) Vertical Acceleration

Figure 3-5: RECORDED ACCELERATION TIME HISTORIES AT THE CENTRAL CREST OF THE MAIN DAM (STATIONS 9, 10 AND 11)



(Horizontal Acceleration: Transverse to the Crest Axis)

Figure 3-6: RECORDED ACCELERATION TIME HISTORY AT CENTER OF THE DOWNSTREAM FACE OF THE MAIN DAM (STATION 12)



(Vertical Acceleration)

Figure 3-7: RECORDED ACCELERATION TIME HISTORY AT THE CREST OF THE MAIN DAM (STATION 8), AT THE TRANSITION FROM THE MAIN EMBANKMENT SECTION TO THE WEST WING SECTION

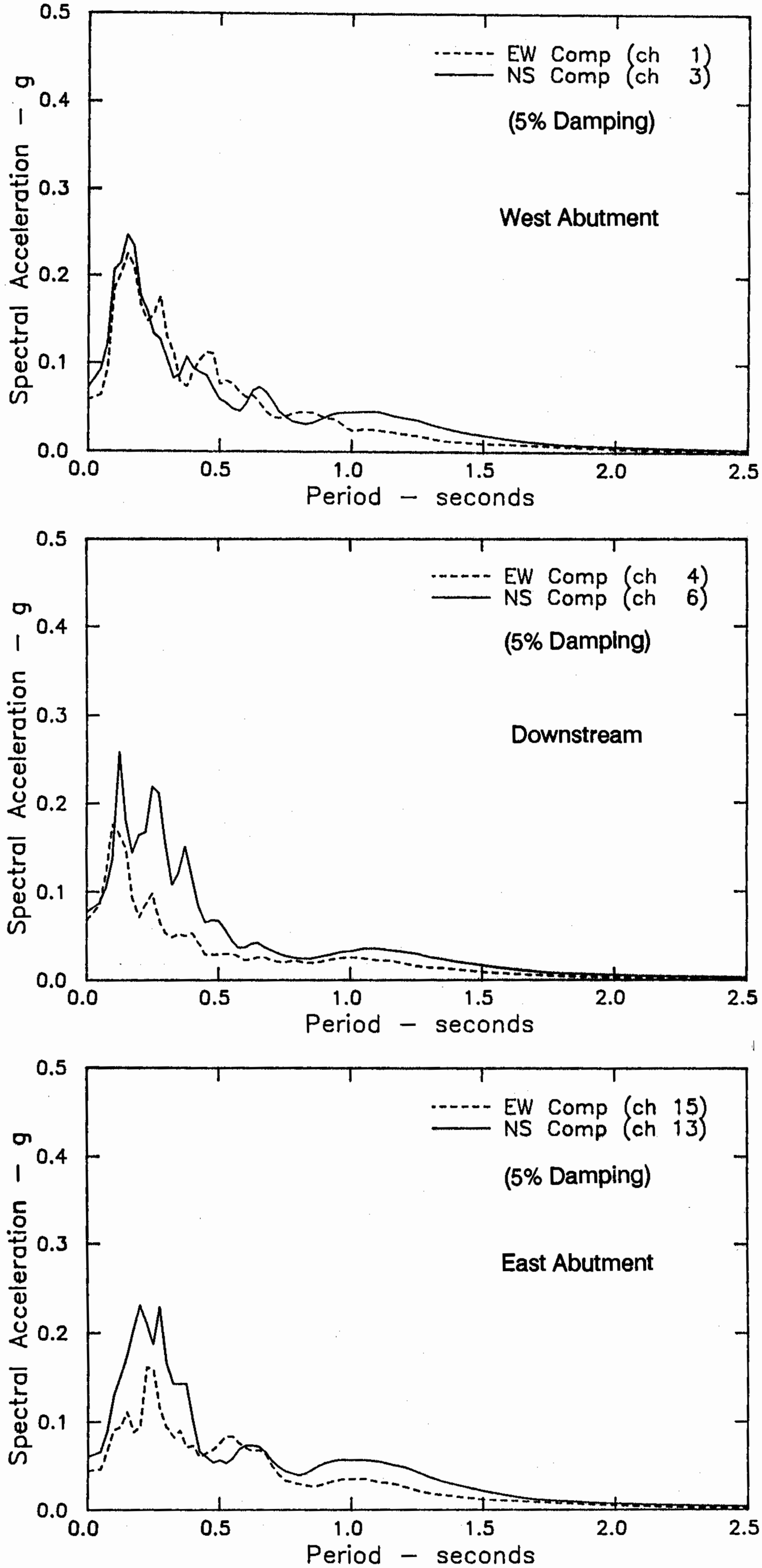


Figure 3-8: RESPONSE SPECTRA FOR THE RECORDED "NEAR" ROCK MOTIONS AT THE EAST AND WEST ABUTMENT, AND DOWNSTREAM OF PUDDINGSTONE DAM

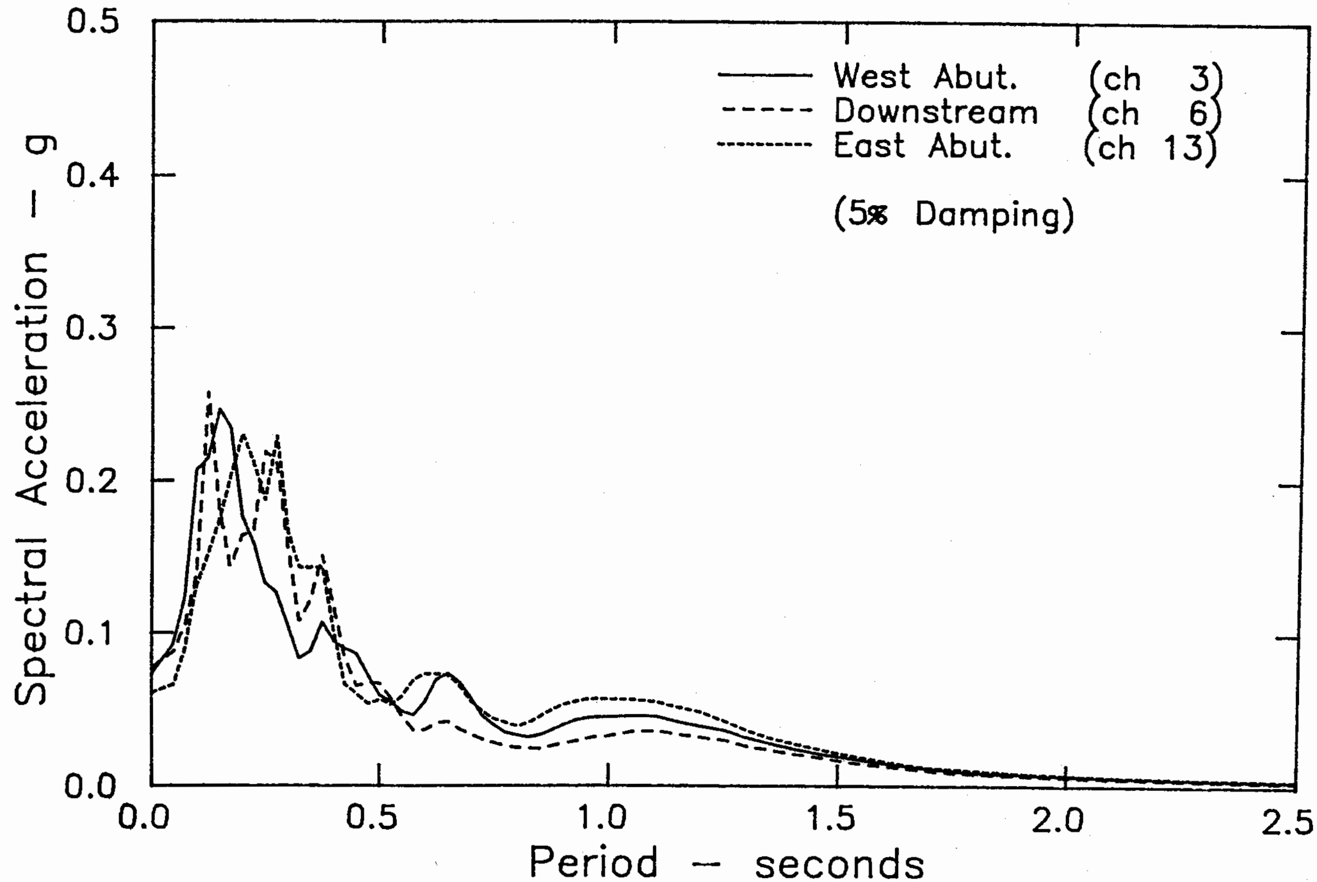
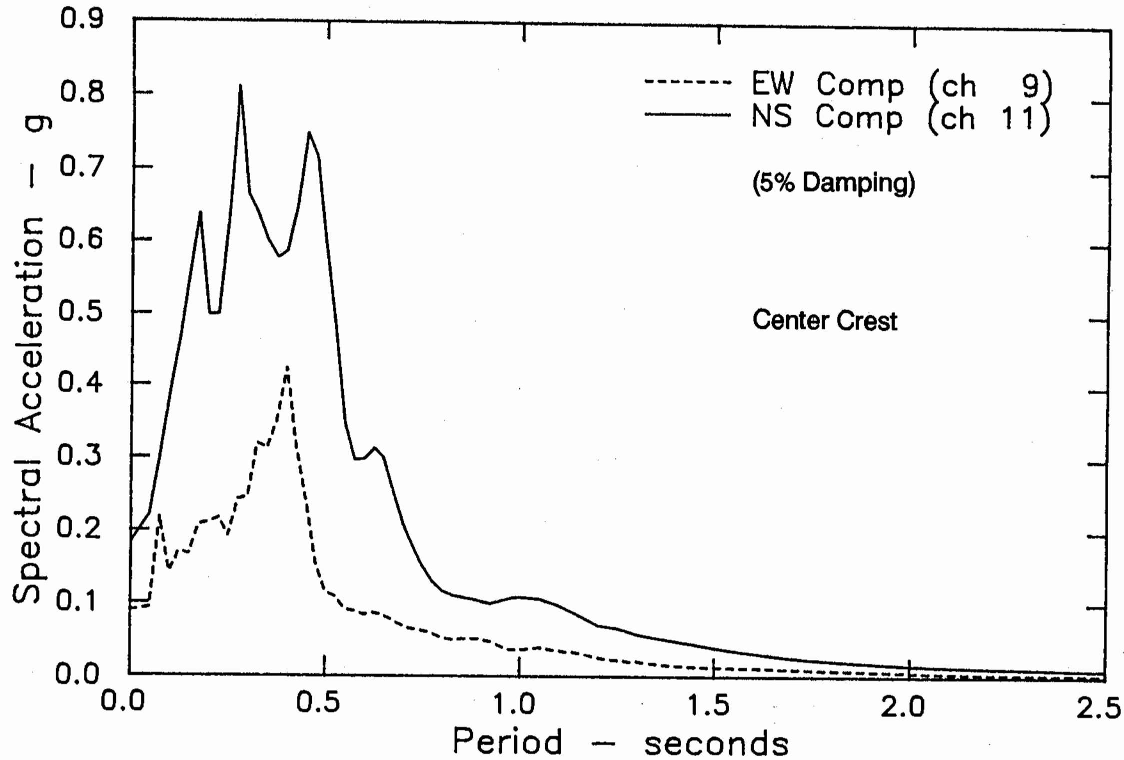


