

RECLAMATION

Managing Water in the West

Dam Safety Office, Research Report No. DSO-05-06

Effect of Spatially Varying Earthquakes on Concrete Gravity, Arch, and Slab-and-Buttress Dams



U.S. Department of the Interior
Bureau of Reclamation
Denver, Colorado

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**By Barbara Mills-Bria, PE
Structural Analysis Group**



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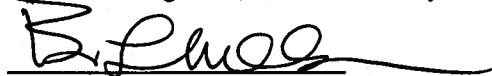
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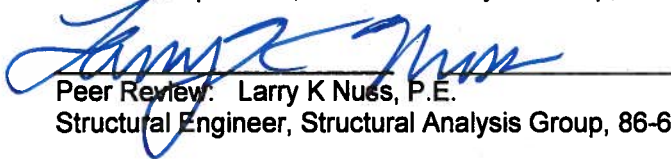
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1.0 Background

The structural analysis of concrete dams at the Bureau of Reclamation (Reclamation) has evolved through the years as new technologies become available. In the recent past, the Technical Service Center's Structural Analysis Group (D-8110) performed dynamic structural finite element analysis by applying the ground motions to a massless foundation resulting in a uniform acceleration reaching the dam/foundation interface at each time step. In reality, ground motions vary across the foundation with time. The larger and deeper the canyon, the more varied the ground motions become as they reach the ground surface in various locations across the canyon. Application of spatially varying ground motions may greatly affect the response of a dam.

The state-of-the-art finite element structural analysis packages, *NIKE3D/DYNA3D* (Lawrence Livermore National Laboratory, 2000), currently used at Reclamation, allow the dam/reservoir/foundation system to be considered. Because the entire system is modeled and because mass is included in the foundation, changes must be made in the way seismic input is applied.

A similar finite element package, *ABAQUS/Explicit* (Hibbit et al., 1998), was used at Reclamation to analyze a slab-and-buttress dam. Because each buttress responds as a separate structural element, and because the foundation material varies across the canyon, ground motions reaching the base of each buttress will vary in time and phase. The response of this structure to these spatially varying ground motions may or may not be different than the response determined by applying the ground motions uniformly across the canyon.

The objective of this research is to study the effects of spatially varying ground motions and, if necessary, incorporate the capability of varying ground motion input into the analysis procedures used by the Structural Analysis Group.

2.0 Slab-and-Buttress Dam

2.1 Model description

To address the seismic evaluation of Stony Gorge Dam, a finite element model was created and analyzed using *ABAQUS/Explicit*. This model was originally analyzed using uniform ground motions (i.e., one set of ground motions applied at the base of all of the buttresses). The analysis was repeated for this study, using 42 sets of ground motions, each determined for a particular buttress location. Shown in figure 2.1, the model consisted of:

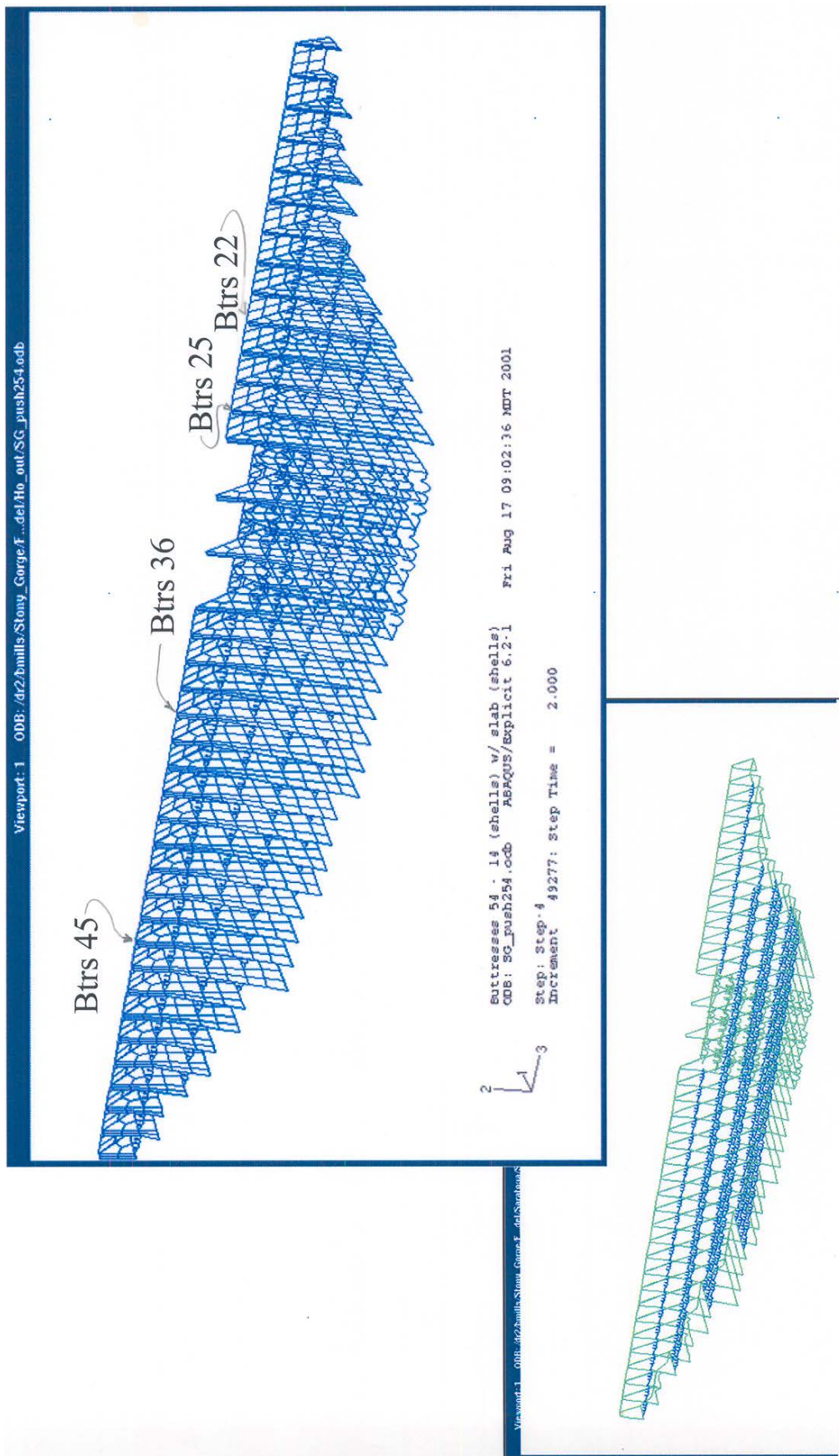


Figure 2.1 Model of Stony Gorge Dam.

- a. Buttresses, 42 in total and numbered 13 through 54, were modeled using shell elements. The buttresses were fixed against displacement at the base in three directions. This allowed the input of ground motions in three directions at this location. Rotations were not fixed. The buttresses are relatively narrow concrete sections founded on concrete footings which are keyed into the rock foundation. Rotation at the footing is considered to be possible, but sliding is not as likely.
- b. Slabs, 41 in total, were modeled using shell elements. The use of shell elements allowed the mass of the slab to be included. Similar to the buttresses, the slabs were fixed against displacement in three directions at the base, but were free to rotate.
- c. Struts, 254 in total, providing lateral support between buttresses were modeled. The struts are simply supported at the buttresses so springs were used with a:
 - (1) Compressive stiffness based on the concrete properties and cross section of the struts
 - (2) Low tensile stiffness
- d. Slabs were connected to the buttresses using multiple point constraints (MPC). The type of MPC used, a slider, keeps a node on a straight line defined by two other nodes, but allows the possibility of moving along the line and allows the line to change length (see figure 2.2). The behavior of this MPC is similar to a ring sliding along a rod. Thus, nodes connecting the slabs to the buttresses were allowed to slide along the upstream face of the buttresses. This allowed the slabs and buttresses to deform without shear resistance or moment transfer. Loads were transferred from the slabs into the buttresses so that the structural response is well represented.

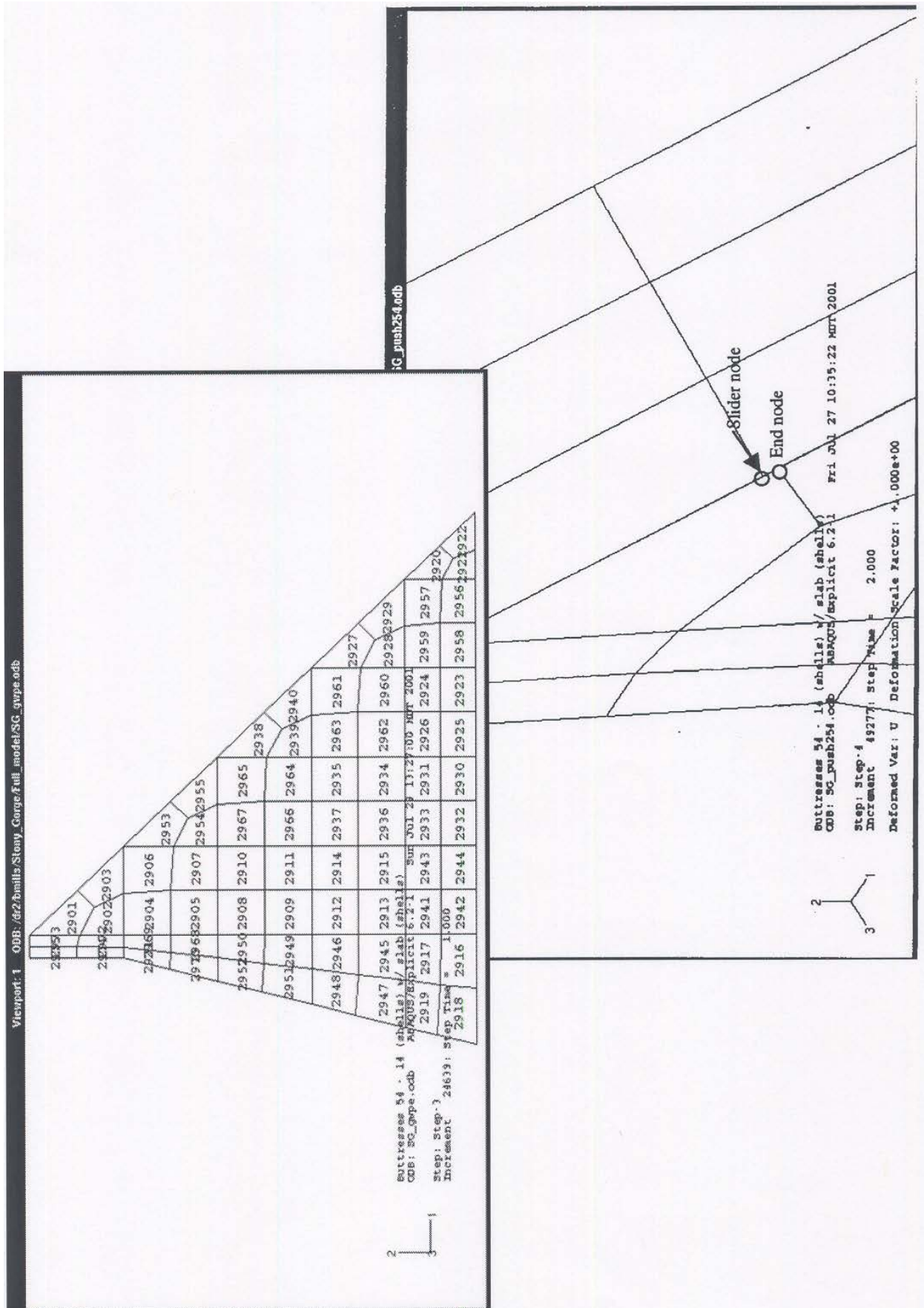


Figure 2.2 Slab-to-buttress connection (multiple-point constraints).

- e. The spillway gate hoist house, steel sliding gates, and overtopping wall on the crest of the dam were modeled as lumped masses.
- f. The concrete material model used was linear elastic and assumed an elastic modulus of 3.66×10^6 pounds per square inch (lb/in²), a Poisson's ratio of 0.15, and a density of 150 pounds per cubic foot (lb/ft³). Rayleigh damping was used. The following values were assumed: α (mass damping) = 1.743584 and β (stiffness damping) = 0.000797 resulting in a combined damping percentage of approximately 6.5 percent for the fundamental mode [frequency = 2.6 hertz (Hz)] and ranging between 3.75 and 4.5 percent for higher modes. (See figure 2.3)

Calculations for Rayleigh Damping Coefficients

Input % Critical Damping for The 2 Frequencies You Want in the Yellow Areas:						
		cps	Rad/sec		Percent	Decimal
Uncracked						
	Freq 1:		0.000	% Damp 1:		0.000
	Freq 2:		0.000	% Damp 2:		0.000
Solve simultaneous equations:			Alpha + w^2 *	Beta = 2*w*Epsilon		
			Alpha +	0.00 Beta =	0	
			Alpha +	0.00 Beta =	0	
Uncracked						
Beta =	0.000797	(stiffness)	Freq Increm	1		
Alpha =	1.743584	(mass)				
		Stiffness	Mass			
Freq vs % Damping		Damping	Damping			
0.50	27.88	0.13	27.75			
1.00	14.13	0.25	13.88			
2.00	7.44	0.50	6.94			
3.00	5.38	0.75	4.63			
4.00	4.47	1.00	3.47			
5.00	4.03	1.25	2.78			
6.00	3.81	1.50	2.31			
7.00	3.73	1.75	1.98			
8.00	3.74	2.00	1.73			
9.00	3.80	2.25	1.54			
10.00	3.89	2.50	1.39			
11.00	4.02	2.75	1.26			
12.00	4.16	3.00	1.16			
13.00	4.32	3.26	1.07			
14.00	4.50	3.51	0.99			
15.00	4.68	3.76	0.93			
16.00	4.87	4.01	0.87			
17.00	5.07	4.26	0.82			
18.00	5.28	4.51	0.77			
19.00	5.49	4.76	0.73			

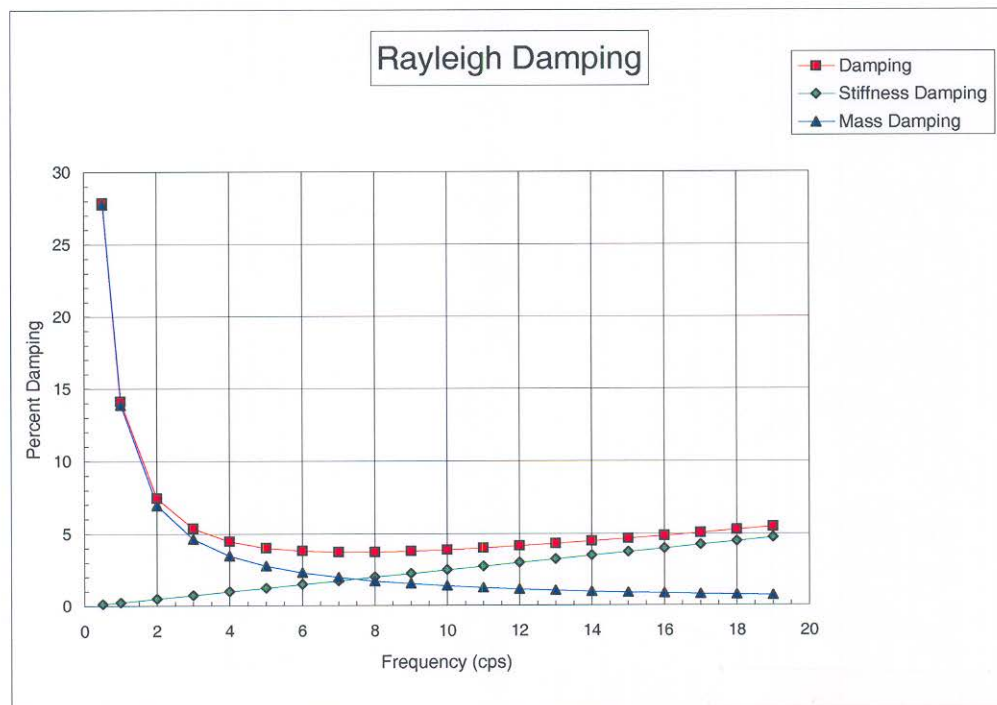


Figure 2.3 Rayleigh Damping

2.2 Ground motion description

The Pleasant Valley earthquake record, from the 1994 Northridge, California, earthquake, is shown in figure 2.4, which is a plot of the cross-canyon, upstream-downstream, vertical motions, and the acceleration response spectra at 5-percent damping for each motion. The peak acceleration for the record used in the cross-canyon direction is 0.65 acceleration of gravity (g) and occurs 6.33 seconds (s) into the record. The peak acceleration for the record used in the upstream/downstream direction is 0.71 g at 9.25 s and for the record used in the vertical direction, the peak acceleration is 0.53 g at 4.26 s. (See figure 2.4)

Stony Gorge 2001 -- Pleasant Valley Ground Motions and Response spectra

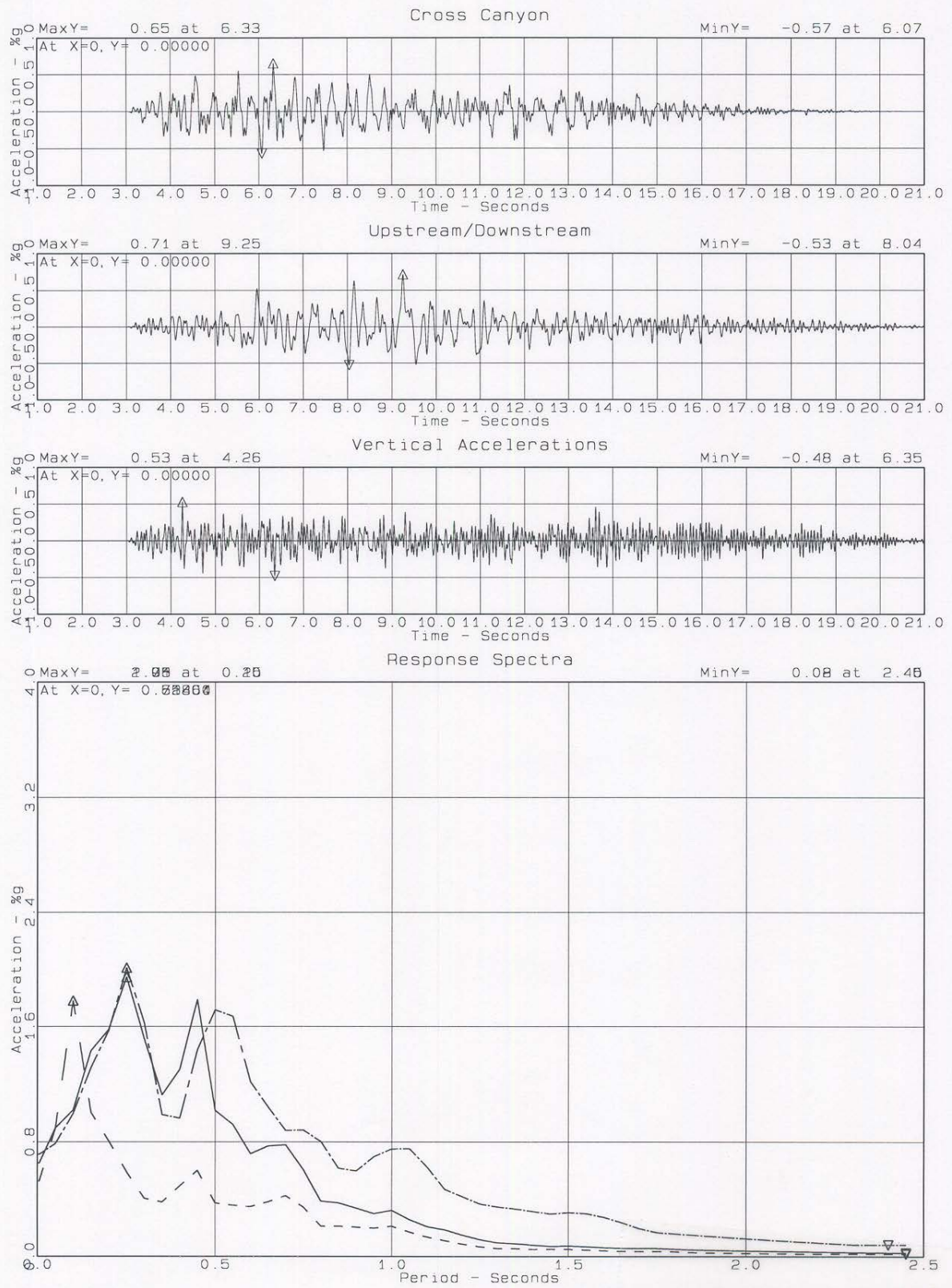


Figure 2.4 Pleasant Valley record.

To specify spatially-varying ground motions, the north and south abutment recordings of 04 January 1998 M 1.5 earthquake from the seismotectonic report (O'Connell, 2001) were used to construct transfer operators to interpolate and scale the Pleasant Valley motions using the northernmost abutment (buttress 45) as the rock reference. A Weiner shaping filter was used to calculate a least-square deconvolution inverse operator of the north-abutment ground velocity seismograms. This deconvolution operator was applied to the south abutment velocity seismograms to produce the transfer function required to convert the north abutment (buttress 45) rock reference motions to south abutment motions (buttress 1).

The Pleasant Valley record was used as the buttress 45 record. As found in the seismotectonic report, south abutment motions are amplified by a ~50-foot-weathered zone that is absent on the north abutment. Consequently, southern buttress ground motions were amplified 10 percent, except for buttresses 26 to 35, which used amplifications of 20 percent to account for complex velocity structure and associated responses documented in the seismotectonic report. A mildly randomized weight function was used to combine north and south abutment responses at each buttress to produce coherence consistent with the findings in the seismotectonic report. The largest randomizations were used for buttresses 26 to 35 because this is the region likely to have the most variable ground motions based on the seismotectonic report. An apparent shear-wave velocity of 3.5 kilometers per second (km/s) was used to calculate propagation delays of 0.0015 s between buttresses, and assuming that seismic energy arrives from the south. Consequently, the arrivals at buttress 45 are delayed 0.066 s relative to the arrivals at buttress 1.

Figure 2.5 shows one set of the spatially varying ground motions, in particular, the set of motions applied to the base of buttress 25. The biggest difference can be seen in the vertical spectra (dashed line) with a less apparent difference shown in the upstream-downstream records. Figures 2.6 to -8 are cross-canyon, upstream-downstream, vertical ground motions, and response spectra, respectively, for three buttresses: buttress 22 (near the left abutment—solid line), buttress 25 (near the spillway-chain—dash line), and buttress 45 (near the right abutment—dashed line). The response spectra in figure 2.6 (cross-canyon motions) indicate that the record for buttress 45 has a slightly different frequency content than the other two records. This reflects the difference in material properties between the left and the right abutments. Figures 2.9 to -11 show a 2-second-interval comparison of the cross-canyon, upstream-downstream, and vertical motions, respectively, for

buttresses 22, 25, and 45. Differences in amplitude and frequency can be seen as well as the phase shifts.

Typical Acc-Stony-Gorge-PVPP, sp-var Ground Motions and Response spectra

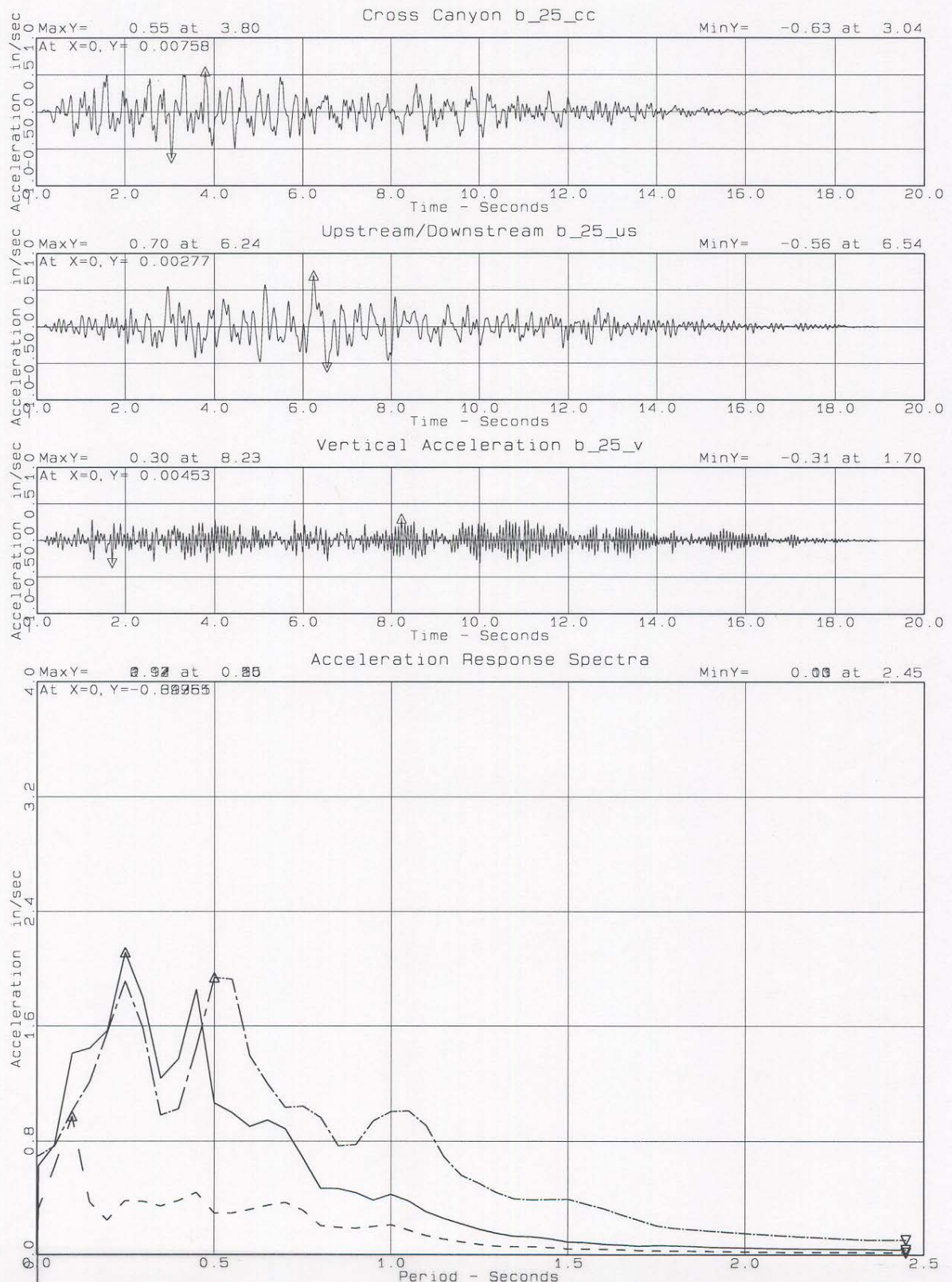


Figure 2.5 Pleasant Valley record—spatially-varying ground motions applied at buttress 25,

Typical Acc-Stony-Gorge-PVPP, sp-var Ground Motions and Response spectra

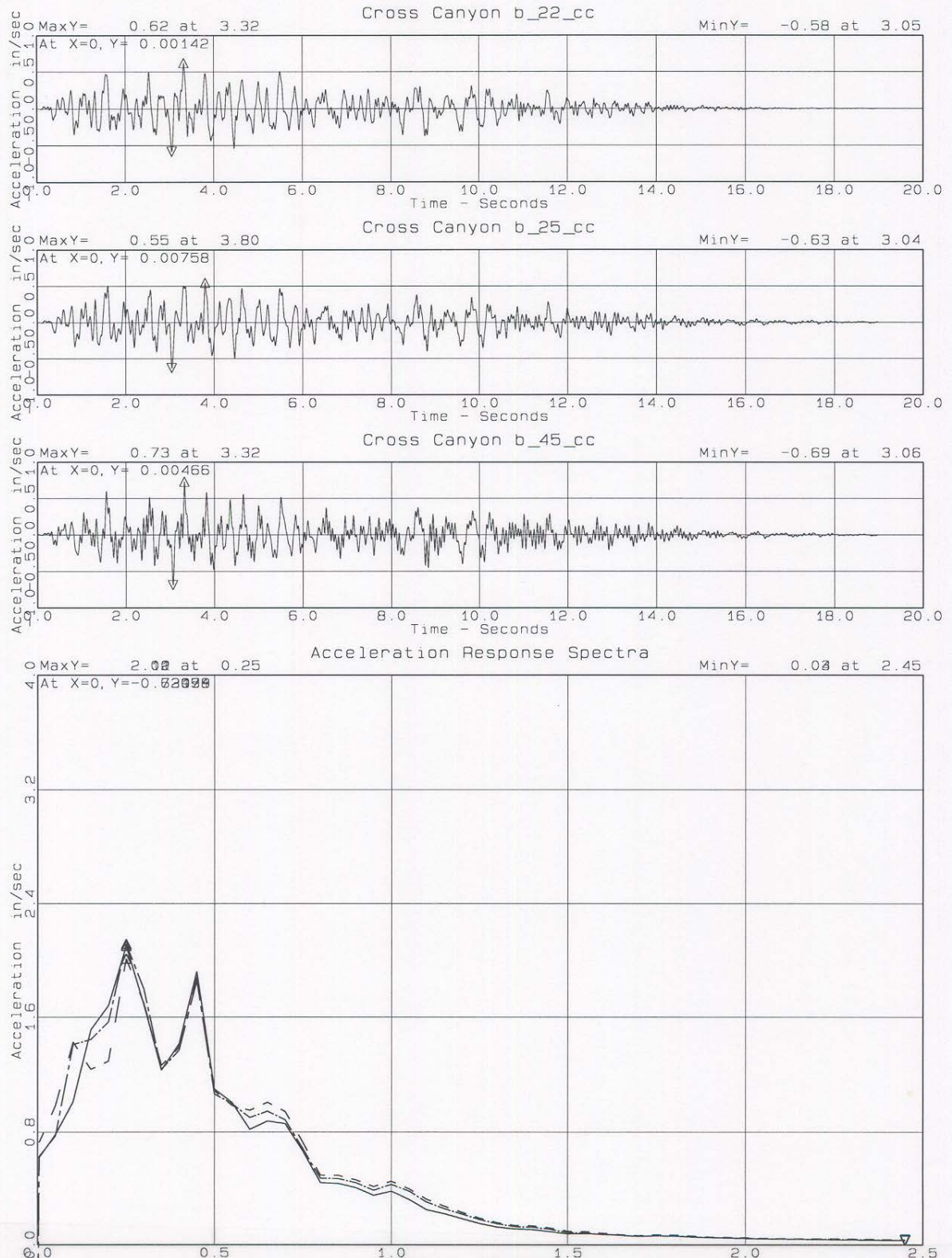


Figure 2.6 Spatially-varying cross-canyon ground motions and response spectra buttresses 22 (left abutment), 25 (near spillway), and 45 (right abutment),

Typical Acc-Stony-Gorge-PVPP, sp-var Ground Motions and Response spectra

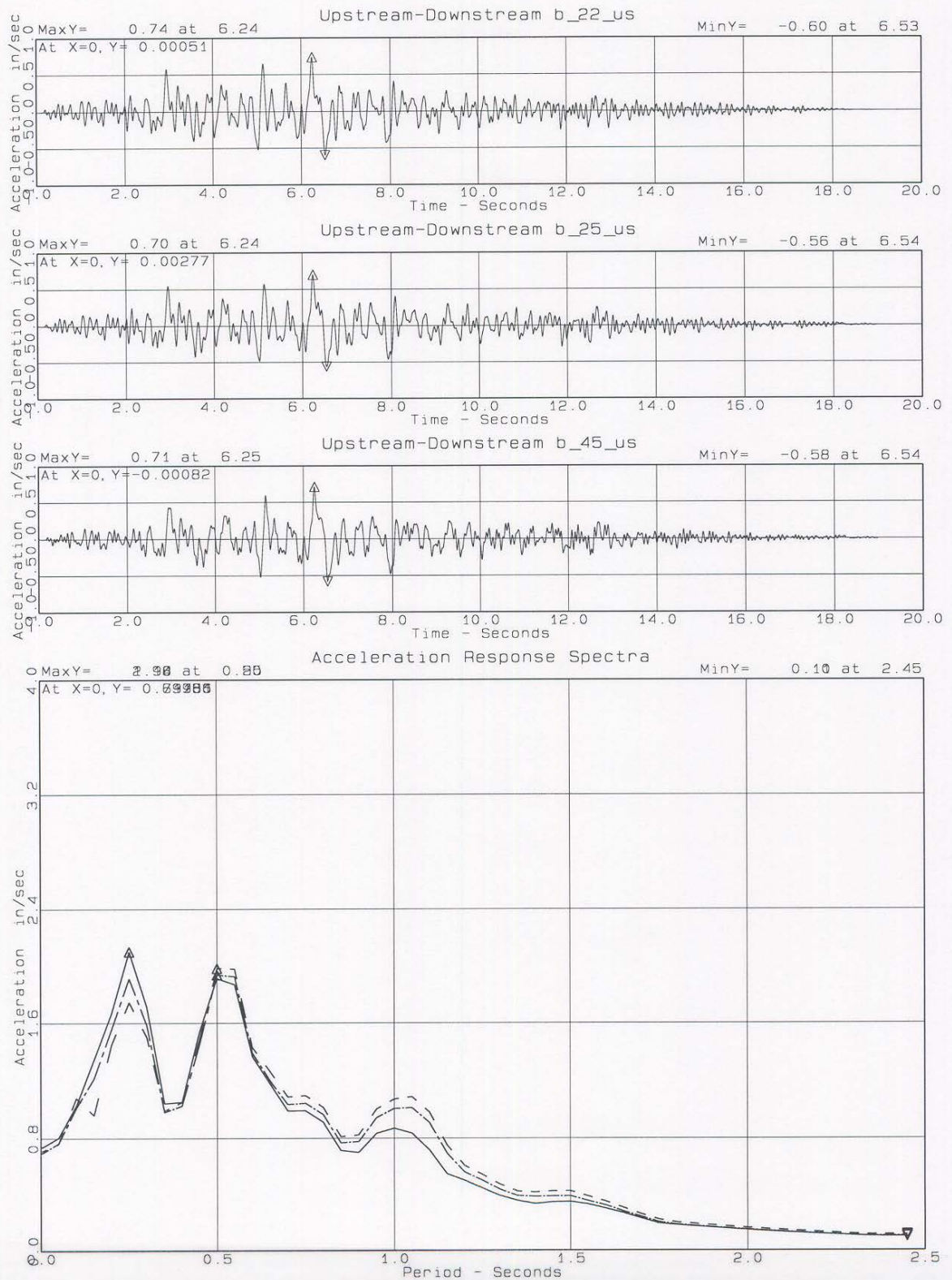


Figure 2.7 Spatially-varying upstream-downstream ground motions and response spectra buttress 22 (left abutment), 25 (near spillway), and 45 (right abutment),

Typical ACC-Stony-Gorge-PVPP, sp-var Ground Motions and Response spectra

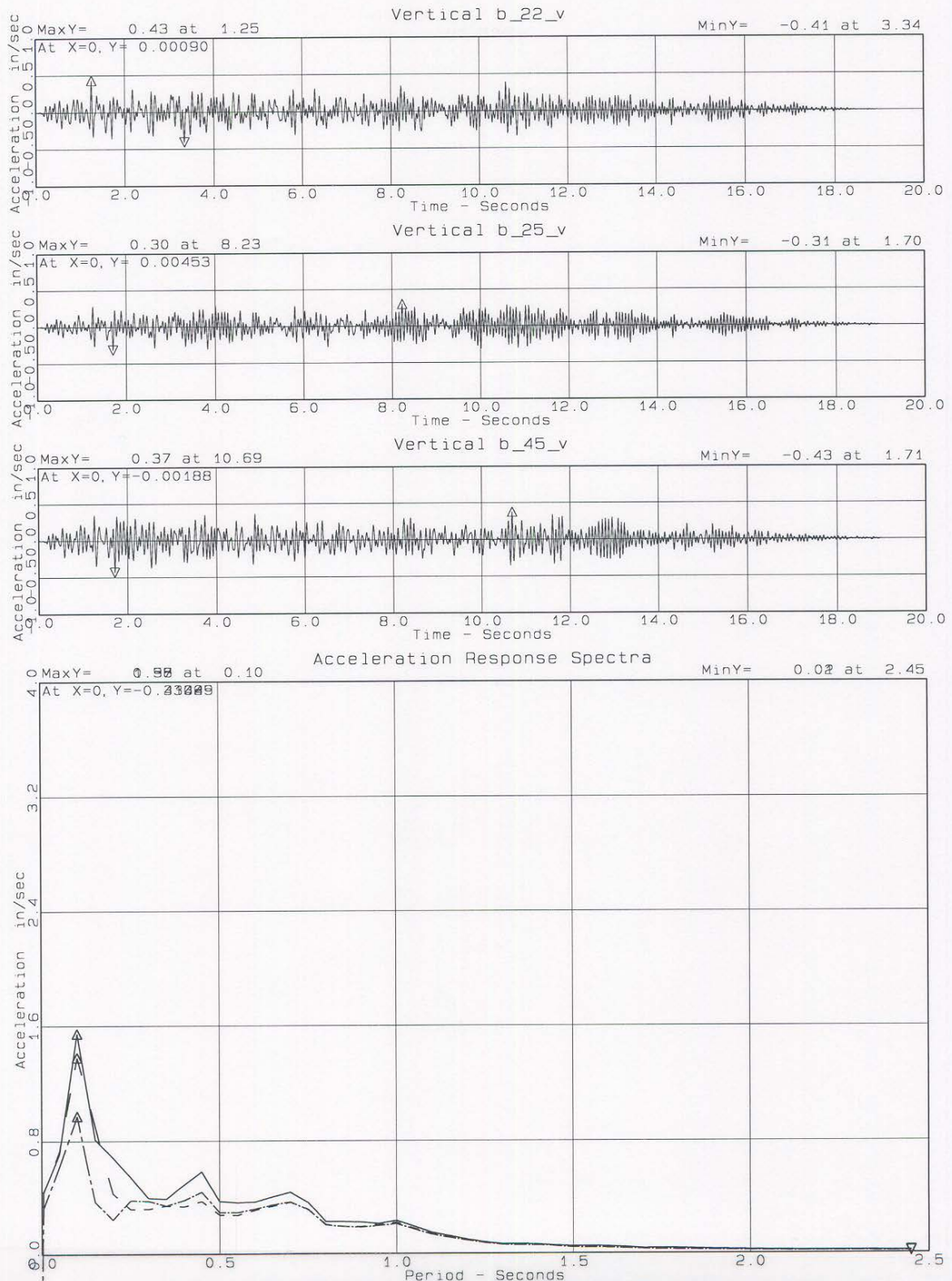


Figure 2.8 Spatially-varying upstream-downstream ground motions and response spectra buttress 22 (left abutment), 25 (near spillway), and 45 (right abutment),

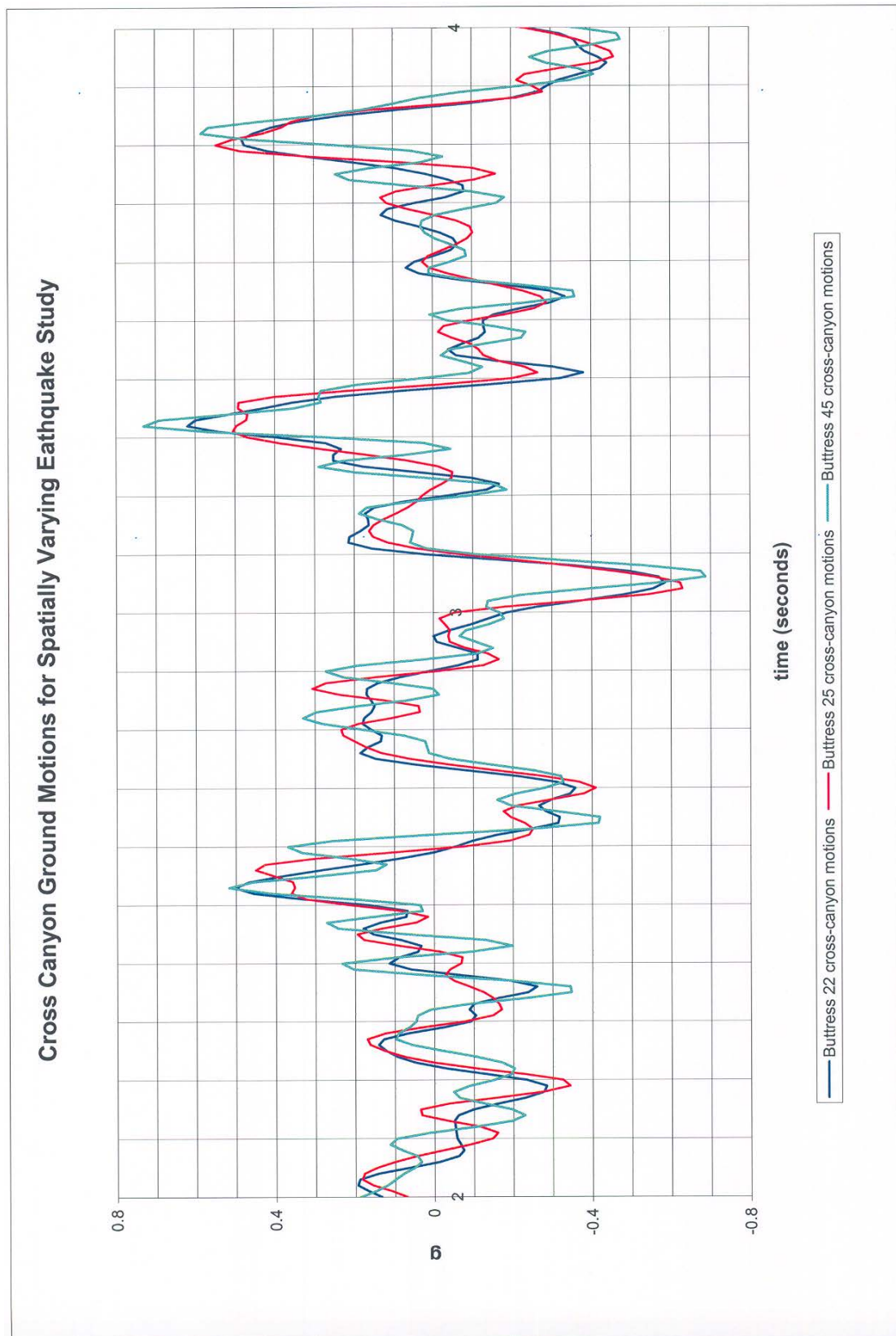


Figure 2.9 Spatially-varying upstream-downstream ground motions and response spectra buttress 22 (left abutment), 25 (near spillway), and 45 (right abutment)—2-second interval,

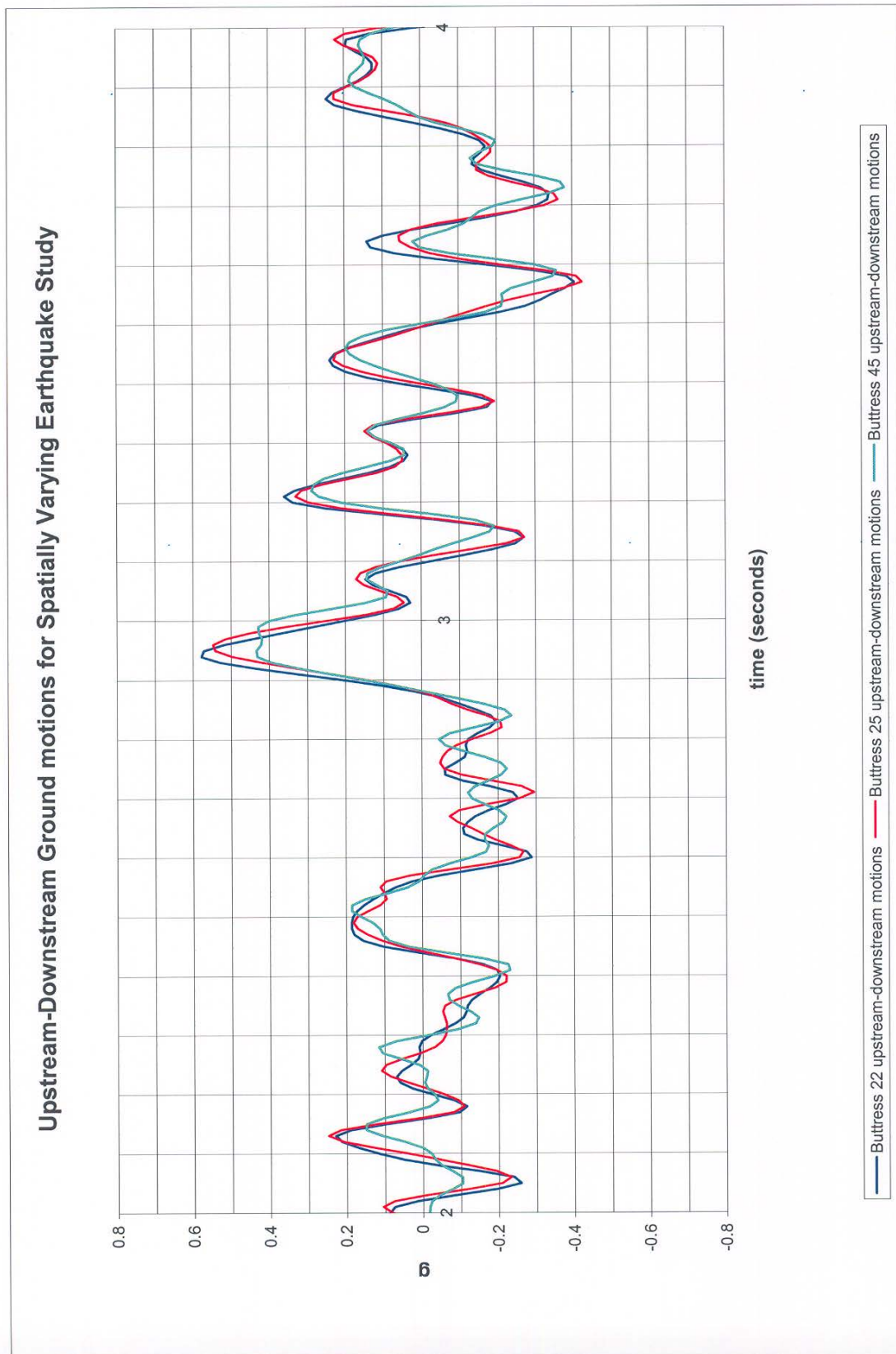


Figure 2.10 Spatially-varying upstream-downstream ground motions and response spectra buttress 22 (left abutment), 25 (near spillway), and 45 (right abutment)—2-second interval,

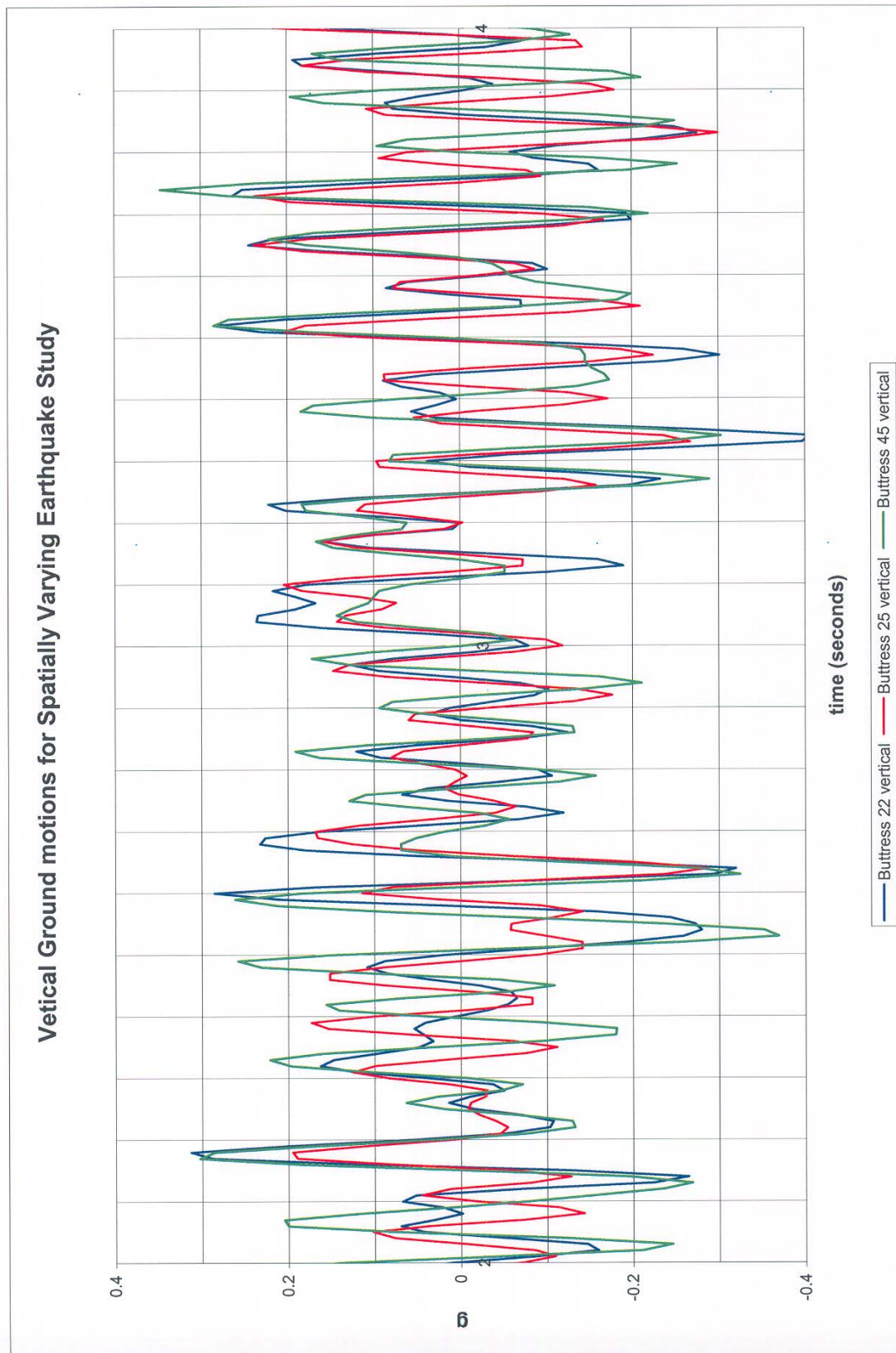


Figure 2.11 Spatially-varying upstream-downstream ground motions and response spectra buttress 22 (left abutment), 25 (near spillway), and 45 (right abutment)—2-second interval,

The values of acceleration spectral intensity (ASI), a measure of the energy in the ground motions between periods of 0.1 to 0.5 s for each motion, are shown in table 1, following. It can be seen that the spatially-varying cross-canyon motions generally contain more energy than the uniform cross-canyon motions; whereas, the vertical motions contain less energy.

Table 1	Spatially-varying Ground Motions)			Uniform Motions
ASI centimeters per second (cm/s)	Buttress 22 (left abutment)	Buttress 25 (near spillway)	Buttress 45 (right abutment)	
Cross-canyon	621	622	593	578
Upstream-downstream	587	552	525	555
Vertical	237	152	186	264

2.3 Results

Comparisons were made of forces measured in several of the struts and maximum principal tensile stresses measured in four of the buttresses. Buttress 22 is located near the left abutment of the dam and is one of the mid-height buttresses. Buttresses 25 (near the spillway) and 36 (near mid-stream) are two of the tallest buttresses. Buttress 45 is near the right abutment. Figure 2.12, which is a plot of a typical buttress, shows the location of the elements for which maximum principal stresses are plotted. The elements for each buttress are numbered from 1 to 73. Elements of buttress 25 range from 2901 to 2973, while elements of buttress 22 range from 3201 to 3273. Therefore, element 2913 and 3213, for example, are in the same relative location on the buttress. The time history plots of buttress stresses are shown in figures 2.13 to 2.16. Figure 2.17 shows the location of the struts for which forces are plotted. The time history plots of forces are shown in figures 2.18 to 2.20.

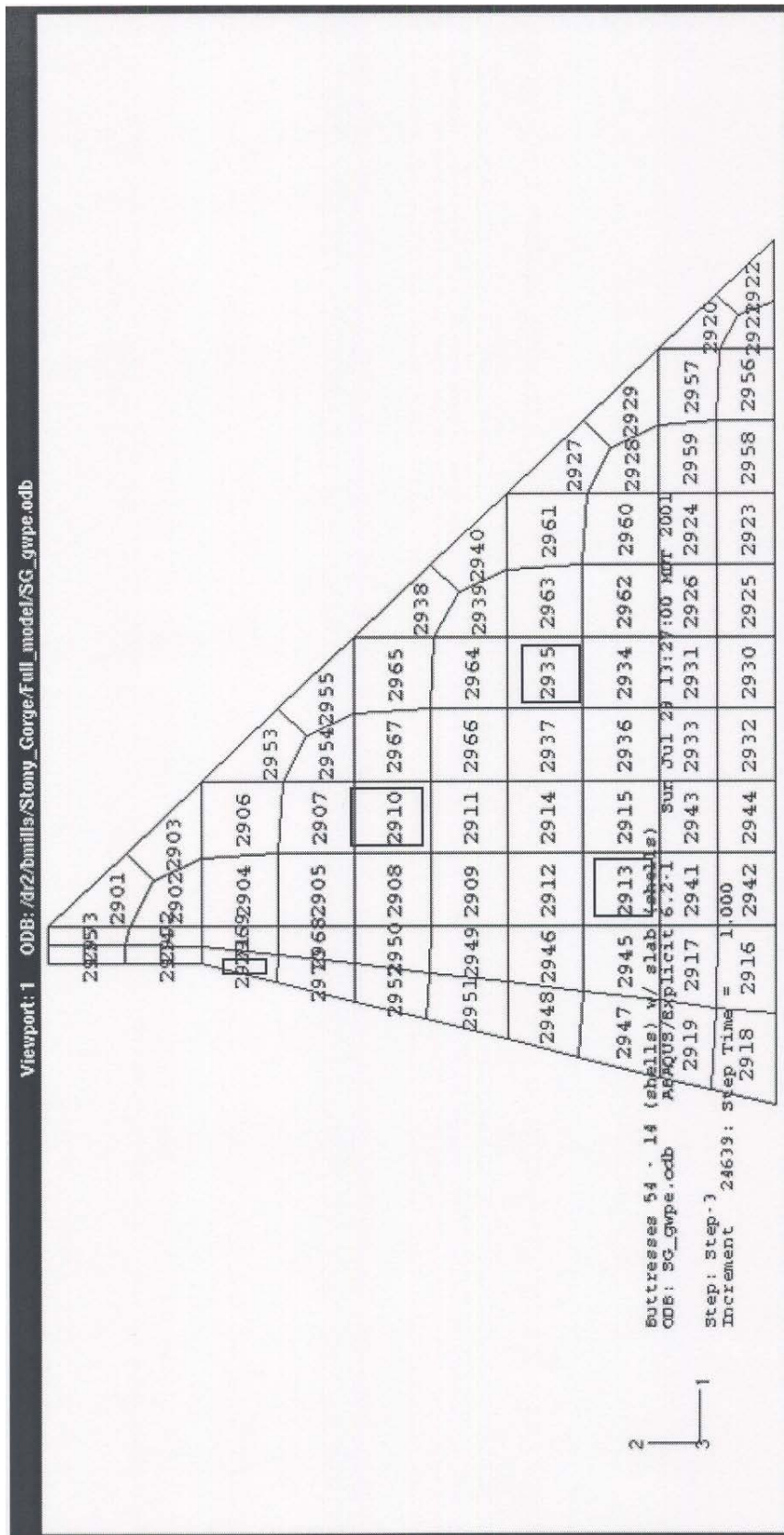


Figure 2.12 Location of elements for which maximum principal stresses are plotted.

