EFFECT OF CONTRACTION JOINT OPENING ON PACOIMA DAM IN THE 1994 NORTHRIDGE EARTHQUAKE

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University of California
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Data Utilization Report CSMIP/00-05 (OSMS 00-07)

California Strong Motion Instrumentation Program

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CALIFORNIA DEPARTMENT OF CONSERVATION
DIVISION OF MINES AND GEOLOGY
OFFICE OF STRONG MOTION STUDIES

DIVISION OF MINES AND GEOLOGY
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DISCLAIMER

The content of this report was developed under Contract No. 1093-558 from the Strong Motion Instrumentation Program in the Division of Mines and Geology of the California Department of Conservation. This report has not been edited to the standards of a formal publication. Any opinions, findings, conclusions or recommendations contained in this report are those of the authors, and should not be interpreted as representing the official policies, either expressed or implied, of the State of California.

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PREFACE

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

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

1. Understand the spatial variation and magnitude dependence of earthquake strong ground motion.
2. Understand the effects of earthquake motions on the response of geologic formations, buildings and lifeline structures.
3. Expedite the incorporation of knowledge of earthquake shaking into revision of seismic codes and practices.
4. Increase awareness within the seismological and earthquake engineering community about the effective usage of strong motion data.
5. Improve instrumentation methods and data processing techniques to maximize the usefulness of SMIP data. Develop data representations to increase the usefulness and the applicability to design engineers.

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

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ABSTRACT

The response of Pacoima dam in the Northridge earthquake of 1994 is examined using the strong motion records obtained by CDMG accelerographs located in the canyon and on the dam body. A post–earthquake inspection of the dam showed the occurrence of minor cracking and block offset in parts of the dam body. The contraction joints between the cantilever monoliths appeared to have opened during the earthquake because of their clean appearance after the earthquake. The joints closed under static forces after the earthquake, except for the left–most joint which had a permanent opening of two inches because the abutment thrust block slid downstream.

By examining mathematical models of the dam, the seismic response of the dam was found to be influenced by opening–closing of contraction joints and horizontal joints, spatial variation of the seismic input, and amplification of seismic waves due topographical effects. Using a simple assumption for the distribution of free–field motion along the dam–foundation interface and considering the opening–closing of the joints, the response of the dam was computed analytically. The computed response from the model is larger in amplitude than the recorded response, but many of the overall characteristics between the two are similar. The differences between the model and the recorded responses illustrate the uncertainty in many factors affecting the earthquake response of concrete dams, such as input motion to the dam, dam-rock interaction, and energy dissipation.

Study of the CDMG processed records of Pacoima Dam in the 1994 Northridge earthquake and comparison with the analytical results from the models indicated the following conclusions: (i) The contraction joints opened during the earthquake, and the effect of the joint opening is an important factor in the response; (ii) the non–uniform free–field ground motion caused by topographic amplification has a significant effect on the dam response; (iii) the computed response using uniform free–field ground motion does not provide an adequate representation of dam performance; (iv) damping due to foundation rock radiation appears to be important; and (v) the pseudo–static effects of the non–uniform ground motion cause high stresses.

APPLICATION TO CODES AND PRACTICE

Seismic analysis and design of arch dams should be performed with proper attention given to the effects of local canyon topography on the free–field motions. To the extent possible, an appropriate seismic wave type should be selected and the response of the canyon to the selected incident wave should be investigated. The effects of the spatial variation of the free–field motion must be accounted for in the analysis.

A nonlinear analysis including joint opening effects should be performed for determining the response of an arch dam to strong ground motions. Based on results of the present study and in view of the performance of Pacoima dam during the Northridge earthquake, a nonlinear analysis procedure considering opening–closing of contraction joints, lift joints, and dam–foundation interface only can be used for most practical cases. It is necessary to use more than three or four joints in the model to represent the effect of joint opening on the stresses, particularly near the abutments, with the non–uniform ground motion. A complete nonlinear analysis including a nonlinear constitutive model for concrete may also be useful especially for very unusual cases, but validated analysis procedures are not available at this time.

The energy loss due to propagation of seismic waves into the foundation must be accounted for in the seismic analysis. To this end, an appropriate radiation damping should be developed and be included in the model.

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1. INTRODUCTION

Concrete arch dams, such as Pacoima Dam in Los Angeles County, are constructed as cantilever monoliths separated by contraction joints. Since the joints, which are usually oriented in vertical planes, cannot transfer substantial tensile stresses in the arch direction, they may open as the dam vibrates in response to earthquake ground motion. The earthquake response of arch dams is further complicated by the effects of dam–water interaction, dam–foundation rock interaction, and the spatial variation of the input motion along the interface between the dam and canyon.

A seismic safety evaluation of an arch dam often involves a linear dynamic analysis assuming the dam is a monolithic structure and the free–field ground motion is uniform for the canyon. The joint opening behavior is not represented, and hence a linear analysis can show unrealistically large tensile stresses in the arch direction. Design criteria have been developed to interpret arch dam behavior from linear earthquake analyses, but they involve ad hoc explanations for the acceptability of large arch tensile stresses. Although the effect of contraction joints had been examined analytically and from shaking table tests, there had not been evidence of contraction joints opening during an earthquake before January 17, 1994, when Pacoima Dam was subjected to the Northridge earthquake. The contraction joints opened during the strong vibration. The joints closed under static forces after the earthquake, except for the left–most joint which had a permanent opening of two inches because the abutment thrust block slid downstream. Minor cracking and block offset occurred in parts of the dam body.

In the 1971 San Fernando earthquake, the dam experienced strong motion close to the epicenter. The left abutment was damaged as in the Northridge earthquake, although the permanent opening was only 3/8–inch after that earthquake, and several rockslides had to be repaired by post–tensioning the thrust block. A single accelerometer above the left abutment recorded a peak acceleration of 1.25 g, one of the largest recorded accelerations up to 1971. The topographic amplification of earthquake vibration in a canyon and the non–uniform input motion for an arch dam were recognized over twenty years ago in a study of Pacoima Dam in the San Fernando earthquake (Reimer, 1973). The peak values computed in that study for the base rock and dam accelerations were remarkably similar to the peak accelerations recorded in the Northridge earthquake. An examination of the role of the contraction joints was later conducted by Dowling and Hall (1989).

The strong motion response of Pacoima Dam in the 1994 Northridge earthquake was recorded by a network of California Division of Mines and Geology (CDMG) accelerometers (CSMIP, 1994). The records show peak accelerations that are among the largest measured during an earthquake. For example, peak accelerations of 1.6 g and 1.2 g were recorded at the left abutment of the dam in horizontal and vertical directions, respectively. Smaller peak accelerations were recorded at the base of the dam, with 0.5 g and 0.4 g peaks for the horizontal and vertical directions, respectively, indicating a significant amplification of ground motion because of the canyon topography.

The strong motion data from the Northridge earthquake offers a unique opportunity to evaluate the contraction joint behavior in an arch dam subjected to a large earthquake. The objective of this study is to examine the response of Pacoima Dam in the Northridge earthquake using the computer program ADAP–88, which has been developed at the University of California, Berkeley, for nonlinear analysis of arch dams considering joint opening effects (Fenves, Mojtahedi et al., 1992; Fenves, Mojtahedi et al., 1992). An additional objective is to examine the effects of non–uniform free–field motion on the response of arch dams. Although there has been at least one study on the effects of non–uniform free–field motion on the earthquake response of arch dams (Nowak and Hall, 1990), there has been no recorded response data available from dams subjected to strong ground motion.

In this study, seismic analyses of the Pacoima dam are performed with the joints assumed closed (monolithic structure), and additional cases examine the response when the joints are allowed to open. The effects of joint opening on the acceleration response and maximum arch and cantilever tensile stresses are examined through a detailed comparison of the computed responses. In view of the importance of the ground motion variation along the abutment of the dam, seismic analyses are performed considering non–uniform ground motion based on simplifying assumptions, and the results are compared with seismic response which would have been obtained using uniform seismic input.

Chapter 2 describes the dam and the model developed for the seismic analyses. Chapter 3 presents the results of the seismic analyses showing the effects of contraction joint opening and non–uniform free–field motion on the response of the dam in the 1994 Northridge earthquake. Finally, Chapter 4 presents the conclusions and recommendations of the study.

2. RESPONSE OF PACOIMA DAM IN THE NORTHRIDGE EARTHQUAKE AND FINITE ELEMENT MODELING

2.1 Description of Pacoima Dam

Pacoima dam is a flood control concrete arch dam located on Pacoima Creek, approximately 4.5 miles northeast of San Fernando, California. The dam was constructed between 1925 and 1929. The dam crest is 365 ft above the base at the crown section, and the crest length is 589 ft. The thickness of the dam at crown section varies from 10.4 ft at the crest to 99 ft at the base. The dam has eleven contraction joints with beveled contraction joint keys that are 12 in. deep. The left abutment of the dam is supported by a concrete thrust block through an approximately 60 ft high contraction joint. A plan, the crown section and the profile of the dam are shown in Figure 2.1. The reservoir level at the time of 1994 Northridge earthquake was 233 ft above the base (132 ft below crest), about two–thirds full.

2.2 Response of the Dam in the Northridge Earthquake

The epicenter of the 1994 Northridge earthquake (M_S=6.8) was approximately 11 miles southwest of the dam. An assessment of the damage is reported in Morrison-Knudsen (1994). The dam and its abutment were subjected to severe shaking causing rock falls and rock slides, extensive cracking of gunite coatings on the rock faces, minor cracking of dam concrete. The contraction joint between the dam and the left thrust block opened and remained open after the earthquake about 2 in. at the crest and 1/4–in. at the bottom. Some differential vertical movement occurred at the joint with the thrust block remaining lower with respect to the dam. The thrust block and its supporting rock mass moved away from the dam causing a large diagonal crack in the thrust block near the bottom of the thrust block joint.

Cracks and offsets also occurred in the dam body. After the earthquake, a fine diagonal crack was visible on the downstream face extending from the bottom of the open thrust block joint to the horizontal joint at 1967 ft elevation (48 ft below the crest). A permanent horizontal offset of 3/8 to 1/2–in. occurred along the horizontal joint at 1967 ft elevation, with the top block moving downstream relative to the bottom block. Both the crack and offset are believed to be associated with the foundation failure and opening of the left thrust block.

No significant damage occurred at the other vertical contraction joints in the dam. However, there were indications that some of the joints have opened and closed during the earthquake. Vertical offsets across the joints were visible at the crest after the earthquake, indicating that the right blocks have dropped slightly relative to the left blocks progressively across the crest of the dam. Visual inspection showed the joints clean of accumulated dirt and debris.

The acceleration response of the dam and the canyon to the earthquake was recorded by a network of CDMG strong motion accelerometers. Figure 2.2 shows the CDMG accelerometer locations on the dam (CSMIP, 1994). It was not possible for CDMG to digitize and process all the strong motion records because the traces were intertwined with large acceleration peaks that exceeded the range of the instruments. Complete, processed records are available for Channels 8 to 11 on the dam, two locations in the canyon, one at a downstream location and the other above the left abutment (see CSMIP Report Nos. 94–12A, 94–13, and 94–15A). Partial unprocessed records for Channels 1 to 6, 12, 13, 15, 16 and 17 are available in CSMIP Report 95–05.

The downstream instrument recorded a peak acceleration of 0.43 g. The three components of acceleration at this location are shown in Figure 2.3. The acceleration records from the instrument at the upper left abutment, shown in Figure 2.4, have a peak acceleration of 1.58 g. The difference between the canyon and upper left abutment records demonstrate the topographic amplification of ground motion in the canyon.

Considering the dam response, the accelerograms from the dam base are shown in Figure 2.5, and the crest accelerograms are shown in Figure 2.6. At the base, the peak acceleration was 0.53 g in the tangential direction and 0.41 g in the vertical direction. At the crest of the dam, the peak acceleration in the radial direction reached 2.3 g at the left quarter point. Figure 2.7 shows the radial accelerations recorded by Channel 8, which is located at the left quarter point at 80% height of the dam; the peak acceleration was 1.31 g. The motion of the dam shows higher frequency components than the downstream and base records, possibly because of higher mode contributions of the dam or impact caused by open contraction joints pounding closed.

Examining the response of the dam in the 1994 earthquake, Figure 2.8 shows the transmissibility function between radial motion at the base (Channel 9) and the radial motion of the dam at 80% height (Channel 8). The transmissibility function is computed from estimates of the power spectral density functions. Furthermore, the poles of the transfer function are determined to better locate the resonant frequencies. The system identification gives resonant peaks of the dam system at 4.0, 6.25, 8.25, 10.1, and 13.5 Hz. More refined analysis of the recorded data to identify vibration mode shapes was not performed because of the limited strong motion data available.

2.3 Finite Element Model of Concrete Arch Dam

The finite element program ADAP–88 allows modeling vertical contraction joints between linear substructures (Fenves, Mojtahedi et al., 1989; Fenves, Mojtahedi et al., 1992). The program has recently been verified by a shaking table test of a 1:300 scale model of an arch dam (Chen, Li et al., 1994).

For the present study, the program has been modified to use standard 3–D solid elements throughout the entire model (the original version used shell elements for the dam body). The new version of the program also allows for nonlinear joint elements to be used for simulating the opening of lift joints and joints at the dam–foundation rock interface. The program was also extended to allow specification of non–uniform free–field motion through displacement histories at the dam–foundation rock interface (Mojtahedi and Tseng, 1994).

The finite element model of the Pacoima dam is shown in Figure 2.9. A total of 588 eight–node 3–D solid elements, resulting in about 3600 degrees–of–freedom, are used to model the dam body. Nonlinear joint elements simulate the opening and closing of the contraction joints, lift joints and the dam–foundation rock interface. Three joint elements (and 3–D solid elements) are used through the thickness of the dam. Tangential slippage of the joints is not allowed in the model. As described later this assumption is valid for the keyed vertical joints, but it may not be true for the horizontal joints or abutment joints.

Although Pacoima dam has eleven vertical contraction joints, only five joints are included in the model: four in the body and one at the left thrust block. A previous study has shown that it is not necessary to include all joints in the model to capture the major effects of joint opening on the earthquake response of an arch dam (Fenves, Mojtahedi et al., 1992). As shown in Figure 2.9, Joint 3 represents the crown joint of the dam while interface of the dam and the thrust block is represented by Joint 5. Joints 1, 2 and 4 are in the interior of the dam. In the previous parameter study, which was performed using uniform free–field ground motion, three joints located at the quarter–points of a dam were found to be sufficient for a seismic analysis. However more joints were included in the Pacoima dam model for the following reasons:

- Preliminary analyses with non–uniform free–field input indicated that more joints are needed in right portion of the model for releasing arch stresses.
- Because of the permanent opening at the interface of dam and thrust block during the Northridge earthquake, the seismic response of the dam without the thrust block is of interest in the study. One joint was therefore used in the model for explicit representation of this interface.

As will be described in Chapter 3, the response of Pacoima dam to the Northridge earthquake indicates large tensile cantilever stresses particularly when contraction joints are allowed to open. These stresses cannot be withstood by the tensile strength of the concrete or the lift joints. Based on the preliminary analyses, it was concluded that the lift joints opened or concrete cracked during the Northridge earthquake. To recognize the opening of lift joints, two horizontal joints were included in the model. These are shown in Figure 2.9 as Joints 6 and 7 located at 202 ft and 97 ft above the base, respectively. Each joint spans an entire horizontal section between right and left abutments.

The material properties used for concrete are: modulus of elasticity = 2400 ksi, Poisson's ratio = 0.20, and unit weight = 150 lb/ft3. Theses properties, which are based on testing of core samples of concrete, were obtained from a previous study of the dam (County, 1983).

The first ten computed natural frequencies of the dam, with the reservoir level at the level during the Northridge earthquake, are : 4.3, 4.4, 6.2, 7.5, 7.8, 8.5, 9.1, 9.8, 10.4, and 11.3 Hz. The first two computed frequencies, 4.3 and 4.4 Hz, are 10% greater than the fundamental frequency indicated by the transmissibility function in Figure 2.9 (which has a broad peak at 4 Hz). Furthermore, the model shows more distinct modes than indicated by the transmissibility function. Considering the uncertainties in the material properties, and the limited processed strong motion data, the correspondence between frequencies from the free vibration analysis and those from transmissibility function was judged to give an adequate calibration of the vibration characteristics of the model.

For the earthquake analyses, Rayleigh damping was assumed for the dam–foundation system with the parameters selected to produce 10% damping at 5 Hz and 20 Hz. The damping was selected after several trial analyses. The assumed damping is reasonable because radiation damping and damage in the foundation rock is not included in the model and the broad peaks are observed in the transmissibility function (see Figure 2.8).

2.4 Foundation Rock Model and Dam–Foundation Rock Interaction

A foundation rock region with a depth approximately equal to the height of the dam is included in the model to account for dam–foundation rock interaction effects. To suppress stresses caused by propagation of seismic waves, the foundation rock is assumed to be massless. For analyses with uniform free–field ground motion, the ground motion is specified at the rigid base of the foundation model. For analyses with non–uniform free–field motion, the free–field ground displacements are specified at the nodes along the dam–foundation rock interface.

In view of the complexity of the foundation geometry and uncertainty in the foundation material properties, a prismatic shape is assumed for the canyon and a coarse mesh is used for finite element modeling of the foundation. A total of 220 3–D solid elements are used for the foundation rock domain. The material properties for the foundation rock are: modulus of elasticity = 2000 ksi, Poisson's ratio = 0.20, again based on previous investigations (County, 1983). The effect of the weak rock near the left abutment was bounded by including or excluding the abutment in different cases of the model.

