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DYNAMIC RESPONSE OF LONG VALLEY DAM IN THE MAMMOTH LAKE EARTHQUAKE SERIES OF MAY 25–27, 1980

by

S. S. LAI
H. BOLTON SEED

A report on research sponsored by
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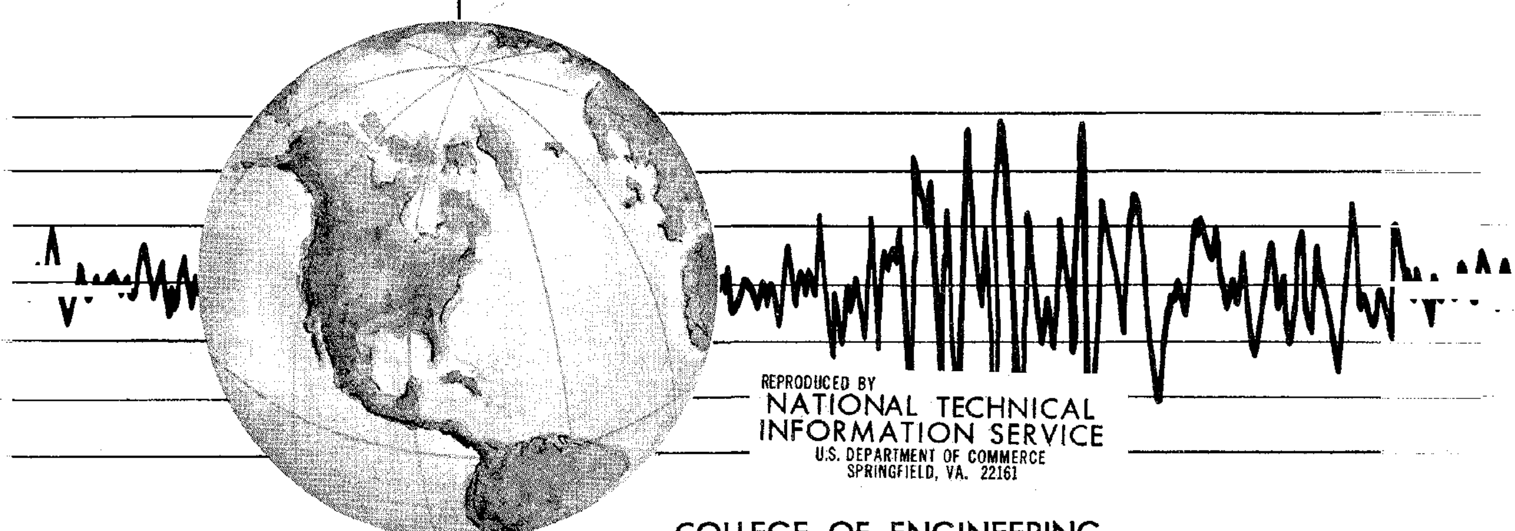

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16. Abstract (Limit: 200 words)
The data recorded on the Long Valley Dam during the Mammoth Lakes earthquake series provided an excellent opportunity to check the accuracy of dynamic analysis procedures for determining the seismic response of embankment dams. In this investigation the recorded seismic performance of Long Valley Dam to the earthquake of May 27, 1980, was successfully simulated by 2-D and 3-D response analyses using appropriate combinations of the values of dynamic shear modulus coefficient $(K_2)_{max}$ in the analyses.

Another 3-D response analysis was carried out to determine the dynamic response of the embankment using the same values of $(K_2)_{max}$ and the new shear modulus curve for gravel material proposed by Seed, et al. This led to better estimates of peak accelerations for Long Valley Dam but poorer overall motion characteristics compared with those obtained using a modulus attenuation curve developed from tests on sands.

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S.S. Lai[1] and H. Bolton Seed[2]

Introduction

In recent years much attention has been given to methods of analyzing the response of earth dams and earth embankments to earthquake shaking. However the applicability of these analytical procedures can only be evaluated when the results are compared with the observed response of prototype structures during actual earthquakes or carefully conducted experimental observations of the response of small-scale structures. An excellent opportunity to check the accuracy of dynamic analysis procedures for determining the seismic response of embankment dams has recently been provided by the excellent data recorded on the Long Valley Dam during the Mammoth Lakes earthquake series of May 25 to 27, 1980. A study of response prediction procedures for this embankment provides a unique opportunity to evaluate the suitability of analytical methods for future applications.

Long Valley Dam, shown in Fig. 1, which retains Lake Crowley Reservoir, is located in Mono County, California, about 22 miles northwest of the City of Bishop and approximately 240 miles north of the City of Los Angeles. Construction of the dam was started in the late 1930's and completed in September 1941. The dam is supported on bedrock and has a maximum height of 126 feet above the original streambed elevation; it has a crest length of about 600 ft. The reservoir has a capacity of about 183,470 acre feet.

The dam is essentially a homogeneous section dam, with the main compacted fill consisting of sand and gravel with sufficient fines to produce a permeability somewhat less than 7×10^{-6} cm/sec, and with outer shells consisting

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FIG. 1 LONG VALLEY DAM

of dumped-sluiced small rock and coarse fines, see Fig. 2. A 10 feet thick layer of rock rip-rap was placed to protect the upstream face of the dam. A subdrain system with longitudinal and transverse drains was provided in the downstream shell to reduce the seepage and saturation problems at the toe of the embankment.

As can be seen from the plan view of the dam (Fig. 3), the upstream geometry of the dam shows a significant concavity from the surrounding abutments and the crest to the bottom and center of the upstream face of the embankment. The outer shells of the embankment have slopes of 3:1 on both upstream and downstream faces; the central compacted earth fill has slopes of 2:1. A side-discharge type spillway structure, 94 feet in length, together with a 10-1/2 feet diameter spillway channel, which served as the diversion tunnel during the construction of the main embankment, are located in the left abutment.

An excellent set of surveillance facilities was installed at the dam site, consisting of seepage measurement devices, observation wells, deformation and settlement indicators, and seismoscopes. The phreatic surface in the embankment, as determined from the observation wells, is relatively high, as shown in Fig. 1. Although the quantity of flow is as high as 5 cfs, little dissolved or suspended solids were found in the observed seepage flow.

The geological profile of the dam along the crest shows that the dam is founded on a rhyolite tuff overlying volcanic ash, which is a part of the Bishop Tuff formation, and the depth of the tuff varies up to a maximum of 750 feet. The construction records show that the streambed alluvium material was removed under the main body of the dam prior to the construction of the embankment, and the dam rests on a firm tuff bedrock foundation.

In order to improve the watertightness of the foundation rock, a minimum

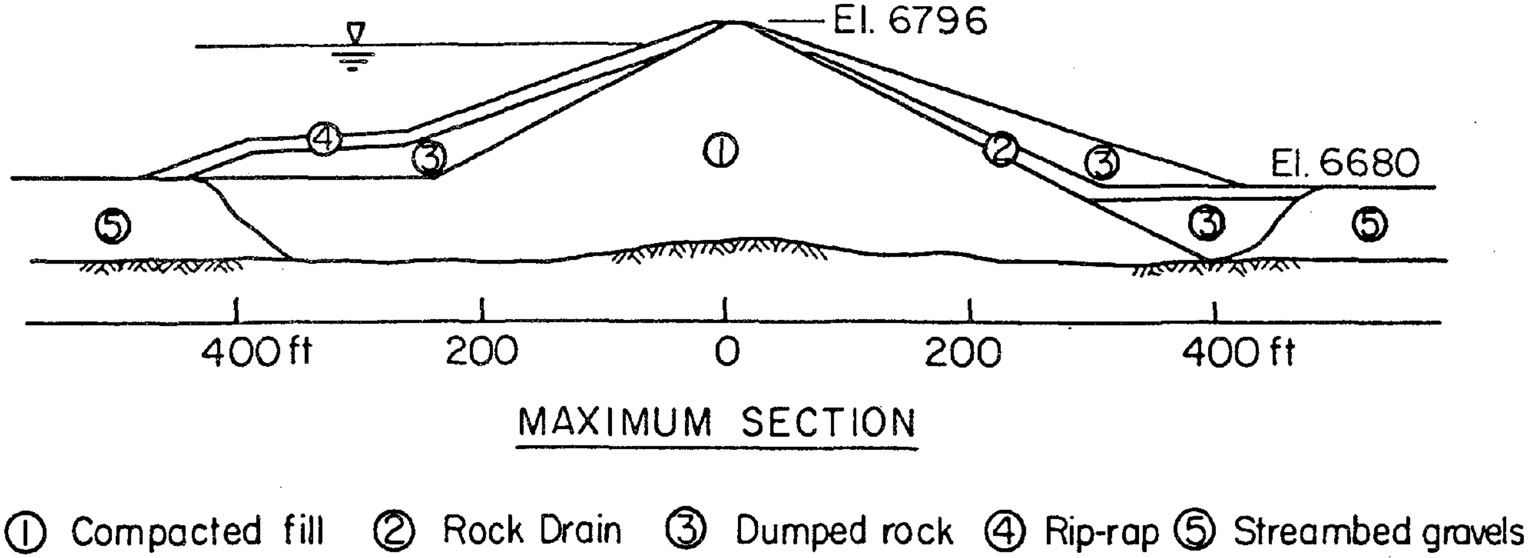


FIG. 2 CROSS-SECTION THROUGH LONG VALLEY DAM

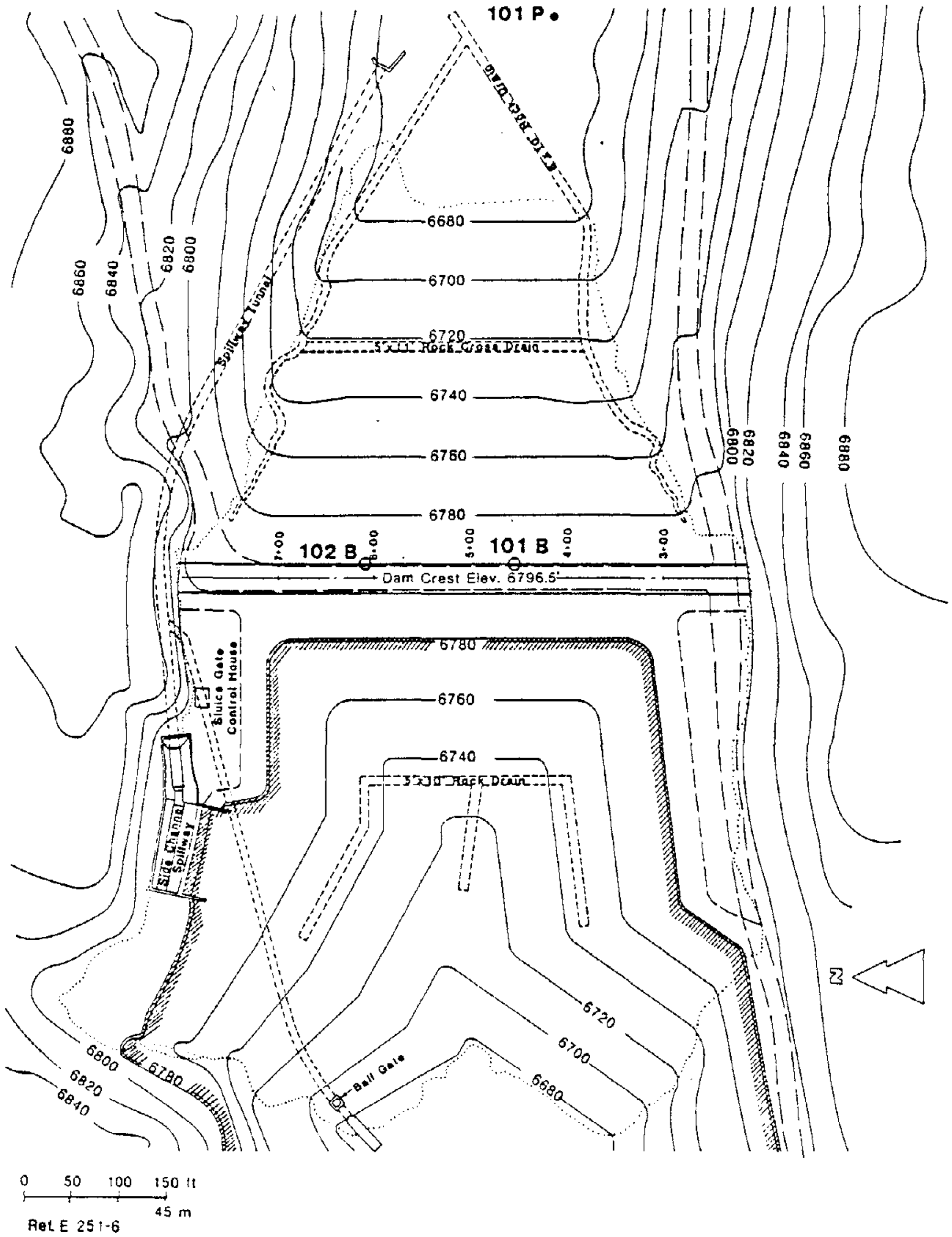


FIG. 3 PLAN VIEW OF LONG VALLEY DAM

of 2 inches of gunite was applied to the contact area between the compacted core and the abutments. In addition, blanket grouting, with grout holes at 5 ft spacing and 50 ft deep, was used in the upper parts of the abutment walls and some very deep grout holes were installed under the main body of the embankment.

The material used for the compacted earthfill was a well-graded silty and gravelly sand consisting of about 23% gravel sizes, 63% sand sizes, and 14% fines. The fill was placed in layers 6-in thick using a sheepsfoot roller, with a minimum of 16 passes. It was placed at a water content about 2% wet of optimum, and it was compacted to a degree of compaction of about 93% based on the Modified AASHO Compaction Test.

The pervious material in the outer shells of the embankment came from the excavated streambed gravels and/or stripped rock material from the abutments. The material was dumped in place in layers about 6 ft thick and then sluiced thoroughly. Following placement, the pervious section was ponded to saturate and densify the material.

Performance of Long Valley in the Mammoth Lakes Earthquake Series During the Period May 25 to 27, 1980

Long Valley Dam is situated in a very active seismic area. As can be seen from the generalized geological map (Fig. 4), the dam site is bounded by several major faults. The Sierra Frontal fault system (Sierra Front and Hilton Creek Faults), about five miles from the dam, is believed to be capable of generating a maximum credible earthquake of magnitude about 8.3, with a duration of about 60 seconds and peak horizontal accelerations of about 0.6g at the dam site. In addition, a maximum credible earthquake of magnitude 8.5 is

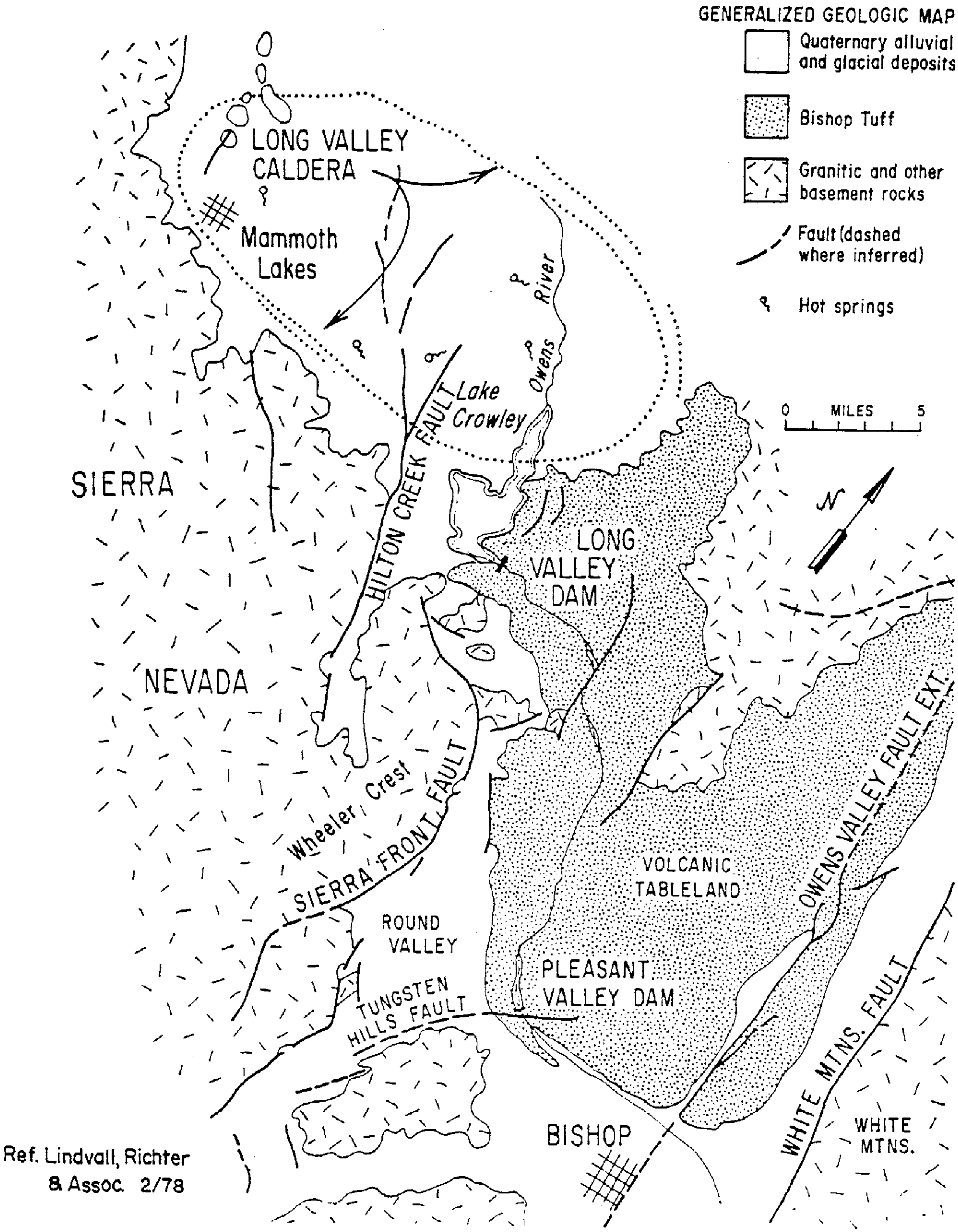


FIG. 4 GENERALIZED GEOLOGIC MAP - NORTHERN OWENS VALLEY REGION

also considered possible on the Owens Valley Fault, about 15 miles from the dam (Lindvall-Richter, 1980).

Because of the high seismicity of the area, Long Valley Dam was selected by the State of California Strong Motion Instrumentation Program for comprehensive instrumentation to investigate the dynamic response of dams to strong earthquake shaking. Accordingly 22 strong motion accelerographs were installed on the dam and in adjacent areas by this program. The distribution of these accelerographs is shown in Fig. 5. The 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.

An excellent record of the seismic response and performance of the dam has been provided through the acceleration data recorded during the Mammoth Lakes earthquake series during the period May 25 to 27, 1980. The earthquakes in this series were located on the Hilton Creek Fault in the Mammoth Lakes area, which passes within a few miles west of the dam site. The focal depths of the earthquakes ranged from 2 to 14 kms. The earthquake series had the following magnitudes:

May 25: M = 6.2 (09:34 a.m.); M = 5.9 (09:49 a.m.)
M = 6.3 (12:45 p.m.); M = 5.6 (13:36 p.m.)

May 26: M = 5.0; M = 5.3 (11:58 a.m.); M = 6.0

May 27: M = 6.2 (07:51 a.m.)

Following the earthquakes, rockfalls and landslides were observed and several surface cracks occurred in the embankment. Cracks were also found in the roadway between the contact of the dam and its north abutment, but no

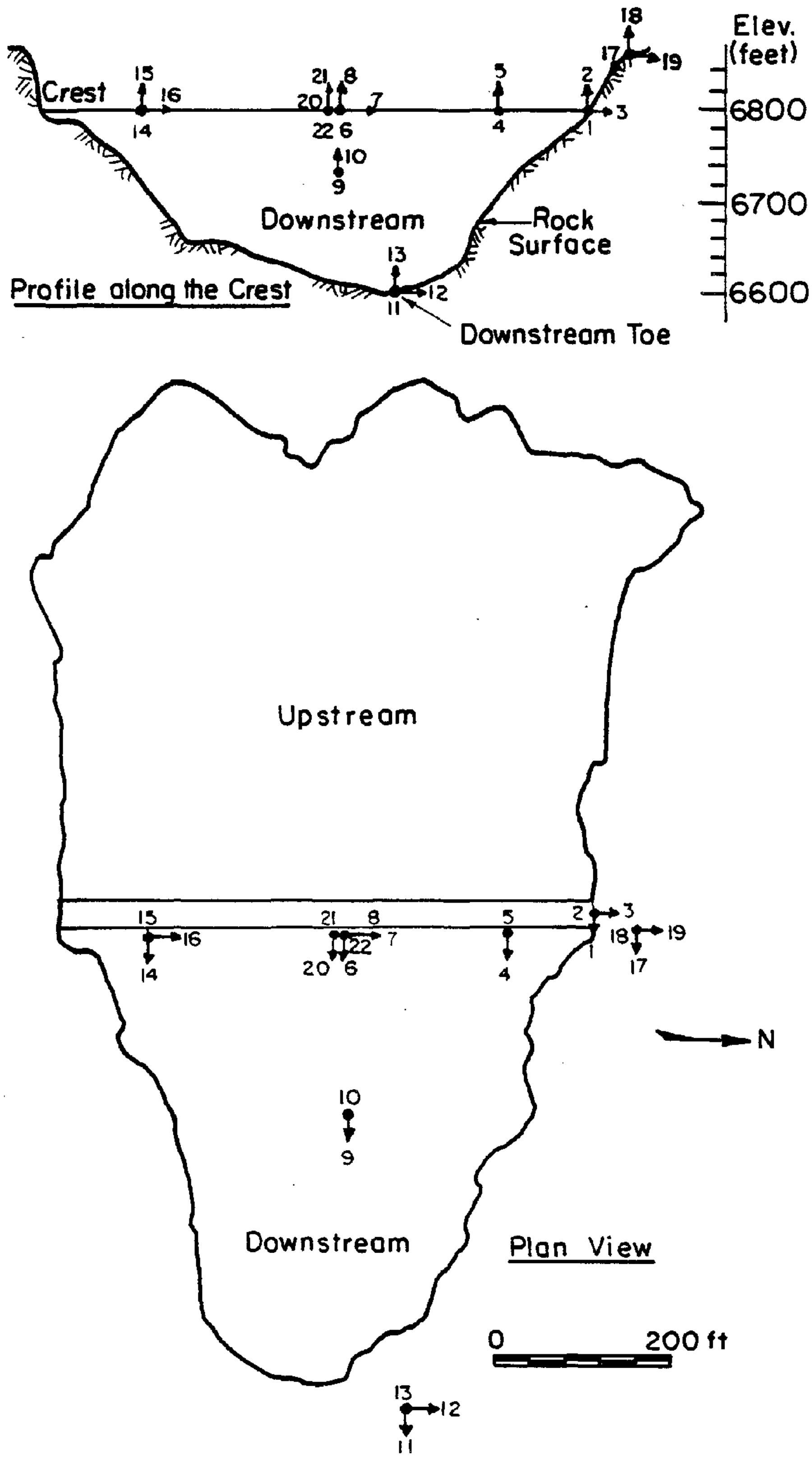


FIG. 5 DISTRIBUTION OF ACCELEROGRAPHS IN LONG VALLEY DAM

significant damage to the dam was detected because of the shallow depths of the cracks.

Although the dam suffered no significant damage during the earthquake series, following the earthquake of May 27 water was observed to flow out of the soil just downstream of the toe of the dam and continue to flow for several minutes after the earthquake shaking stopped. In this region, the materials were dumped and loosely compacted. It is interesting to note that even a moderate earthquake (magnitude = 6.2) with peak accelerations of about 0.2g in bedrock could cause the materials to decrease in volume and develop excess pore pressures sufficient to cause liquefaction and expulsion of water.