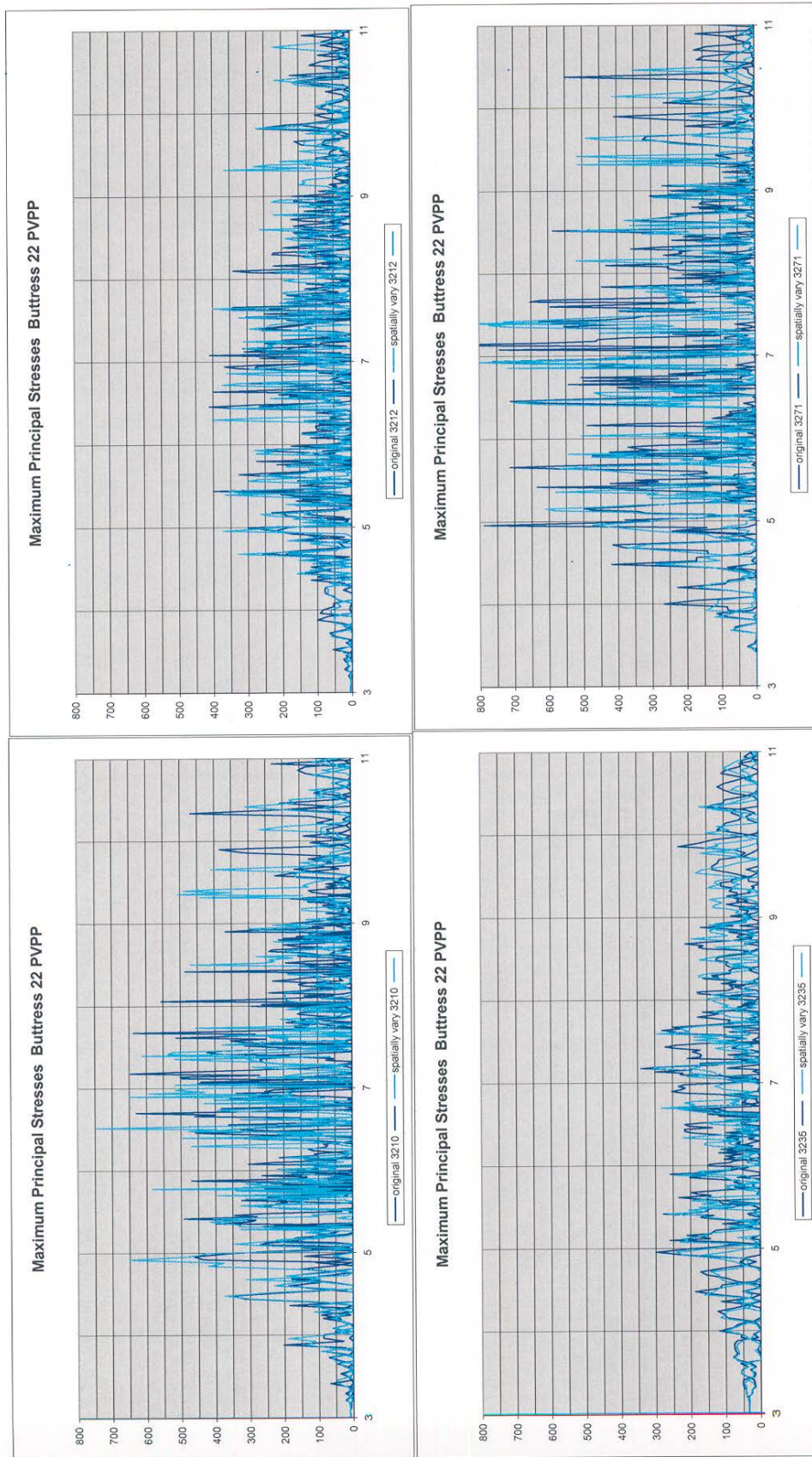


Figure 2.13 Time history of stresses—butter 22.

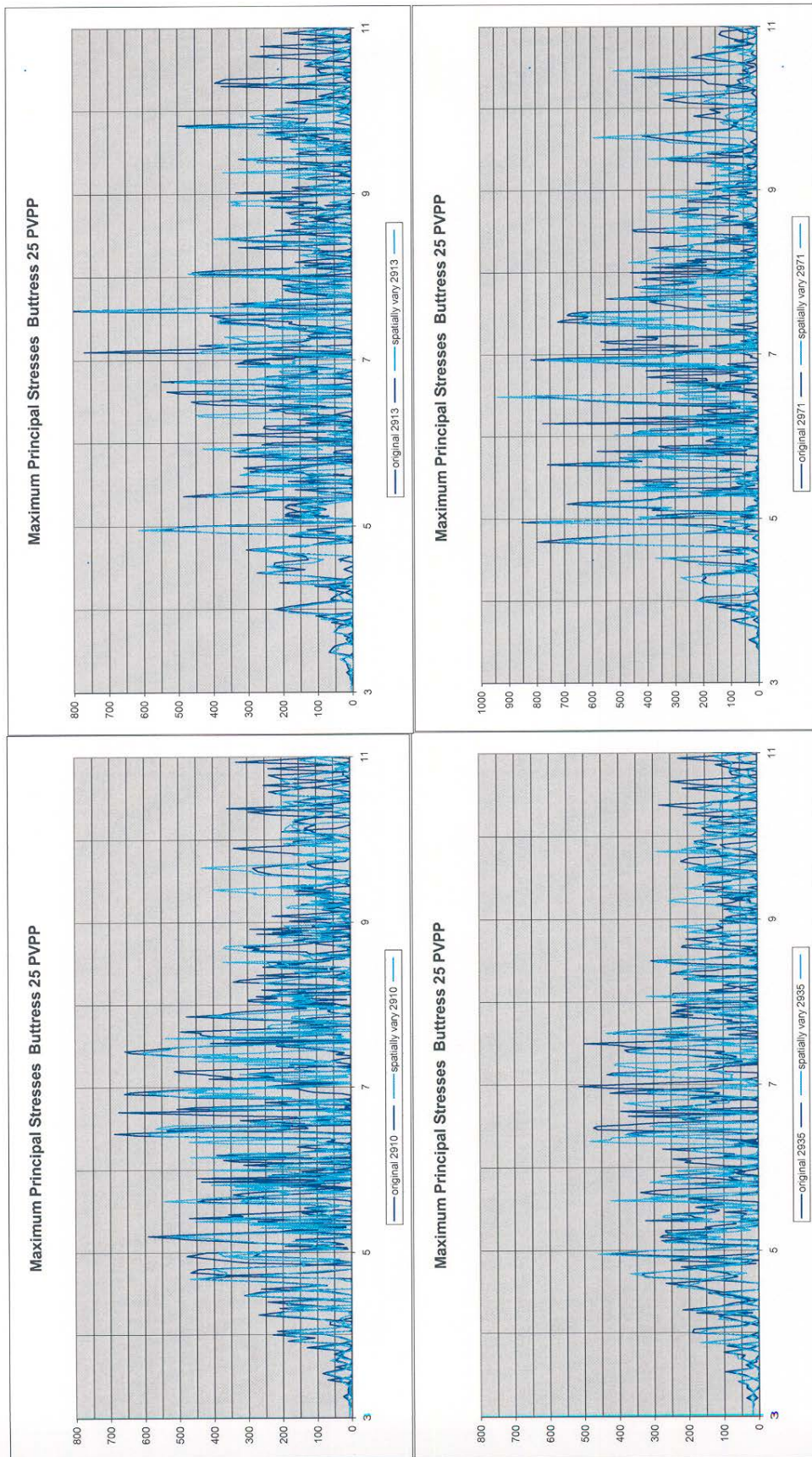


Figure 2.14 Time history of stresses—butter 25.

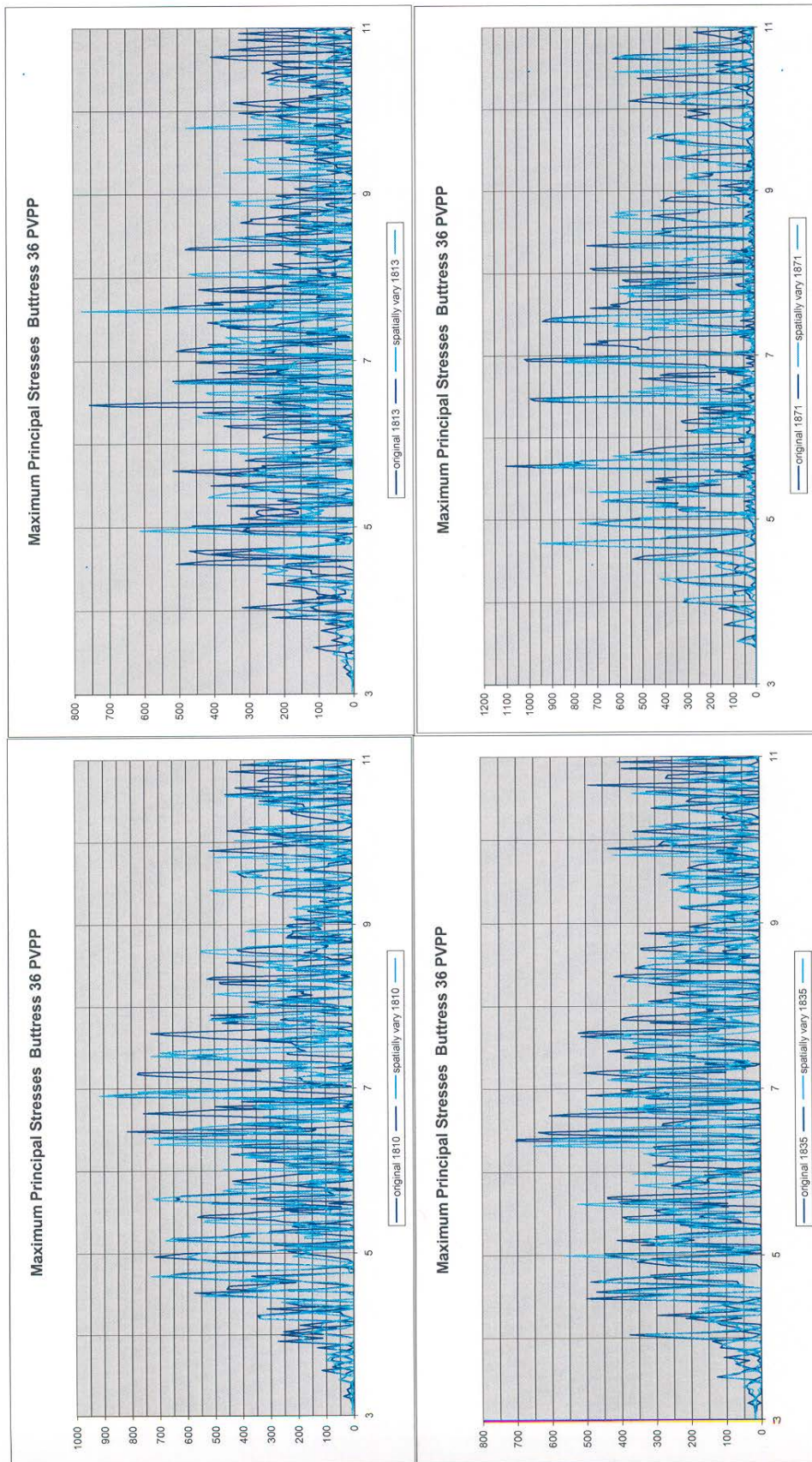


Figure 2.15 Time history of stresses—butter 36.

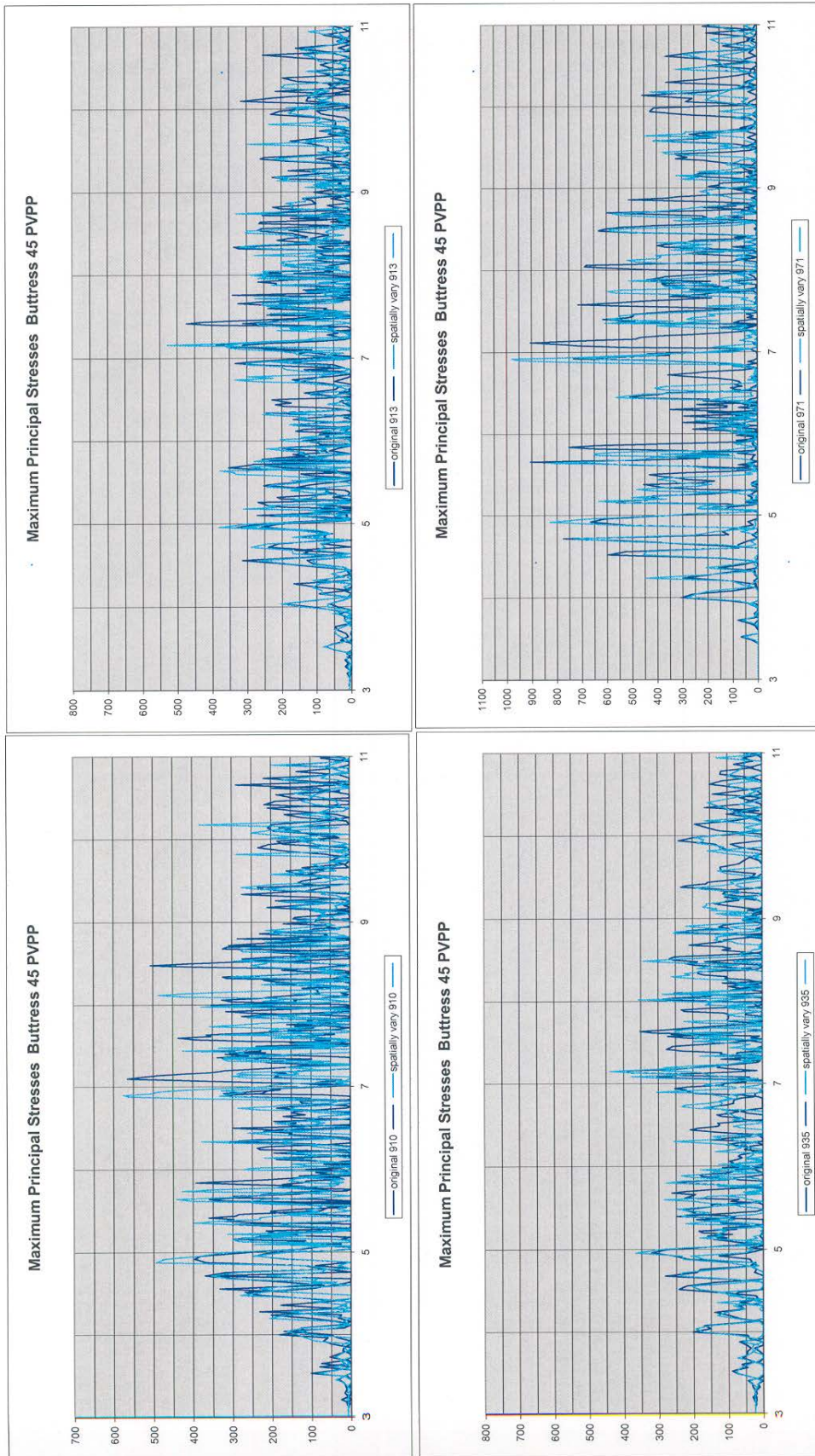


Figure 2.16 Time history of stresses—butter 45.

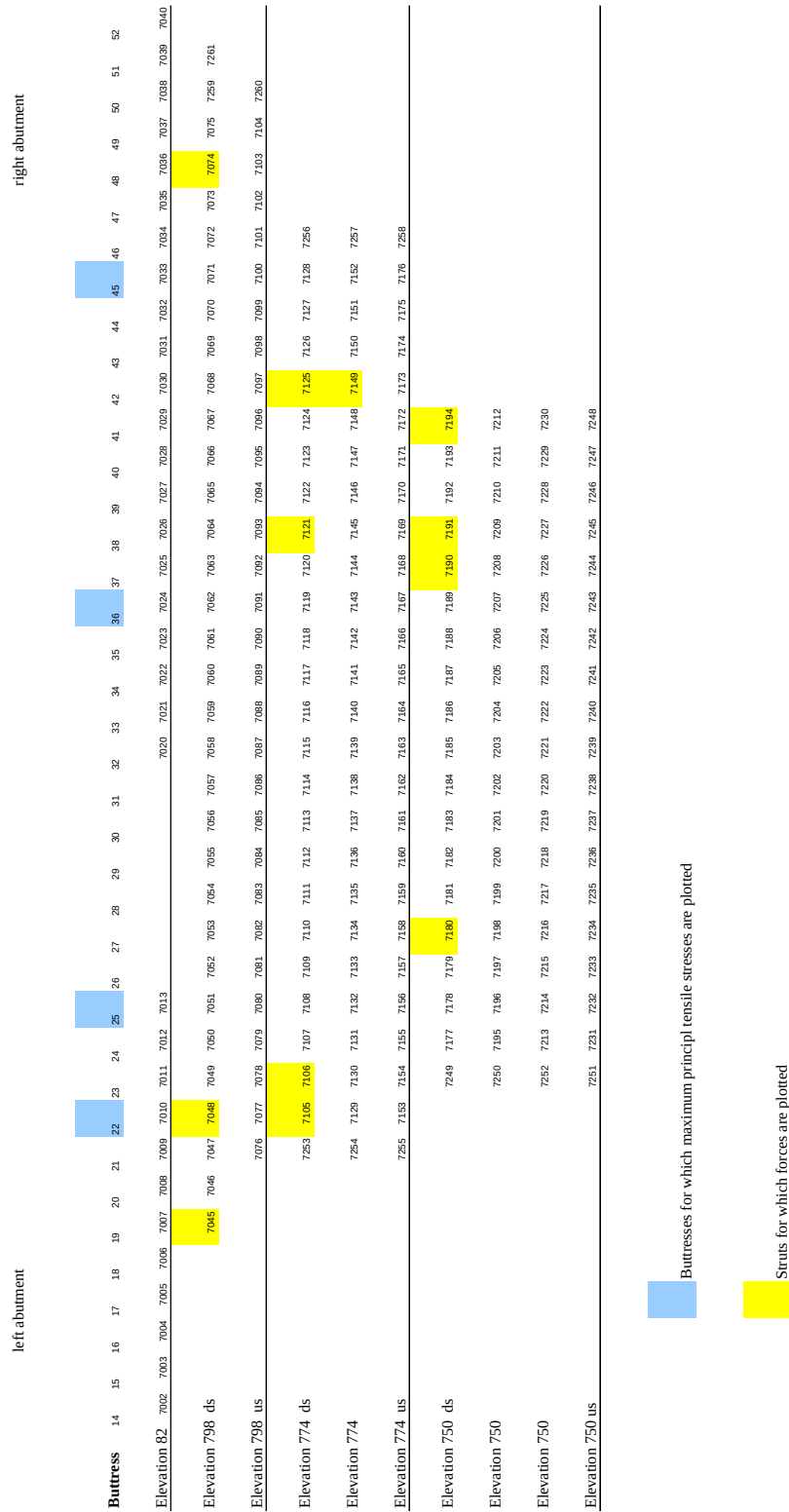


Figure 2.17 Location of Struts and Buttresses for which Forces are Plotted

Figure 2.17 Location of struts and buttresses for which forces are plotted.

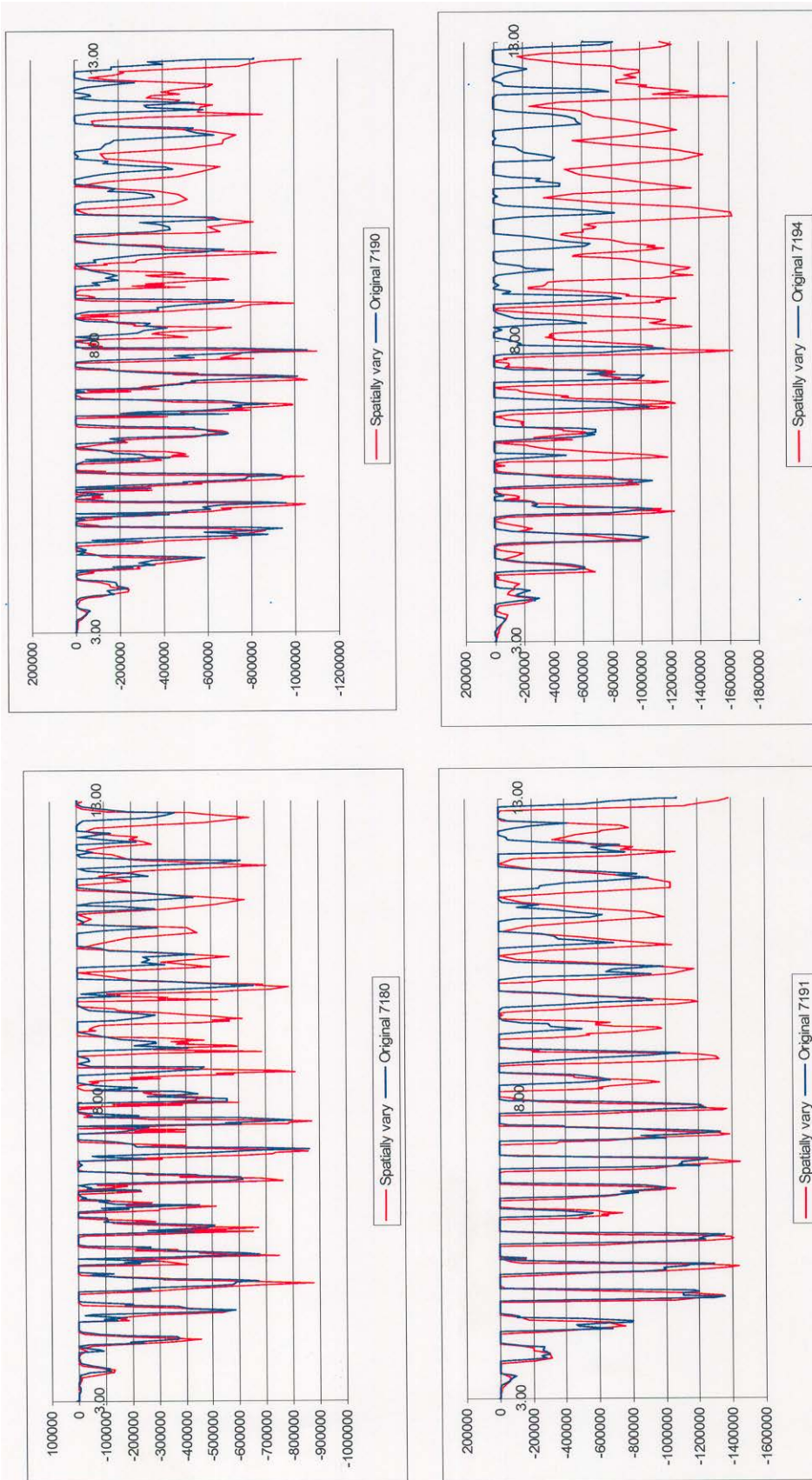


Figure 2.18 Forces in selected struts: bottom row and near downstream face.

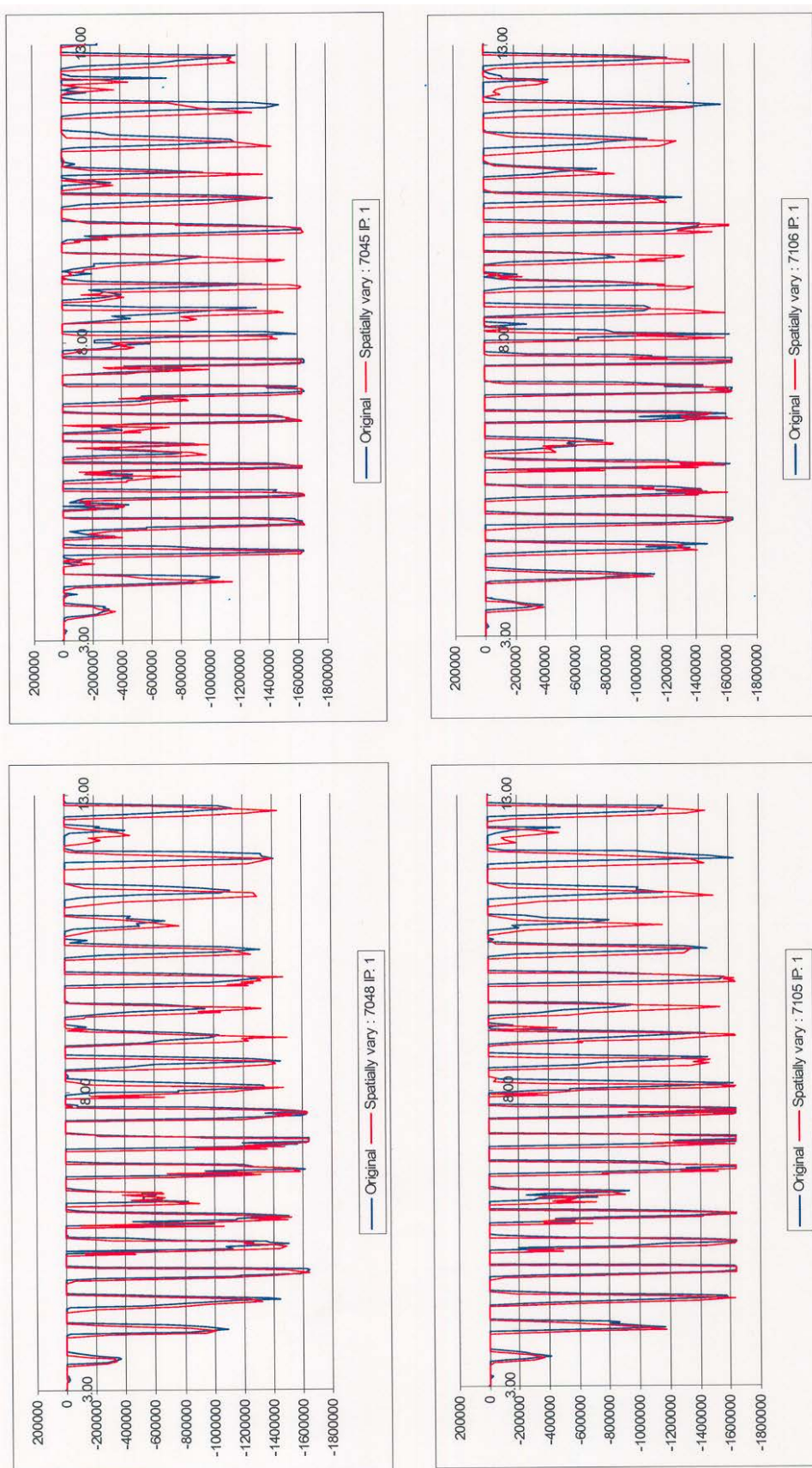


Figure 2.19 Forces in selected struts: near top of dam, left abutment.

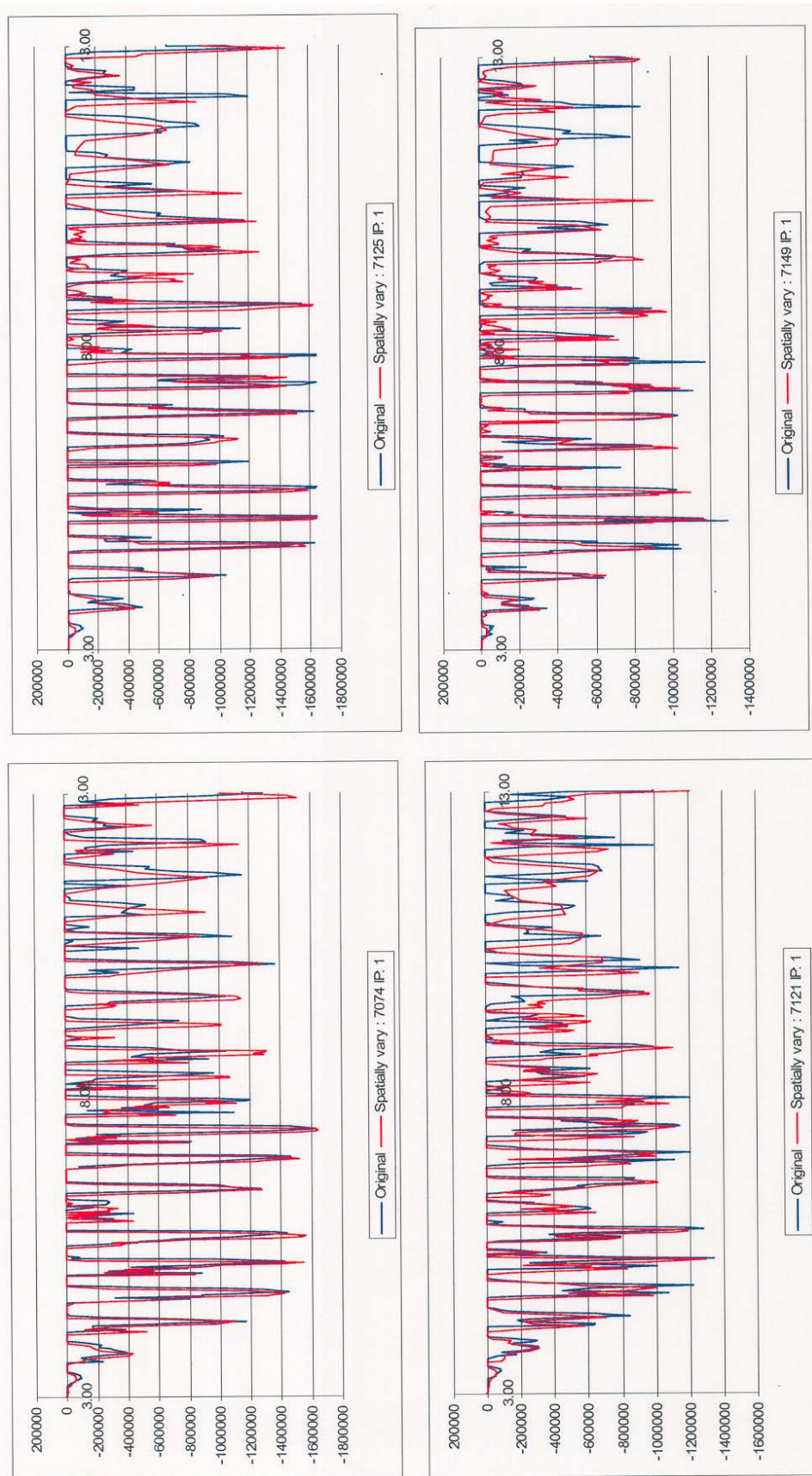


Figure 2.20 Forces in selected struts: near top of dam, right abutment.

The values of the buttress maximum principal tensile stresses in response to the spatially-varying ground motions remained close to the values of the stresses experienced under the uniform ground motions. High stresses occurred at different times due to the phase shift between the sets of spatially-varying motions. Stresses in elements 3210 and 3271 (buttress 22, mid-height and near the change in slope) and in 910 and 971 (buttress 45, mid-height and near the neck) reach values above 400 lb/in^2 , which indicates cracking could occur in either case. All of the elements for which maximum principal tensile stresses are plotted for buttresses 25 and 36 experience stresses high enough to cause cracking. The stresses are higher for elements near the top of the dam and near the downstream face and are higher overall in buttress 36 which is located near midstream, away from the spillway.

The impact of the spatially-varying ground motions is seen in the forces experienced by the struts. Figures 2.18 to 2.20 are plots of the forces in selected struts. For struts near the top of the dam (figures 2.19 and 2.20), the values remain close to the values calculated under the uniform ground motions. However, for struts in the bottom row and near the downstream face (figure 2.18), the values are consistently higher when the spatially-varying motions were input by as much as 400,000 pounds. This is especially true later in the record. The seismotectonic report (on which the spatially-varying ground motions are based) indicated that cross-canyon compressional strains could range up to about 0.007 during this ground motion, with the largest expected strains located beneath the spillway and toward the left abutment. This is reflected in the forces experienced in the lowest level of struts (figure 2.18). The finite element model, consisting of non-linear (compression-only) springs and multiple-point constraints which prevent shear restraint along the slab/buttress interface is flexible enough to accommodate these strains in the buttresses above the bottom row of struts.

The analysis using uniform earthquake motions indicated that many of the struts above the lower level would experience forces greater than the compressive capacity of the struts and would fail. The analysis using the spatially-varying earthquake indicates that many of the lower struts would also fail in an earthquake of that magnitude. However, the cracking indicated in the buttresses during the spatially-varying ground motions remains the same as in the analysis using the uniform earthquake.

3.0 Foundation Studies

Several finite element models were analyzed to determine the effect of foundation size on the propagation of the motions. The analyses varying the size of the foundation led to criteria for width and depth of the foundation. Once criteria were established for foundation size, the mesh density was varied to determine appropriate element size. The appropriate element size is dependent upon the wavelength of the shear waves traveling up through the foundation from the point of application of the input motions. The goal is to reduce the number of elements (finite element models with fewer, and therefore, larger elements have faster run times) without adversely affecting the impact of the ground motions at the surface of the model.

In the past, the Seismotectonic Group has supplied surface motions which were used as input in models which had massless foundations. Input motions for the current models in which mass is included in the foundation must be deconvolved to allow for magnification of the acceleration amplitudes and changes in frequency content as the motions travel up through the foundation. The process of deconvolving the ground motions is dependent upon the material properties and depth of the applied ground motions. In addition, if free-field motions are supplied, the adjustments to the foundation (and element) size should be made based on a foundation with a flat upper surface (i.e., no topography added). If the ground motions supplied are site specific (i.e., measured in a canyon) the foundation model should include the topography of the site.

3.1 Foundation – free-field motions

Seven level surface foundation models were studied to determine the effect of horizontal dimensions and depth on the propagation of ground motions. Deconvolved ground motions were applied on a horizontal plane 100-feet above the base of the finite element model. These motions and their response spectra at 5-percent damping are plotted in figures 3.1-1a (acceleration), -1b (velocity), and -1c (displacement). Plus and minus peak accelerations are shown in table 2.

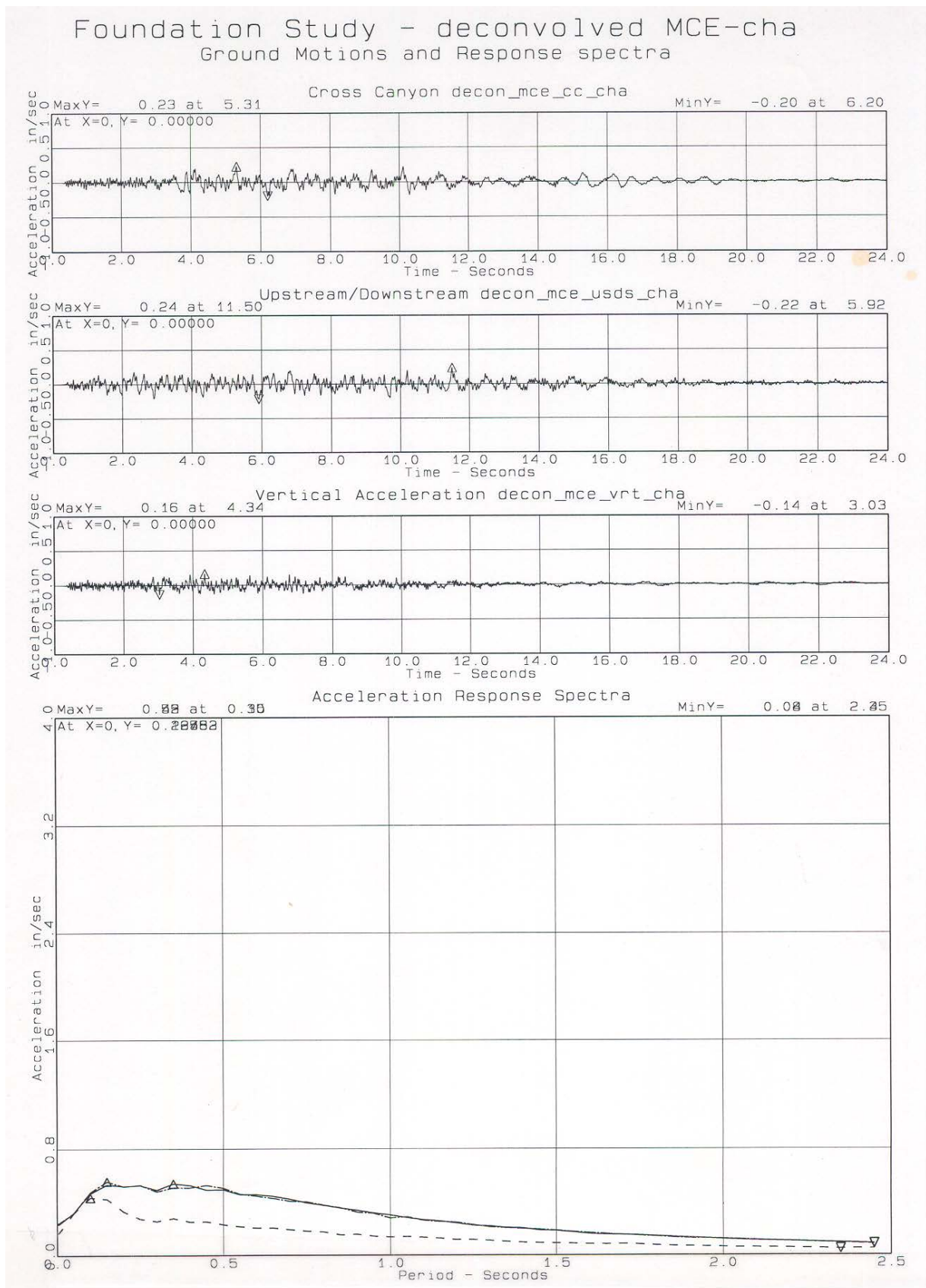


Figure e.3.1-1a Time history accelerations and response spectra for input ground motions.

Foundation Study - decon-MCE-cha

Ground Motions and Response spectra

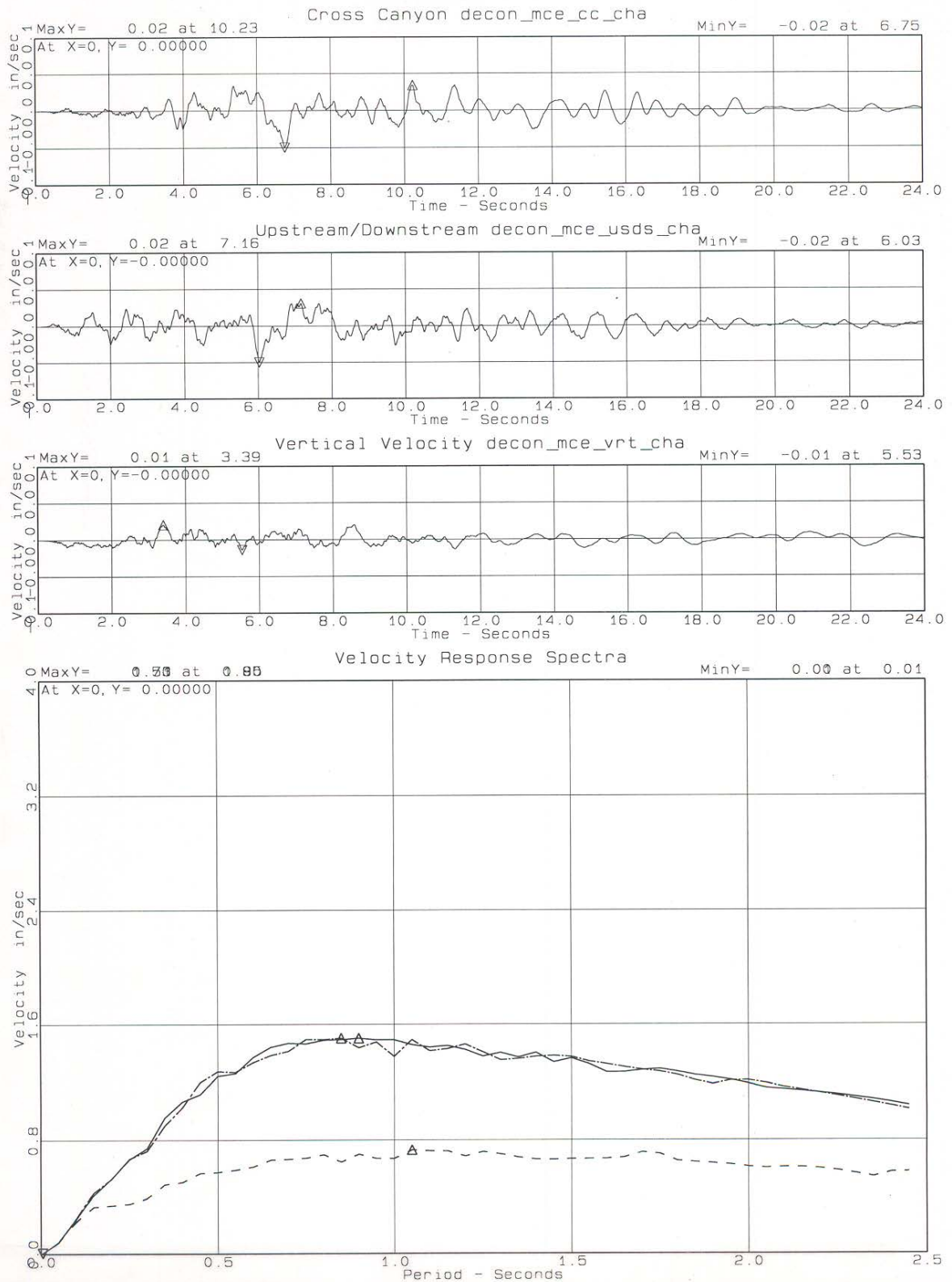


Figure 3.1-1b Time history velocities and response spectra for input ground motions.

Foundation Study - decon-MCE-cha

Ground Motions and Response spectra

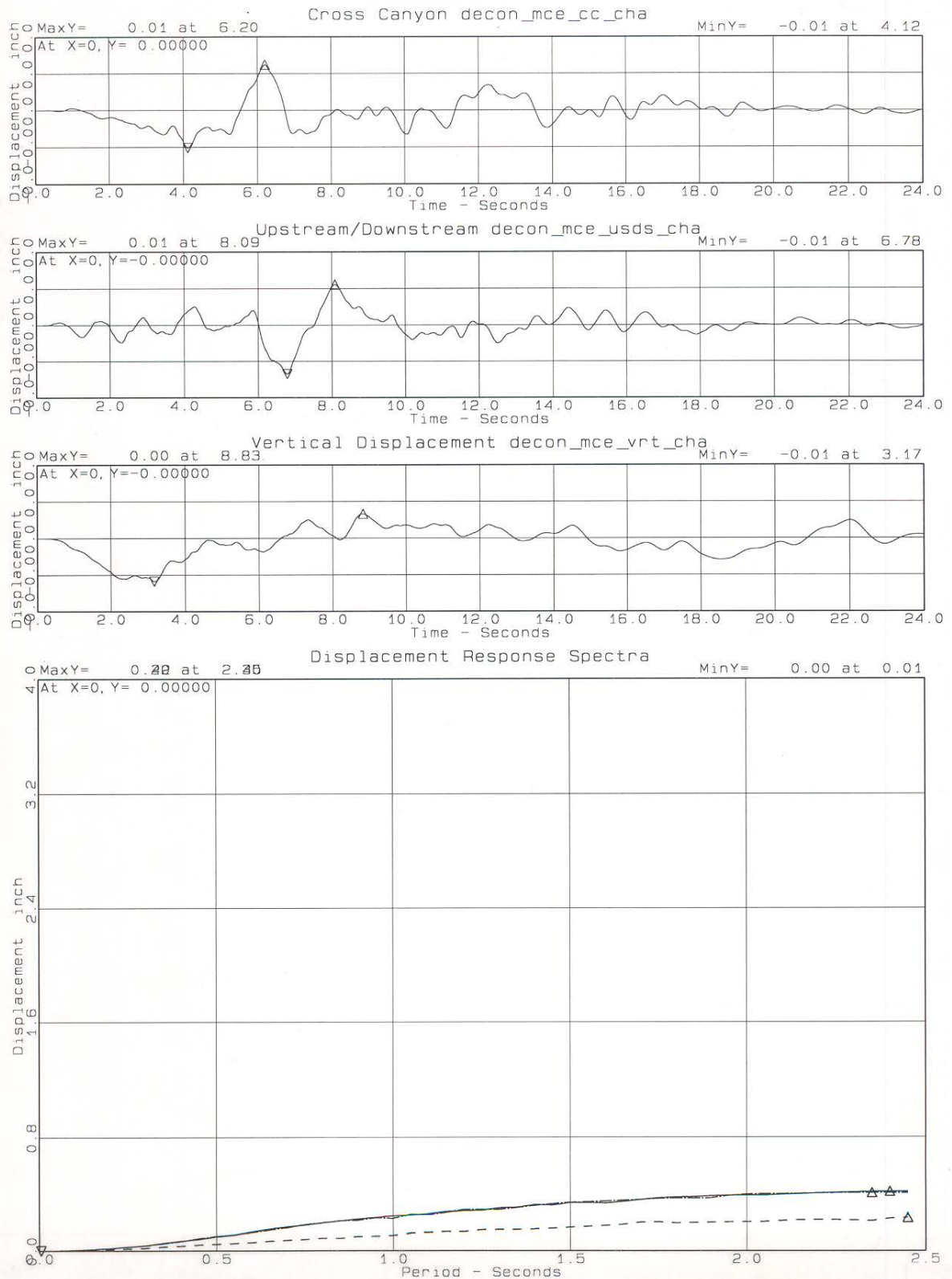


Figure 3.1-1c Time history displacements and response spectra for input ground motions.

Table 2	Peak Accelerations (g)		
Deconvolved Ground Motions (input motions)	Cross-Canyon	Upstream- Downstream	Vertical
	-0.23	-0.24	-0.16
	-0.20	-0.22	-0.14

A Rayleigh damping value of $\alpha = 3.25$ and $\beta = 0.0$ was applied during the first several analyses. A plot of percent damping versus frequency for this value of α is shown in figure 3.1-2.

Consideration of the plot in figure 3.1-2 shows that for this amount of mass damping, a frequency of 1 Hz is damped 26 percent, 2 Hz is damped 13 percent, and 10 Hz is damped 2.6 percent. It is apparent that very low frequencies are highly damped for this value of α .

Calculations for Rayleigh Damping Coefficients

Input % Critical Damping for The 2 Frequencies You Want in the Yellow Areas:					
	cps	Rad/sec		Percent	Decimal
Uncracked					
Freq 1:	6.000	37.699	% Damp 1:	10.00	0.100
Freq 2:	15.000	94.248	% Damp 2:	10.00	0.100
Solve simultaneous equations:					
	Alpha +	w ² *	Beta =	2*w*Epsilon	
	Alpha +	1421.22	Beta =	7.5398224	
	Alpha +	8882.64	Beta =	18.849556	
Uncracked					
Beta =	0.000000	(stiffness)	Freq Increm	1	
Alpha =	3.250000	(mass)			
Freq vs % Damping	Stiffness Damping	Mass Damping			
0.50	51.73	0.00	51.73		
1.00	25.86	0.00	25.86		
2.00	12.93	0.00	12.93		
3.00	8.62	0.00	8.62		
4.00	6.47	0.00	6.47		
5.00	5.17	0.00	5.17		
6.00	4.31	0.00	4.31		
7.00	3.69	0.00	3.69		
8.00	3.23	0.00	3.23		
9.00	2.87	0.00	2.87		
10.00	2.59	0.00	2.59		
11.00	2.35	0.00	2.35		
12.00	2.16	0.00	2.16		
13.00	1.99	0.00	1.99		
14.00	1.85	0.00	1.85		
15.00	1.72	0.00	1.72		
16.00	1.62	0.00	1.62		
17.00	1.52	0.00	1.52		
18.00	1.44	0.00	1.44		
19.00	1.36	0.00	1.36		

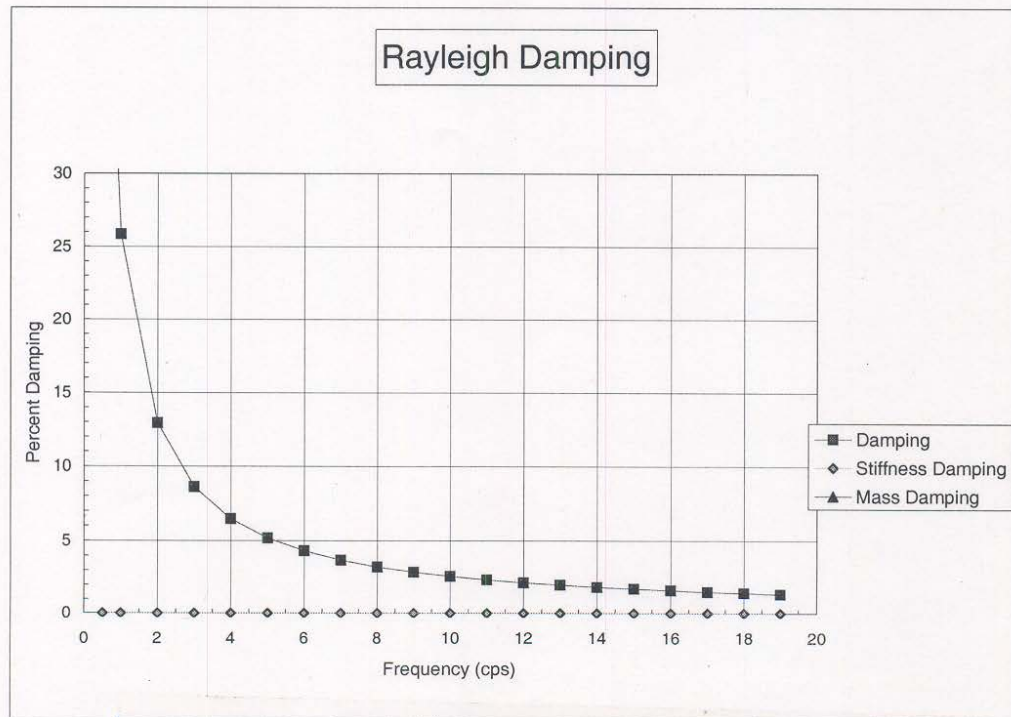


Figure 3.1-2 Rayleigh mass damping for foundation studies.

Finally, an estimate of appropriate finite element size can be made by considering the material properties of the foundation and calculating the shear wave velocity based on these material properties. Given a range of frequencies of interest, in this case frequencies that can be expected to excite concrete gravity or arch dams, a range of wavelengths can be determined. For the propagation of the wave to be modeled accurately, there should be 6 to 10 elements per wavelength. For the foundation material properties in these analyses ($E = 4 \times 10^6 \text{ lb/in}^2$, Poisson's ratio, $\nu = 0.33$, density, $\gamma = 165 \text{ lb/ft}^3$), and for frequencies greater than 2 Hz and less than 20 Hz, the wavelength ranged from 325 to 3,250 feet. In order to ensure that the wave travels through a minimum of 6 elements vertically, the elements would be approximately 50 feet on a side. Figure 3.1-3 shows the spreadsheet used to calculate the wavelength information. The equations represented on this spreadsheet are as follows:

- Shear modulus, $G = E / 2(1+\nu)$
- Shear wave velocity = $[G/\rho]$, where ρ is the mass density ($\rho = \gamma/386.4$)
- Shear Wavelength = shear wave velocity/frequency

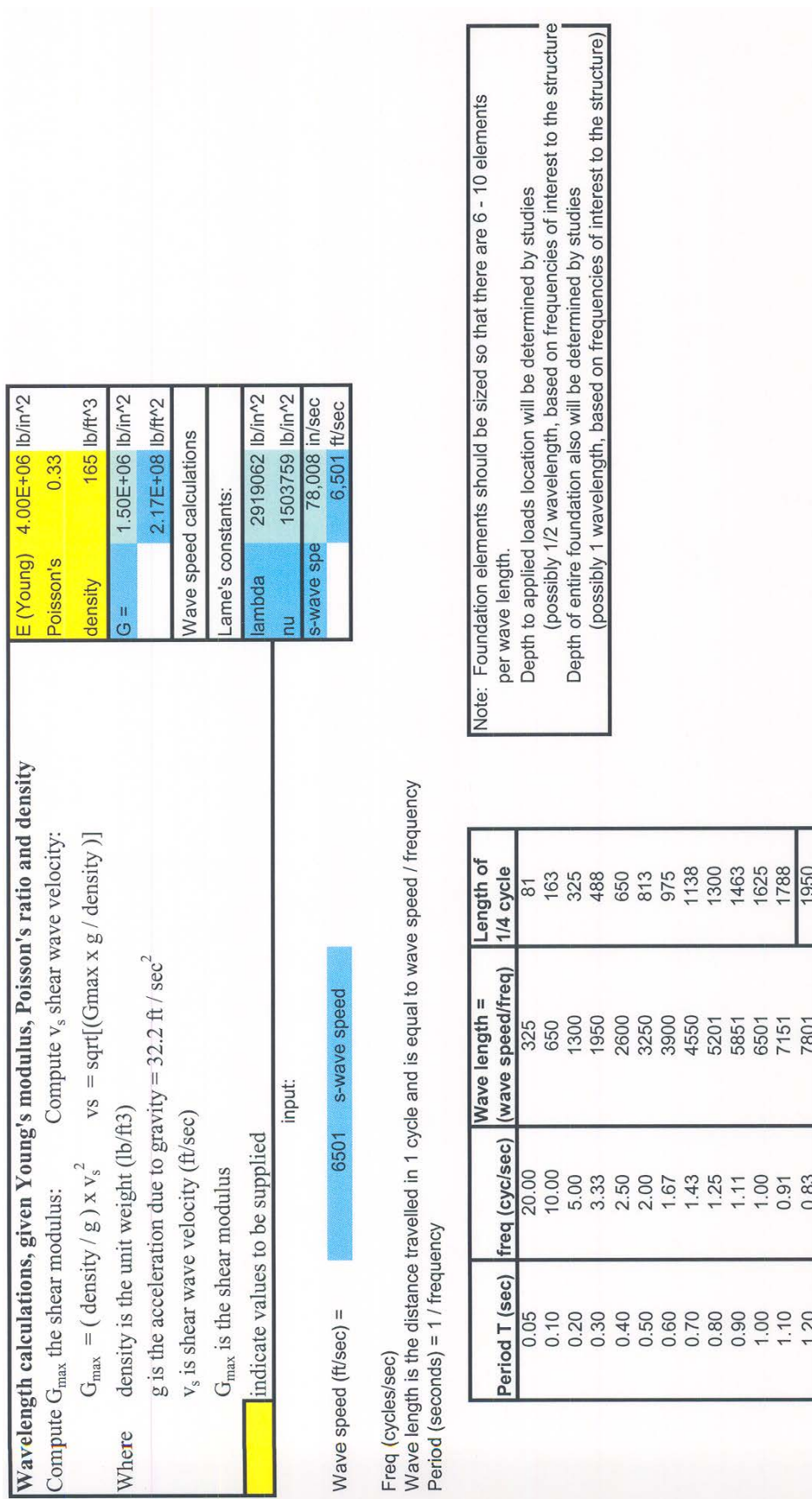


Figure 3.1-3 Calculations of wave speed and wavelength.

3.1.1 Variation in foundation depth

For this study, three foundation models were used. The horizontal dimensions were 600 by 600 feet. The vertical dimensions varied from 600 (from the ground surface down to the plane of ground motion application) to 400 feet and, finally, to 200 feet. Because *DYNA* does not allow prescribed accelerations at nodes that are defined as non-reflecting boundaries, the ground motions are applied on a plane 100-feet above the base of the model. The base of the model extended below the plane of ground motion application 100 feet. Non-reflecting or “transmitting” boundary conditions were applied on the sides and base of the model. These boundary conditions simulate semi-infinite domains and prevent spurious wave reflections due to the finite extent of the mesh. The three models are shown in figures 3.1.1-1, -6, and -11.