The foundation rock model is a linear substructure and is represented in the solution process by its stiffness matrix defined at the degrees–of–freedom on the dam–foundation rock interface. This matrix couples all of the DOFs at the interface and increases the computation time of the analysis. To reduce the foundation coupling and hence the computation time, coupling is neglected for widely separated nodes on the dam-foundation rock interface. The effects of localization of foundation on the dam response have been previously found to be negligible.

2.5 Dam–Reservoir and Reservoir Model

The solution procedure in ADAP–88 is based on a time integration of the nonlinear equations of motion. The hydrodynamic pressure of the impounded water acting on the dam is represented by an added mass matrix neglecting compressibility of water (Kuo, 1982). The water compressibility can have important effects on the earthquake response of arch dams (Fok and Chopra, 1987). However, the frequency analysis required to include compressibility cannot be combined with the time domain analysis required to account for the nonlinear joint opening effects. Hence, the added mass approach is used in ADAP–88. For computing the added mass, the reservoir is assumed to be bounded by a cylindrical surface obtained by translating the dam–reservoir interface in the upstream direction. The silt was not included in the model because water at a low elevation has little effect on the earthquake response of the dam.

The seismic response of Pacoima dam has been studied for three levels of water in the reservoir:

- Water level of the time of Northridge earthquake, 233 ft above the base.
- Water level at the crest of spillway (normal pool condition), 300 ft above the base, which is used for the standard seismic safety evaluation of arch dams.
- A hypothetical extreme condition with the water level at 360 ft above the base, representing simultaneous flood and earthquake conditions.

The vibration frequencies of dam with water level of the 1994 Northridge earthquake were found to be essentially identical to the frequencies of the dam with empty reservoir. It is therefore concluded that dam–water interaction did not have much effect on the response of the dam to the Northridge earthquake. The hydrodynamic effects for water at elevation 300 ft (82% full) are still small. Only for the extreme full condition do hydrodynamic effects become important.

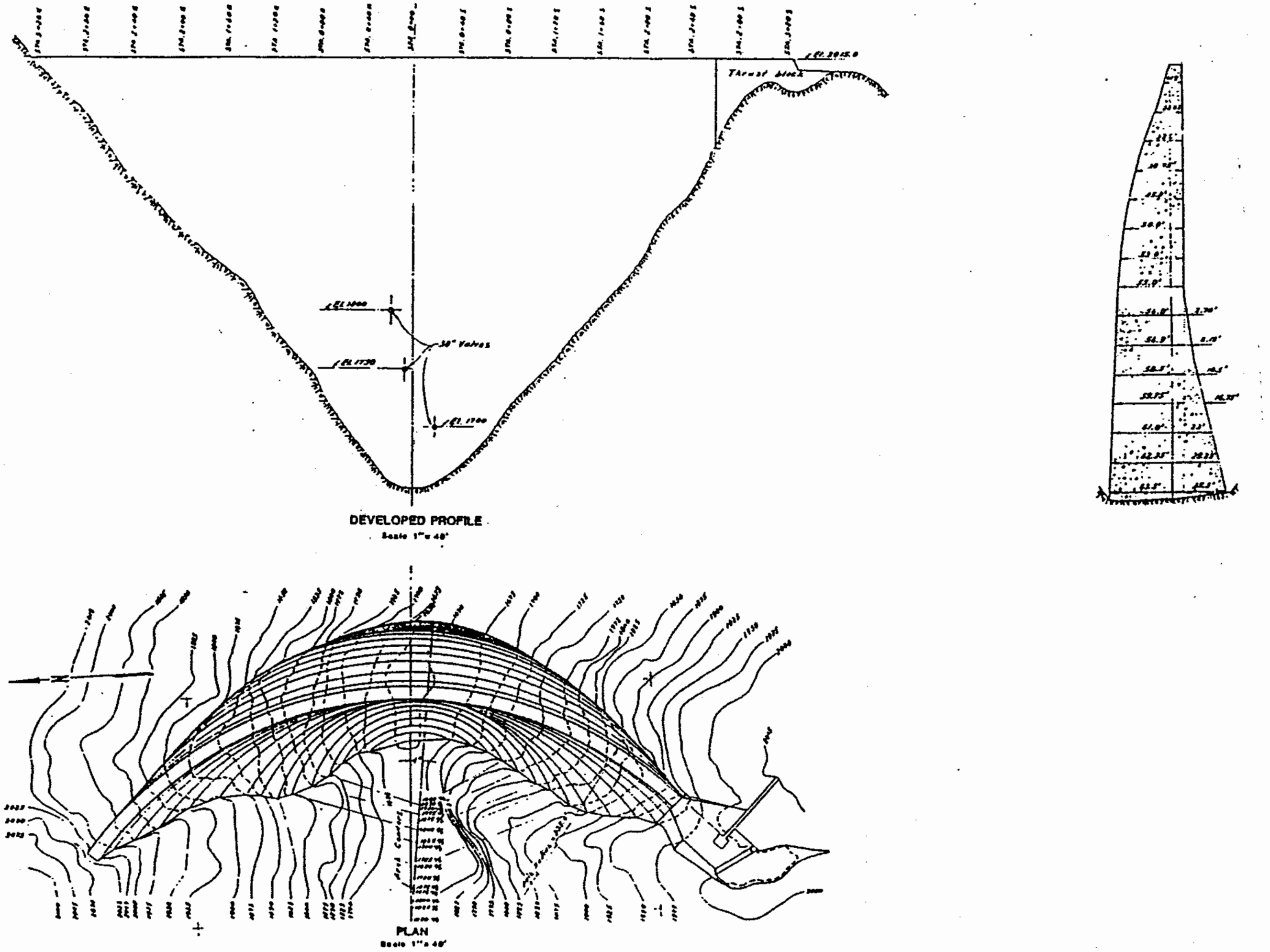


Figure 2.1 Plan, Profile and Crown Section of Pacoima Dam from Original Construction Drawings (County, 1983)

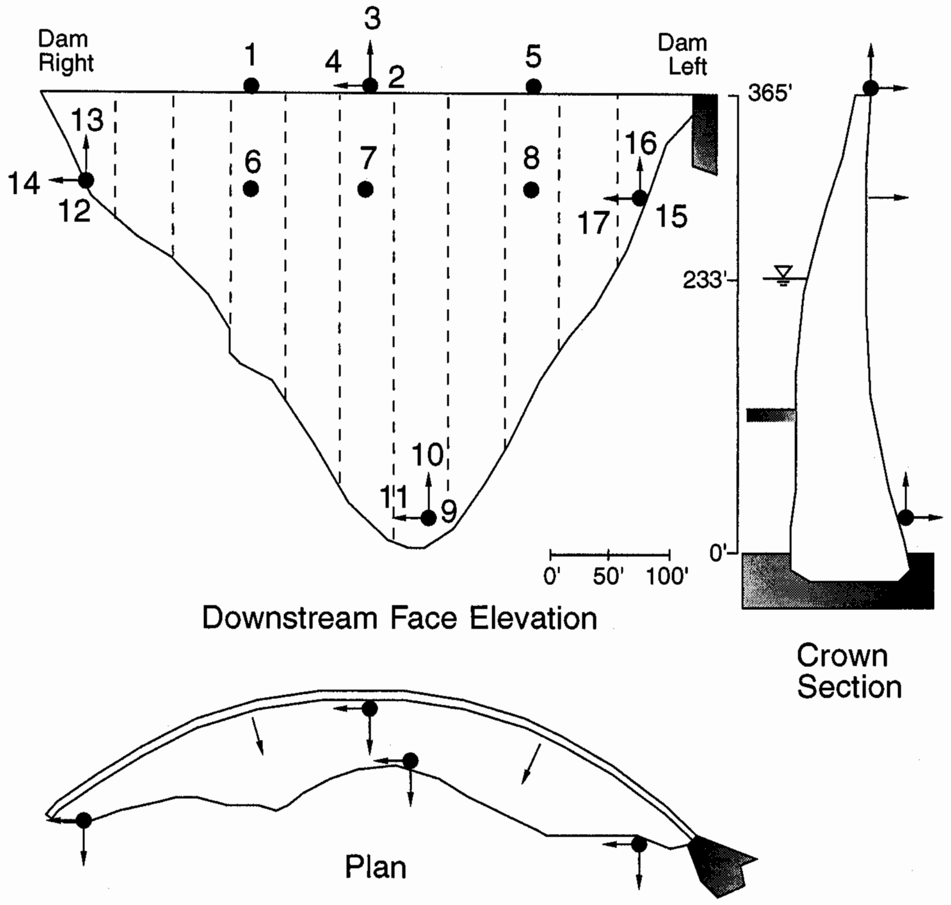


Figure 2.2 Instrumentation Plan for Pacoima Dam (CSMIP, 1994)

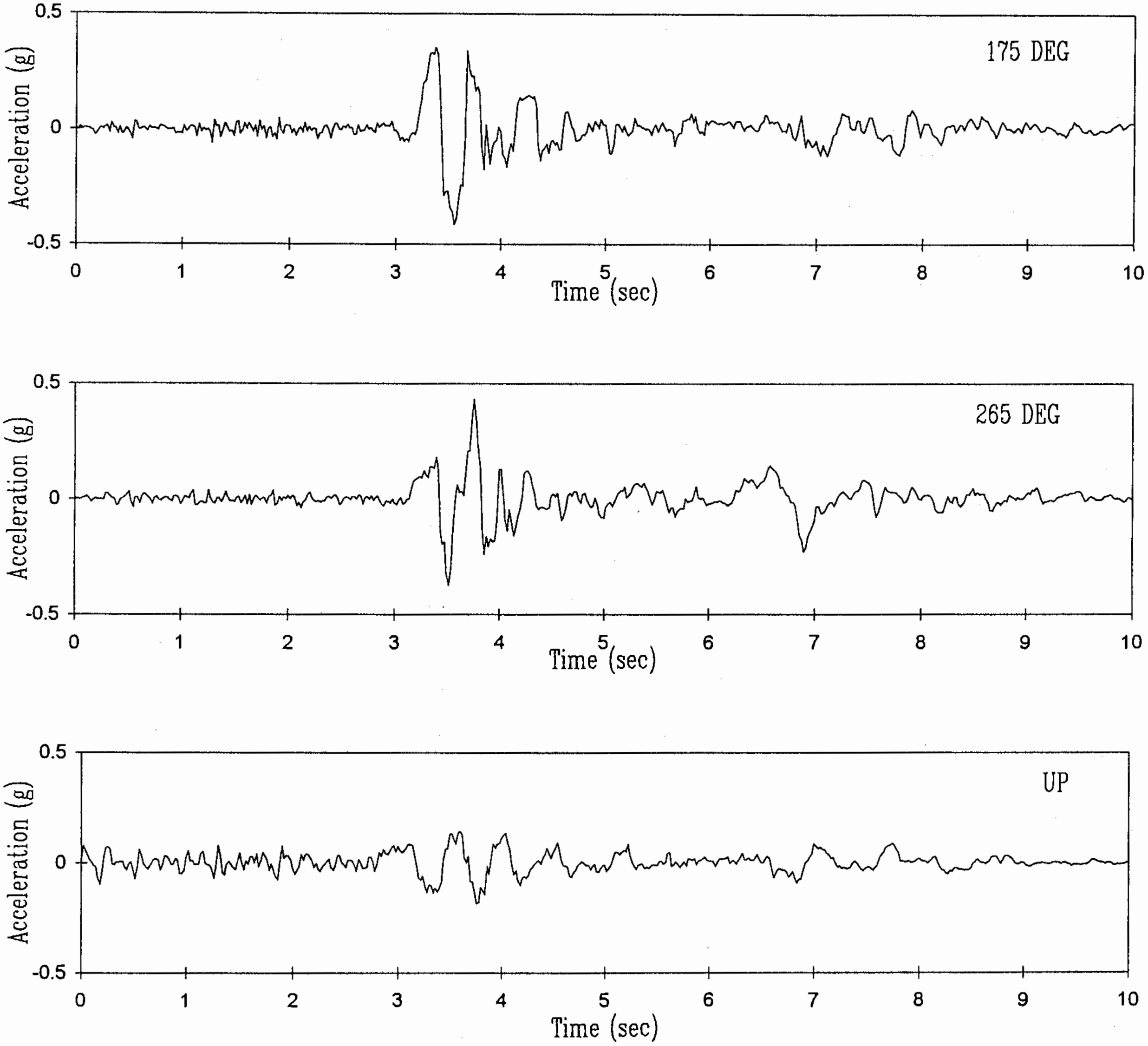


Figure 2.3 Accelerograms Recorded by the Downstream Instrument in Canyon. Cross-stream direction is Approximately 175 Deg and Stream Direction is Approximately 265 Deg.

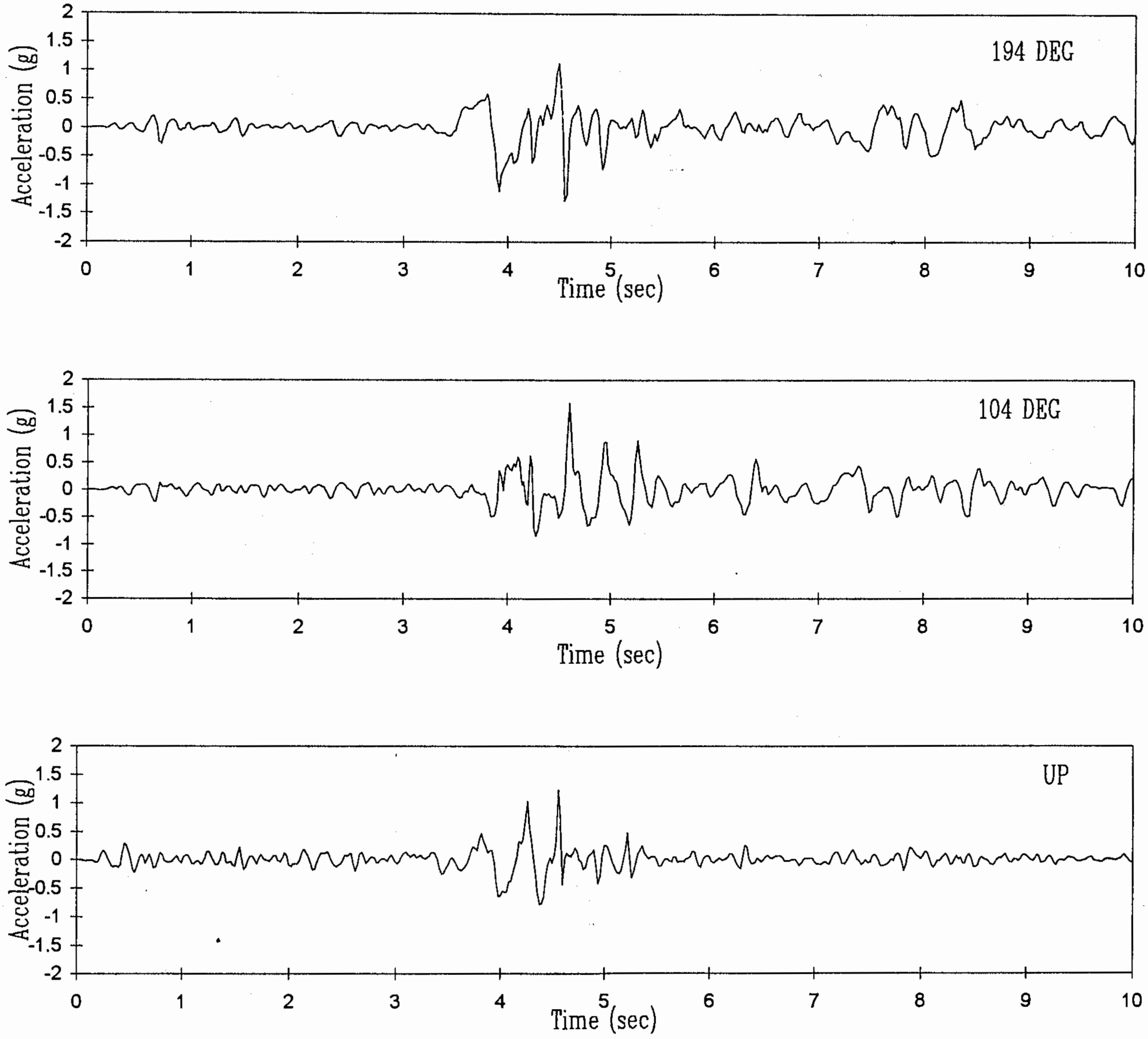


Figure 2.4 Accelerograms Recorded at Upper Left Abutment. Cross-stream is Approximately 194 Deg and Stream Direction is Approximately 104 Deg.