It is also interesting to note that following the earthquakes the flow in the toe drains increased from 200 to 460 gpm but the water remained clear; also in the spillway channel drain, where water flows into the tunnel through the weep holes and narrow cracks, the flow increased from 310 gpm to 790 gpm after the earthquakes. The flows of these drainage systems have become stable and they have remained at the higher level since the earthquakes occurred. The increased flow indicates some slight loosening of the structures of pervious materials and the abutments, but no significant settlement and movement were recorded after the earthquakes.

Characteristics of Recorded Accelerograms of the 1980 Mammoth Lakes Earthquake Series

Among the earthquake motions recorded at the dam site during the period May 25 to 27, 1980, five sets of recorded accelerograms, with up to 22 channels of recorded accelerations for each earthquake, have been processed. The recorded peak accelerations at the 22 instrument locations are shown in Table 1. In order to throw more light on the characteristics of the earthquake

Table 1 Recorded Peak Accelerations for 1980 Mammoth Lakes Earthquakes

Dam Crest

Location		Trace No.	5/25 09:34 PDT Acc. (G)	5/25 12:45 PDT Acc. (G)	5/25 13:36 PDT Acc. (G)	5/26 Acc. (G)	5/27 Acc. (G)
Sta. 3+32	(H)	14	0.15	----	----	0.10	0.48
Sta. 5+56	(H)	6	0.22	0.13	0.09	----	0.44
Armco Bldg	(H)	20	0.23	0.12	0.09	0.12	0.26
Sta. 7+13	(H)	4	0.14	0.11	0.08	----	0.27
Sta. 3+32	(L)	16	0.20	----	----	0.08	0.40
Sta. 5+56	(L)	7	0.23	0.20	0.16	----	0.31
Armco Bldg	(L)	22	0.23	0.21	0.16	0.09	0.29
Sta. 3+32	(V)	15	0.11	----	----	0.02	0.25
Sta. 5+56	(V)	8	0.15	0.13	0.11	----	0.15
Armco Bldg	(V)	21	0.15	0.13	0.11	0.03	0.19
Sta. 7+13	(V)	5	0.17	0.14	0.10	----	0.24

Downstream Slope

Location		Trace No.					
Mid. Slope	(H)	9	0.19	0.11	0.10	----	0.30
Mid. Slope	(V)	10	0.17	0.16	0.10	----	0.20

Bedrock

Location		Trace No.					
865'Dwn. Strm.	(H)	11	0.07	0.06	0.04	----	0.18
L. Abutment	(H)	1	0.08	0.08	0.06	----	0.20
L. Abutment	(H)	17	0.27	0.19	0.14	0.07	0.35
865'Dwn. Strm.	(L)	12	0.10	0.11	0.08	----	0.21
L. Abutment	(L)	3	0.12	0.08	0.10	----	0.21
L. Abutment	(L)	19	0.38	0.37	0.30	0.11	0.83
865'Dwn. Strm.	(V)	13	0.08	0.07	0.07	----	0.09
L. Abutment	(V)	2	0.10	0.07	0.05	----	0.11
L. Abutment	(V)	18	0.12	0.11	0.13	0.03	0.29

Note: H - Horizontal comp.; L - Longitudinal comp.; V - Vertical comp.
All records were instrument-corrected and bandpass-filtered.

motions recorded at the dam site for this earthquake series, the peak horizontal and longitudinal accelerations recorded in bedrock, corresponding to the stations at the toe, on the abutment at crest elevation, and at the station above the crest of the embankment are also tabulated in Table 2. It may be seen that in each earthquake the peak horizontal and longitudinal accelerations at the stations located on rock at the toe and at the crest elevation of the dam were generally similar but the peak accelerations recorded on rock at the station above the crest were much higher. Thus, a complicated wave propagation phenomenon in the upper rock formation at the dam site is clearly apparent.

As can be seen from Table 1, the earthquake of May 27 induced very high accelerations in the embankment. In addition, it is interesting to note that the earthquake of May 27 (magnitude = 6.2 and focal depth $\simeq$ 8.8 miles) produced much higher recorded peak accelerations in bedrock than those recorded during the earthquake of May 25, at 09:34 a.m. (magnitude = 6.2 and focal depth $\simeq$ 5.6 miles) although the two earthquakes had comparable magnitudes and epicentral distances. The motions recorded at the stations on rock during these two earthquakes are shown in Figs. 6(a) and 6(b). However the two earthquakes of May 25, (at 09:34 a.m. and at 12:45 p.m.) showed quite similar peak acceleration patterns for corresponding points on bedrock as shown in Figs. 7(a) and 7(b). It is clear from these records that earthquake motions in bedrock are determined not only by the earthquake magnitude and the distance of the energy source but also by other factors such as bedrock formations, local site characteristics, wave types and travel paths, etc.

The event on May 27 caused the most severe motions at the dam site and this event was considered especially significant because of the detailed instrumentation of the project and the extremely high peak longitudinal

Table 2 Peak Accelerations in Rock

Earthquakes	at the Toe of the Dam		Station on the Abutment at the Crest Elevation		Station on the Abutment above the Crest Elevation	
	Long.	Trans.	Long.	Trans.	Long.	Trans.
5/25/1980 (09:34 a.m.)	0.10 g	0.07 g	0.12 g	0.08 g	0.38 g	0.27 g
5/25/1980 (12:45 p.m.)	0.11 g	0.06 g	0.08 g	0.08 g	0.37 g	0.19 g
5/25/1980 (13:36 p.m.)	0.08 g	0.04 g	0.10 g	0.06 g	0.30 g	0.14 g
5/26/1980	----	----	----	----	0.11 g	0.07 g
5/27/1980	0.21 g	0.18 g	0.21 g	0.20 g	0.83 g	0.35 g

Note: Long. = Longitudinal Accelerations

Trans. = Transverse Accelerations

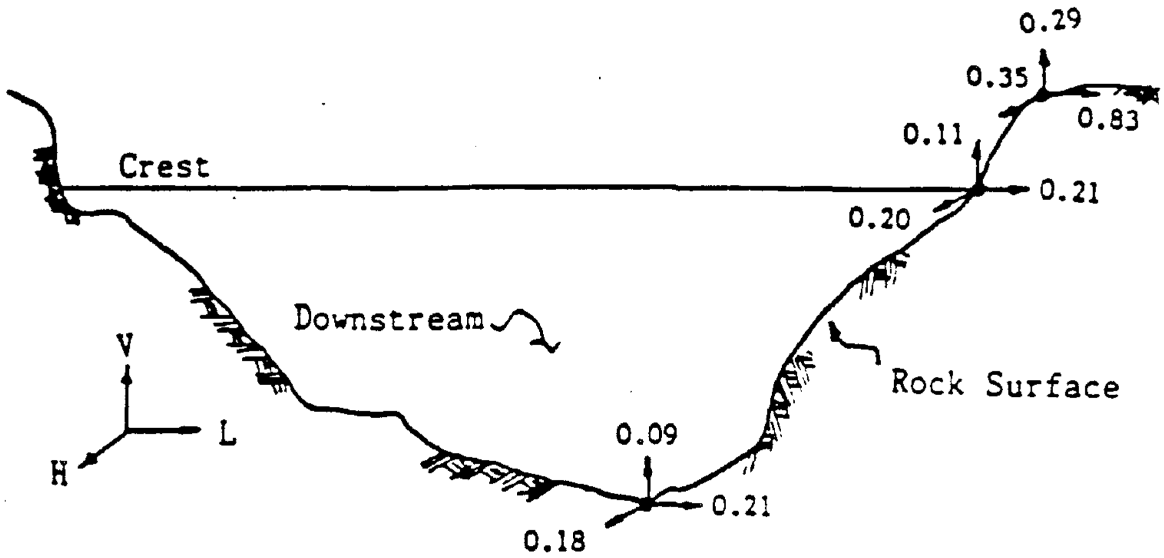


FIG. 6(a) PEAK ACCELERATIONS (G) IN ROCK FOR THE EARTHQUAKE OF 5-27-1980
(MAGNITUDE = 6.2, FOCAL DEPTH ≃ 8.8 MILES
EPICENTRAL DISTANCE = 5 MILES)

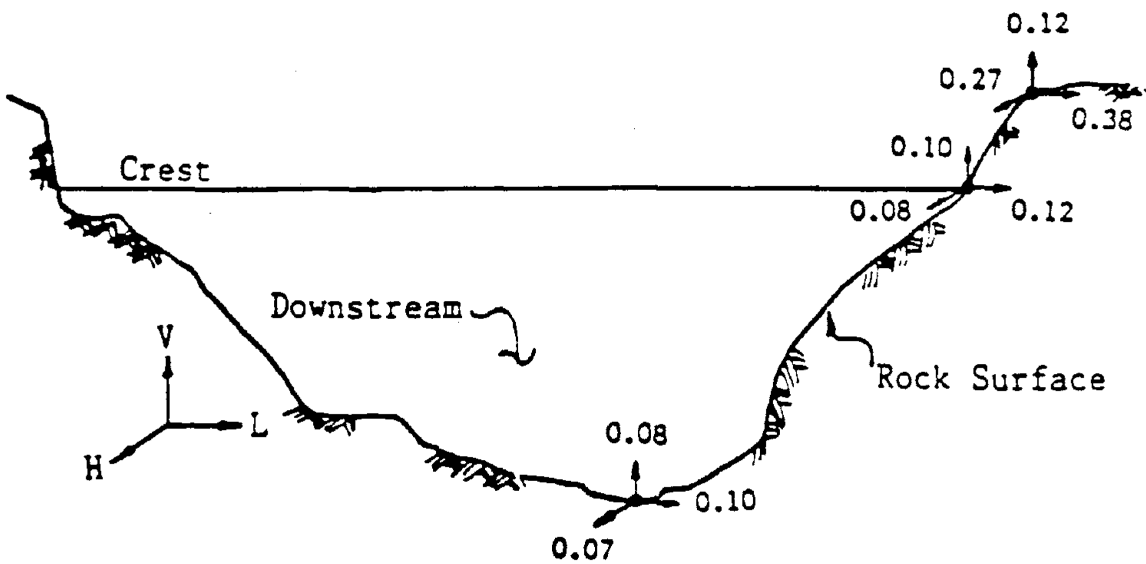


FIG. 6(b) PEAK ACCELERATIONS (G) IN ROCK FOR THE EARTHQUAKE OF 5-25-1980
AT 09:34 A.M.
(MAGNITUDE = 6.2, FOCAL DEPTH ≃ 5.6 MILES
EPICENTRAL DISTANCE = 5 MILES)

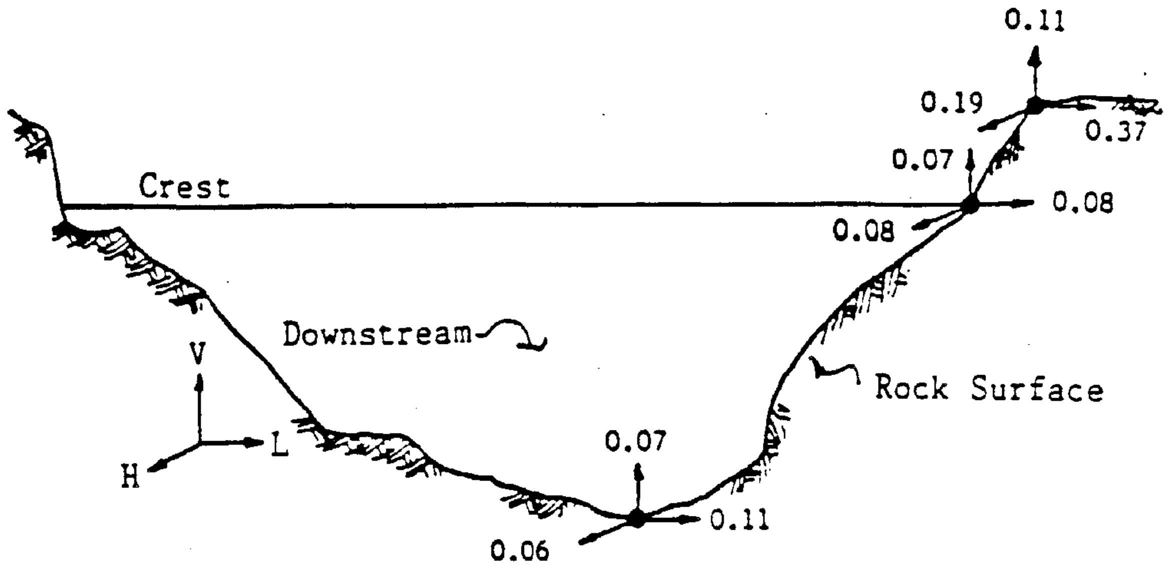


FIG. 7(a) PEAK ACCELERATIONS (G) IN ROCK FOR THE EARTHQUAKE OF 5-25-1980 AT 12:45 P.M.
(MAGNITUDE = 6.3, FOCAL DEPTH ≃ 10 MILES
EPICENTRAL DISTANCE ≃ 5 MILES)

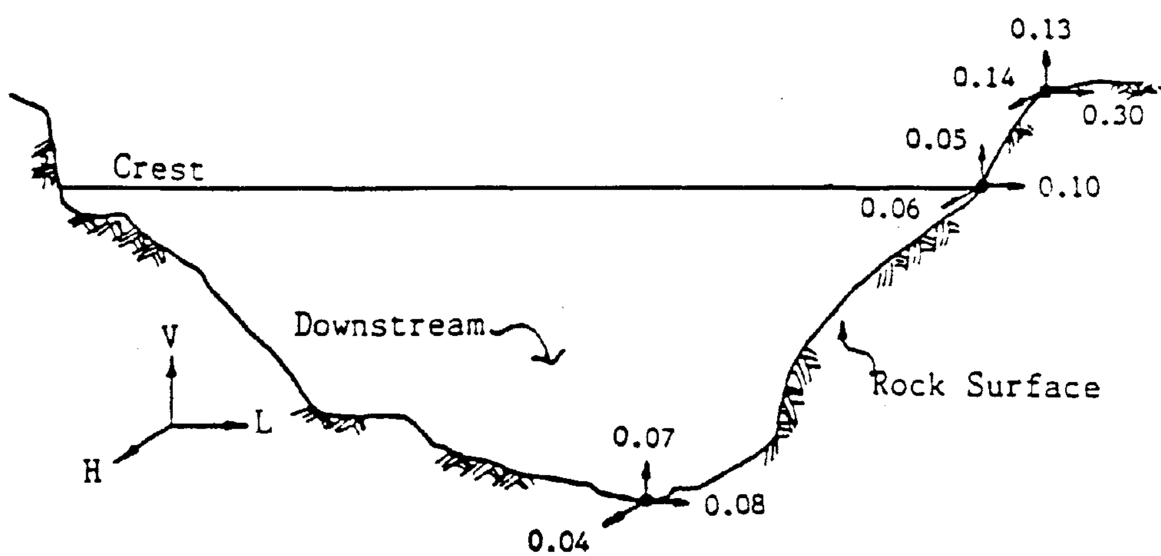


FIG. 7(b) PEAK ACCELERATIONS (G) IN ROCK FOR THE EARTHQUAKE OF 5-25-1980 AT 13:36 P.M.
(MAGNITUDE = 5.6, FOCAL DEPTH ≃ 1.3 MILES
EPICENTRAL DISTANCE ≃ 5 MILES)

acceleration of about 0.83g which was recorded on a rock outcrop above the left abutment (Fig. 6(a)).

Since only the transverse components of the recorded earthquake motions are generally considered in a response analysis, it is interesting to examine the distribution of the peak horizontal accelerations, normal to the axis of the dam, recorded at the dam site for all five earthquakes. Values of these accelerations are shown in Fig. 8. An examination of the peak horizontal accelerations recorded for channel 6, located near the center of the crest of the embankment, and channel 11, situated on bedrock downstream, shows amplification factors of about 2.2 to 3.1 for all the recorded earthquakes. It may also be noted that there is almost no difference between the recorded peak horizontal accelerations for channels 6 and 20 (both located near the center of the crest) for all five earthquakes.

An important assumption of the analytical procedures generally used to compute the seismic response of embankment dams is that the dam is constructed on a rigid base, and all points on the rigid boundary have the same motion and move in phase. To throw some light on the validity of this assumption a comparison was made of the time histories and acceleration response spectra (5% damping) recorded on rock at the toe of the embankment, on rock on the abutment at the crest elevation of the embankment and on a rock surface above the crest of the embankment, for the earthquake of May 27, 1980 (see Figs. 9 and 10). It is clear that the recorded motions of channel 1, located on bedrock near the downstream toe, have very similar characteristics to those of channel 11, located on the left abutment. On the other hand, the recorded motions of channel 17, located on a rock outcrop above the left abutment and not far away from the location of channel 1, were much higher, both in terms of peak acceleration and response spectral ordinates, than those of the motions