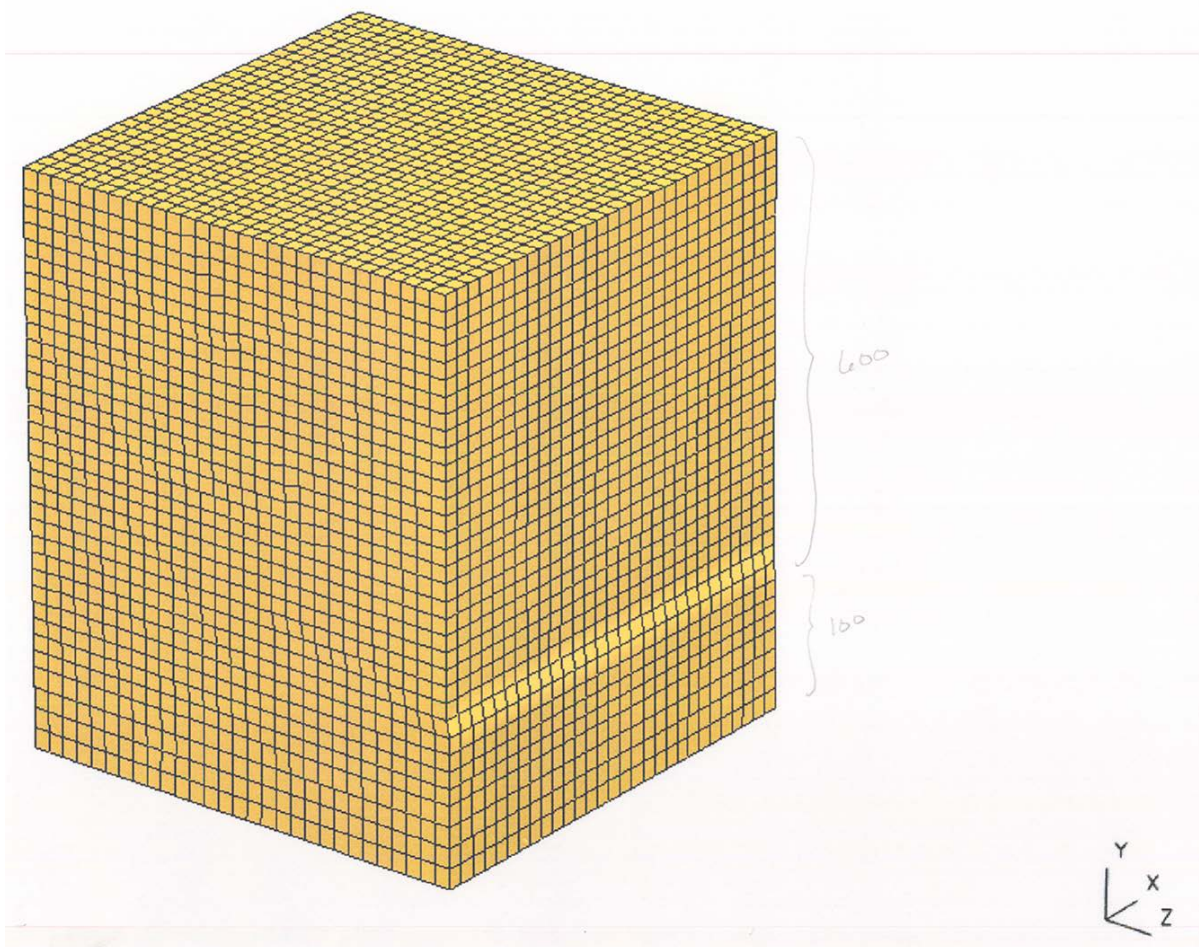


Figure 3.1.1-1 Model for free-field motions (600 by 600 by 600 feet).

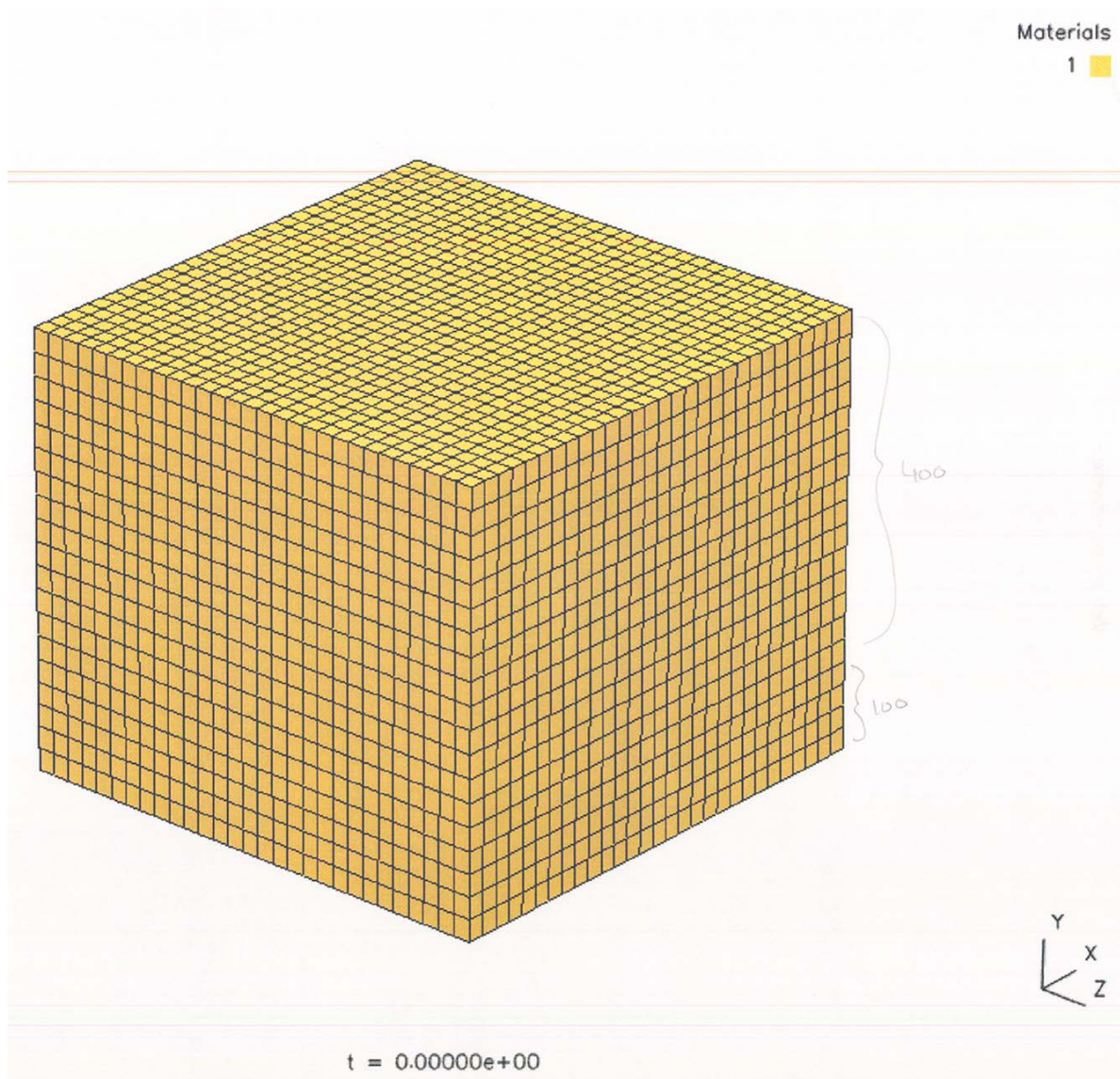


Figure 3.1.1-6 Model for free-field motions (600 by 600 by 400 feet).

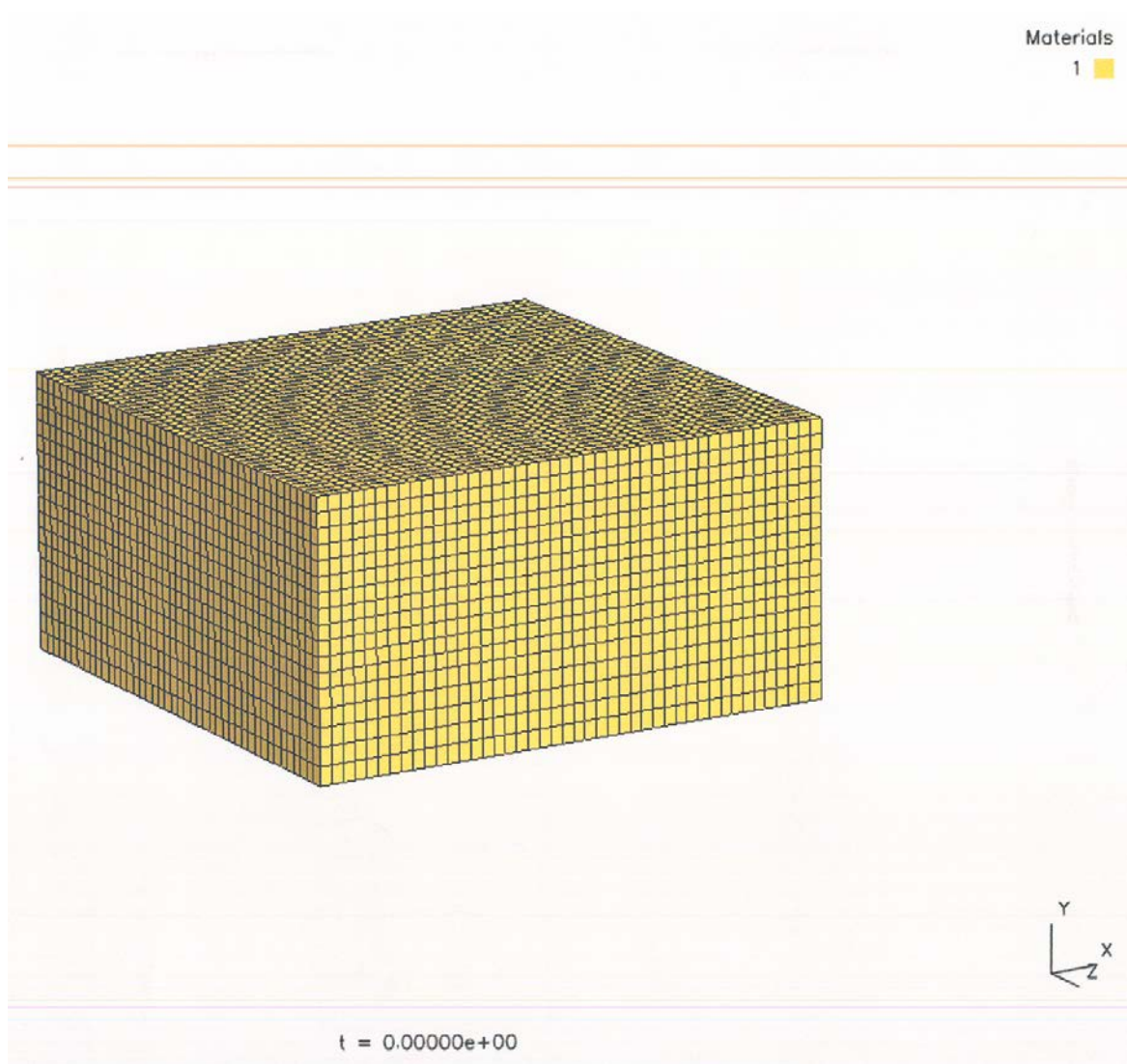


Figure 3.1.1-11 Model for free-field motions (600 by 600 by 200 feet).

For the dynamic analysis, accelerations in all three directions were extracted for a line of nodes along the x-axis at the surface of the model. For the 600- by 600- by 600-foot model, these accelerations are plotted in figures 3.1.1-2 through -4. Similar plots are shown in figures 3.1.1-7 through -9 for the 600- by 600- by 400-foot model and in figures 3.1.1-12 through -14 for the 600- by 600- by 200-foot model. Figures 3.1.1-5, -10, and -15 are time history plots of the accelerations in all three directions at the center of the model surfaces. The acceleration response spectra for these motions are also plotted on these three plots. In order to compare all of this information, the average of the maximum and absolute value of the minimum accelerations for each record was found. These average values are plotted in figure 3.1.1-16. The pale green bar shows the maximum average value of the input acceleration. The blue, red, and yellow bars show the maximum average value for the foundations with a 600-, 400-, and 200-foot depth, respectively.

Spatially Varying EQ -- x-Acc (xaxis) Folsom- decon-mce-cha

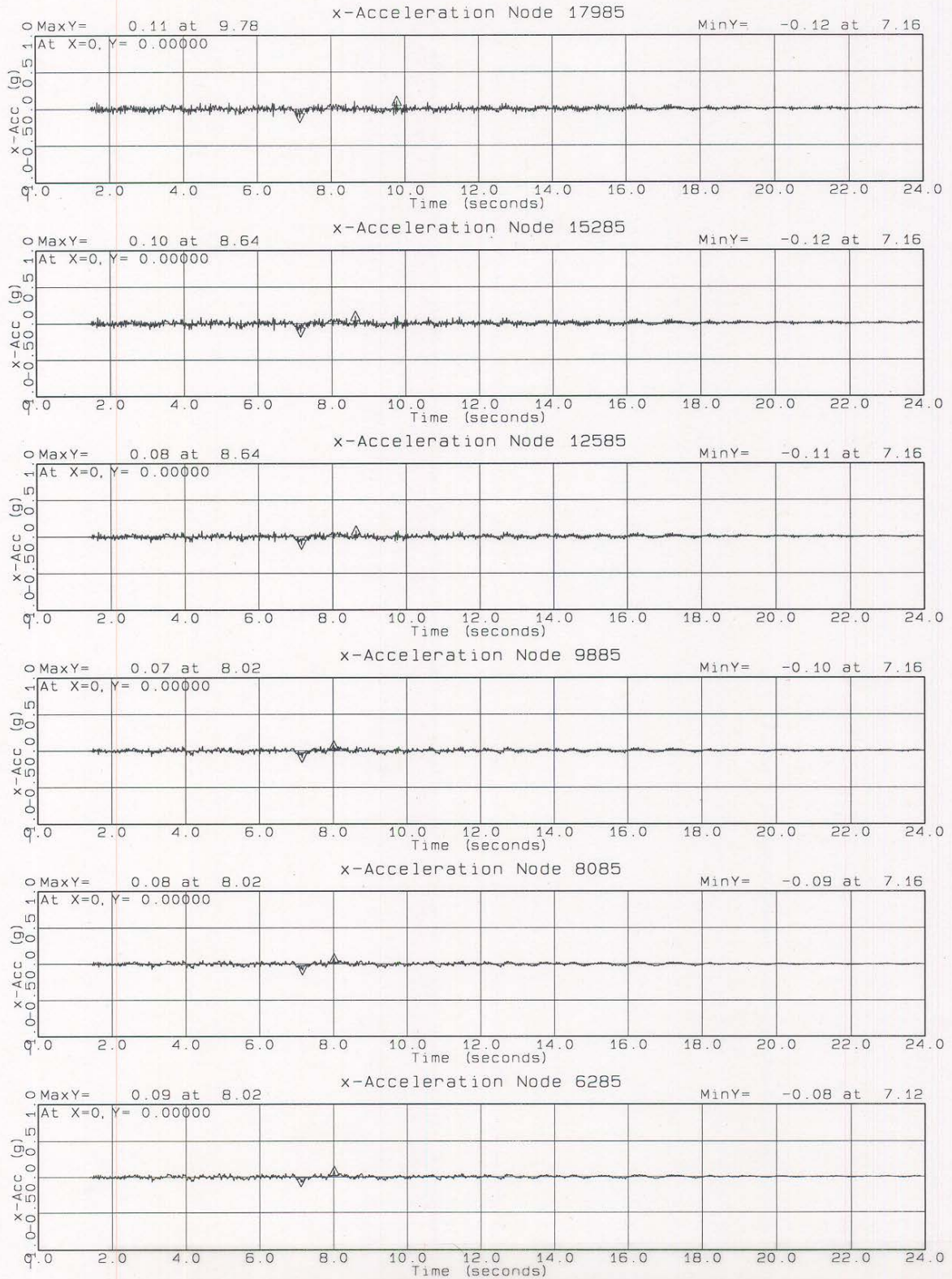


Figure 3.1.1-2 X-direction surface accelerations along the x-axis.

Spatially Varying EQ -- y-Acc (xaxis) Folsom- decon-mce-cha

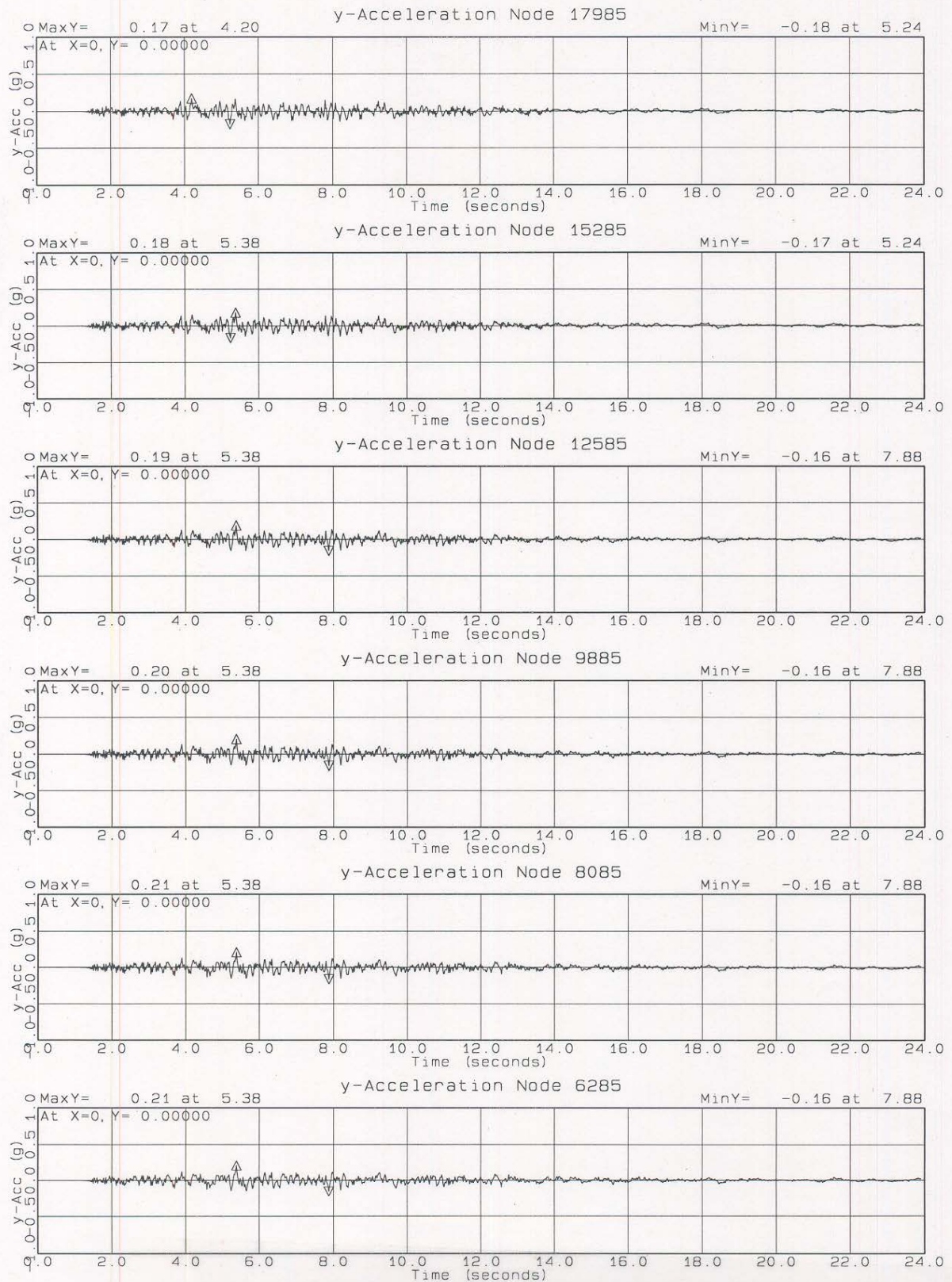


Figure 3.1.1-3 Y-direction surface accelerations along the x-axis.

Spatially Varying EQ -- z-Acc (xaxis) Folsom- decon-mce-cha

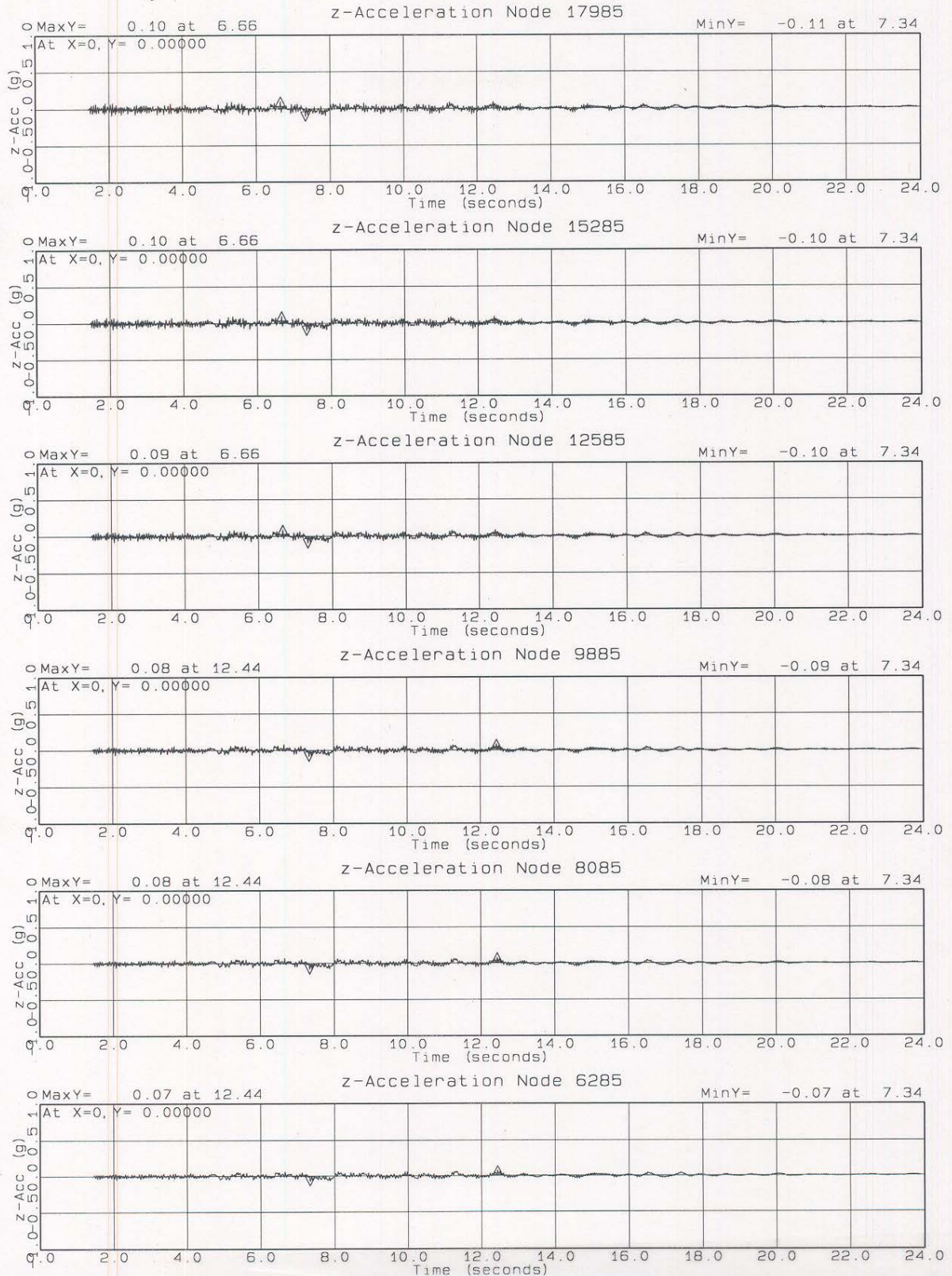


Figure 3.1.1-4 Z-direction surface accelerations along the x-axis.

Spatially Varying EQ -- x-Acc (xaxis) Folsom- decon-mce-cha

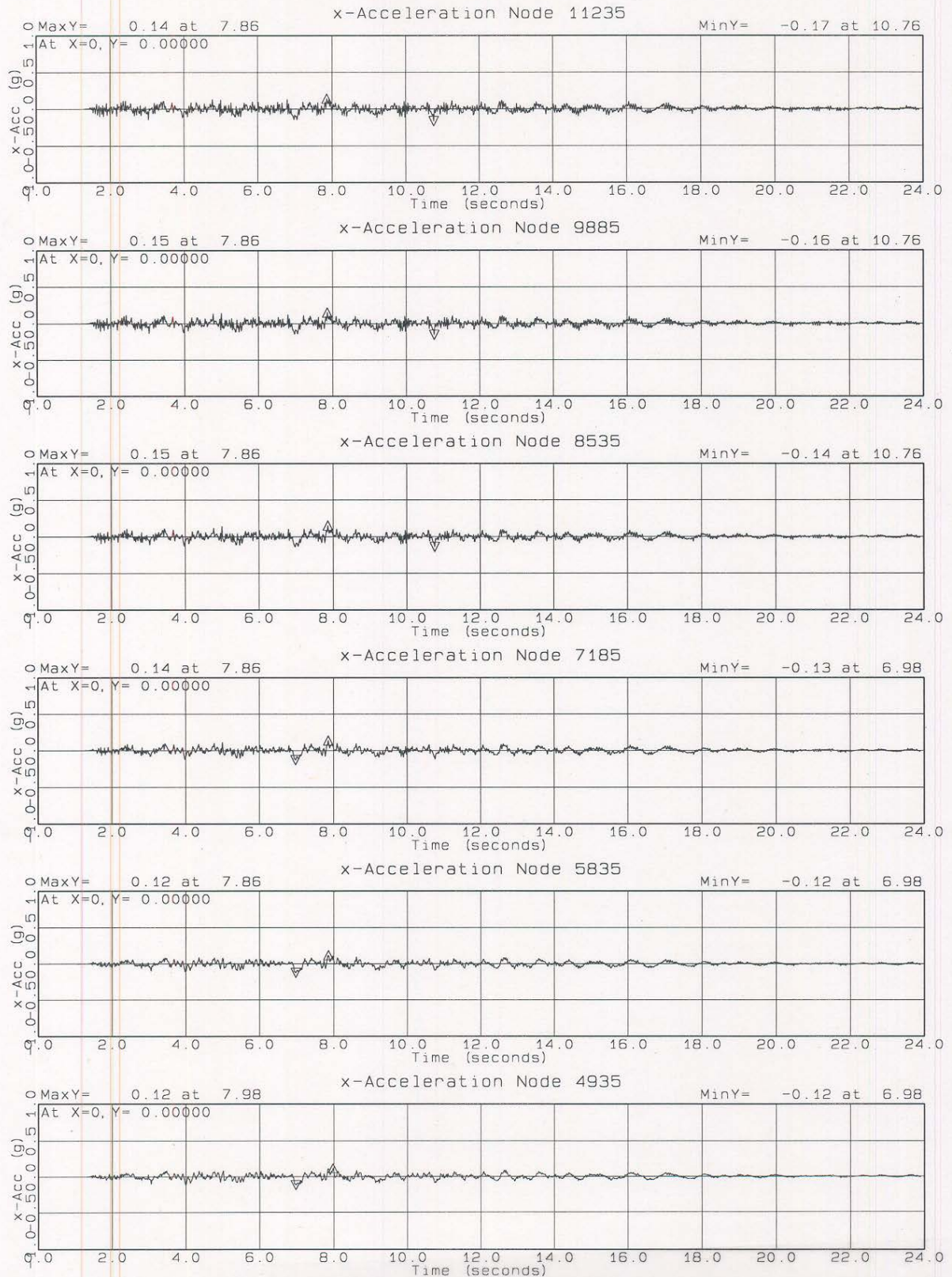


Figure 3.1.1-7 X-direction surface accelerations along the x-axis.

Spatially Varying EQ -- y-Acc (xaxis) Folsom- decon-mce-cha

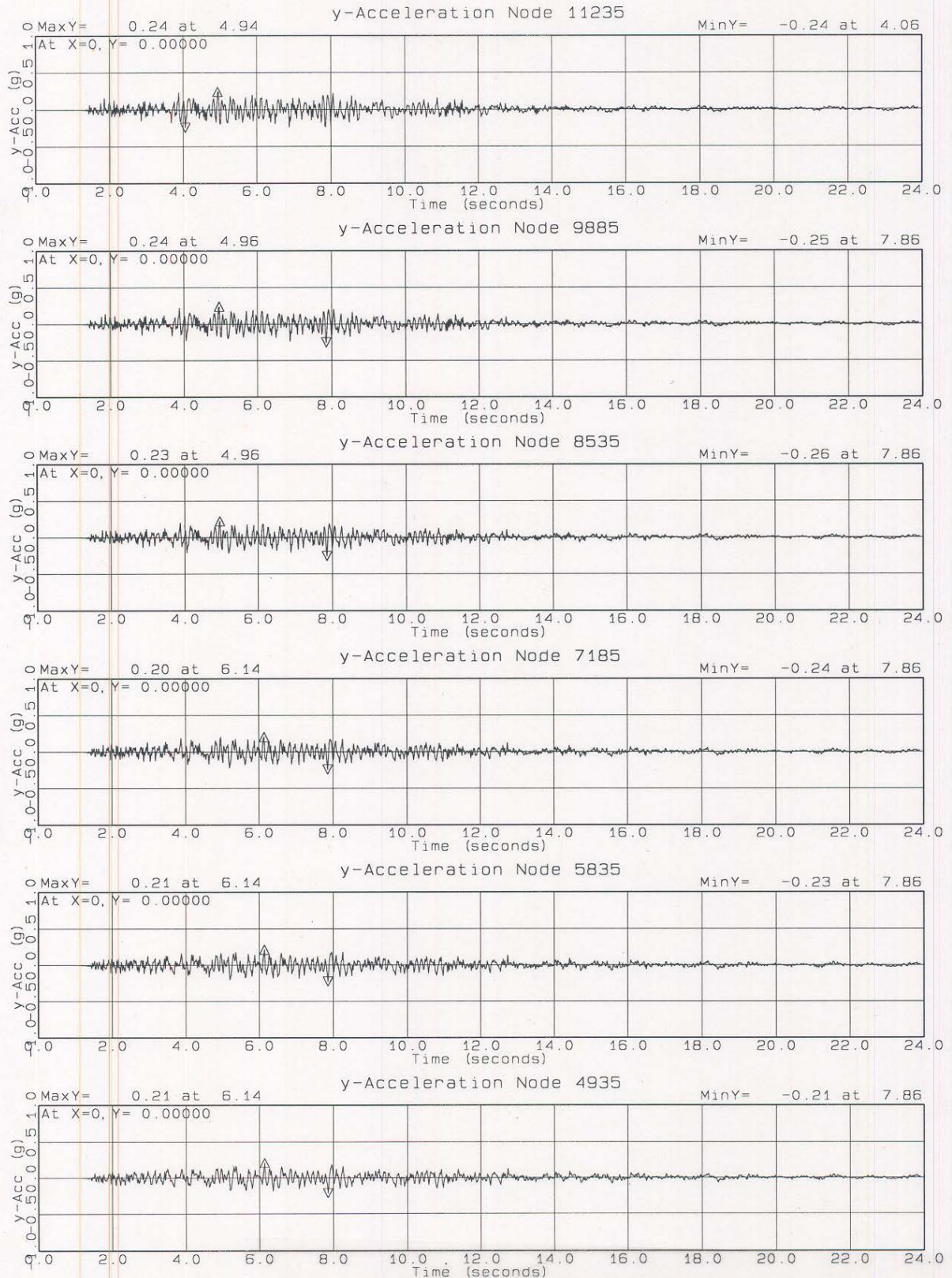


Figure 3.1.1-8 Y-direction surface accelerations along the x-axis.

Spatially Varying EQ -- z-Acc (xaxis) Folsom- decon-mce-cha

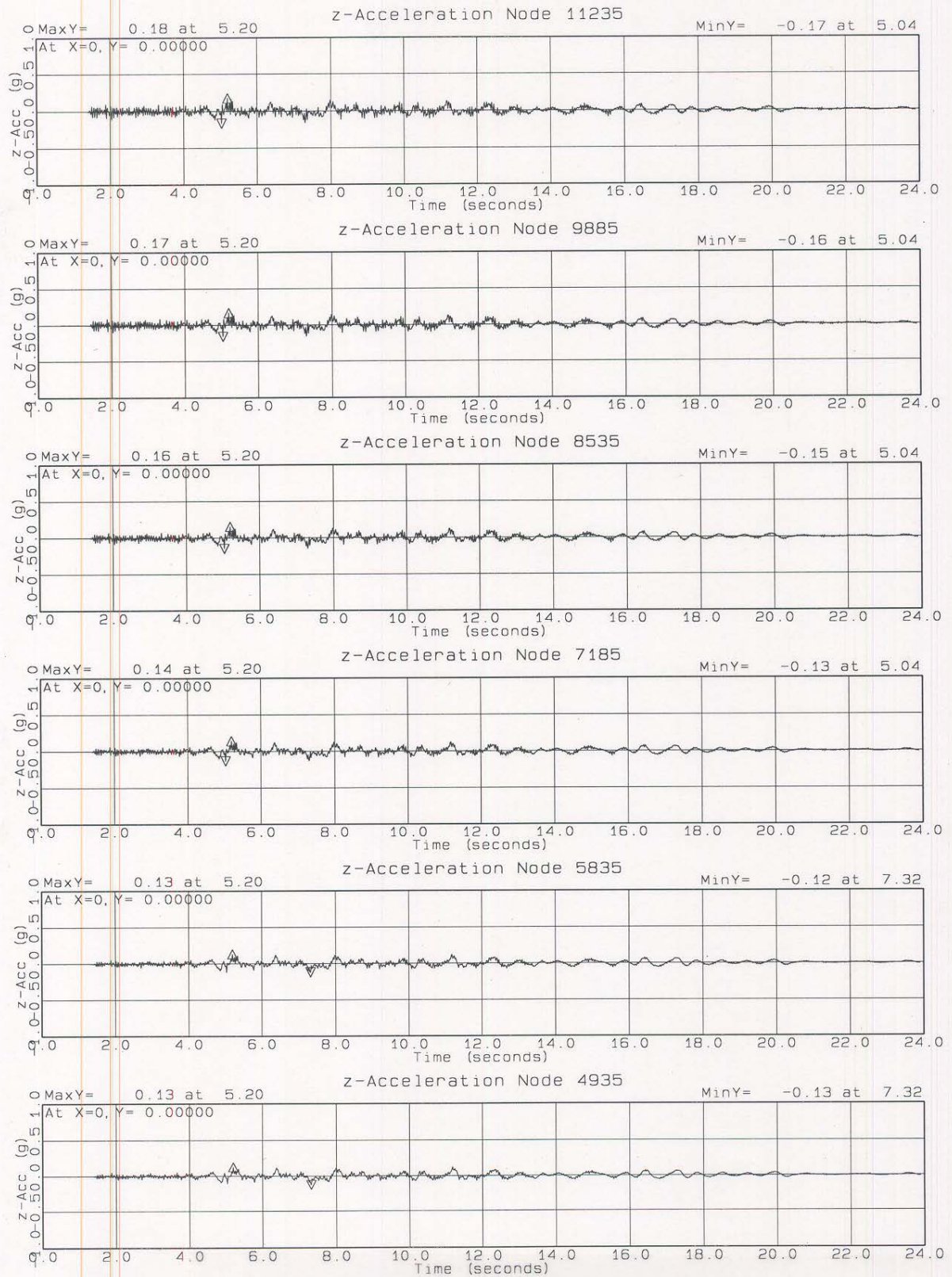


Figure 3.1.1-9 Z-direction surface accelerations along the x-axis.

Spatially Varying EQ -- x-Acc (xaxis) Folsom- decon-mce-cha

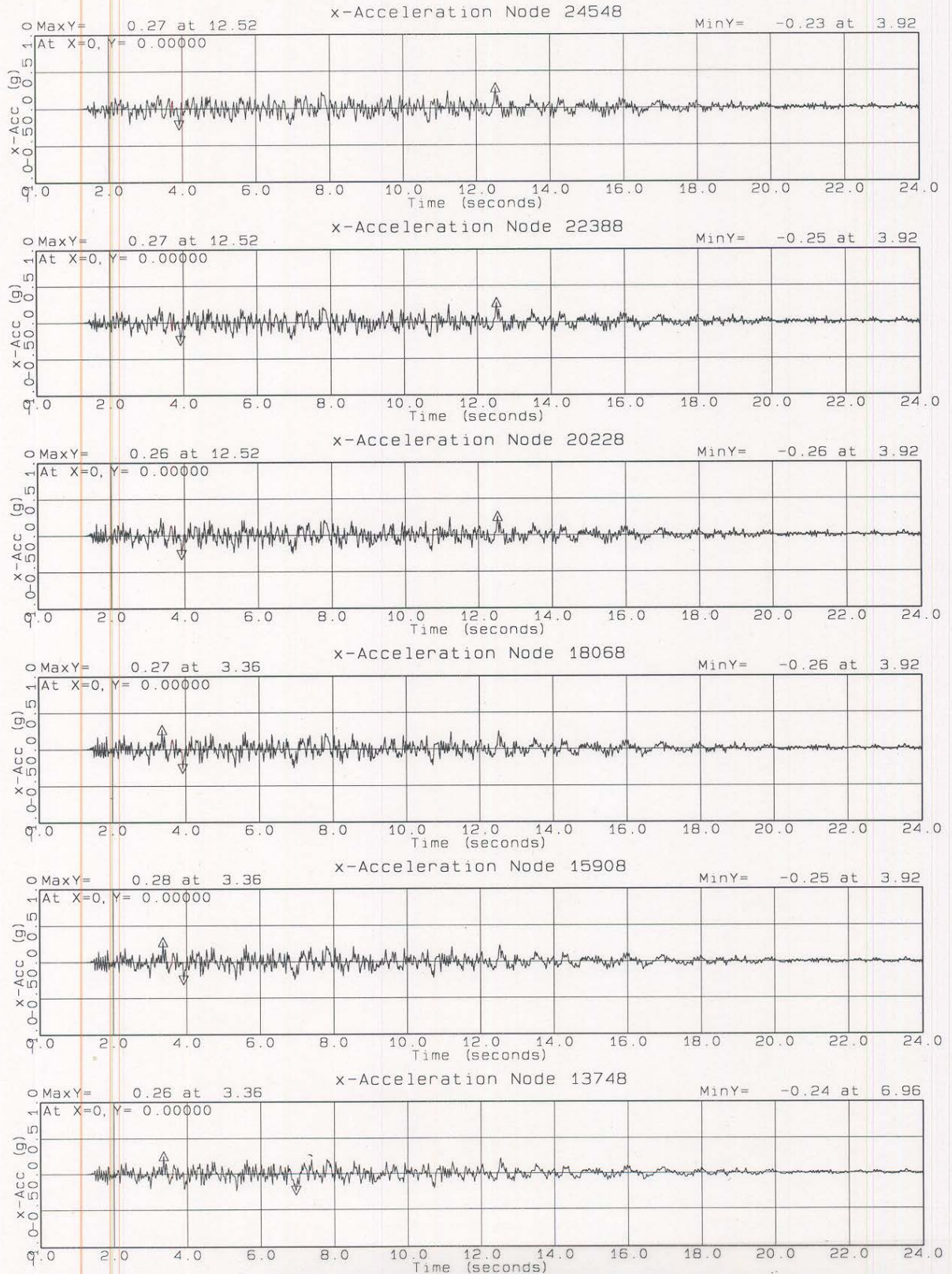


Figure 3.1.1-12 X-direction surface accelerations along the x-axis.

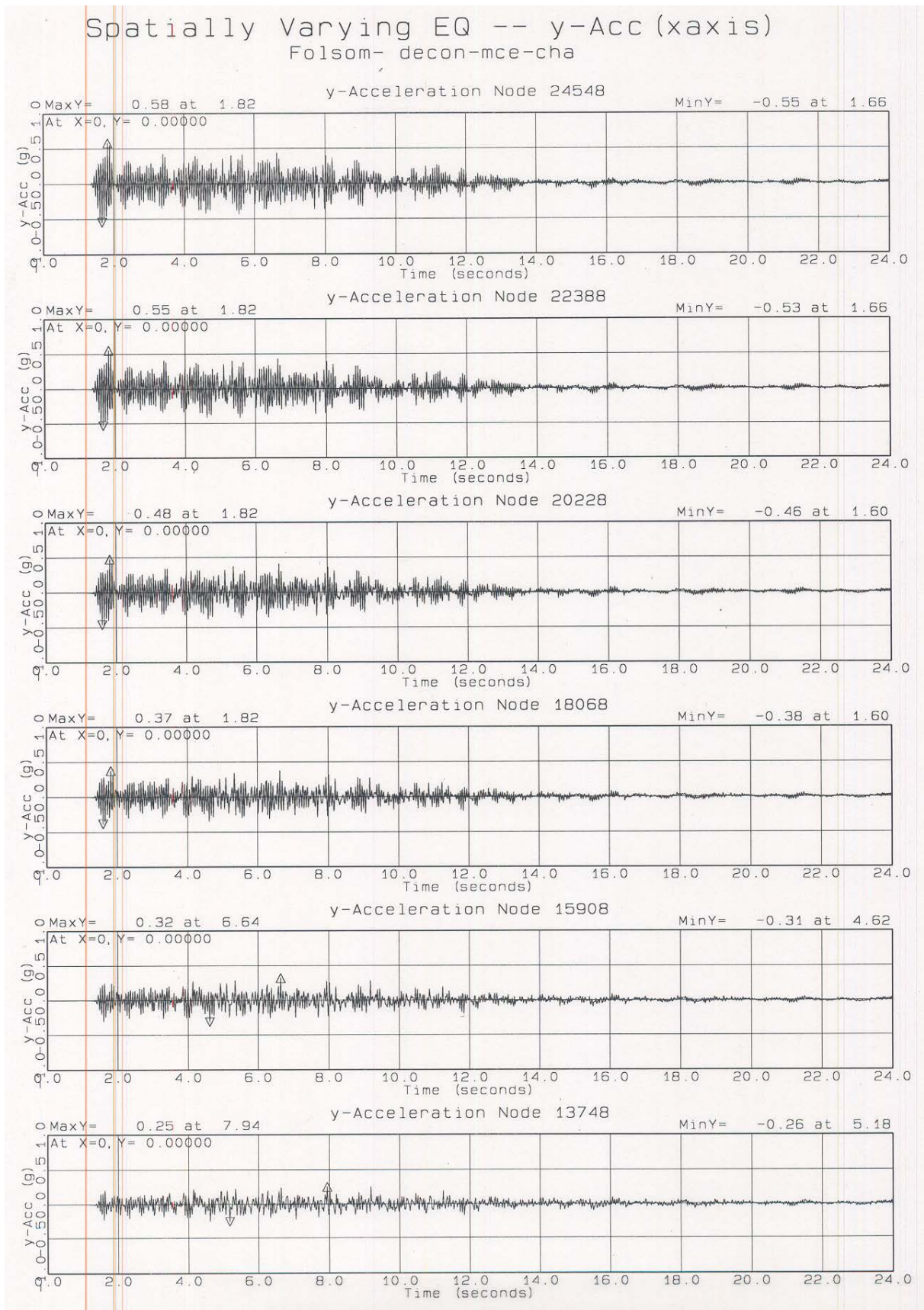


Figure 3.1.1-13 Y-direction surface accelerations along the x-axis.

Spatially Varying EQ -- z-Acc (xaxis) Folsom- decon-mce-cha

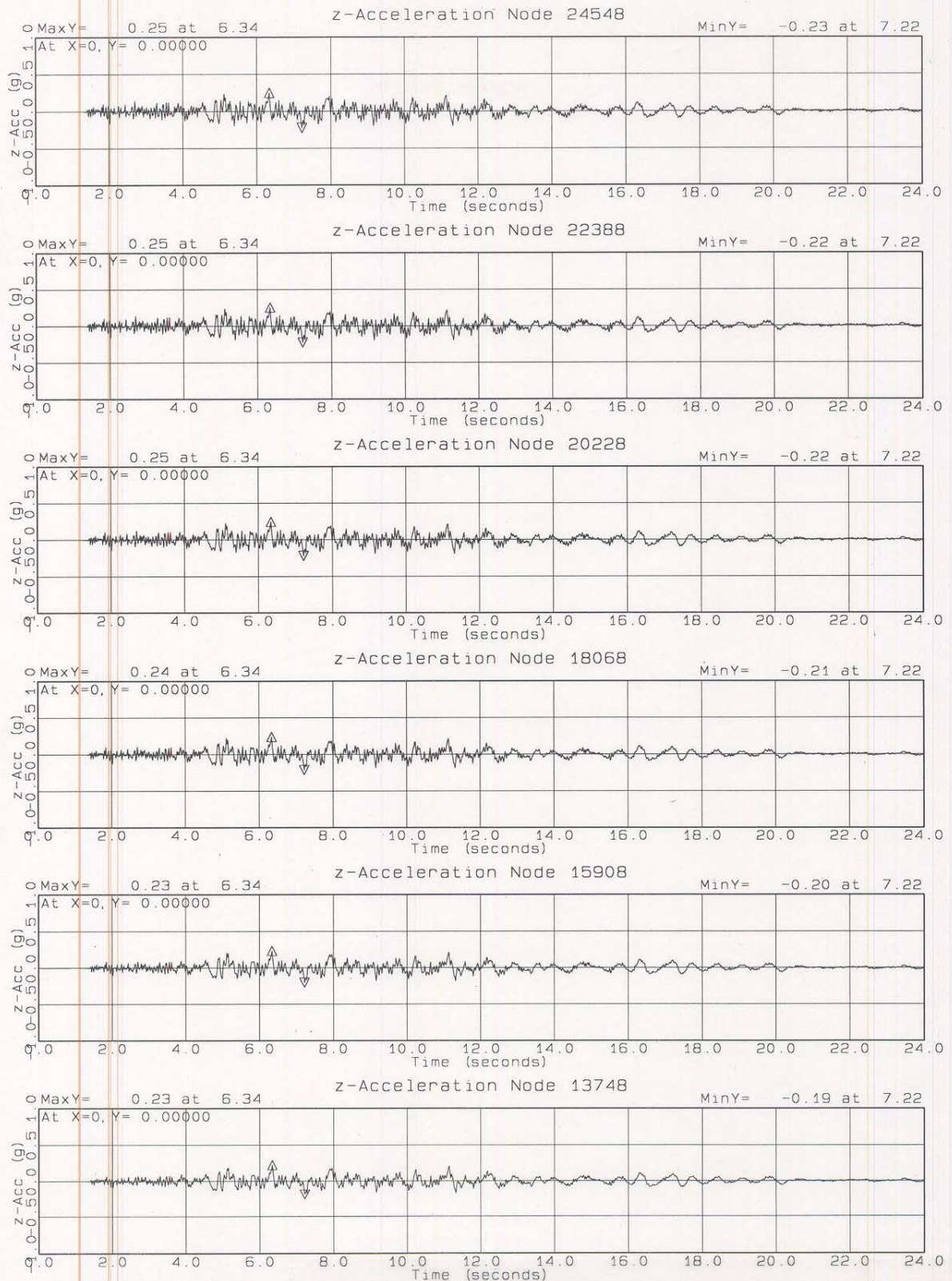


Figure 3.1.1-14 Z-direction surface accelerations along the x-axis.

Free Field Motions- Center of 600' deep Foundation Surface Motions and Response spectra

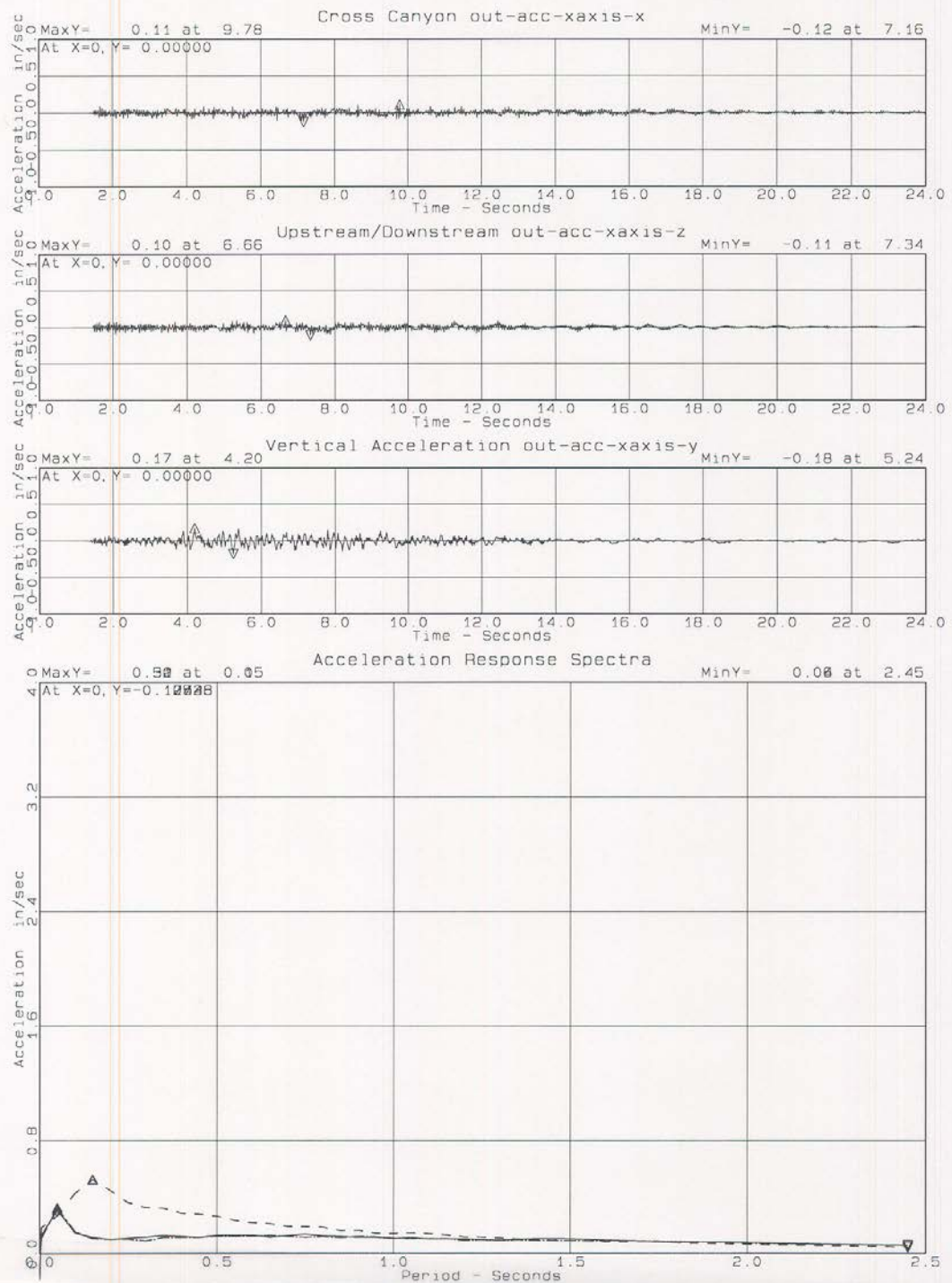


Figure 3.1.1-5 Cross-canyon, upstream-downstream, and vertical surface accelerations and response spectra—600-foot-deep model.

Free Field Motions- Center of 400' deep Foundation Surface Motions and Response spectra

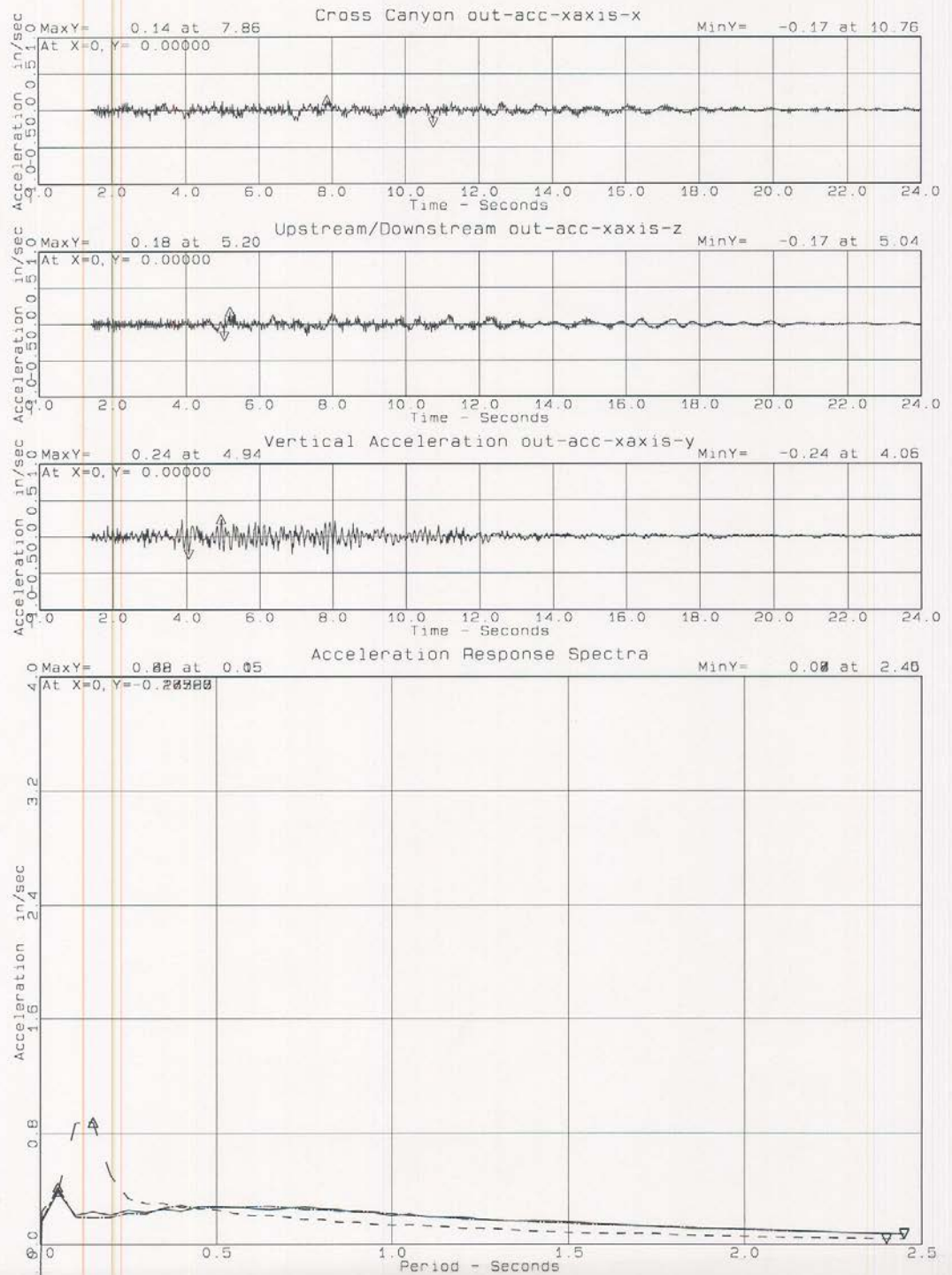


Figure 3.1.1-10 Cross-canyon, upstream-downstream, and vertical surface accelerations and response spectra—400-foot-deep model.

Free Field Motions- Center of 200' deep Foundation Surface Motions and Response spectra

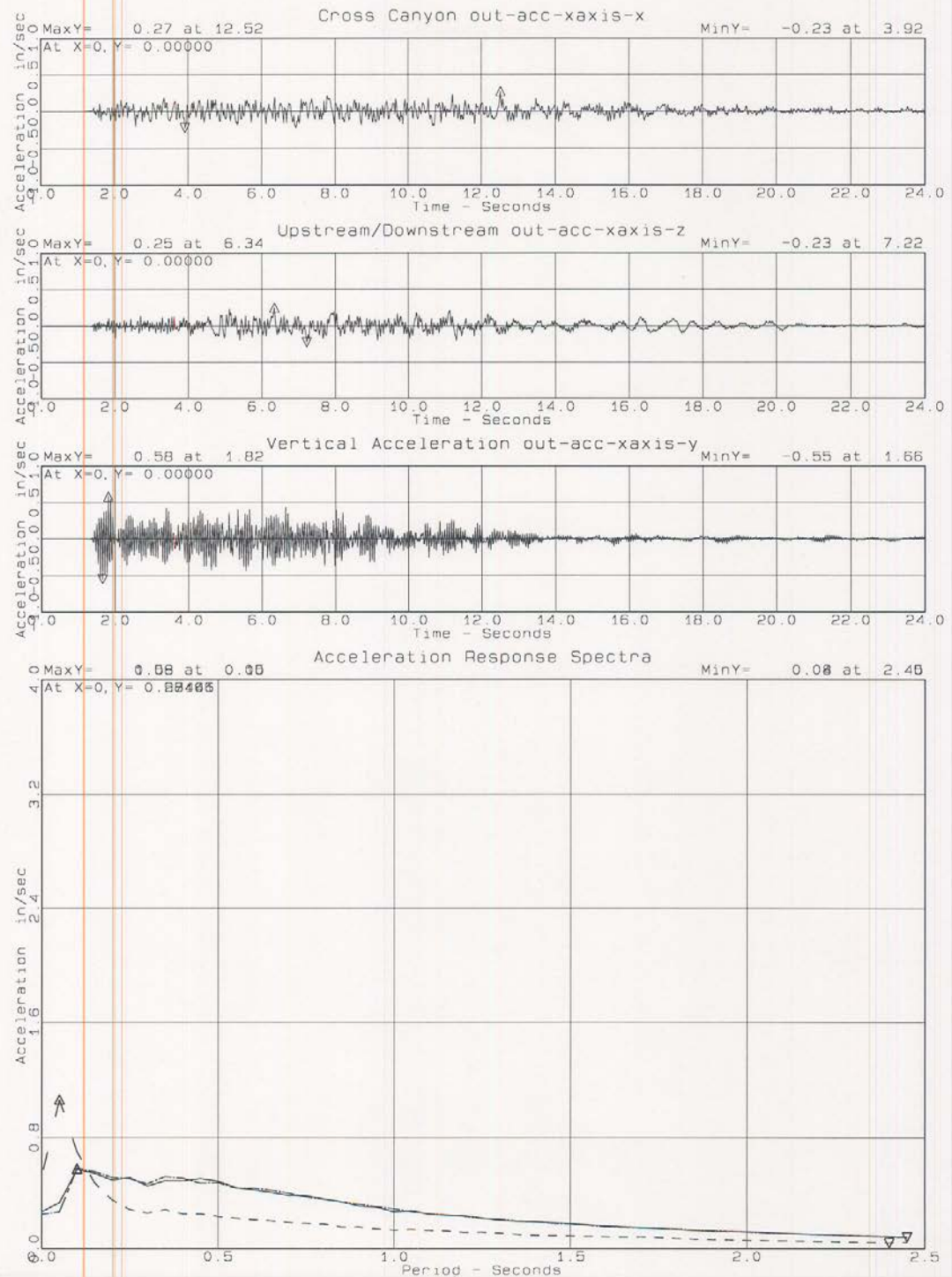


Figure 3.1.1-15 Cross-canyon, upstream-downstream, and vertical surface accelerations and response spectra—200-foot-deep model.