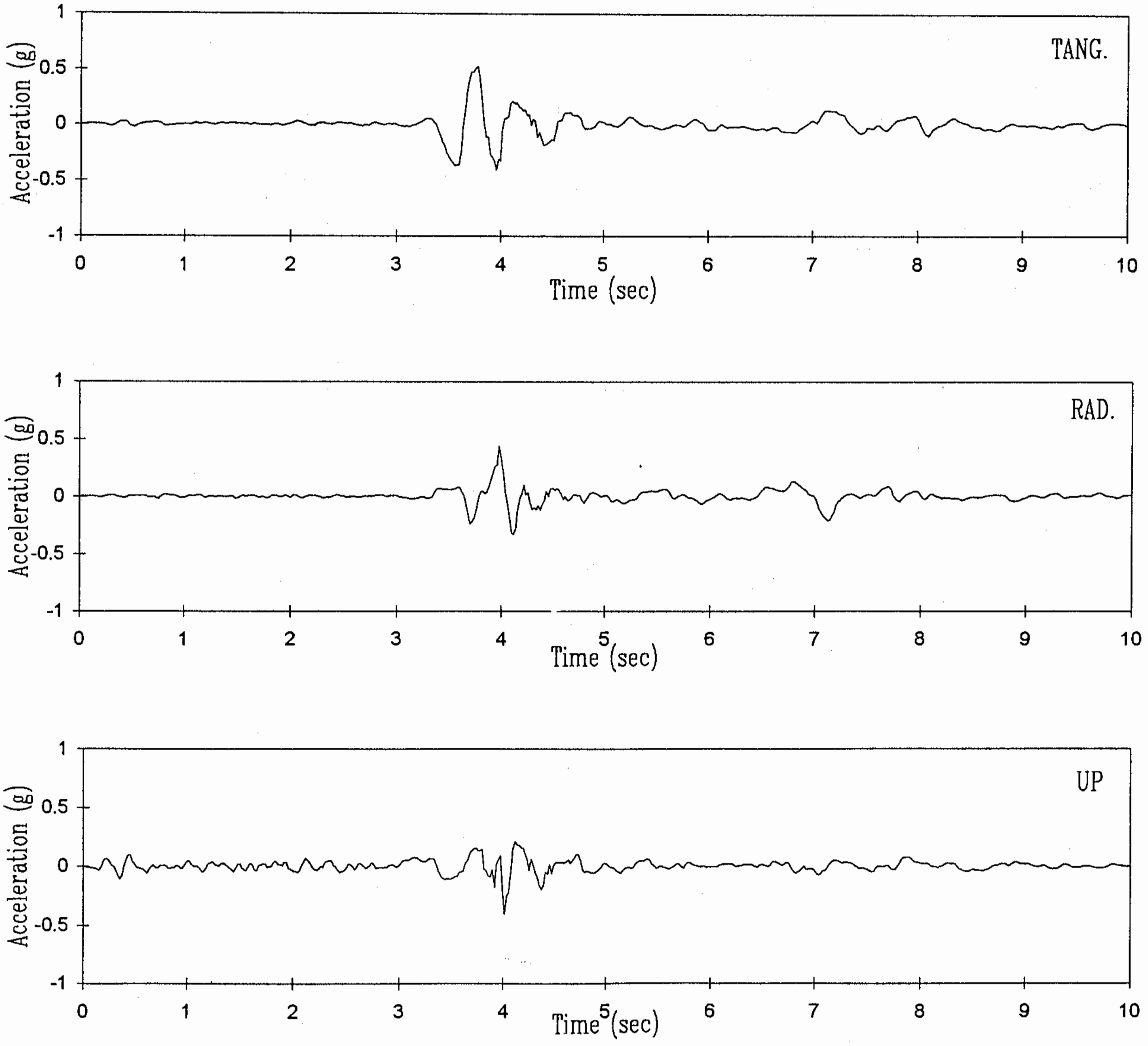


Figure 2.5 Accelerograms Recorded at Base of Dam in Local Coordinate System for Dam (Channels 9, 10, and 11).

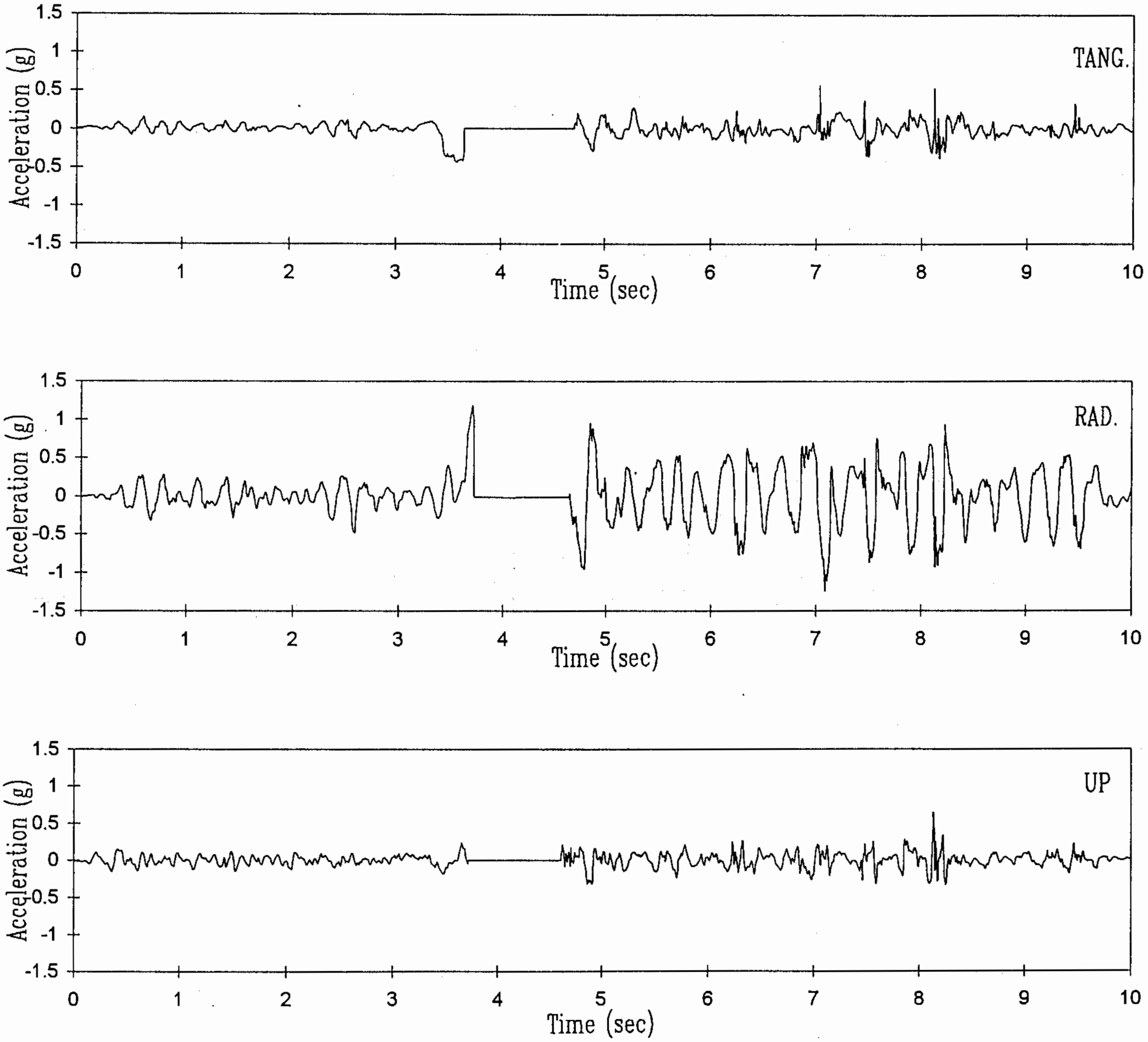


Figure 2.6 **Accelerograms Recorded at Center of Crest in Local Coordinate System for Dam (Channels 2, 3, 4).**

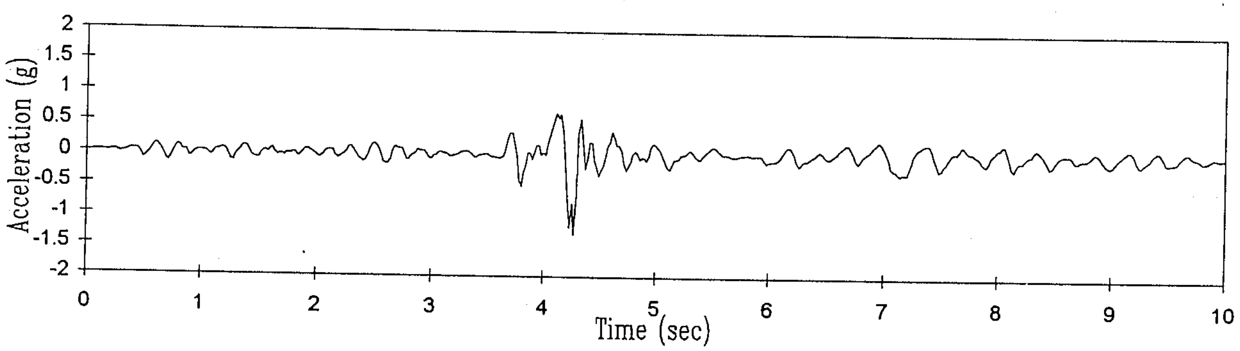


Figure 2.7 Accelerograms Recorded at Channel 8 (Radial Direction)

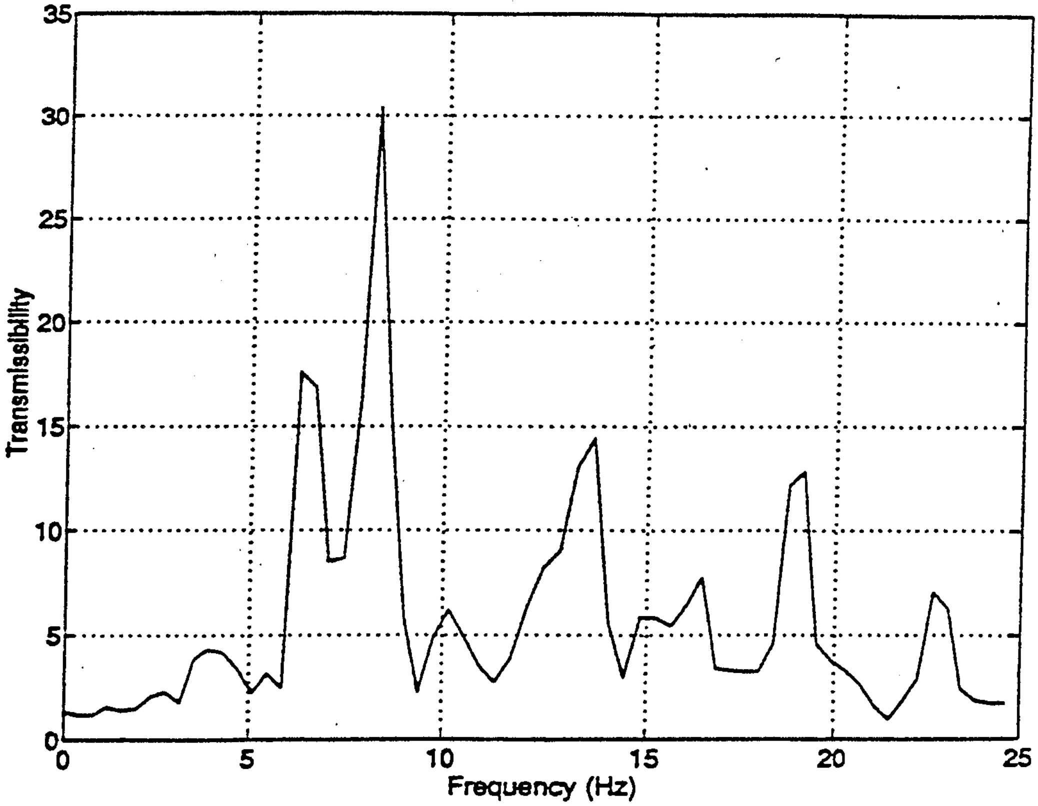


Figure 2.8 Transmissibility Function from Base to Channel 8 Location

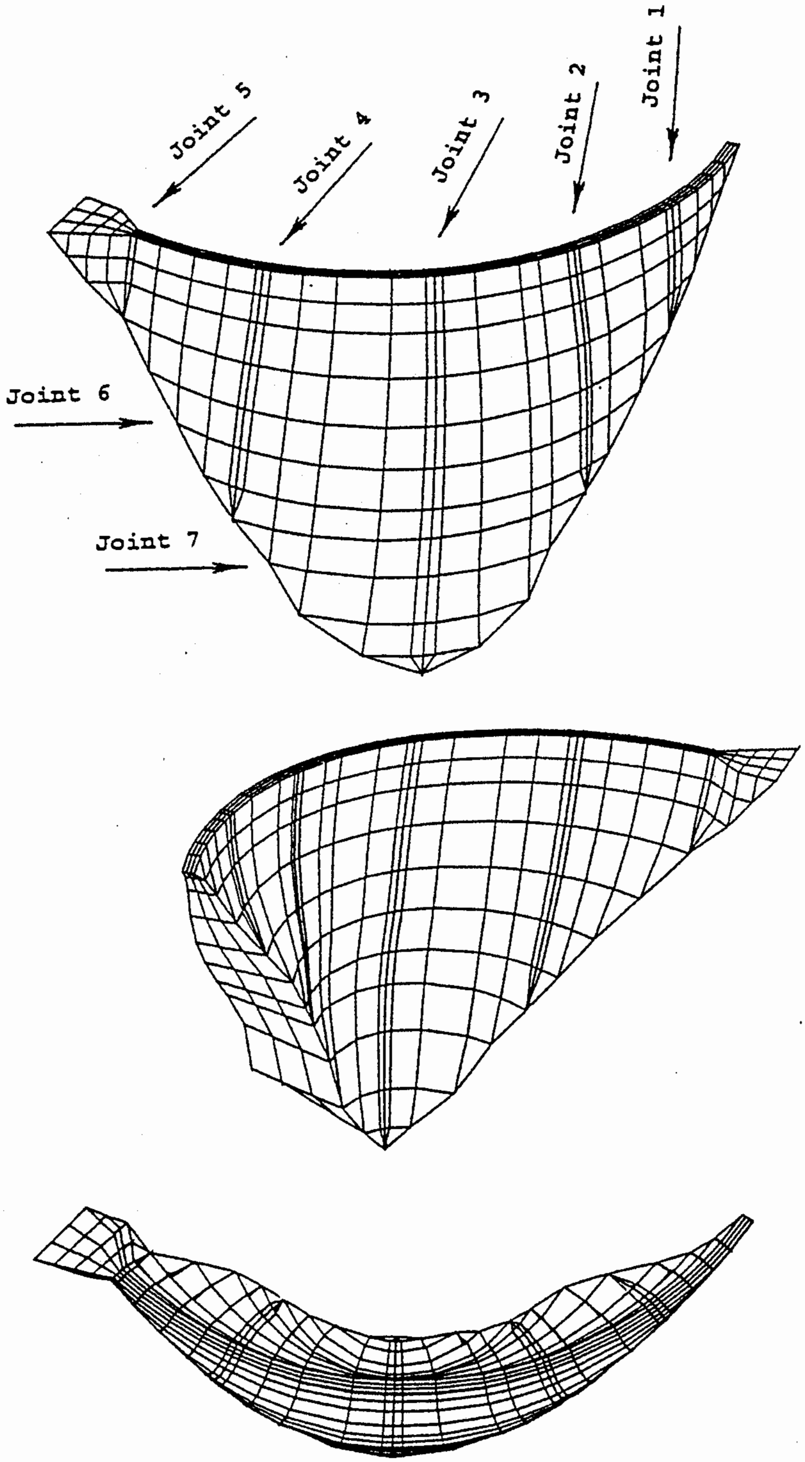


Figure 2.9 Finite Element Model of Pacoima Dam, Showing Locations of Vertical and Horizontal Joints. (Reservoir and Foundation Rock Not Shown.)

3. SEISMIC ANALYSIS OF PACOIMA DAM

3.1 Introduction

This chapter presents the seismic analyses of Pacoima dam using the strong motion accelerograms recorded by the CDMG network during 1994 Northridge earthquake as the input motion for the dam. The principal objectives of the analyses are to investigate two issues concerning the response of the dam to the Northridge earthquake:

- The amount of the contraction joint opening and the consequent effects on stresses in the dam.
- The significance of non–uniform free–field motion on the response of the dam.

In addition, other questions are addressed in the seismic analyses. As mentioned earlier, the interface of the dam and the thrust block on the left side had a permanent opening of about 2 in. at the crest. To assess the effects of the loss of arch support, an analysis was performed disregarding the resistance of the thrust block. The effects of the vertical ground motion component on the earthquake response of concrete dams have received attention in the past. Analyses of the dam with and without the vertical component in the Northridge earthquake were performed to evaluate the effect of the vertical component on the response of the dam. Another issue is the effect of the reservoir level on the earthquake response of the dam. Since the reservoir was only about two–thirds full at the time of the earthquake, the hydrodynamic effects on the dam were small. Additional analyses were performed with the reservoir level at the normal pool elevation (82% full at the level of the spillway), and a hypothetical extreme full condition with 5 ft freeboard.

Finally, the seismic response of the dam assuming monolithic cantilevers produces large tensile cantilever stresses, exceeding the tensile strength of concrete and indicating horizontal cracking of concrete or opening of lift joints. To account for such behavior, the response of the dam was analyzed using horizontal joint elements in addition to the standard vertical joint elements.

3.2 Assumptions for Input Motion

The earthquake input motion for the analysis of the dam is the free–field ground motion at the dam-foundation rock interface. The downstream accelerogram can be assumed to represent the free–field ground motion record at the downstream location. The recorded motion at the base of the dam is different than the free-field motion because of dam-foundation rock interaction effects. The records obtained from accelerometers at the base of the dam and the abutments include the modification of the free-field motion by topographic effects and dam–foundation rock interaction effects. To identify the free–field motion from the recorded motion, it would be necessary to perform a deconvolution analysis for the complete system.

In this study, however, a deconvolution analysis was not performed and the recorded motion at the base and left abutment were assumed to represent the free–field motion of the canyon. In other words, dam–foundation rock interaction effects were neglected for purposes of specifying the free–field motion at the canyon interface. For all cases, the specified motion is the free–field motion, the ground motion assumed to occur at the dam interface if the dam were not present.