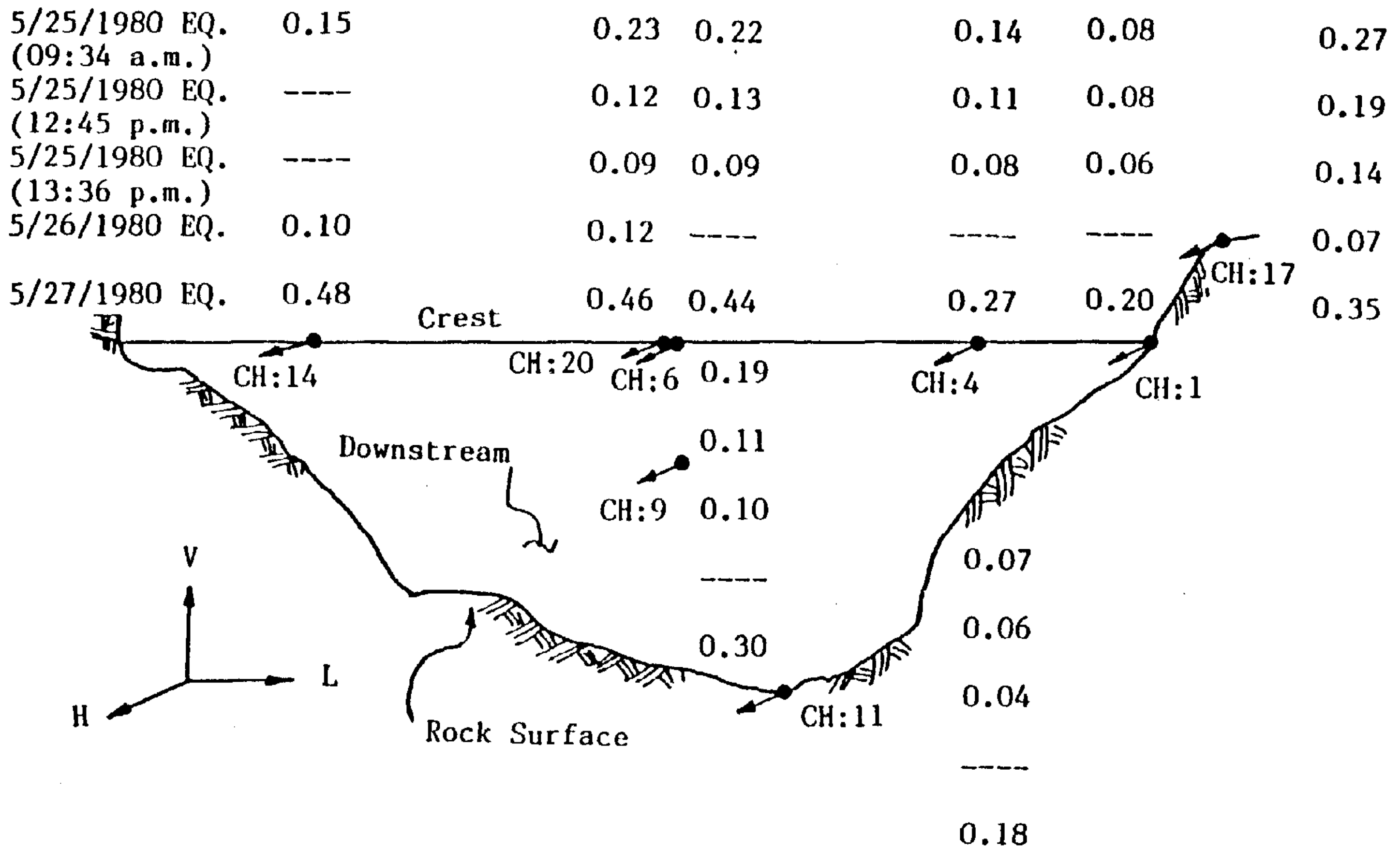


FIG. 8 PEAK ACCELERATIONS (G) NORMAL TO THE AXIS OF THE EMBANKMENT

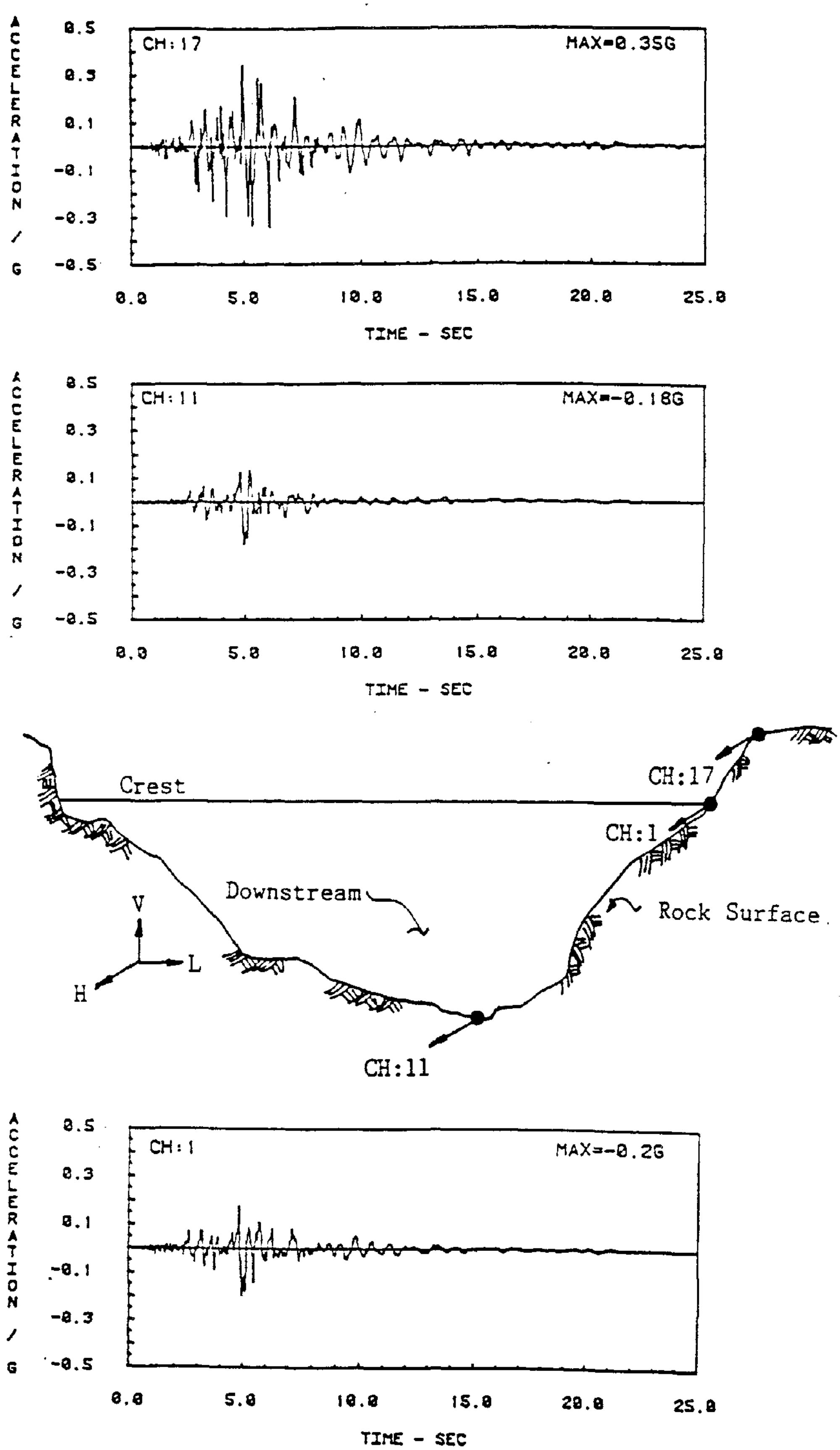


FIG. 9 RECORDED TIME HISTORIES (TRANSVERSE COMPONENTS) AT THE LEFT ABUTMENT TO THE EARTHQUAKE OF MAY 27, 1980

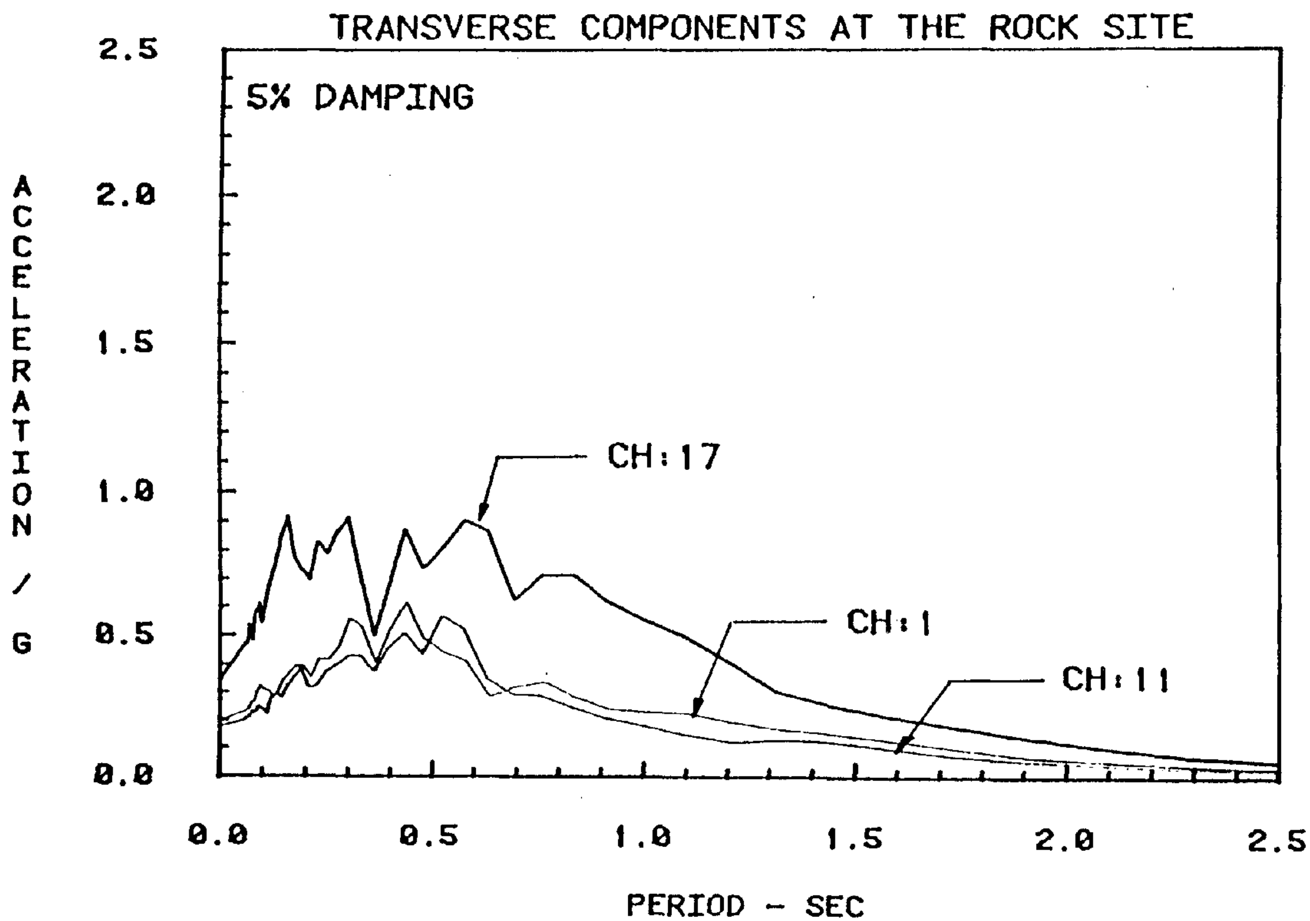


FIG. 10 RECORDED RESPONSE SPECTRA (TRANSVERSE COMPONENTS) AT THE LEFT ABUTMENT TO THE EARTHQUAKE OF MAY 27, 1980

recorded at channels 1 and 11. The same phenomenon was observed in a comparison of the recorded peak longitudinal accelerations for channels 3, 12 and 19 (Figs. 11 and 12) at the same locations for the same earthquake.

It would appear from these results and similar observations from the other earthquakes (Figs. 6(b), 7(a) and 7(b)) that the motions in rock at elevations below the crest of the embankment were generally similar but that important amplifications occurred in the rock formations at higher elevations, possibly due to topographic effects.

Finite Element Models

Since the coupling effects between the components of the recorded motions are not likely to be significant, only the transverse components of the recorded accelerations were considered in this study. Due to the very complex geometry existing in the upstream part of the embankment (concave configuration toward the bottom of the valley), it was considered desirable to simplify the model geometry to some extent in order to reduce the computational effort. Thus, a modified maximum cross section (Fig. 13), which leveled off the upstream face of the embankment and extended both the upstream and downstream faces all the way down to bedrock, was adopted in the analyses; it was believed that no significant errors would be introduced by this simplification.

Because the crest length to height ratio, L/H, is about 3:1, it was considered necessary to perform a 3-D dynamic analysis. For 3-D response analyses with input motions in the transverse direction, it is also convenient to choose a geometrical model which is symmetrical about the maximum section of the dam; this selection makes it possible to use only half of the complete embankment in the analyses. From previous experience it was considered desirable to use 12 sections across the valley to simulate the variation of

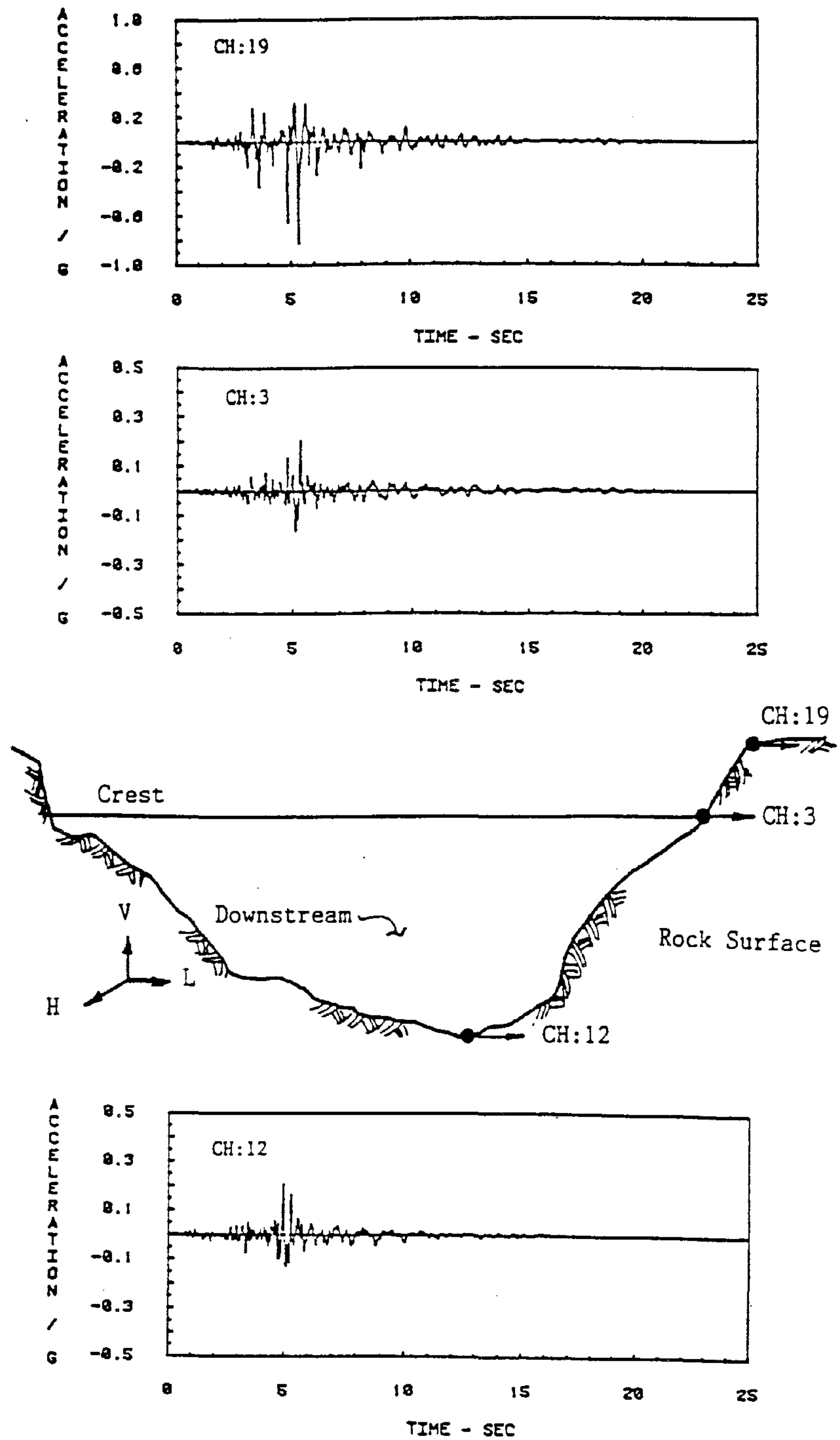


FIG. 11 RECORDED TIME HISTORIES (LONGITUDINAL COMPONENTS) AT THE LEFT ABUTMENT TO THE EARTHQUAKE OF MAY 27, 1980

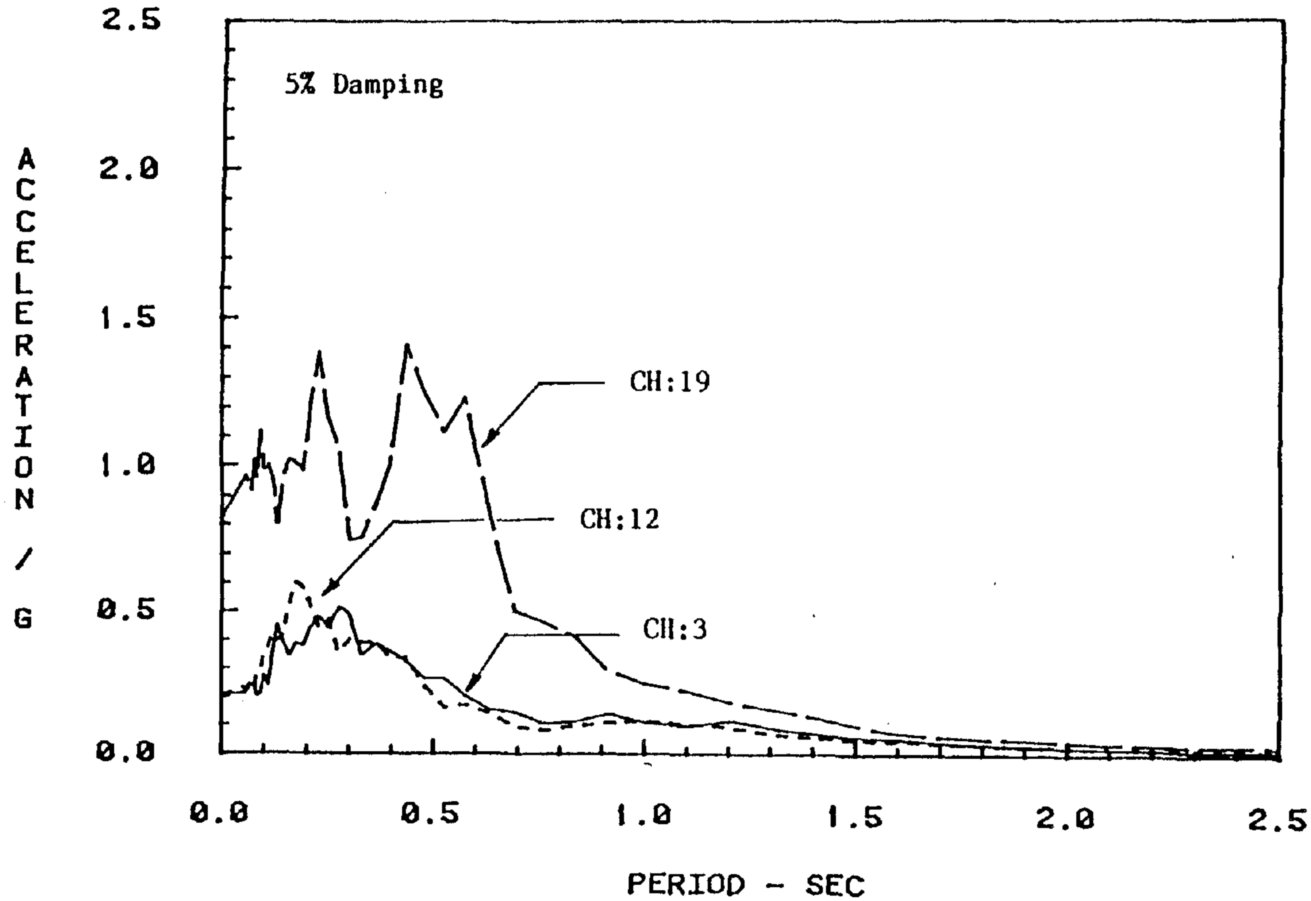


FIG. 12 RECORDED RESPONSE SPECTRA (LONGITUDINAL COMPONENTS) AT THE LEFT ABUTMENT TO THE EARTHQUAKE OF MAY 27, 1980

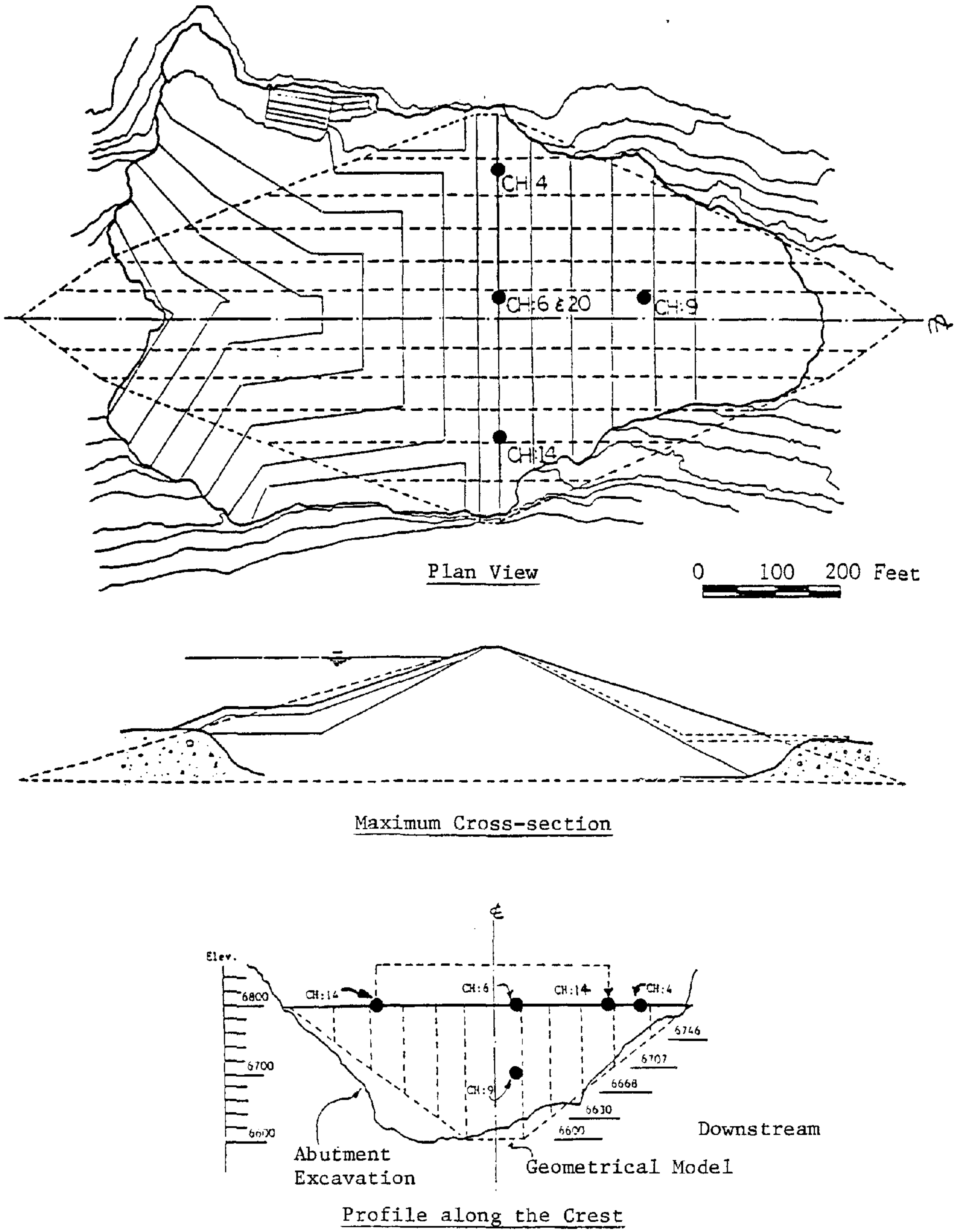


FIG. 13 GEOMETRICAL MODEL OF LONG VALLEY DAM

motions in the longitudinal direction. Based on these considerations, a 3-D finite element model, which represents only half of the embankment, was used in the dynamic response analyses. The model consists of 512 eight-node isoparametric solid elements, 582 nodal points (458 free nodal points with three translational degrees of freedom at each node), and 6 discretized sections across the valley (Fig. 14). Three materials were modeled in the analyses; compacted earthfill (gravelly sand with fines), shell material (coarse fines and small rock), and streambed alluvium (sand and gravel).

2-D response analyses were also performed. The finite element model for the 2-D analyses was chosen to be the same as that for the main section of the dam in the 3-D studies. Based on considerations with regard to mesh size requirements, a model with 129 solid elements and 142 nodes (117 free nodes) was constructed for use in the 2-D dynamic analyses (Fig. 15).

Initial Static Stress Analysis

A knowledge of the initial static stresses in the embankment is necessary to assess the dynamic shear moduli of granular materials such as sands, gravels, and rockfill for use in dynamic response analyses. To determine these stresses a 2-D plane strain analysis was carried out, using the computer program FEADAM, to determine the stress distribution throughout the main section of the dam, and then the stresses in the main section were projected horizontally to each cross-section along the dam axis. The static soil parameters used in the analysis are presented in Table 3. The seepage forces were considered in the analysis by using the computer program SEEP.

The contours of the computed effective major and minor principal stresses in the main section of the embankment are shown in Figs. 16(a) and 16(b). The contours are generally parallel to the slopes of the embankment except in the lower portion of the central region of the embankment; such distributions of

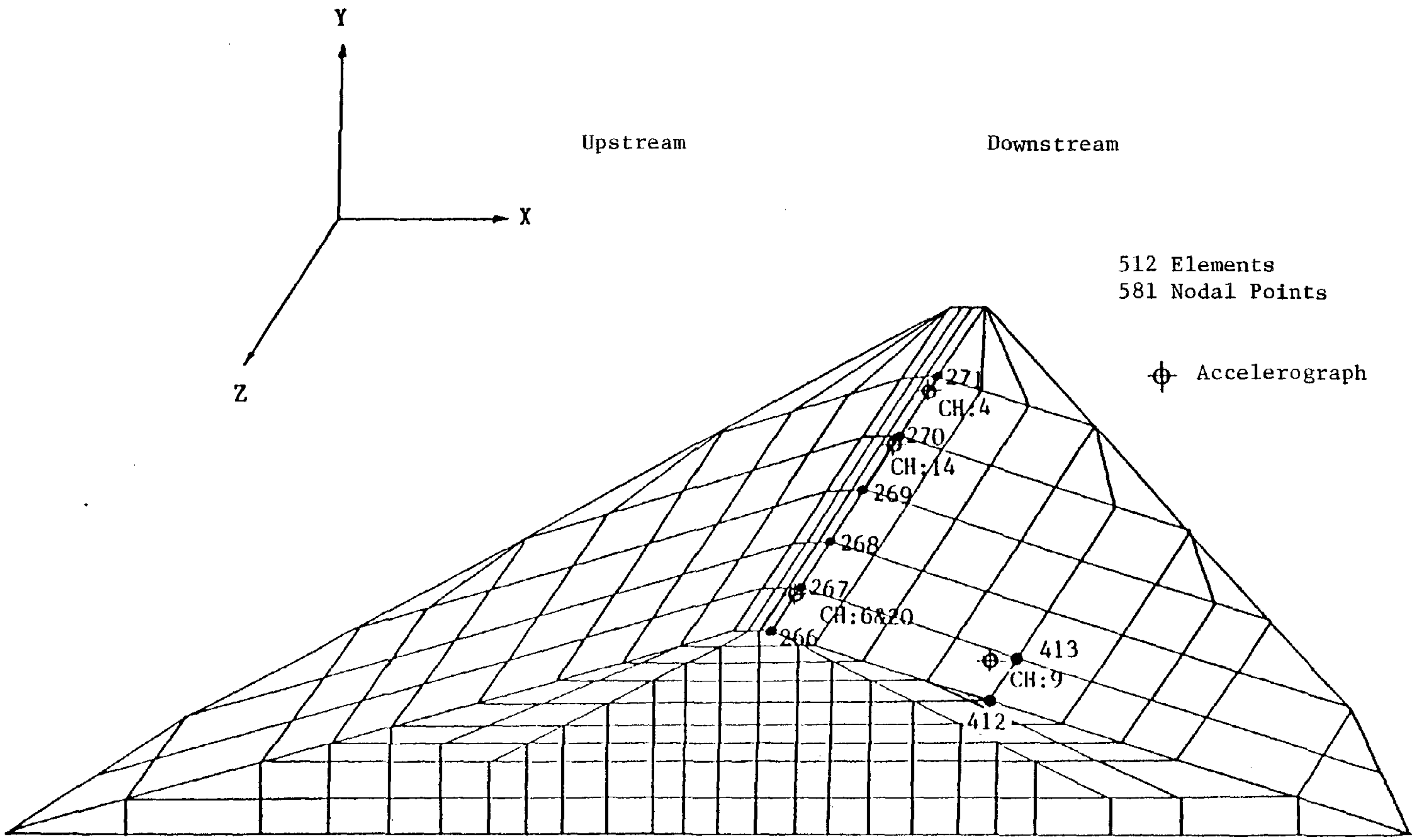


FIG. 14 THREE-DIMENSIONAL FINITE ELEMENT MODEL OF LONG VALLEY DAM

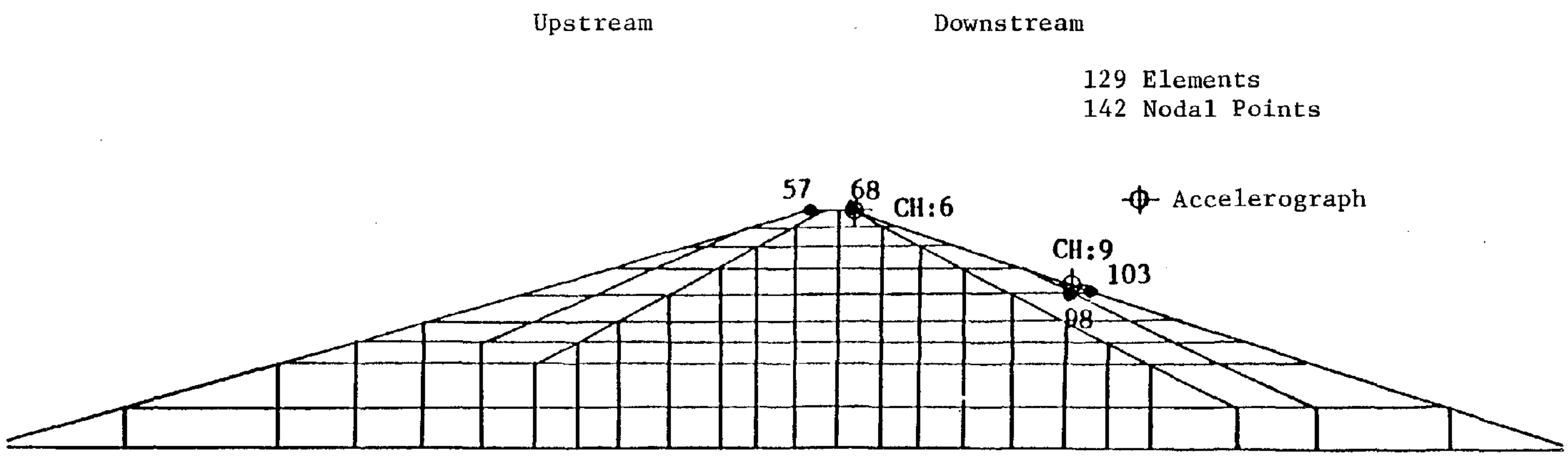


FIG. 15 TWO-DIMENSIONAL FINITE ELEMENT MODEL OF LONG VALLEY DAM

Table 3 Soil Parameters for Initial Static Stress Analysis (Long Valley Dam)