Figure 3-9: COMPARISON OF RESPONSE SPECTRA FOR RECORDED TRANSVERSE MOTIONS AT "NEAR" ROCK SITES



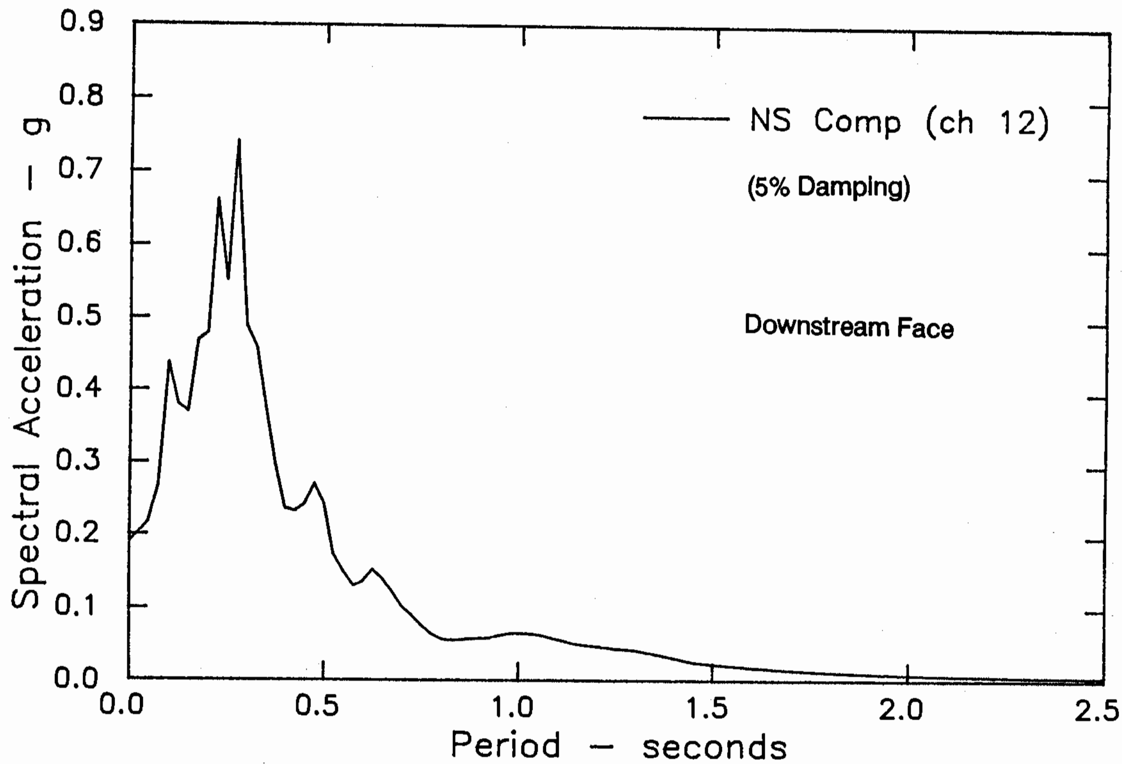


Figure 3-10: RESPONSE SPECTRA FOR THE RECORDED MOTIONS AT THE CREST AND DOWNSTREAM FACE OF PUDDINGSTONE DAM

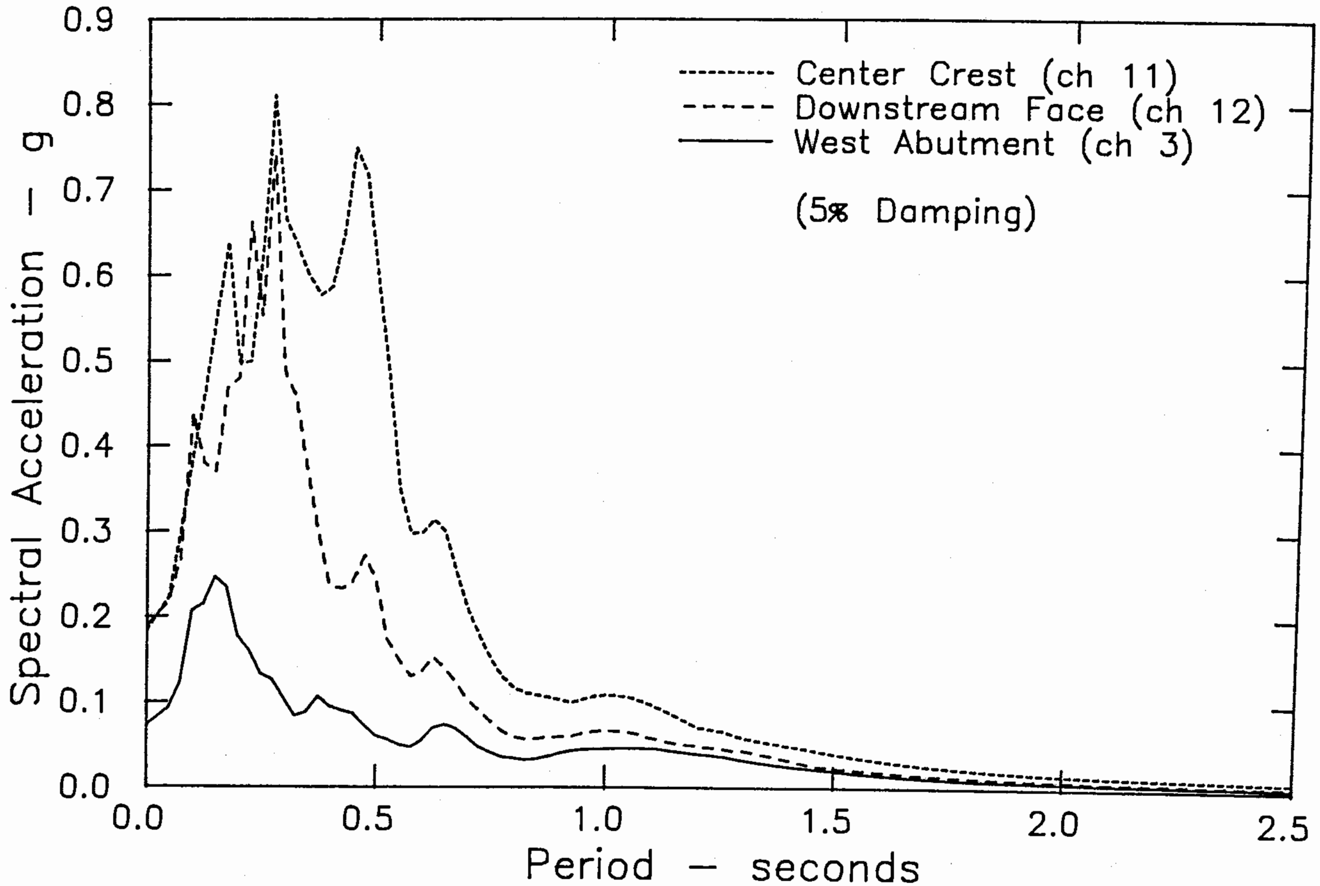


Figure 3-11: COMPARISON OF RESPONSE SPECTRA FOR RECORDED TRANSVERSE MOTIONS AT THE CREST, DOWNSTREAM FACE, AND WEST ABUTMENT OF PUDDINGSTONE DAM

0.07 g, maximum spectral accelerations which range from 0.23 g to 0.26 g, and predominant periods which range from 0.2 seconds to 0.25 seconds. The transverse components of each of these horizontal "near" rock response spectra are all shown together in Fig. 3-9. As suggested earlier when examining their acceleration time histories, it seems reasonable to conclude that the "near" rock motions are similar in terms of the engineering accuracy obtainable in seismic site analysis. These response spectra thus provide support for the important assumption that the dam is constructed on a rigid base, and that all points on the rigid boundary have largely the same motions. Most widely used analytical procedures rely upon this assumption when computing the seismic response of embankment dams.

A closer inspection of these three "near" rock response spectra, however, suggests that Channel 3 with its higher concentration of energy at higher frequencies corresponds to the most "rock-like" recording among the three. On the other hand, Channel 6 which was sited on shallow, stiff soil does display some deviation from the classic rock recording with its amplification of accelerations at lower frequencies. Channel 13's response spectra is intermediate, although quite similar to that of Channel 3. It thus appears appropriate to use Channel 3 as the standard input rock motion in subsequent analyses.

The acceleration response spectra for the transverse motions recorded at Sensors 11 and 12 are shown in Figure 3-10. The character of the response spectra for the embankment dam crest and slope motions is greatly different from that recorded at the "near rock" sites. The transverse embankment crest and slope acceleration response spectra indicate peak ground accelerations of 0.19 g and 0.18 g, maximum spectral accelerations which range from 0.74 g to 0.81 g, and predominant periods which range from 0.26 seconds to 0.45 seconds. Maximum spectral accelerations at the top of the embankment surface have been amplified by a factor of 4 and peak ground accelerations have been tripled. The predominant period of the earth structure appears to be in the range of 0.25 to 0.45 seconds. The "near rock" and embankment response spectras can be readily compared by studying Fig. 3-11. The Whittier Narrow earthquake has provided us with an exceptional opportunity to apply our analytical tools to a well-documented case study to ascertain their validity and to improve our understanding of this phenomenon.

In summary, the State of California Strong Motion Instrumentation Program instrumented Puddingstone Dam with a well organized array of 18 strong motion accelerograms. These devices were triggered by the 1987 Whittier Narrows earthquake and ground motions were recorded. These motions have been processed to provide acceleration time histories and acceleration response spectra for the horizontal motions at various locations at the dam site. These sites can be categorized as "near rock" abutment sites, the embankment crest site, and the embankment mid-downstream face site. Studying the variation of the transverse motions at these sites indicates that accelerations within the rock around the dam were fairly uniform and that accelerations were strongly amplified within the earth embankment. These recorded motions provide a rare opportunity to validate our seismic design and response analysis procedures for embankment dams and to enhance our understanding of the dynamic response of dams to strong motion shaking.