Two types of earthquake analysis were performed depending on the specification of free–field motion. The first type of analysis used an assumed uniform free–field motion, and the second type used an assumed non–uniform free–field motion. Various records were used for the analysis cases with uniform free–field motion at the interface, which is the typical assumption used for seismic safety evaluations of arch dams.

The problem of determining the actual spatial variation of the non–uniform free–field motion in a canyon is difficult because the free–field motion depends on complicated wave propagation effects for seismic waves emanating from the extended source and scattering on the canyon and vibration of the canyon itself. The instrumentation for Pacoima dam is not sufficient to define the complete spatial variation of input motion. Even if it were sufficient, the recorded motion of the abutments would include dam–foundation rock interaction effects. A simple approach was adopted for capturing the overall effects of the canyon topography on the free–field motion for the 1994 Northridge earthquake. The free–field motion is computed from an interpolation of the recorded motion at the dam base and the recorded motion at the left abutment, which is the only processed abutment record available. The interpolation function should represent the dynamic characteristics of the canyon. For this study, a linear interpolation function is selected in lieu of a detailed model for wave propagation effects. Consequently, each component of the free–field motion at an elevation is a linear interpolate between the recorded motions at the base and the upper left abutment. As an example of the interpolation procedure, Figure 3.5 shows the acceleration histories for the input motion at elevation 280 ft of the model.

3.3 Cases Analyzed

To examine the issues identified in the previous sections, the finite element model of Pacoima dam described in Chapter 2 was analyzed for ten cases. Uniform free–field motion was used for the first three cases, whereas non–uniform free–field motion was used in the other seven cases. The conditions for the ten cases are listed in Table 3.1. The cases are distinguished by:

- The free–field motion is uniform or non–uniform, including or excluding the vertical component.
- The vertical contraction joints are allowed to open or are constrained closed.
- The joints at the abutment are allowed to open or are constrained closed.
- The horizontal lift joints are allowed to open or are constrained closed.

- The reservoir level is the level at the time of the earthquake (233 ft), the normal pool elevation (300 ft), or an extreme full condition (360 ft).
- The thrust block at the left abutment is included or omitted.

Table 3.1 Cases for Seismic Analysis of Pacoima Dam in the 1994 Northridge Earthquake

Case	Free–Field Motion	Contraction Joints	Abutment Joints	Lift Joints	Reservoir Depth (ft)	Thrust Block
1	uniform–downstream	closed	closed	closed	233	included
2	uniform–2/3 upper left abutment	closed	closed	closed	233	included
3	uniform–2/3 upper left abutment	open	open	closed	233	included
4	non–uniform	closed	closed	closed	233	included
5	non–uniform	open	open	closed	233	included
6	non–uniform, excluding vertical component	open	open	closed	233	included
7	non–uniform	open	open	closed	233	omitted
8	non–uniform	open	open	open	233	omitted
9	non–uniform	open	open	open	300	omitted
10	non–uniform	open	open	open	360	omitted

3.4 Seismic Response for Uniform Free–Field Motion

To assess the effects of the canyon topography on the earthquake response of the dam, Cases 1 and 2 were analyzed with uniform free–field motion at the canyon interface. Case 1 uses the downstream acceleration records; and Case 2 uses the acceleration records from the upper–left abutment, reduced by one–third to represent approximately the average motion of the canyon. The free–field motions for the two cases are very different because of the topographic amplification of the canyon; but they have been proposed as bounds for computing the dam response to the actual non–uniform ground motion (Dowling and Hall, 1989).

Cases 1 and 2 were analyzed with all the joints prevented from opening. This is the typical assumption used in the seismic safety evaluation of arch dams. To assess the effects of joint opening on the dam response, Case 3 was analyzed with the same free–field input as used in Case 2 (2/3 of the upper–left abutment record) but with the contraction and abutment joints allowed to open. The reservoir depth for all three cases was 233 ft, as at the time of earthquake.

The acceleration was computed in the radial direction at the Channel 8 location and also for three directions at the center of the crest. Channel 8 records the radial motion at the left quarter point at about 80% height of the dam. Except for channels along the abutment, this is the only channel on the dam for which a complete processed accelerogram from the Northridge earthquake was available.

The recorded and computed accelerations for Channel 8 are presented in Figure 3.1. The computed accelerations for Cases 1 and 2 differ in phase and amplitude, showing the significant influence of the assumed uniform free–field motion. Case 2 shows a better agreement in phase with the recorded response compared with Case 1. The first large pulse in the recorded response of the dam is represented fairly well by the computed response. However, the Case 2 response has one large amplitude acceleration cycle, which is not present in the recorded motion. The overestimation of the vibration is most likely caused by the lack of radiation damping in the foundation rock model, although the differences could be attributed to possibly incorrect assumptions for the input motion.

When the vertical contraction joints are allowed to open in Case 3, the first large acceleration pulse in the recorded motion is represented better than in Case 2. However, the subsequent computed motion has three large acceleration spikes not present in the recorded motion. The additional spikes compared with Case 2 are most likely caused by impact as the contraction joints close in the model.

Figures 3.2, 3.3 and 3.4 present the computed accelerations in the cross–canyon, stream and vertical directions, respectively, at the crest of crown section for each of the cases. The influence of the assumed uniform free–field motion on the response can be seen again from the comparison of Cases 1 and 2. With joint opening allowed in Case 3, the cross–canyon acceleration increases significantly due to impact of the joints closing, whereas the stream and vertical accelerations are not as affected by joint opening.

For the purpose of comparison, the partially digitized and unprocessed acceleration records at the crest of crown section were shown in Figure 2.6. The strong motion portion of the response could not be digitized and is shown by a zero–acceleration level. Although a direct comparison of the unprocessed accelerograms and the computed accelerations is not possible (Figures 3.2 to 3.4 compared with Figure 2.6), the accelerations are qualitatively similar for the recovered portions of the records. As with the case of Channel 8 in Figure 3.1, however, the computed accelerations overestimate the recorded accelerations because the foundation rock model does not include radiation damping.

Considering the tensile stresses computed for the models of the dam, the envelopes of maximum arch and cantilever tensile stresses for Cases 1, 2 and 3 are shown, respectively, in Figures 3.6, 3.7 and 3.8. The stress contour plots are viewed looking in the downstream direction and tension is positive. Again, the influence of the assumed uniform free–field motion can be seen by comparing the stresses for Case 1 with those for Case 2. The maximum arch and cantilever stresses for Case 2 (Figure 3.7) are about twice as large as the stresses for Case 1 (Figure 3.6).

As seen in Figure 3.8, the opening of the contraction joints in Case 3 significantly reduces the maximum arch stresses of the dam, and increases the cantilever stresses as seismic loads are transferred from arch action to cantilever action. This transfer of load has been observed in previous studies on the effects of contraction joint opening (Fenves, Mojtahedi et al., 1992; Fenves, Mojtahedi et al., 1992).

The envelopes of maximum opening of contraction joints for Case 3 is shown in Fig. 3.16. Joint opening is negligible at the contact of the dam and abutment and increases with the elevation. The analysis for Case 3 shows a maximum joint opening of 2.6 in. at the crest of crown joint. At a given elevation, the crown joint has a larger opening at upstream face than the downstream face because of the upstream bending of the arches. However, the opposite is occurs for the other joints (particularly 1 and 5) because of a reverse curvature of the stiffer monoliths. This is consistent with the distribution of curvature due to upstream displacement of the arches.

3.5 Seismic Response for Non–Uniform Free–Field Motion

The remaining cases use the assumed non–uniform free–field motion described in Section 3.2. To examine how joint opening affects the dam response to non–uniform free–field motion, Case 4 assumes the joints are closed and Case 5 allows the contraction and abutment joints to open.

A comparison of the Channel 8 histories for Cases 2 and 4 in Figure 3.1 indicates that the agreement between the computed and recorded accelerations improves when non–uniform free–field motion is used compared with the response to uniform free–field motion. The first acceleration peak is captured well in Case 4 and the amplitudes of subsequent acceleration spikes are reduced, as compared with Case 2. When the joints are allowed to open in Case 5, the agreement with the recorded channel improves further.

The acceleration histories computed at the crest of crown section for Case 4 (Figures 3.2, 3.3 and 3.4) are quite different compared with the corresponding histories for Case 2, especially for the vertical components. With contraction joint opening allowed for the contraction joints in Case 5, the acceleration at the crest center increases, especially in the cross–canyon and vertical directions. With joints allowed to open, the response to non–uniform free–field motion has smaller acceleration amplitudes compared with the response to uniform free–field motion (compare Cases 3 and 5).

The envelopes of maximum tensile arch and cantilever stresses at the upstream and downstream faces of the dam for Case 4 are shown in Figure 3.9. The contours are very different than those for the uniform free–field motion of Case 2 in Fig. 3.7. For Case 4 the largest arch and cantilever stresses develop near the abutments, in contrast with Case 2 in which the largest stresses occur at the center of the dam. When the spatial variation of free–field motion is accounted for in Case 4, tensile arch stresses exceeding 3000 psi are obtained near the upper right abutment at both faces and the tensile cantilever stresses reach 1400 psi in the lower right portion of the dam. As will be seen in Section 3.8, the large stresses near the abutments are caused by the relative displacements of the non–uniform free–field motion at the interface. The effects of the relative displacements at the interface tend to reduce towards the center of the dam where dam vibration contributes more to the response. From these results, it is apparent that the non-uniform ground motion produces a substantially different stress distribution than the uniform free–field ground motion.

With joints allowed to open in Case 5, the maximum arch stress reduces to 600 psi at the upstream face and 1000 psi at the downstream face, as shown in Figure 3.10. The maximum cantilever stress on the upstream face also reduces due to joint opening at the dam–foundation interface. However, the maximum tensile cantilever stress at the downstream face increases to 1800 psi as seismic forces are transferred from arch action to cantilever action.

The envelopes of maximum joint opening of contraction joints for Case 5 are shown in Figure 3.17. A comparison of Figure 3.16, showing the joint opening envelopes for Case 3 (uniform free–field motion), with Figure 3.17 for Case 5 demonstrates the significant effects of the non–uniform free–field motion on the opening displacements of the contraction joints. For Case 3 the maximum joint opening is less than 1 in. except for the crown joint which opens about 2.6 in. near the crest. When the non–uniform free–field motion is used in Case 5, openings of 4, 2.4, 3.4, 1.6 and 2.4 in. occur at the crest of Joints 1, 2, 3, 4, and 5, respectively. Also, while no joint opening occurs at dam–foundation interface in the case of uniform free–field motion, significant opening occurs along the interface in the case of non–uniform input. The larger joint openings are caused by the relative displacements along the dam–canyon interface. The effects of non–uniform free–field motion on the joint opening are discussed further in Section 3.8.

3.6 Effect of Vertical Component of Free–Field Motion

The effects of the vertical component of the free–field motion on the stresses in dam can be seen from a comparison of the stress contours for Case 5 in Fig. 3.10 with those shown in Fig. 3.11 for Case 6. With the vertical component disregarded in Case 6, the maximum tensile cantilever stress at the downstream face reduces to 1000 psi from 1800 psi, indicating the importance of the vertical component of the ground motion on the cantilever stresses. Arch stresses are less affected by the vertical ground motion component, although neglecting the vertical component reduces the maximum arch tensile stress from 1000 psi to 600 psi.

3.7 Effect of Thrust Block

In view of the permanent opening of the interface between dam and its thrust block after the Northridge earthquake, the thrust block did not provide effective restraint for the entire duration of earthquake. To neglect the thrust block in the analysis, small stiffness values were assigned for the joint elements that represent its interface with the dam. A comparison of the Channel 8 histories for Cases 5 and Case 7 (Figure 3.1) indicates that removing the thrust block from the model does not affect the comparison between the recorded and computed accelerations. This is because any role of the thrust block in providing arch support is limited to elevations higher than the Channel 8 location.

From the comparison of the crest acceleration histories of Cases 5 and 7 (Figures 3.2 to 3.4), the motion at the crest is not affected by removal of the thrust block from the model, except for slight increase in the cross–canyon direction. The maximum stresses for Case 7, shown in Fig. 3.12, are essentially identical to those for Case 5, demonstrating the small influence of the thrust block on the earthquake response.

The comparison of joint opening envelopes of Cases 5 and 7 in Figures 3.17 and 3.18 indicates that removal of the thrust block from the model has a minor effect on opening of the contraction joints, except for the separation of the dam from the thrust block. The magnitude of the separation increases from 2.4 in. for Case 5 to about 4 in. for Case 7, which disregards the thrust block.

3.8 Effect of Horizontal Joint Opening

The seismic response of Pacoima dam computed with the assumed non–uniform free–field motion indicates large tensile cantilever stresses, which exceed tensile strength of concrete and lift joints. To recognize the limited tensile capacity of the lift joints, horizontal joints are included in the model for Case 8.

From comparison of accelerations in Figure 3.1 for Cases 7 and Case 8, both with the thrust block omitted from the model, it can be seen that opening of horizontal joints does not affect the Channel 8 acceleration significantly. This is a reasonable result considering that Channel 8 is 90 ft above the nearest horizontal joint in the model and, as described later, the opening displacements at the lift joints are small. The opening of horizontal lift joints also has little influence on the crest acceleration for the same reason.

From the cases considered in this study, Case 8 represents the most realistic model for the response of Pacoima dam to the Northridge earthquake. The acceleration computed for Channel 8 in Case 8 and the corresponding acceleration record has the same general character, except for the acceleration spikes which are attributed to the lack of radiation damping in the model. The strong motion pulses within the first 4.5 sec of the response history are represented very well by the Case 8 model. The agreement between the computed and unprocessed recorded accelerations at the crest of crown appears to be reasonable in the terms of the frequency content and amplitude of vibrations after the main (and unprocessed) pulses.

The opening of horizontal joints in Case 8 simulates the release of cantilever tensile stresses at the lift joints. The effects of horizontal joint opening on stresses in the dam can be seen from comparison of the stress contours for Cases 7 and 8 in Figures 3.12 and 3.13. The arch stresses are not affected by opening of the horizontal joints, as expected. Neither are the cantilever stresses at upstream face affected because of the small magnitude of these stresses. The cantilever stresses at the downstream face, however, are significantly reduced due to opening of the horizontal joints. The maximum stress reduces to about 1000 psi from 1800 psi.

A comparison of joint opening displacement envelopes for Cases 7 and 8 in Figures 3.18 and 3.19 indicates that opening of the horizontal joints has a minor effect on the opening of the vertical contraction joints. The histories of joint opening displacement at the upstream and downstream faces for Case 8 are shown in Figure 3.22 at the crest of the vertical contraction joints and at three points of each horizontal joint. The opening displacements of the horizontal joints (less than 0.50 in.) are much less than the opening of the contraction joints. Furthermore, the upstream and downstream opening of the horizontal joints is generally out–of–phase indicating that the cantilevers rock at the lift surface. Complete separation through the thickness of the dam is indicated for some of the horizontal joints. The complete separation is limited in extent, however, and considering the small opening displacements at the horizontal joints, it is not considered a mechanism for the failure of the dam.

To examine further the effect of non–uniform free–field motion on the joint opening, the histories of joint opening for Case 8 are shown in Figure 3.23. In that figure, the solid lines correspond to the total dynamic response, and the dashed lines show the pseudo–static component of joint opening: the opening that would occur due to the slow application of the free–field ground motion. Upstream joint opening is shown for the vertical joints and downstream opening is shown for the horizontal joints. Joint 1, near the right abutment, has the largest opening of 4 in., but nearly all of that opening is associated with the pseudo–static response. The vibration produces less than 1 in. of the opening. The opening caused by vibration is more significant for Joints 2 and 3, with maximum values of about 1.5 in. and 3 in., respectively, during the strong vibration response. Negligible opening, less than 0.5 in., is seen in Joint 4. The opening shown for Joint 5 is fictitious because it is associated with small joint stiffness that was used to simulate the removal of the thrust block from the model.

As shown in Figure 3.23, the opening of the horizontal joints is nearly all caused by the pseudo–static response to the non–uniform free–field motion. The only significant dynamic opening is at 4 sec at the right end of joint 6 and between 5 and 6 sec for the center of joint 6.

3.9 Effect of Reservoir Level

The reservoir depth for Case 8 is 233 ft, which was the depth at the time of the 1994 Northridge earthquake. Two additional cases (9 and 10) are identical to Case 8 except for the reservoir level. For Case 9 the reservoir level is assumed at the crest of spillway, 300 ft above the base, the level normally assumed for a seismic safety evaluation a dam. For Case 10 the reservoir level is increased to 360 ft (98% full with 5 ft freeboard) to examine the effects of dam–water interaction for the unlikely occurrence of simultaneous flood and earthquake.

From Figures 3.2 to 3.4, the computed crown crest accelerations for Case 9 are similar to those obtained in Case 8. In contrast, the accelerations for Case 10 are significantly larger, approximately 7.5 g. Although perhaps unrealistic, the case is rather extreme with the reservoir filled to the crest producing large hydrodynamic mass and hence acceleration.

The stress contours obtained for Case 9 in Figure 3.14 are similar to the contours for Case 8, except for slightly larger cantilever stresses at the downstream face away from the horizontal joints of the model. With water surface level increased to 360 ft in Case 10, the cantilever stresses of the downstream face near the center of dam increases to about 600 psi compared with 200 psi in Case 9. From comparison of the joint opening envelopes for Cases 8 and 9 in Figures 3.19 and 3.20, the opening is not affected significantly when reservoir level is increased from 233 ft to 300 ft. However, as seen in Figure 3.21 significant reduction of contraction joint opening (to 2.2 in. from 4.0 in at the crown) occurs when the reservoir level is increased to 360 ft because the hydrostatic forces tend to close the joints.