Parameter	Symbol	Compacted Fill (Dry)	Compacted Fill (Sat.)	Shell Material (Dry)	Shell Material (Sat.)	Streambed Gravels (Sat.)
Unit Weight (pcf)	r	130	144	124	140	147
Modulus Number	K	600	600	600	600	600
Elastic Unloading Modulus Number	K_{ur}	1200	1200	1200	1200	1200
Modulus Exponent	n	0.25	0.25	0.40	0.40	0.41
Failure Ratio	R_f	0.70	0.70	0.70	0.70	0.68
Bulk Modulus Number	K_b	450	450	175	175	170
Bulk Modulus Exponent	m	0.10	0.10	0.20	0.20	0.21
Friction Angle	ϕ_o	44	44	50	50	48
Decrease in Friction Angle	$\Delta\phi$	8.	8.	7.	7.	9.
Earth Pressure Coefficient	K_o	0.36	0.36	0.31	0.31	0.30

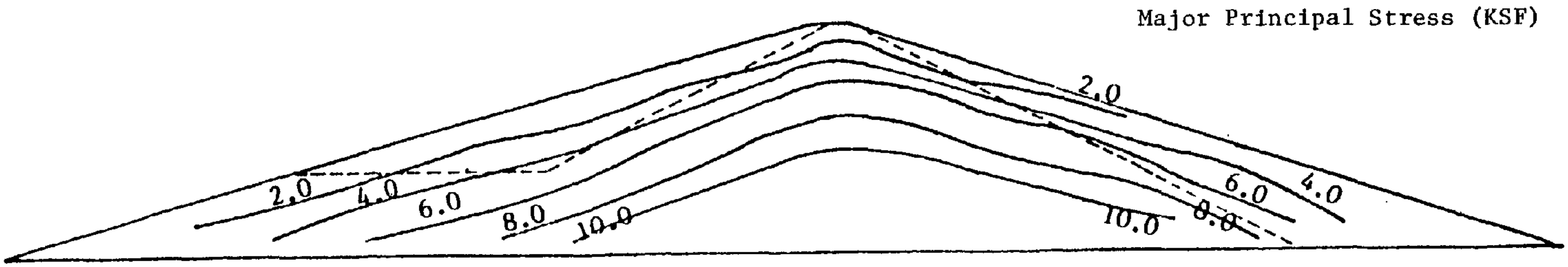


FIG. 16(a) STRESS CONTOUR OF EFFECTIVE MAJOR PRINCIPAL STRESS

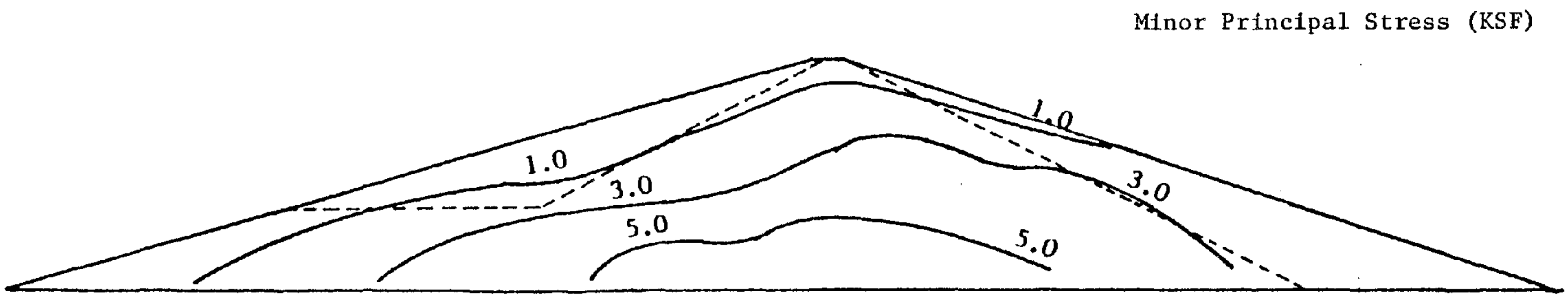


FIG. 16(b) STRESS CONTOUR OF EFFECTIVE MINOR PRINCIPAL STRESS

effective principal stresses are quite common in earth dams. It is convenient to adopt an average value of the principal stress ratio, σ_3'/σ_1' (ratio of effective minor principal stress to effective major principal stress), which is representative of the overall stress state within the embankment, in order to avoid possible numerical difficulties in discretizations and mathematical assumptions for the computational model. For Long Valley Dam a value of the stress ratio, σ_3'/σ_1', of about 0.42 was obtained by averaging the individual stress ratio in each element of the embankment.

Due to the lack of information on the intermediate principal stresses, σ_2', a value of $\sigma_2' = 0.6\sigma_1'$ was assigned to this stress. Thus, values of the effective mean principal stress, σ_m', were determined to be 0.68 σ_1'. The stresses throughout the embankment were obtained by projecting the stresses in the main section of the dam horizontally to the other sections across the valley.

Dynamic Response Analysis

Because only the transverse accelerations were considered in the dynamic analyses, and the recorded motions at channel 11, located on bedrock near the downstream toe, were very similar to those recorded at channel 1, located on rock on the left abutment, the recorded motions at channel 11 were used as the input motions for the dynamic response analyses. The equivalent-linear complex-response method was employed to compute the response. The computed motions in the analytical model were compared with the motions recorded at the stations corresponding to channels 9, 6 and 14.

For the main study, the motions for the earthquake of 5-27-1980 were used because this event caused the most severe response of the embankment. The recorded peak accelerations in the embankment and abutment walls for this

earthquake are shown in Fig. 17. Since 2-D dynamic response analyses can provide useful information about the dynamic response of an embankment prior to performing 3-D dynamic analyses, 2-D analyses were carried out in addition to the 3-D analyses.

Dynamic response analyses were made using the programs FLUSH (Lysmer et al., 1975) for the 2-D analyses and TLUSH (Kagawa et al., 1981) for the 3-D analyses. Representative dynamic material properties for the embankment soils were determined by determining the response which best matched the recorded motions by varying the material properties used in the computations. The dynamic soil properties used in the computations. The dynamic soil properties obtained from the first 2-D analysis were used to compute the response of the embankment to the earthquake of 5/25/1980, at 09:34 a.m. However, no further attempt was made to check the applicability of the dynamic soil properties obtained in the 3-D response analysis to other earthquakes because previous studies (Lai, 1985) had indicated that the 3-D analytical technique is quite capable of predicting the dynamic response of embankment dams with very complicated 3-D configurations provided good selections of the dynamic properties are made.

2-D Response Analysis for the Earthquake of May 27, 1980

Four accelerograms (channels 4, 6, 20, and 14) recorded at the crest of the dam together with one record (channel 9) on the downstream face were processed and used as the recorded motions to be compared with the computed results from the analytical procedures described above. The recorded accelerogram at channel 11, which is located on rock near the downstream toe of the embankment, was used as the input motion at the base of the analytical models.

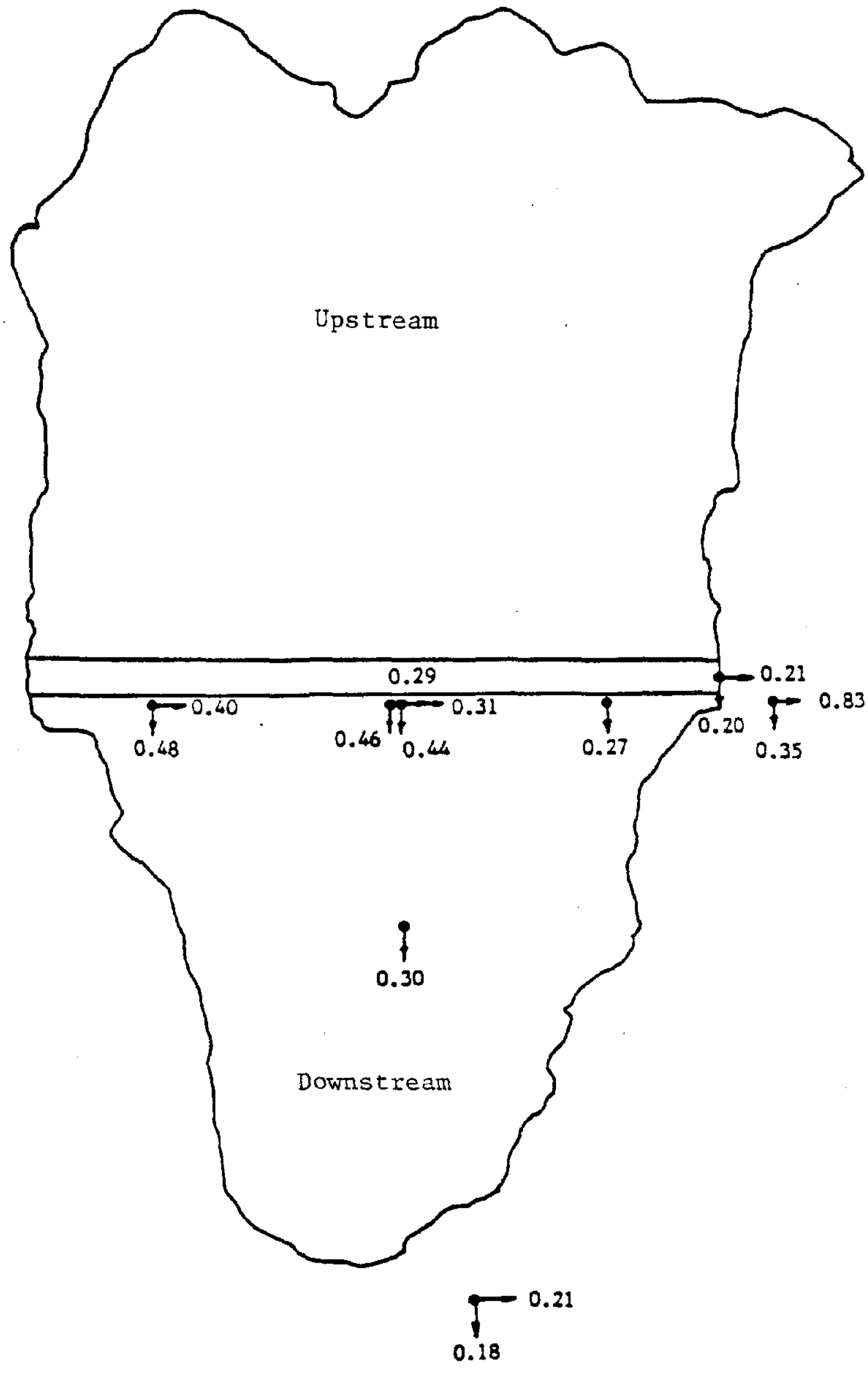


FIG. 17 PEAK ACCELERATIONS (G) RECORDED IN THE EMBANKMENT FOR THE EARTHQUAKE OF MAY 27, 1980

One way to check the quality of the recorded motions is to compare the response spectra for the motions recorded at channels 6 and 20, which are situated next to each other on the crest of the dam. The spectra for these two motions are shown in Fig. 18. It is apparent that there is no significant difference in these response spectra. Thus, it seems reasonable to conclude that the recorded accelerograms are quite consistent and the quality of the instrumentation at the dam site is reliable. As a result, only the accelerogram of channel 6 was used to represent the recorded motions at the center of the crest of the dam in the comparative studies.

Fig. 19 shows the recorded time histories in the main section of the embankment for channels 11, 9, and 6, corresponding to the stations on bedrock at the downstream toe, on the downstream face, and at the center of the crest respectively. The peak accelerations are 0.18g on the base rock, 0.3g at the mid point of the downstream face, and 0.44g at the crest. The records show that the peak accelerations of the recorded motions occurred at essentially the same absolute time on all records (about 5.0 seconds after the start of the recording). Futhermore the acceleration in bedrock was gradually amplified at the upper elevations of the embankment (e.g. at the downstream face and the crest of the dam). The corresponding 5% damped response spectra are shown in Fig. 20. As can be seen from the response spectra, there was little response for periods lower than about 0.15 second (i.e. 6.6 Hz). Based on this observation, it was concluded that the highest frequency used in the analyses could be about 10.0 Hz without losing any significant accuracy in the results.

It is also interesting to note that the response spectra for all three motions have very similar shapes. Unlike the case of El Infiernillo Dam, where the predominant frequencies of the base motions were found to be significantly lower than those of the crest motions, the predominant period of the base

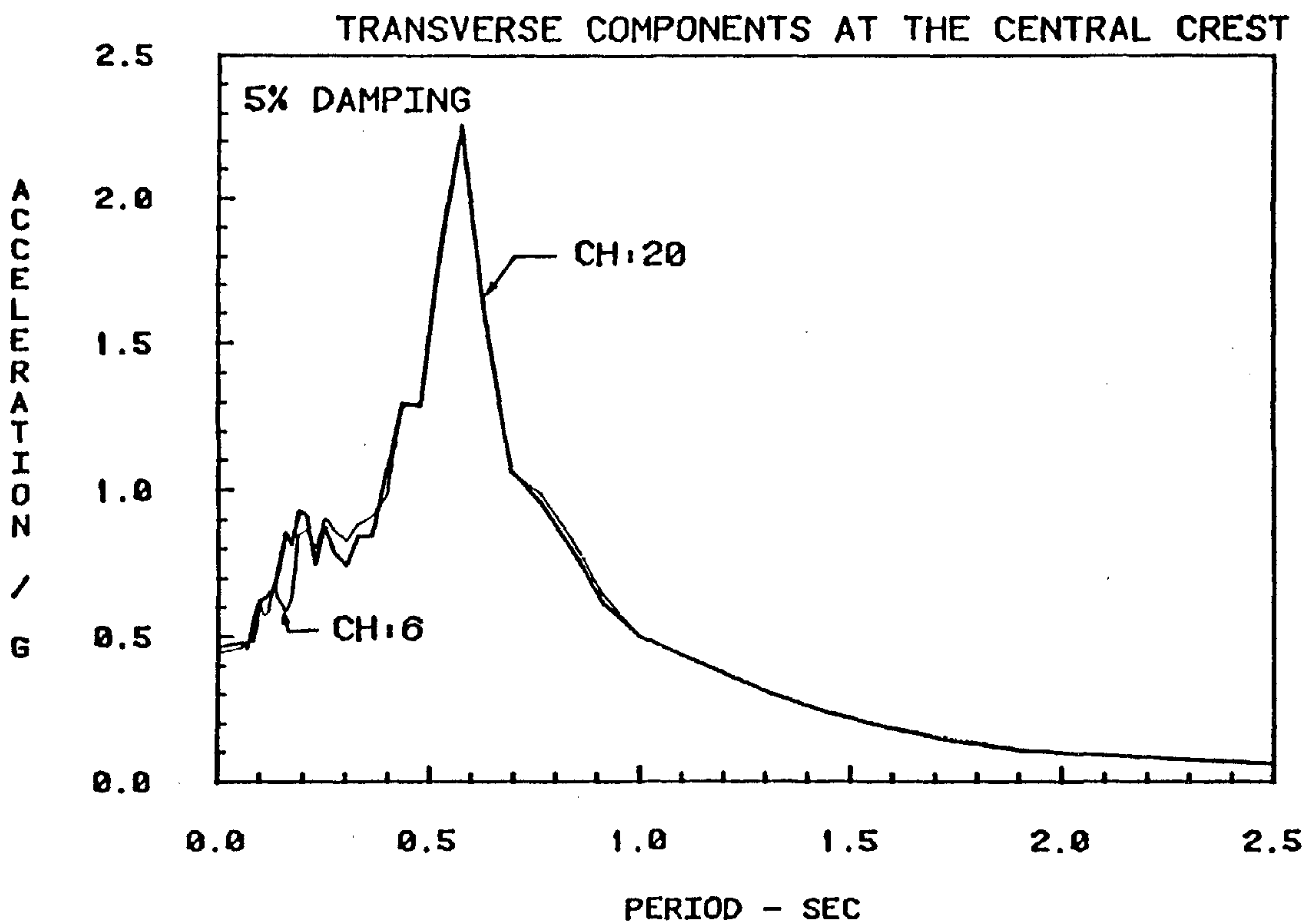


FIG. 18 RECORDED RESPONSE SPECTRA (MAY 27, 1980) AT THE CENTER OF THE CREST

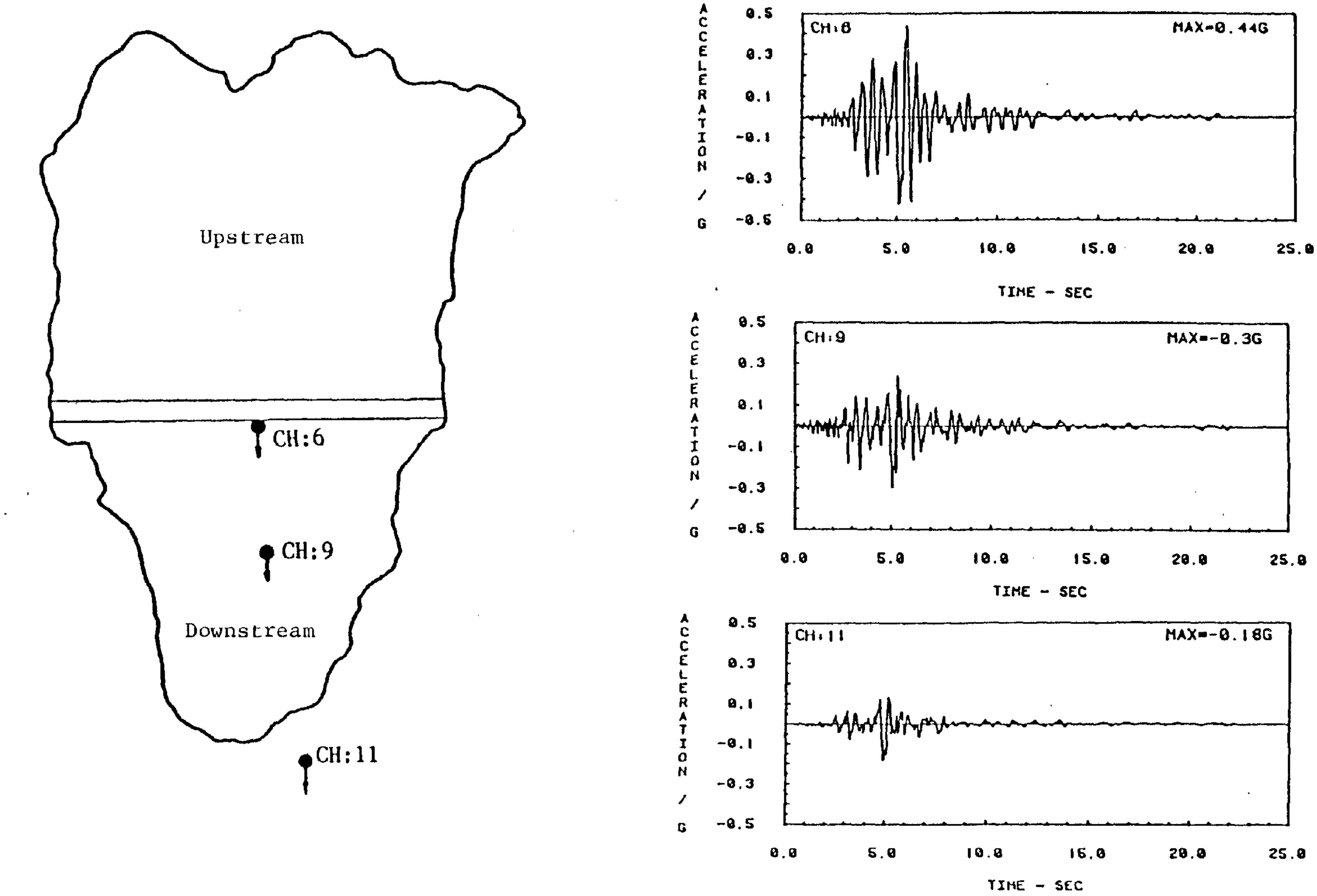


FIG. 19 RECORDED TIME HISTORIES (MAY 27, 1980 EARTHQUAKE) IN THE MAIN SECTION OF LONG VALLEY DAM

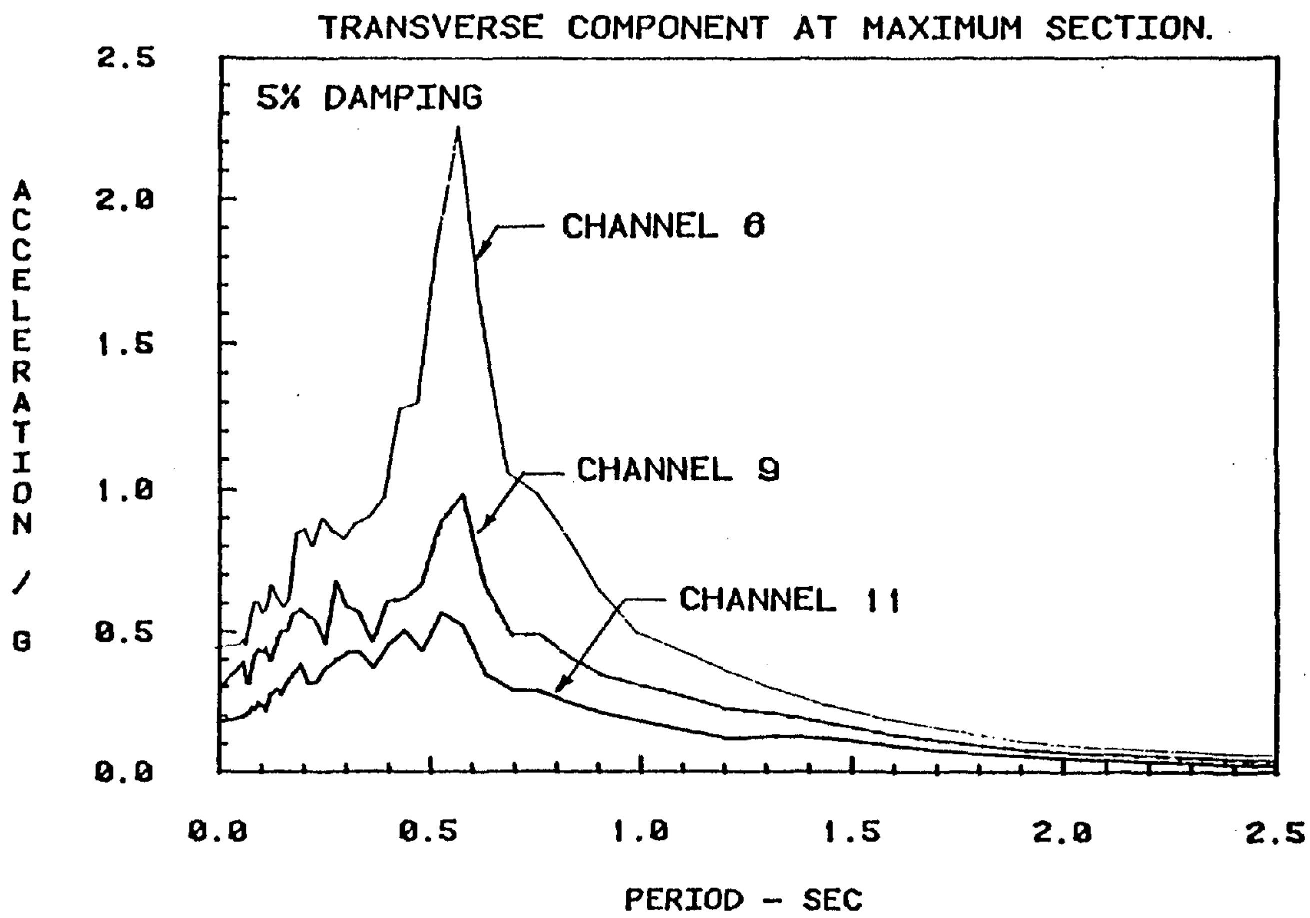
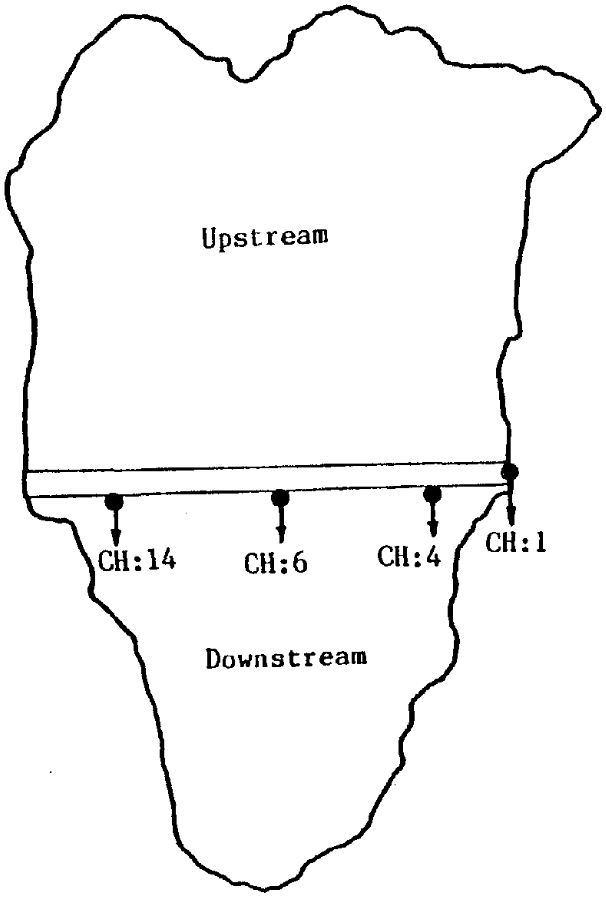


FIG. 20 RECORDED RESPONSE SPECTRA (MAY 27, 1980 EARTHQUAKE) IN THE MAIN SECTION OF LONG VALLEY DAM

motion for Long Valley Dam was about 0.51 second while the motions at the downstream face and on the crest both had a predominant period of about 0.58 second.