For the two models with the greatest depths, the accelerations at the surface in the two horizontal directions (x and z) are less than the input accelerations. Only for the model with the 200-foot depth have the horizontal accelerations been magnified. This indicates that there is a danger of energy loss in the model if the foundation depth is too great or if the ratio of the horizontal extent of the model-to-vertical extent is too low. The acceleration in the vertical direction is magnified in all three models, with the most pronounced magnification in the model with the 200-foot depth. It is postulated that this effect occurs because the input motions are applied along a horizontal plane. The motions in the horizontal directions are allowed to travel up through the foundation as shear waves, while the vertical accelerations are applied as compression waves (p-waves).

Acceleration response spectra were plotted for the accelerations measured at the center of the top surface of each model. For the two models with dimensions 600 by 600 by 600 feet and 600 by 600 by 400 feet, the horizontal acceleration spectra did not match the input motion in frequency content. Each had a frequency content of predominantly 20 Hz compared to a dominant frequency of less than 10 Hz for the input motion. The horizontal acceleration response spectra for the 200-foot-deep model compared closely with the input motion in magnitude and in frequency content. When vertical motions are compared, the 600-foot-deep model agreed more closely with the input motion than did the other two models. As the depth of the model decreased, the predominant frequency of the vertical motions increased (6.6 to 10 Hz for the 400-foot-deep model and 20 Hz for the 200-foot-deep model) along with the magnitude of the response.

One additional notable phenomenon is that the surface accelerations are higher in the center of the model than at the edges. This has been referred to as the “halo effect” and is true in all three directions for all three models (with the exception of the x-acceleration for the model with the 200-foot depth). This indicates that when the foundation model is created, the size should be adjusted so that the accelerations reaching the surface near the location of the dam are the desired surface accelerations and that the pronounced halo is away from this location.

3.1.2 Variation of mesh density

Two free-field models were analyzed to check the effect of a fine versus a coarse mesh. The dimensions of these models were 1,400 by 1,650 by 700 feet. In the first model, the element size was 31.8 by 50 by 31.5 feet for the elements below the plane of application of accelerations and 31.8- by 37.5- by 37.5-feet above the plane of application of accelerations (figure 3.1.2-1). In the second model, the element size was 63.6 by 83.3 by 75 feet for the elements below the plane of application of accelerations and 63.6- by 75- by 75-feet above the plane of application of accelerations (figure 3.1.2-5). The material properties indicate that the wavelengths of interest are 325 to 3,250 feet for a range of frequencies from 20 to 2 Hz. The element dimensions should not exceed 32.5 feet for the higher frequencies and 325.0 feet for the lower frequencies of the range of frequencies of interest. Considering the higher frequency, the first model has elements of approximately the desired size and the second model has elements which are twice as large.

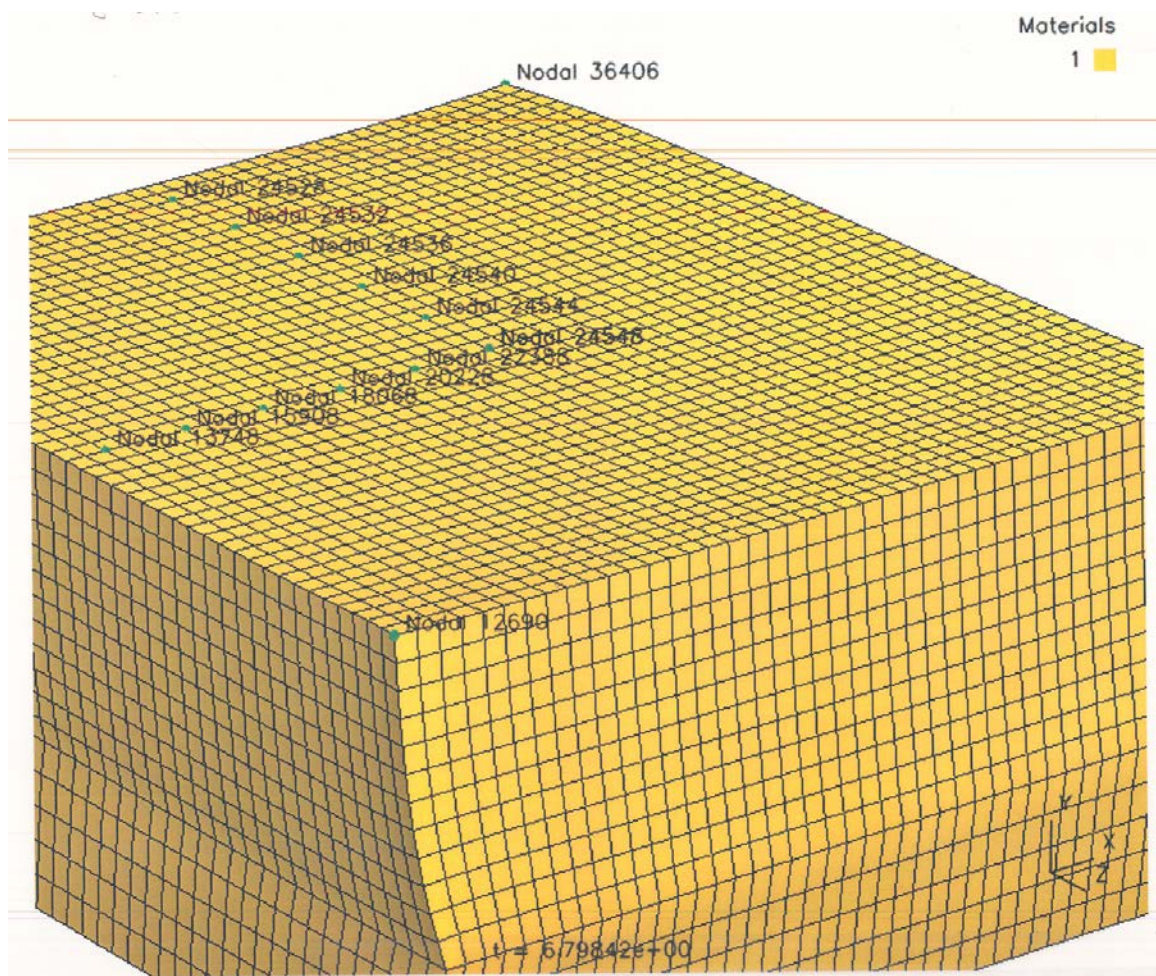


Figure 3.1.2-1 Model for free-field motions (1,400 by 1,650 by 700 feet)—fine mesh.

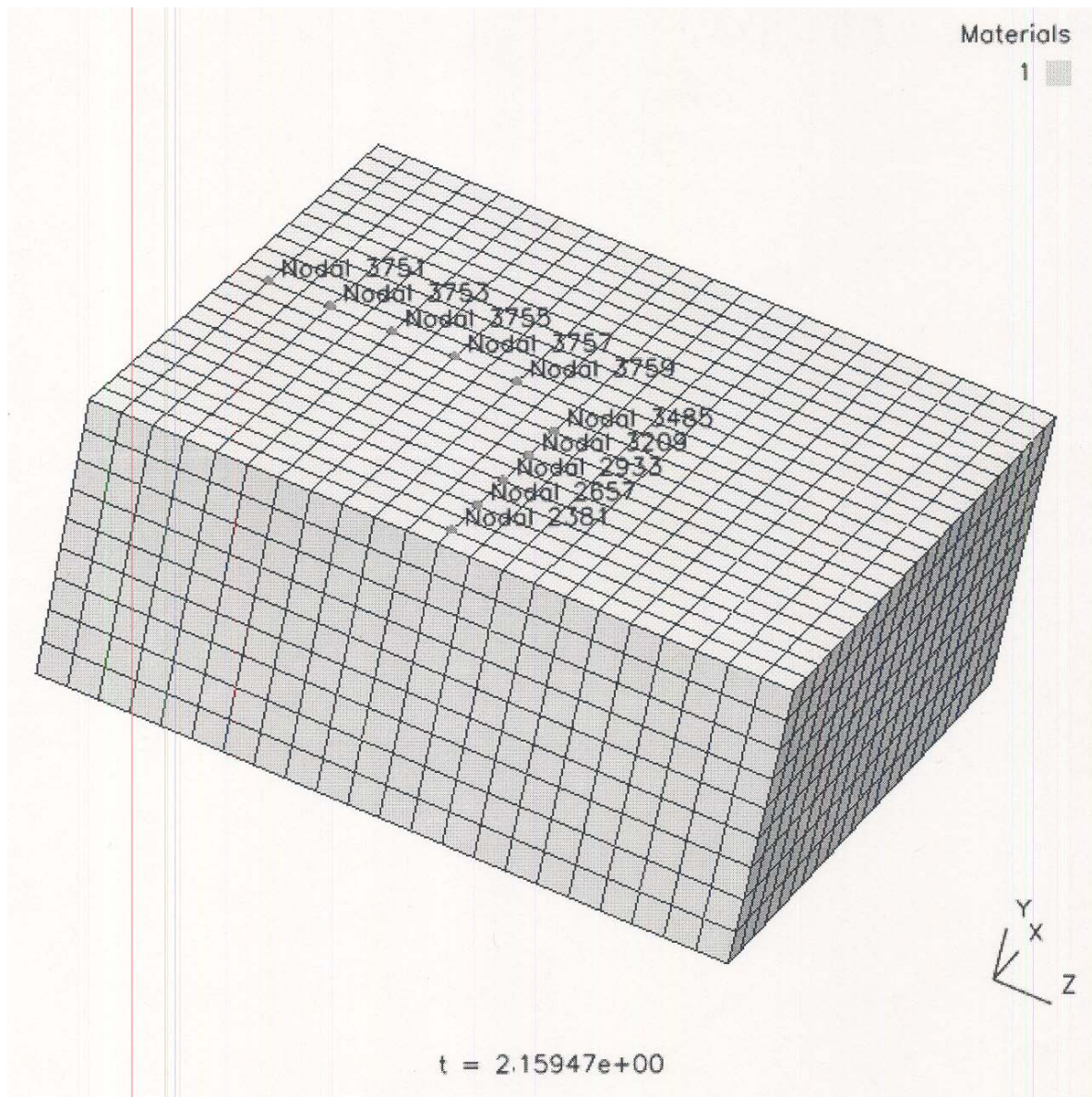


Figure 3.1.2-5 Model for free-field motions (1,400 by 1,650 by 700 feet)—coarse mesh.

Accelerations in all three directions were extracted for a line of nodes on the surface along the x-axis at the center of the model. Figures 3.1.2-2 through -4 are accelerations from the coarse mesh and figures 3.1.2-6 through -8 are accelerations from the fine mesh. As before, the average maximum accelerations are compared in the three bar charts shown in figure 3.1.2-9. In these plots, the blue bars represent the coarse mesh and the red bars represent the fine mesh. The pale blue bars represent the input acceleration. In this case, the mesh density affected the surface accelerations very little. This is probably because the predominant frequency of the foundation is approximately 4 Hz, corresponding to a wavelength of 1,625 feet and a critical element size of 162.5 feet.

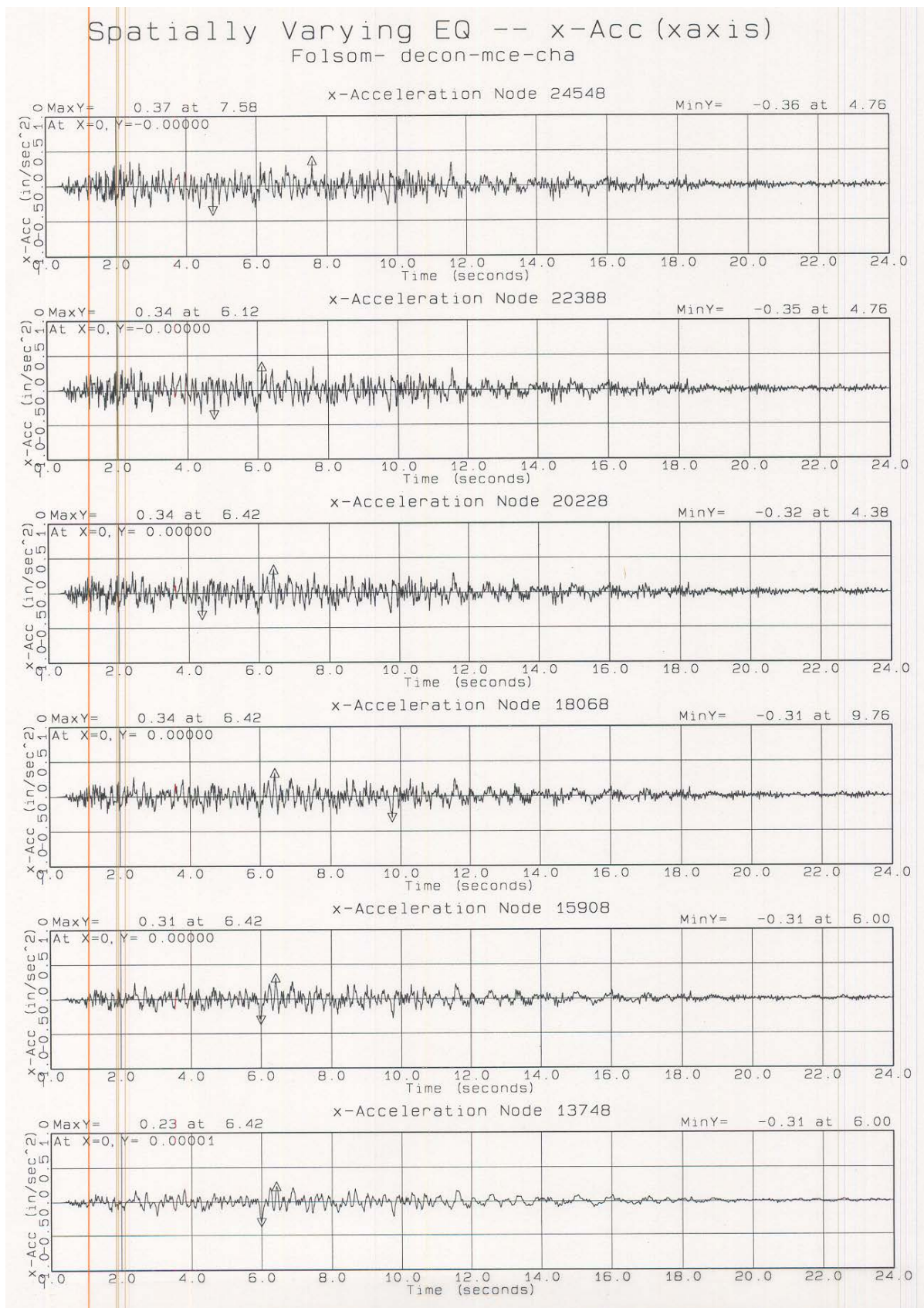


Figure 3.1.2-2 X-direction surface accelerations along the x axis.

Spatially Varying EQ -- y-Acc (xaxis) Folsom- decon-mce-cha

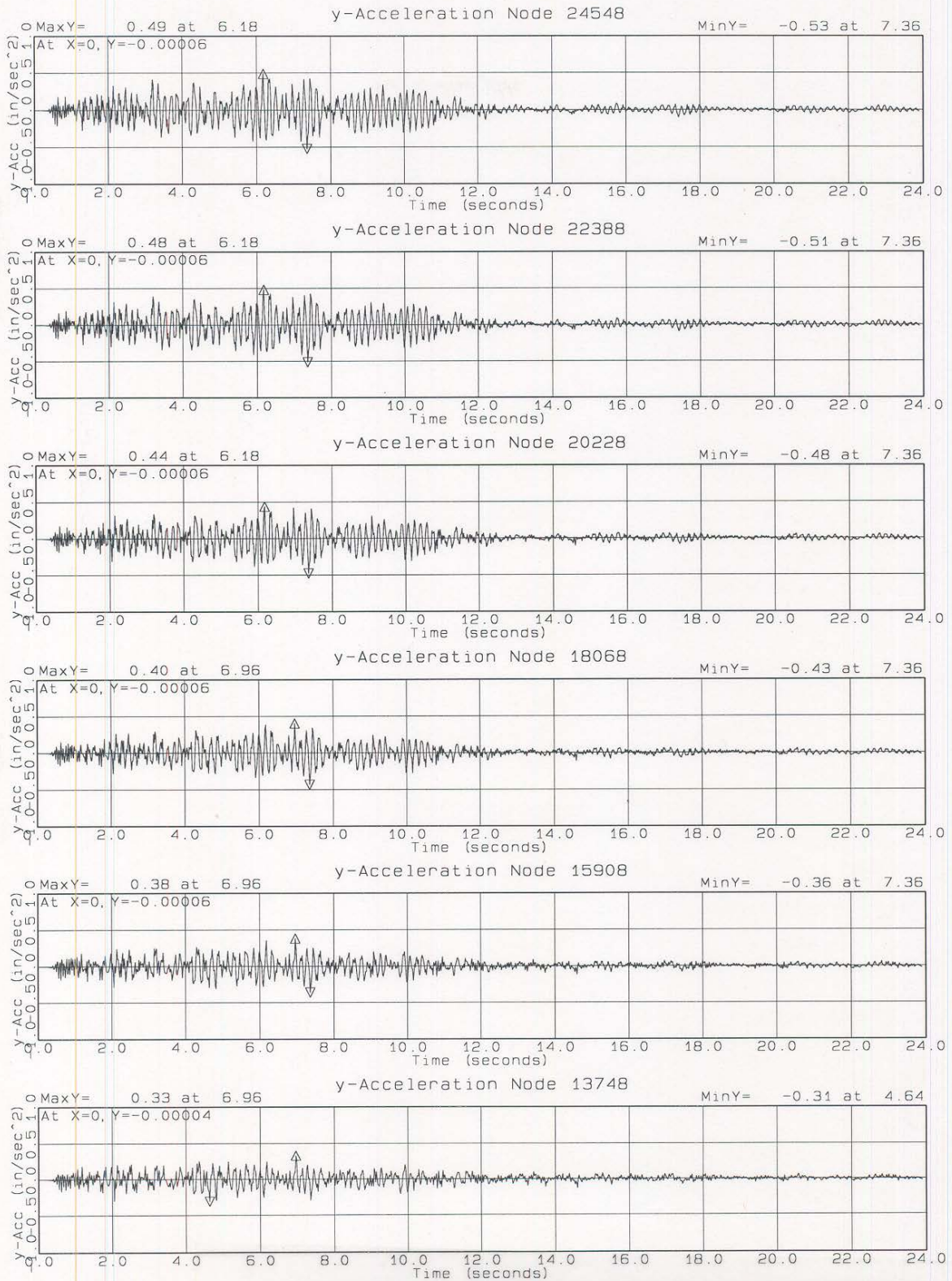


Figure 3.1.2-3 Y-direction surface accelerations along the x axis.

Spatially Varying EQ -- z-Acc (xaxis) Folsom- decon-mce-cha

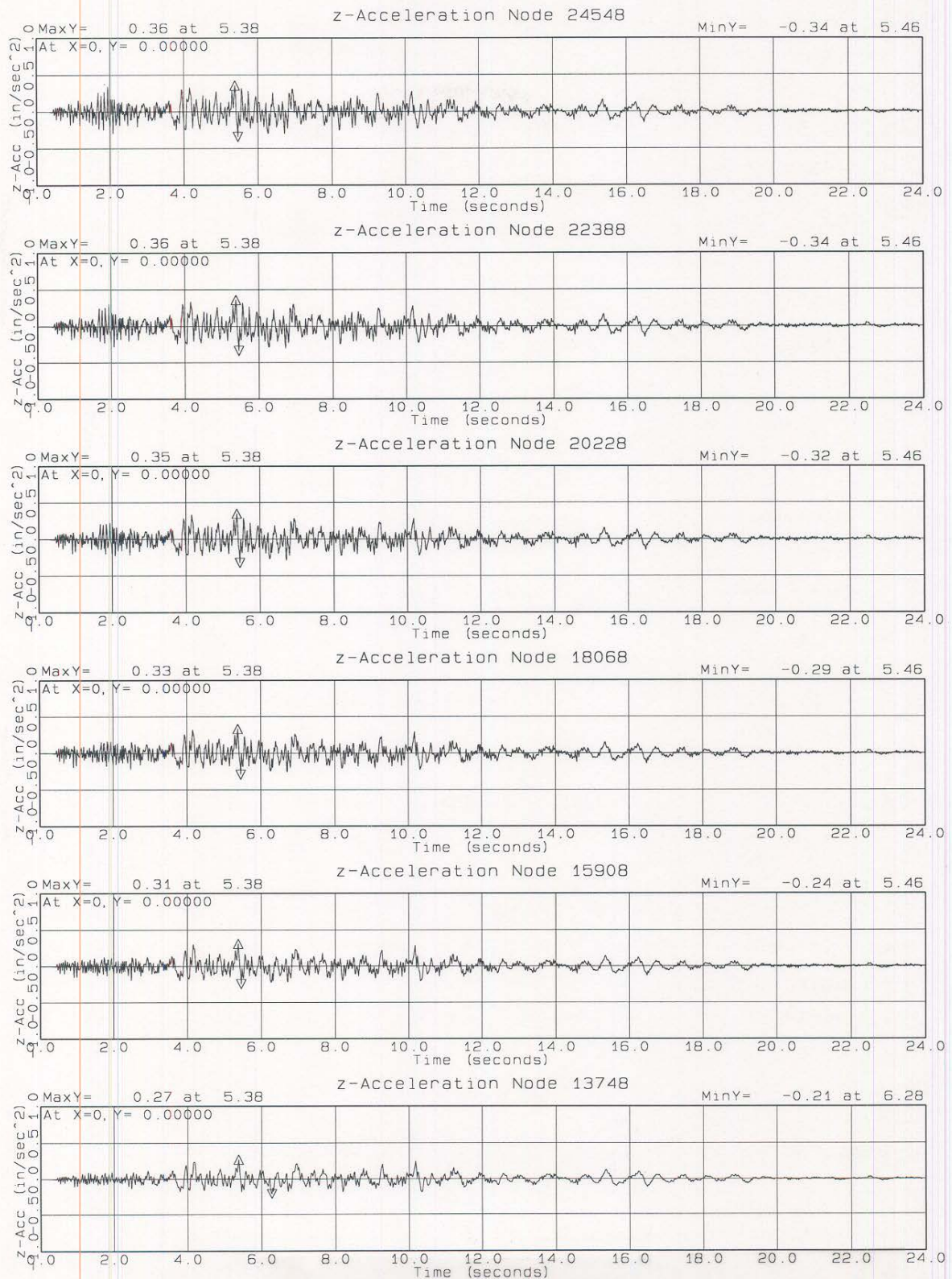


Figure 3.1.2-4 Z-direction surface accelerations along the x axis.

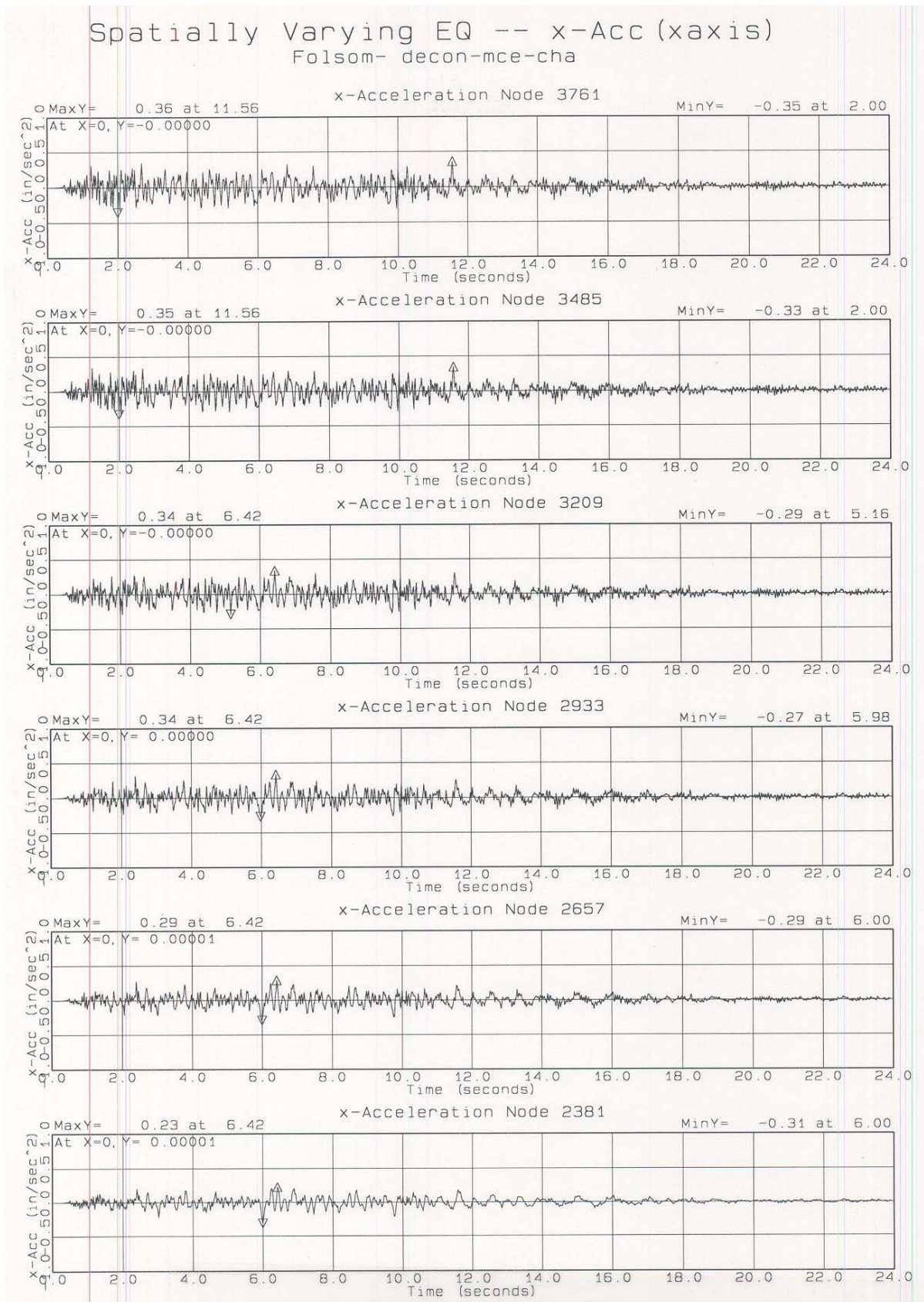


Figure 3.1.2-6 X-direction surface accelerations along the x axis.

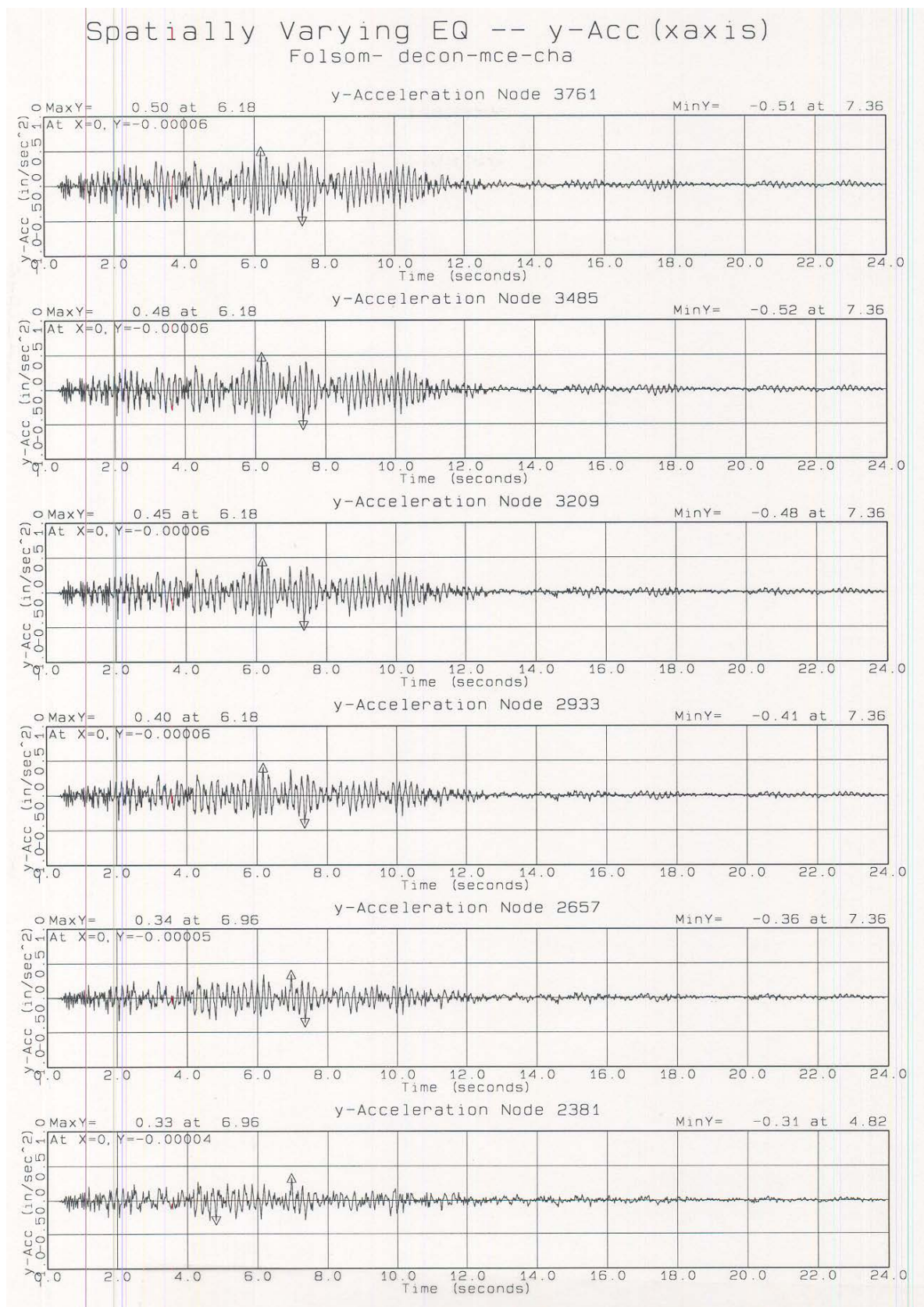


Figure 3.1.2-7 X-direction surface accelerations along the x axis.

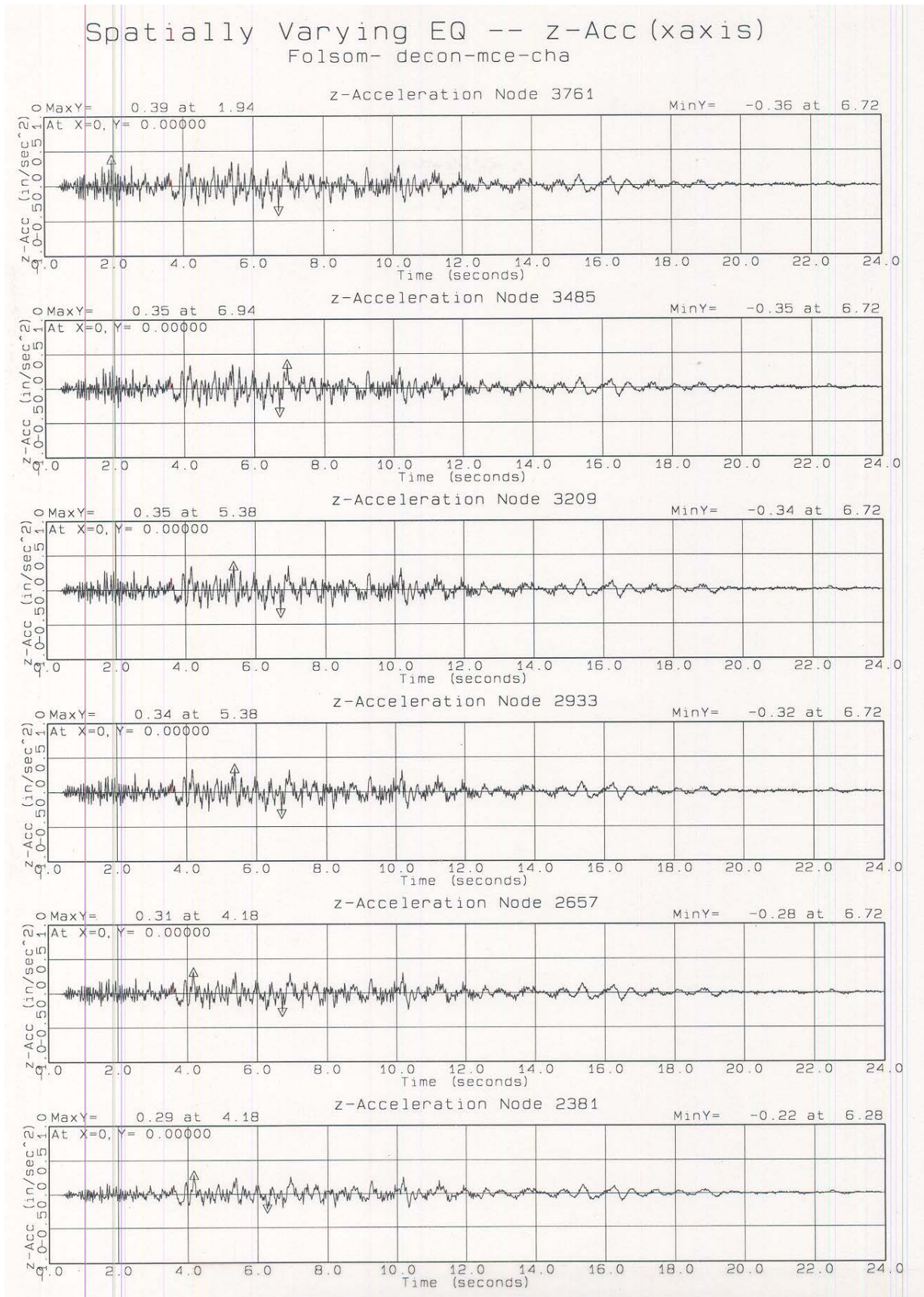


Figure 3.1.2-8 X-direction surface accelerations along the x axis.

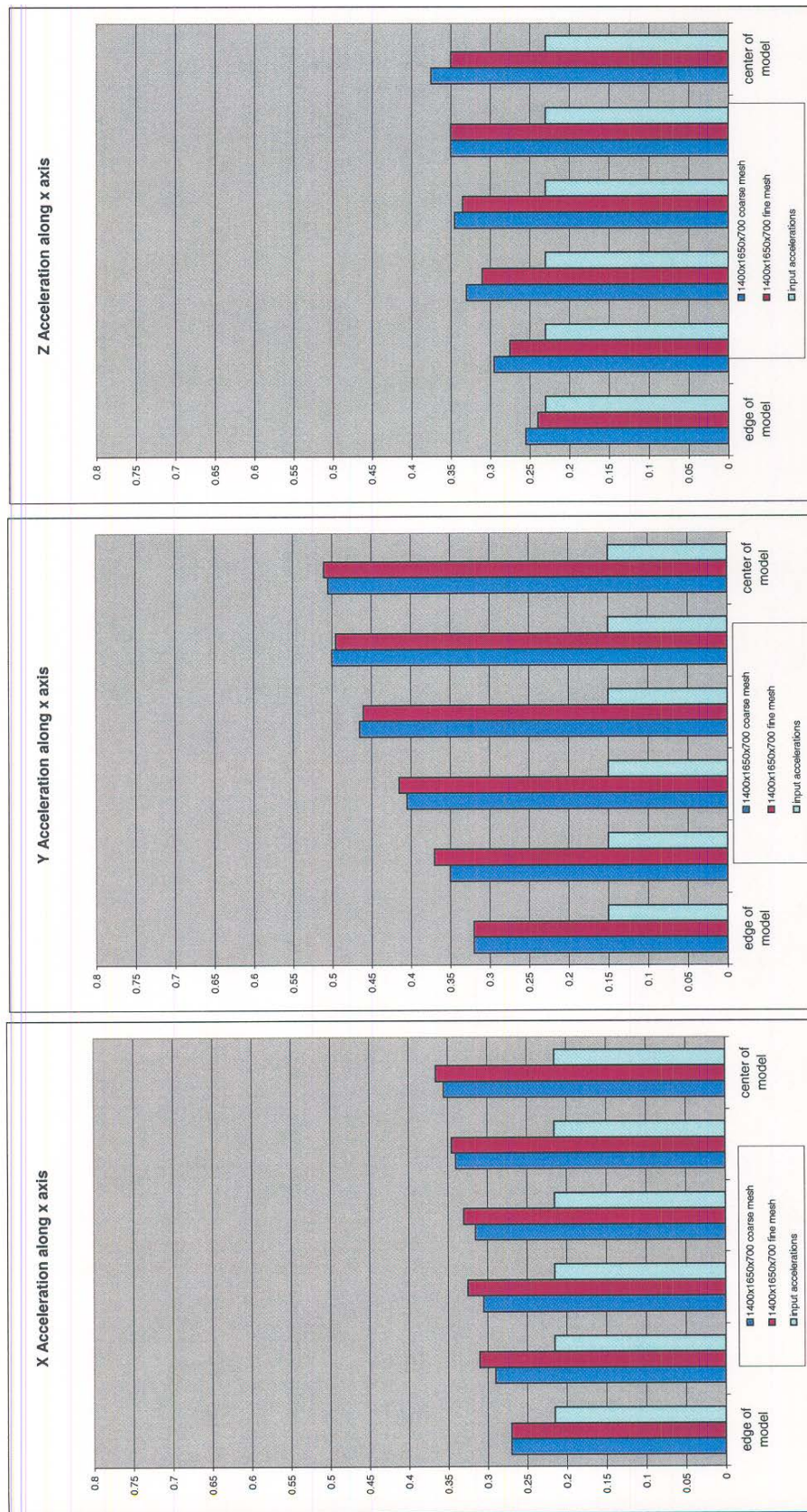


Figure 3.1.2-9 Comparison of average maximum accelerations.

3.1.3 Variation in Damping

A third set of models was analyzed in which the effect of damping was considered on foundations with a flat surface. The models were 1,350 by 1,350 by 550 feet in depth (see figure 3.1.3-1). Ground motions were applied on a plane, 450-feet deep in the foundation which was 100-feet above the base of the model. The material properties remain the same. Elements were 61 by 61 by 75 feet. This is within the appropriate size limits.

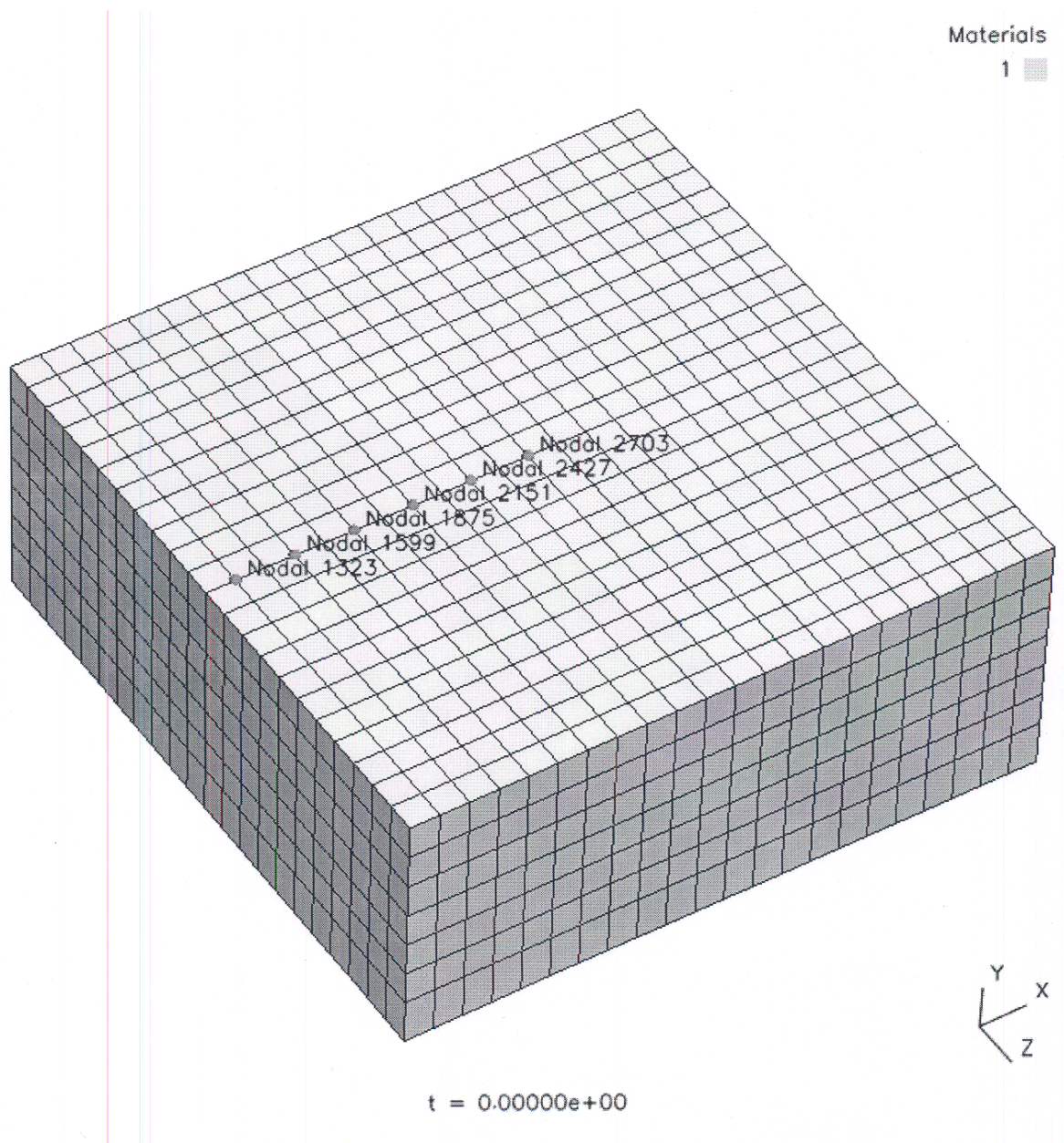


Figure 3.1.3-1 Model for free-field motions (1,350 by 1,350 by 550 feet).

Accelerations in all three directions were extracted for a line of nodes on the surface along the x-axis at the center of the model. For the case with no damping, the accelerations at the surface on a row of nodes along the x-axis are shown in figures 3.1.3-2 through -4. The accelerations in all three directions at the center of the surface of the foundation and their acceleration, velocity, and displacement response spectra at 5-percent damping are shown in figures 3.1.3-5a through -5c. A mass damping value of $\alpha = 3.25$ was then added and the analysis re-run. For this case, the accelerations at the surface on a row of nodes along the x-axis are shown in figures 3.1.3-6 through -8. The accelerations, velocity, and displacement response spectra at 5-percent damping in all three directions at the center of the surface of the foundation and their acceleration response spectra at 5-percent damping are shown in figures 3.1.3-9a through -9c.

Spat Vary EQ - 1350x1350x550- x-Acc (xaxis)
Folsom- decon-mce-cha

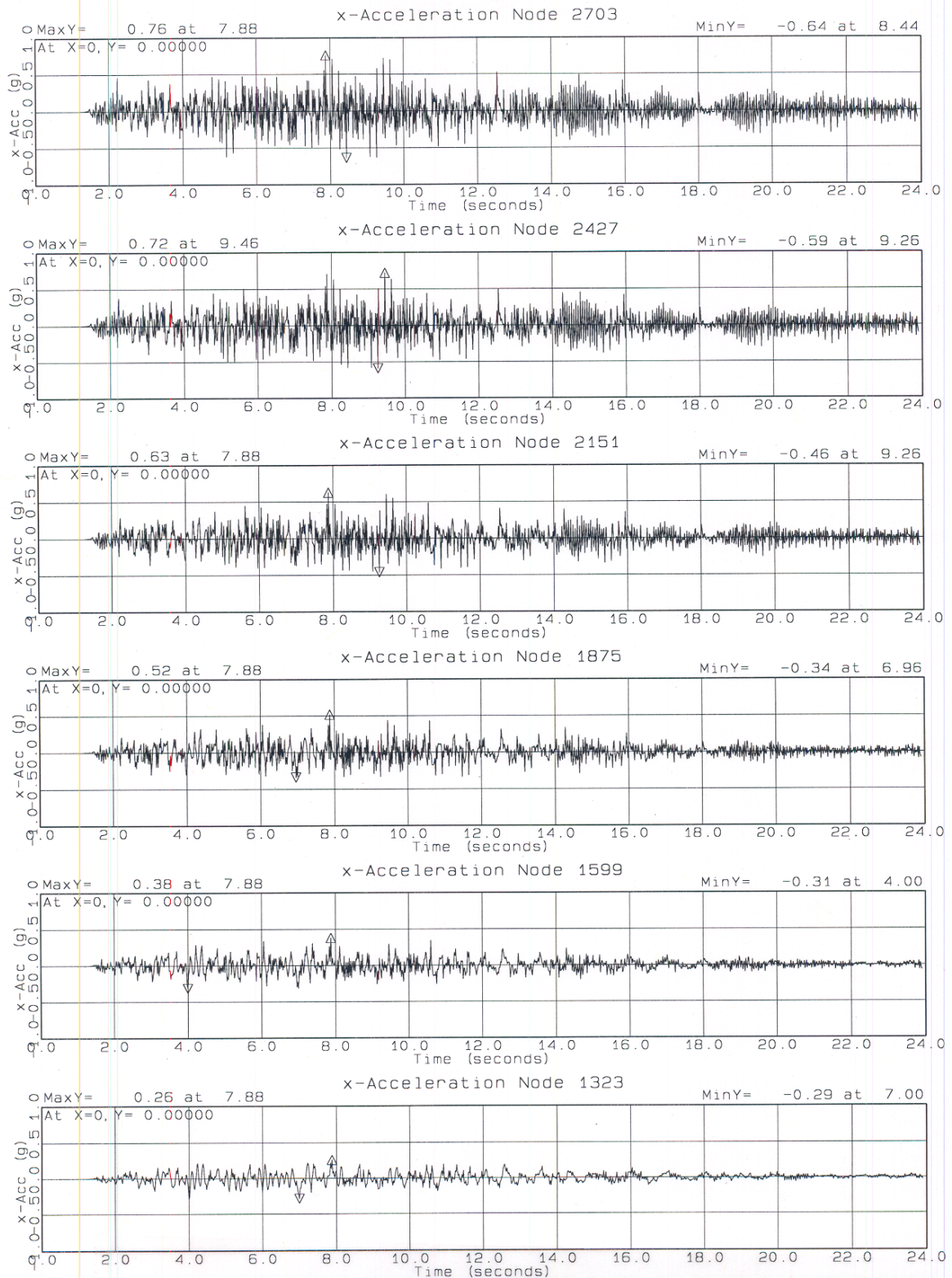


Figure 3.1.3-2 X-direction surface accelerations along the x axis—no damping.

Spat VaryEQ -1350x1350x550- y-Acc (xaxis)
Folsom- decon-mce-cha

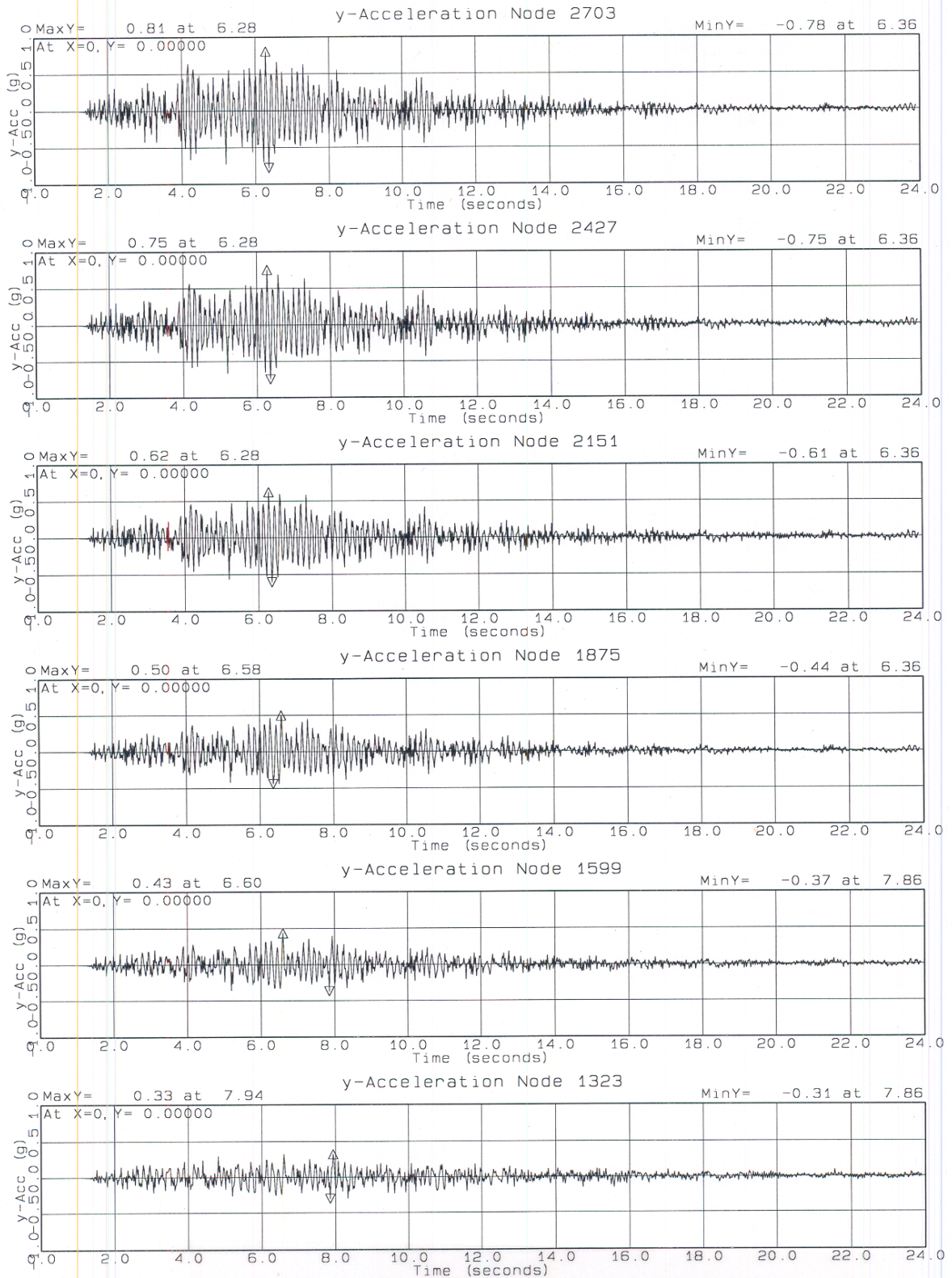


Figure 3.1.3-3 Y-direction surface accelerations along the x axis—no damping.

Spat VaryEQ -1350x1350x550- z-Acc (xaxis)
Folsom- decon-mce-cha

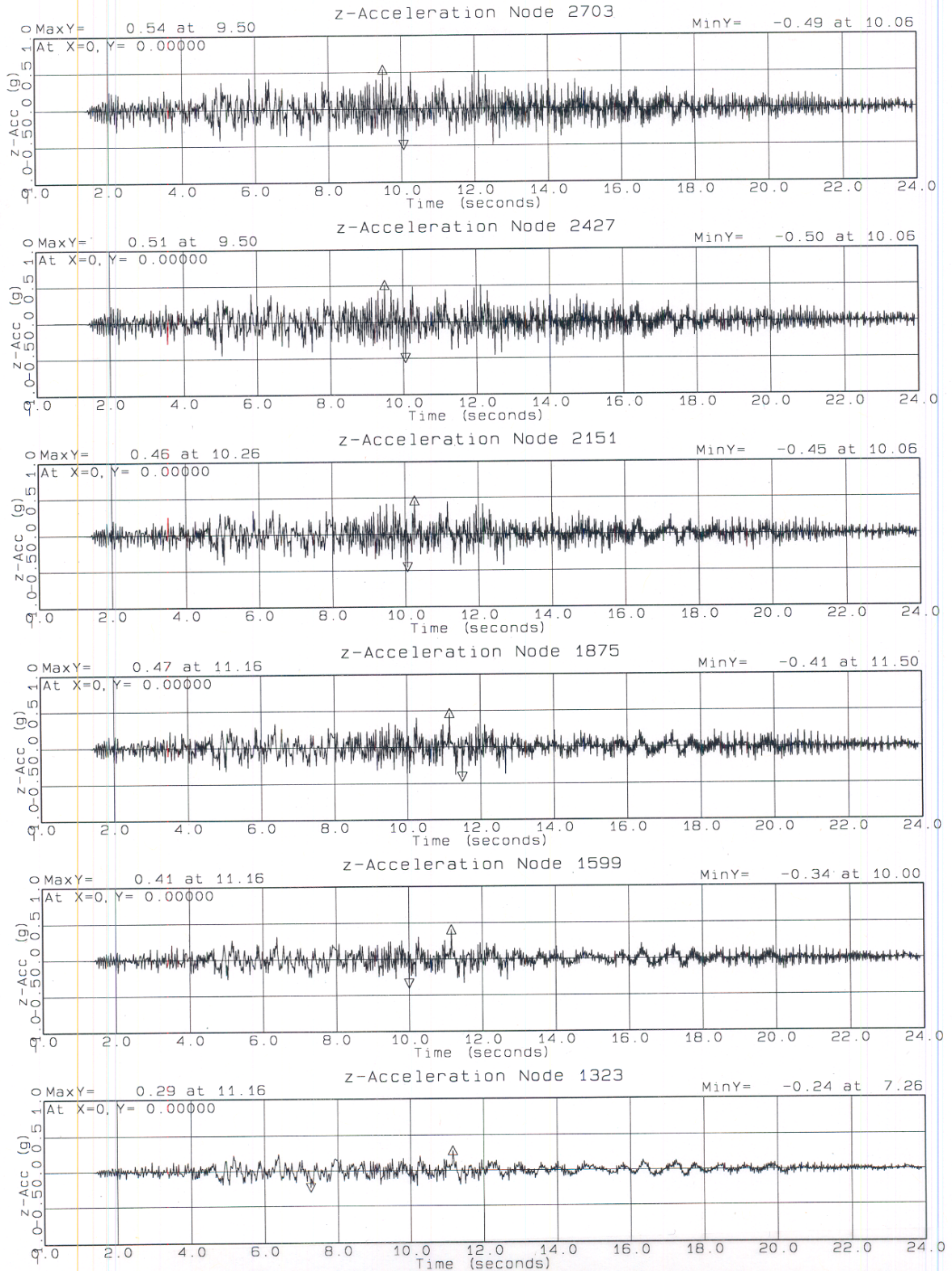


Figure 3.1.3-4 Z-direction surface accelerations along the x axis—no damping.