The opening histories of the joints for Cases 9 and 10 are shown in Figures 3.24 and 3.25, respectively. Opening displacements are shown for upstream and downstream faces to give the phase relationship through the dam thickness. The maximum opening of the vertical joints for Case 10 is still 4 in, but the maximum horizontal joint opening increases to 1.2 in for a brief duration of time at the center of Joint 6. The upstream and downstream openings of the contraction joints are in–phase as in the previous cases. In–phase upstream and downstream opening also occurs for portions of the horizontal joints, particularly at the center of Joint 6 in Case 10. This indicates complete separation near the center of the horizontal joint when the reservoir depth is full at 360 ft elevation.

3.10 Concrete Compressive Stresses

The compressive stresses in an arch dam are generally not as important as the tensile stresses for seismic safety evaluation. The trends in compressive stress are briefly described in this section. With the joints assumed closed and the ground motion as two-thirds of the upper left abutment record (Case 2), the maximum compressive arch stress is 700 psi at the upstream face and 600 psi at the downstream face. The maximum cantilever stresses at the upstream and downstream faces are 500 psi and 400 psi, respectively. Joint opening produces an increase in the maximum arch stresses at both faces (1000 psi upstream and 800 psi downstream) as the arch loads transfer to the cantilevers. The maximum cantilever stresses also increase to 800 psi upstream and 500 psi downstream.

Comparison of compressive stresses for Cases 2 and 4 show that the spatial variation of ground motion has a significant effect. When non-uniform free-field motion is considered the maximum arch compressive stress increases to 2000 psi at the upstream face and 1600 psi at the downstream face. The maximum cantilever stress increases to 1600 psi and 1000 psi at the upstream and downstream faces, respectively. The non-uniform ground motion also affects the distribution of compressive stresses. For example, the maximum upstream arch stress for Case 2 is near the center of the crest, whereas the largest compressive stresses for Case 4 are near the ends of the crest. Considering joint opening with non-uniform ground motion, the joint opening relieves the large compression stresses near the abutments. When joints are allowed to open (Case 8), the maximum arch compressive stress reduces to 1600 psi at the upstream face and 1400 psi at the downstream face. The maximum compressive cantilever stress reduces to 800 psi at the downstream face and is unchanged at the upstream face.

The large hydrodynamic loads for Case 10, with a completely full reservoir, increases the compressive stresses. For Case 10, the maximum compressive arch stress is 2000 psi upstream and 1600 psi downstream. The distribution of compressive stresses for Cases 8 and 10 are similar.

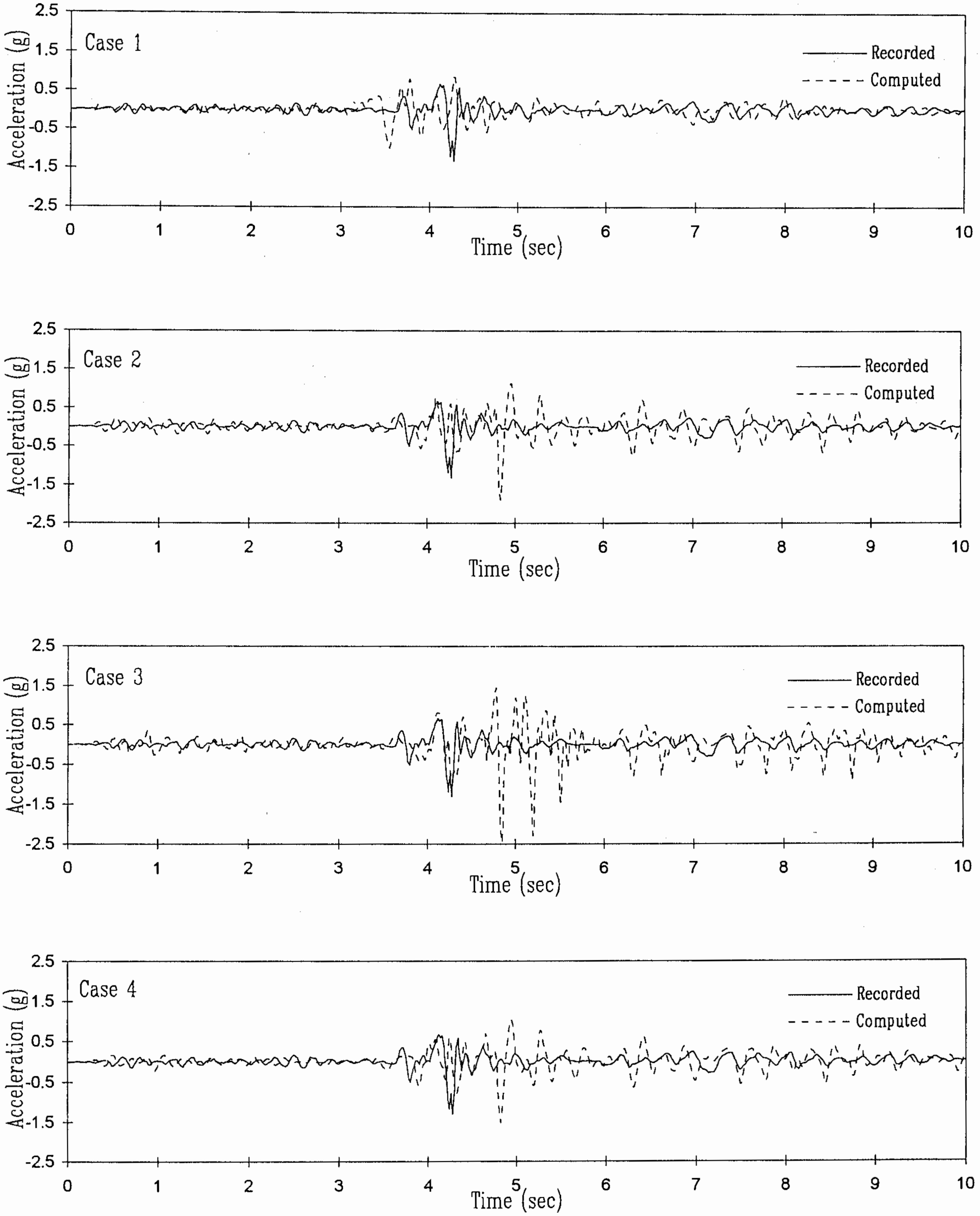


Figure 3.1 Acceleration of Model Computed for Channel 8 Compared with Recorded Acceleration. See Table 3.1 for Definition of Cases.

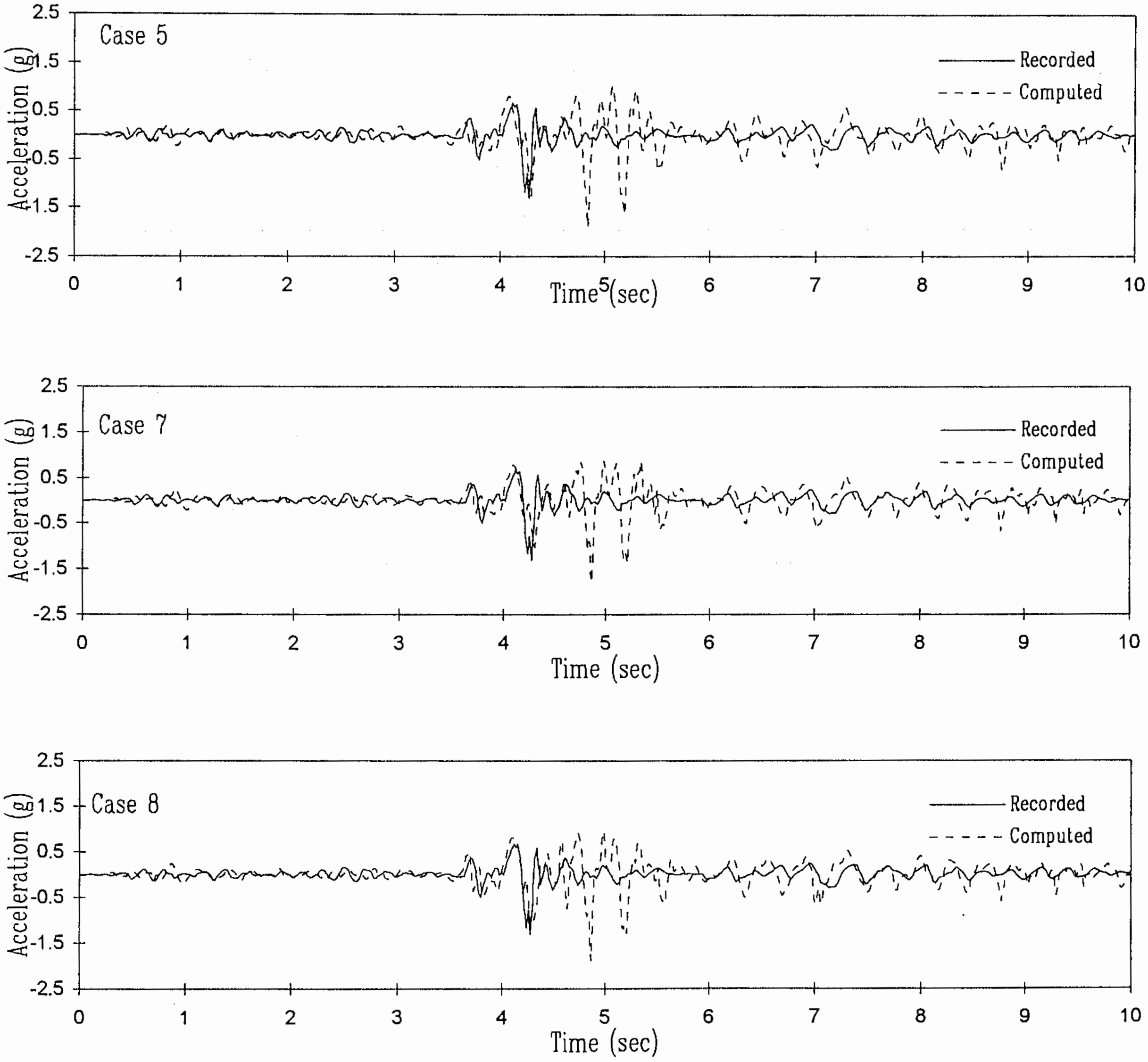


Figure 3.1 Acceleration of Model Computed for Channel 8 Compared with Recorded Acceleration. See Table 3.1 for Definition of Cases (cont'd).

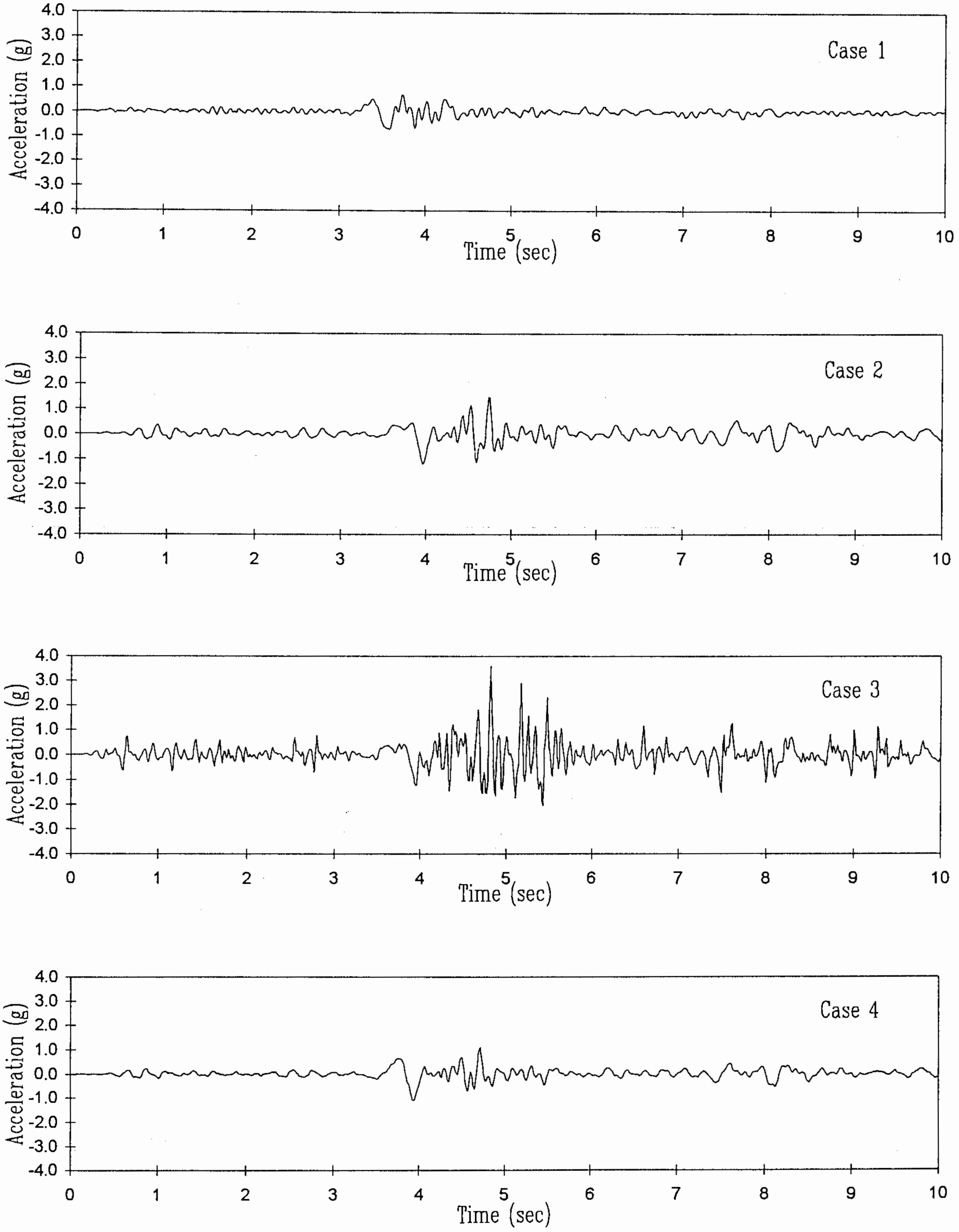


Figure 3.2 Acceleration Computed at Crest of Crown Section in Cross-Canyon Direction. See Table 3.1 for Definition of Cases.

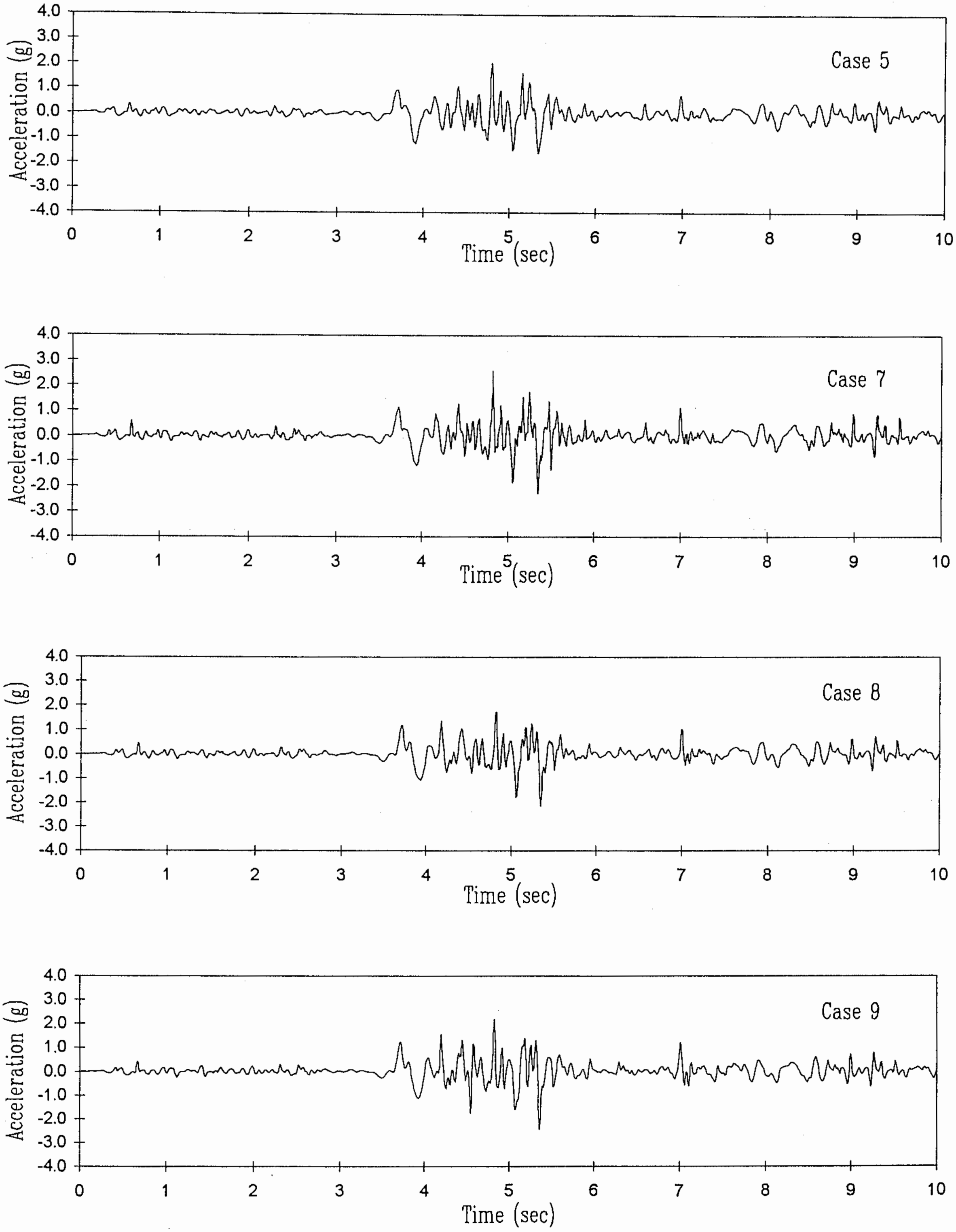


Figure 3.2 Acceleration Computed at Crest of Crown Section in Cross-Canyon Direction. See Table 3.1 for Definition of Cases. (cont'd)