In order to provide a better understanding of the characteristics of the accelerograms recorded at the dam site, the recorded time histories of the motions for channels 1, 4, 6, and 14, corresponding to the stations along the crest of the dam are shown in Fig. 21. Again, the peak accelerations for all four channels occurred at approximately 5.0 seconds. The trend for the recorded accelerations to increase from the abutments towards the center of the dam can also be seen.

In the section on Finite Element Models it was noted that the gometrical model was chosen in such a way that the model is symmetrical about the main section of the embankment so that only half of the dam would need to be considered in the response analyses. Based on this assumption, the recorded motions at channels 4 and 14, which are located at similar locations in the symmetrical model (Fig. 13) should have comparable accelerations with respect to overall frequency content, predominant frequency, maximum acceleration, and peak spectral accelerations. In Fig. 21 the accelerograms recorded at channels 4 and 14, are seen to exhibit quite similar patterns of acceleration-time histories but the motion recorded at channel 14 had higher amplitudes of overall acceleration than that recorded at channel 4. This phenomenon is also clearly indicated by the response spectra for the recorded motions shown in Fig. 22. The two spectra are quite similar in shape and differ only in the magnitude of spectral accelerations. Although there is some difference in these two recorded accelerograms, the use of a symmetrical model in the dynamic analyses was still considered adequate for the present study. Therefore, the



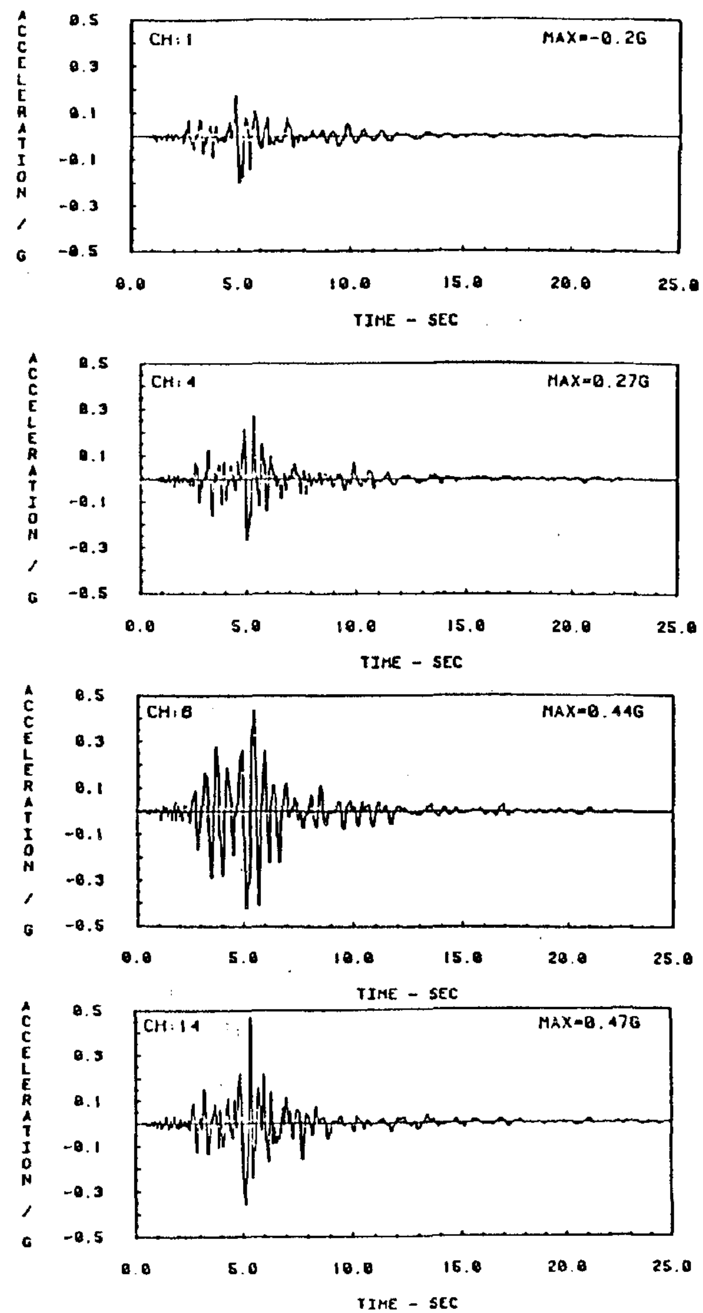
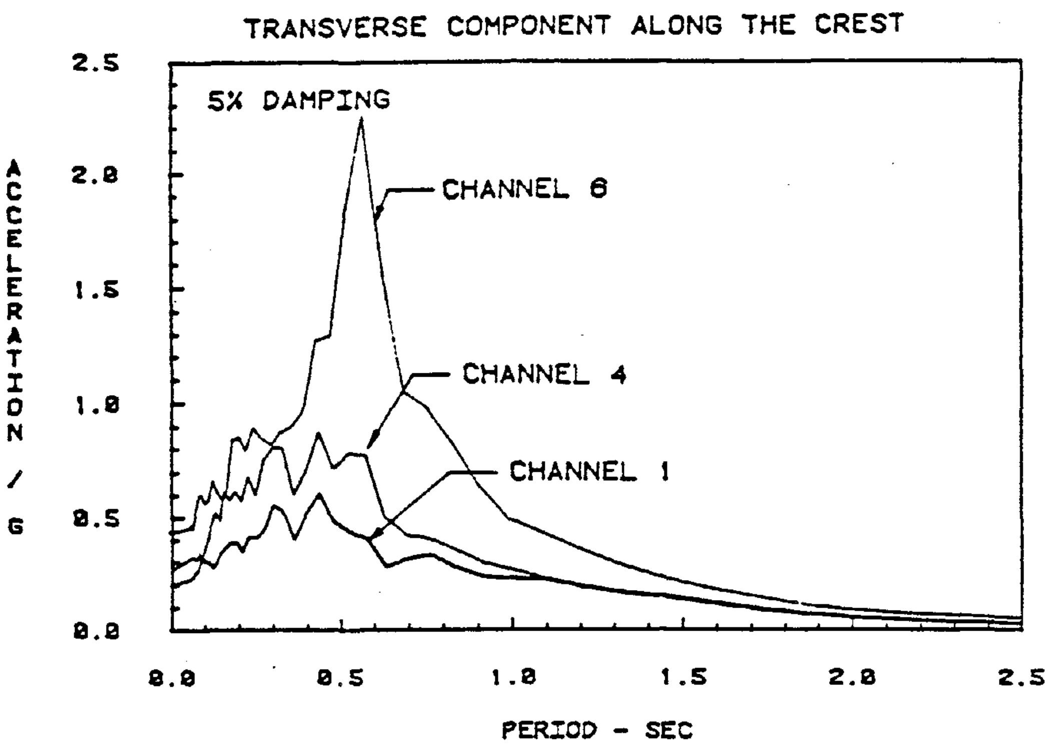


FIG. 21 RECORDED TIME HISTORIES (5-27-1980 EARTHQUAKE)
ALONG THE CREST OF LONG VALLEY DAM



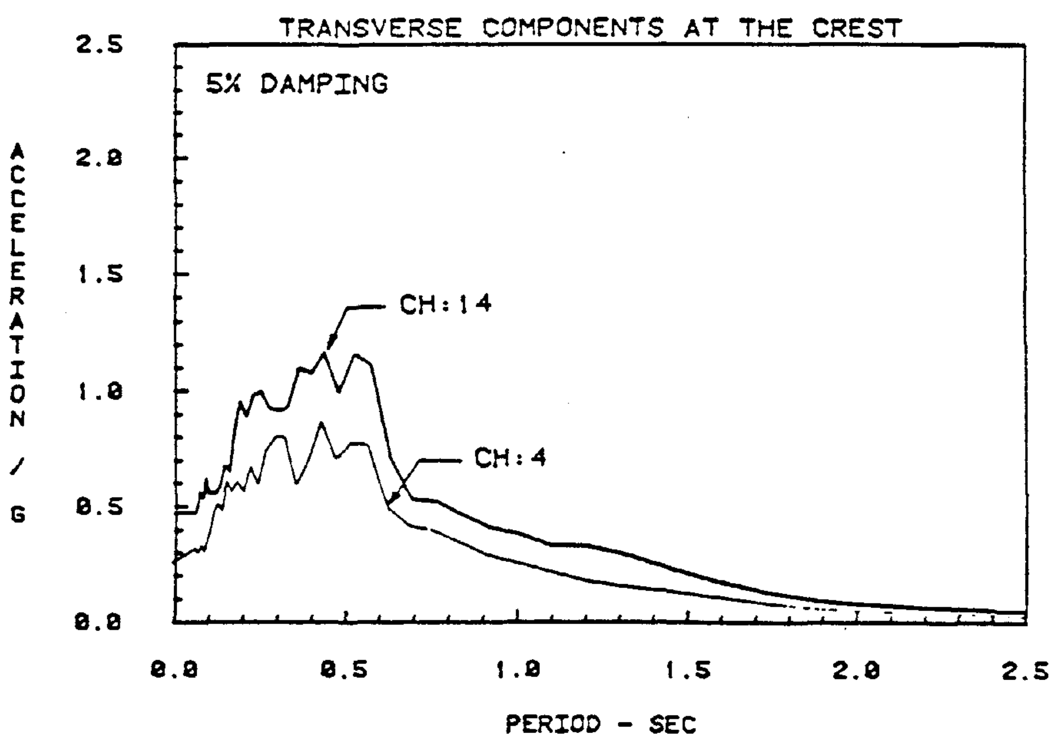


FIG. 22 RECORDED RESPONSE SPECTRA (MAY 27, 1980 EARTHQUAKE) ALONG THE CREST OF LONG VALLEY DAM

location of channel 14 was symmetrically projected to a position next to nodal point 270 in the computational model (Fig. 14).

Dynamic Soil Properties

The dynamic properties of the soils in the embankment were characterized in the present study by means of the shear modulus and damping values proposed by Seed et al. (1984). Thus the shear modulus, G, of the cohesionless soil at any point was determined by the expression:

$$G = 1000\ (K_2)\ (\sigma_m')^{1/2}$$

where σ_m' = the effective mean principle stress at the point

and K_2 = a soil modulus coefficient whose value varies with the strain level induced in the soil and the grain-size distribution of the soil involved; thus the maximum value of K_2 for any soil, designated (K_2)max, is developed at a low strain level of the order of 10^{-4}%.

For sands and many other cohesionless soils it has been found that the value of $K_2/(K_2)$max varies with strain as shown in Fig. 23 and this relationship is often referred to as the standard modulus attenuation curve for sands.

A corresponding average damping curve for cohesionless soils, expressing the damping ratio as a function of shear strain has also been proposed by the same authors and this curve has been widely used for dynamic response analyses of many earth structures and deposits. This relationship is shown in Fig. 24, together with upper bound and lower bound values.

The relationships shown in Figs. 23 and 24 were adopted for this study. By this means the dynamic stiffness of any given soil could be completely

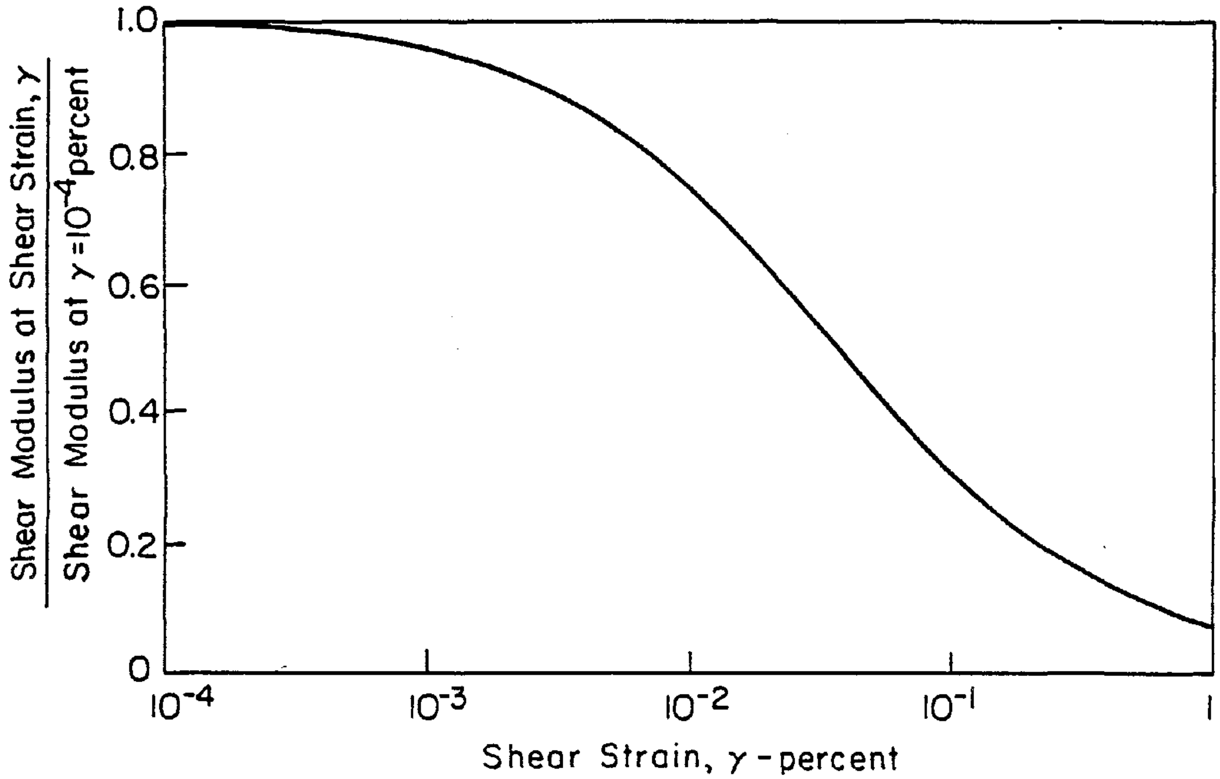


FIG. 23 MODULUS ATTENUATION CURVE FOR SANDS

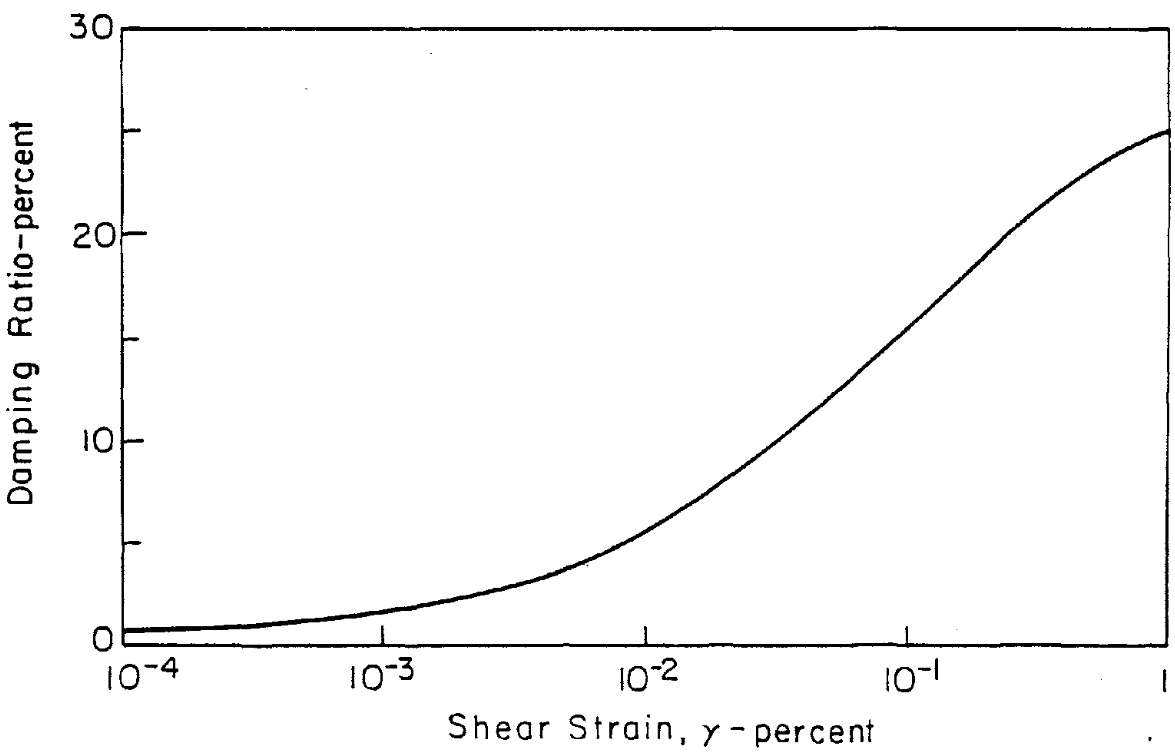


FIG. 24 DAMPING RATIO VERSUS SHEAR STRAIN RELATIONSHIP FOR SANDS (after Seed and Idriss, 1970)

characterized by its assigned value of (K_2)max. However in the dynamic analyses the shear modulus varied with strain and with confining pressure throughout the embankment in accordance with equation (1) above. Damping ratios also varied throughout the embankment depending on the induced strain, but relationships were varied to correspond either with the lower bound, average or upper bound relationships shown in Fig. 24.

Analytical Studies

In this investigation analyses were made to determine the values of (K_2)max for the compacted fill (gravelly sand with fines), dumped shell material (coarse sand and small rock) and streambed alluvium (sand and gravel) which gave best agreement between the computed and observed response of the embankment. The shear moduli for the embankment materials (mostly granular materials) were determined from Equation 1 using values of effective mean principal stress, σ_m', of about 0.68 σ_1' for each element.

For the 2-D plane strain dynamic analyses, it is only possible to compare the computed response with the recorded performance at channel stations 6 and 9. Thus, the computed responses at nodal points 57 and 68 in Fig. 15 were compared with the recorded accelerogram at channel 6, while the computed responses at nodal points 98 and 103 was compared with the recorded motions of channel 9. Several combinations of possible (K_2)max values for the embankment materials were tried to determine the best agreement between these motions.

The equivalent-linear computer program FLUSH was used to compute the response. A cut-off frequency of 10 Hz was imposed on the response computations. Table 4 shows a summary of the dynamic soil properties used in the response analyses providing best agreement between computed and recorded motions.

Table 4 Dynamic Properties for Response Analysis of Long Valley Dam

Parameter	Compacted Fill (Gravelly Sand)	Shell (Coarse Material)	Alluvium (Gravel & Sand)
(K2)max	60 (2D) 50 (3D)	90 (2D) 75 (3D)	120(2D) 100(3D)
Modulus Reduction* Curve	Mean Value	Mean Value	Mean Value
Damping** Curve	Mean Value	Mean Value	Mean Value
Poisson's Ratio	0.3 (Dry) 0.4 (Sat.)	0.3 (Dry) 0.4 (Sat.)	0.3 (Dry) 0.4 (Sat.)
Density (pcf)	130 (Dry) 144 (Sat.)	124 (Dry) 140 (Sat.)	147 (Sat.)

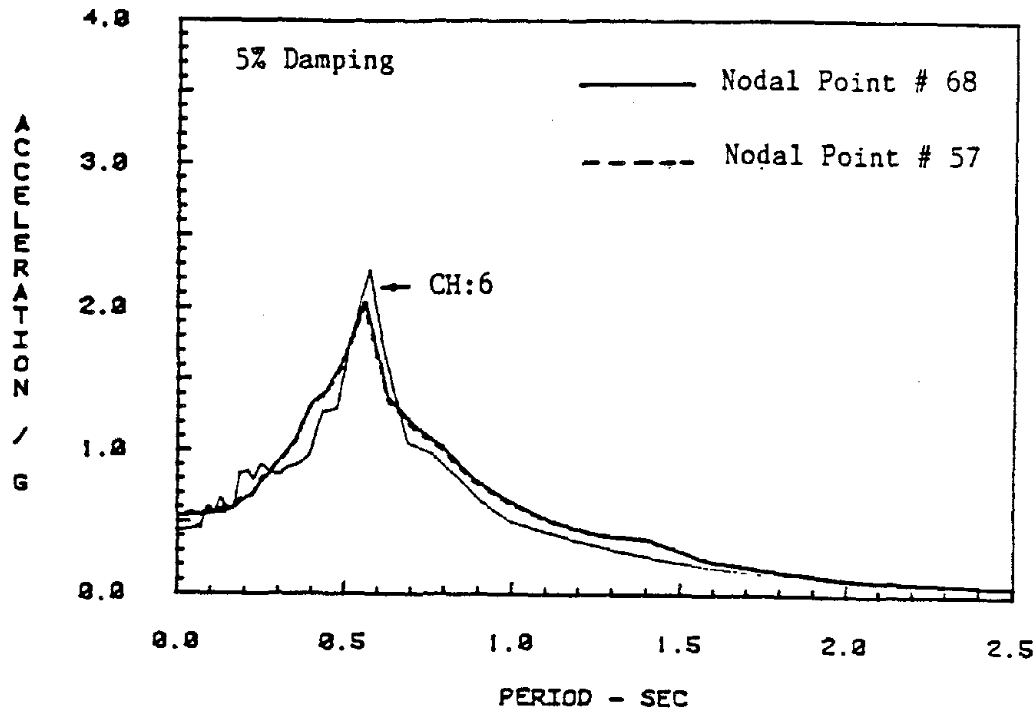
Note: A cut-off frequency of about 10 Hz was used in the analyses.