Chapter 4

SEISMIC RESPONSE ANALYSES OF PUDDINGSTONE DAM

Introduction:

The Whittier Narrows earthquake provides an opportunity to apply analytical models and techniques to a well-documented case study to ascertain their suitability for this class of problem. The dynamic response of Puddingstone Dam may be studied through the use of empirical correlations, one-dimensional (1-D) analyses, two-dimensional (2-D) analyses, and three-dimensional (3-D) analyses. These analytical procedures may incorporate nonlinear dynamic soil properties or equivalent linear dynamic soil properties or even linear elastic dynamic soil properties to model strain dependent moduli and damping characteristics. Additionally, the analyses may or may not incorporate such complexities as pore pressure generation, strain softening, geometric damping, or travelling waves at the boundary of the modelled region.

In this study, empirical techniques, as well as one-dimensional and two-dimensional analyses will be employed. Since the dam possesses a crest length to height ratio of approximately 4.5 to 1 ($L/H \approx 4.5$), it is probable that three-dimensional effects will be fairly minor. In fact, previous studies by Mejia and Seed (1980) and Makdisi, et al. (1982) suggest that two-dimensional analytical procedures provide acceptable accuracy for earth dams with crest length to height ratios greater than 3 or 4 to 1. Furthermore, the inaccuracies created when analyzing a three-dimensional problem by two-dimensional analysis can be compensated for by slightly modifying the dynamic soil properties used to model the materials comprising the earth dam. For example, the 3-D geometry provides greater confinement for the soil within the embankment and provides side shear resistance, and thus increases the overall stiffness of the dam. A 2-D analytical procedure which utilizes slightly higher dynamic moduli might therefore improve its accuracy. This will be explored later in this report.

Numerous studies have validated the suitability of the equivalent linear method for modelling nonlinear (shear dependent) dynamic soil properties in response analyses. This involves the use of a shear strain-dependent linear elastic shear modulus and a shear strain-dependent damping ratio in each individual layer or element of the modelled soil region. These moduli and damping ratios are selected as those representative of a shear strain level equal to 65% of the maximum shear strain occurring in the given soil element or layer. As the shear strains calculated are, in turn, affected by the moduli and damping levels employed, analyses are performed iteratively until convergence is achieved and the moduli and damping in all elements are compatible with the calculated strains within the elements. All analyses of dynamic response performed as part of these studies utilized the equivalent linear method to model strain dependent moduli and damping characteristics. One-dimensional (columnar) response analyses were performed using the program SHAKE (Schnabel et al., 1972), and two-dimensional (plane strain) finite element analyses were performed using the program FLUSH (Lysmer et al., 1975). As transverse motions are the motions of principal engineering interest, and because the coupling effects between the various components of the recorded motions are not likely to be significant, only the transverse components of the recorded accelerations were considered in detail in this study.

Dynamic Soil Properties

Before any analyses can be performed, the dynamic properties of the soil must be evaluated and modelled. As discussed in Chapter 2 of this report, International Engineering Company's 1973 and 1976 geotechnical investigations of the Puddingstone Dam produced data useful in determining the engineering properties of the embankment materials. These data, along with construction reports describing materials and placement procedures, were used as the basis for evaluation of the dam's dynamic soil properties.

The principal material of interest is the compacted sandy silty clay (crushed weathered shale) which comprises almost the entire section of the main Puddingstone Dam (Dam No. 1). The soil's key dynamic properties are its strain dependent shear moduli (G) and damping ratio. Characterization of the soil's shear modulus values requires development of appropriate estimates of maximum shear moduli or G_{max} (the value of G at "small" shear strains of on the order of 1×10^{-4} %), and the choice of a suitable form of the modulus reduction curve (the relationship between G/G_{max} vs shear strain) for this material.

The 1973 geotechnical study of Puddingstone Dam found that the embankment material was fairly uniform in terms of density, water content, and strength with depth. For example, SPT values ranged from 21 to 41 blows/ft with most values near the average value of 33 blows/ft. No consistent pattern of increasing SPT values with depth emerged. Improved compaction procedures and desiccation of the clayey soil near the top of the embankment may partially explain this phenomenon. It does seem reasonable, however, that stiffness should at least slightly increase with increased overburden and confinement, so a slight increase in modulus with increased overburden was assumed to occur. The analyses performed were not sensitive to this assumption of a slight increase in the stiffness of the soil with overburden depth. Thus, for the balance of this study, an overall average maximum ("small" strain) modulus value ($(G_{max})_{avg}$) was used to characterize the majority of the embankment soil mass. Moduli actually used at any location varied slightly with confinement, with a G_{max} value approximately 10 - 15% less than $(G_{max})_{avg}$ being used at the crest and faces of the dam, and a G_{max} value about 10 - 15% greater than $(G_{max})_{avg}$ being used deep within the interior of the embankment.

The embankment fill's average G_{max} value was evaluated using a variety of techniques to develop a feel for the range of the possible values of the dynamic shear moduli of the embankment material. Utilizing the SPT data ($N_{avg} \approx 33$) and the empirical relationship $G_{max} \approx 130$ N ksf (Seed et al., 1984), G_{max} was found to be approximately 4300 ksf. Strength evaluations from IECO's consolidated undrained (CU) testing and the approximate empirical relationship that $G_{max}/S_u \approx 1000$ to 2000 suggest that $G_{max} \approx 2500$ ksf to 7000 ksf. Estimates of the initial modulus of elasticity from stress-strain data from these CU tests put G_{max} in the neighborhood of 3000 ksf, and this might be interpreted as an approximate estimate of the likely value. Finally, Hardin & Drnevich (1970) developed the following expression for estimating G_{max} of soils:

$$G_{max} = 14.760 \times \frac{(2.973 - e)^2}{1 + e} (OCR)^a (\sigma'_m)^{\frac{1}{2}} \qquad \text{(Eq. 1)}$$

Where G_{max} = maximum shear modulus, in ksf

e = void ratio

OCR = overconsolidation ratio

a = a parameter that depends on the plasticity index of the soil

σ'_m = mean principal effective stress, in psf

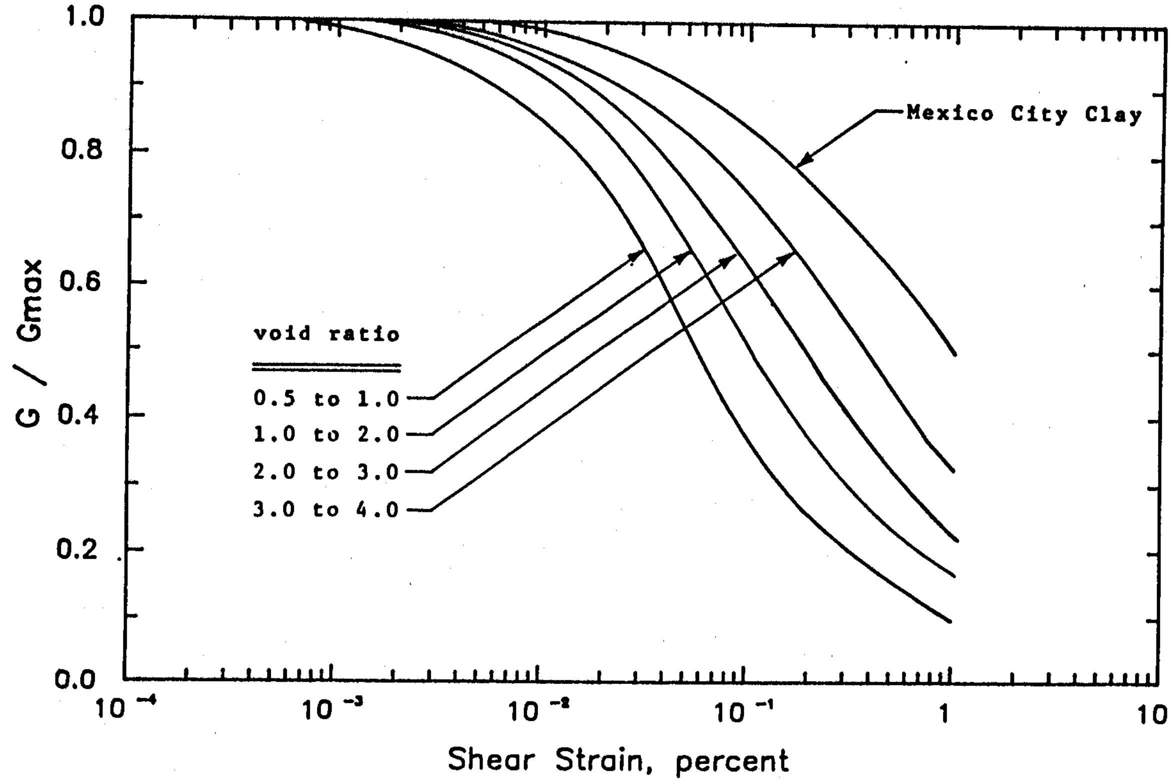
Based on the information developed by IECO's studies in the early 1970's, $e \approx 0.92$ and $a \approx 0.24$ for the compacted crushed shale fill, and the soil's OCR which was estimated to be around 2, the Hardin & Drnevich equation then calculates G_{max} in the range of 2500 ksf to 4000 ksf. In general, there is fairly good agreement between these various estimates of G_{max}, and representative "overall average" values of $(G_{max})_{avg} \approx 3000$ ksf to 4000 ksf were selected for use in the response analyses described herein. Actual values used in analyses varied slightly with confinement, as described previously and shown in Table 4-1.

Given an estimate of G_{max}, one must next determine how the shear moduli reduce at higher levels of strain. A number of investigators have recently examined shear strain dependent modulus degradation of cohesive soils as a result of the "unusual" dynamic properties exhibited by Mexico City clays in the Mexico City Earthquake of September 19, 1985 (e.g. Sun, et al., 1988, Vucetic and Dobry, 1988). Two of the resulting modulus reduction relationships are presented in Fig. 4-1. The normalized modulus reduction curve for clays with a void ratio in the range of 0.5 to 1.0 and the normalized modulus reduction curve for clays with a plasticity index of 10-20 are nearly identical, and correspond best with the actual index properties of the Puddingstone Dam's sandy, silty clay material if the actual plasticity index of 20-30 is reduced by about 10 to allow for the actual gradation of the soil. Additionally, these curves agree well with the modulus reduction trends exhibited by other silty clays detailed in the Sun et al. (1988) report, and are in good agreement with the modulus reduction curve for clayey soils with PI $\approx$ 15 proposed by Vucetic and Dobry (1988). This characterization also agrees well with the high strain level dynamic shear moduli determined from tests performed on the soil in the 1976 geotechnical study (see Figure 2-3). Accordingly, this normalized modulus reduction relationship will be used in conjunction with the previous estimates of G_{max} to characterize the strain dependent dynamic moduli of the embankment soil. Table 4-2 lists the shear modulus reduction vs. shear strain curve used in the analyses described herein.