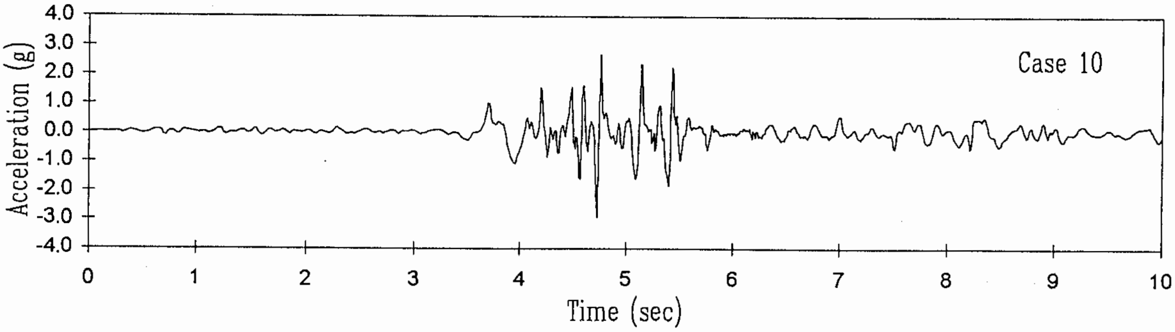


Figure 3.2 Acceleration Computed at Crest of Crown Section in Cross-Canyon Direction. See Table 3.1 for Definition of Cases. (cont'd)

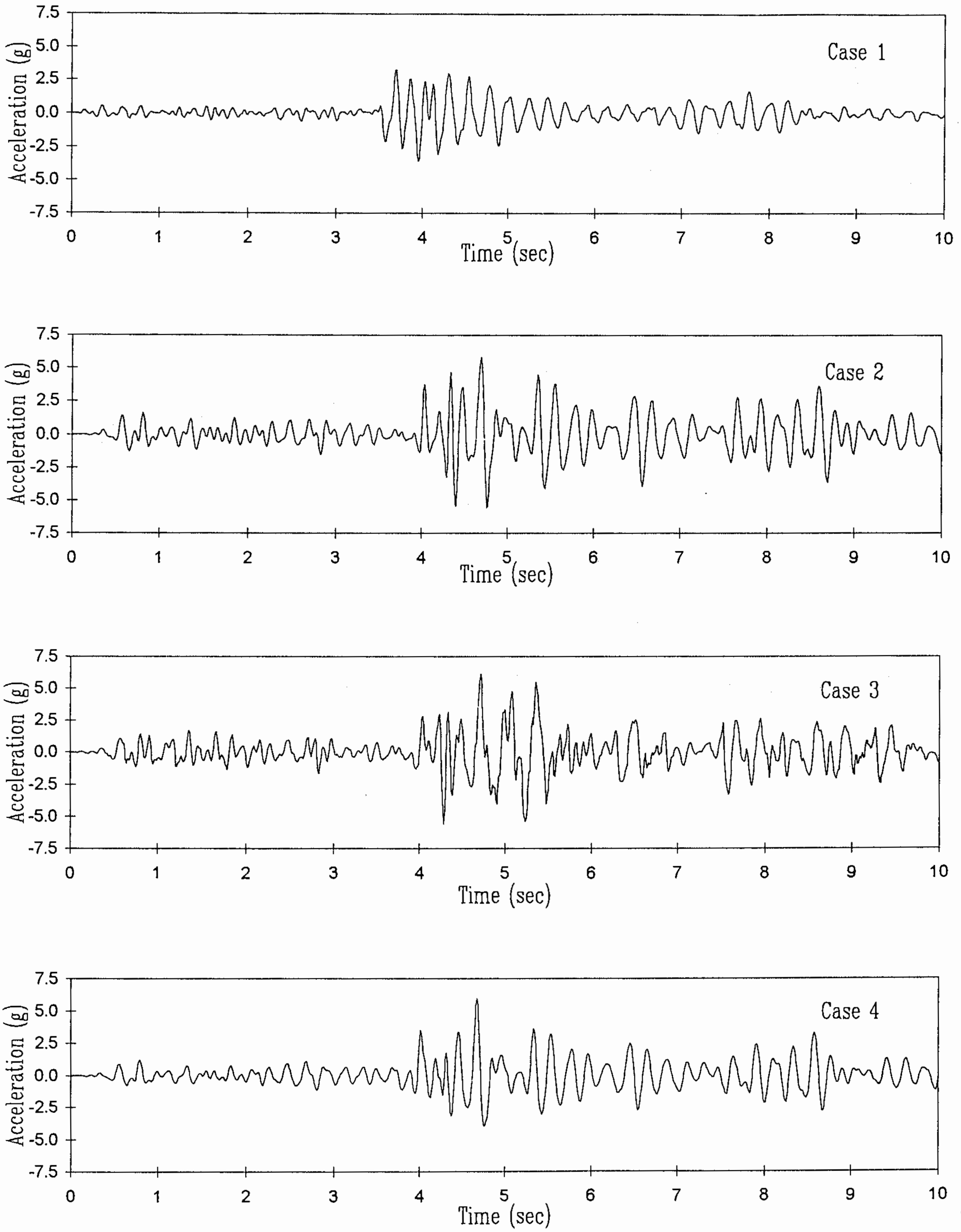


Figure 3.3 Acceleration Computed at Crest of Crown Section in Stream Direction. See Table 3.1 for Definition of Cases.

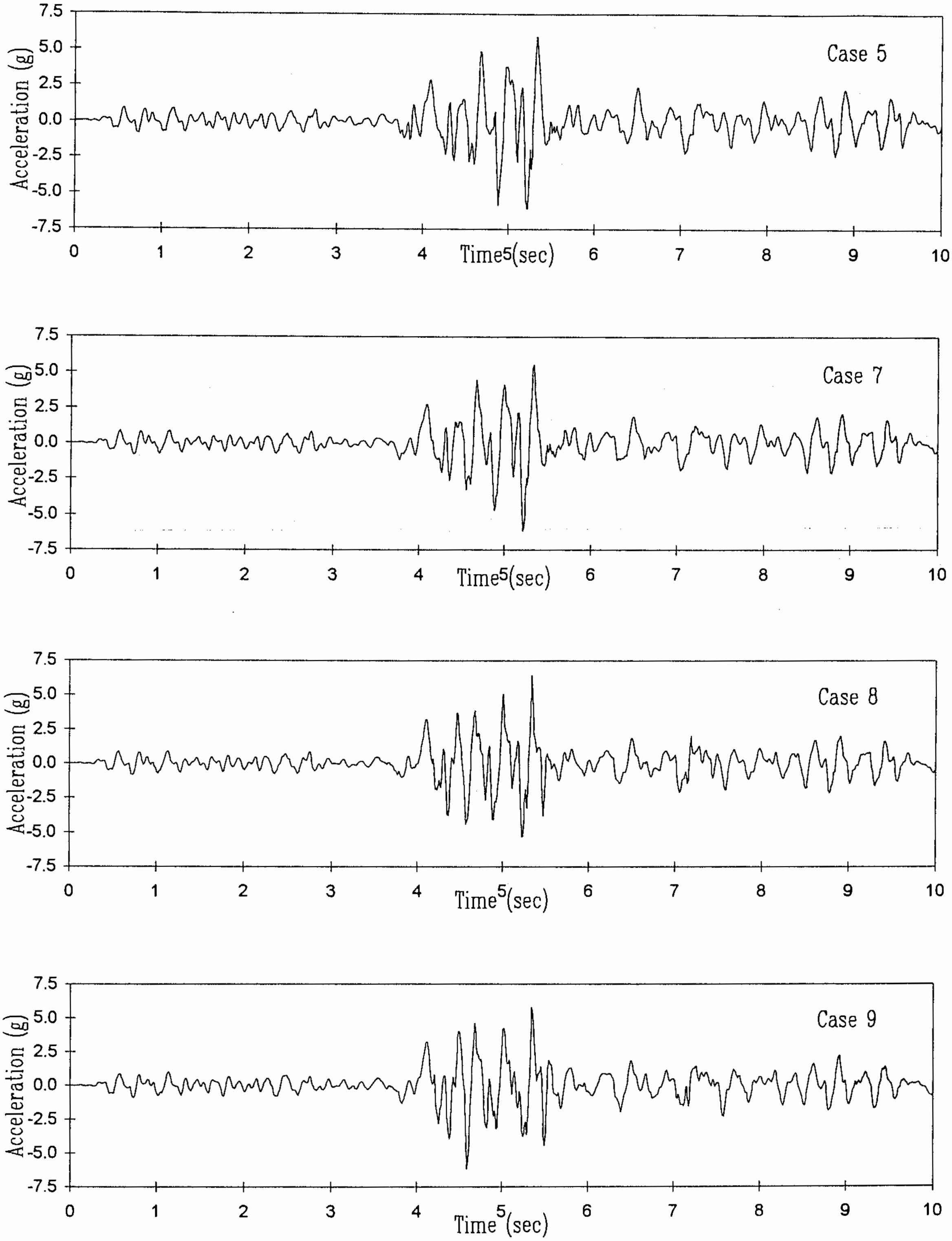


Figure 3.3 Acceleration Computed at Crest of Crown Section in Stream Direction. See Table 3.1 for Definition of Cases. (cont'd)

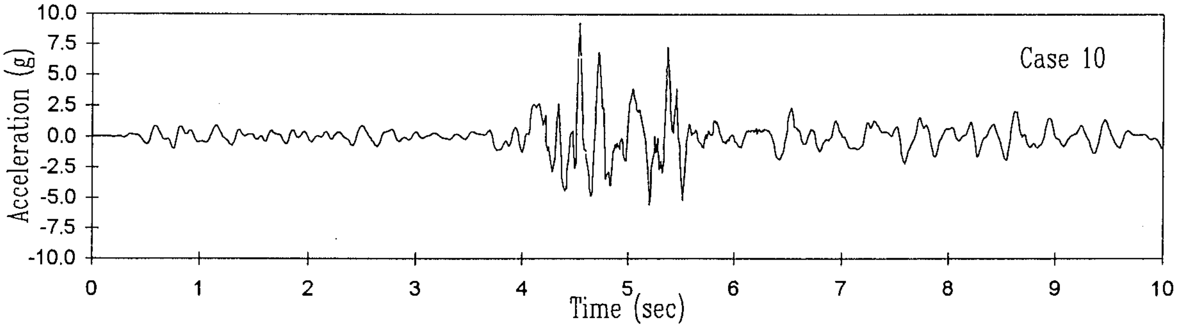


Figure 3.3 Acceleration Computed at Crest of Crown Section in Stream Direction, See Table 3.1 for Definition of Cases. (cont'd)

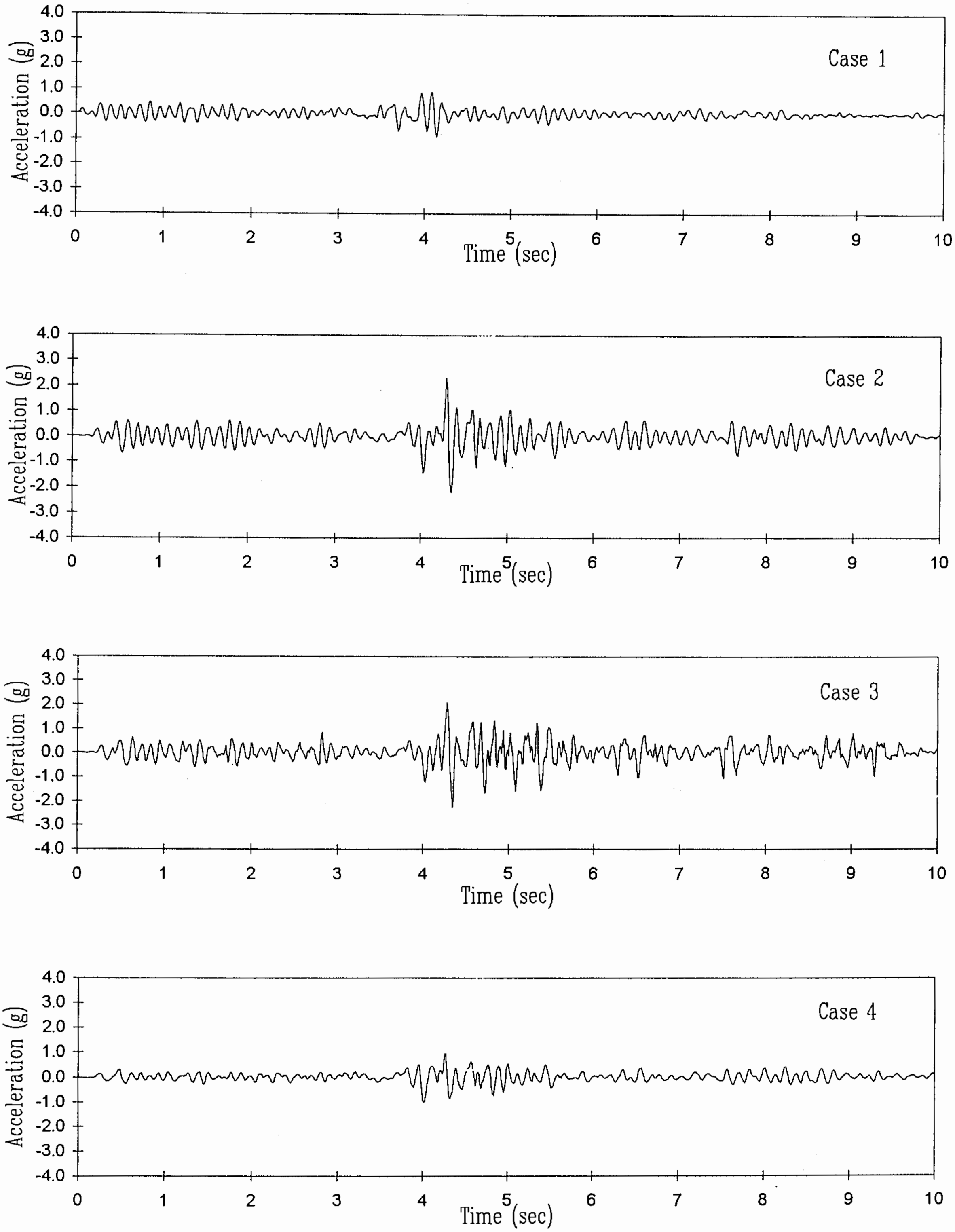


Figure 3.4 Acceleration Computed at Crest of Crown Section in Vertical Direction. See Table 3.1 for Definition of Cases.