* : Seed and Idriss shear modulus reduction curve (1970) for 2-D and 3-D analyses; new shear modulus reduction curve for gravels (Seed Private Communication) for the 3-D reanalysis.

** : Seed and Idriss damping curve (1970).

The acceleration response spectra for the computed motions based on the set of dynamic properties shown in Table 4 with values of $(K_2)max$ of 60 for the compacted earthfill, 90 for the shell material, and 120 for the streambed alluvium, are compared with the spectra for the recorded motions in Fig. 25. Although the computed peak accelerations at nodal points 68 and 57 have a value of about 0.55g (25% higher than the value of 0.44g recorded at channel 6), the overall agreement between the spectra for the computed and recorded motions is excellent with respect to predominant period (about 0.56 second), peak spectral acceleration, and overall frequency content. It may also be noted that there was almost no difference between the computed motions at nodal points 68 and 57, 30 feet apart at the crest. The same excellent agreement between the recorded and observed motions was found in comparing the spectra for the computed motions at nodal points 98 and 103 with that for the motion recorded at channel 9 on the downstream face. Based on these comparisons, it may be concluded that the use of the dynamic properties indicated in Table 4 provides excellent agreement with the observed motions in the 2-D response analysis. Thus, values of $(K_2)max$ of 60 for the compacted fill (gravelly sand with sufficient fines), 90 for the dumped shell material (coarse sand and small rock), and 120 for the streambed alluvium (gravel and sand) obtained from the above back-calculation procedure are considered to be representative of the dynamic properties of Long Valley Dam embankment materials for 2-D response analyses. It may be noted that these values are in good accord with typical values for such materials, as summarized by Seed et al. (1984).

The distribution of computed peak accelerations in the main section of the embankment, as determined by the 2-D analysis is shown in Fig. 26. It can be seen that peak accelerations in the main section of the embankment increase from the base to the crest of the dam and that the computed peak accelerations



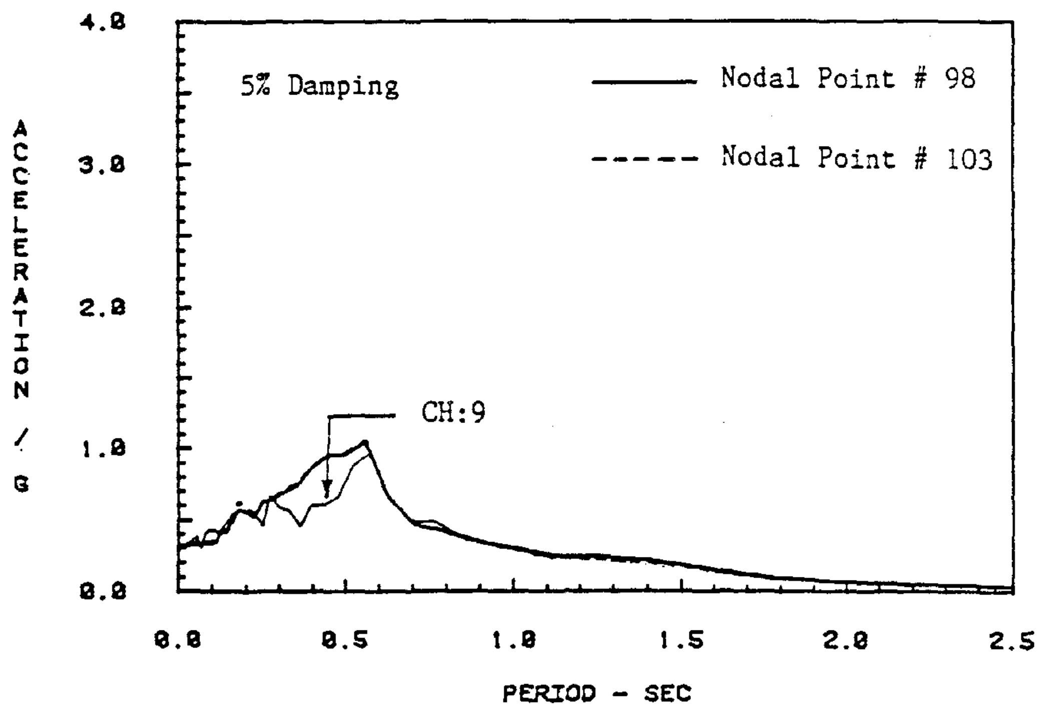
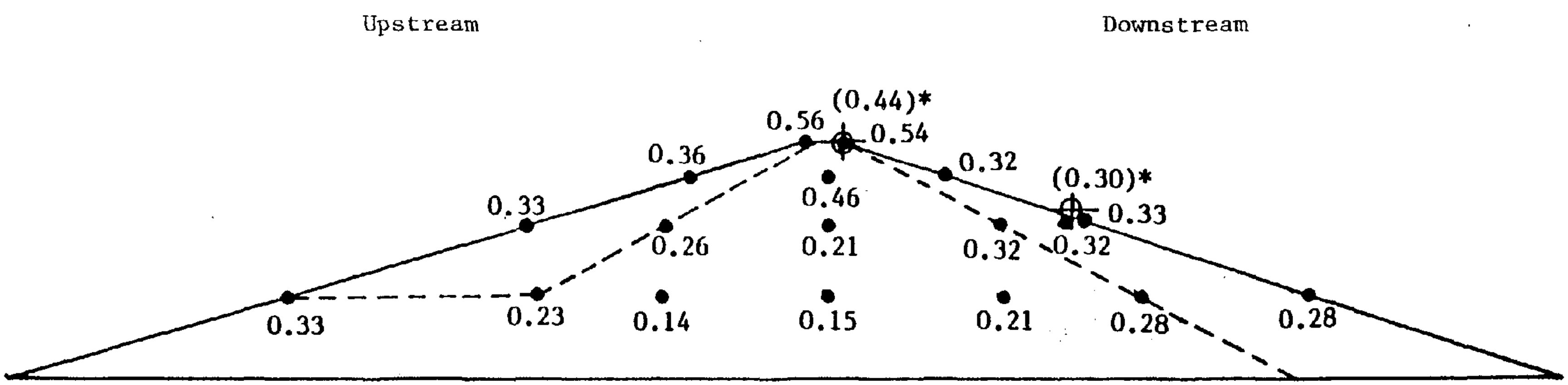


FIG. 25 COMPUTED 2-D RESPONSE WITH (K2)max (FILL) = 60
(May 27, 1980 EARTHQUAKE) (K2)max (SHELL) = 90
(K2)max (ALLUVIUM) = 120

(K2)max (Fill) = 60
(K2)max (Shell) = 90
(K2)max (Alluvium) = 120

*: Recorded Peak Acceleration
⊕ Accelerograph



(Maximum Base Acceleration = 0.18g)

FIG. 26 PEAK ACCELERATIONS IN MAXIMUM SECTION FROM 2-D ANALYSIS TO THE EARTHQUAKE OF MAY 27, 1980

for the upstream face are comparable to those for the downstream face of the dam. There is good agreement between the computed and recorded peak accelerations at the mid point of the downstream face of the dam.

3-D Response Analysis for the Earthquake of May 27, 1980

The detailed measurement of 22 channels of earthquake motions at the dam site and the significant 3-D configuration of the embankment, provide an excellent opportunity to study the applicability of the 3-D response analysis procedure to predict the distribution of motions in an embankment and to investigate the in-situ dynamic properties of the embankment materials in Long Valley Dam.

Based on previous studies, it would be expected that the true in-situ dynamic shear moduli which should be incorporated in a 3-D response analysis would be somewhat lower than those used in the 2-D response analysis, as a result of the stiffening effect resulting from the boundary constraint from the steep canyon walls. However, as a preliminary trial, the same set of dynamic properties for the embankment materials as those used for the 2-D response analyses was used to compute the response of the embankment using the 3-D response analysis program TLUSH.

The computed response spectra (5% damping) determined in this analysis are shown in Fig. 27. The response spectrum corresponding to nodal point 267 represents the computed response at the recording station for channel 6, which is located near the mid-point of the crest of the dam (Fig. 14). Similarly, the recorded motion at channel 9, located on the downstream face, is represented by the computed motion at nodal point 413. The computed response at nodal point 270 was compared with the recorded accelerogram of channel 14, which is symmetrically located just next to nodal point 270 in the model (Figs.

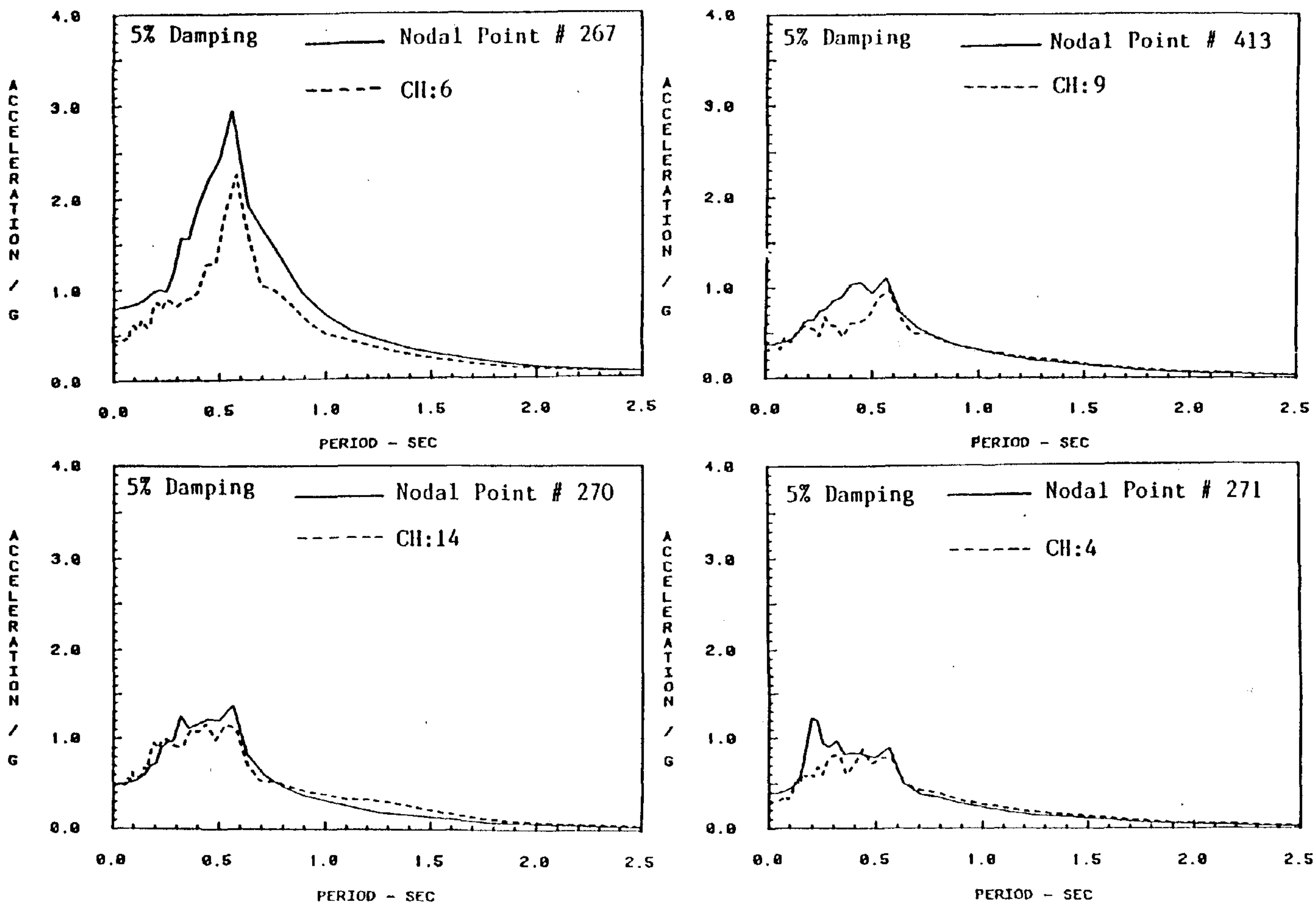


FIG. 27 COMPUTED 3-D RESPONSE SPECTRA WITH (K2)max (FILL) = 60
(K2)max (SHELL) = 90
(K2)max (ALLUVIUM) = 120

13 and 14). The recorded motion of channel 4 was compared with the computed motion at nodal point 271, which is close to the station of channel 4.

As can be seen from Fig. 27, there is generally good agreement between the spectra for the recorded and computed responses at channels 9, 14 and 4 with respect to values of peak accelerations, predominant periods, and overall frequency contents of the recorded motions. However the agreement between the computed and recorded motions is not so good for channel 6. The somewhat higher values of observed embankment response indicated by these computations can be attributed to the fact that the selected dynamic properties ($(K_2)max$ = 60 for compacted earthfill, 90 for dumped shell material, and 120 for streambed alluvium) are generally higher than would be expected to give good results in 3-D dynamic response analyses.

Accordingly, a set of slightly lower values of $(K_2)max$ was adopted and the 3-D analysis was repeated. The response spectra for the computed motions in this analysis are presented in Fig. 28, where they are again compared with the spectra for the recorded motions. It may be seen that the use of these lower values of dynamic moduli led to a significant improvement in the degree of agreement between the computed and observed motions at channel 6. However no significant improvement was achieved for the computed response of channel 9, located on the downstream face of the dam. It is interesting to note that the computed motions for nodal point 270 located near channel 14, had almost the same peak accelerations as the recorded value. In addition, the computed peak acceleration for nodal point 271 near channel 4 showed closer agreement with the recorded motion than before. Generally speaking, the overall agreement between the computed and recorded motions is quite good, although some discrepancies still exist.

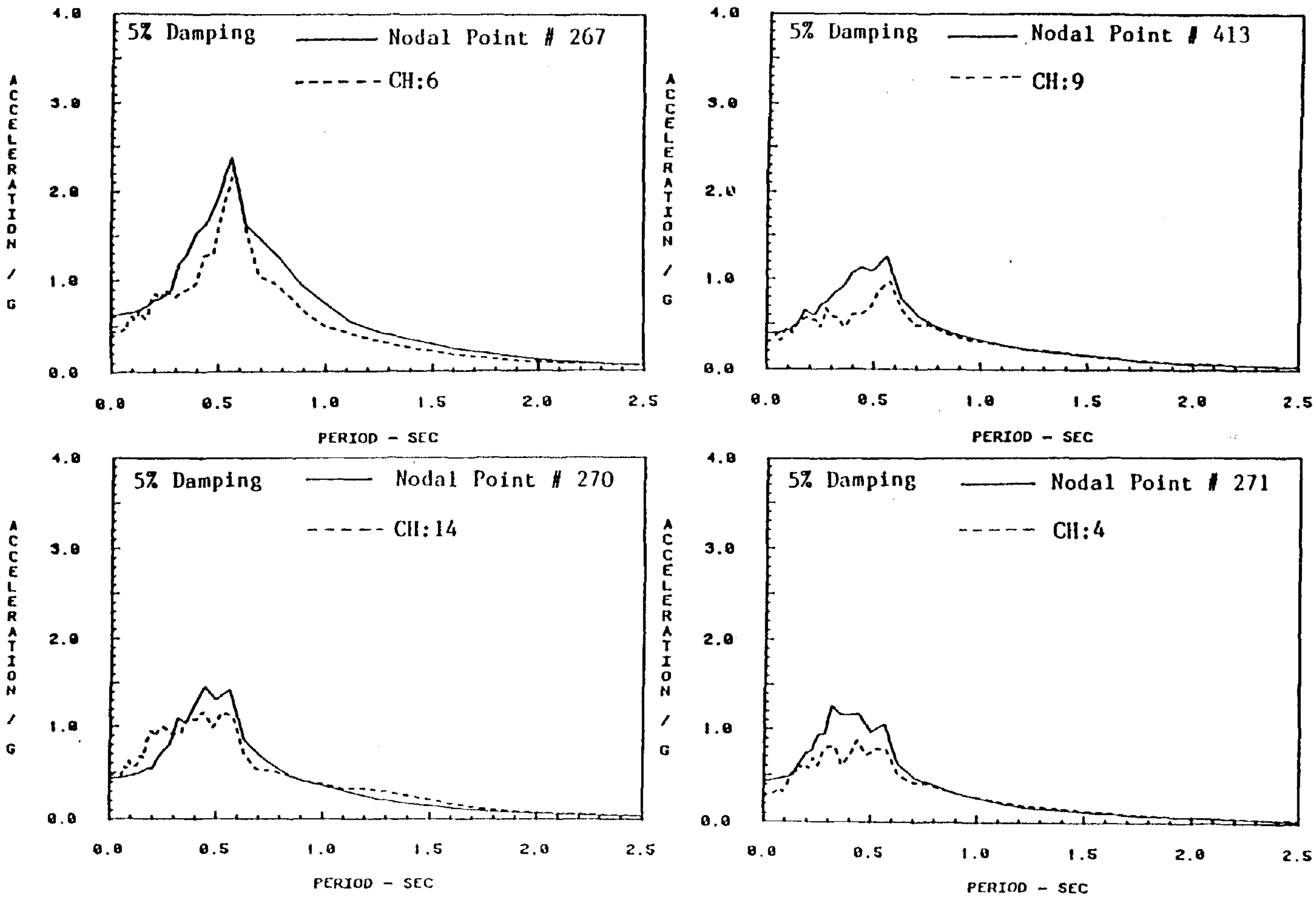


FIG. 28 COMPUTED 3-D RESPONSE SPECTRA WITH (K2)max (FILL) = 50
(K2)max (SHELL) = 75
(K2)max (ALLUVIUM) = 100

It may be noted that a close examination of the computed acceleration amplification functions (Fig. 29) in this analysis shows a very clear picture of the distribution of natural frequencies of vibration of the embankment to this earthquake shaking. For the three nodal points along the crest, the first natural frequency of the embankment in this earthquake is about 1.12 Hz; it may also be seen that the higher modes of vibration of the dam are less important for points near the center of the embankment whereas they contribute significantly for points close to the abutments.

Since the embankment consists primarily of the compacted gravelly sand, the computed response is only slightly affected by the properties of the shell material and the foundation soils used in the dynamic response analyses. Thus the critical properties are those of the compacted gravelly sand at the induced strain levels; for the motions developed by the earthquake of May 27, the induced shear strains were about 1×10^{-1} percent on the average. The corresponding values of the modulus stiffness coefficient K_2 for the 3-D analysis with $(K_2)max = 50$ for this material are shown in Fig. 30(a). Also shown in this figure are the values of the modulus stiffness coefficient for this soil determined by cyclic loading triaxial compression tests in the LADWP laboratories during the seismic evaluation studies before the earthquake occurred. It may be seen that the laboratory-determined values of modulus coefficient are in excellent agreement with those deduced from the embankment response, indicating that meaningful values of soil moduli for use in dynamic analyses can be determined from laboratory tests.

A similar comparison for values of damping for the gravelly sand is shown in Fig. 30(b). It may be seen that in this case the agreement between laboratory-determined values of damping ratio and those determined from observations of response is not so good, with the laboratory values being