The other critical soil parameter required in dynamic response analysis is the strain dependent damping ratio relationship. Not as much soil specific data is available to characterize a soil's strain dependent damping ratio. A review of the damping vs. shear strain data presented by Sun et al. (1989) and Seed et al. (1984), however, suggest that sandy silty clays possess a damping curve which is intermediate between the curves

Table 4-1: Puddingstone Dam: Dynamic Soil Properties Modelled

Material	G_{max} (ksf)	Unit Weight: γ (pcf)	Poisson's Ratio: ν	Shear Wave Velocity: V_s (ft/sec)
Embankment:				
Shallow	2700-3600	117	0.45	--
Average	3000-4000	117	0.45	--
Deep	3300-4400	117	0.45	--
Toe Drain	2600-3200	130	0.35	--
Rock Abutment	--	145	0.40	5000



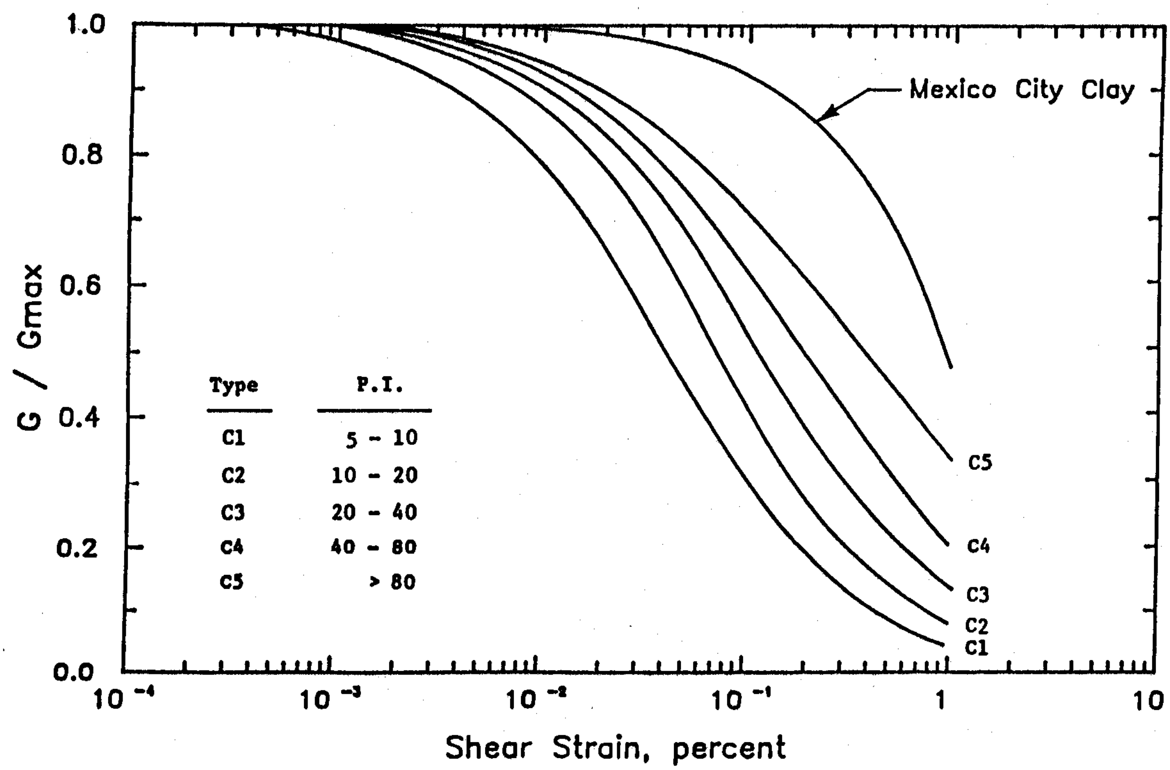


Figure 4-1: NORMALIZED MODULUS REDUCTION RELATIONSHIPS FOR CLAYS WITH DIFFERENT VOID RATIOS AND PLASTICITY INDEXES (After Sun et al., 1988)

Table 4-2: Strain-Dependent Embankment Soil Properties Modelled: Compacted Sandy Silty Clay

Shear Strain (γ)	Normalized Dynamic Modulus $\frac{G}{G_{max}}$	Damping Ratio
1×10^{-4}%	1.00	2%
3×10^{-4}%	1.00	3%
1×10^{-3}%	0.99	4%
3×10^{-3}%	0.97	5%
1×10^{-2}%	0.86	7.5%
3×10^{-2}%	0.65	11%
1×10^{-1}%	0.40	15%
3×10^{-1}%	0.20	21%
1.0%	0.10	26%

Table 4-3: Strain-Dependent Embankment Soil Properties Modelled: Cohesionless Toe Drain Material

Shear Strain (γ)	Normalized Dynamic Modulus $\frac{G}{G_{max}}$	Damping Ratio
1×10^{-4}%	1.00	1%
3×10^{-4}%	0.99	1.3%
1×10^{-3}%	0.95	1.6%
3×10^{-3}%	0.87	3.1%
1×10^{-2}%	0.72	5.8%
3×10^{-2}%	0.53	9.5%
1×10^{-1}%	0.53	15%
3×10^{-1}%	0.20	21%
1.0%	0.11	25%

recommended for cohesive soils and sandy soils in these two references. This intermediate curve corresponds to a curve just under the upper bound damping ratio curve originally proposed by Seed and Idriss (1970) for cohesive soils shown in Fig. 4-2 (Sun, et al., 1988). This curve also agrees well with the dynamic laboratory testing results presented in the 1976 geotechnical study of the Puddingstone Dam sandy silty clay material (See Fig. 2-4), and provides damping ratios about 3% to 6% higher than the damping vs. shear strain relationship proposed by Vucetic and Dobry (1988) for clays with PI $\approx$ 15. This upper bound curve from Fig. 4-2 will be used in most of the analyses, though comparing results from analyses performed utilizing the average damping curve for clays from Fig. 4-2 will provide insight into the sensitivity of the damping ratio curve in dynamic response analyses of this type of embankment. The upper bound damping curve, taken as a "best estimate" for use in the response analyses described herein, is presented in Table 4-2.

In addition to the main embankment material, dynamic properties were required for the toe drain composed of boulder and gravel sized particles and the stiff abutment rock. Dynamic properties of the cohesionless toe drain were relatively unimportant due to the small size of this zone and its location, and were modelled using modulus degradation and damping vs. shear strain relationships recommended for gravelly soils by Seed et al. (1984), with $K_{2,max}$ = 90. The relationships used are presented in Table 4-3. The abutment rock shear wave velocity was found to have a relatively minor influence on calculated peak ground accelerations when varied from 3600 ft/sec to 8000 ft/sec. Since it was found not to be a highly sensitive parameter, the rock shear wave velocity was modelled as v_s = 5000 ft/sec for the balance of the analyses performed. Finally, a few other relatively nonsensitive soil parameter values were required. They are presented in Table 4-1 along with a summary of the principal "typical" dynamic soil properties previously described.

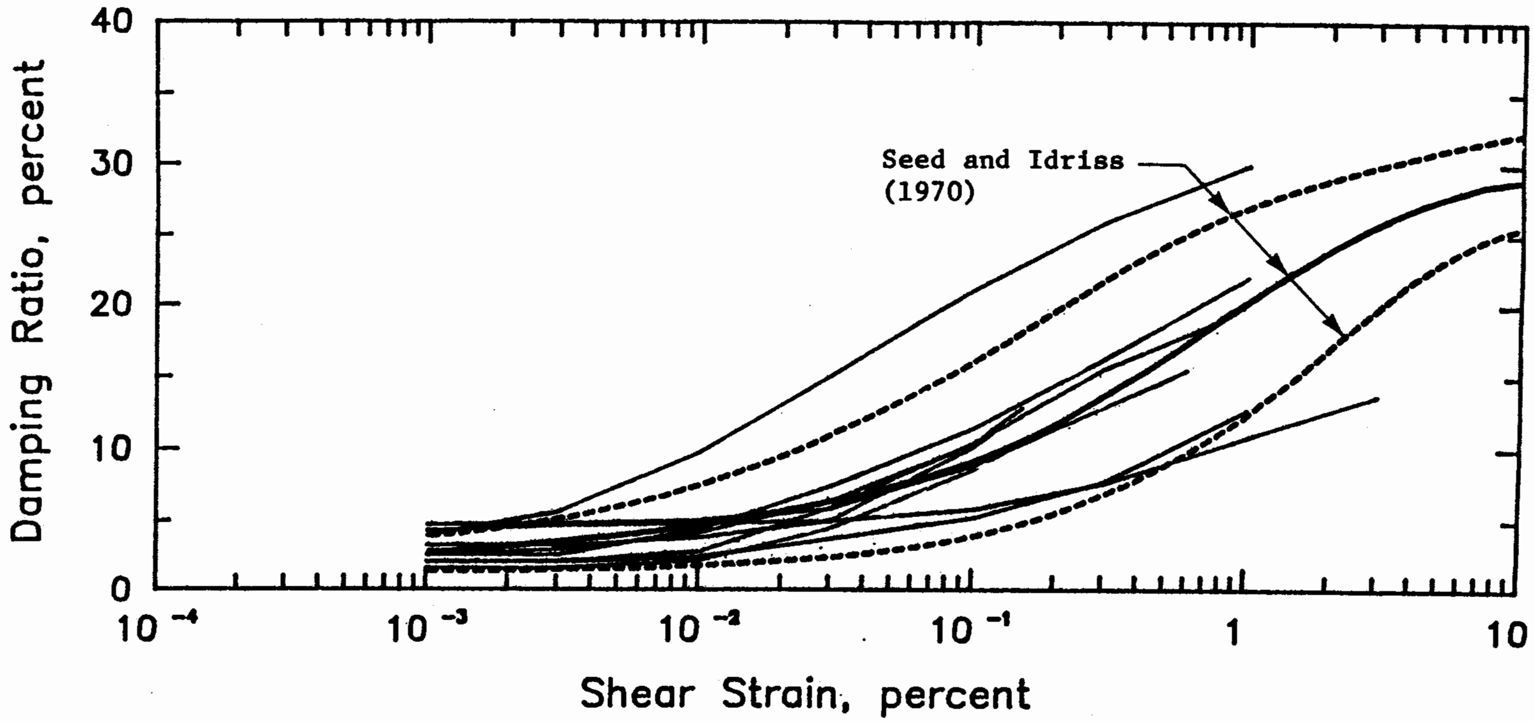


Figure 4-2: GENERALIZED DAMPING vs. SHEAR STRAIN RELATIONSHIP FOR COHESIVE SOILS AS PROPOSED BY SUN et al., 1988

Analytical Studies

The dynamic response of the Puddingstone Dam to the 1987 Whittier Narrows earthquake was illustrated in Fig. 3-11. As discussed in Chapter 3, maximum spectral accelerations at the crest of Puddingstone Dam appear to have been amplified by a factor of nearly 4 ($(S_a)_{CREST} \approx 0.81$ g : $(S_a)_{ROCK} \approx 0.24$ g) and peak ground accelerations to have been tripled ($(a_{max})_{CREST} \approx 0.19$ g : $(a_{max})_{ROCK} \approx 0.07$ g). The analytical studies described herein were performed in order to investigate whether these types of one- and two-dimensional analytical procedures can correctly predict this magnitude of amplification of accelerations in the Puddingstone Dam, and the general observed response characteristics. Specifically, this study attempts to evaluate the predominant period of the earth structure, predict the peak ground acceleration and maximum spectral acceleration at the crest and face of the dam, provide the correct shape of the acceleration response spectra at the crest and downstream face, and finally estimate the actual dynamic properties of Puddingstone Dam.