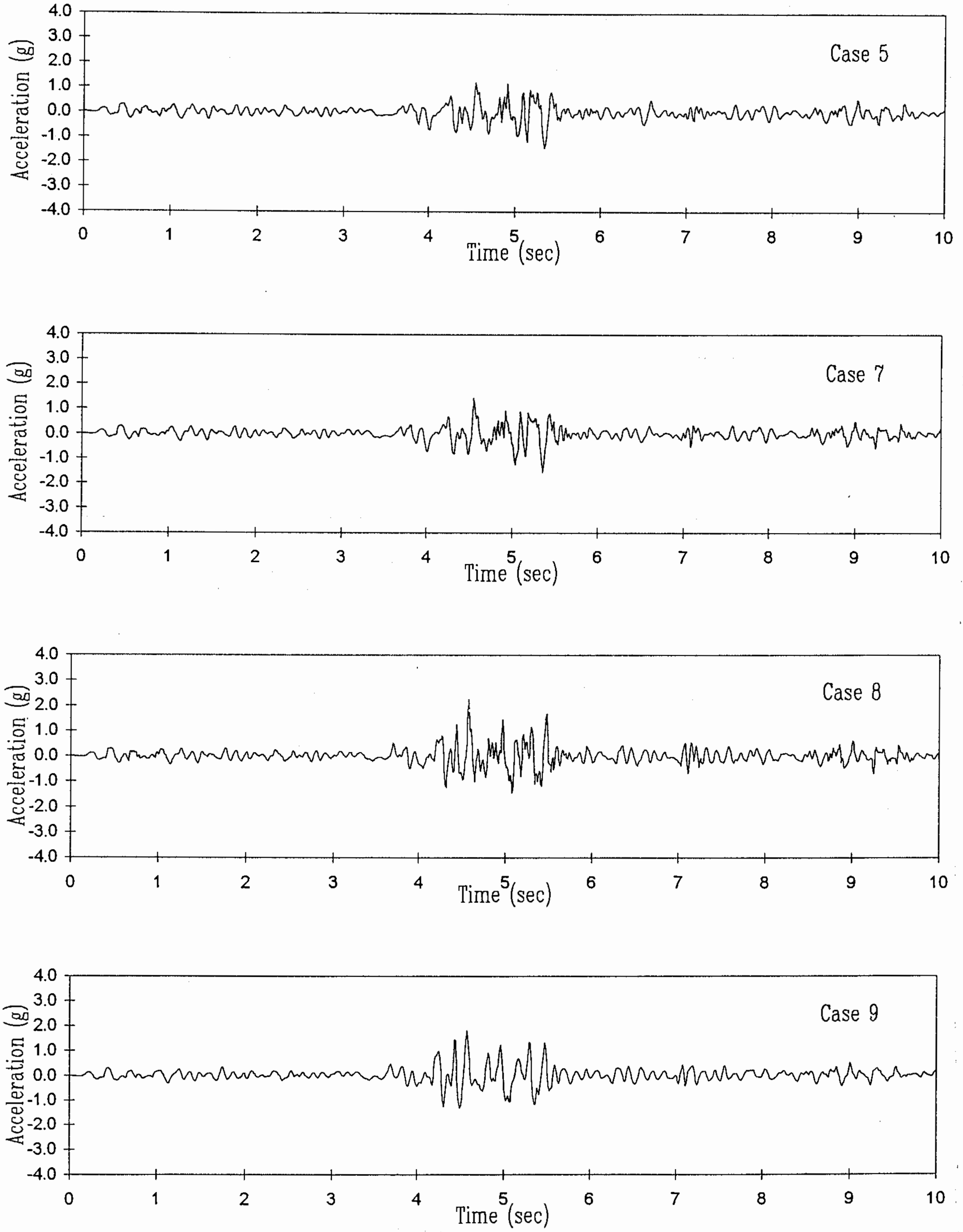


Figure 3.4 Acceleration Computed at Crest of Crown Section in Vertical Direction. See Table 3.1 for Definition of Cases. (cont'd)

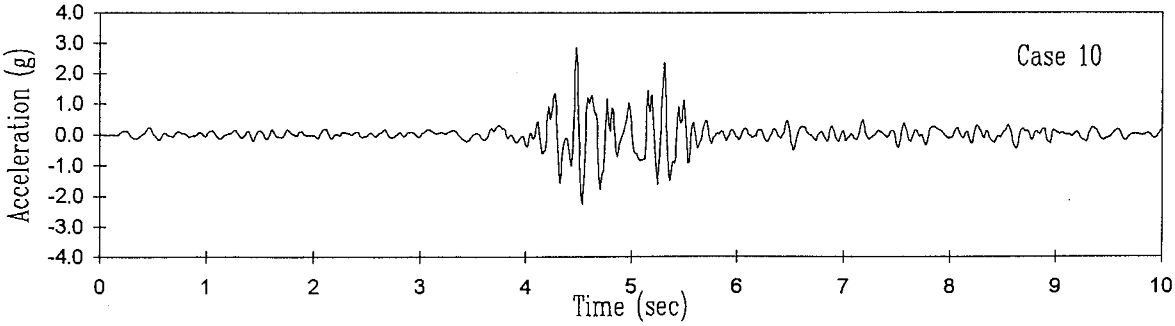


Figure 3.4 Acceleration Computed at Crest of Crown Section in Vertical Direction. See Table 3.1 for Definition of Cases. (cont'd)

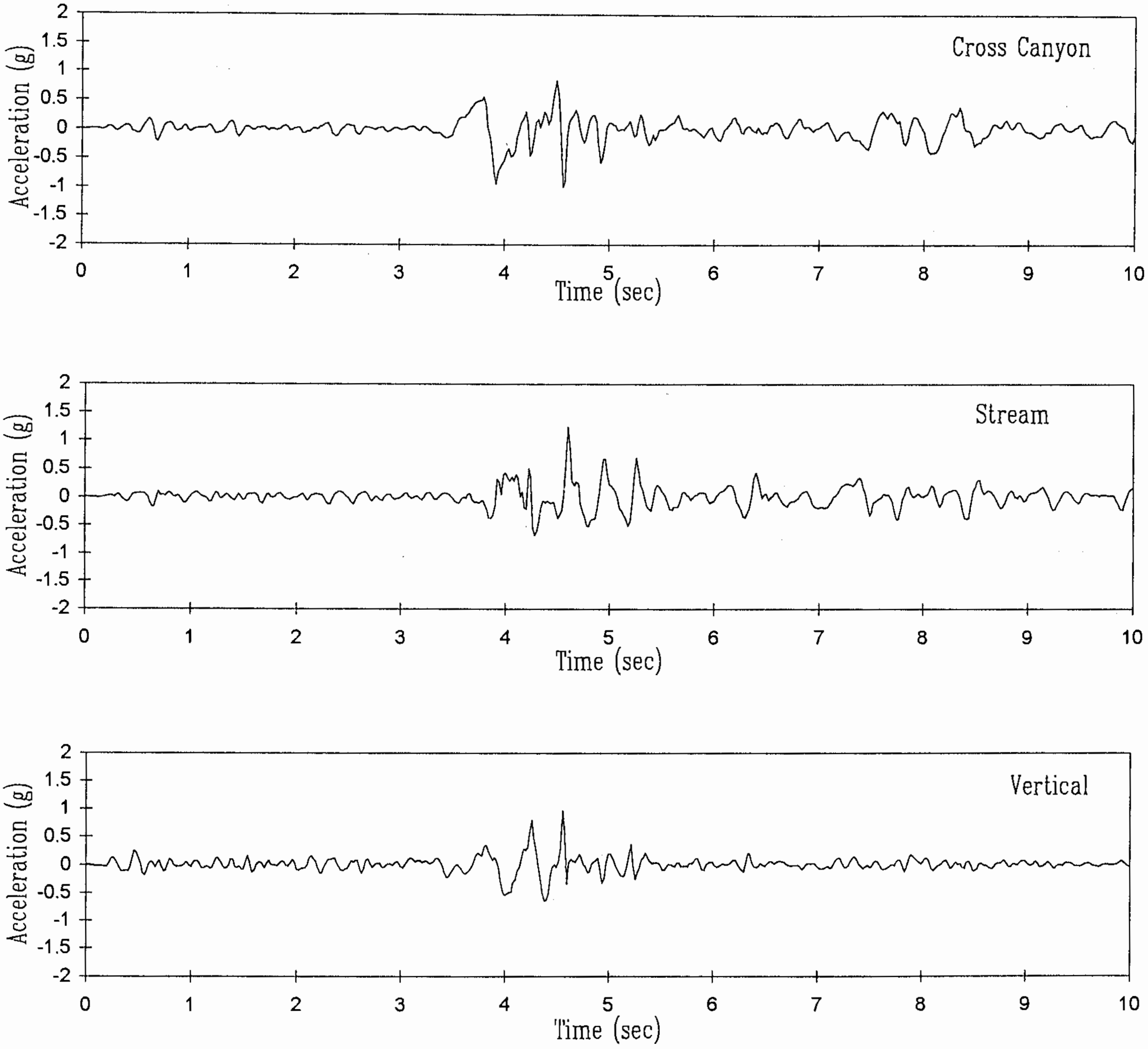


Figure 3.5 Assumed Free-field Acceleration for 280 ft Height Interpolated from Base and Crest Recorded Motions

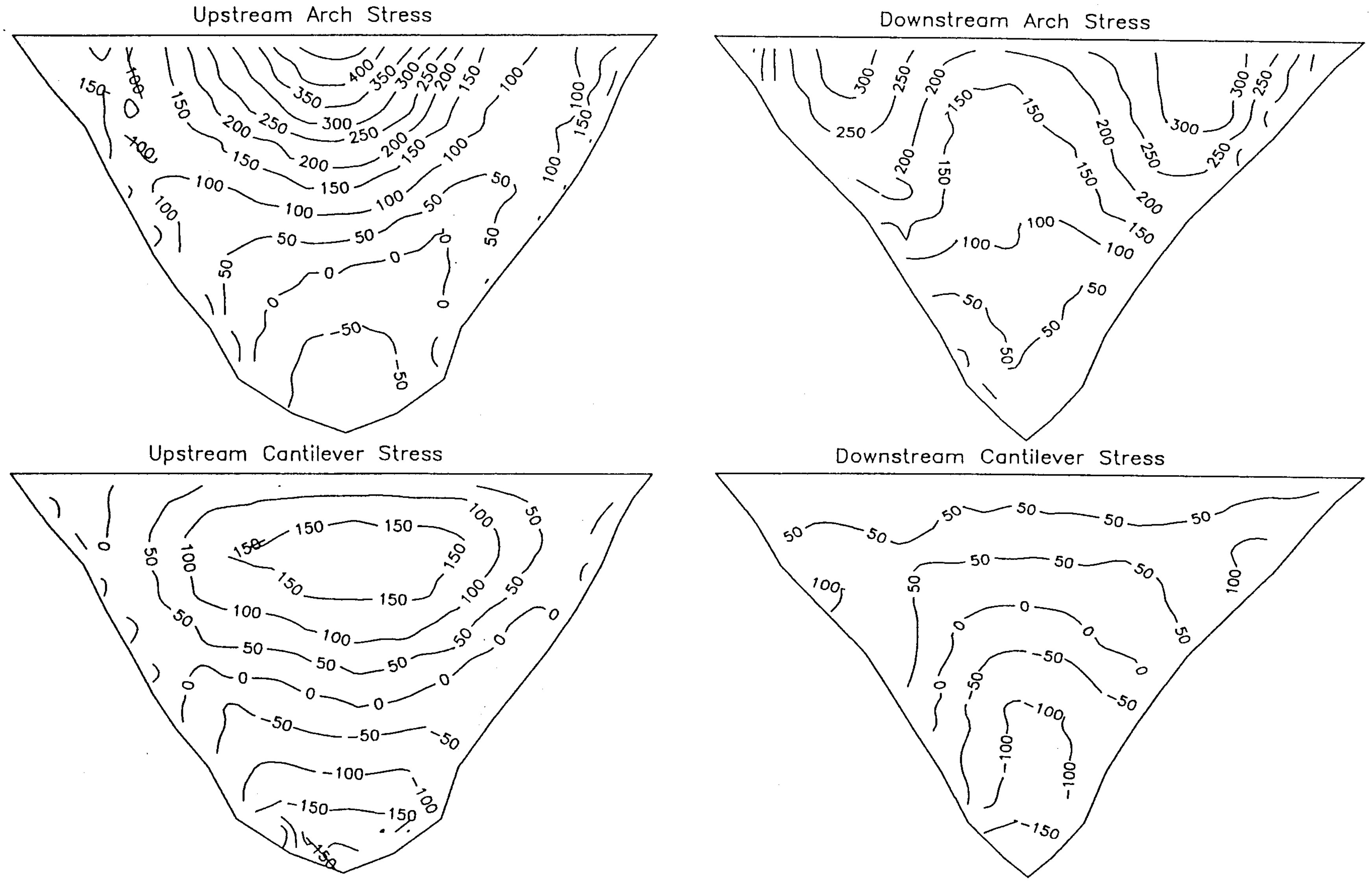


Figure 3.6 Envelopes of Maximum Stress (in psi) for Case 1: Uniform Free-field Motion with Downstream Record, Closed Joints, 233 ft Reservoir.

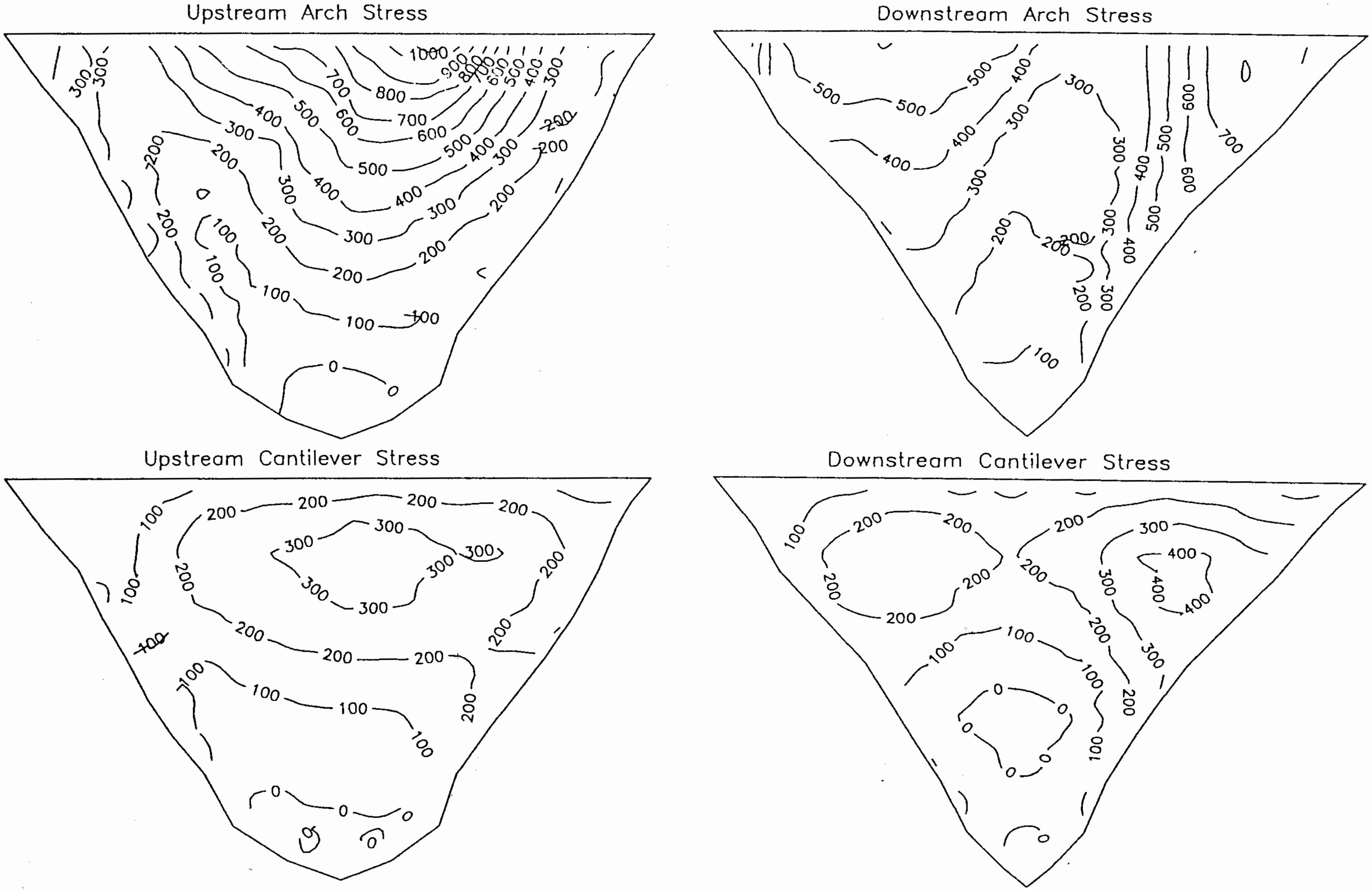


Figure 3.7 Envelopes of Maximum Stress (in psi) for Case 2: Uniform Free-field Motion with 2/3-Upper Left Abutment Record, Closed Joints, 233 ft Reservoir.