Foundation Study -no damp 1 decon MCE-cha Ground Motions and Response spectra

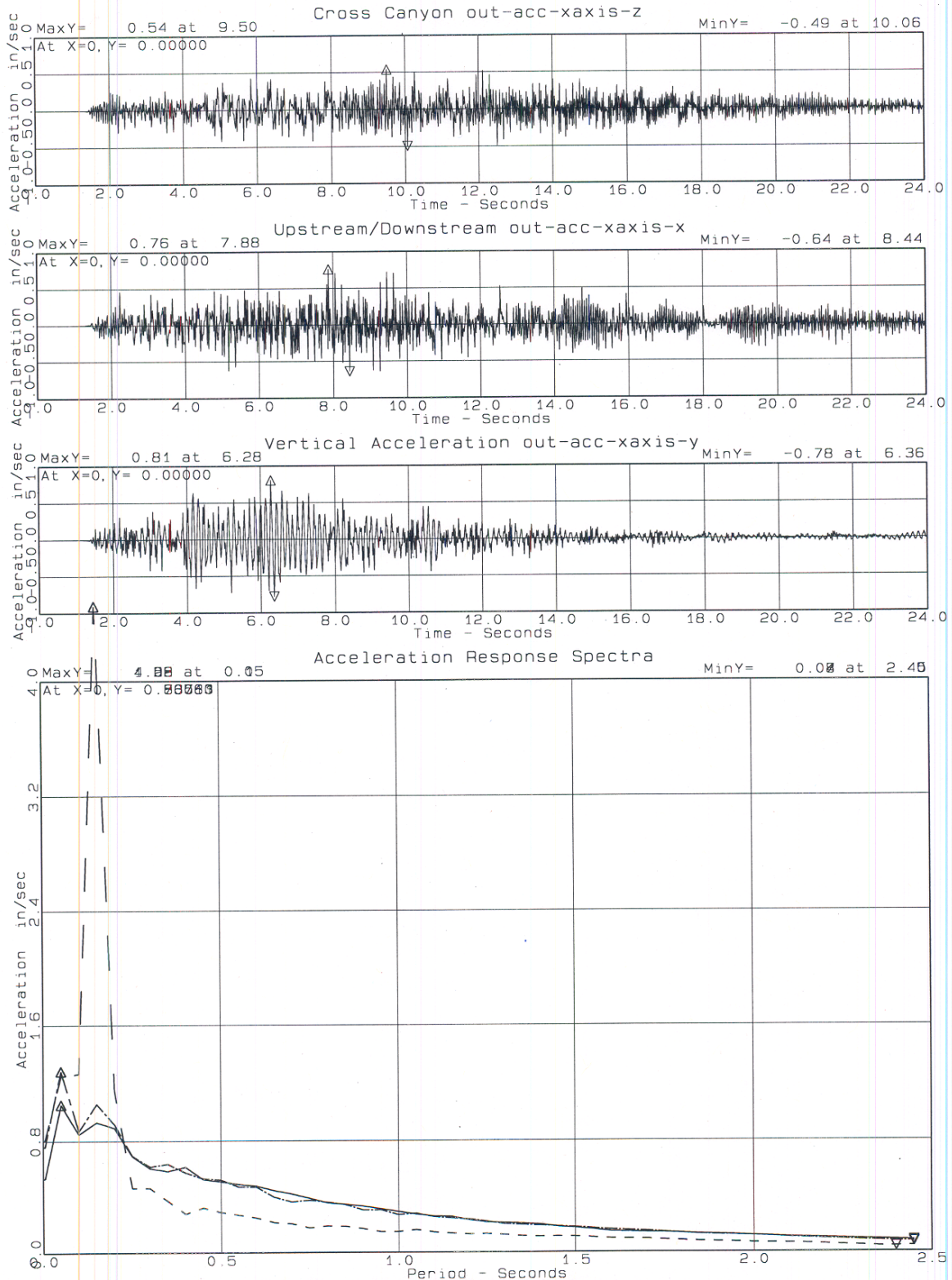


Figure 3.1.3-5 Time history accelerations and response spectra for surface ground motions at the center of the model—no damping.

Spat Vary EQ-1350x1350x550-a=3.25-x-Acc (xaxis)
Folsom- decon-mce-cha

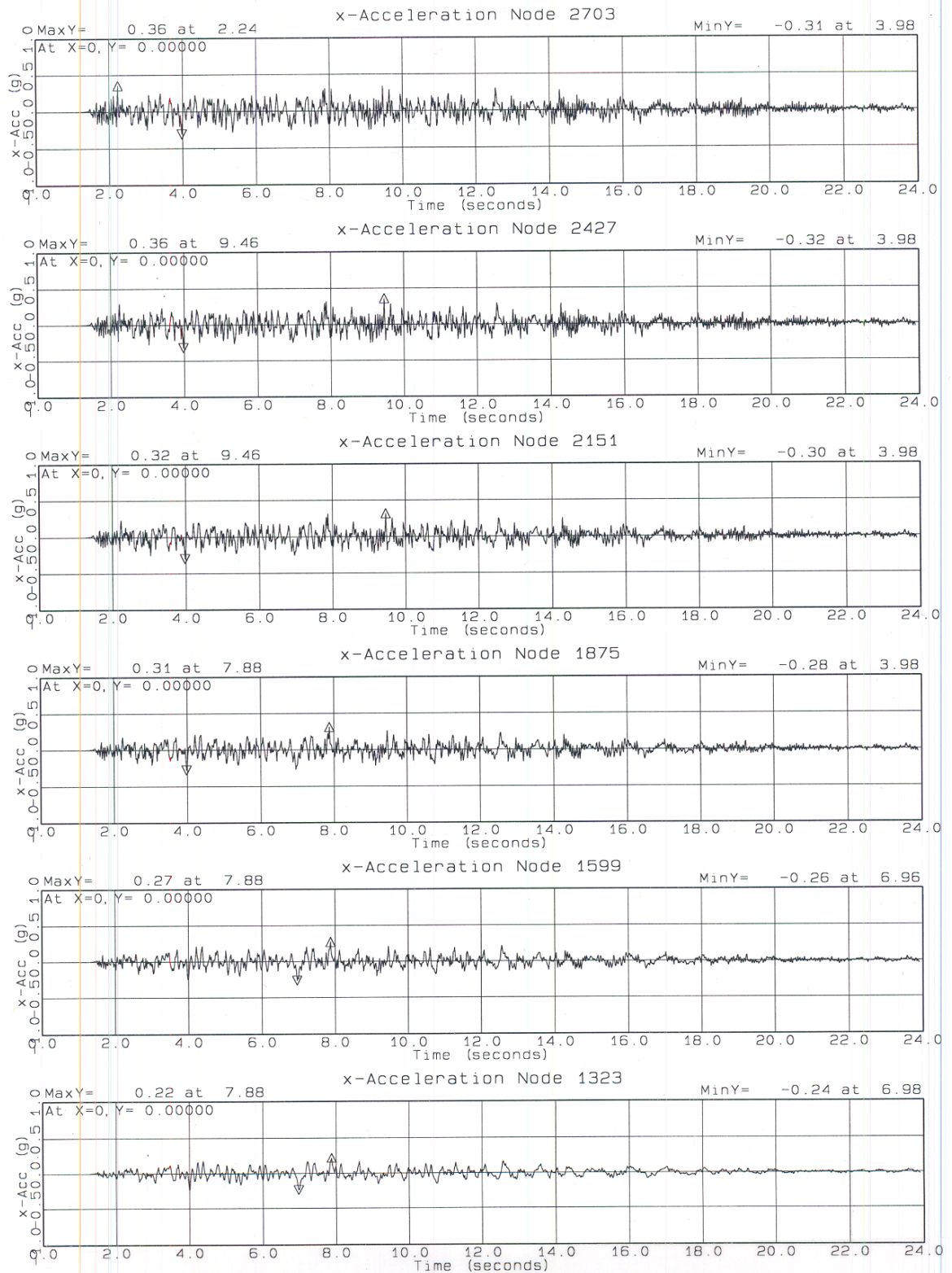


Figure 3.1.3-6 X-direction surface accelerations along the x axis—with damping.

Spat VaryEQ-1350x1350x550-a=3.25 y-Acc (xaxis)
Folsom- decon-mce-cha

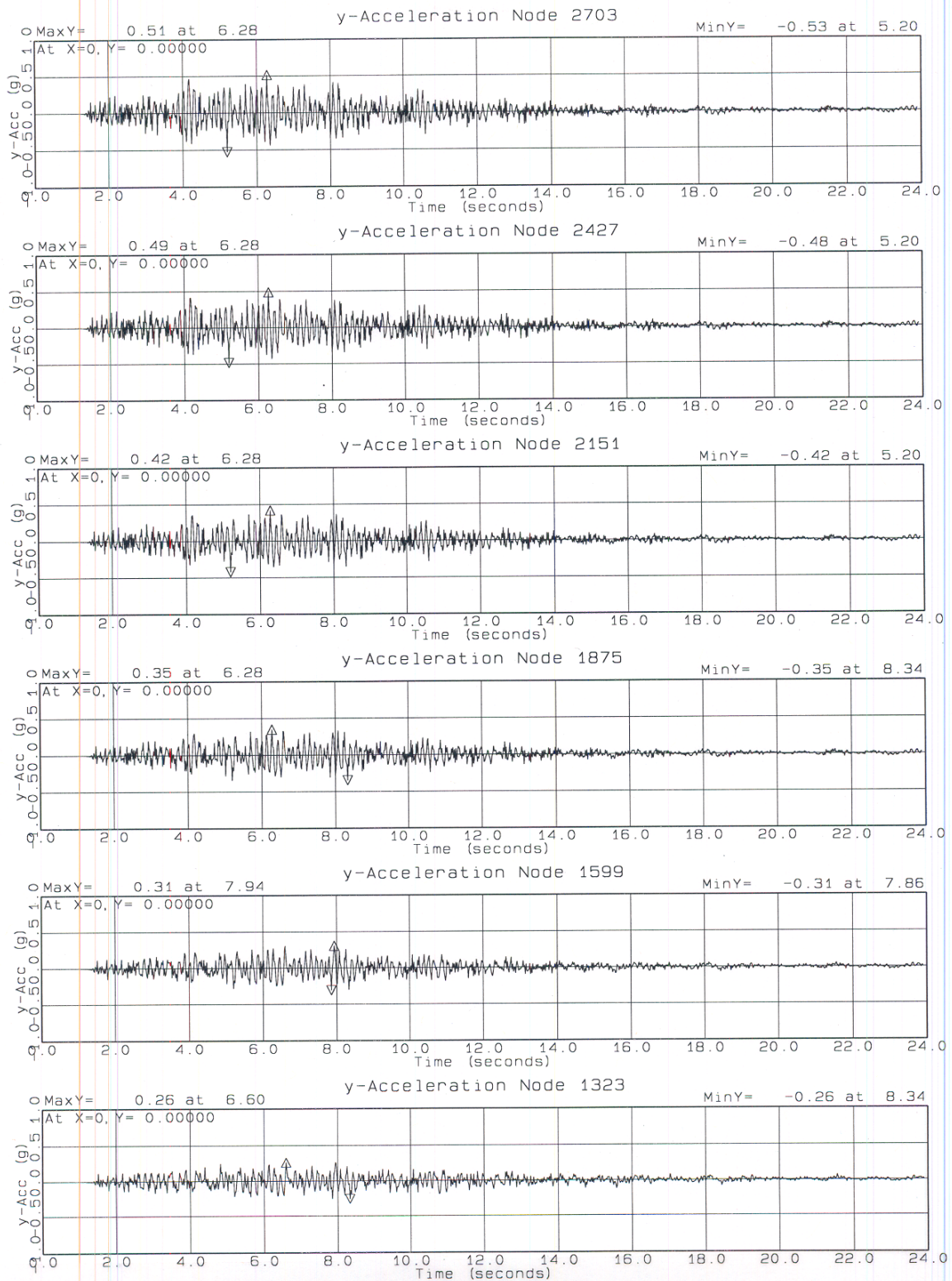


Figure 3.1.3-7 Y-direction surface accelerations along the x axis—with damping.

Spat VaryEQ-1350x1350x550-a=3.25 z-Acc (xaxis)
Folsom- decon-mce-cha

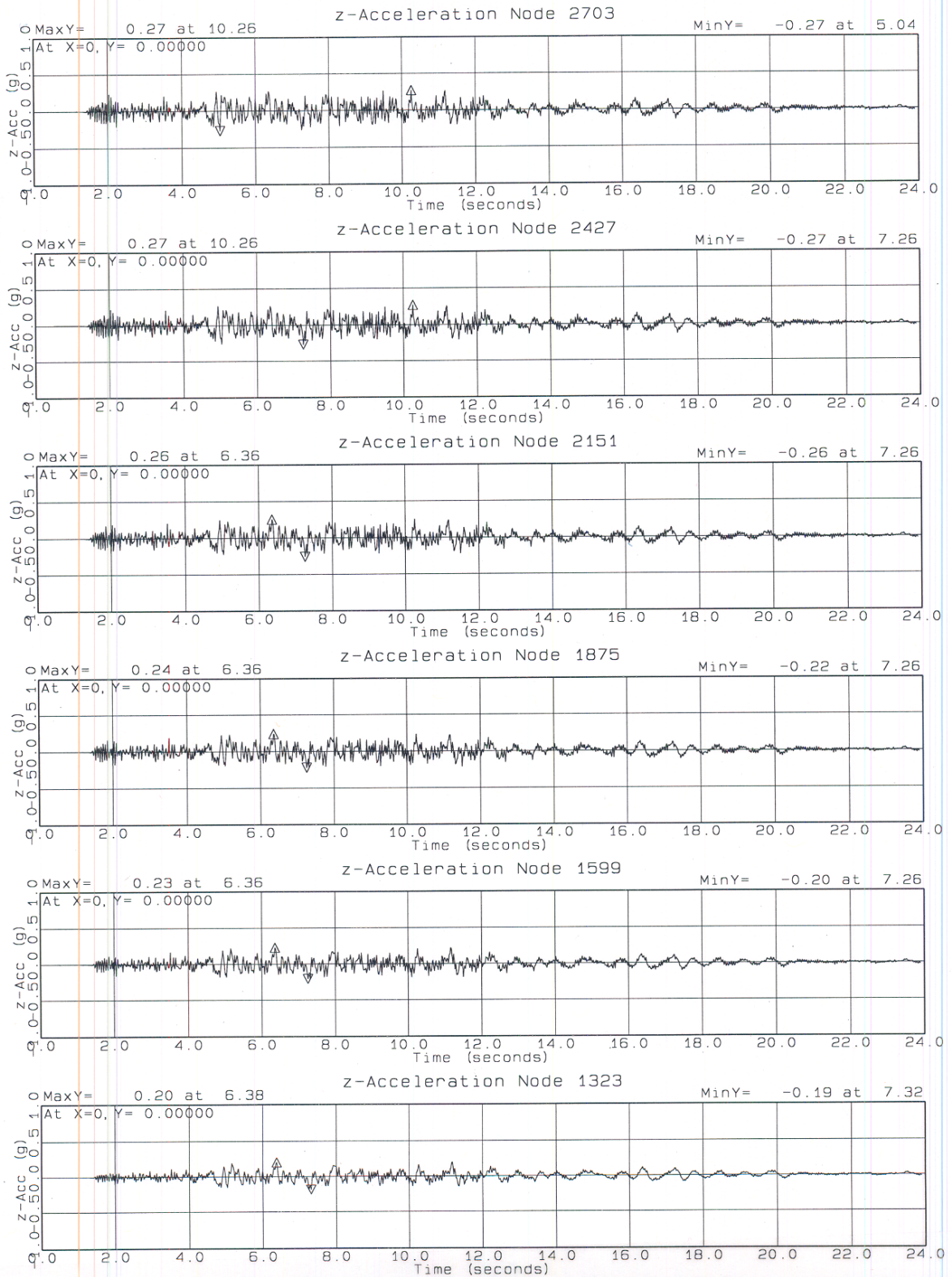


Figure 3.1.3-8 Z-direction surface accelerations along the x axis—with damping.

Foundation Study -a=3.25- decon MCE-cha Ground Motions and Response spectra

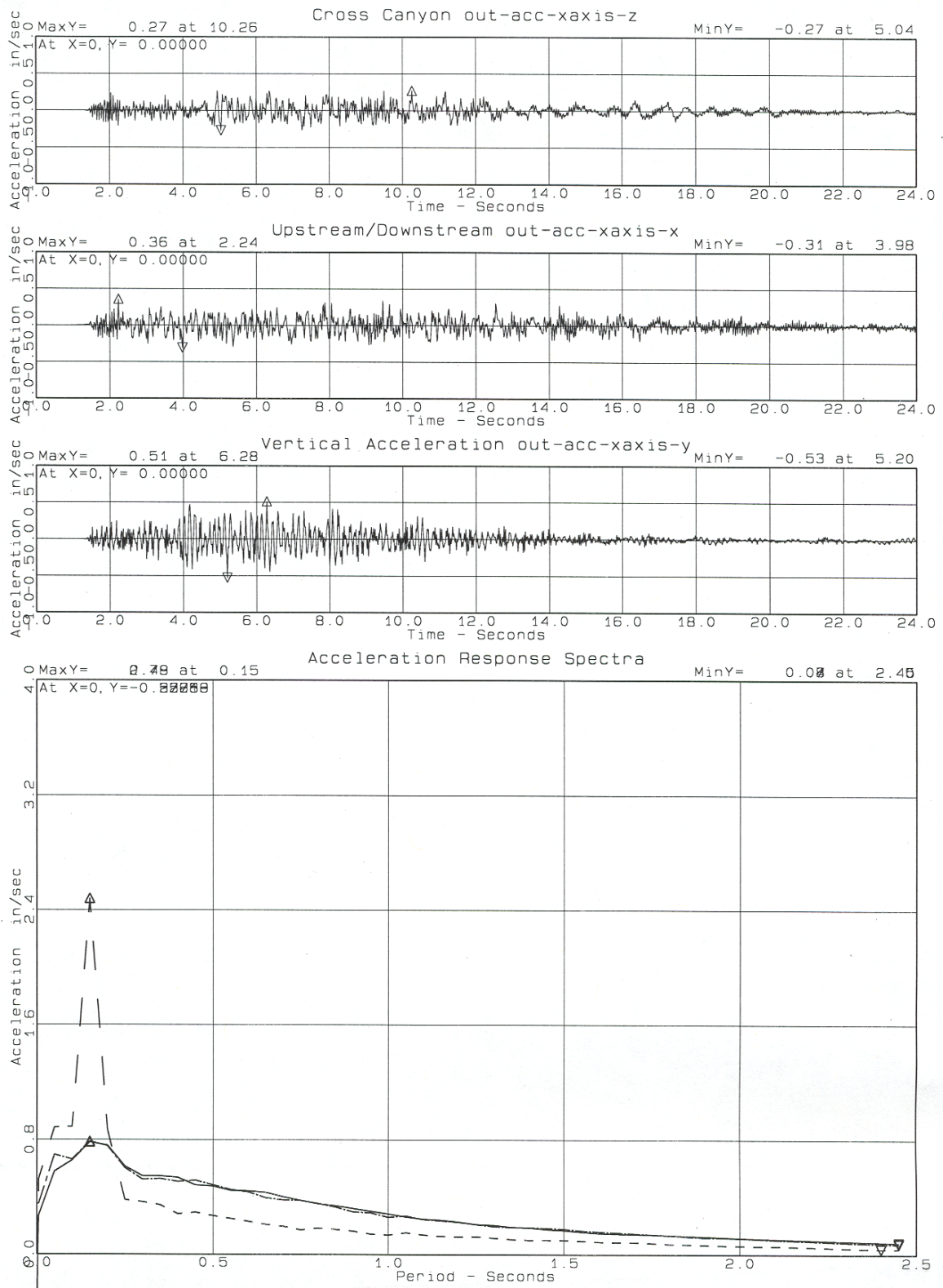


Figure 3.1.3-9 Time history accelerations and response spectra for surface ground motions at the center of the model—with damping.

The average maximum accelerations are compared in the three bar charts shown in figure 3.1.3-10. The blue bars represent the model with a flat surface and no damping applied. The red bars represent the model with a flat surface and an $\alpha = 3.25$ applied. The pale blue bars represent the input motions. As expected, the addition of damping reduced the surface accelerations in all three directions. In the x-direction the maximum average acceleration at the center of the surface of the model went from 0.7 to 0.34g. In the y-direction, the maximum average acceleration at the center of the surface of the model went from 0.79 to 0.52g. In the z-direction, the maximum average acceleration at the center of the surface of the model went from 0.52 to 0.27g. In both cases, the y-accelerations were characterized by a predominant frequency of approximately 6.67 Hz. This is most easily seen in the response spectra plots on figures 3.1.3-5 and -9. This phenomenon is due to the fact the input motions are applied along a horizontal plane. The motions in the horizontal directions are allowed to travel up through the foundation as shear waves, while the vertical accelerations are applied as p-waves or compression waves.

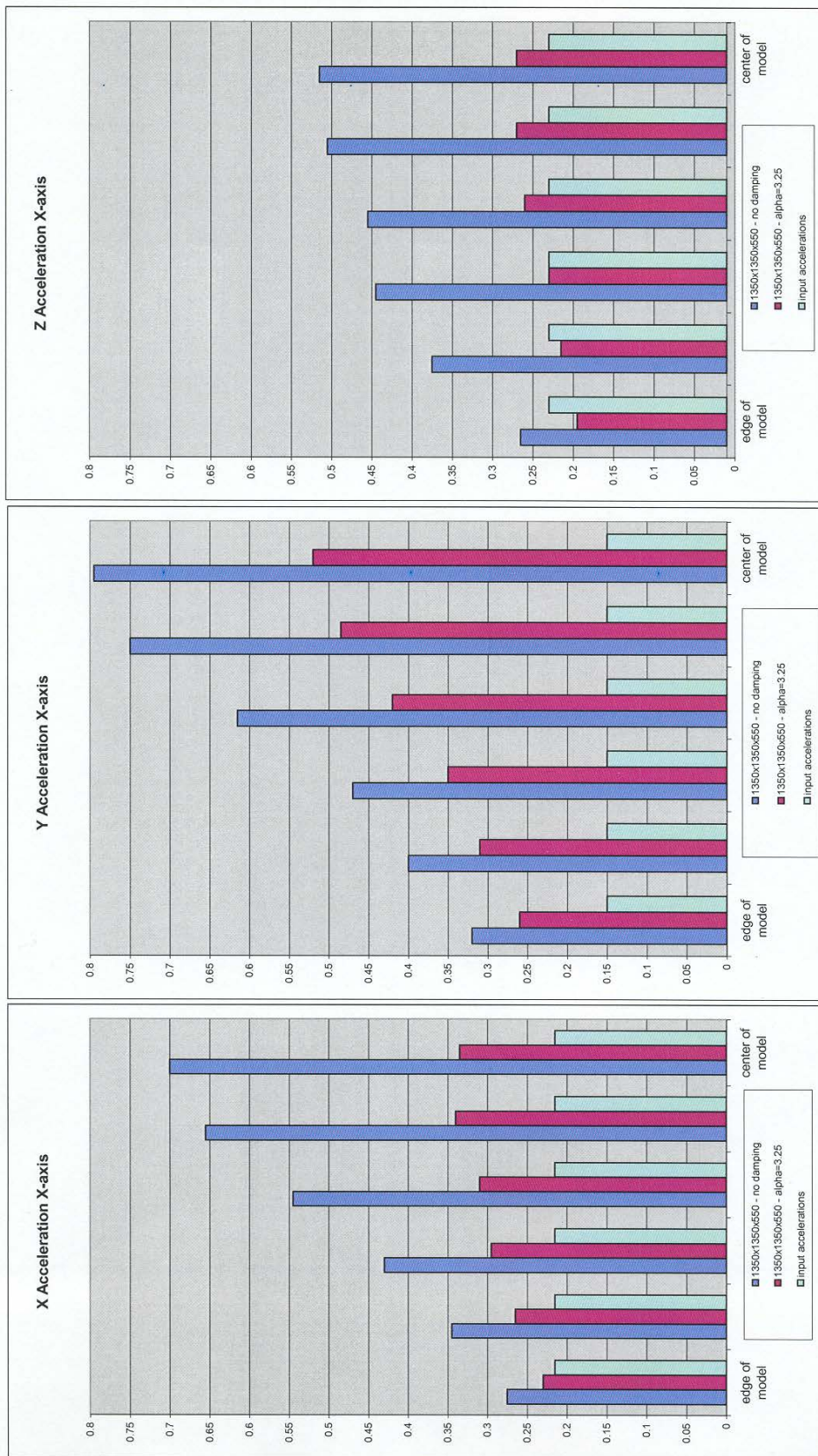


Figure 3.1.3-10 Comparison of average maximum accelerations.

3.1.4 Foundation – Site specific motions (Addition of topography)

The 1,350- by 1,350- by 550-foot model discussed above was modified to include the surface topography. The model is shown in figure 3.1.4-1. The nodes in the canyon remain approximately 450- to 500-feet above the plane of input application. The nodes on the abutment rise a maximum of 250-feet above this elevation. A mass damping factor of $\alpha = 3.25$ was applied and the material properties remained the same. Because the canyon lies along the x-axis and the topography changes lie along the z-axis, accelerations at the surface center along the z-axis were considered, as well as along the x-axis. Time history plots of the accelerations at the surface along the x-axis are shown in figures 3.1.4-2 through -4 and time history plots of accelerations along the z-axis are shown in figures 3.1.4-5 through -7. Time history plots of the accelerations in all three directions and their response spectra are shown in figure 3.1.4-8a. Time history plots of the velocities in all three directions and their response spectra are shown in figure 3.1.4-8b. Time history plots of the displacements in all three directions and their response spectra are shown in figure 3.1.4-8c. Figure 3.1.4-9 compares the average maximum accelerations for the free-field motions with a damping value $\alpha = 3.25$ to the average maximum accelerations for the model with topography and the same damping value.

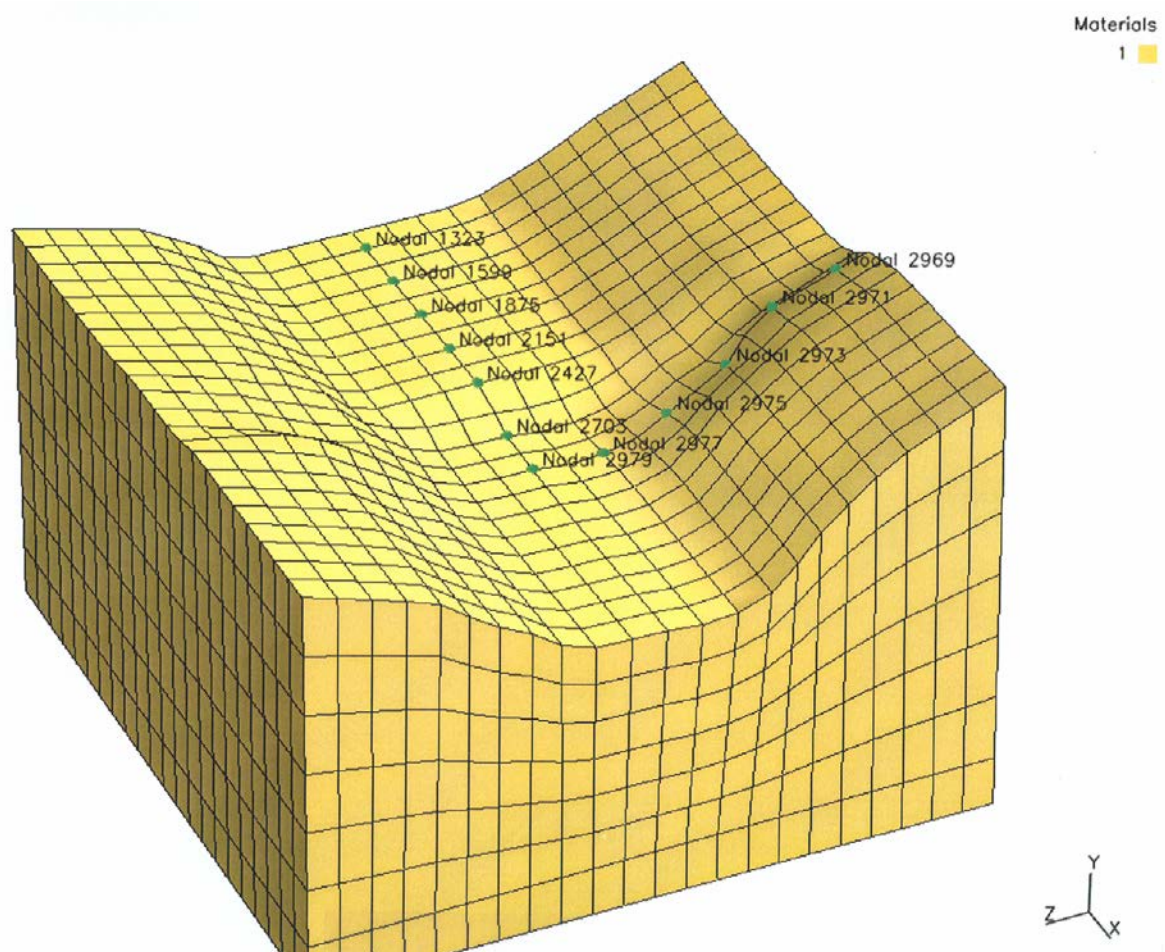


Figure 3.1.4-1 Model for site specific motions (1,350 by 1,350 by 550 feet).

Spat Vary EQ -topo CF 1350x1350x550- x-Acc (xaxis)
Folsom- decon-mce-cha

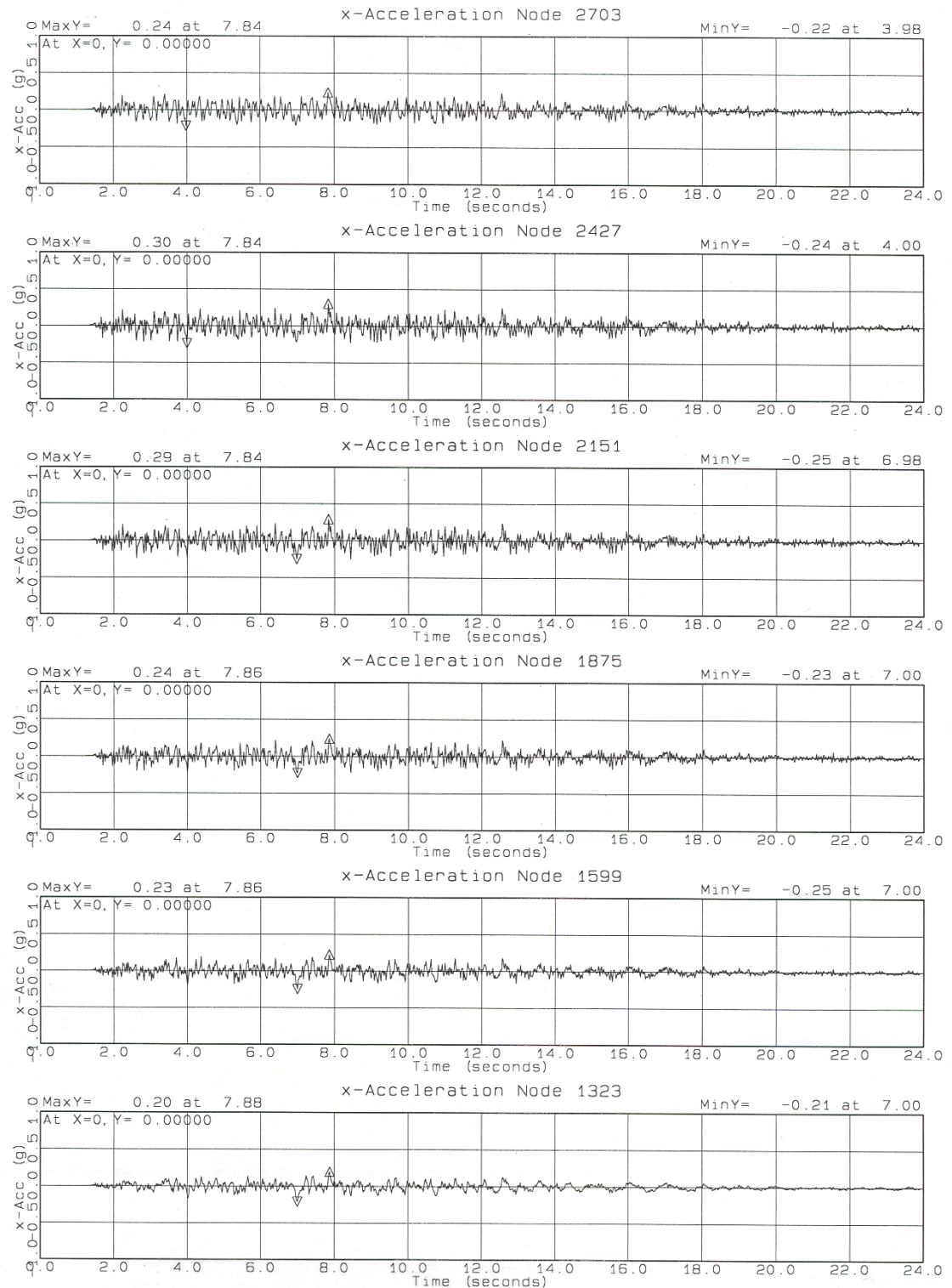


Figure 3.1.4-2 X-direction surface accelerations along the x-axis.

Spat VaryEQ topo CF-1350x1350x550- y-Acc (xaxis)
Folsom- decon-mce-cha

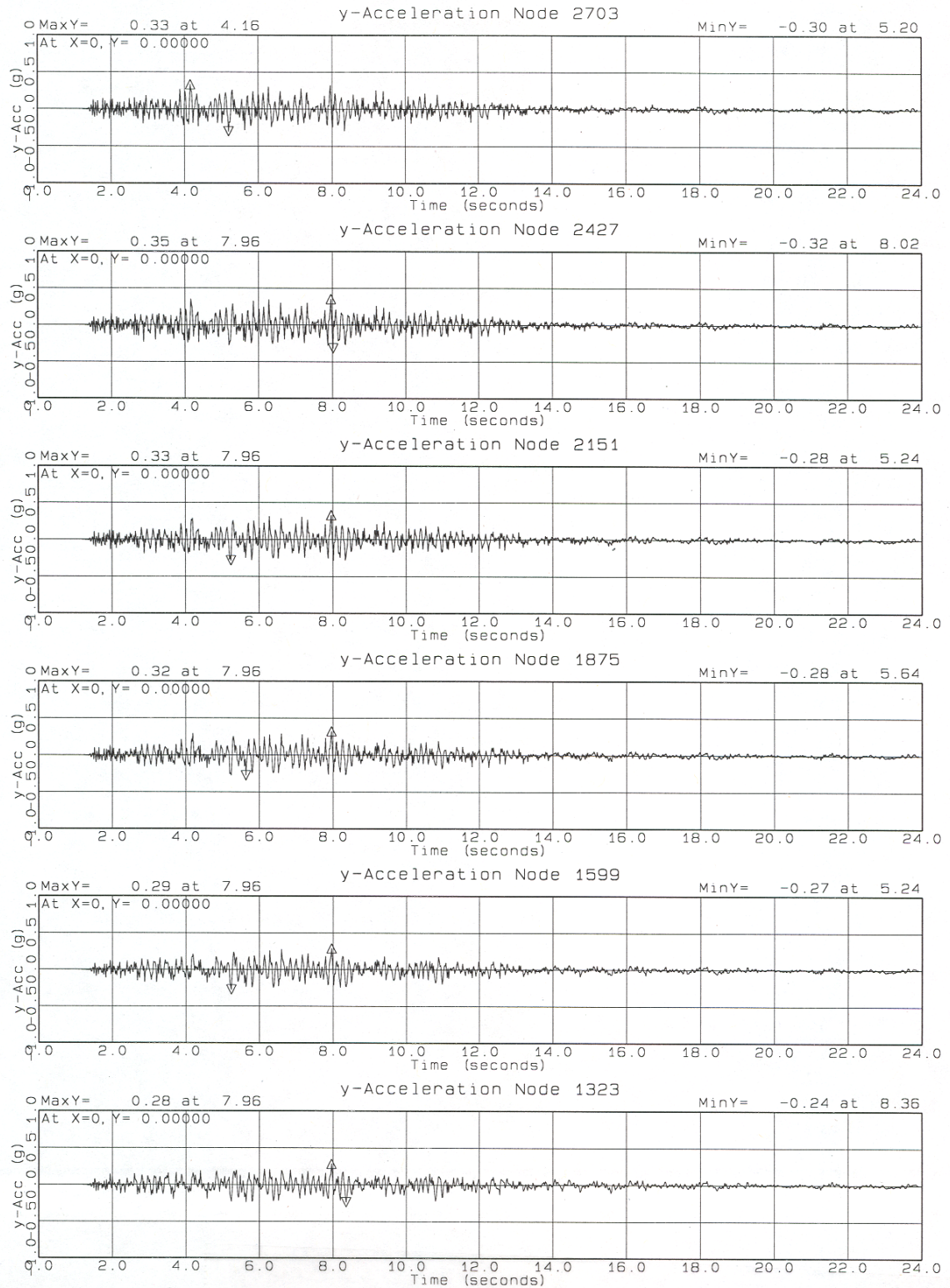


Figure 3.1.4-3 Y-direction surface accelerations along the x-axis.

Spat VaryEQ topo CF -1350x1350x550- z-Acc (xaxis)
Folsom- decon-mce-cha

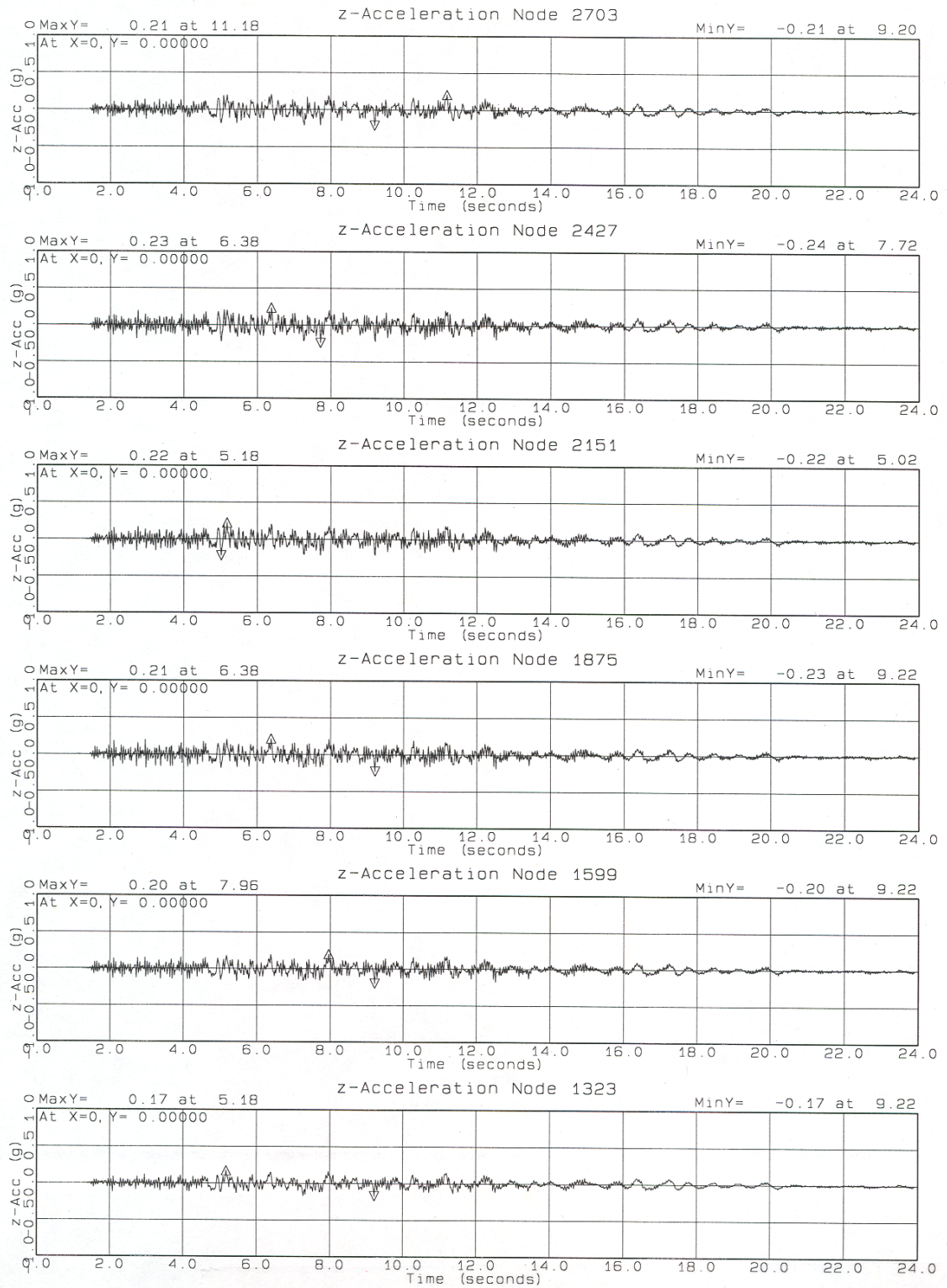


Figure 3.1.4-4 Z-direction surface accelerations along the x-axis.

Spat Vary EQ-1350x1350x550-x-Acc (zaxis)
Folsom- decon-mce-cha

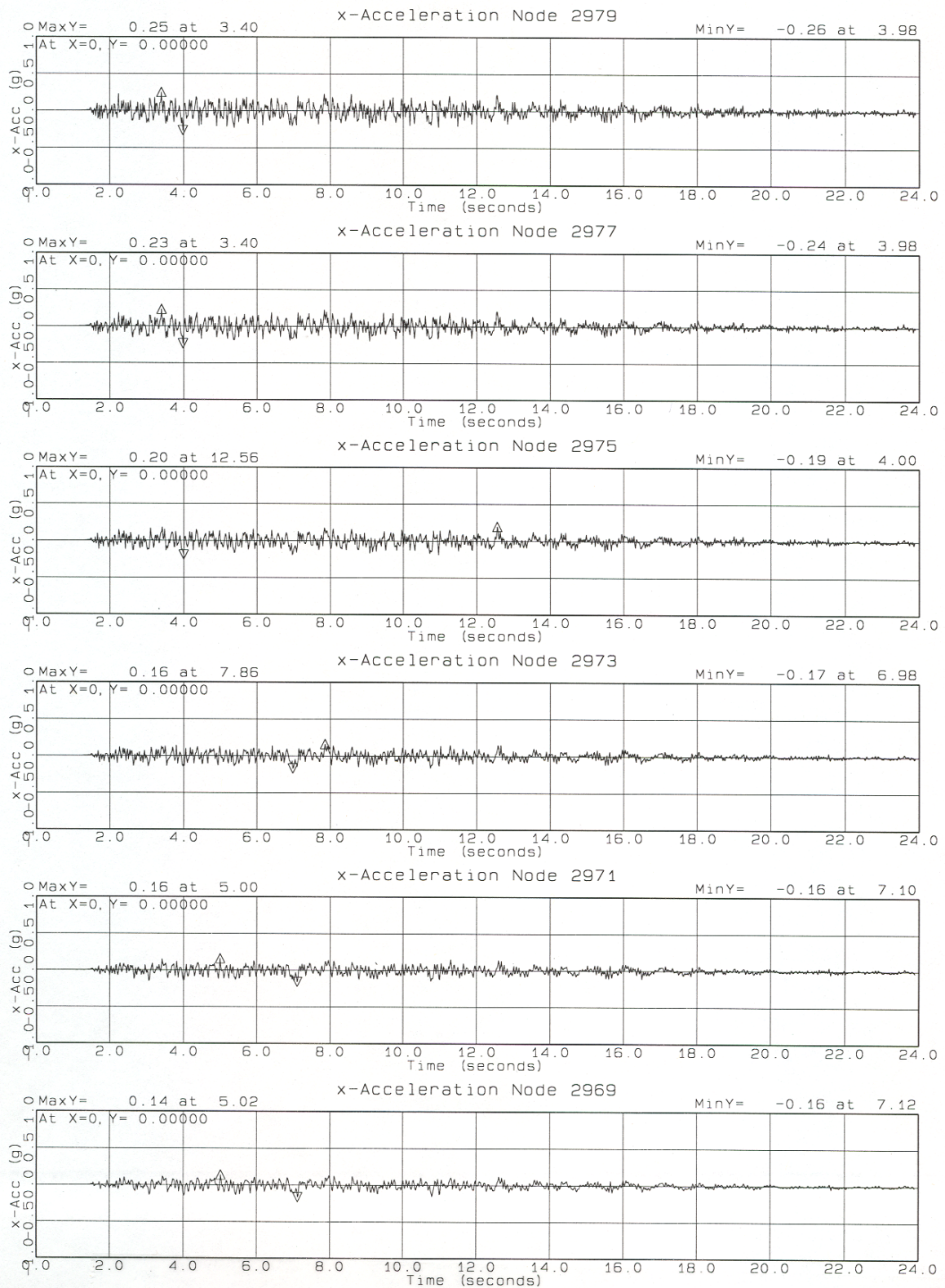


Figure 3.1.4-5 X-direction surface accelerations along the z-axis.

Spat Vary EQ-1350x1350x550-y-Acc (zaxis)
Folsom- decon-mce-cha

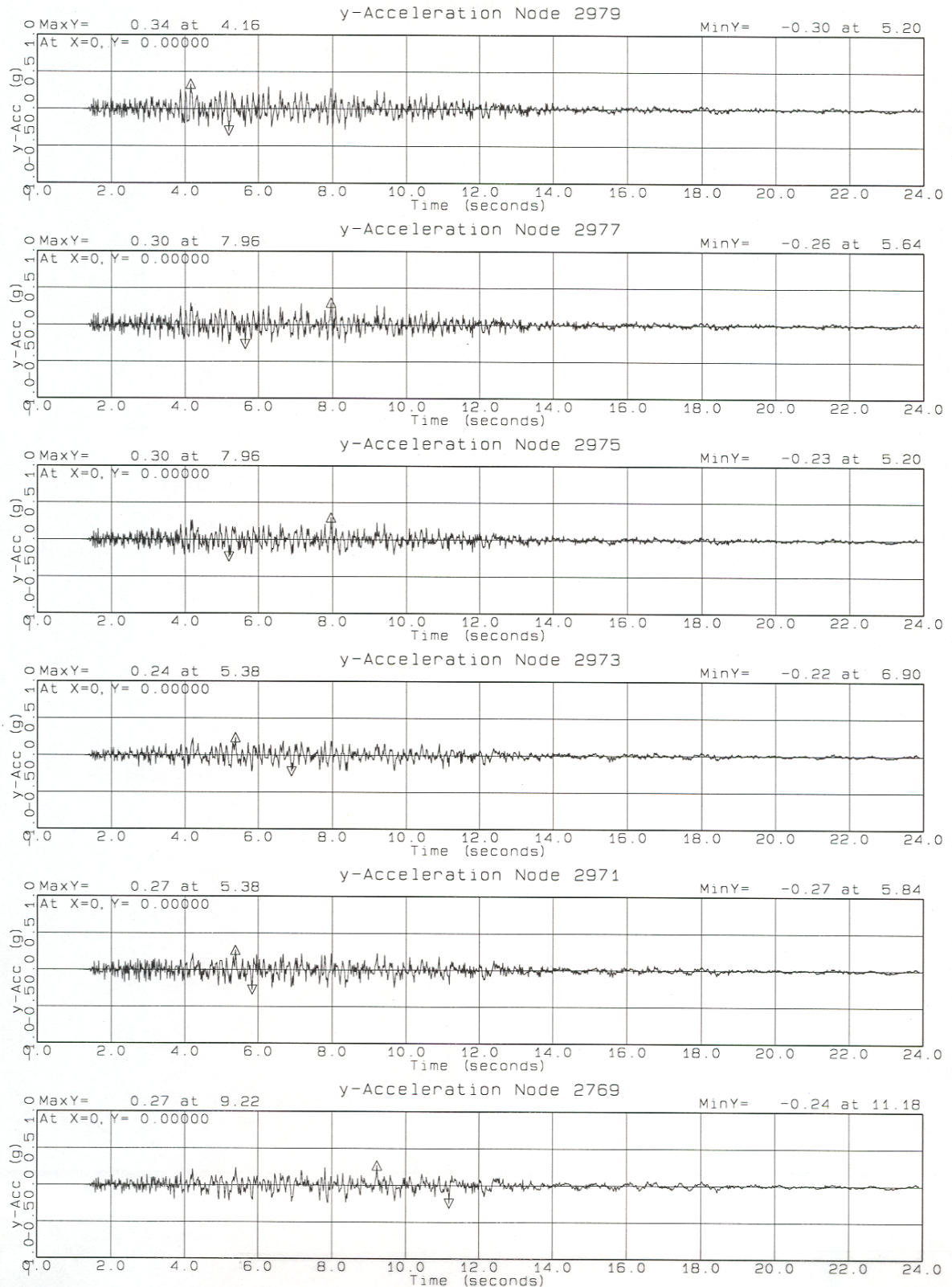


Figure 3.1.4-6 Y-direction surface accelerations along the z-axis.

Spat Vary EQ -1350x1350x550- z-Acc (zaxis)
Folsom- decon-mce-cha

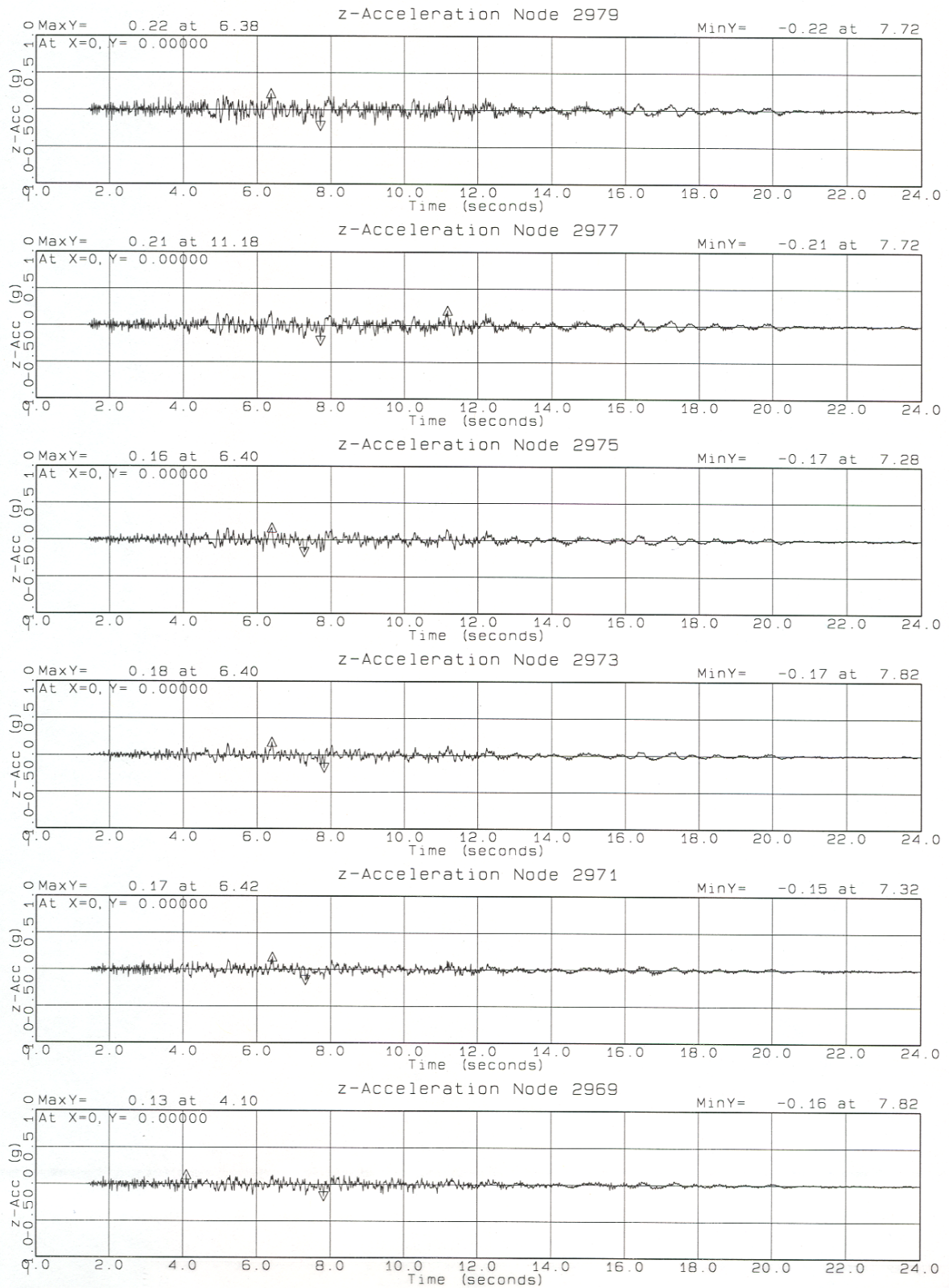


Figure 3.1.4-7 Z-direction surface accelerations along the z-axis.

Found Stdy-topo CF -a=3.25- decon MCE-cha
Ground Motions and Response spectra

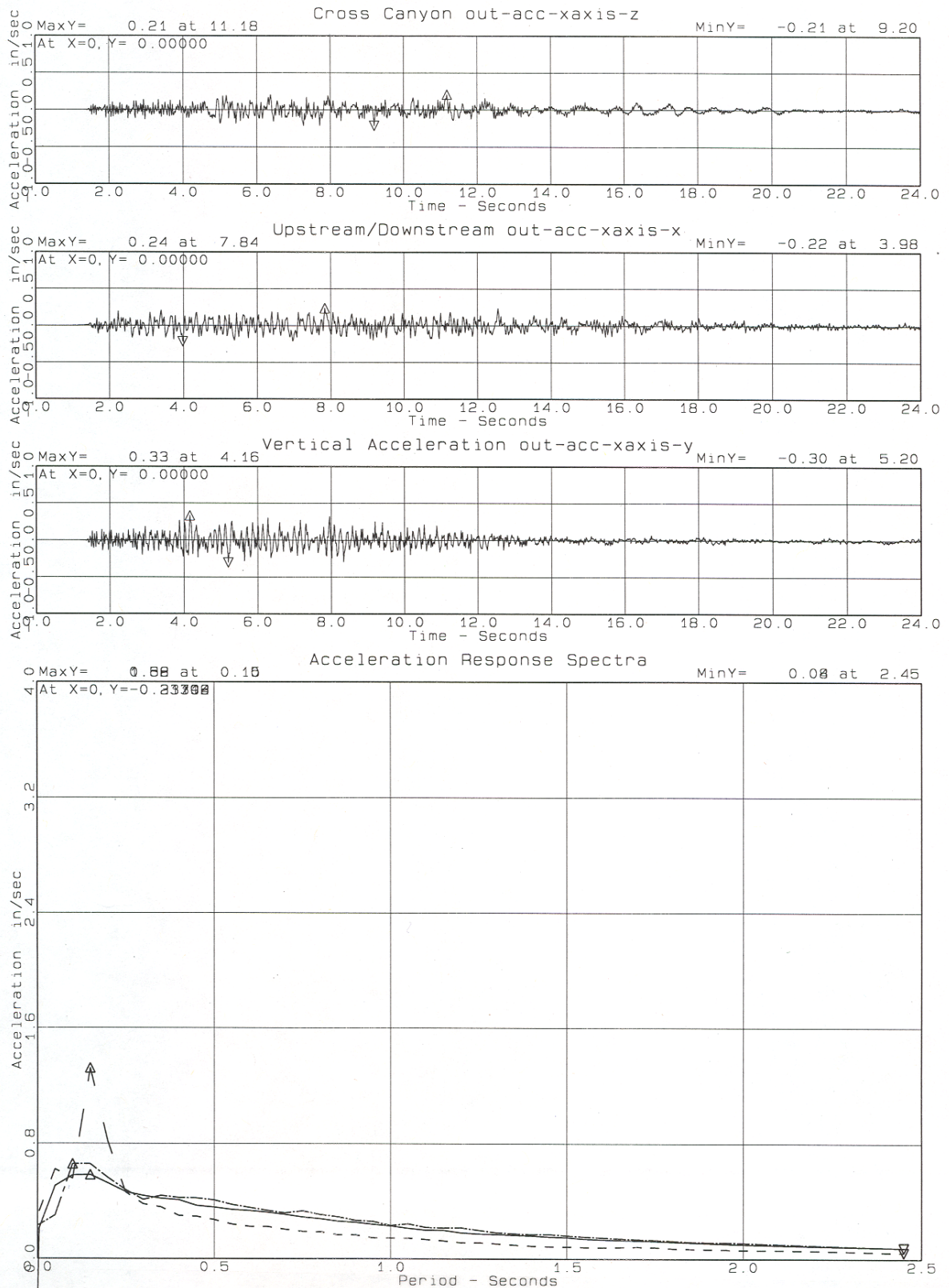


Figure 3.1.4-8a Time history accelerations and response spectra for surface ground motions at the center of the model.