Two approaches were taken to evaluate the observed 3-D predominant period of the Puddingstone Dam. The recorded crest response motion suggested a predominant period of either 0.26 seconds or 0.45 seconds, as shown in Fig. 3-11. Because of the spectra's multiple peaks, there was some question as to whether the slightly higher peak at a period of 0.26 seconds actually indicated the dam's true predominant period. This peak at 0.26 seconds might represent interaction of the dam's second mode of response with the high frequency input motions, in which case a slightly lower peak on the spectral response might better represent the dam's predominant period at the observed strain levels. Accordingly, a section of the crest response accelerogram representing the initial period of decay of strong shaking was analyzed, and found to have a predominant period of about 0.41 to 0.45 seconds, as shown in Fig. 4-3. This technique was developed by Mejia and Seed (1980) to evaluate the predominant period of the Oroville Dam. After the strong input shaking has ceased, the dam should respond during largely

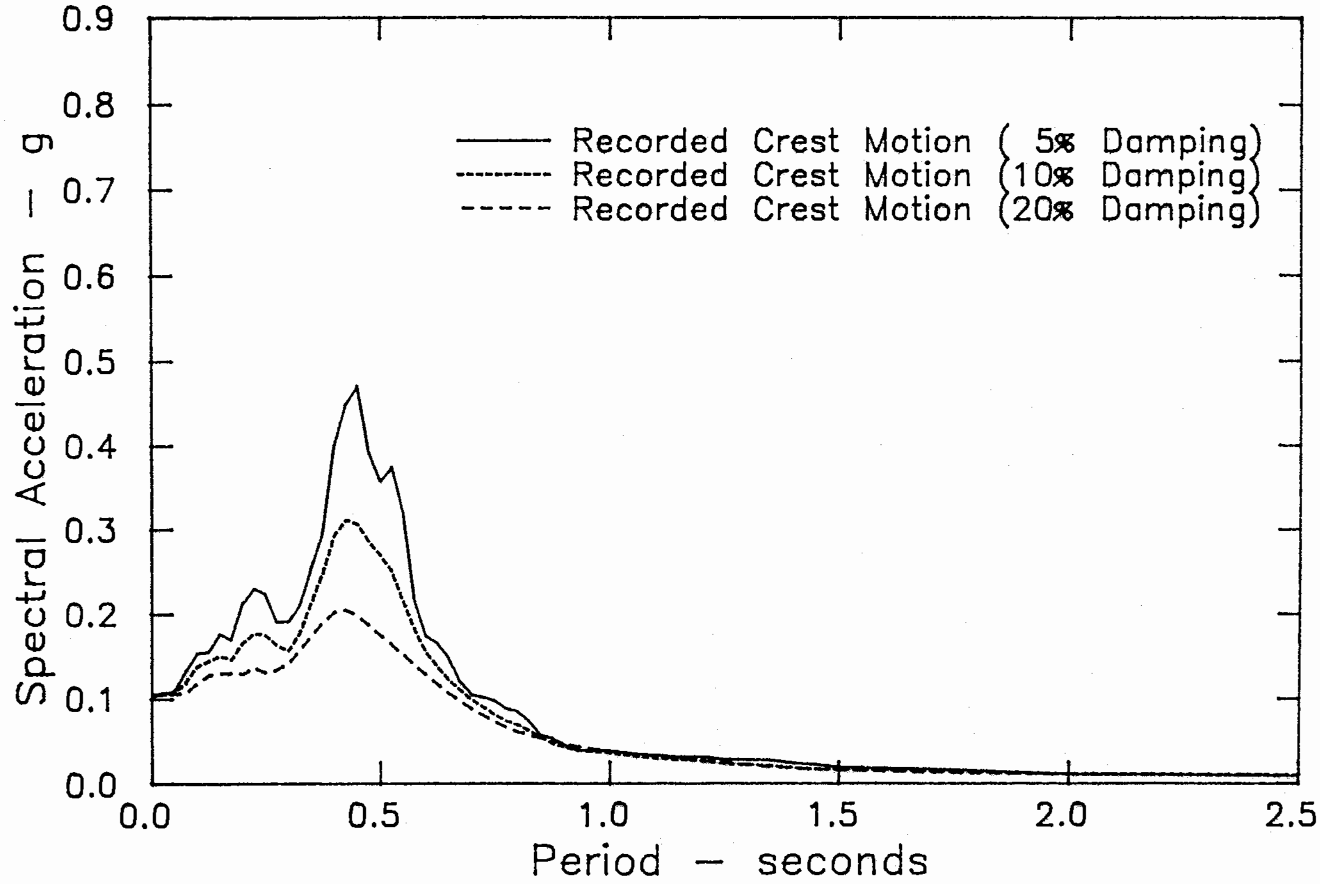


FIG. 4-3: CREST TRANSVERSE RESPONSE SPECTRA RECORDED FROM T = 7 SEC TO 12 SEC

free vibrations at its predominant period. Observing the crest's response spectra for motions after the initial 7 seconds (the duration of strong shaking) of the earthquake record clearly indicates one main peak at the dam's predominant period. These two approaches find the dam's predominant period to be $T_p \approx 0.41$ to 0.45 seconds.

The analytical techniques applied to Puddingstone Dam provide estimates of the earth structure's plane strain maximum section (2-D) predominant period. By scaling for the actual 3-D geometry effects as described by Makdisi et al. (1982), a 2-D (plane strain) predominant period can be transformed into a corresponding fully 3-D predominant period. For a dam in a rectangular/triangular canyon with $L/H \approx 4.5$, $(T_p)_{3\text{-}D} \approx 0.8 \times (T_p)_{2\text{-}D}$. This would indicate that the "equivalent" 2-D period corresponding to the actual 3-D period of 0.41 to 0.45 seconds would be approximately $(T_p)_{2\text{-}D} \approx 0.5$ to 0.55 seconds.

Ambraseys and Sarma (1967) developed a relationship for estimating the predominant period of 2-D planar dam sections as

$$T_p \approx 2.61 \times H/V_s \qquad \text{(Eqn. 2)}$$

where V_s is the average shear wave velocity (based on G_{avg}, the average shear modulus) within the embankment, and H is the embankment height. For the levels of shear strain likely to have been induced within the Puddingstone Dam by the Whittier Narrows Earthquake, the representative G_{avg} is likely to have been about 85% of G_{max}, so that $(T_p)_{2D}$ is estimated to be about 0.47 to 0.54 seconds. From the Fourier Amplification Function computed by FLUSH for the cases where $(G_{max}) \approx 3000$ to 4000 ksf, $(T_p)_{2\text{-}D} \approx 0.47$ to 0.55 seconds. Scaling for 3-D geometry effects, as described above, these analytical procedures estimate the 3-D predominant period of the Puddingstone Dam to be approximately 0.38 to 0.43 seconds. Although a bit low, this range of values satisfactorily agrees with the observed 3-D predominant period of 0.41 -

0.45 seconds. Hence, these simple empirical procedures appear to provide acceptable estimates of the Puddingstone Dam's predominant period. Furthermore, the results of 2-D dynamic finite element analyses suggest that the appropriate $(G_{max})_{avg}$ value of the sandy silty clay embankment material is nearer to 3000 ksf. With this information, one might narrow the range of estimated $(G_{max})_{avg}$ values of the Puddingstone Dam's sandy silty clay material to 3000 to 3500 ksf.

Figure 4-4 shows the finite element mesh used for the 2-D plane strain dynamic finite element (FEM) analyses of Puddingstone Dam. Comparative analyses showed that a frequency cut-off above 12 Hz provided a negligible loss of accuracy in performing these analyses. More than fifteen separate analyses were performed using the program FLUSH to study the sensitivity of the results to variations in the critical soil parameters used in the analyses. When a set of embankment dynamic moduli values were chosen that produced the correct 3-D predominant period of 0.41 to 0.45 seconds (e.g. $(G_{max})_{avg} \approx$ 4000 ksf), FLUSH computed a dynamic response which most closely mirrored the response actually observed. Figure 4-5 shows a comparison between the response spectra for the resulting predicted crest and mid-downstream face motions when $(G_{max})_{avg} \approx$ 4000 ksf vs. those actually recorded. The predicted peak acceleration of a_{max} = 0.21 g at the crest agrees well with the recorded peak of a_{max} = 0.19 g, and the predicted crest response spectra is in fair general agreement with the observed crest motions. The predicted peak acceleration of a_{max} = 0.15 g at the downstream face station also agrees well with the recorded a_{max} = 0.18 g, and the spectral response agreement is fairly good here too. The 2-D FEM analyses using slightly stiffer soil properties to account for 3-D geometric effects provided a good prediction of the observed response, with an accuracy level which was sufficient as to provide a good basis for engineering analyses.