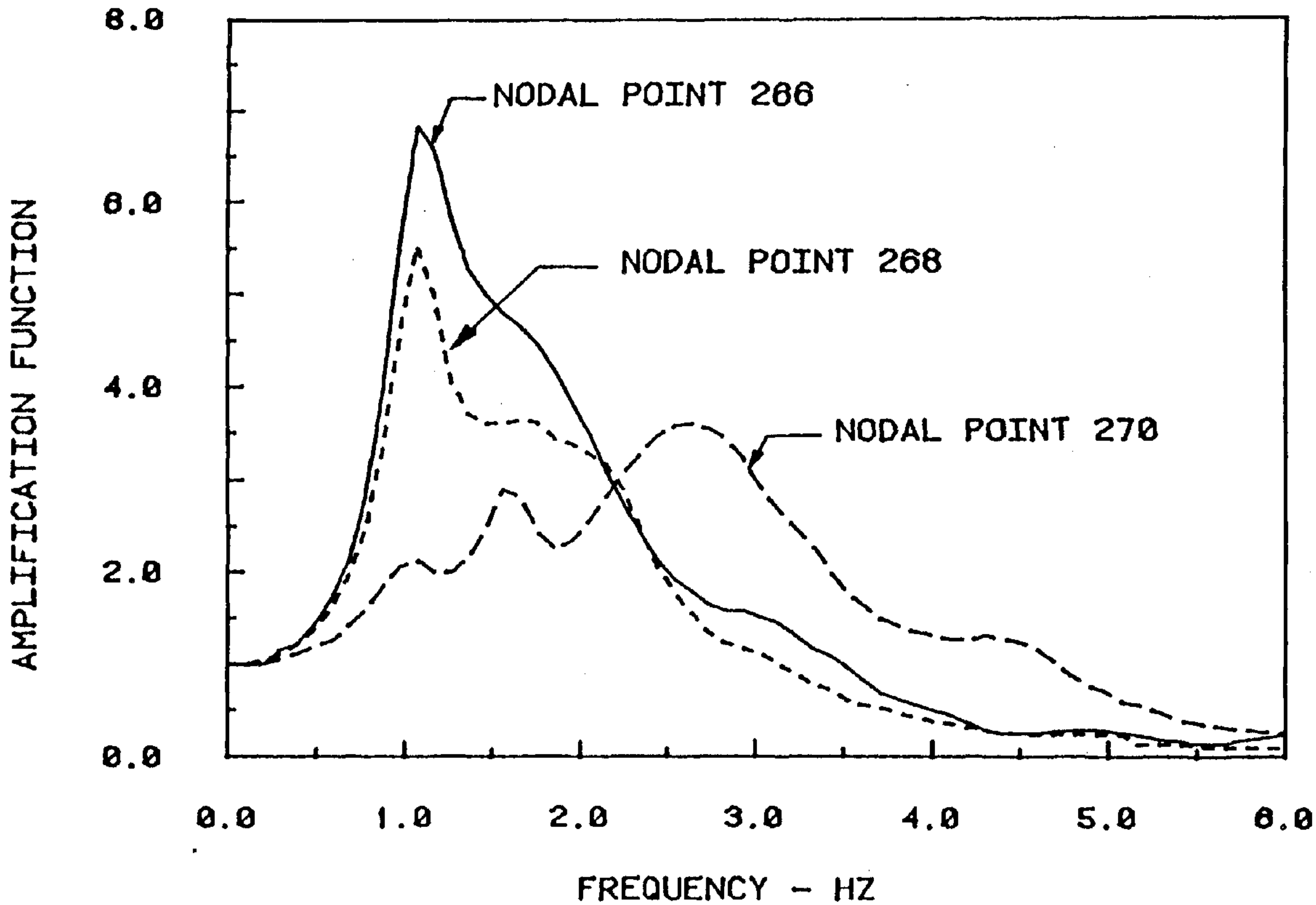


FIG. 29 COMPUTED ACCELERATION AMPLIFICATION FUNCTIONS AT THE CREST TO THE EARTHQUAKE OF MAY 27, 1980

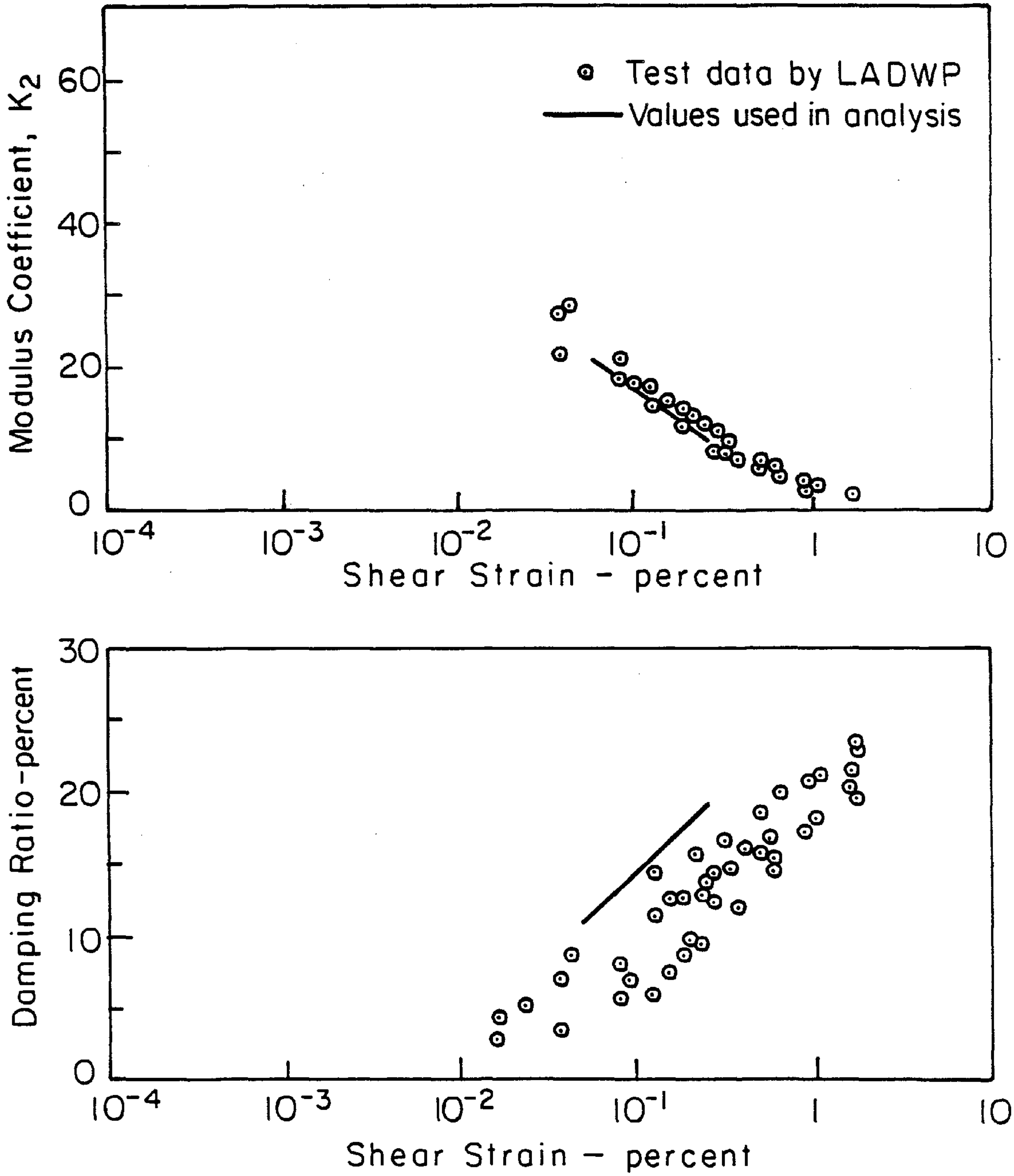


FIG. 30 MODULUS COEFFICIENTS, K_2, AND DAMPING RATIOS FOR RESPONSE ANALYSES AND LABORATORY TESTS

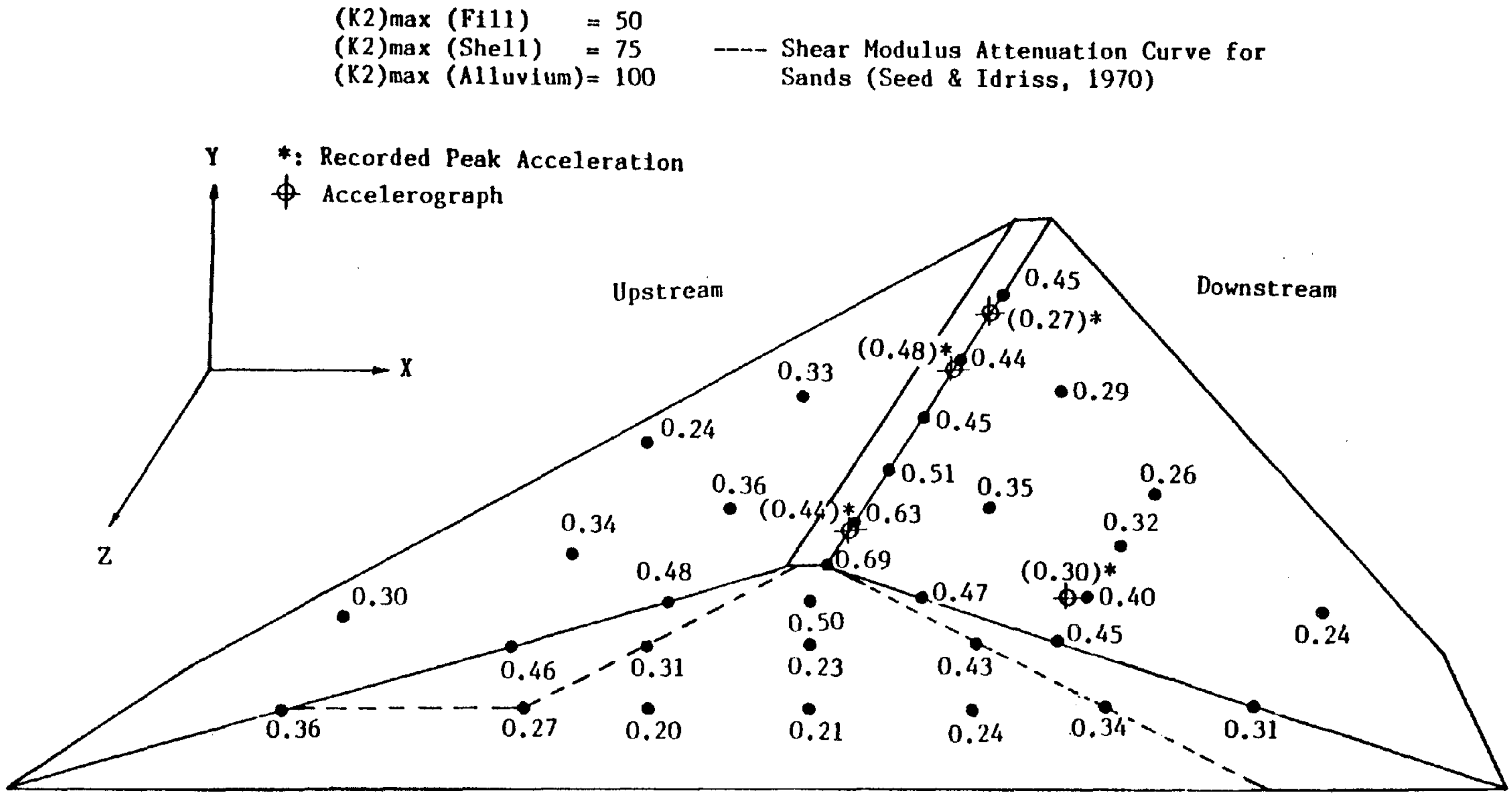
somewhat lower than those indicated by the dynamic response analyses.

A comparison between computed and recorded values of peak accelerations at different points in the embankment is shown in Fig. 31. It may be seen that the agreement is good but it seemed likely that it would be improved by a further reduction in the values of the stiffness for the different soils. Recently a new shear modulus attenuation curve for gravels has been suggested by Seed et al. (1984). Therefore, the possible applicability of this new curve for improving the results of the dynamic response analysis of this dam to the same earthquake will be investigated in the following section.

Reanalysis of 3-D Response for the Earthquake of May 27, 1980

Since the material comprising the main body of compacted fill for the embankment contained about 23% of gravel sizes, 63% of sand sizes and 14% fines, while the shell material varied from coarse sand to small rock, it is of interest to investigate the computed response of the embankment to the same earthquake with the different attenuation curve recently proposed by Seed et al. for gravels.

The shear modulus attenuation curve for sands (Seed & Idriss, 1970) has been widely used for granular materials in response calculations. In engineering practice, this attenuation curve has also been considered appropriate for most gravels. Mejia (1981) studied the dynamic response of Oroville Dam, which was constructed mainly of gravelly material, and he concluded that good results were obtained using this curve and the same 3-D analytical procedure as that described in the previous chapter. The shear modulus reduction curve for gravels recently published by Seed et al. (1984) is shown in Fig. 32. It may be noted that the curve falls below the curve for sands proposed by Seed and Idriss (1970). This suggests that



(Maximum Base Acceleration = 0.18g)

FIG. 31 PEAK ACCELERATIONS IN THE EMBANKMENT FROM 3-D ANALYSIS TO THE EARTHQUAKE OF MAY 27, 1980

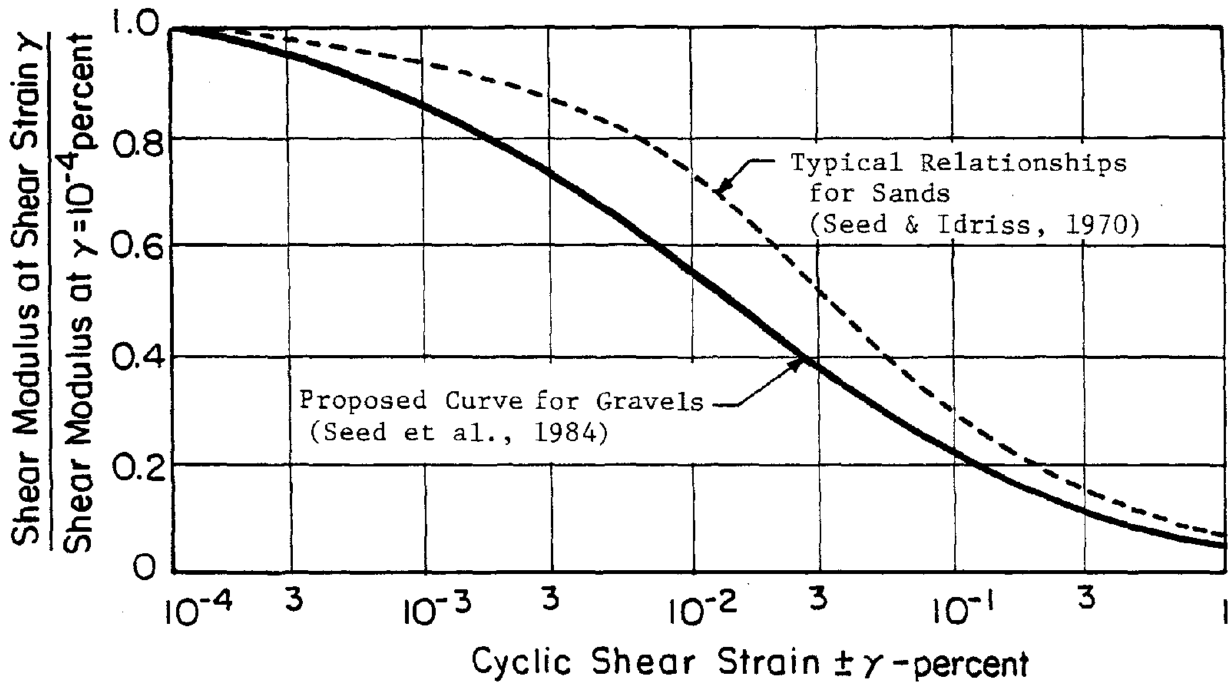


FIG. 32 SHEAR MODULUS ATTENUATION CURVES FOR GRANULAR MATERIALS

the computed response might be lower if this curve was used to represent the properties of the gravelly sand rather than the curve for sands.

Following the same 3-D analytical procedure and using the same set of values of (K_2)max, (100 for the foundation alluvium, 75 for the shell material and 50 for the compacted fill) as before, the computed 3-D response spectra (5% damping) at various points in the embankment are shown in Fig. 33. It may be noted that the degree of agreement between the response spectra for the computed and recorded motions was not improved significantly over that obtained in the previous 3-D analysis using the shear modulus attenuation curve for sands. The computed peak acceleration and the peak spectral acceleration at nodal point 267 were reduced to some extent, but the predominant period was unchanged. There was no discernible change in the computed response at nodal point 413, corresponding to the recording station at the mid-downstream face, and surprisingly the computed responses at nodal points 270 and 271 were somewhat higher than those obtained in the previous analysis.

The computed peak acceleration distribution in the embankment is shown in Fig. 34. The overall values of the computed peak accelerations are lower than those shown in Fig. 31 as a result of the use of the lower shear modulus reduction curve. A comparison of the computed accelerations from the two analyses, performed using different modulus attenuation curves, with the recorded motions is shown in Table 5. A significant reduction in peak acceleration was achieved by using the modulus attenuation curve for gravels and the computed values were much closer to the recorded values, especially at the midpoint of the crest.

Generally speaking, the agreement between computed and recorded motion characteristics obtained in this analysis was not significantly better than that obtained in the previous 3-D response analysis, possibly because the

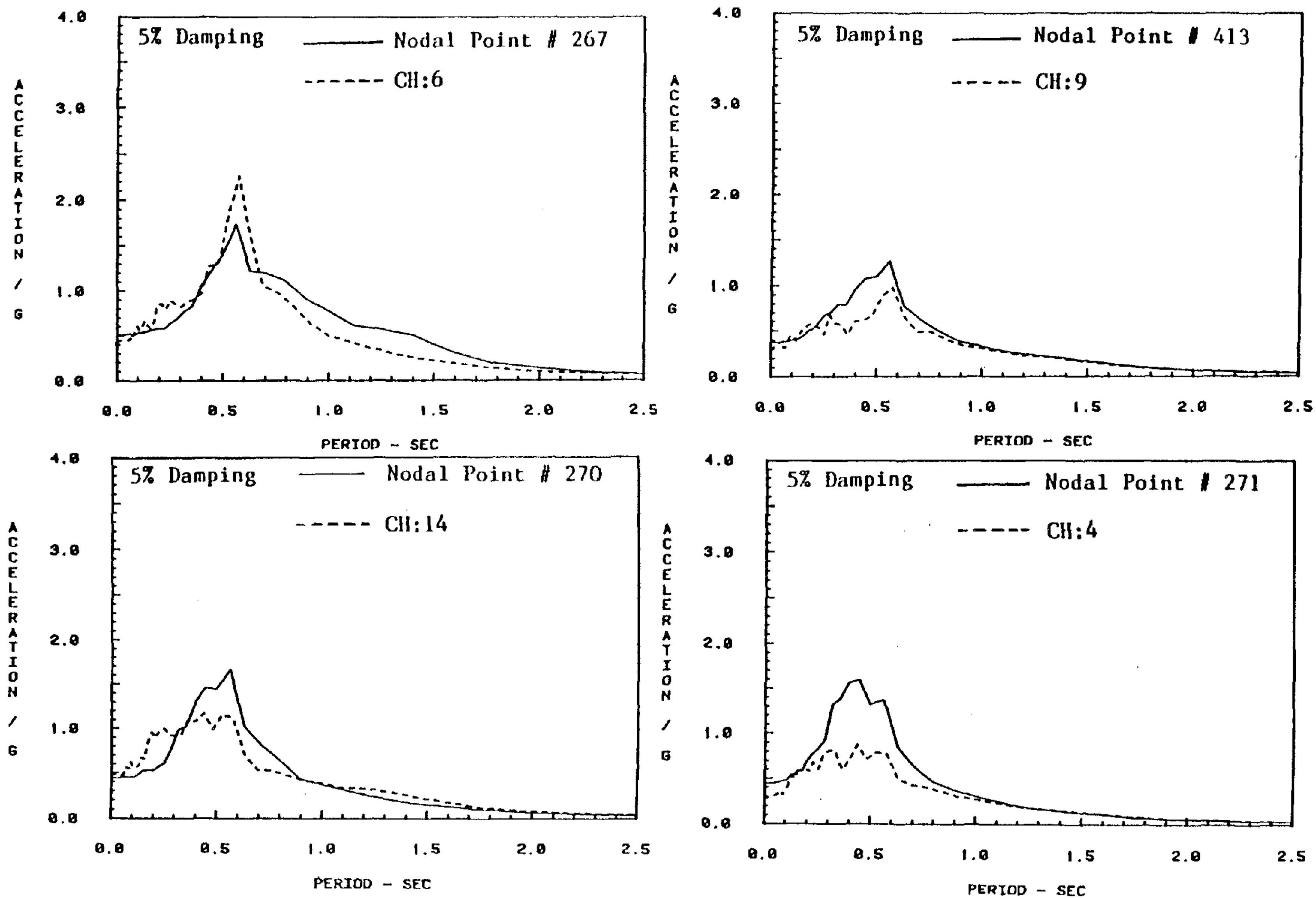


FIG. 33 COMPUTED 3-D RESPONSE SPECTRA WITH NEW SHEAR MODULUS ATTENUATION CURVE AND (K2)max (FILL) = 50, (K2)max (SHELL) = 75, (K2)max (ALLUVIUM) = 100 (MAY 27, 1980 EARTHQUAKE)

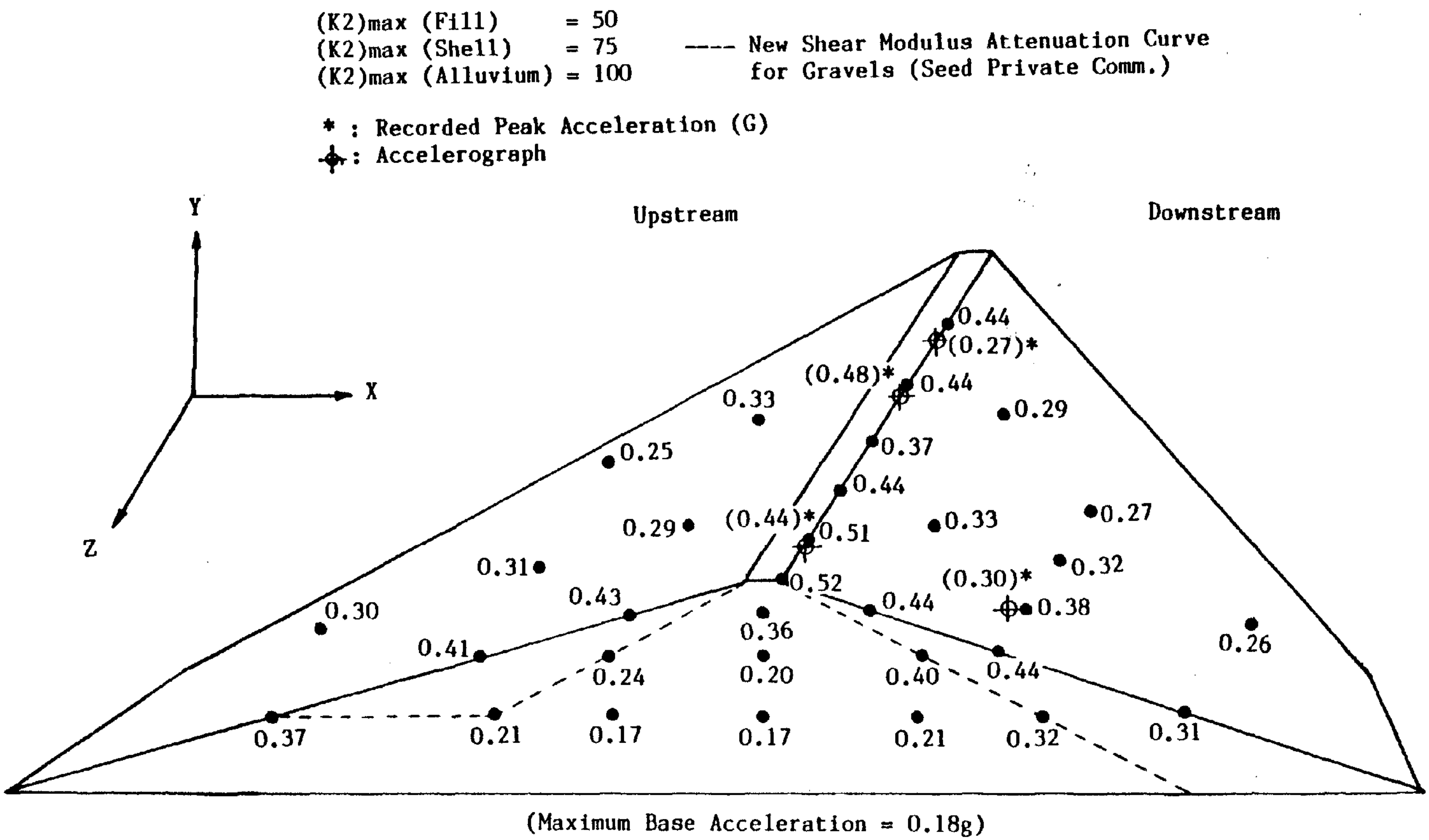


FIG. 34 PEAK ACCELERATIONS IN THE EMBANKMENT FROM 3-D ANALYSIS WITH NEW SHEAR MODULUS ATTENUATION CURVE TO 5-27-1980 EARTHQUAKE

Table 5 Comparisons of the Computed and Recorded Peak Accelerations with Different Modulus Attenuation Curves (5-27-1980 Eq.)

Locations	Recorded (G)	Computed (1)* (G)	Computed (2)** (G)
Crest Center (Channel 6)	0.44	0.63	0.51
Crest (Right) (Channel 14)	0.48	0.44	0.44
Crest (Left) (Channel 4)	0.27	0.45	0.44
Downstream (Mid.) (Channel 9)	0.30	0.40	0.38

(1)* : Computed results using standard modulus attenuation curve for sands (Seed and Idriss, 1970).

(2)** : Computed results using new modulus attenuation curve for gravels (Seed et al., 1984).

sand-size material is dominant in the determination of the dynamic shear moduli for the Long Valley embankment material. However the results of this analysis do show that a small change in the form of the modulus attenuation curve used in the analysis can significantly affect the computed values of peak accelerations.

On the basis of these results, it may be concluded that the set of values of (K_2)max: 50 for the compacted fill, 75 for the shell material and 100 for the foundation alluvium, together with the shear modulus attenuation curve for sands best represent the in-situ dynamic properties of Long Valley embankment materials under earthquake loading conditions and these values are in good agreement with values determined by laboratory tests on representative samples.

2-D Response Analysis to the Earthquake of May 25, 1980 (09:34 a.m.)

Previous studies (Lai, 1985) have shown that the applicability of the 2-D response analysis procedure for the prediction of dynamic response of embankment dams with complex geometries and 3-D configurations is uncertain, and accordingly, the corresponding dynamic properties to be used in such analyses may be significantly different from the actual properties. Thus, it is interesting to check if the dynamic properties obtained from the previous 2-D dynamic analysis are able to predict the dynamic response of the embankment to other earthquakes.

Examination of the other available accelerograms recorded at the Long Valley damsite (Table 1) indicates that the recorded data for the earthquake of May 25, 1980 at 09:34 a.m. are probably the best with regard to the integrity of the recording channels (22 channels) and the comparison of acceleration amplification functions with those for the earthquake motions recorded on May 27. Thus, this set of recorded data was used to

check the applicability of the modulus values determined by the 2-D analysis procedure. The corresponding peak transverse and longitudinal accelerations recorded in the embankment for the earthquake of May 25, at 09:34 a.m. are shown in Fig. 35.

The response spectra (5% damping) for the motions recorded at channels 11, 6, and 9, corresponding to stations on bedrock at the downstream toe, on the downstream face, and on the crest near the center of the dam are shown in Fig. 36. The recorded motions in bedrock show significant high frequency components but the spectra for channel stations 6 and 9 show a predominant period of about 0.55 second, which presumably indicates the fundamental period of the dam.