Found Stdy-topo CF -a=3.25- decon MCE-cha
Ground Motions and Response spectra

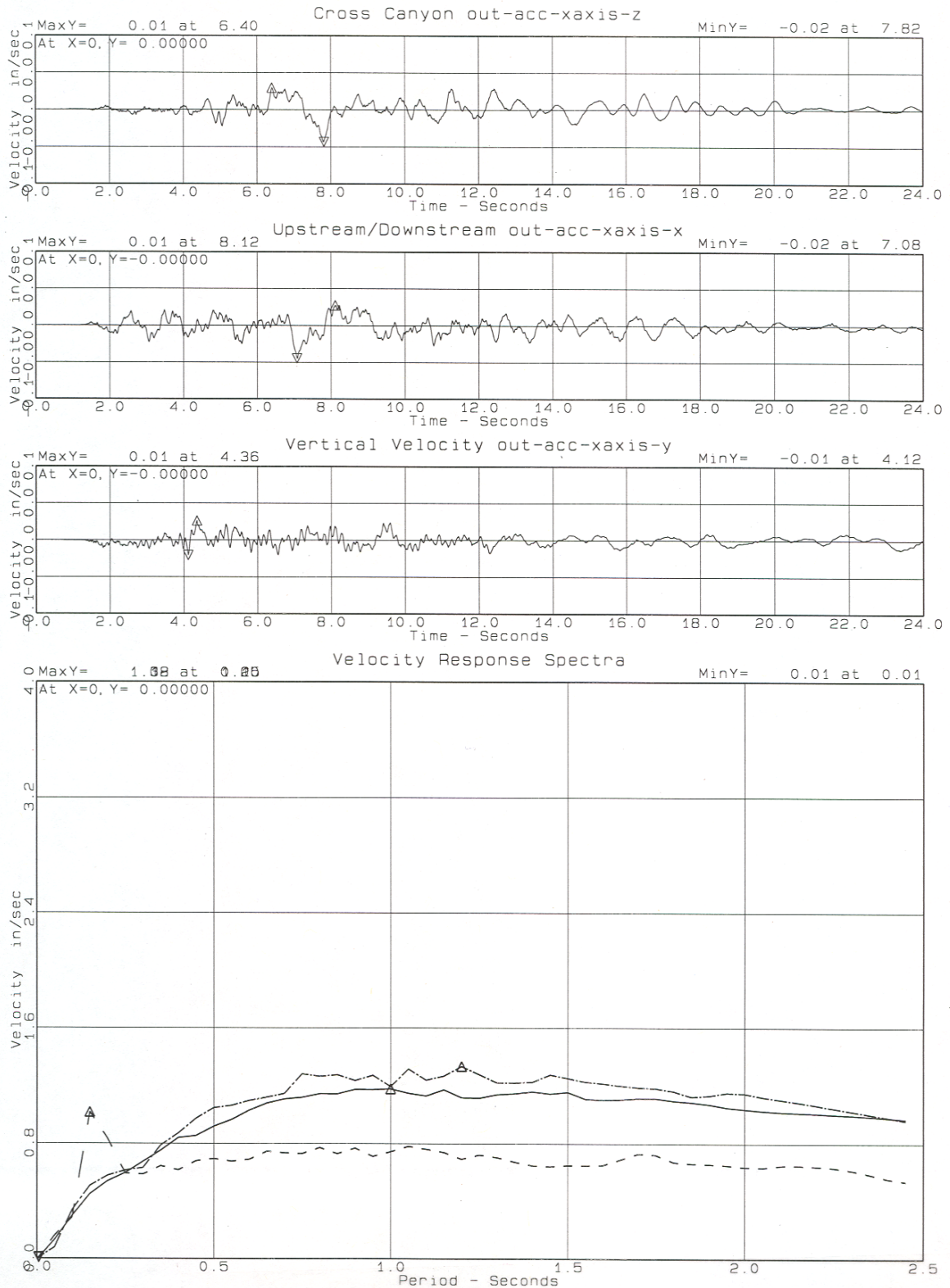


Figure 3.1.4-8b Time history velocities and response spectra for surface ground motions at the center of the model.

Found Stdy-topo CF -a=3.25- decon MCE-cha
Ground Motions and Response spectra

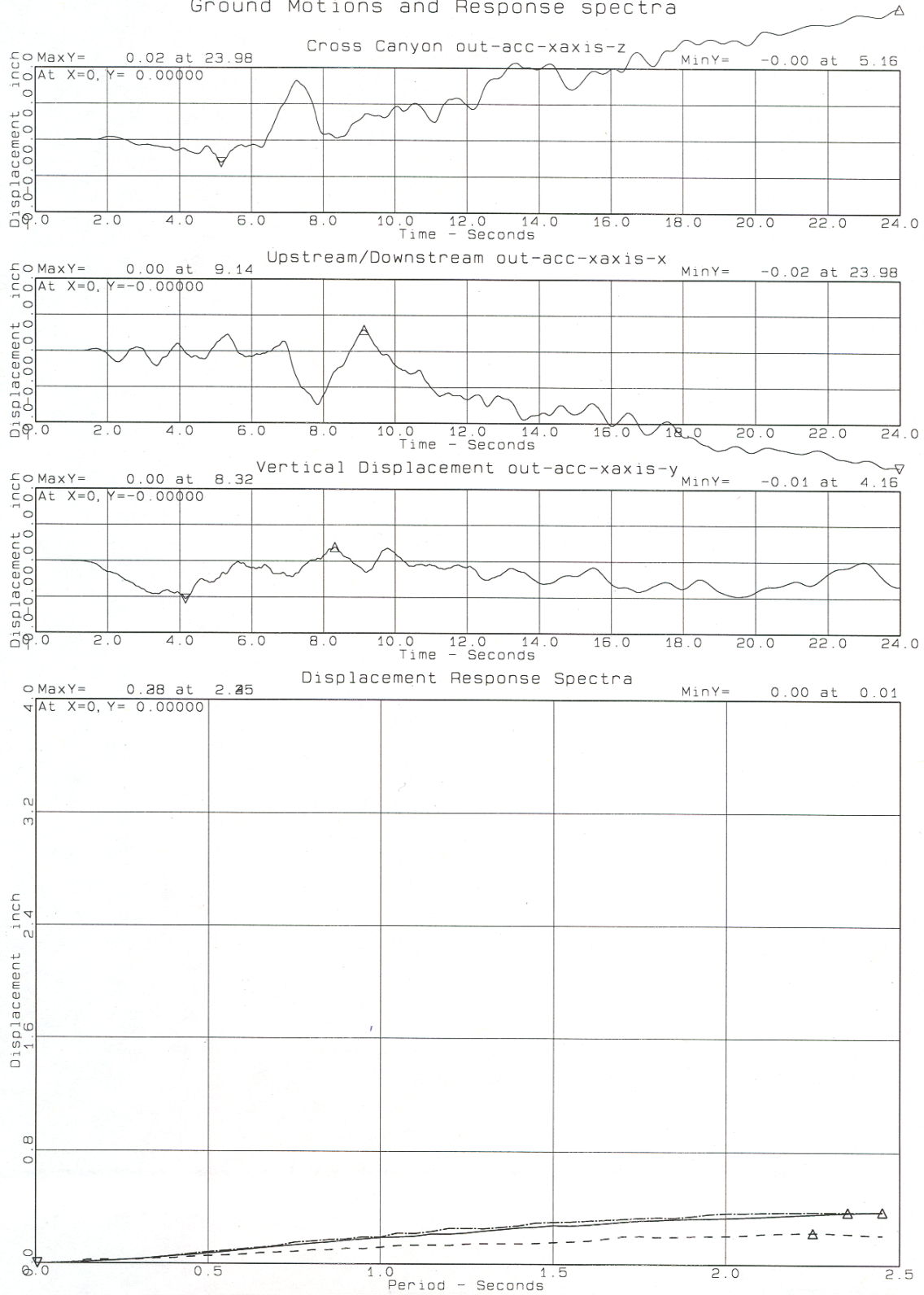


Figure 3.1.4-8c Time history displacements and response spectra for surface ground motions at the center of the model.



Figure 3.1.4-9 Time history accelerations comparisons of average maximum accelerations.

The addition of topography affected the surface accelerations along the x-axis (in the canyon) and the z-axis (along the abutment) differently. Along the x-axis (in the canyon, pale yellow bars on figure 3.1.4-9) the surface accelerations parallel to the canyon (x-direction) were lower than the surface motions in the free-field model analyses, but they were still higher than the input motions. The accelerations along the x-axis near the outside edge, but in the cross canyon direction (z-direction) were lower than the input motions for both the free-field model and the site specific model with the accelerations from the free-field being slightly higher than those from the site specific model. At the center of the model (in the canyon), the maximum average x-acceleration at the surface dropped from 0.34 (free-field) to 0.25g (site specific); the maximum average z-acceleration dropped from 0.27 (free-field) to 0.21g (site specific); and the maximum average y-acceleration dropped from 0.52 (free-field) to 0.31g (site specific).

Along the z-axis (pale orange bars on figure 3.1.4-9), at the edge of the model (on the abutment), the maximum average x-acceleration at the surface dropped from 0.23 (free-field) to 0.15g (site specific); the maximum average z-acceleration dropped from 0.19 (free-field) to 0.14g (site specific); and the maximum average y-acceleration remained 0.26g for the free-field model and the site specific model. In both the x-direction (upstream-downstream) and the z-direction (cross-canyon), the surface accelerations near the outside edges of the site specific model were less than the input motions.

They (vertical) accelerations along the x-axis (in the canyon) and along the z-axis (up the abutment) were markedly reduced with the addition of site specific topography. At the center they dropped from 0.52 (free-field) to 0.32g (site specific).

This study indicates that the transmission of the input accelerations up through the foundation must be considered before the analysis of the dam begins. Currently, the Structural Analysis Group receives a set of deconvolved motions (which are based on the Young's modulus, Poisson's ratio, and density of the foundation) and a set of corresponding surface motions from the Seismotectonic Group. If the deconvolved represent free-field motions, they are input into the free-field foundation model and

measured at the surface to be sure that the ground motions that will be experienced by the dam are comparable to the supplied surface motions. The horizontal and vertical extent of the model must be adjusted so that the resulting surface motions are comparable to the supplied surface motions. If the deconvolved motions are site specific, the topography must be added to the foundation model. It also can be seen that the middle third of the model has fairly consistent surface motions. This canyon was approximately 225-feet deep. This implies that six times the dam height is a reasonable guideline for horizontal extent of the model.

4.0 Gravity Dam/Reservoir/Foundation System

4.1 Introduction

In addition to the determination of foundation size, the effect of damping values and the ratio of elastic modulus of the foundation to the elastic modulus of the concrete (E_f / E_c) were studied. A model of a typical gravity dam and a model of a typical arch dam were used in these studies. The gravity dam is considered in this section.

A 3-dimensional finite element model of a simplified gravity dam was created (see figures 4.1-1 and -2) which included the reservoir, the foundation, and three monoliths of the dam. The contraction joints were modeled using sliding contact surfaces. The effect of friction on these surfaces was determined by assuming values of $\mu = 0.0$ and $\mu = 0.9$. The height of the dam was 150 feet, the crest width was 20 feet, and the crest length was 265 feet. The width at the base was 85 feet and the downstream face slope was 1:2. This model was analyzed for static and dynamic loads. The ground motions used were deconvolved ground motions based on the Cerro Prieto record. The ground motions are plotted in figure 4.1-3 along with the acceleration response spectra at 5-percent damping for the three components.

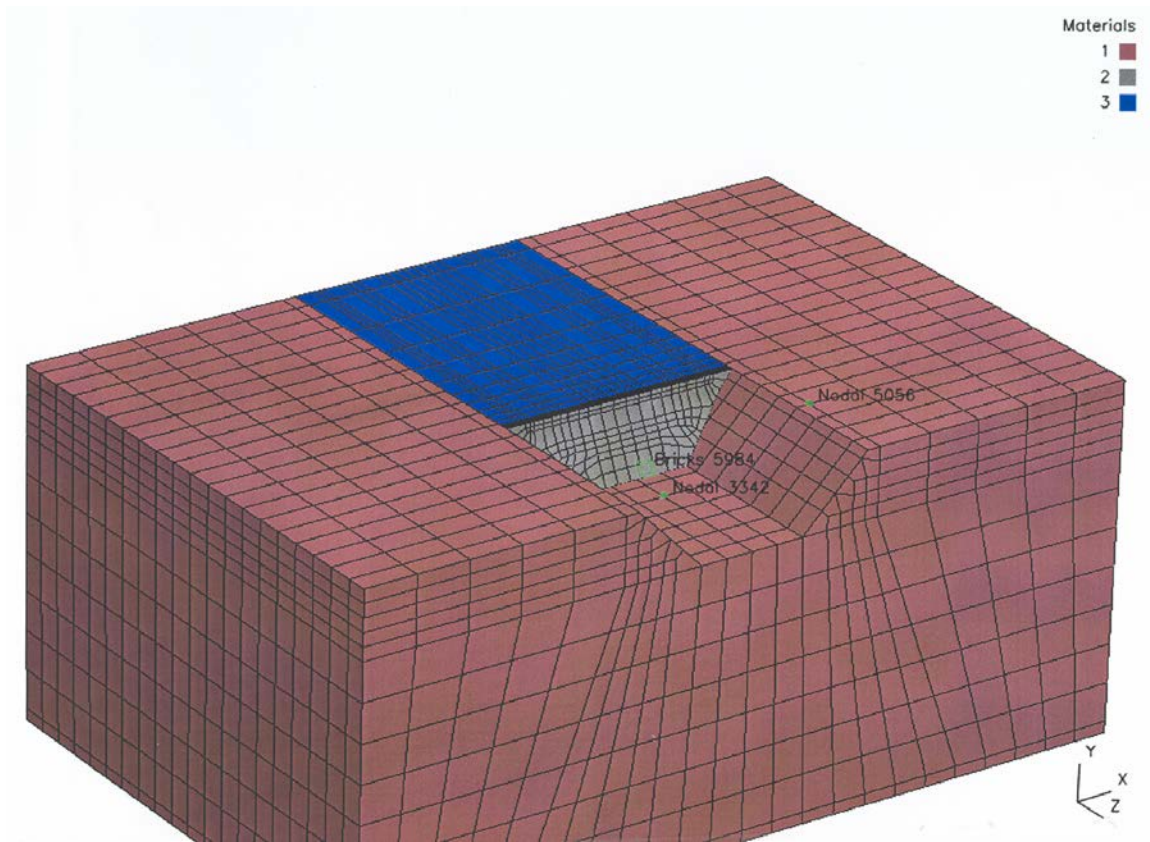


Figure 4.1-1 Model of a typical gravity dam/foundation/reservoir system.

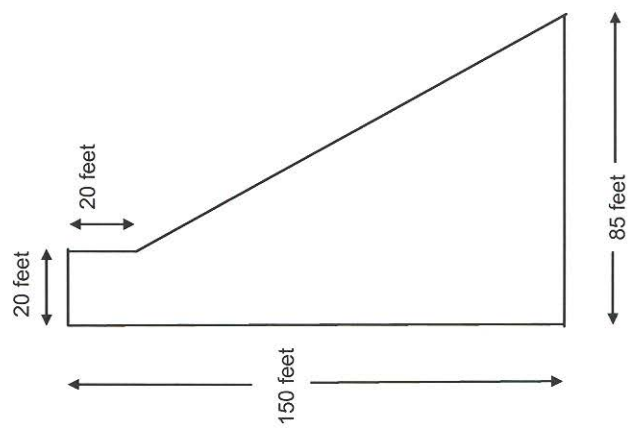
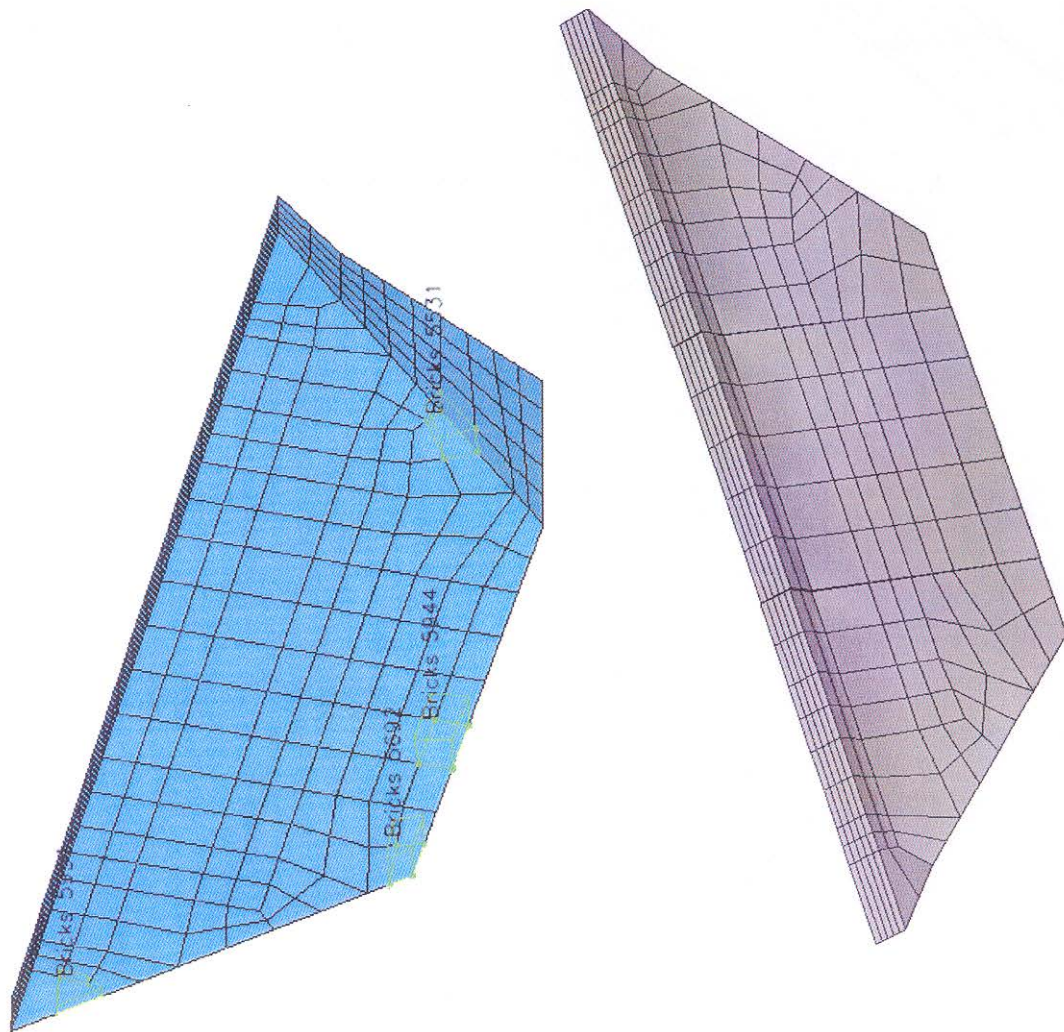


Figure 4.1-2 Model of typical gravity dam.

Foundation Study - deconvolved CerroPrieto-50H Ground Motions and Response spectra

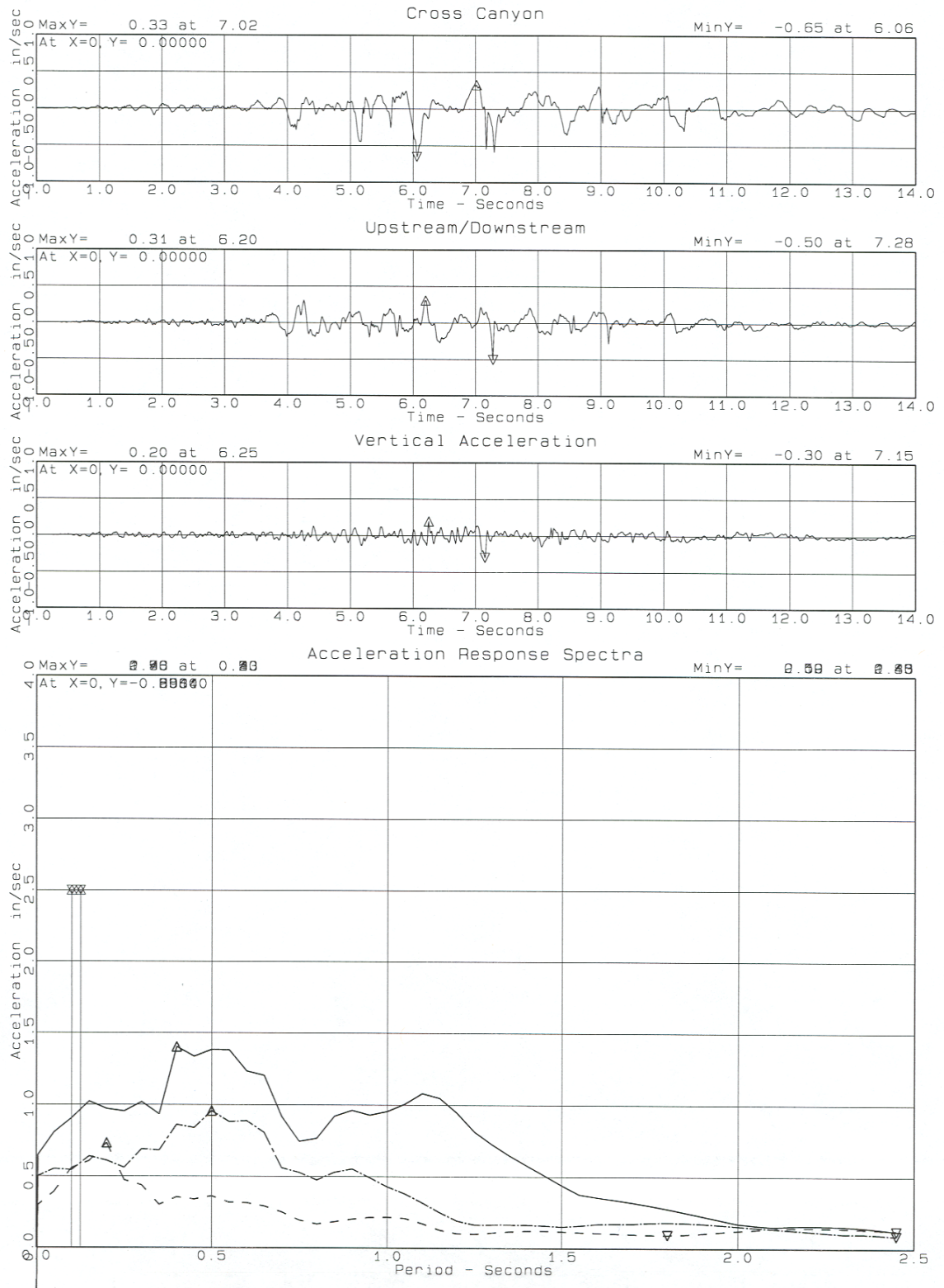


Figure 4.1-3 Deconvolved ground motions based on the Cerro Prieto record and acceleration response spectra for the three components.

The main purpose of the dynamic runs was to evaluate the effect of varying ratios of elastic modulus in the foundation to elastic modulus in the dam (E_f/E_c). Typically for an *EACD-3D-96* (Tan et al., 1996) analysis, a ratio of 1/1.25 is the minimum allowed if the material properties are unknown since lower ratios can over-damp the system. Past analyses have proven that this ratio can greatly affect the stresses calculated in the dam and a ratio of $E_f/E_c = 1/7$ can produce unreasonably low stresses. For these reasons, the ratios considered here were $E_f/E_c = 1/1.25$ (where $E_c = 4 \times 10^6 \text{ lb/in}^2$ and $E_f = 3.2 \times 10^6 \text{ lb/in}^2$) and $E_f/E_c = 1/7$ (where $E_c = 4 \times 10^6 \text{ lb/in}^2$ and $E_f = 0.57 \times 10^6 \text{ lb/in}^2$). Note that the stiffness of the concrete remained unchanged and the stiffness of the foundation varied. In addition, three methods to determine the fundamental frequency and inherent damping in the gravity dam/foundation/reservoir system were studied.

4.2 Damping studies

It is very important that damping values be applied correctly in the gravity dam/reservoir/foundation models. Currently at Reclamation, the mass damping component of Rayleigh damping is the only method for applying damping. In *DYNA*, stiffness damping slows the analysis too much to be practical. The way in which mass damping affects the behavior of the system must be understood. Figures 4.2-1 to -3 are plots of Rayleigh mass damping for three values of α : 1, 2, and 4. The percent of critical damping for a given frequency can be determined from these plots. For example, for a frequency of 1 Hz, the percent of critical damping varies from 7.96 ($\alpha = 1$) to 15.92 ($\alpha = 2$) to 31.83 percent ($\alpha = 4$). It is apparent that over-damping could occur if the user is not careful.

Calculations for Rayleigh Damping Coefficients

		Input % Critical Damping for The 2 Frequencies You Want in the Yellow Areas:			
		cps	Rad/sec	Percent	Decimal
Uncracked	Freq 1:	0.500	3.142	% Damp 1:	10.00
	Freq 2:	30.000	188.496	% Damp 2:	10.00
Solve simultaneous equations:		Alpha + w ² * Beta = 2*w*Epsilon			
		Alpha + 9.87 Beta = 0.6293185			
		Alpha + 35530.58 Beta = 37.699112			
Uncracked		Freq Incentr 1			
Beta = 0.000000 (stiffness)					
Alpha = 1.000000 (mass)					
Period	Freq vs % Damping		Mass		
	Period	Damping	Stiffness	Damping	
2.000	0.50	15.92	0.00	15.92	
1.000	1.00	7.96	0.00	7.96	
0.500	2.00	3.98	0.00	3.98	
0.333	3.00	2.65	0.00	2.65	
0.250	4.00	1.99	0.00	1.99	
0.200	5.00	1.59	0.00	1.59	
0.167	6.00	1.33	0.00	1.33	
0.143	7.00	1.14	0.00	1.14	
0.125	8.00	0.99	0.00	0.99	
0.111	9.00	0.88	0.00	0.88	
0.100	10.00	0.80	0.00	0.80	
0.091	11.00	0.72	0.00	0.72	
0.083	12.00	0.66	0.00	0.66	
0.077	13.00	0.61	0.00	0.61	
0.071	14.00	0.57	0.00	0.57	
0.067	15.00	0.53	0.00	0.53	
0.063	16.00	0.50	0.00	0.50	
0.059	17.00	0.47	0.00	0.47	
0.056	18.00	0.44	0.00	0.44	
0.053	19.00	0.42	0.00	0.42	

freq damping
3.57 13
3.84 8

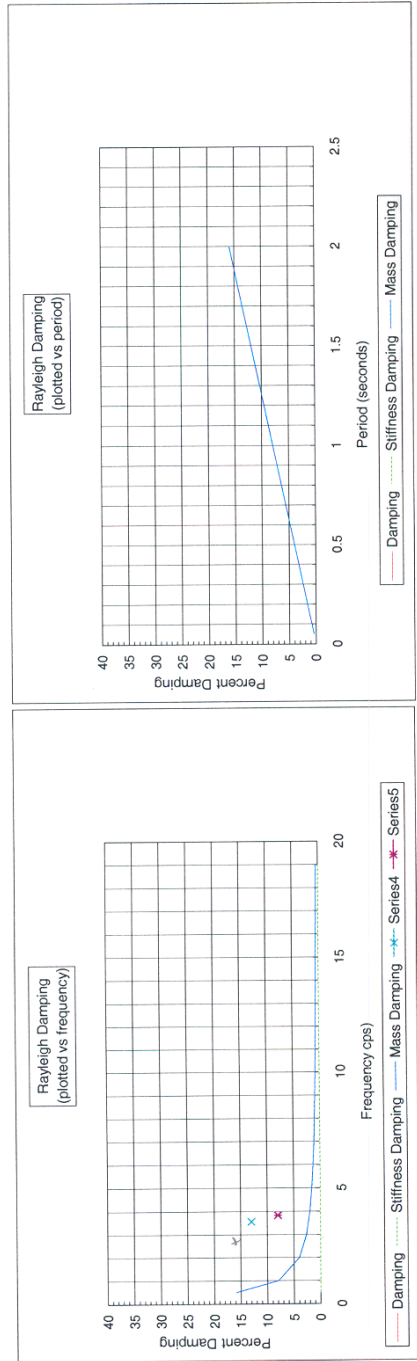


Figure 4.2-1 Rayleigh mass damping, $\alpha = 1$.

Calculations for Rayleigh Damping Coefficients

Input % Critical Damping for The 2 Frequencies You Want in the Yellow Areas:						
Uncracked		cps	Rad/sec	Percent	Decimal	
Freq 1:		0.500	3.142	% Damp 1:	10.00	0.100
Freq 2:		30.000	188.496	% Damp 2:	10.00	0.100
Solve simultaneous equations:						
		Alpha + w^2 *		Beta = 2*w*Epsilon		
		Alpha +		9.87 Beta = 0.6283185		
		Alpha +		35530.58 Beta = 37.699112		
Uncracked						
Beta =		0.000000 (stiffness)		Freq Incent		1
Alpha =		2.000000 (mass)				
Period	Freq vs % Damping	Stiffness		Mass		
		Damping	Damping	Damping	Damping	
2.000	0.50	31.83	0.00	31.83		
1.000	1.00	15.92	0.00	15.92		
0.500	2.00	7.96	0.00	7.96		
0.333	3.00	5.31	0.00	5.31		
0.250	4.00	3.98	0.00	3.98		
0.200	5.00	3.18	0.00	3.18		
0.167	6.00	2.65	0.00	2.65		
0.143	7.00	2.27	0.00	2.27		
0.125	8.00	1.99	0.00	1.99		
0.111	9.00	1.77	0.00	1.77		
0.100	10.00	1.59	0.00	1.59		
0.091	11.00	1.45	0.00	1.45		
0.083	12.00	1.33	0.00	1.33		
0.077	13.00	1.22	0.00	1.22		
0.071	14.00	1.14	0.00	1.14		
0.067	15.00	1.06	0.00	1.06		
0.063	16.00	0.99	0.00	0.99		
0.059	17.00	0.94	0.00	0.94		
0.056	18.00	0.88	0.00	0.88		
0.053	19.00	0.84	0.00	0.84		
				freq	damping	
				3.85	11	

freq damping
3.85 11

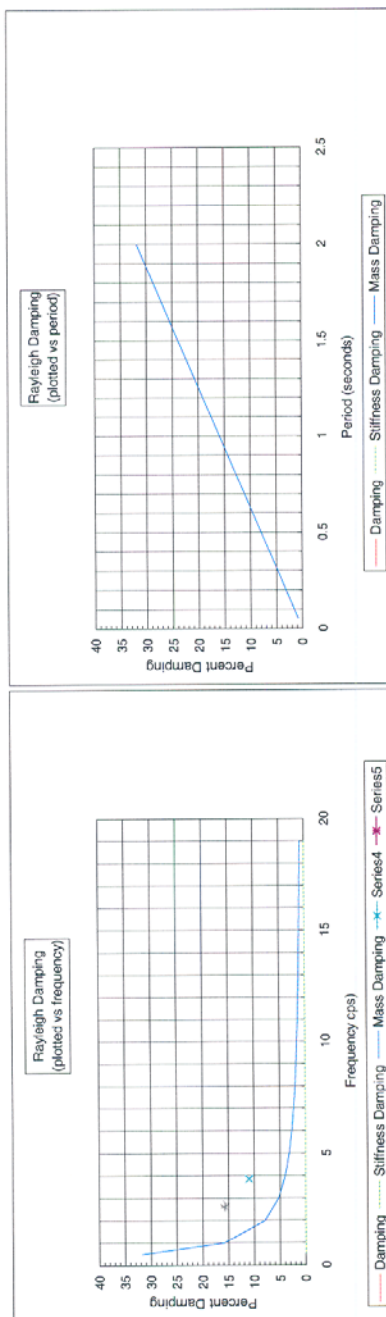


Figure 4.2-2 Rayleigh mass damping, $\alpha = 2$.

Calculations for Rayleigh Damping Coefficients

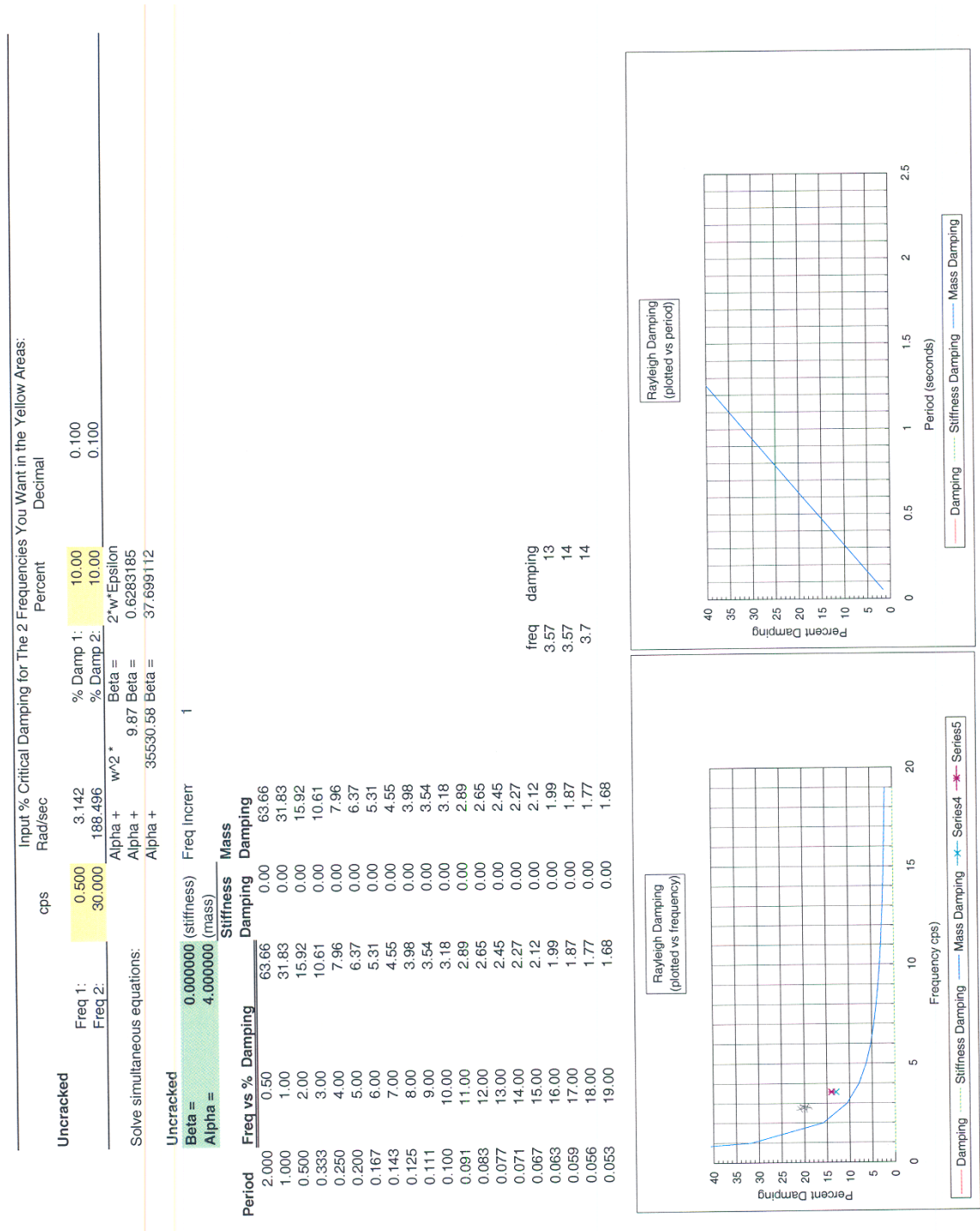


Figure 4.2-3 Rayleigh mass damping, $\alpha = 4$.

Rayleigh mass damping applies to all components of the system when it is used. The inherent damping of the gravity dam/reservoir/foundation system cannot be directly determined from the finite element model by extracting eigenvalues in an implicit software, like *NIKE* due to the high level of non-linearity of the model. For this reason, three methods of determining the fundamental frequency and inherent damping of the gravity dam/reservoir/foundation system (with an $E_t/E_c = 1/1.25$) were considered. In each method, the goal was to excite the system and then allow the vibrations to die down. It was hoped that in the time period in which the system was quieting, a parameter (displacement, velocity, stress) which was oscillating with decreasing amplitude would indicate the frequency and the amount of damping in the system. In real structures, an “effective” viscous damping ratio can be determined by exciting the structure and letting the vibrations decay. In a similar manner, a finite element model can be excited and the vibrations allowed to decay. Figure 4.2-4 shows a plot of an oscillating displacement with decreasing amplitude. The frequency can be determined by the time difference between two adjacent maximum amplitudes. The percent critical damping can be determined by the amplitude difference between two adjacent maximum amplitudes. (Note: the percent critical damping, ξ , appears on both sides of the equation. On the right side of the equation, for a typical value of $\xi = 0.05$, the square root of $[1 - \xi^2] \approx 1$.)

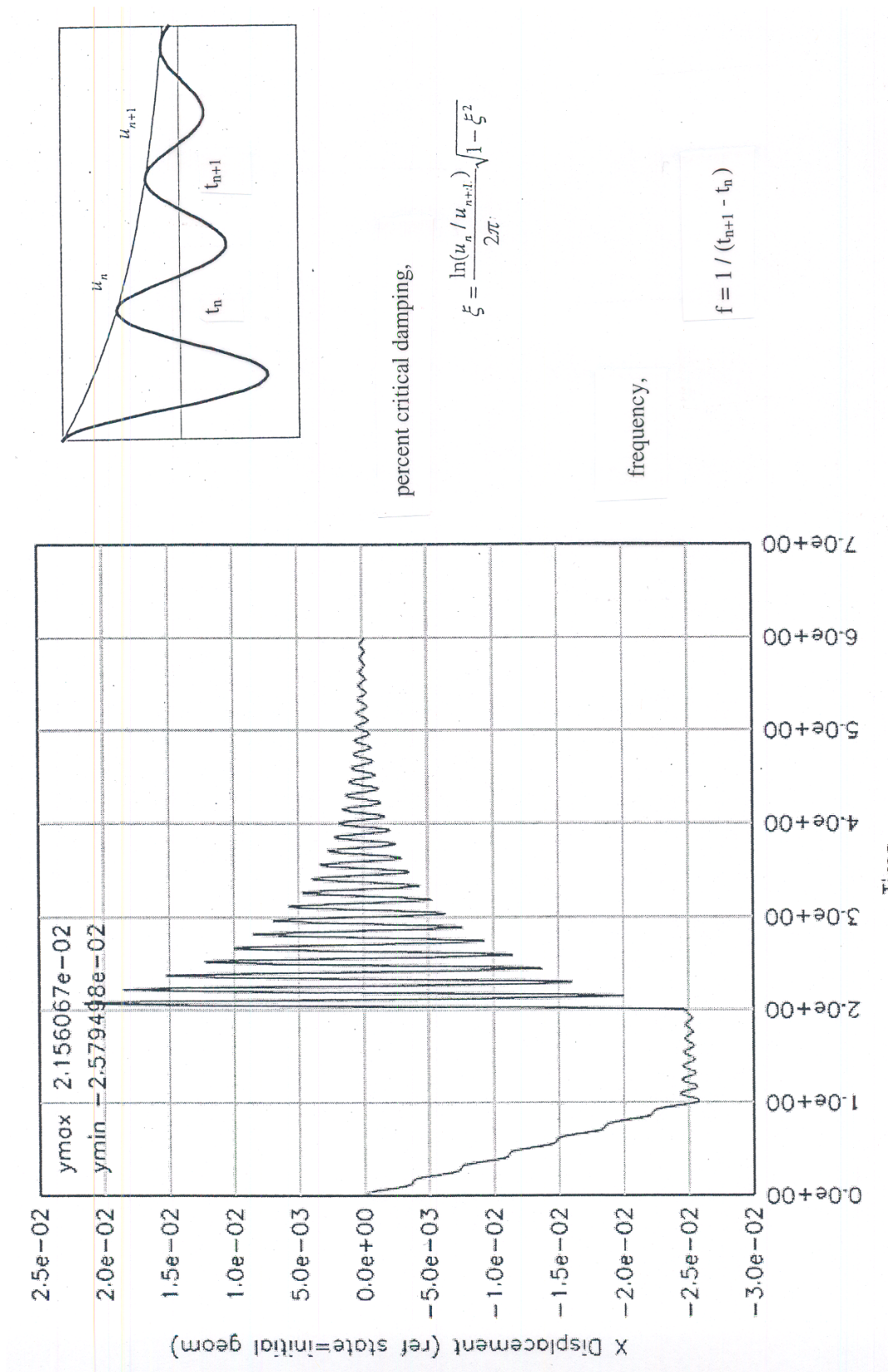


Figure 4.2-4 Effective viscous damping.

Method 1: In the direction of interest, apply an acceleration pulse in the foundation. For the gravity dam model discussed here, the fundamental frequency would be a symmetrical upstream-downstream mode so the acceleration pulse would be applied in the z-direction (upstream-downstream). Apply the pulse in a 1 s interval and allow the system to quiet for at least 2 s.

Plots of the z-displacement of a node at the crest are shown on figure 4.2-5 for the various parameter changes (the parameters which were varied include application of friction with a 0.9 coefficient of friction on the sliding surfaces representing the contraction joints and variation in the applied damping of $\alpha = 1, 2,$ and 4). From this figure, it is apparent that the displacements in the z-direction decay, but not in a uniform manner. This is due to the interaction of the waves traveling through the water elements and impacting the upstream face of the dam. This interaction causes the dam to be excited by the impacting waves. Whether or not friction is assumed between the monoliths does not affect the frequency of the response. Only the amplitude is affected. The same is true of the damping assumed. The result of these analyses is that the fundamental frequency of the dam can be determined in this manner, but the inherent amount of damping in the system cannot.

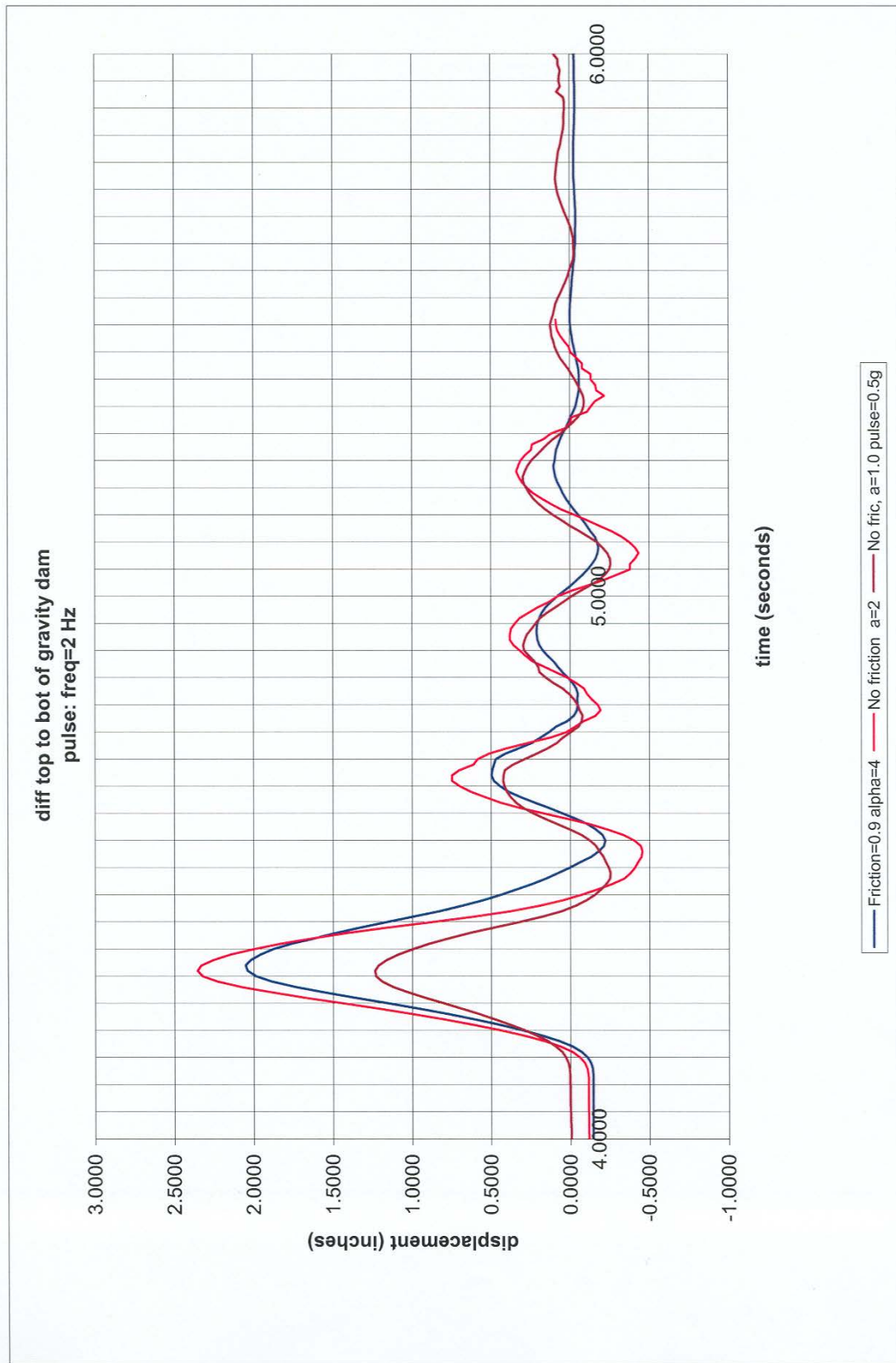


Figure 4.2-5 Decaying oscillations from application of acceleration pulse.

Method 2: In the direction of interest, apply a force at the crest of the dam. For the gravity dam model discussed here, the fundamental frequency would be an upstream-downstream mode so the force would be applied in the z-direction (upstream-downstream). The force was applied in 1 s and removed.

Plots of the z-displacement of a node at the crest are shown in figure 4.2-6 for analyses, assuming two values for damping and three magnitudes of applied forces. Once again, it is seen that the displacements do not decay in a uniform manner. The result of these analyses is that the fundamental frequency of the dam can be determined in this manner, but the inherent amount of damping in the system cannot.

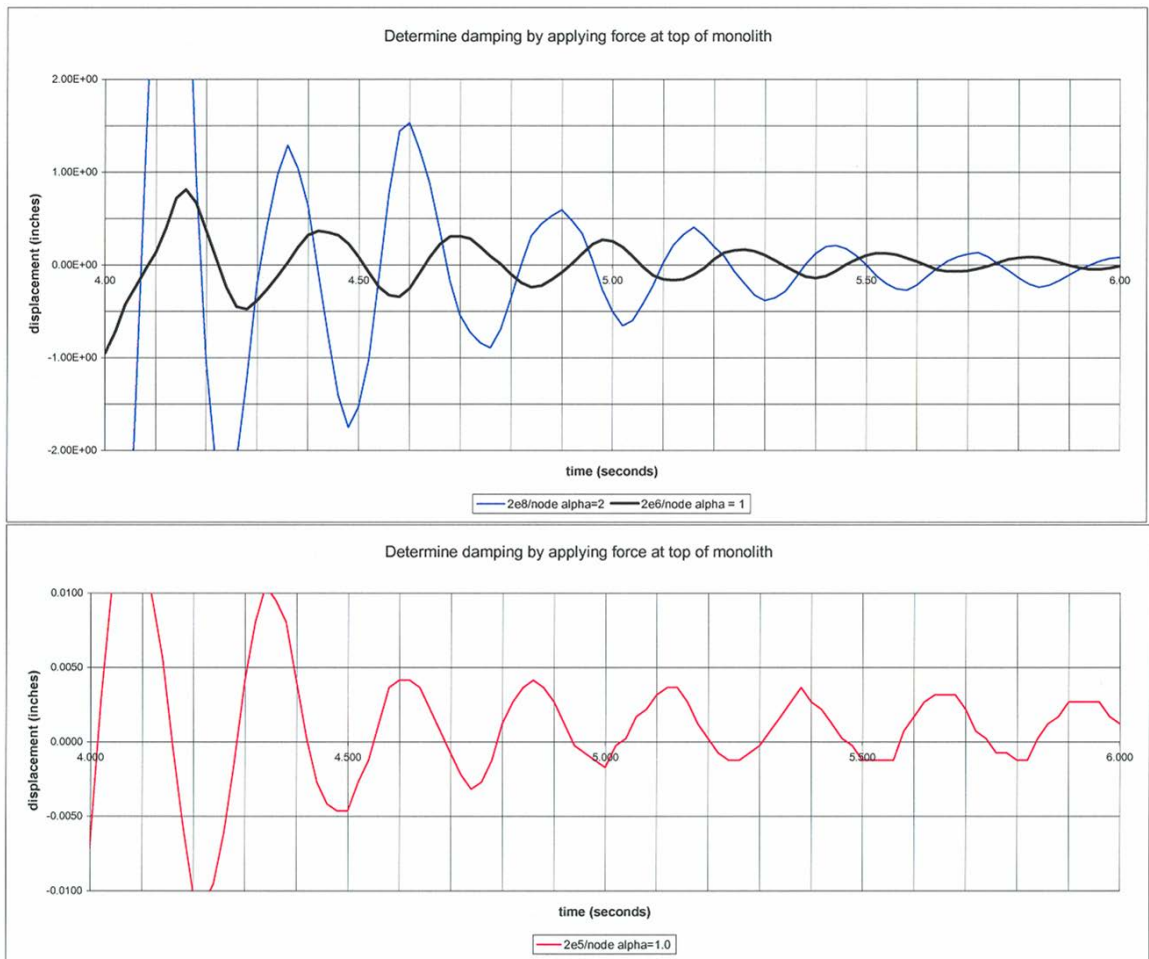


Figure 4.2-6 Decaying oscillations from application of concentrated forces.

Method 3: Apply gravity to the gravity dam/reservoir/foundation system and allow the vibrations to decay. The benefit to this method is that additional analyses in which the acceleration pulse or the forces are applied are not needed. The gravity load can be applied, quiet time can be allotted to allow any vibrations to decay, and then the dynamic loads can be applied.

Plots of the z-displacement of a node at the top of the dam are plotted for the case in which no damping was applied to the model and for the case in which a damping value of $\alpha = 1$ was applied (see figure 4.2-7). In the case of no damping, the vibrations are seen to begin to die down until the dam is impacted by a wave traveling through the water. When a damping value of $\alpha = 1$ is applied, the waves traveling through the water are damped sufficiently to avoid the impact on the structure. It should also be noted that this displacement is taken for a node at the top of the center monolith which has a clear fundamental upstream-downstream mode which is excited by the static water pressure being applied. The effective damping for adjacent peaks for the undamped model is 2.2 percent. The frequency of the response is 3.57 Hz. For the model with damping applied, the effective damping for two adjacent peaks is 4.9 percent and the frequency is once again 3.57 Hz. From the Rayleigh plot for $\alpha = 1$, at a frequency of 3.57 Hz, the damping value is 2.23 percent. This study implies that the inherent damping is 2.2 percent and when the mass damping is applied, the damping in the system is close to the inherent damping plus the applied damping.

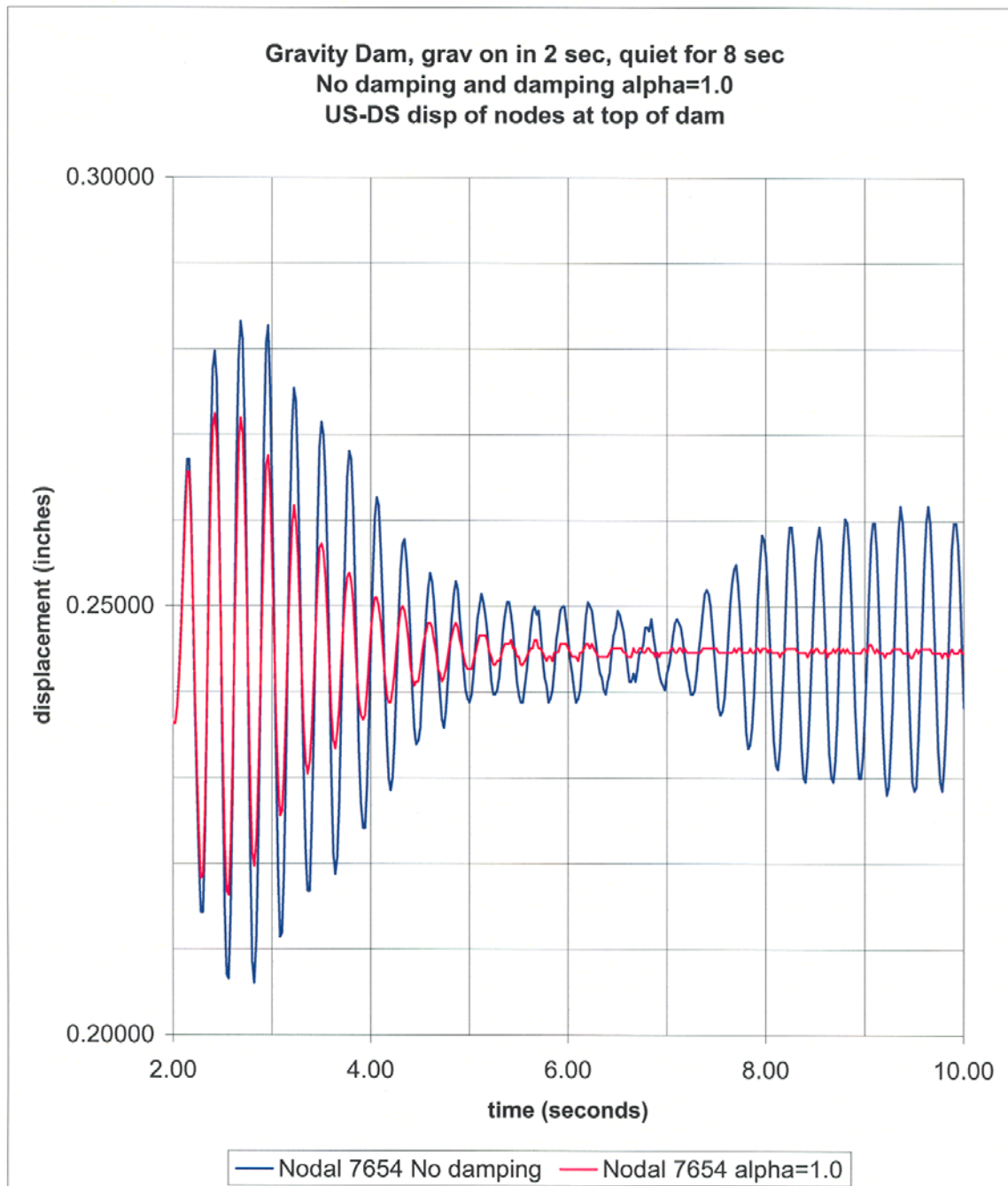


Figure 4.2-7 Decaying oscillations from application of gravity loads.

4.3 Effect of Varying the Ratio of the Foundation Modulus/Dam Modulus

As stated in the introduction, the main purpose of the dynamic runs was to evaluate the effect of varying ratios of elastic modulus in the foundation to elastic modulus in the dam (E_f/E_c). Therefore, following the application of gravity, the acceleration time histories were applied along a horizontal plane 100-feet above the base of the model. This was done for two models, each with a different ratio: $E_f/E_c=1/1.25$ and $E_f/E_c=1/7$. All other variables were kept constant. To determine the effect of the varying ratios, accelerations of two surface nodes (one in the canyon and one on the abutment) were considered. Figure 4.1-1 shows the location of the two nodes (5056 on the abutment; 3343 in the canyon) for which accelerations were extracted. In addition, vertical stresses in several elements on the upstream face of the dam near the base and for a downstream element of the dam near the base were considered. The location of the downstream element is shown in figure 4.1-1, and the locations of the upstream elements are shown in figure 4.1-2.

4.3.2 Acceleration on surface of foundation

Accelerations in the three directions were extracted for the two nodes on the surface of the foundation model. The time history accelerations for these nodes are shown in figures 4.3-1 and -2 (canyon node) and 4.3-3 and -4 (abutment node), along with the acceleration response spectra (cross canyon—solid line; upstream-downstream—chain-dash line; vertical—dashed line). For the canyon node and a ratio, $E_f/E_c=1/7$, in the all three records there is a predominance of frequencies between 5.0 and 6.67 Hz (or a range in period of 0.2 to 0.15 s). For the y-direction acceleration, there is a second peak around 1.5 Hz (or a period of 0.65 s). When a ratio $E_f/E_c = 1/1.25$ is considered, the first peak shifts to a range of higher frequencies between 10 to 20 Hz (or a range in period of 0.05 to 0.1 s). In the x-direction (cross canyon) and in the z-direction (upstream-downstream) there is a second peak around 2 Hz (0.5 s). In the y-direction (vertical), the second peak is around 3.33 Hz (0.3 s). A comparison of the time history plots on figures 4.3-1 and -2 indicates that the amplitudes of the accelerations for a node in the canyon remain close in value, but when ratio E_f/E_c is changed, the frequency content of the resulting surface motions changes.