Figure 4-6 shows that varying the representative maximum dynamic moduli values from 3500 ksf to 4500 ksf does not appreciably change the results of the FLUSH

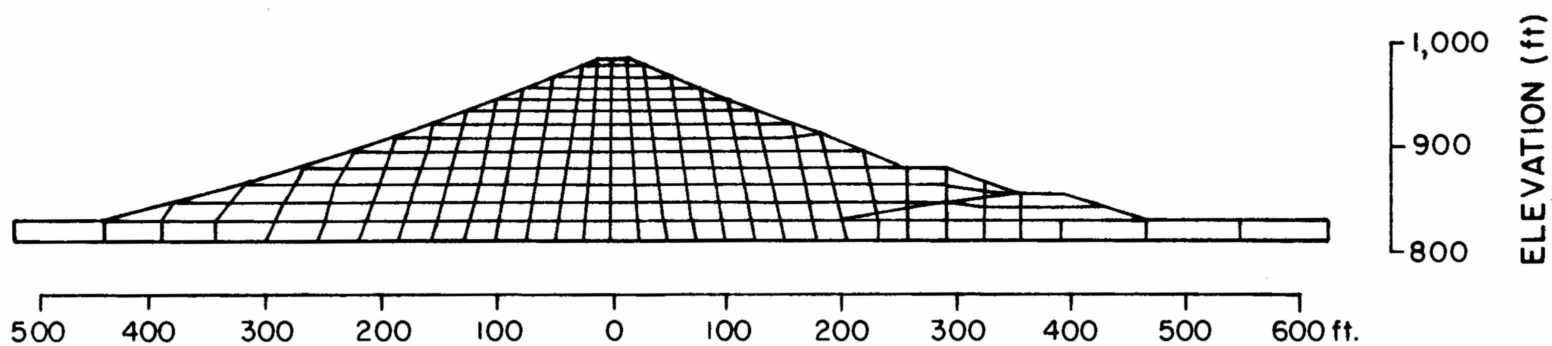
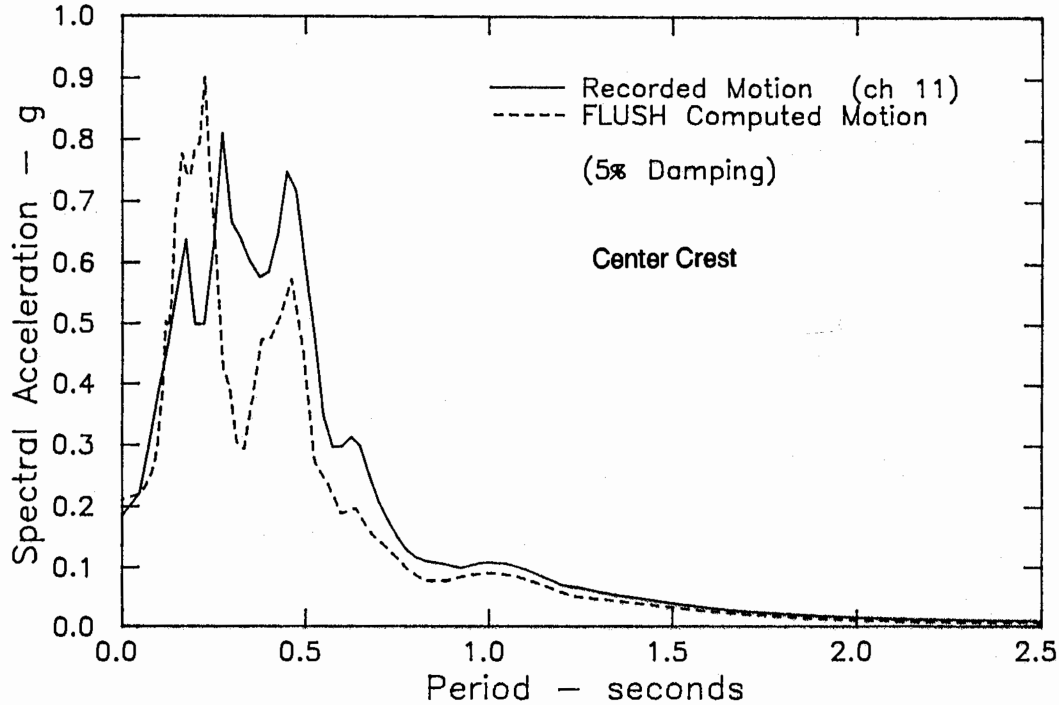


Figure 4-4: FINITE ELEMENT MESH USED TO MODEL PUDDINGSTONE DAM



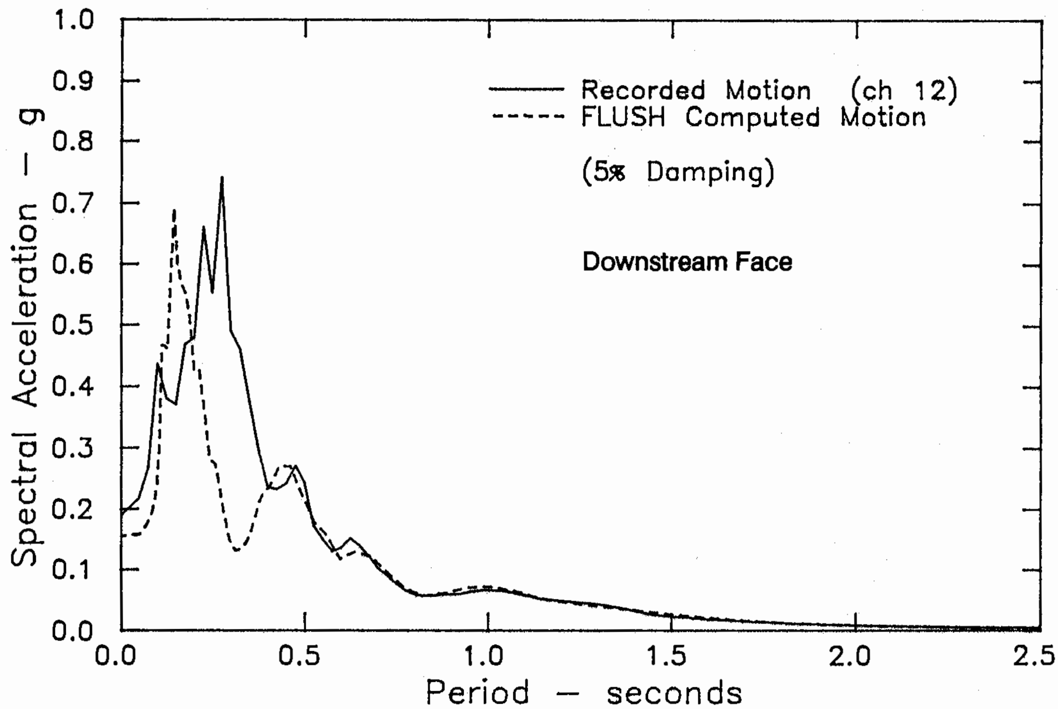


FIG. 4-5: COMPARISON BETWEEN PREDICTED AND OBSERVED RESPONSE SPECTRA: 2-D FEM ANALYSES WITH $(G_{max})_{avg} \approx 4000$ KSF

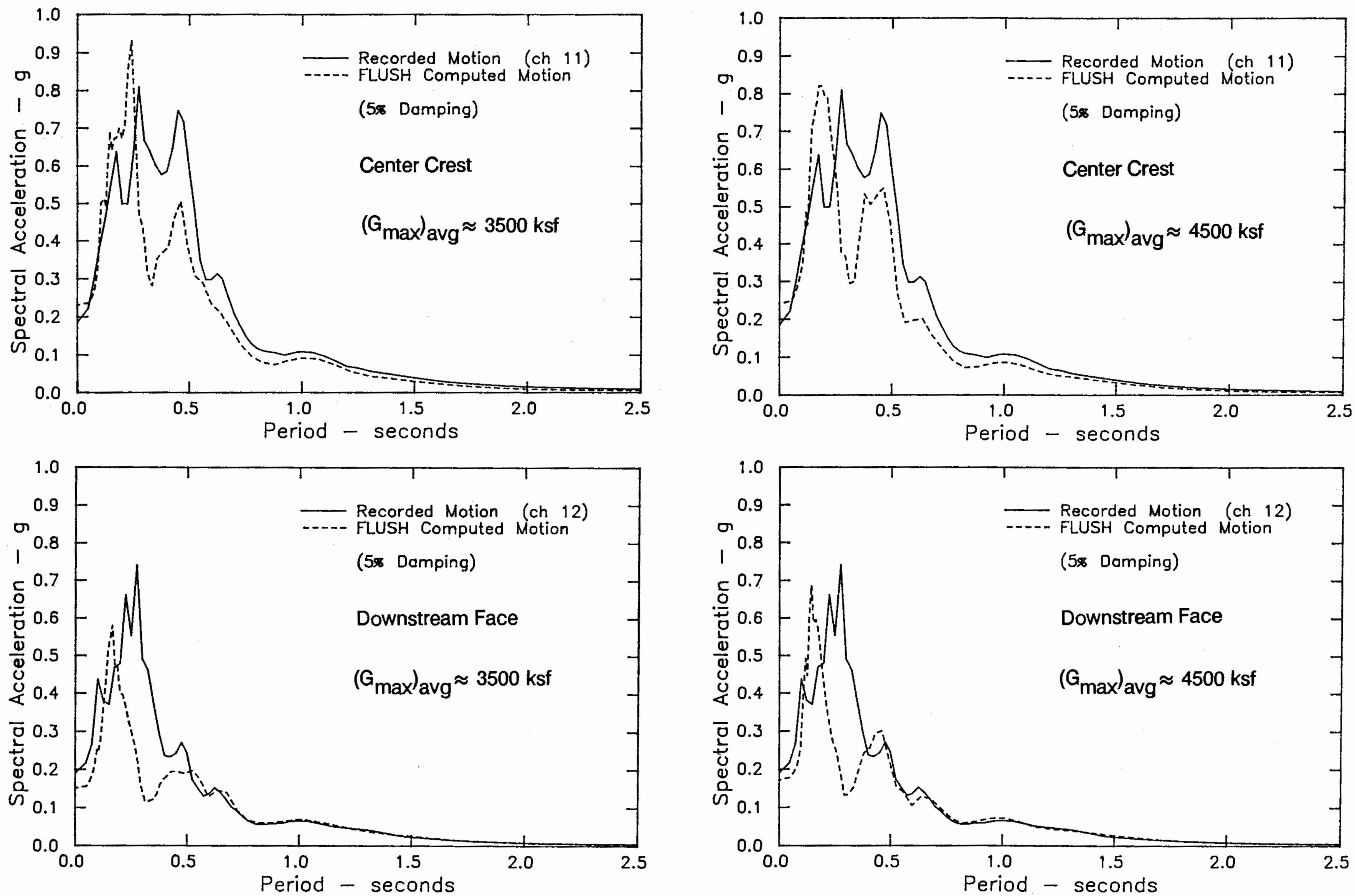
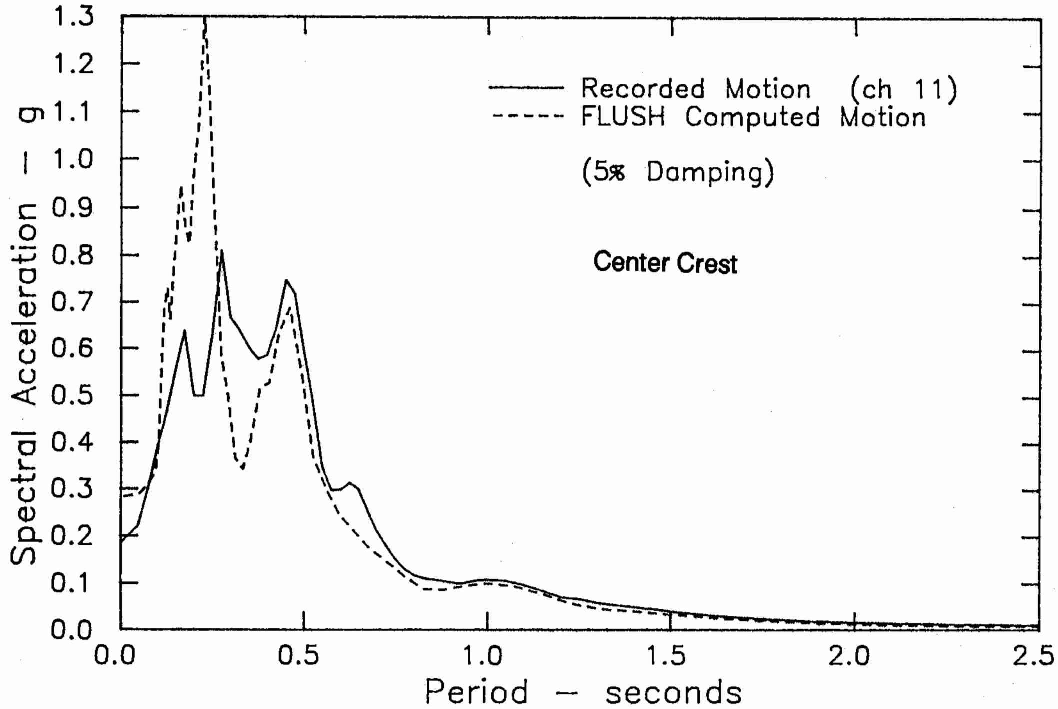