Fig. 37 shows the response spectra for the recorded motions at channel stations 1, 11, and 17 (transverse components), which are all located on rock on the left abutment. It may be noted again that the recorded motions at channel 1 (bedrock near the toe) and 11 (bedrock on the left abutment at the elevation of the crest) are quite similar and can be considered identical from an analytical point of view. The recorded motion for channel 17 (rock outcrop above the crest elevation) shows significantly different frequency contents and a much higher amplitude of motions. Good agreement is also observed in the recorded response spectra for channels 3 and 12 (longitudinal components) at the stations on rock at the toe of the dam and at crest elevation.

A 2-D dynamic analysis was performed using the same dynamic shear moduli and damping characteristics for the embankment materials, as those determined from the previous 2-D analysis: ((K_2max = 60 for compacted fill, 90 for dumped rockfill, and 120 for streambed alluvium). The response spectra for the computed motions in this study are presented in Fig. 38. Fairly good agreement was obtained between the characteristics of the computed and recorded motions

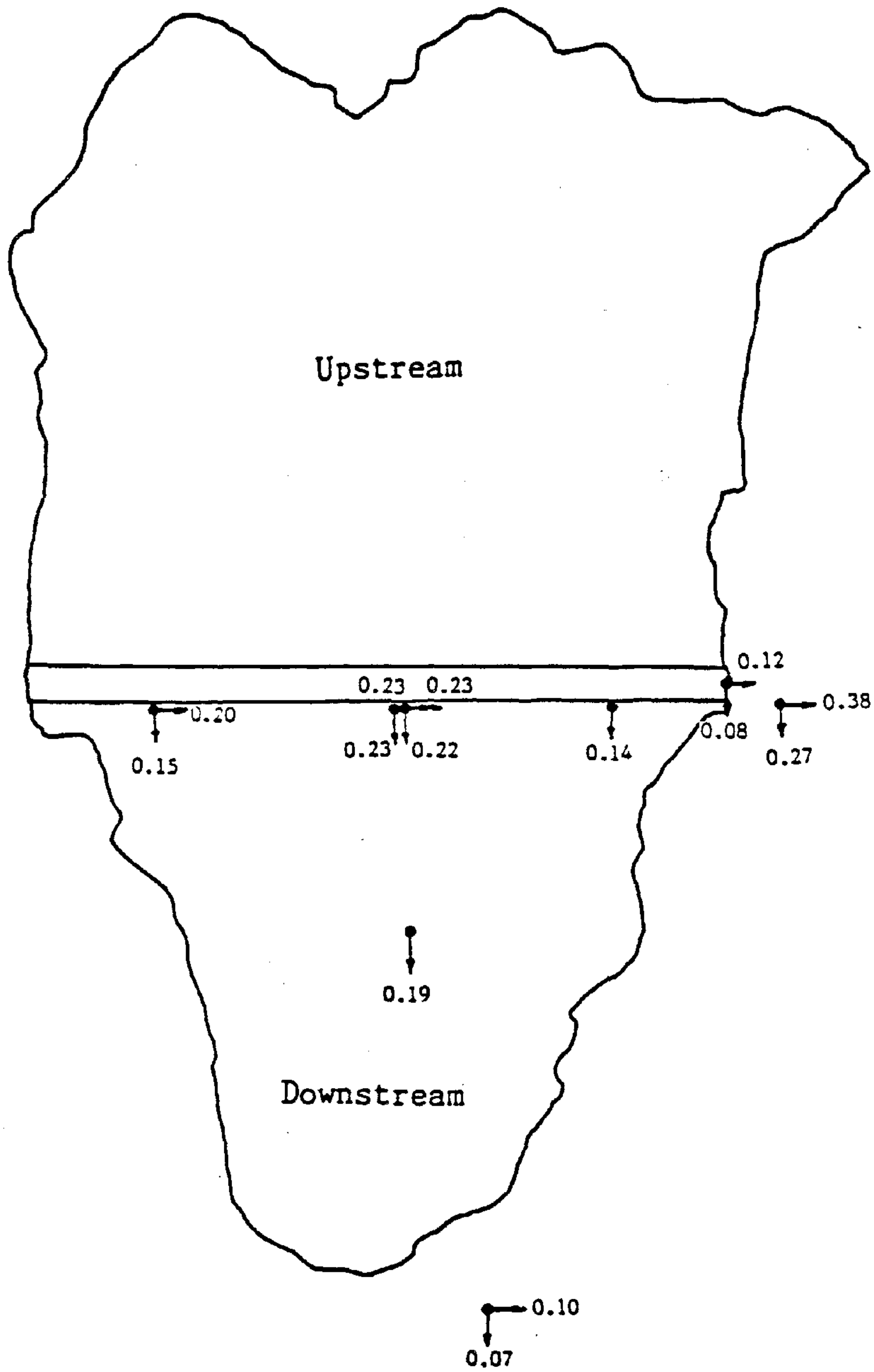


FIG. 35 PEAK ACCELERATIONS (G) RECORDED IN THE EMBANKMENT FOR THE EARTHQUAKE OF MAY 25, 1980 AT 09:34 A.M.

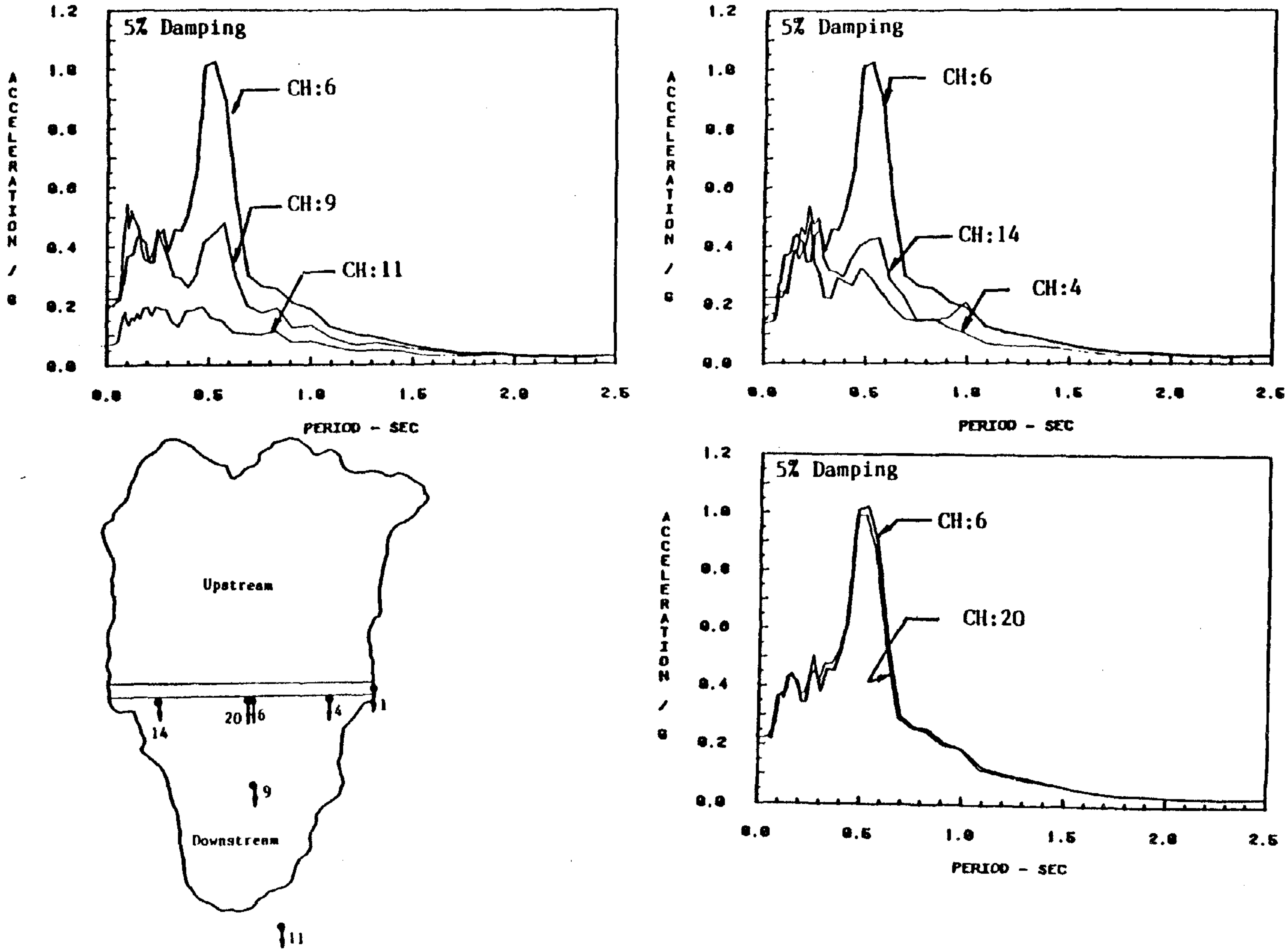


FIG. 36 RECORDED RESPONSE SPECTRA AT THE DAM SITE TO THE EARTHQUAKE OF MAY 25, 1980 AT 09:34 A.M.

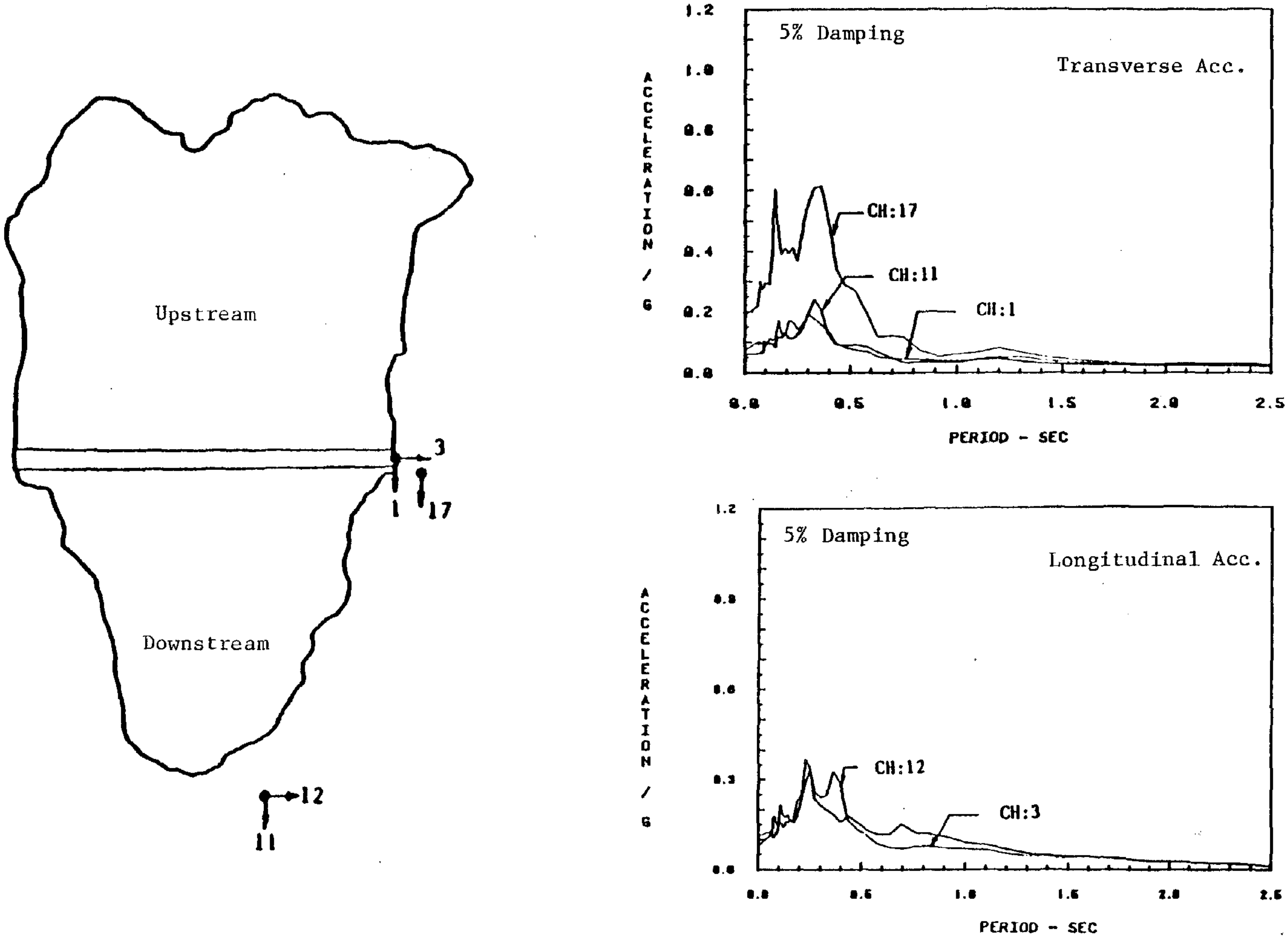
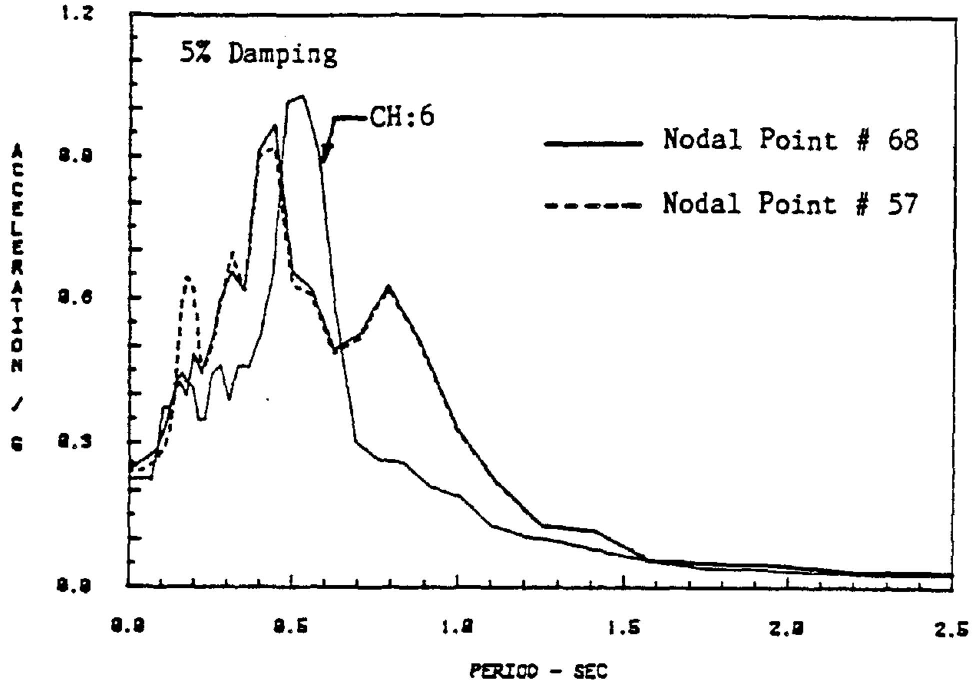


FIG. 37 RECORDED RESPONSE SPECTRA AT THE LEFT ABUTMENT TO THE EARTHQUAKE OF MAY 25, 1980 AT 12:45 P.M.



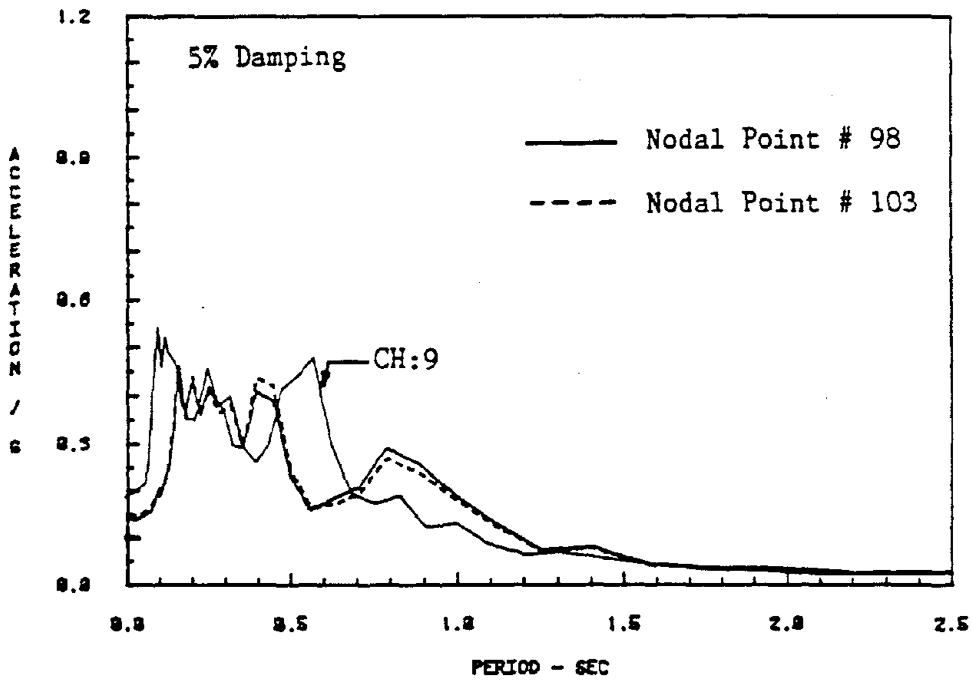


FIG. 38 COMPUTED 2-D RESPONSE SPECTRA TO THE EARTHQUAKE OF MAY 25, 1980 AT 09:34 A.M. WITH (K2)max (FILL) = 60
(K2)max (SHELL) = 90
(K2)max (ALLUVIUM) = 120

at stations for channel 6 on the crest and channel 9 on the downstream face of the embankment.

The computed peak accelerations at nodal points 57 and 68 at the crest of the dam are very close to the recorded peak acceleration for channel 6, located on the crest of the dam. A significant spectral peak at a period of 0.8 second is found in the computed response and the recorded motion has very little frequency content at this period. However the overall frequency content seems to be in reasonably good agreement with that for the recorded motions.

For the selected dynamic soil properties, the computed peak acceleration distribution in the embankment determined by the 2-D response analysis is shown in Fig. 39. The results are in good agreement with the observed peak accelerations.

Conclusions

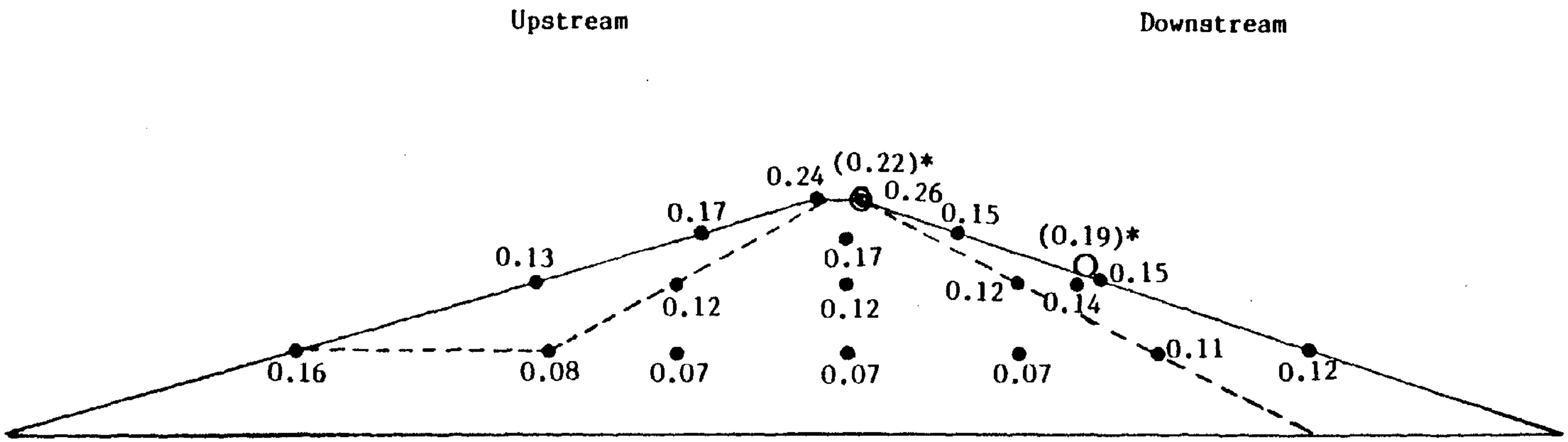
A comprehensive set of data on the seismic performance of the Long Valley Dam has been provided by the California Strong Motion Instrumentation Program during the Mammoth Lakes earthquake series from May 25 to 27, 1980. In this investigation, 2-D and 3-D dynamic analysis procedures were used to check the applicability of the equivalent-linear complex-response method to predict the seismic response of the dam which has a complicated 3-D configuration. 2-D dynamic analyses were performed to study the dynamic response during the earthquakes of May 27 and May 25 (at 09:34 a.m.). However because of the good results obtained only the seismic event of May 27 was used in the 3-D response analysis.

In order to investigate the suitability of a new shear modulus attenuation curve for the Long Valley embankment materials, another 3-D response analysis was also carried out to determine the dynamic response of the embankment using

(K2)max (Fill) = 60
(K2)max (Shell) = 90
(K2)max (Alluvium)= 120

*: Recorded Peak Acceleration

Accelerograph



(Maximum Base Acceleration = 0.07g)

FIG. 39 PEAK ACCELERATIONS IN MAXIMUM SECTION FROM 2-D ANALYSIS TO THE EARTHQUAKE OF MAY 25, 1980 AT 09:34 A.M.

the new shear modulus attenuation curve and the same values of $(K_2)max$ as in the previous 3-D analysis.

The dynamic properties of the embankment materials which provide the best agreement between the computed and recorded motions in the various analyses were determined; the results of these determinations are summarized in Table 6. The values determined are in good agreement with values determined by laboratory tests (report of LADWP) on the soil before the earthquakes.

Other important conclusions drawn from the above investigations are listed below:

(1) The recorded seismic performance of Long Valley Dam to the earthquake of May 27, 1980 has been successfully simulated by 2-D and 3-D response analyses using appropriate combinations of the values of dynamic shear modulus coefficient $(K_2)max$ in the analyses.

(2) A set of dynamic soil properties, $(K_2)max = 50$ for the compacted earthfill (gravelly sand with fines), $(K_2)max = 75$ for the dumped shell material (sluiced coarse sand, gravel and small rock), and $(K_2)max = 100$ for the streambed alluvium (gravels and sands), were determined from the 3-D dynamic analysis and considered to be best representative of the in-situ dynamic soil properties of Long Valley Dam embankment materials to earthquake shaking.

(3) Good agreement between computed and observed response was obtained using slightly higher values of dynamic moduli in a 2-D dynamic analysis of the dam, as follows: $(K_2)max = 60$ for compacted earthfill, $(K_2)max = 90$ for dumped shell material, and $(K_2)max = 120$ for streambed alluvium. The same property values which gave good results for the earthquake of 5-27-1980 also gave fairly good

Table 6 Summary of Results of Analytical Studies for Long Valley Dam

2-D Analysis

Values of (K2)max

Earthquake	Compacted Fill	Dumped Rockfill	Streambed Alluvium	Comments
May 27, 1980	60	90	120	Excellent agreement with recorded motions at two recording stations in the main section.
May 25, 1980 (09:34 a.m.)	60	90	120	Fairly good agreement with recorded motions at two recording stations in the main section.

3-D Analysis

Values of (K2)max

Earthquake	Compacted Fill	Dumped Rockfill	Streambed Alluvium	Comments
May 27, 1980	50	75	100	Very good agreement with recorded motions at four recording stations throughout the entire embankment.
May 27, 1980	50*	75*	100*	Fair agreement with the recorded motions at four recording stations, but excellent agreement with peak acceleration values at four recording stations.

* : Using the new shear modulus reduction curve for gravels (Seed et al., 1984).

agreement between computed and recorded motions for the earthquake of May 25, 1980 at 09:34 a.m.

(4) The new shear modulus attenuation curve, proposed for gravel material by Seed et al. (1984), leads to better estimates of peak accelerations for Long Valley Dam but poorer overall motion characteristics compared with those obtained using a modulus attenuation curve developed from tests on sands. This may be so because the main embankment material for Long Valley Dam consists mainly of sand containing only a little gravel.

Acknowledgement

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