Typical Grav Dam-Res-H2O-Ef/Ec=1/7
Grnd Mns and Rspns spectra-fnd-canyon

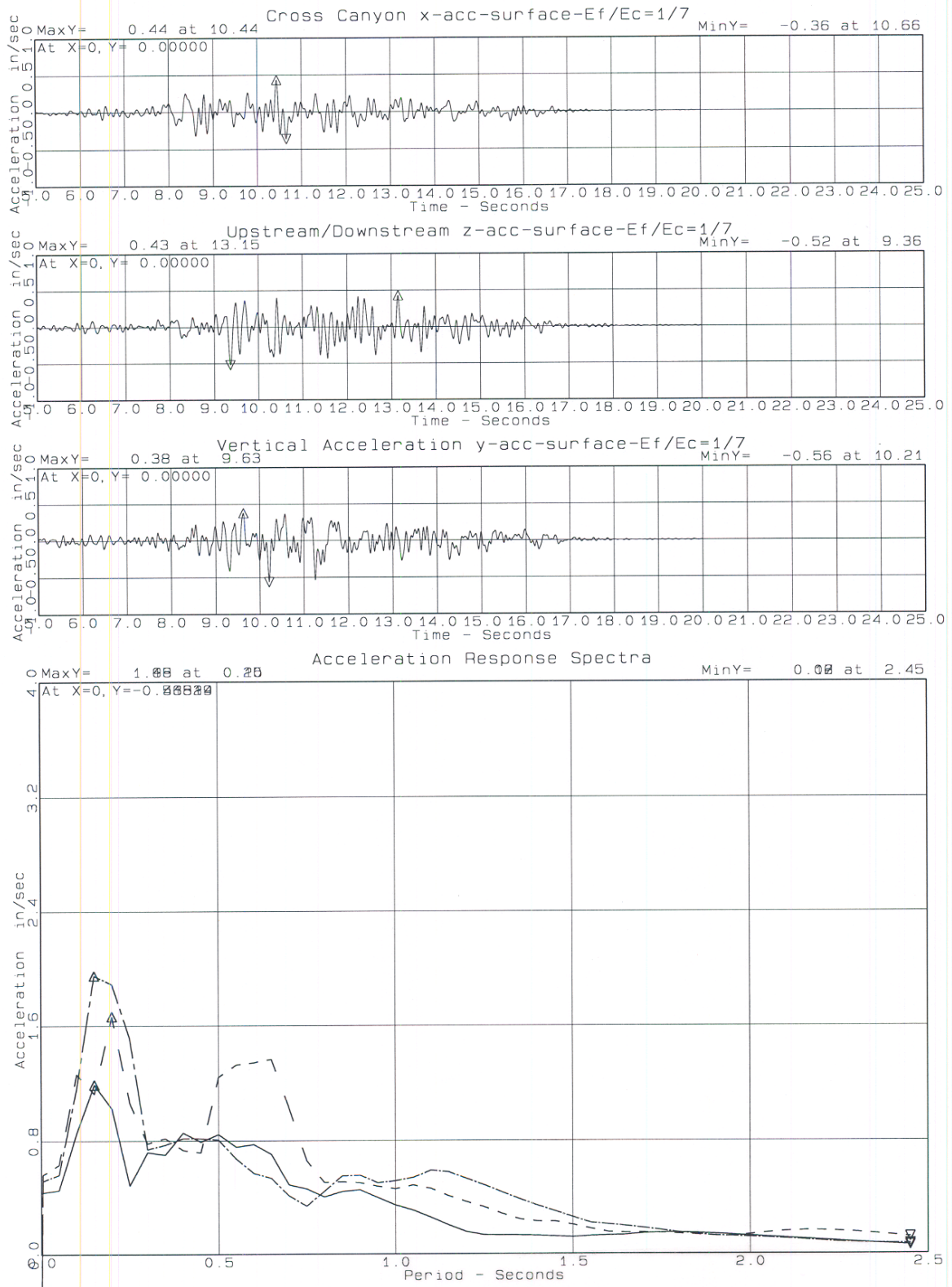


Figure 4.3-1 Time history accelerations—canyon node.

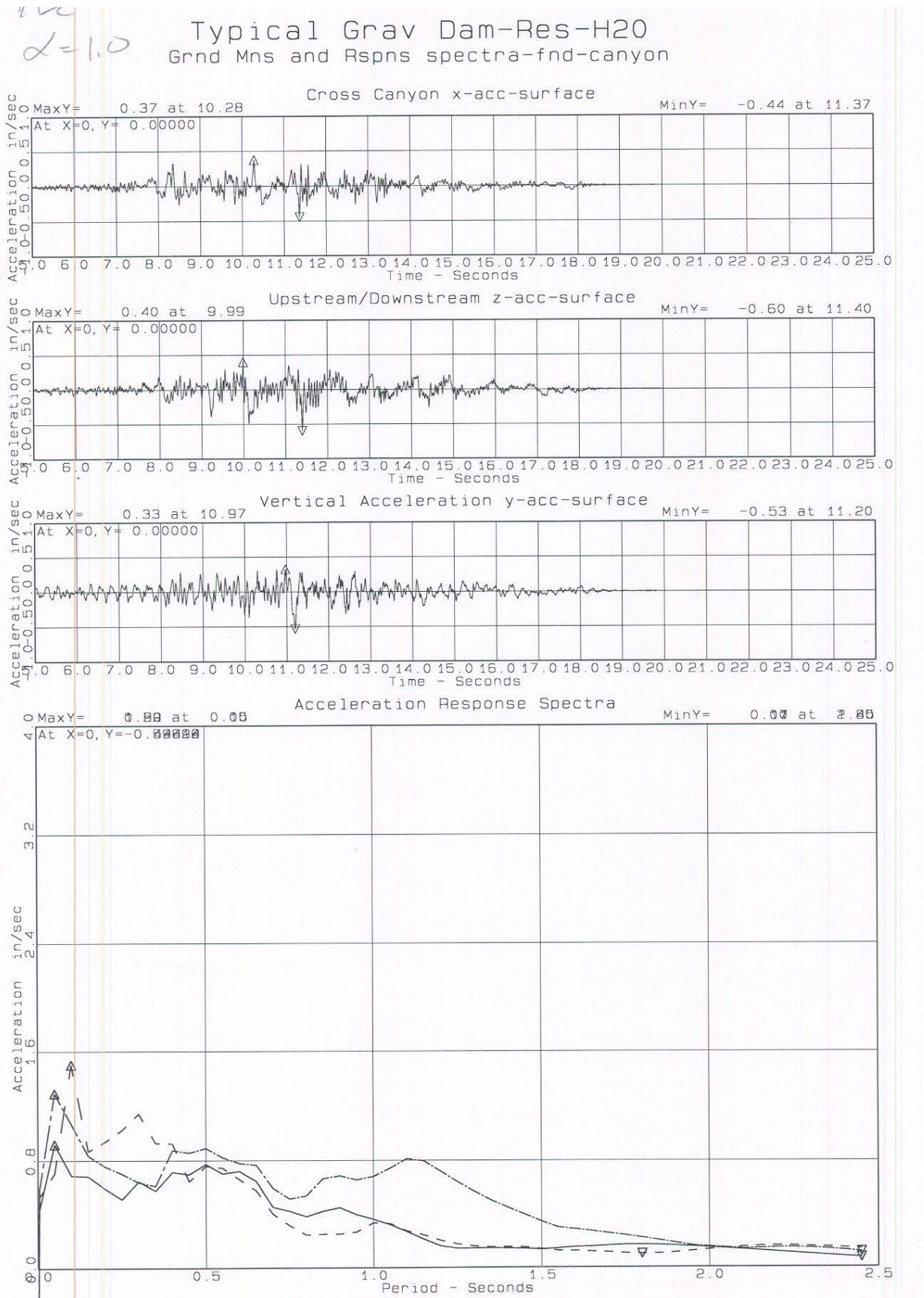


Figure 4.3-2 Time history accelerations—canyon node.

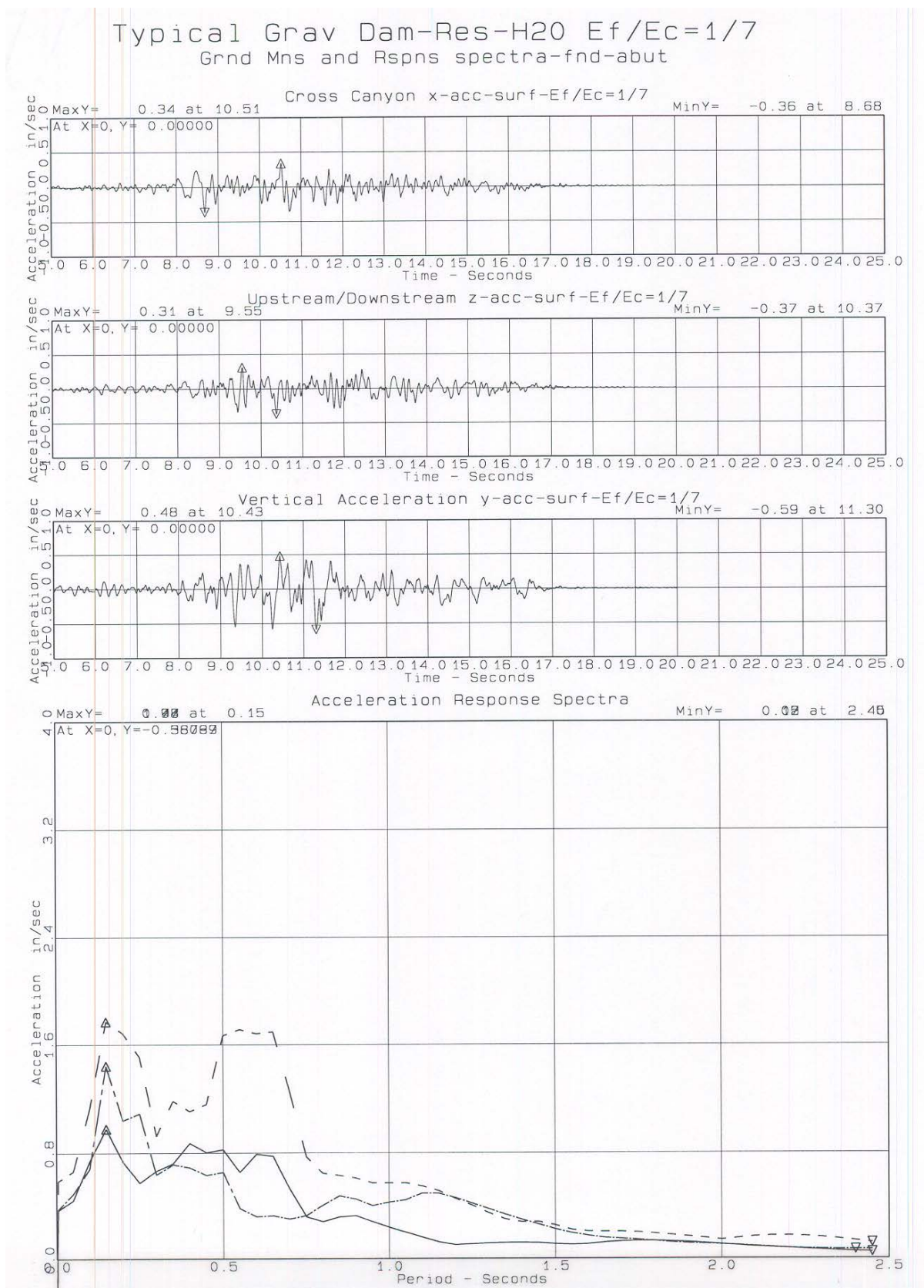


Figure 4.3-3 Time history accelerations—abutment node and acceleration response spectra.

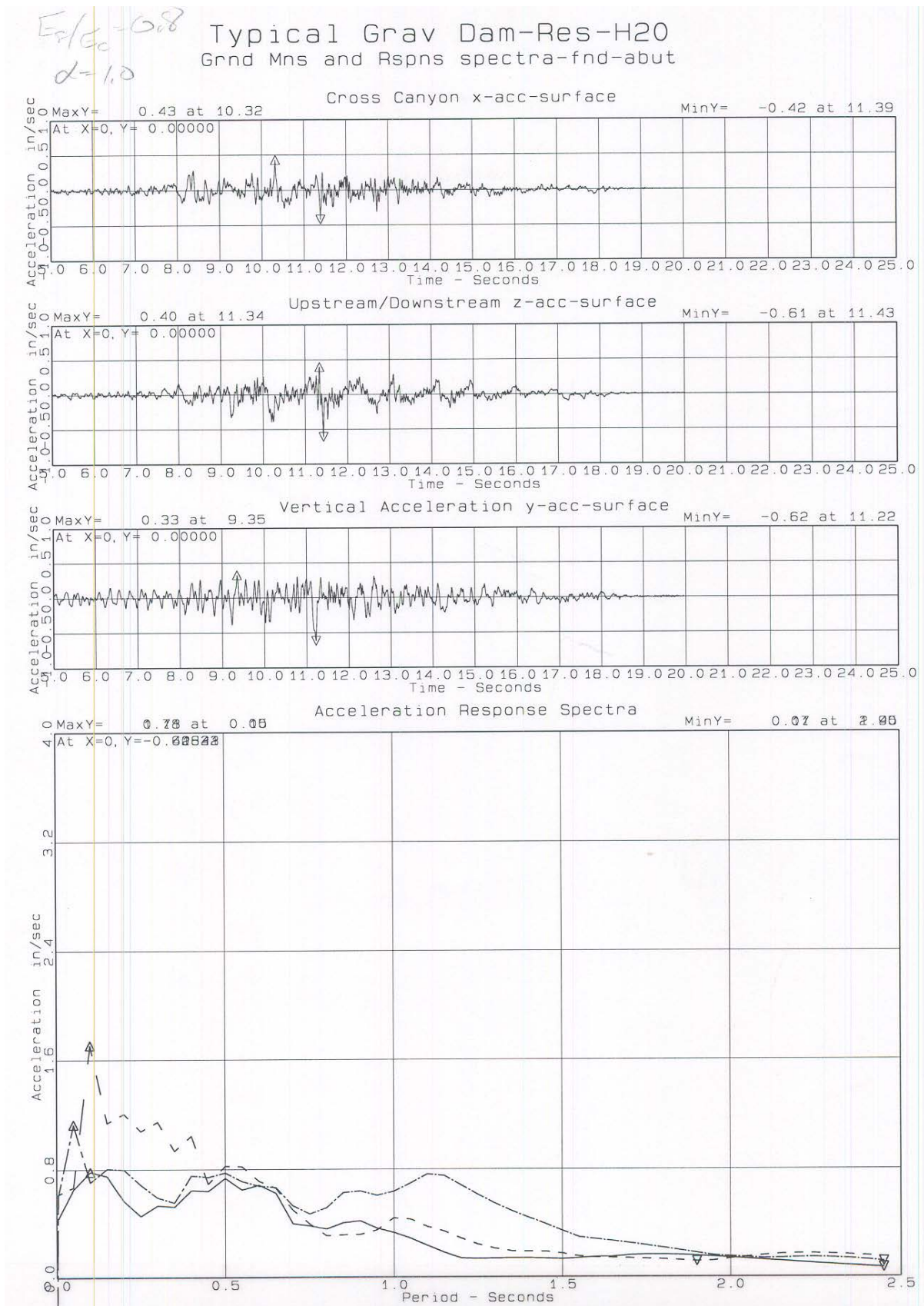


Figure 4.3-4 Time history accelerations—abutment node and acceleration response spectra..

For the abutment node and a ratio, $E_f/E_c=1/7$, in all three records, there is a peak of around a frequency of 6.67 Hz (or a period of 0.15 s). For the y-direction acceleration, there is a second peak around 1.8 Hz (or a period of 0.55 s). When a ratio $E_f/E_c = 1/1.25$ is considered, the first peak in the x-direction (cross-canyon) record and the y-direction record shifts to a higher frequency of 10 Hz (or a period of 0.1 s). In the z-direction (upstream-downstream), there is a peak around 20 Hz (or 0.05 s), a second peak around 2.5 Hz (or 0.4 s), and a third peak at approximately 0.9 Hz (or a period of 1.1 s). A comparison of the time history plots on figures 4.3-1 and -2 indicates that the amplitudes of the accelerations for an abutment node remain close in value, but when ratio E_f/E_c is changed, the frequency content of the resulting surface motions changes.

For both the canyon node and the abutment node, an increase in the ratio E_f/E_c from $1/7$ to $1/1.25$ results in a change in frequency content and, in particular, a shift of the predominant frequency from 6.67 to 10-20 Hz.

4.3.2 Stresses in dam elements

The plots of vertical stress time histories for several brick elements located on the upstream face of the dam are shown in figure 4.3-5. Vertical stresses on the upstream and downstream faces of the center monolith of the dam near the base are shown in figure 4.3-6. From these plots, it is seen that the tensile stresses experienced by the elements on the upstream are higher when the ratio is $1/1.25$. The stress response spectrum for the element at the base of the center monolith on the upstream face (element 5944) was calculated and plotted on figure 4.3-7 along with the stress time histories from the two analyses with the varying ratios. When the ratio was $1/1.25$, there was a high amplitude around a frequency of 4 Hz. The tension stress reached a peak of 195 lb/in^2 for the model with ratio of $E_f/E_c= 1/1.25$ and was only 81 lb/in^2 for the model with the ratio of $E_f/E_c = 1/7$.

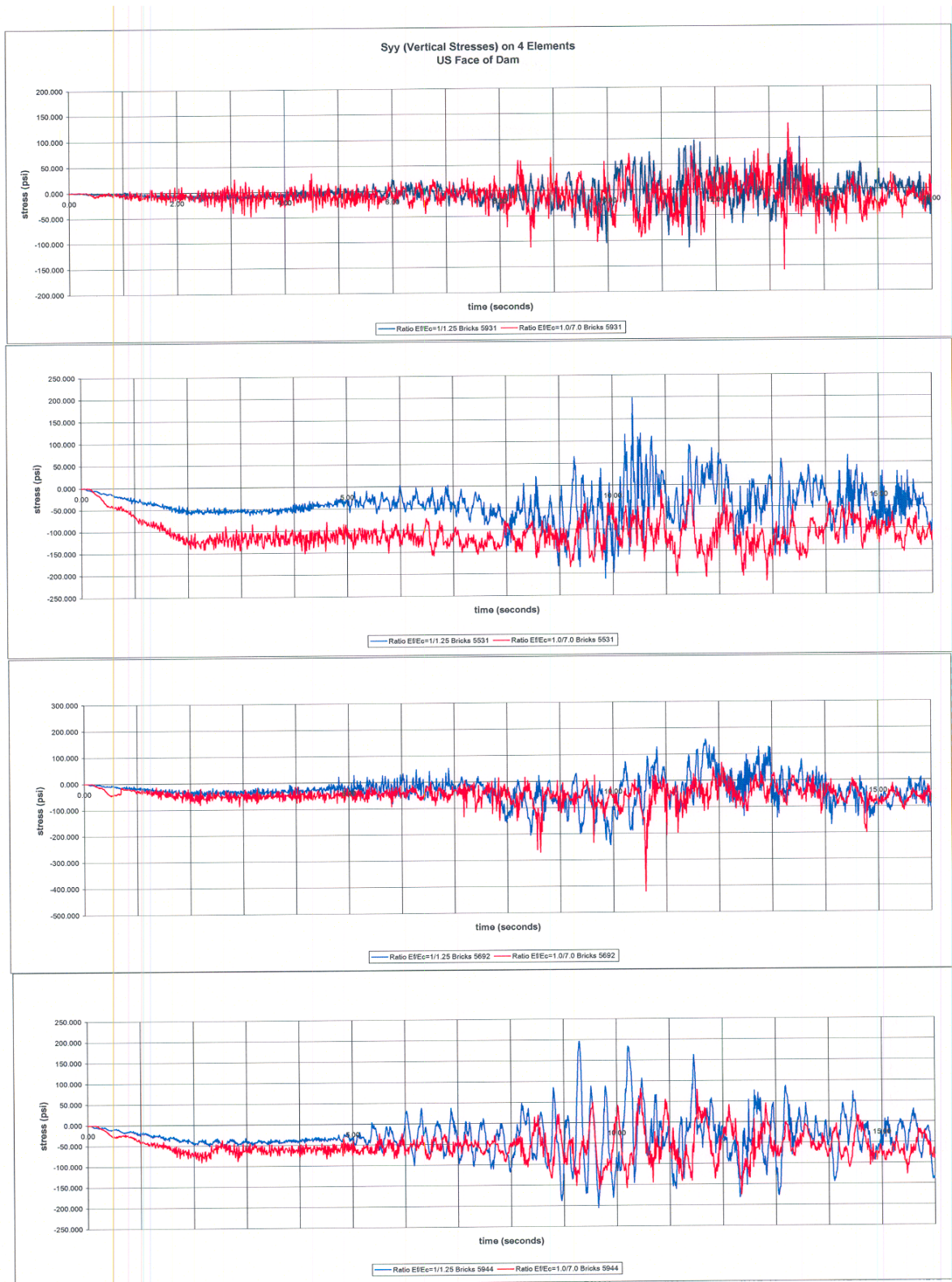


Figure 4.3-5 Time history vertical stresses upstream element at the base of the center monolith (gravity dam).

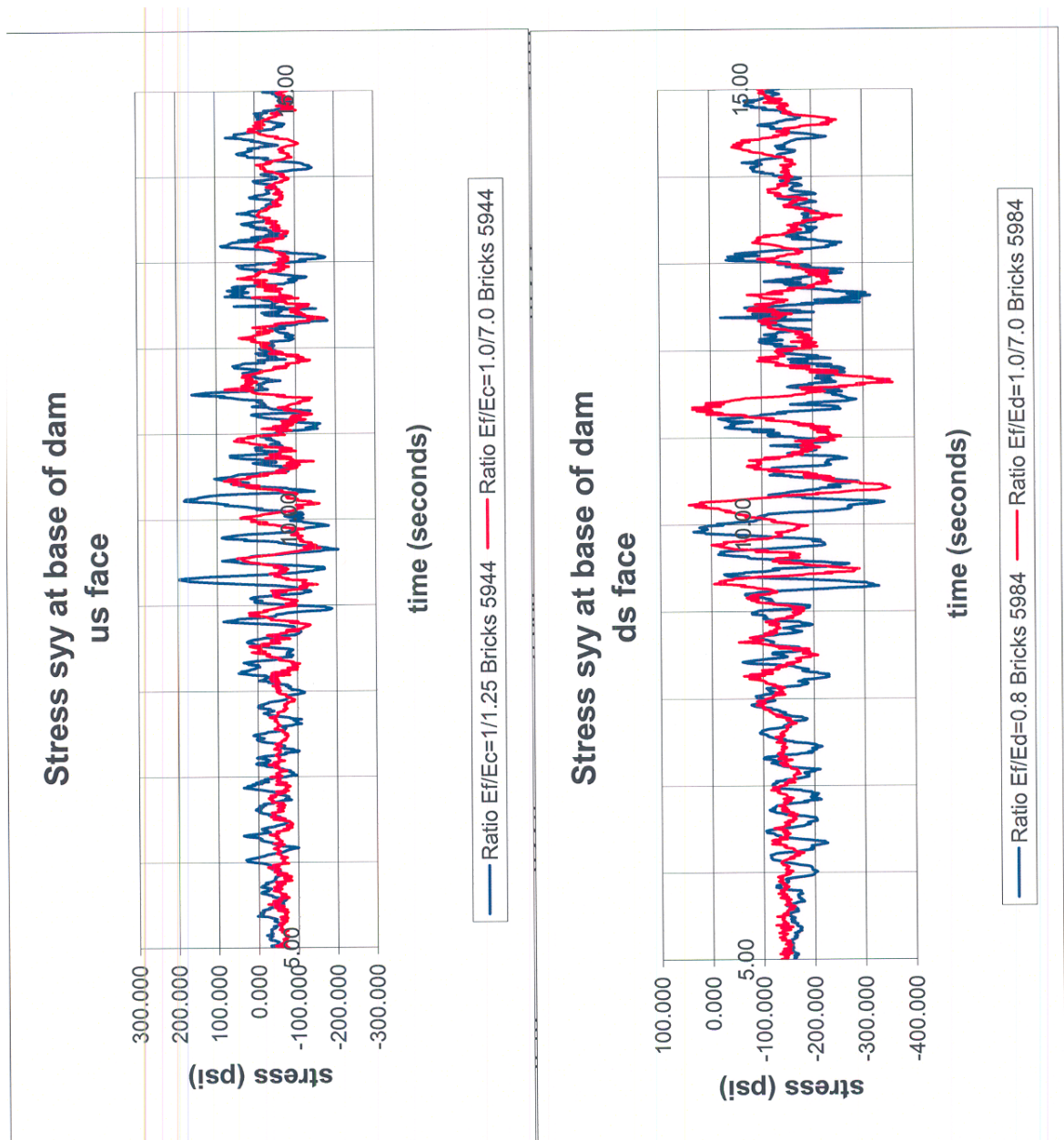


Figure 4.3-6 Stress time histories for two brick elements located at the base of the center monolith (gravity dam).

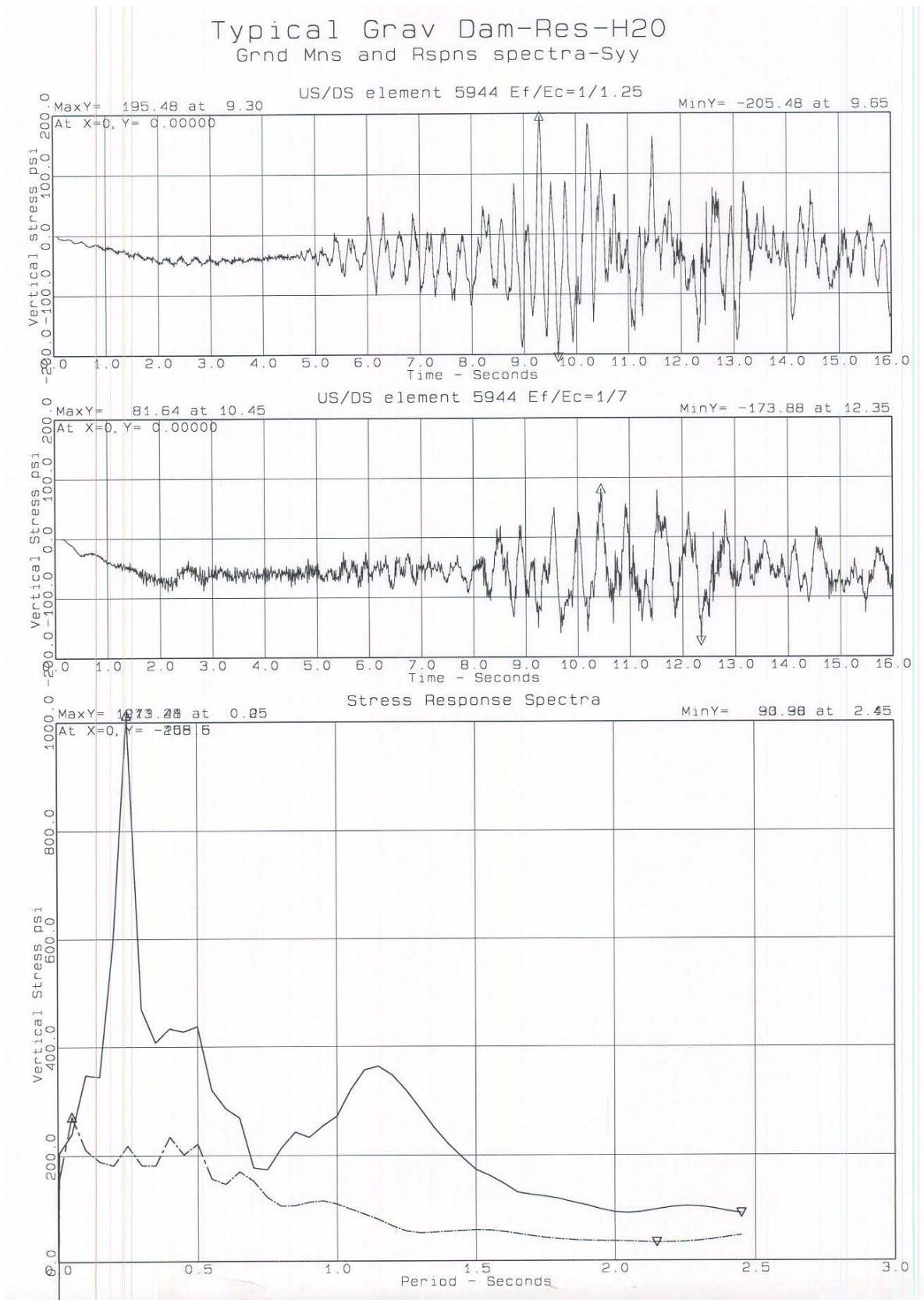


Figure 4.3-7 Stress time histories for element located at the base of the center monolith $E_f/E_c = 1/1.25$ and $E_f/E_c = 1/7$, and stress response spectra.

4.3.3 Summary

This study points out the importance of selecting material properties for the dam and foundation. Surface accelerations must be considered and should agree closely with recommended surface values from the Seismotectonics Group. In addition, if material properties are unknown, a suite of analyses may be necessary to bracket the calculated stress values or, if forced vibration tests are available, calibration of the model may be possible.

5.0 Arch Dam/Reservoir/Foundation System

5.1 Introduction

A model of a simplified arch dam was created (see figures 5.1-1 and -2) which included the reservoir, foundation, and three cantilever sections of the dam. The height of the dam was 570 feet, the crest width was 35 feet, and the crest length was 950 feet. The width at the base was 130 to 150 feet. The ground motions were applied 166 feet up from the base of the foundation. The bottom of the canyon was 2,500-feet above this elevation and the top of the abutments were 600 feet above the base of the canyon. This model was analyzed for static and dynamic loads. The ground motions used were deconvolved ground motions. The ground motions are plotted in figure 5.3-1 along with the acceleration response spectra at 5 percent damping for the three components.

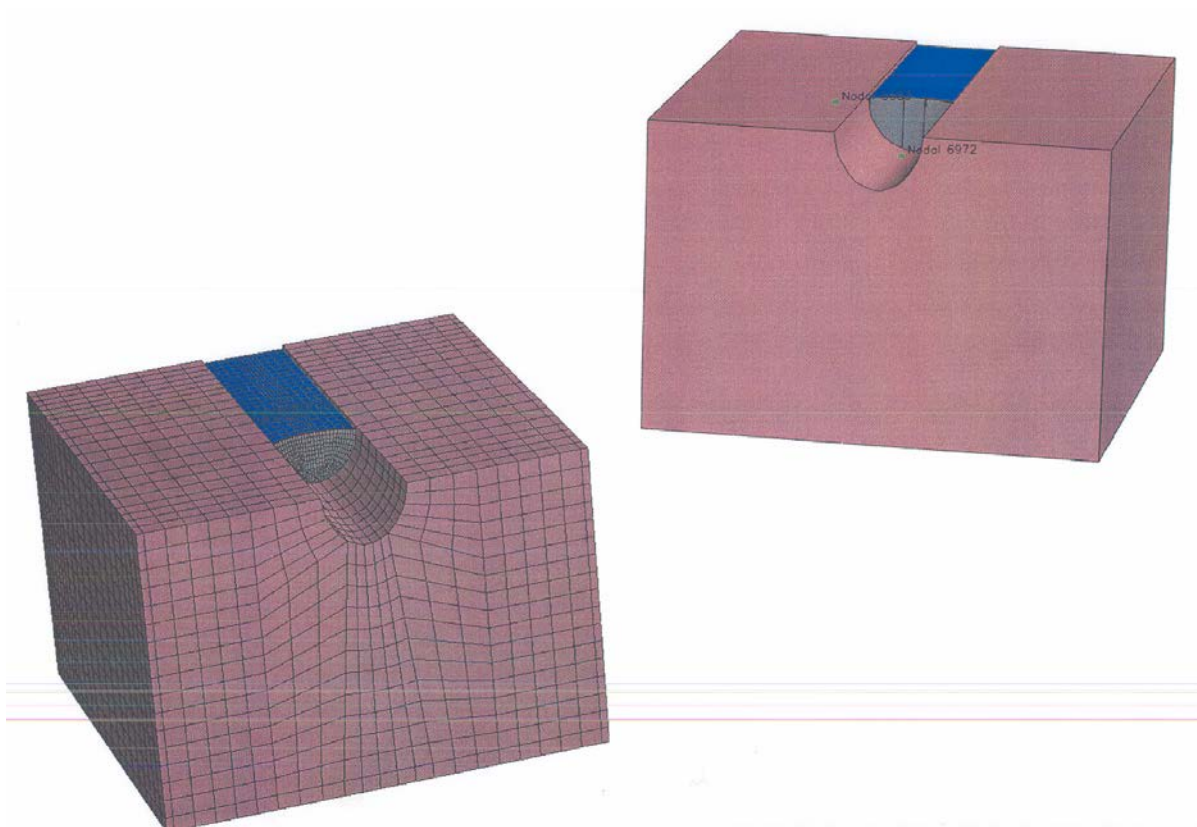


Figure 5.1-1 Arch dam/reservoir/foundation system.

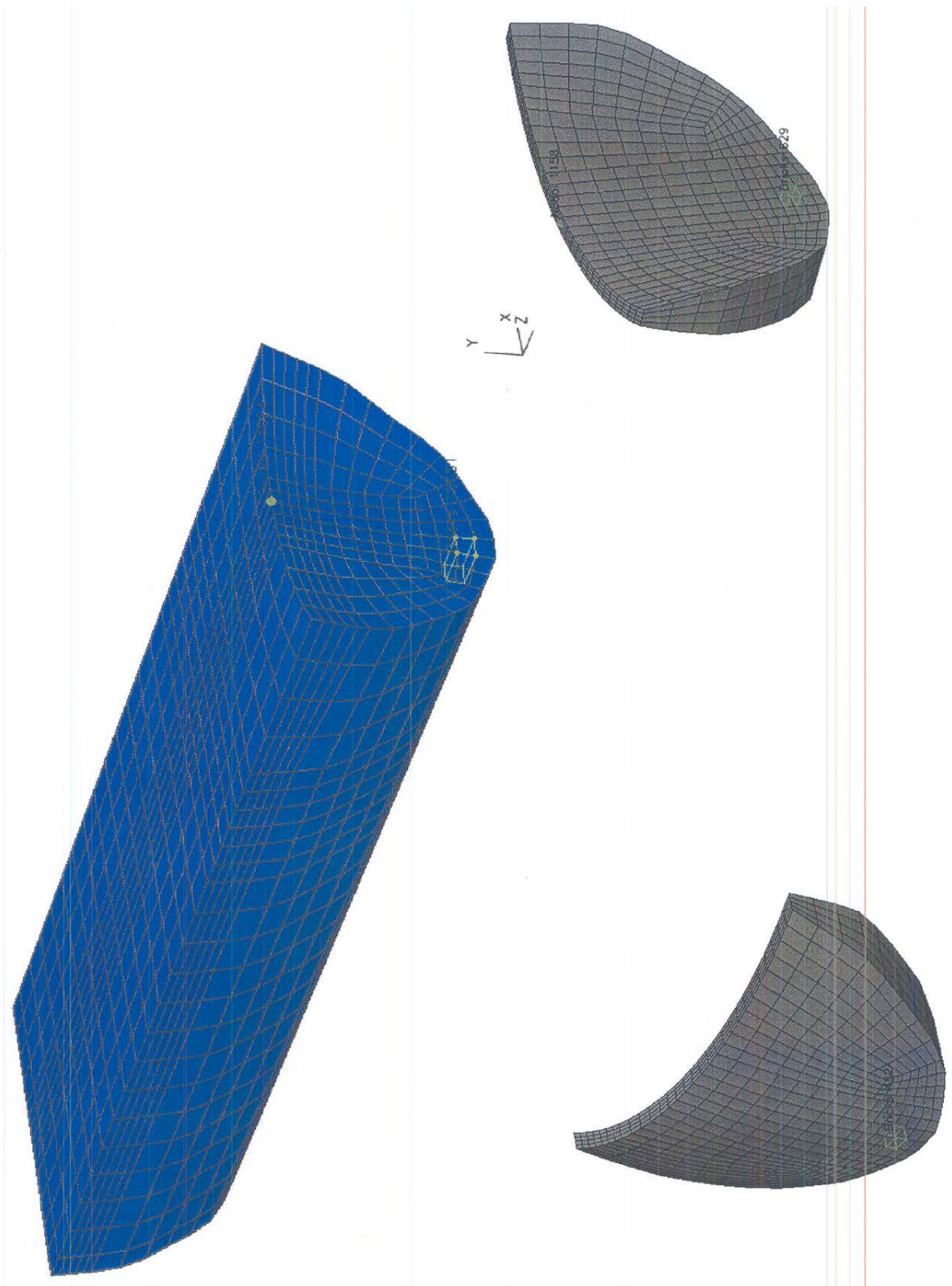


Figure 5.1-2 Dam and water.

As in the gravity dam/reservoir/foundation system, the main purpose of the dynamic runs was to evaluate the effect of varying ratios of elastic modulus in the foundation to elastic modulus in the dam (E_f/E_c). The ratios considered here were, as above, $E_f/E_c = 1/1.25$, (where $E_c = 4 \times 10^6 \text{ lb/in}^2$ and $E_f = 3.2 \times 10^6 \text{ lb/in}^2$) and $E_f/E_c = 1/7$ (where $E_c = 4 \times 10^6 \text{ lb/in}^2$ and $E_f = 0.57 \times 10^6 \text{ lb/in}^2$). Note that the stiffness of the concrete remained unchanged and the stiffness of the foundation varied. In addition, the damping values of $\alpha = 1.0$ and $\alpha = 2.0$ were considered. Finally, applying atmospheric pressure to any exposed surface of the model was considered. In particular, atmospheric pressure was applied to the exposed surface of the foundation, the top of the reservoir, and the top and downstream faces of the dam.

5.2 Damping studies

For the following analyses gravity was applied to the dam/reservoir/foundation system over a 2 s period of time. This was followed by 4 s of quiet time to allow the vibrations to decay. Figures 5.2-1 to -4 show plots of the vertical stress in an element on the upstream face of the dam; the water pressure near the base of the upstream face of the dam; the vertical displacement of a node on the water surface; and the upstream-downstream displacement of a node at the top of the dam near the crown. These variables are plotted for the following cases: 1) no damping and no atmospheric pressure applied; 2) atmospheric pressure was applied but damping was not applied; 3) atmospheric pressure was applied and a damping value of $\alpha = 1.0$ was assumed; 4) atmospheric pressure was applied and a damping value of $\alpha = 2.0$ was assumed. For each of these plots, gravity was applied in 2 s, followed by 4 s of quiet time to allow the vibrations to decay.

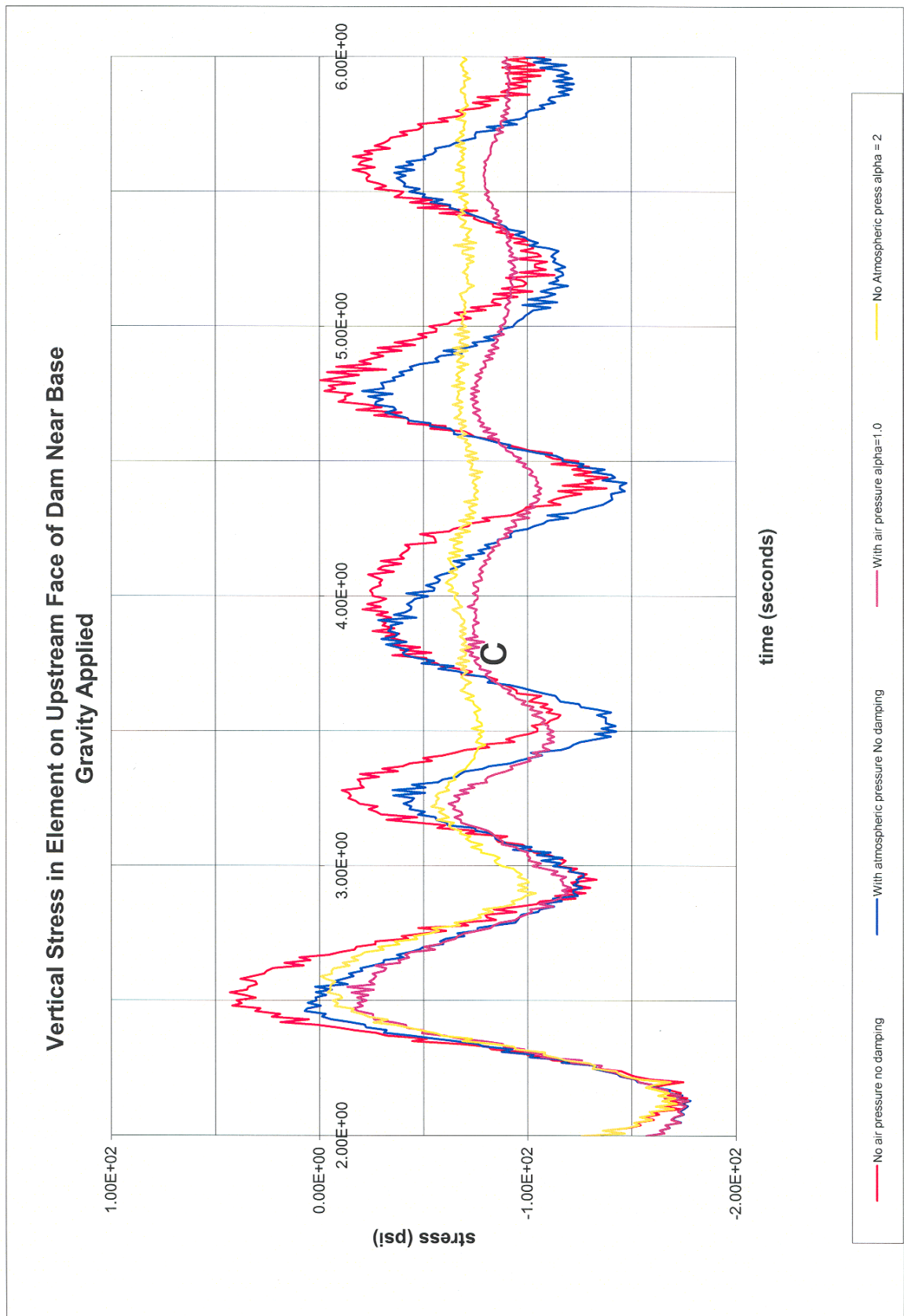


Figure 5.2-1 Vertical stress in an element on the upstream face of the dam.

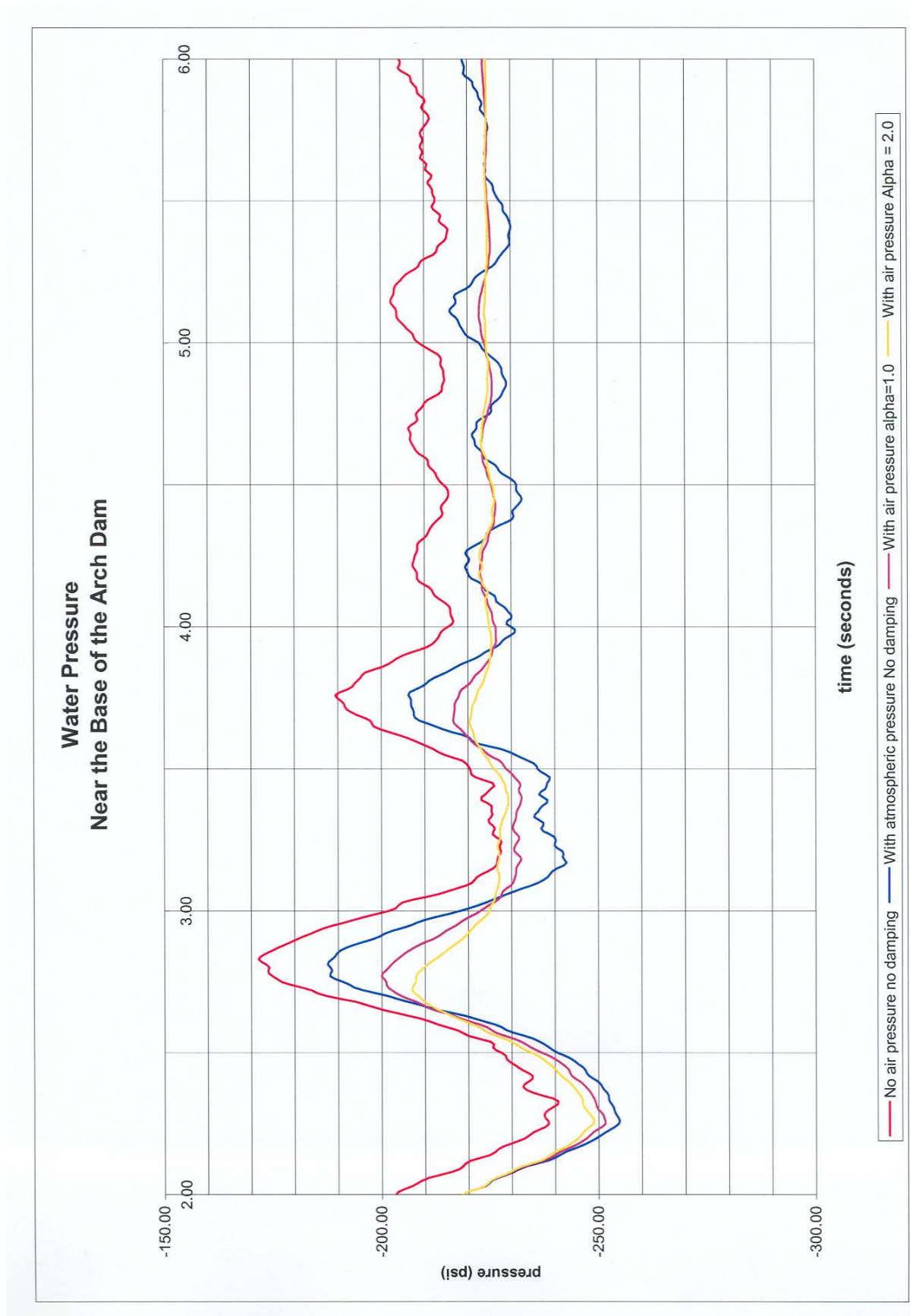


Figure 5.2-2 Water pressure near the base of the upstream face of the arch dam.

Vertical Displacement of Node at Water Surface Gravity Applied

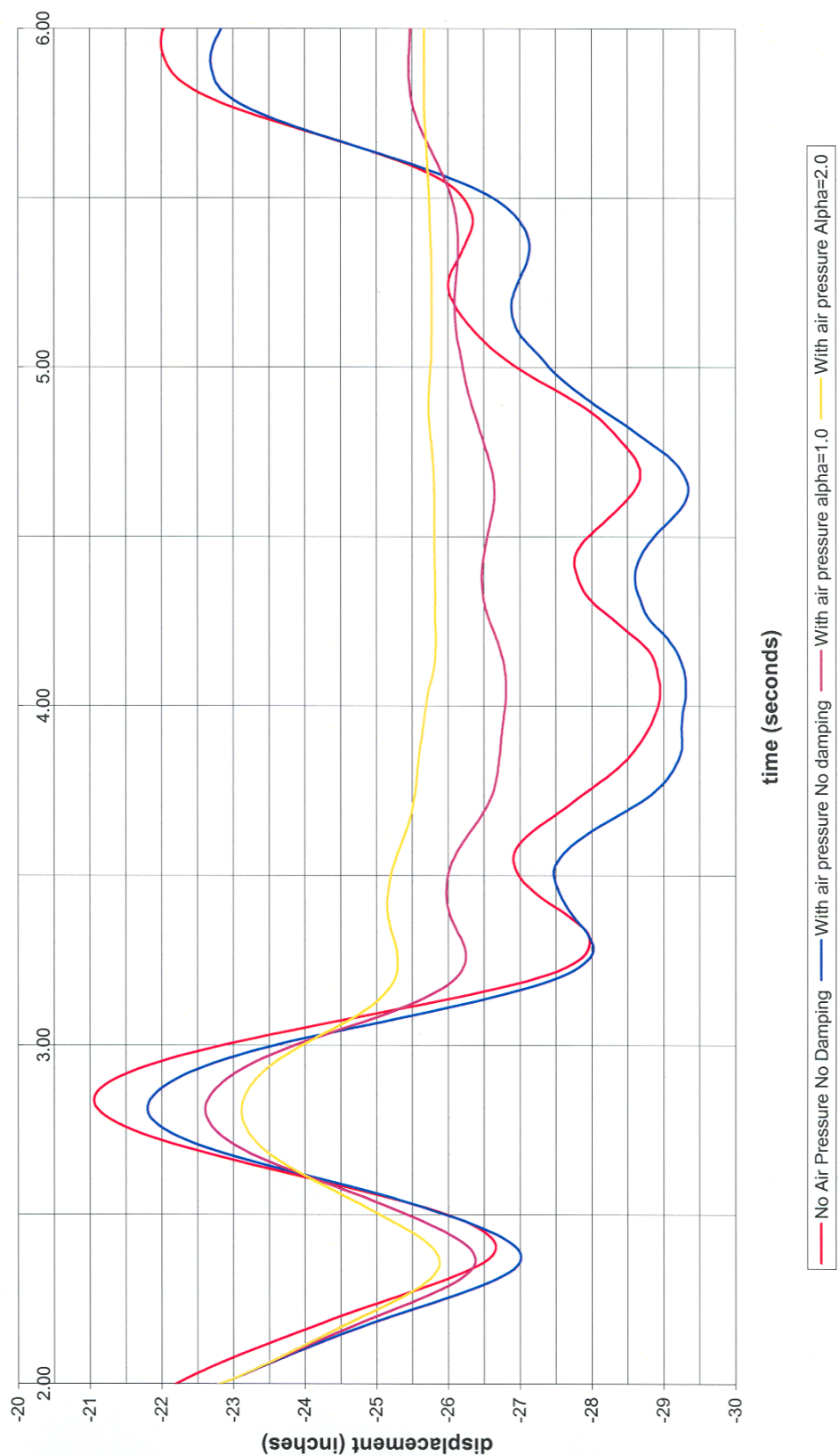


Figure 5.2-3 Vertical displacement of a node on the water surface.

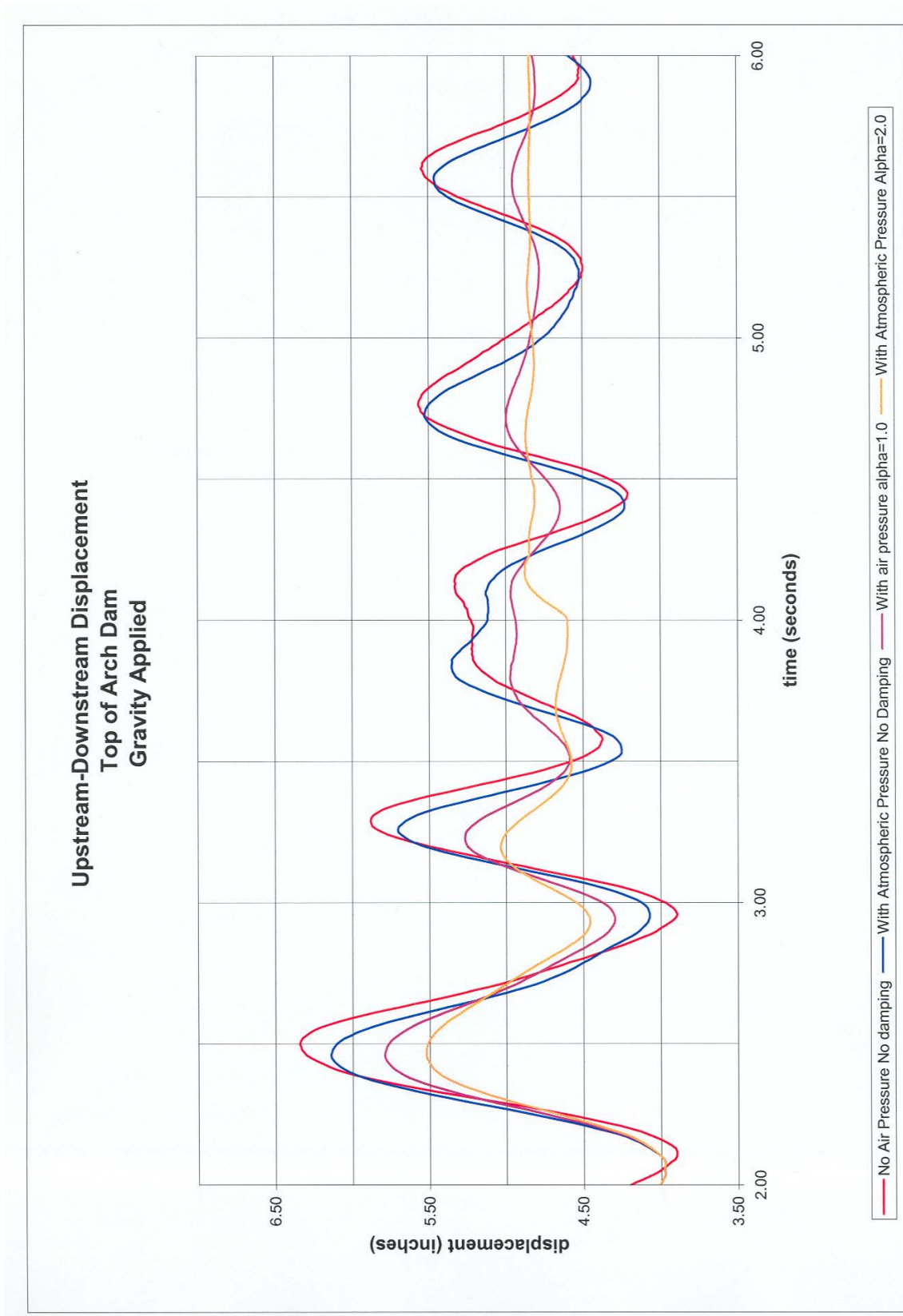


Figure 5.2-4 Upstream-downstream displacement of a node at the top of the arch dam near the crown.

Figure 5.2-1 compares the vertical stresses in an element on the upstream face of the dam near the base. The effect of applying atmospheric pressure is apparent from the red and blue curves. Application of the atmospheric pressure increases the vertical compressive stress in the element, so the blue curve is shifted down approximately 40 lb/in². The effect of damping is evident in a comparison of the red and/or blue curves to the gold and brown curves. When a value of $\alpha = 1.0$ is applied (brown), the oscillations in the time history during the quiet period have not quite decayed to zero by 6.0 s. When a value of $\alpha = 2.0$ (yellow) is applied the oscillations have decayed to zero by 5 s. During the application of gravity, a higher damping value could result in a savings in the time the analysis must be run.

Figure 5.2-2 compares the water pressure near the base of the dam. The application of atmospheric pressure (red versus blue curves) results in an increase in water pressure by the amount of the atmospheric pressure (14.7 lb/in²). When $\alpha = 1.0$ (brown curve), the oscillations decay by approximately 6.0 s. When $\alpha = 2.0$ (gold curve), the oscillations decay by approximately 5.5 s.

Similar conclusions can be drawn from the comparison of curves in figures 5.2-3 and -4. In all cases, the increase in damping value reduces the time in which the oscillations decay to a constant value. If no damping is applied, the system does not reach a constant state. The addition of atmospheric pressure does not affect the damping of the system, but changes slightly the value about which the system oscillates.

5.3 Effect of varying the ratio of the foundation/dam modulus

5.3.1 Acceleration on surface of foundation

For two nodes, one on the abutment and one in the canyon, accelerations were extracted and compared to each other and to input motions and desired surface motions. The time history plot of the input motions and their response spectra are shown in figure 5.3-1. The maximum horizontal accelerations were 0.23g (cross canyon), 0.24g (upstream-downstream), and 0.16g vertically. The primary frequency content was 2 to 6.7 Hz. The time history accelerations for a node on the abutment and a node in the canyon for the three analyses discussed below are plotted in figures 5.3-3 through -8.