FIG. 4-6: COMPARISON BETWEEN COMPUTED AND OBSERVED RESPONSE SPECTRA: 2-D FEM ANALYSES WITH $(G_{max})_{avg} \approx$ 3500 KSF AND 4500 KSF

analyses. In fact, unless unrealistically low or high values of this soil parameter are used in the FLUSH analysis, G_{max} and its associated normalized modulus reduction curve are not extremely sensitive. This is not the case, however, when chosing the strain dependent damping ratio curve. If one does not account for the large amounts of silt and sand size particles in this material and uses an average clay damping curve, FLUSH computes high values of peak ground acceleration (0.29 g) and maximum spectral acceleration (1.31 g) at the crest as shown in Fig. 4-7. On the other hand, this 2-D representation of the Puddingstone Dam might require slightly higher damping values to compensate for the inability of this 2-D analyses to model the 3-D geometric effects. Previous 2-D FLUSH analyses have shown better agreement with observed dynamic response of earth embankments when higher, but still reasonable levels of damping were employed.

The response at the mid-downstream face is not so sensitive to variations in damping, however, and reasonable values of $a_{max} \approx 0.18$ g and $(S_a)_{max} \approx 0.57$ g were computed using both damping curves. The face response exhibited a relatively low level of insensitivity to even major variations in all of the soil parameters studied. The lowest and highest values of peak ground acceleration computed were 0.12 g and 0.18 g, respectively. These analyses included G_{max} values of 2500 ksf to 8800 ksf, average and upper bound clay damping curves, rock shear wave velocities from 3600 fps to 8000 fps, and the use of Channels 3 or 13 as input rock motions. This insensitivity suggests that if it is difficult to reliably characterize the materials of an earth embankment, one might want to calculate the mid-downstream face peak ground acceleration and scale it by the empirically derived 1.5 factor to provide a rough estimate of the crest peak ground acceleration for all but very high levels of maximum acceleration. In this case study, $a_{max} \approx 0.13$ g to 0.15 g at the face for reasonable variations of the input parameters, so one would predict $a_{max} \approx 0.19$ g to 0.22 g at the crest of the embankment. This range of values agrees well with the observed peak ground acceleration of 0.19 g.



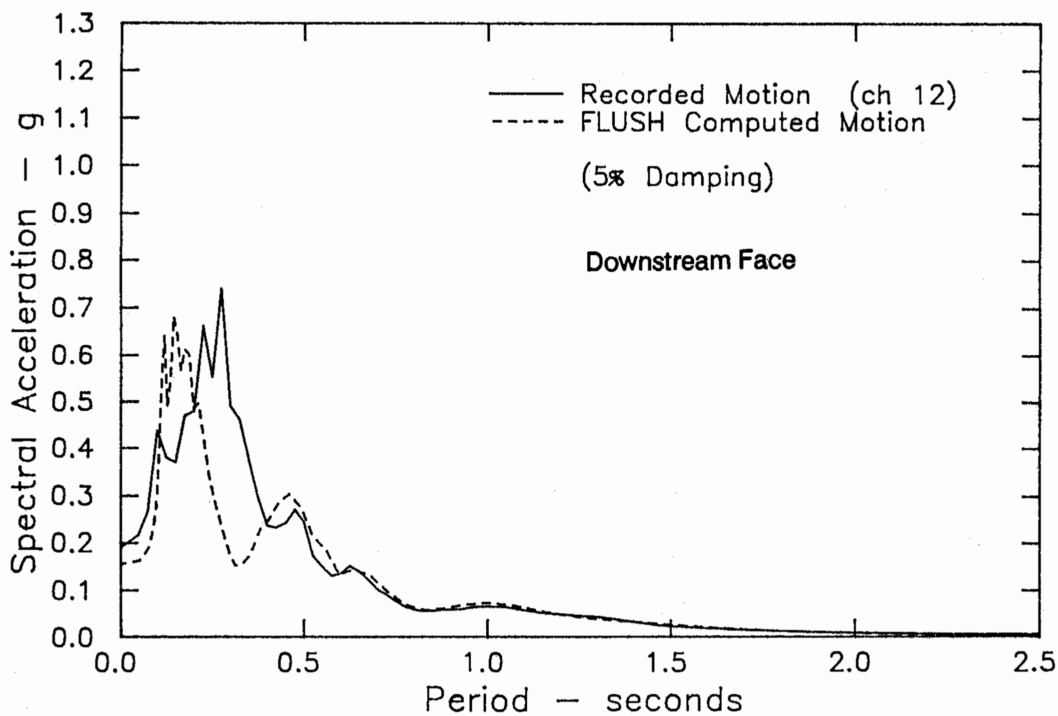
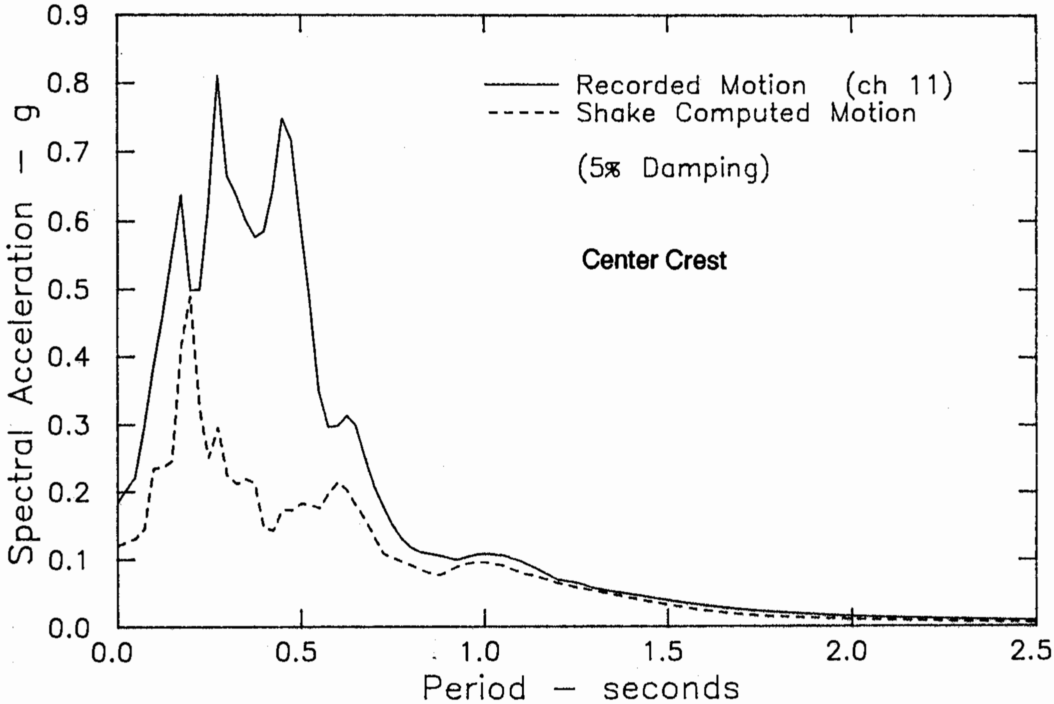


FIG. 4-7: COMPARISON BETWEEN COMPUTED AND OBSERVED RESPONSE SPECTRA: 2-D FEM ANALYSES WITH $(G_{max})_{avg} \approx 4000$ KSF USING LOW DAMPING RATIOS

Finally, the FLUSH analyses did demonstrate sensitivity to the selected input rock motions. As discussed in Chapter 3 of this report, the Channel 3 recording displayed the most "rock-like" motion. Although the Channel 13 rock motion did contain some amplification of spectral accelerations at lower frequencies, one might not expect this largely similar rock input motion to change the FLUSH results much. Yet, using Channel 13 as the input rock motion increases calculated maximum spectral accelerations by 25% to 40%. Predicted peak ground accelerations, however, agreed well with those calculated utilizing Channel 3 and with those observed. It is important to employ a fairly "clean" rock motion as the input motion if one hopes to predict the correct shape of the embankment crest's response spectra.

Overall, these results support the use of 2-D FEM analyses in engineering studies of earth structures like the Puddingstone Dam. Similar analyses were performed using 1-D analyses of "representative" columnar sections through the crest and downstream face of Puddingstone Dam. These SHAKE analyses were invaluable in our preliminary sensitivity analyses because of their simplicity and low cost. The SHAKE results, using $G_{max,avg} \approx$ 4,000 ksf, are presented in Figure 4-8. As expected, these analyses greatly under-predicted both peak acceleration and the spectral response at the crest station, as the one-dimensional SHAKE analyses cannot model the effects of the dam's geometry, but they provided a somewhat better prediction of the observed response at the downstream face station. In fact, if the 1.5 crest to mid-face scaling factor is applied to the computed $a_{max} \approx 0.14$ g at the face of the dam, one would predict a crest peak ground acceleration of 0.21 g which agrees well with the observed $a_{max} \approx 0.19$ g. Thus, it may be possible in some cases to use one-dimensional (columnar) response analyses to provide a reasonable estimate of the peak acceleration at the face of the dam from which a rough estimate of peak crest accelerations can be made.



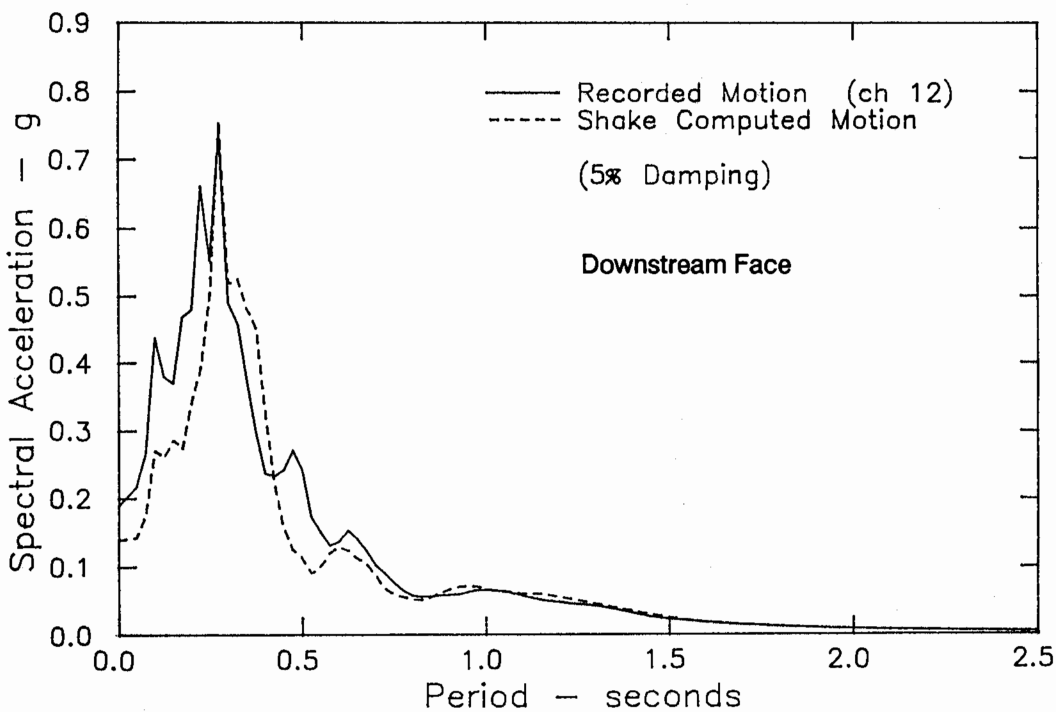


FIG. 4-8: COMPARISON BETWEEN PREDICTED AND OBSERVED RESPONSE SPECTRA: 1-D COLUMNAR ANALYSES WITH $(G_{max})_{avg} \approx$ 4000 KSF

Chapter 5

SUMMARY AND CONCLUSIONS

The Whittier Narrows Earthquake of October 1, 1987 provided an excellent record of the seismic response and performance of Puddingstone Dam. Specifically, records obtained by sensors on the embankment's crest and mid-downstream face, in conjunction with recorded "near rock" motions on the surrounding abutments, provided a valuable opportunity to study the variation of strong motions transverse to the dam, and thus to study the dam's response characteristics of principal engineering interest. The records indicated that bedrock motions were amplified as they passed through the earth embankment to the slope face and crest of the dam. In fact, maximum spectral accelerations at the crest of the embankment were amplified by a factor of 4 and peak ground accelerations were tripled. The Puddingstone Dam's predominant period was found to be in the range of 0.41 - 0.45 seconds at the levels of shaking produced by this earthquake.

Good agreement between the observed response characteristics of Puddingstone Dam and the response characteristics predicted using both simple empirical methods as well as 2-D finite element analyses, based on established methods for evaluation and modelling of strain-dependent dynamic shear moduli and damping, provides good support for these modelling and analytical techniques. Proper interpretation of the analytically predicted response characteristics requires appropriate consideration of three-dimensional effects not modelled in the 1-D and 2-D analyses performed. These effects were only moderate, however, for this dam with a crest length to dam height ratio of $L/H \approx 4.5{:}1$, and the moderately calibrated 2-D finite element analyses provided response predictions for both the crest and mid-downstream face motions which were in sufficient agreement with observed response as to provide a good basis for engineering analyses. Even the simplier 1-D analyses provided good approximate predictions of peak accelerations for the downstream face, though these simpler analyses were unable to provide a reasonable prediction of the observed crest response.

Although these analytical procedures appear to have been suitable for the task of predicting the observed dynamic response of Puddingstone Dam, it should also be noted that soil parameter sensitivity studies suggested that the particular strain-dependent damping ratio curve selected for modelling of the main embankment material in the 2-D response analyse could significantly alter the computed response. 2-D dynamic analysis procedures utilizing current "average" clay damping curves appear to provide conservative estimates of peak ground accelerations at the crest of this dam, but the use of a strain-dependent damping relationship intermediate between those normally used for clayey soils of moderate plasticity and those used to model sandy (cohesionless) soils provided good results for the sandy, silty clay material which comprises the main embankment section. The analytical techniques applied in this study were less sensitive to variations in other soil parameters, and it was concluded that use of a "reasonable" range of values of dynamic moduli provided good analytical agreement with the observed field response.

Finally, it must be noted that although the level of agreement between computed and measured peak horizontal transverse accelerations at both the crest and central downstream face were very good, the spectral content of the computed motions was only in fair agreement with the observed motions. This appears to have been due, at least in part, to three-dimensional effects not modelled in the 2-D, plane strain dynamic finite element analyses performed. Accordingly, it would be interesting to repeat these analyses using fully 3-D dynamic finite element analysis procedures. Such studies are, however, beyond the scope of this current effort.

In summary, the relatively simple 2-D dynamic finite element analyses performed, using only the relatively scant soil data available, and using the "equivalent linear" method to model nonlinear, shear strain-dependent dynamic moduli and damping, provided amply adequate modelling of the observed dam response characteristics for most engineering applications. Lack of more nearly perfect agreement with the observed (recorded) response characteristics, however, render this case study a potentially valuable one with regard to calibration and verification of increasingly sophisticated dynamic analysis and modelling techniques, and such further studies should be encouraged.

REFERENCES

Ambraseys, N. N. (1960) "On the Shear Response of a Two Dimensional Wedge Subjected to an Arbitrary Disturbance," Bulletin of the Seismological Society of America, Vol. 50, Jan., 1960, pp. 45-56.

Bolt, B. A. and Abrahamson, N. A. (1982) "Attenuation Relation for Peak and Expected Accelerations of Strong Ground Motions," Bulletin of the Seismological Society of America, Vol. 72, No. 6, pp. 2307-2321.

Boulanger, R. W., Seed, R. B., Seed, H. B. and Bray, J. D. (1989) "Analyses of the Seismic Response of the Cogswell Dam in the 1987 Whittier Narrows Earthquake," report in preparation for the California Strong Motion Instrumentation Program (CSMIP), Dept. of Civil Engineering, University of California at Berkeley.

Campbell, K. W. (1981) "Near Source Attenuation of Peak Horizontal Acceleration," Bulletin of the Seismological Society of America, Vol. 1, pp. 2039-2070.

International Engineering Company, Inc. (IECO) "Puddingstone Dam, Geotechnical Investigation and Stability Analysis," report prepared for the Los Angeles County Flood Control District, June, 1970.

International Engineering Company, Inc. (IECO) "Puddingstone Dam, Geotechnical Investigation and Dynamic Stability Analysis," report prepared for the Los Angeles County Flood Control District, June, 1976.

Joyner, W. B. and Boore, D. M. (1981) "Peak Horizontal Acceleration and Velocity from Strong-Motion Records Including Records from the 1979 Imperial Valley, California Earthquake," Bulletin of the Seismological Society of America, Vol. 71, pp. 2011-2038.

Lai, S. S. and Seed, H. B. (1985) "Dynamic Response of the Long Valley Dam in the Mammoth Lake Earthquake Series of May 25-27, 1980," Report No. UCB/EERC-85/12, Earthquake Engineering Research Center, University of California, Berkeley, November.

Lysmer, J., Udaka, T., Tsai, C-F. and Seed, H. B. (1975) "FLUSH - A Computer Program for Approximate 3-D Analysis of Soil-Structure Interaction Problems," Report No. EERC 75-30, Earthquake Engineering Research Center, University of California, Berkeley, November.

Makdisi, F. I., Kagawa, T. and Seed, H. B. (1982) "Seismic Response of Earth Dams in Triangular Canyons," Journal, Geotechnical Engineering Division, ASCE, Vol. 108, No. GT10, October, pp. 1328-1337.

Mejia, L. H. and Seed, H. B. (1981) "Three Dimensional Dynamic Response of Earth Dams," Report No. UCB/EERC-81/15, Earthquake Engineering Research Center, University of California, Berkeley, September.

Mejia, L. H. and Seed, H. B. (1984) "Comparison of 2-D and 3-D Dynamic Analyses of Earth Dams," Journal, Geotechnical Engineering Division, ASCE, Vol. 109, No. GT11, November, pp. 1383-1398.

Schnabel, P. B., Lysmer, J. and Seed, H. B. (1972) "SHAKE - A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites," Earthquake Engineering Research Center, Report No. EERC 72-12, University of California, Berkeley, December.

Seed, H. B. and Idriss, I. M. (1970) "Soil Moduli and Damping Factors for Dynamic Response Analyses," Report No. EERC 70-10, Earthquake Engineering Research Center, University of California, Berkeley, December.

Seed, H. B., Wong, R. T., Idriss, I. M. and Tokimatsu, K. (1984) "Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils," Report No. UCB/EERC-84/14, Earthquake Engineering Research Center, University of California, Berkeley, September.

State of California: The Resources Agency, Department of Water Resources and Division of Safety of Dams (1979) "Puddingstone Dam: Phase I: Inspection Report for December 1979," National Dam Inspection Program: Department of the Army, The Corps of Engineers, Sacramento District, File #32-9, December.

Sun, J. I., Golesorkhi, R. and Seed, H. B. (1988) "Dynamic Moduli and Damping Ratios for Cohesive Soils," Report No. UCB/EERC-88/15, Earthquake Engineering Research Center, University of California, Berkeley.