Foundation Study - deconvolved MCE-cha

Ground Motions and Response spectra

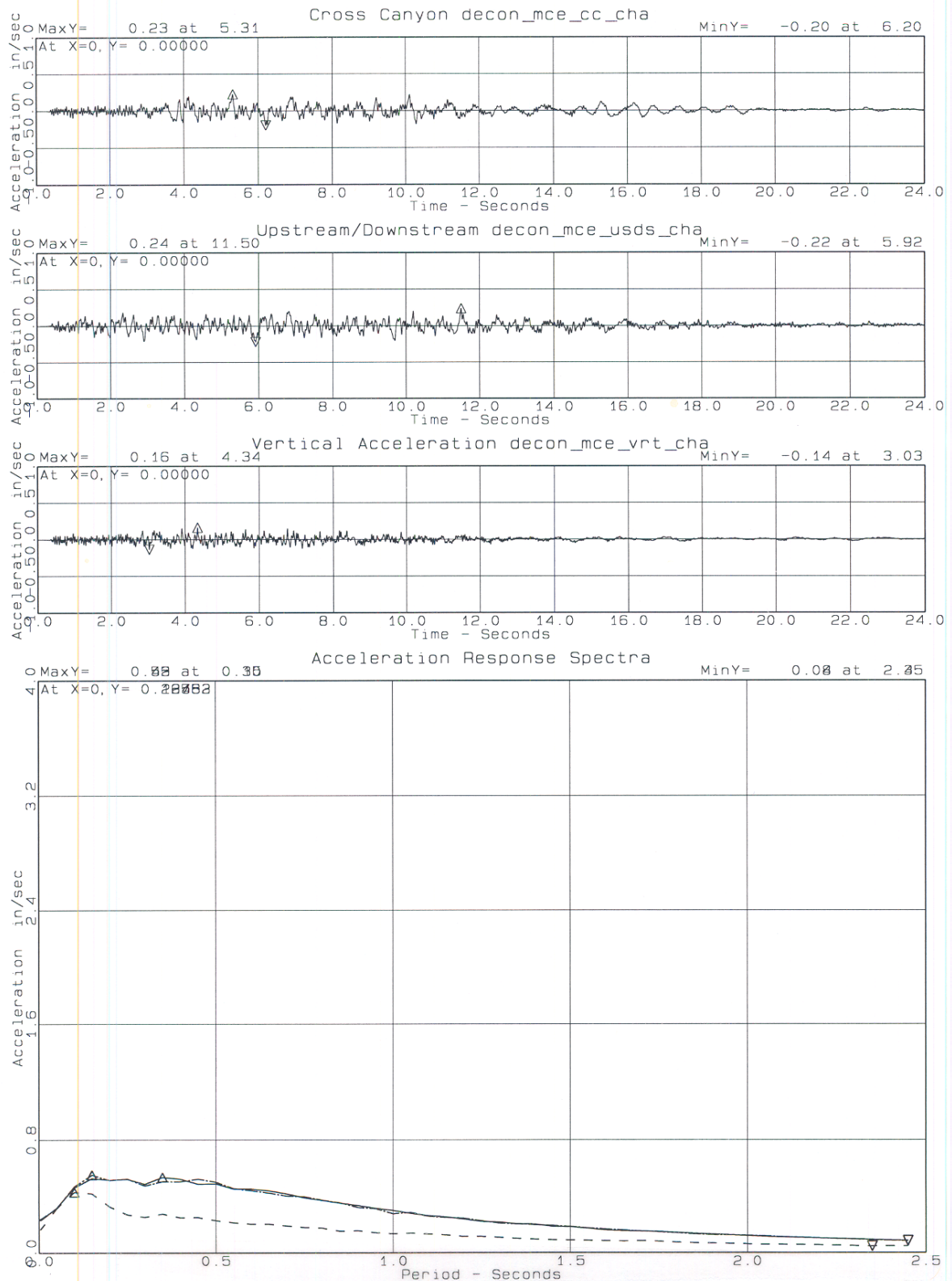


Figure 5.3-1 Time history plot of the input motions and response spectra.

Arch Dam-Acc-canyon, $\alpha=1$, $E_f/E_c=.8$, ap
Ground Motions and Response spectra

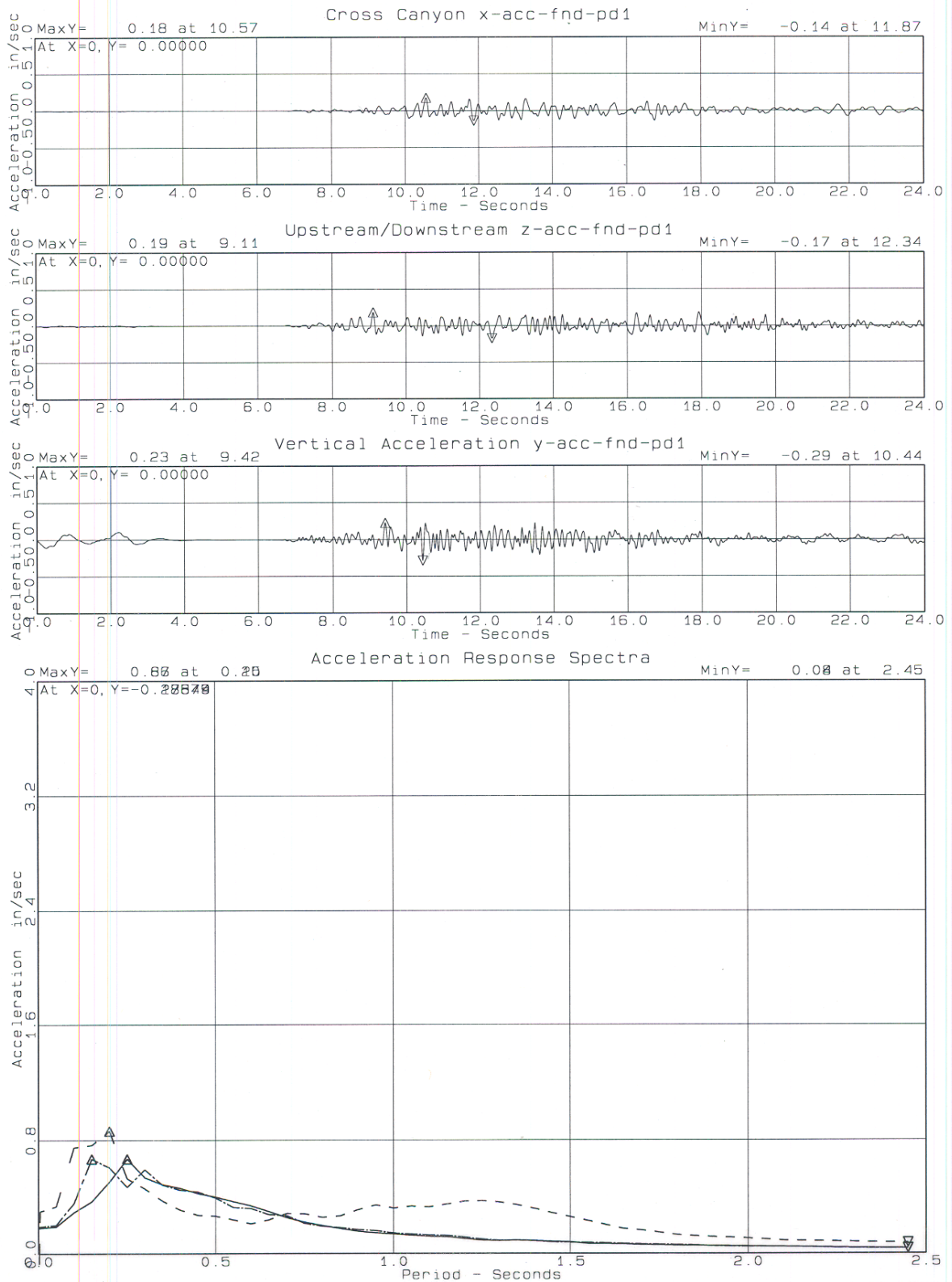


Figure 5.3-3 Time history accelerations and response spectra, $\alpha = 1.0$, $E_f/E_c = 1/1.25$, canyon node.

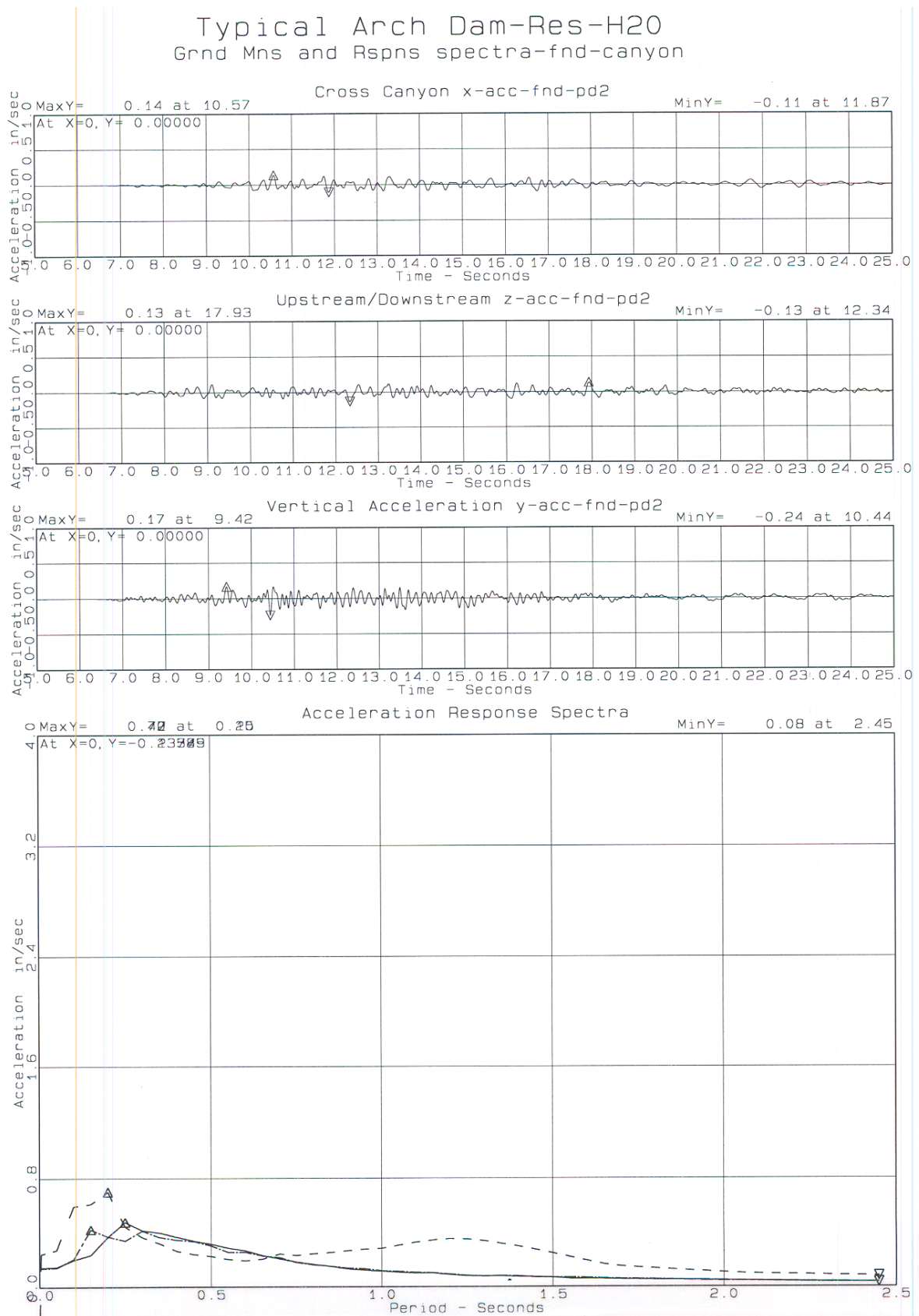


Figure 5.3-4 Time history accelerations and response spectra, $\alpha = 2.0$, $E_f/E_c = 1/1.25$, canyon node.

Typical Arch Dam-Res-H20-Ef/Ec=1/7
Grnd Mns and Rspns spectra-fnd-canyon

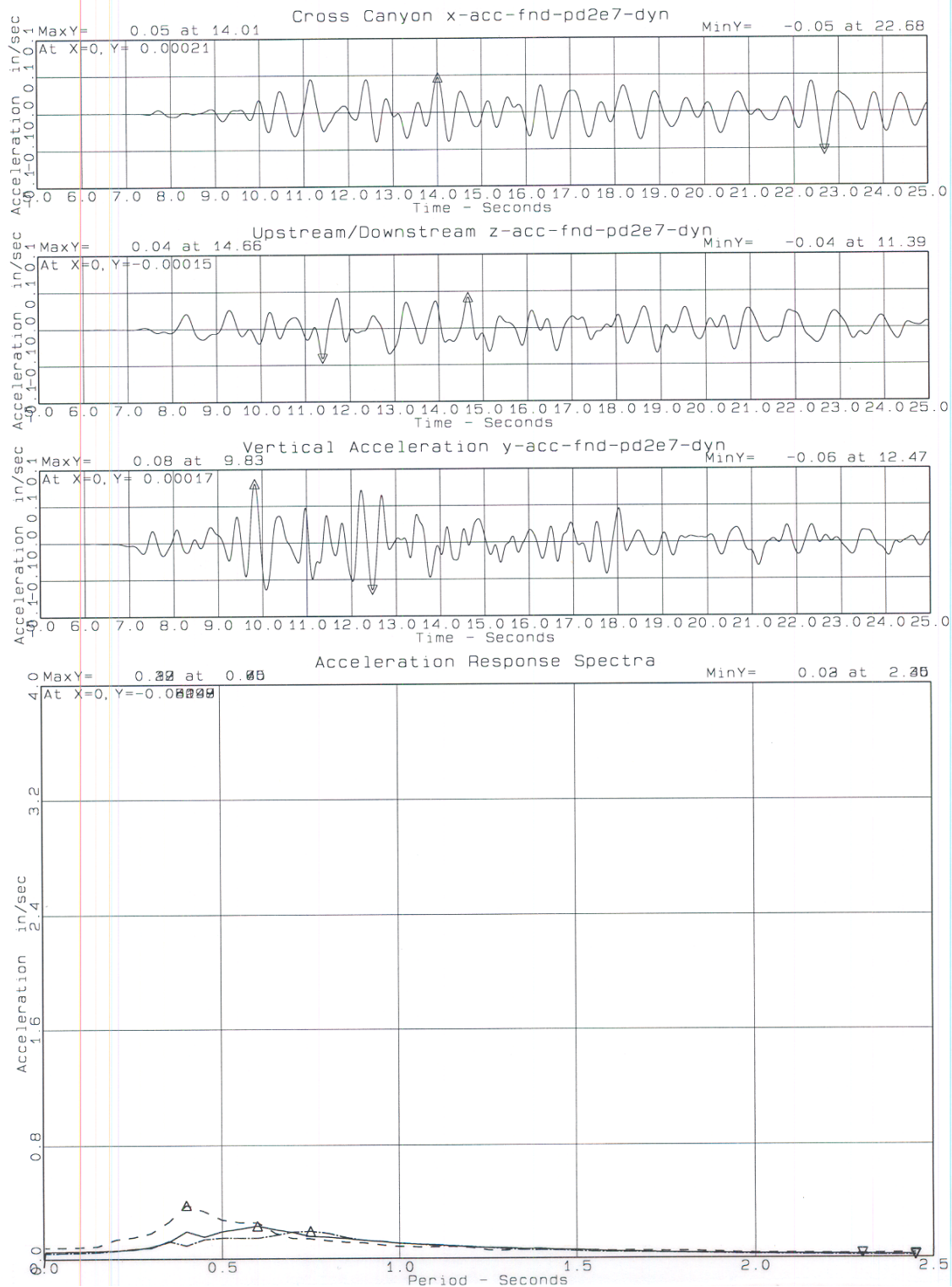


Figure 5.3-5 Time history accelerations and response spectra, $\alpha = 2.0$, $E_f/E_c = 1/7$, canyon node.

Arch Dam-Acc-Abut- $\alpha=1$, $E_f/E_c=.8$, ap
Ground Motions and Response spectra

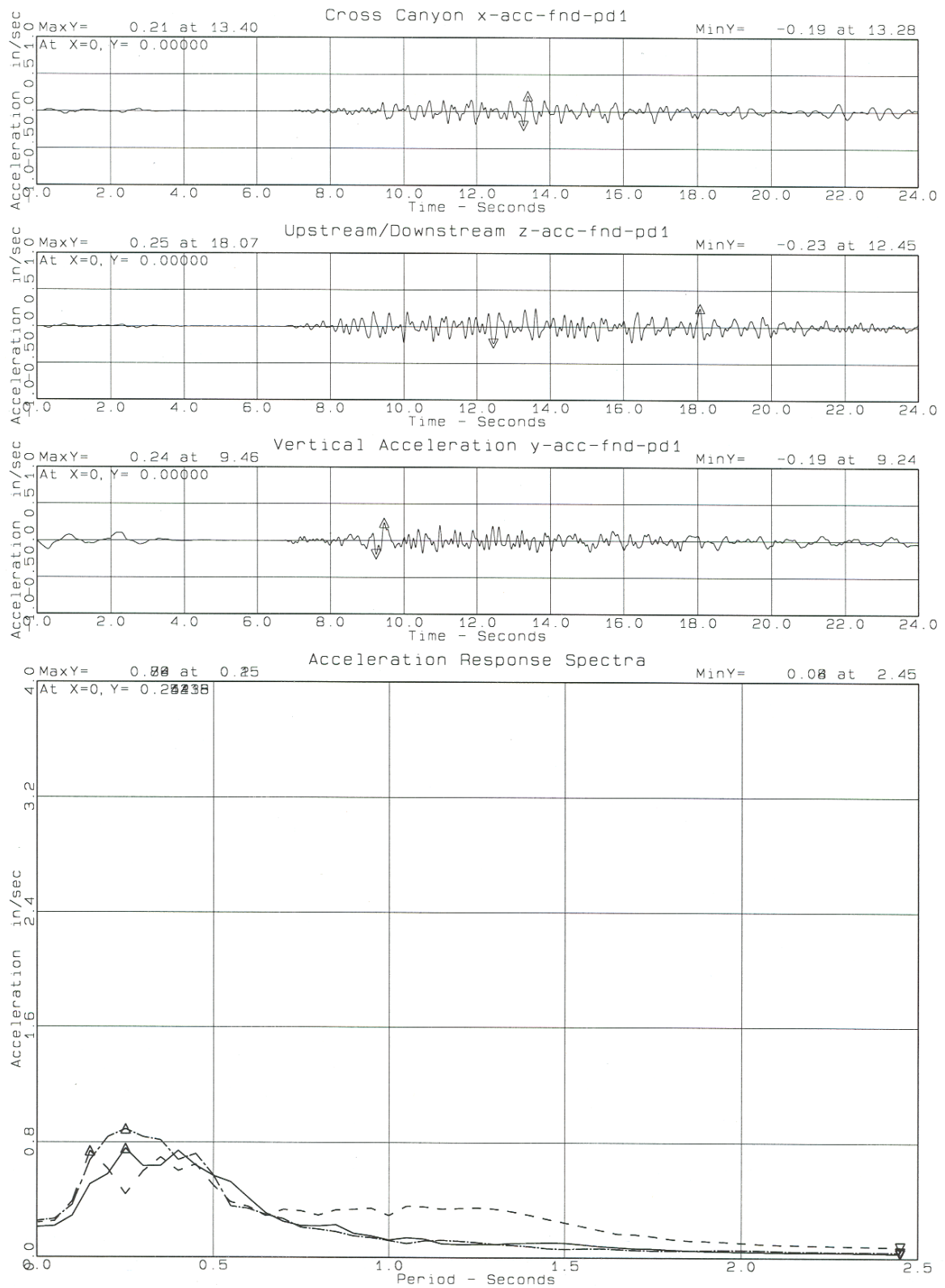


Figure 5.3-6 Time history accelerations and response spectra, $\alpha = 1.0$, $E_f/E_c = 1/1.25$, abutment node.

Typical Arch Dam-Res-H20 Grnd Mns and Rspns spectra-fnd-abut

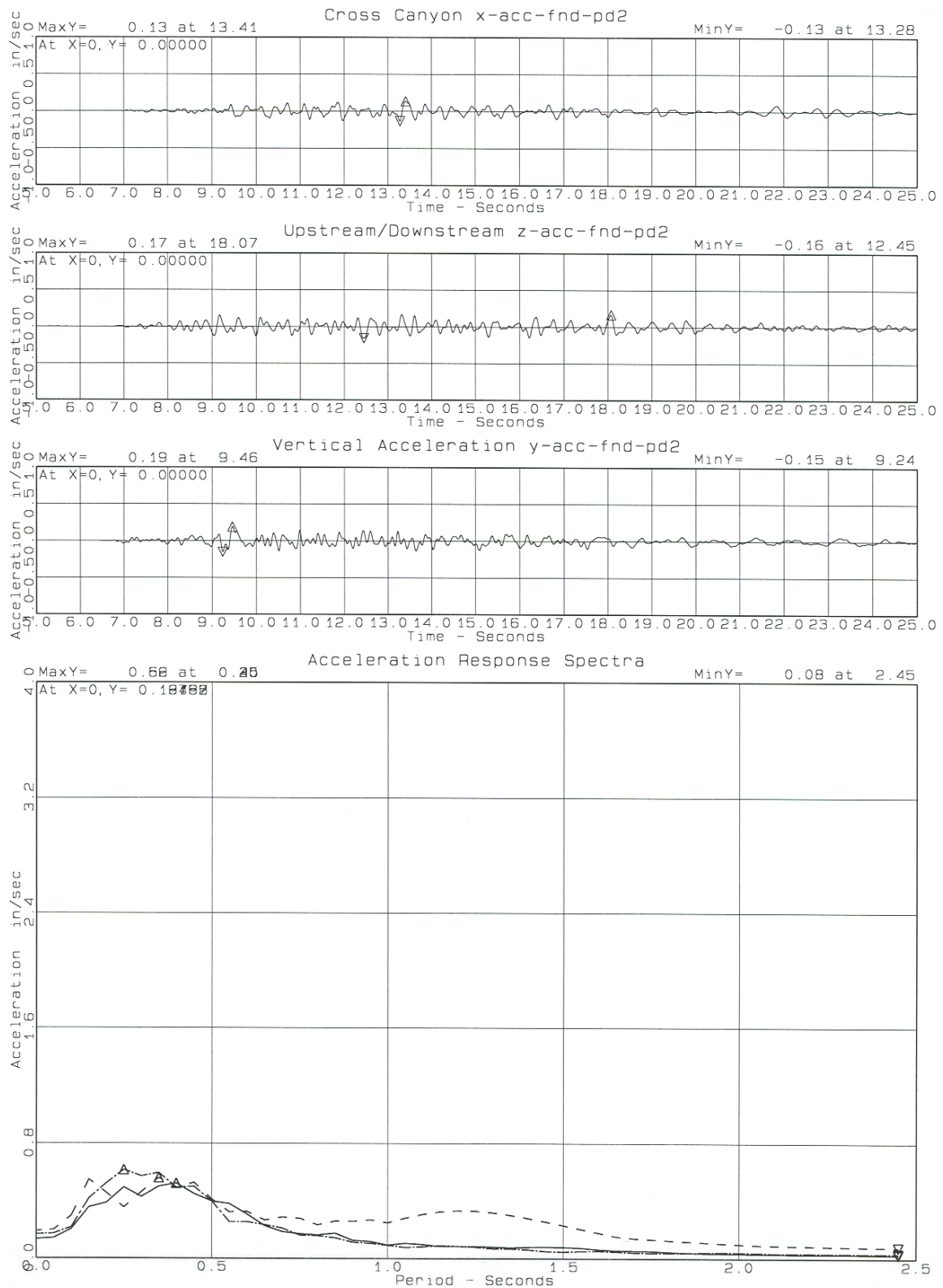


Figure 5.3-7 Time history accelerations and response spectra, $\alpha = 2.0$, $E_f/E_c = 1/1.25$, abutment node.

Typical Arch Dam-Res-H20-EF/Ec=1/7
Grnd Mns and Rspns spectra-fnd-abut

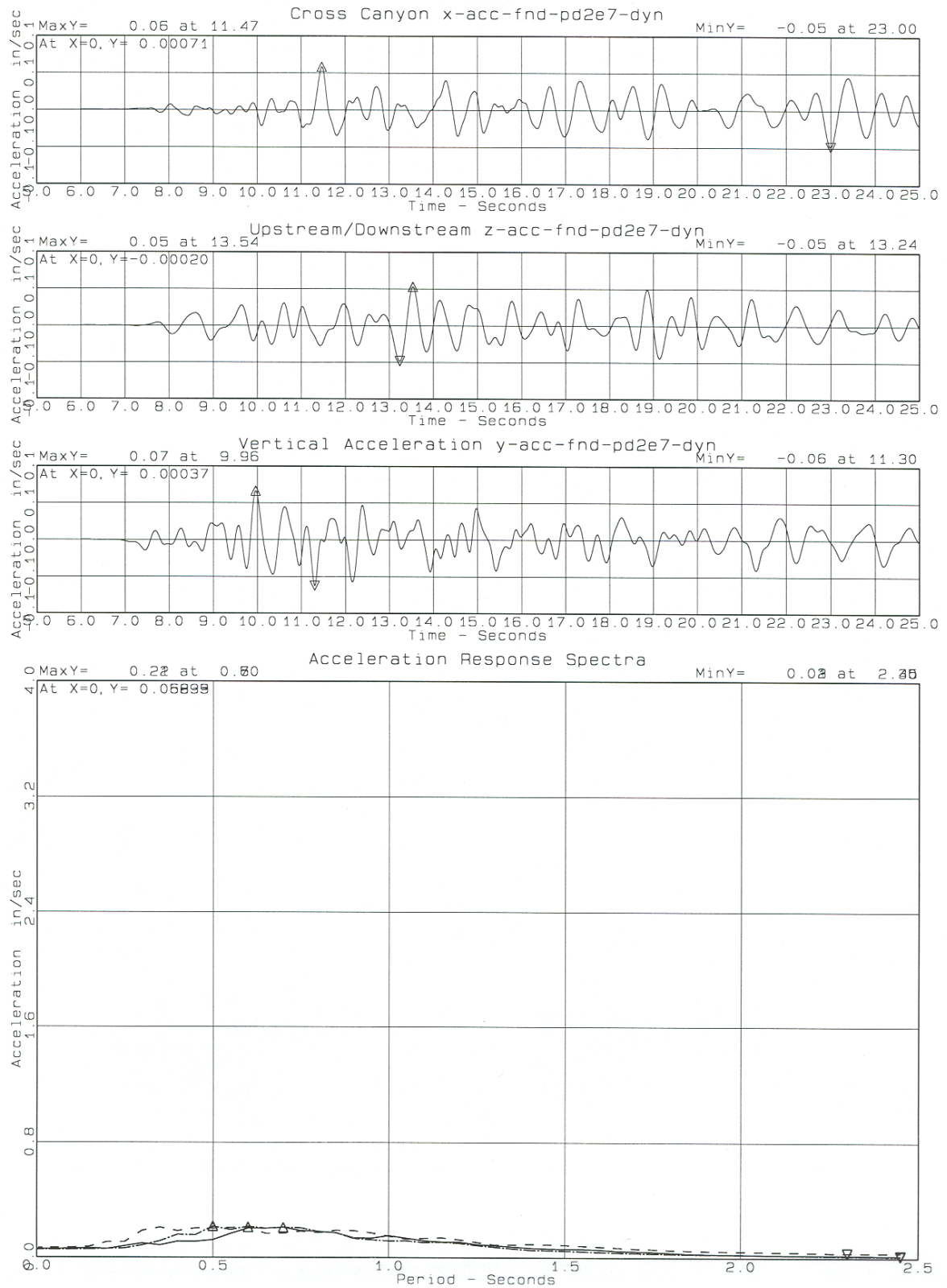


Figure 5.3-8 Time history accelerations and response spectra, $\alpha = 2.0$, $E_f/E_c = 1/7$, abutment node.

The first discussion concerns a node in the canyon, just directly downstream of the arch dam. For a ratio $E_f/E_c = 1/1.25$, two analyses are considered here: one with $\alpha = 1.0$ and the other with $\alpha = 2.0$. For both of these analyses, the maximum accelerations were smaller than the input accelerations, with values of 0.14 (cross-canyon), 0.13(upstream-downstream), and 0.24g vertically for $\alpha = 2.0$ and 0.18(cross-canyon), 0.19(upstream-downstream), and 0.29g vertically for an $\alpha = 1.0$. The response spectra for the analysis with $\alpha = 1.0$ have the same frequency content as the response spectra for the analysis with $\alpha = 2.0$ (5 to 10 Hz), but the higher damping value results in a 20-percent decrease in the maximum value. When the ratio, $E_f/E_c = 1/1.25$ is reduced to $E_f/E_c = 1/7$, the frequency content is shifted to higher frequencies, around 21 Hz, and the acceleration values are 30 to 40 percent of the comparable accelerations from the analysis with the higher E_f/E_c ratio.

The node on the abutment showed similar results for both the acceleration magnitudes and the frequency content. The analyses with a ratio of $E_f/E_c = 1/1.25$ resulted in maximum accelerations with values of 0.21g (cross-canyon), 0.25g (upstream-downstream), and 0.24g vertically for an $\alpha = 1.0$ and 0.13(cross canyon), 0.17(upstream-downstream), and 0.19g vertically for an $\alpha = 2.0$. The predominant frequency content was in the range of 4 to 5Hz for both analyses. When a ratio of $E_f/E_c = 1/7$ was assumed, the frequency content shifted to 1 to 2 Hz and the accelerations were reduced to 30 to 40 percent of the values when $E_f/E_c = 1/1.25$. All of the information discussed in these paragraphs is summarized in the tables 3 and 4 following.

Table 3	Accelerations								
	Cross-Canyon			Upstream-Downstream			Vertical		
Canyon Node	Max Acc (g)	ASI* cm/s	Frequency content	Max Acc (g)	ASI* cm/s	Frequency content	Max Acc (g)	ASI* cm/s	Frequency content
$E_f/E_c = 1/1.25$ $\alpha = 1.0$	0.18	185	3.3 - 5	0.19	199	4 - 6	0.23	199	4 - 10
$E_f/E_c = 1/1.25$ $\alpha = 2.0$	0.14	138	3.3 - 4	0.13	137	3.3 and 6.7	0.24	163	4 - 10
$E_f/E_c = 1/7$ $\alpha = 2.0$	0.05	41	2	0.04	34	1.2 – 1.4	0.08	82	2.5

Table 4	Accelerations								
	Cross-Canyon			Upstream-Downstream			Vertical		
Abutment Node	Max Acc (g)	ASI* cm/s	Frequency content	Max Acc (g)	ASI* cm/s	Frequency content	Max Acc (g)	ASI* cm/s	Frequency content
$E_f/E_c = 1/1.25$ $\alpha = 1.0$	0.21	243	2 - 5	0.25	299	3 - 5	0.24	236	2.2 – 6.7
$E_f/E_c = 1/1.25$ $\alpha = 2.0$	0.13	170	2.5 - 3	0.17	199	3 - 4	0.19	185	2.2 – 6.7
$E_f/E_c = 1/7$ $\alpha = 2.0$	0.06	34	1.4 – 1.6	0.05	41	1.3 – 1.8	0.07	60	1.6 – 2.5

*ASI is acceleration spectral intensity which is the area under the acceleration response curve at 5% 5-percent damping between 0.1 and 0.5 s. It is a measure of the energy in the record.

5.3.2 Stresses in Dam Elements

Vertical stresses in two elements at the base of the dam were compared for the analyses with $E_f/E_c = 1/1.25$ and $E_f/E_c = 1/7$. Stresses in the element on the upstream face are plotted in figure 5.3-9. Stresses in the element on the downstream face are plotted in figure 5.3-10. In both of these figures there is an additional stress plot that indicates the effect of varying the value of α .

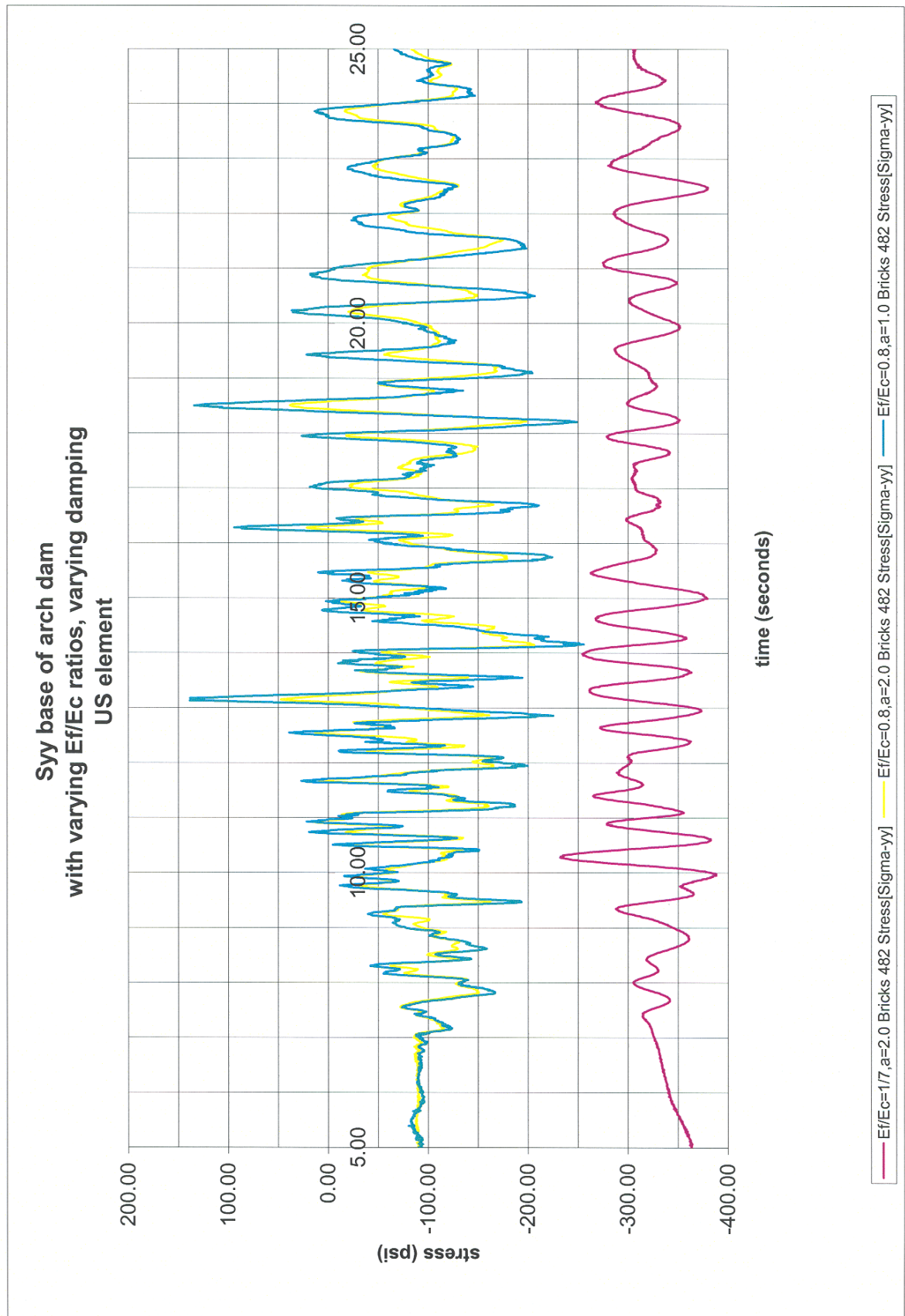


Figure 5.3-9 Stresses in the element on the upstream face of the arch dam.

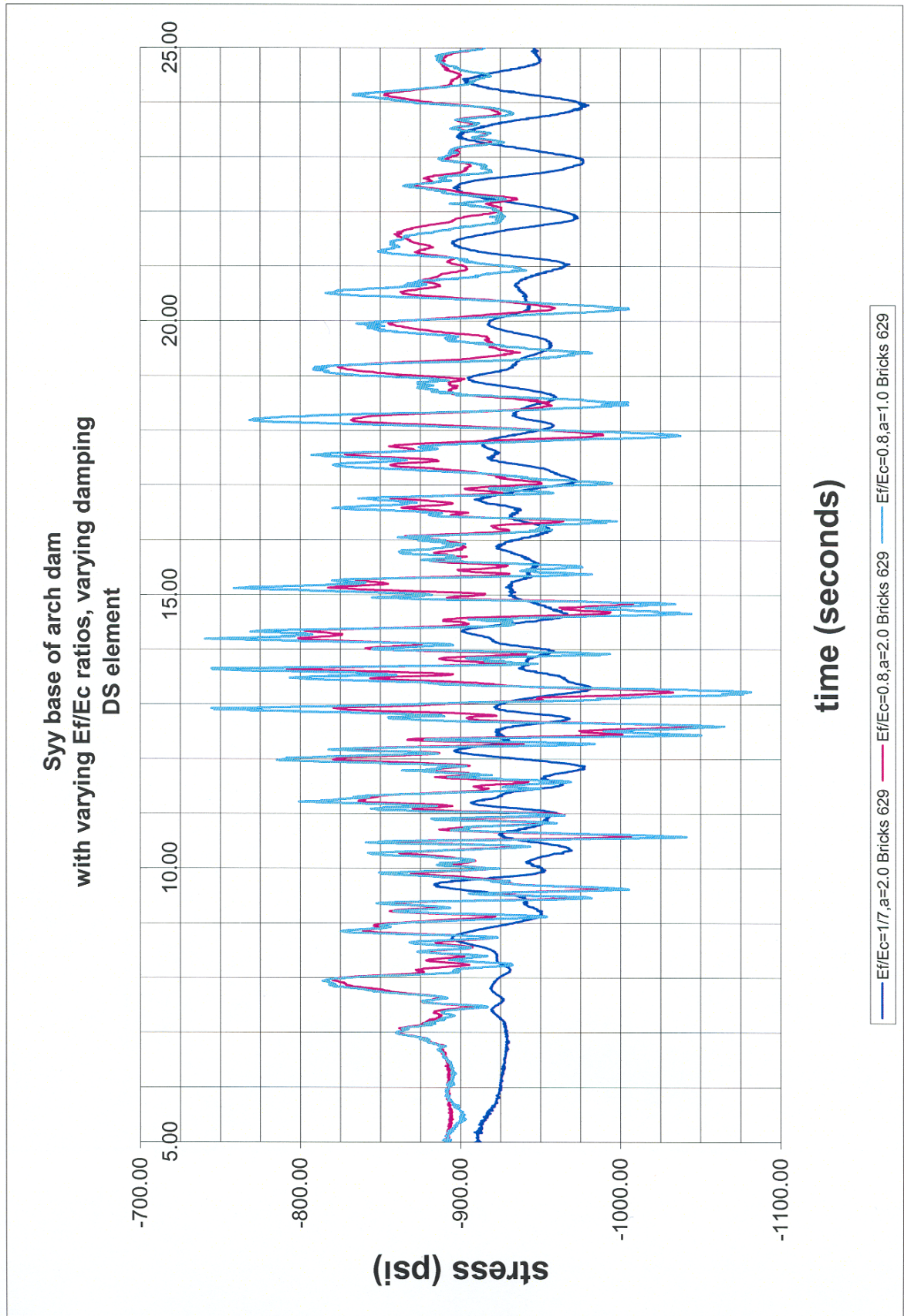


Figure 5.3-10 Stresses in the element on the downstream face of the arch dam.

In figure 5.3-10, it can be seen that for the downstream element, when the ratio E_f/E_c is equal to $1/7$ and an $\alpha = 2.0$ is used, the stresses in the element vary between approximately 880 and 980 lb/in² compression, a difference of 100 lb/in². When the ratio is $E_f/E_c = 1/1.25$, the stresses vary between 795 and 1,050 lb/in² compression, a difference of 255 lb/in². The softer foundation allows less energy from the ground motions to be transmitted into the dam or results in more radiation damping (i.e. more energy is allowed to leave the dam and radiate outward into the foundation). When the ratio is reduced, there is a reduction in the stresses of approximately 50 percent and the frequency content of the time history record changes. For the ratio $E_f/E_c = 1/7$, the predominant frequency of the response is approximately 2 Hz. For the ratio $E_f/E_c = 1/1.25$, the time history response contains higher frequencies, as well. When the damping value is reduced to $\alpha = 1.0$, the stresses vary between 745 and 1,075 lb/in² compression. The reduction in the damping value causes an increase in the stresses of about 30 percent, but does not cause any change in the frequency content of the response. There is also a shift in the value about which the stresses oscillate. For the analysis with a ratio of $E_f/E_c = 1/7$, the stresses oscillate about 935 lb/in² compression. For the analysis with a ratio of $E_f/E_c = 1/1.25$, the stresses oscillate about 900 lb/in² compression. This means that when gravity is applied, the dam with the softer foundation is in a higher compressive state.

In figure 5.3-9, the difference in the stresses of an upstream element can be seen. The same impact on the stresses as described above is seen in this plot. The stress about which the values oscillate varies greatly for the upstream element discussed here. When a ratio $E_f/E_c = 1/1.25$ is assumed, the stress values oscillate around approximately 90 lb/in² compression. Tension values in the element reach a maximum value of 145 lb/in² when $\alpha = 1.0$ is used. When the damping is increased to $\alpha = 2.0$, the maximum stress decreases to 50 lb/in². When the ratio $E_f/E_c = 1/7$ is used, this stress about which the dynamic stress values oscillate becomes approximately 325 lb/in² compression. The primary impact of this is that the element stays in compression during the entire dynamic analysis. The difference in frequency content is also seen clearly in these plots. When the ratio E_f/E_c is $1/7$, the higher frequencies are not present and the primary frequency content is 1 to 2 Hz.

5.3.2 Static and Dynamic Water Pressures

Figures 5.3-11 through -13 show plots of static water pressure and dynamic water pressure for the models with different E_t/E_c ratios and damping values. The dynamic water pressure was taken at one time step of the analysis. The three plots display water pressures from 1) an analysis with a ratio $E_t/E_c = 1/1.25$ and a damping value of $\alpha = 1.0$; 2) an analysis with a ratio $E_t/E_c = 1/1.25$ and a damping value of $\alpha = 2.0$; and 3) an analysis with a ratio $E_t/E_c = 1/7$ and a damping value of $\alpha = 2.0$. For all of these runs, static atmospheric pressure was also applied. Static water pressures were compared for all three runs and found to compare extremely well with theoretical static values.

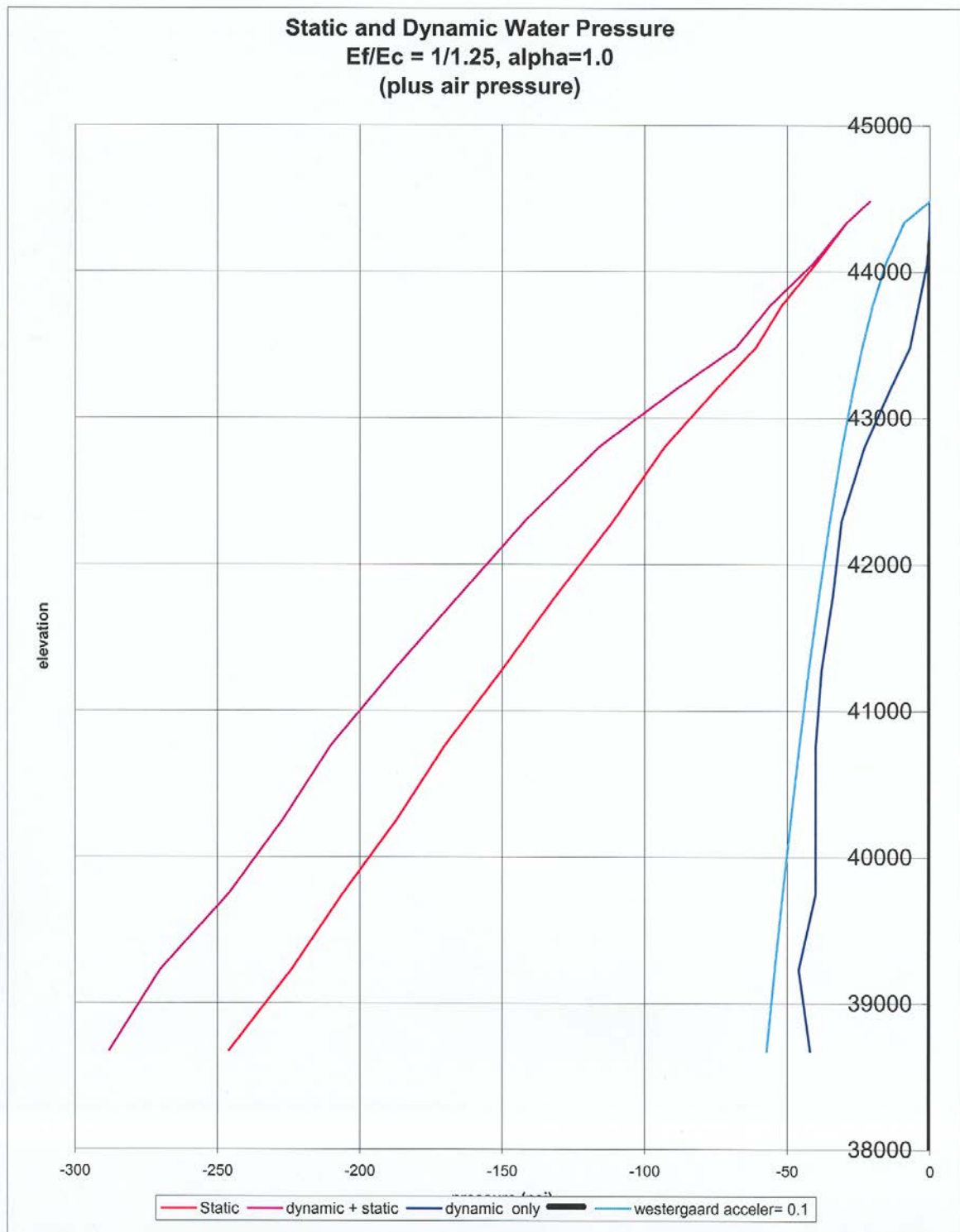


Figure 5.3-11 Static and dynamic water pressure— $E_f/E_c = 1/1.25$, $\alpha = 1.0$.

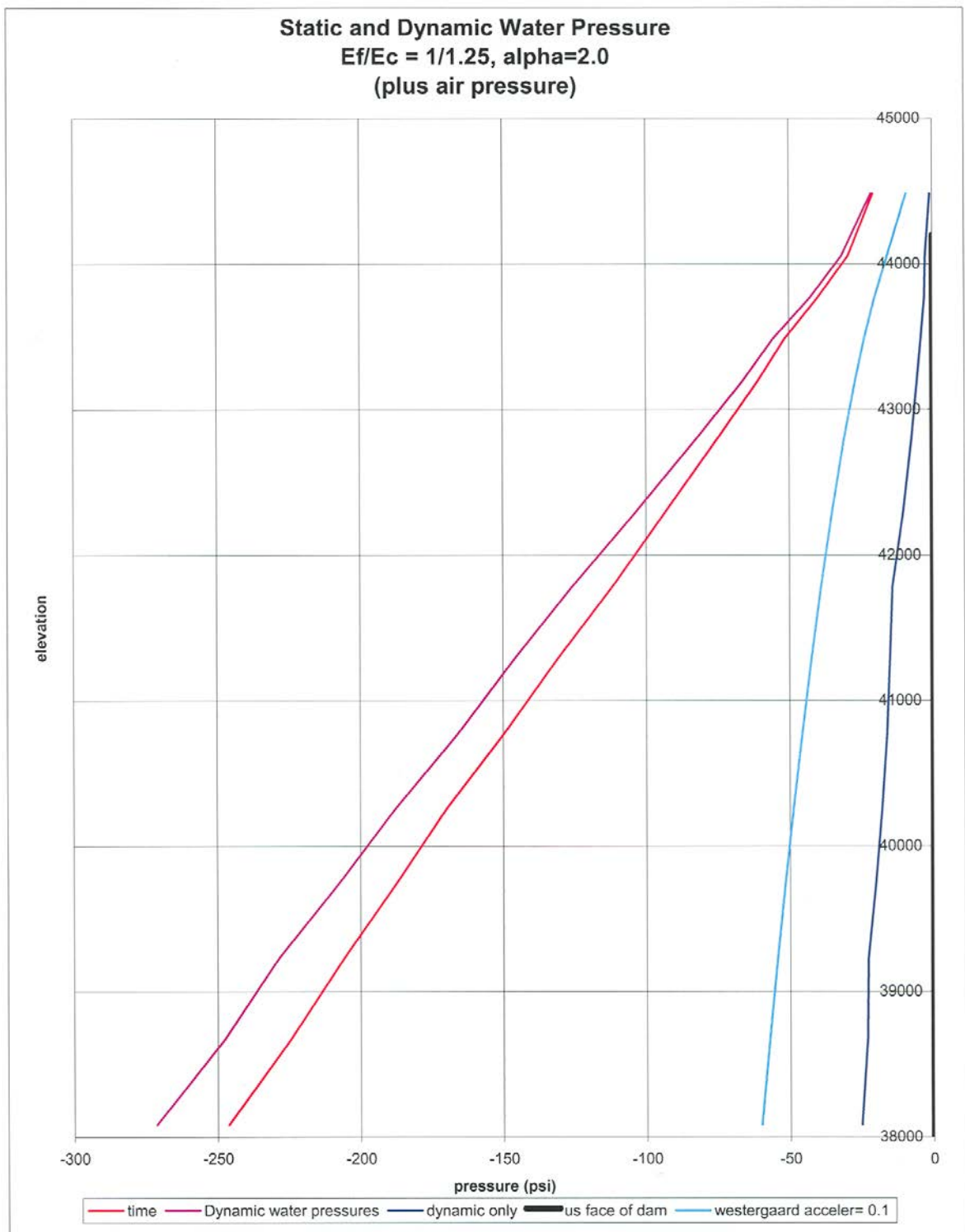


Figure 5.3-12 Static and dynamic water pressure— $E_f/E_c = 1/1.25$, $\alpha = 1.0$.

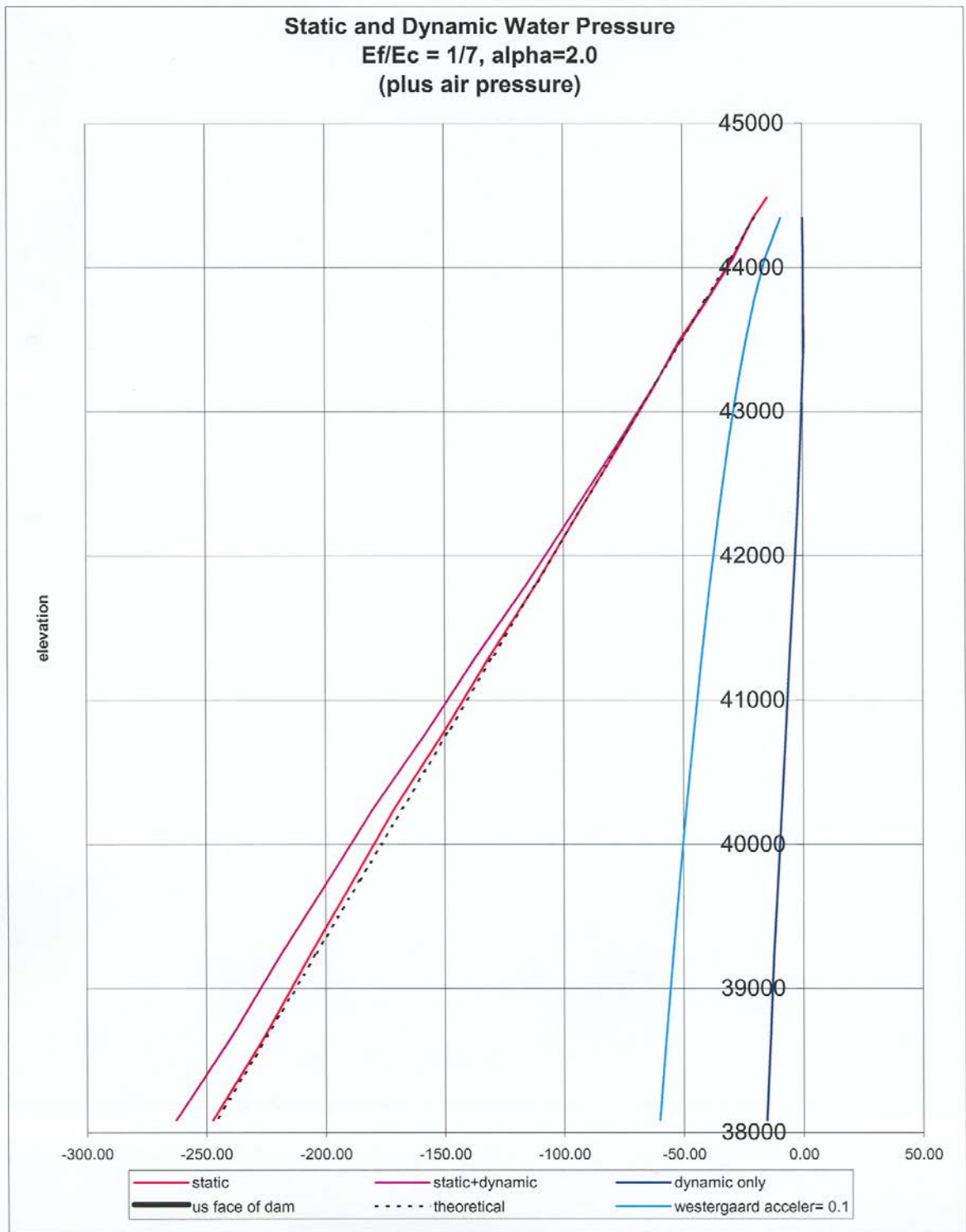


Figure 5.3-13 Static and dynamic water pressure— $E_f/E_c = 1/1.25$, $\alpha = 1.0$.

Consideration of dynamic values indicates that when a ratio $E_f/E_c = 1/1.25$ and a damping value of $\alpha = 1.0$ are assumed, the dynamic water pressures at one time step compared closely to Westergaard's (Westergaard, 1931) pressures for an acceleration of 0.1g. (The particular time step for which data was plotted was chosen by determining the time at which a maximum pressure near the base of the dam occurred.) When the damping value was changed to $\alpha = 2.0$, the dynamic water pressure was greatly decreased. Finally, for a ratio $E_f/E_c = 1/7$ and a damping value of $\alpha = 2.0$, the dynamic water pressures were further reduced. (For this analysis, the maximum water pressure at the base of the dam occurred at a different time in the record.)

6.0 Conclusions

The goal of this study was to capture some of the unknowns related to the effect of various parameters on finite element analysis results and to find methods to successfully model and apply the desired loads to concrete gravity, arch, and slab-and-buttress dams, so that the behavior of the structure is correctly analyzed. The structural finite element methods used currently at Reclamation are state-of-the-art and require significant background knowledge.

Important conclusions determined from these studies include:

- When spatially varying ground motions are included for fairly wide canyons, in particular for slab-and-buttress dams, some aspects of the response (forces in lower struts) are likely to increase over uniform motions. However, other factors (maximum principal tensile stresses in the buttresses) may remain largely unchanged.
- When modeling foundations with mass included, it is important to check deconvolved ground motions using the foundation model only. The foundation depth should be less than the horizontal dimensions to properly capture the horizontal ground motions, and the canyon topography should be modeled to help mitigate the tendency for over-amplification of the vertical ground motions. For nine times, the dam height appears to be a reasonable guideline for the horizontal extent of the model.
- Foundation mesh density should be the coarsest possible to improve run times, based on consideration of material properties, wavelengths, and the predominant frequency of the foundation.

- Reclamation Dam Safety decisions are currently being made within a risk-based context, requiring estimates of failure likelihood. This has necessitated a push toward non-linear analysis. Determining the natural frequencies and damping of non-linear systems is not trivial. The method of applying gravity load and monitoring oscillations and decay of pertinent parameters as the system settles down seems to work well. It is important to adjust Rayleigh damping factors such that the overall system response (including Rayleigh damping and inherent non-linear damping) provides the appropriate range (usually between 5 and 10 percent) of critical damping. Reasonable results are seen when α values are between 1 and 2.
- The ratio between the foundation and concrete modulus is an important consideration when performing explicit dynamic finite element analysis. If this ratio is small, the system can be over-damped due to large radiation damping. If vibration testing or other means to calibrate the model are not available, small values for this ratio should be avoided.
- Fluid elements in the explicit code *DYNA* appear to provide good estimates for static and dynamic reservoir pressures. However, damping affects the reservoir elements, offering another reason to keep α values between 1 and 2.

The studies undertaken for this research are currently being applied to analyses performed by the Structural Analysis Group.

7.0 References

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