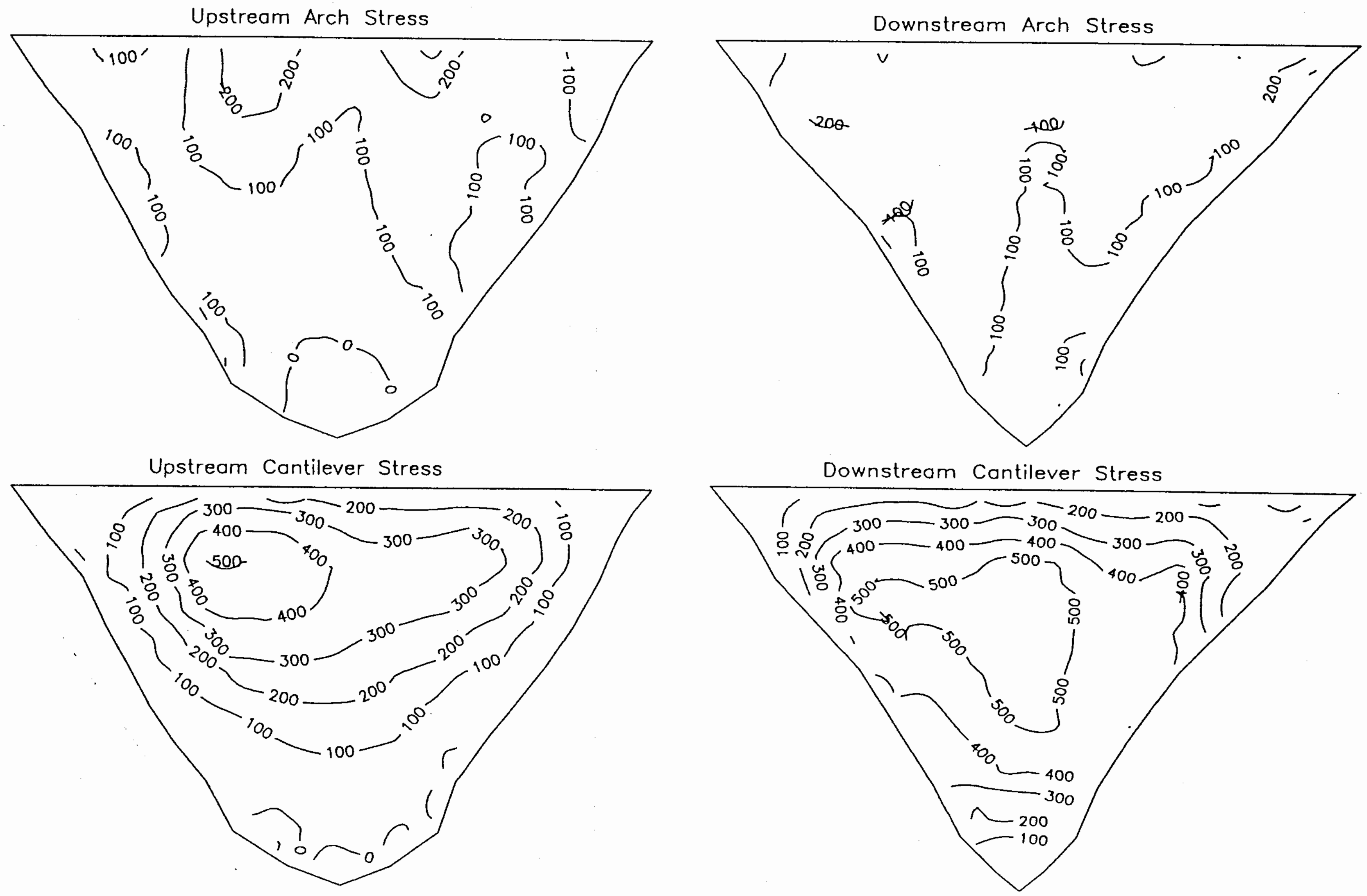
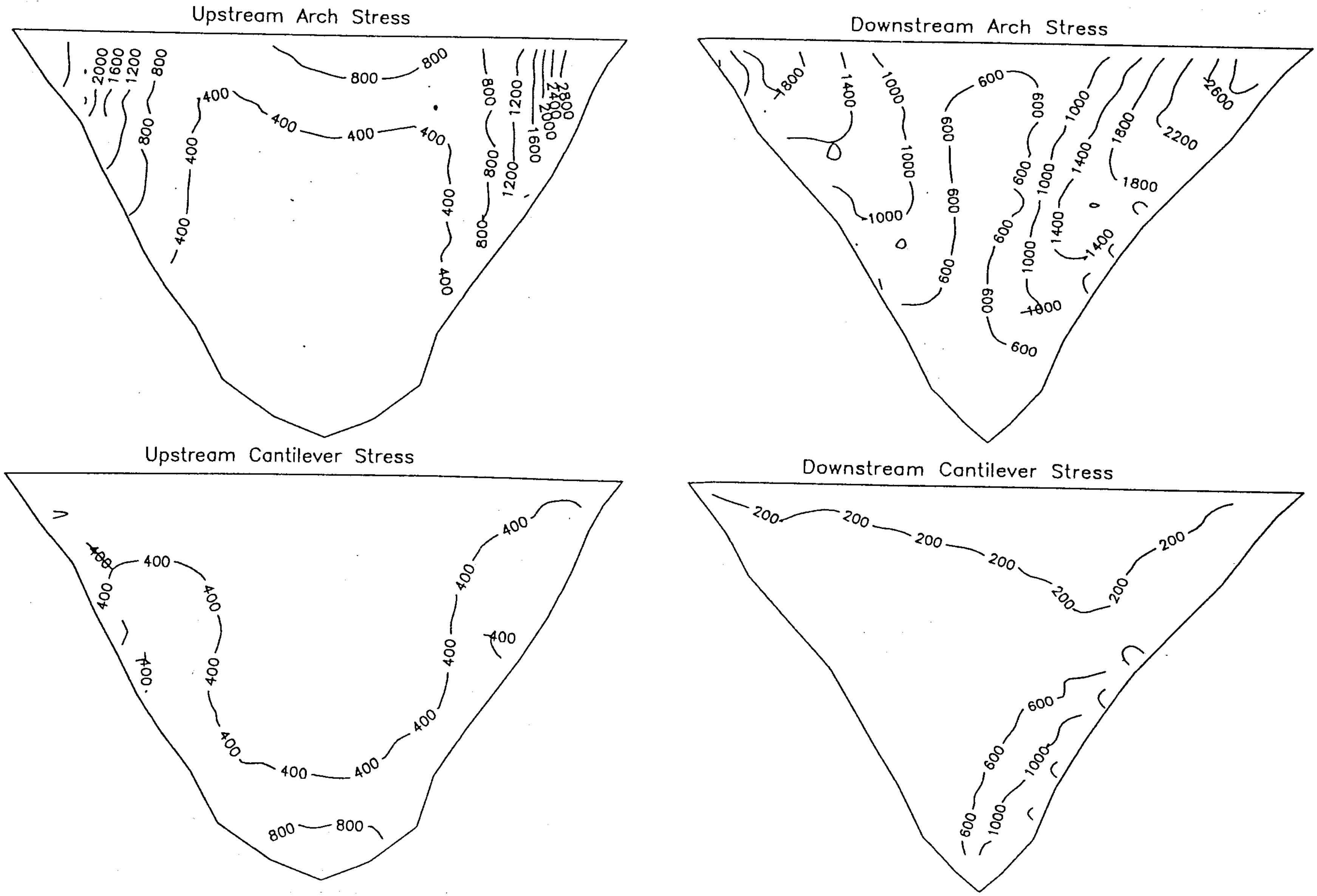


Figure 3.8 Envelopes of Maximum Stress (in psi) for Case 3: Uniform Free-field Motion with 2/3-Upper Left Abutment Record, Vertical Joints Open, 233 ft Reservoir.


Upstream Arch Stress

Downstream Arch Stress

Upstream Cantilever Stress

Downstream Cantilever Stress

Figure 3.9 **Envelopes of Maximum Stress (in psi) for Case 4: Non-uniform Free-field Motion, Joints Closed, 233 ft Reservoir.**

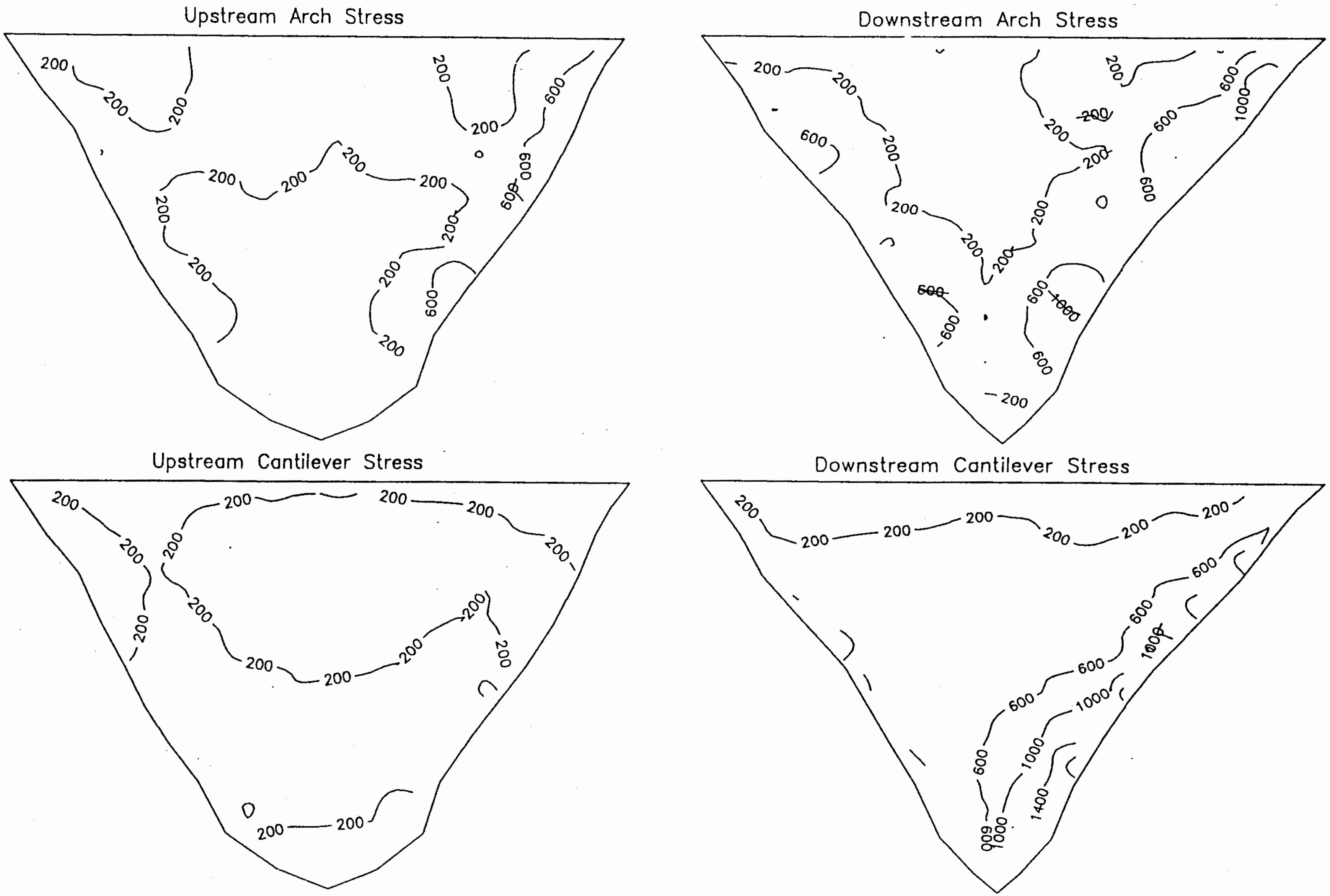


Figure 3.10 **Envelopes of Maximum Stress (in psi) for Case 5: Non-uniform Free-field Motion, Vertical Joints Open, 233 ft Reservoir.**

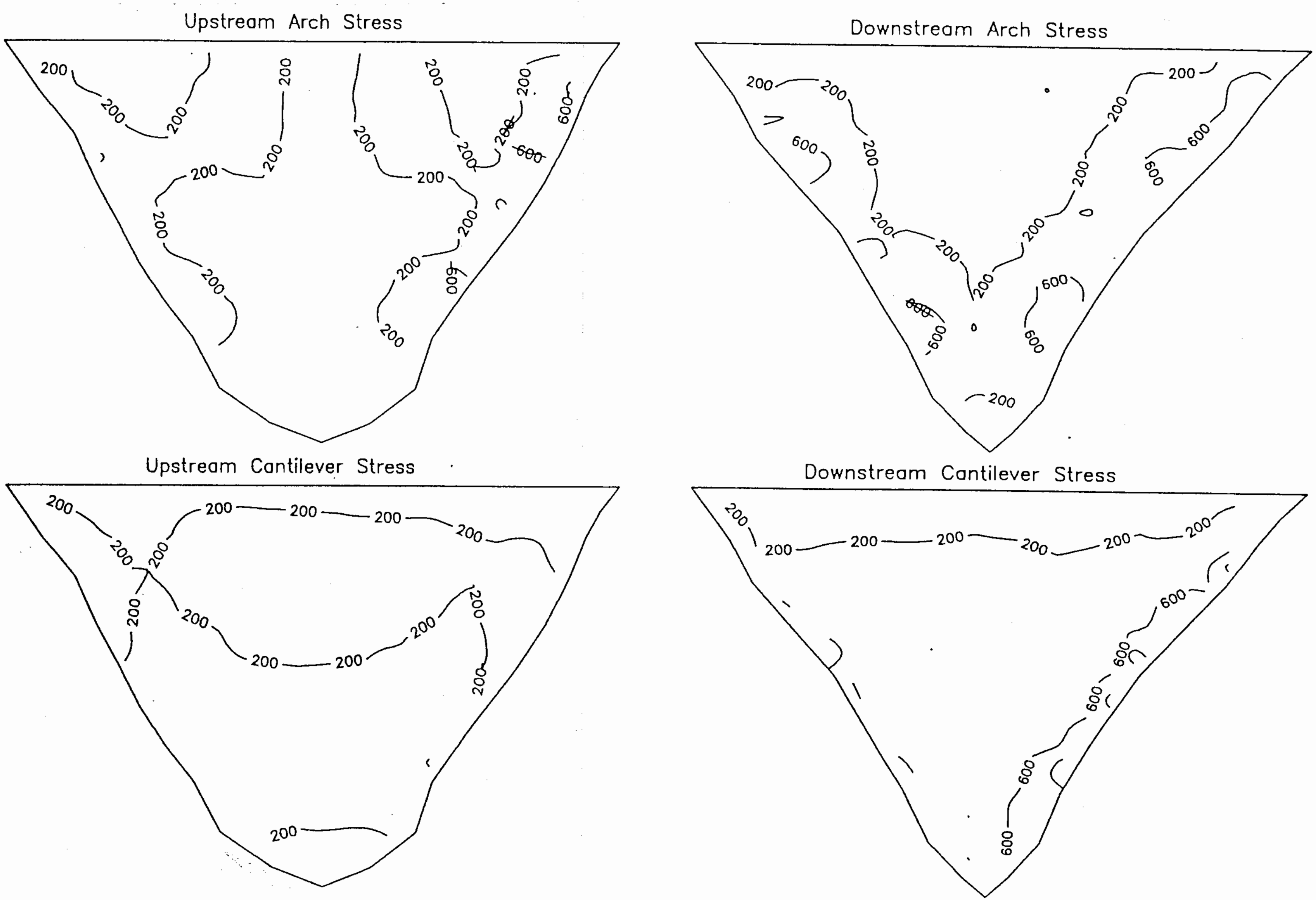


Figure 3.11 Envelopes of Maximum Stress (in psi) for Case 6: Non-uniform Free-field Motion without Vertical Component, Vertical Joints Open, 233 ft Reservoir.

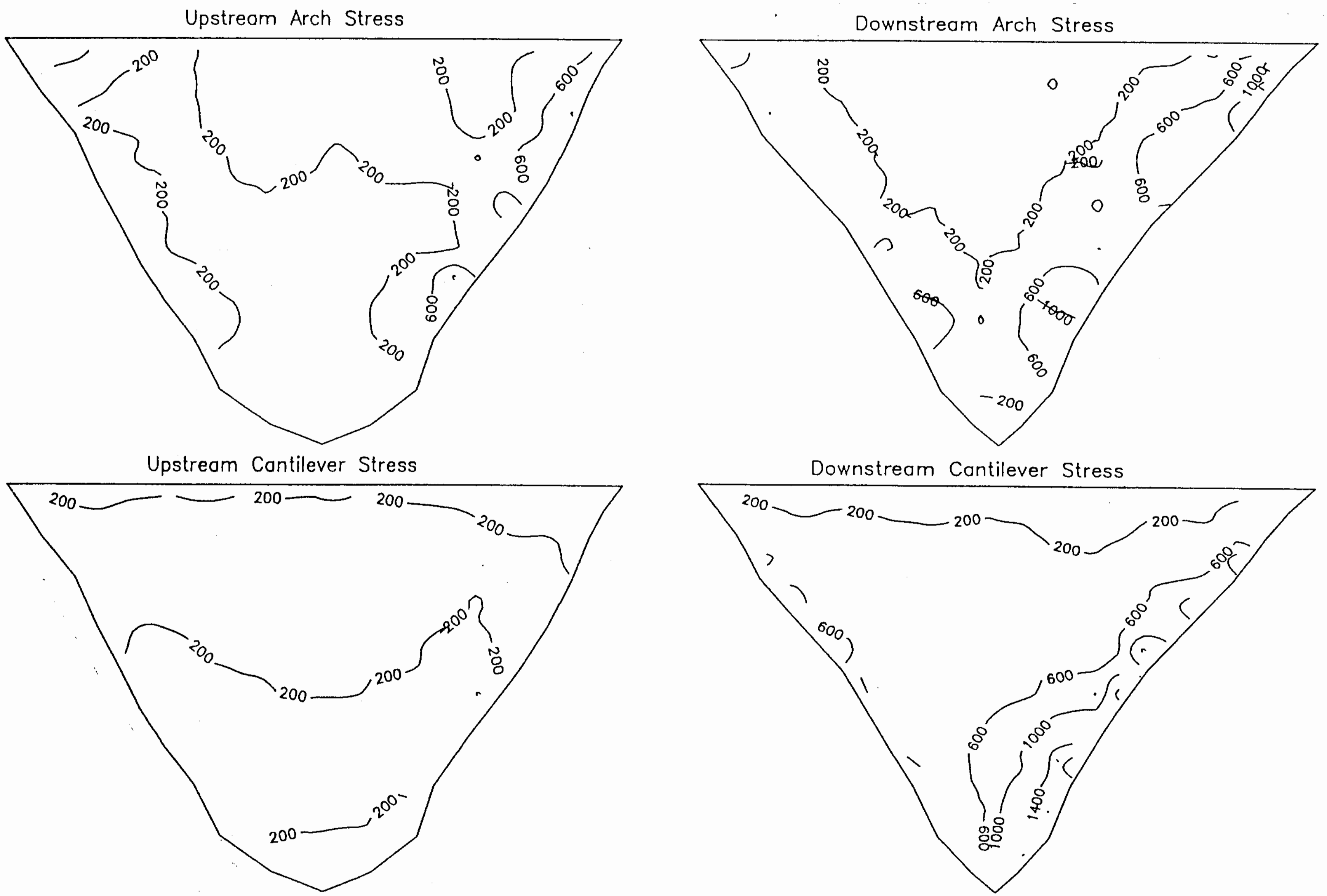


Figure 3.12 Envelopes of Maximum Stress (in psi) for Case 7: Non-uniform Free-field Motion, Vertical Joints Open, Thrust Block Omitted, 233 ft Reservoir.

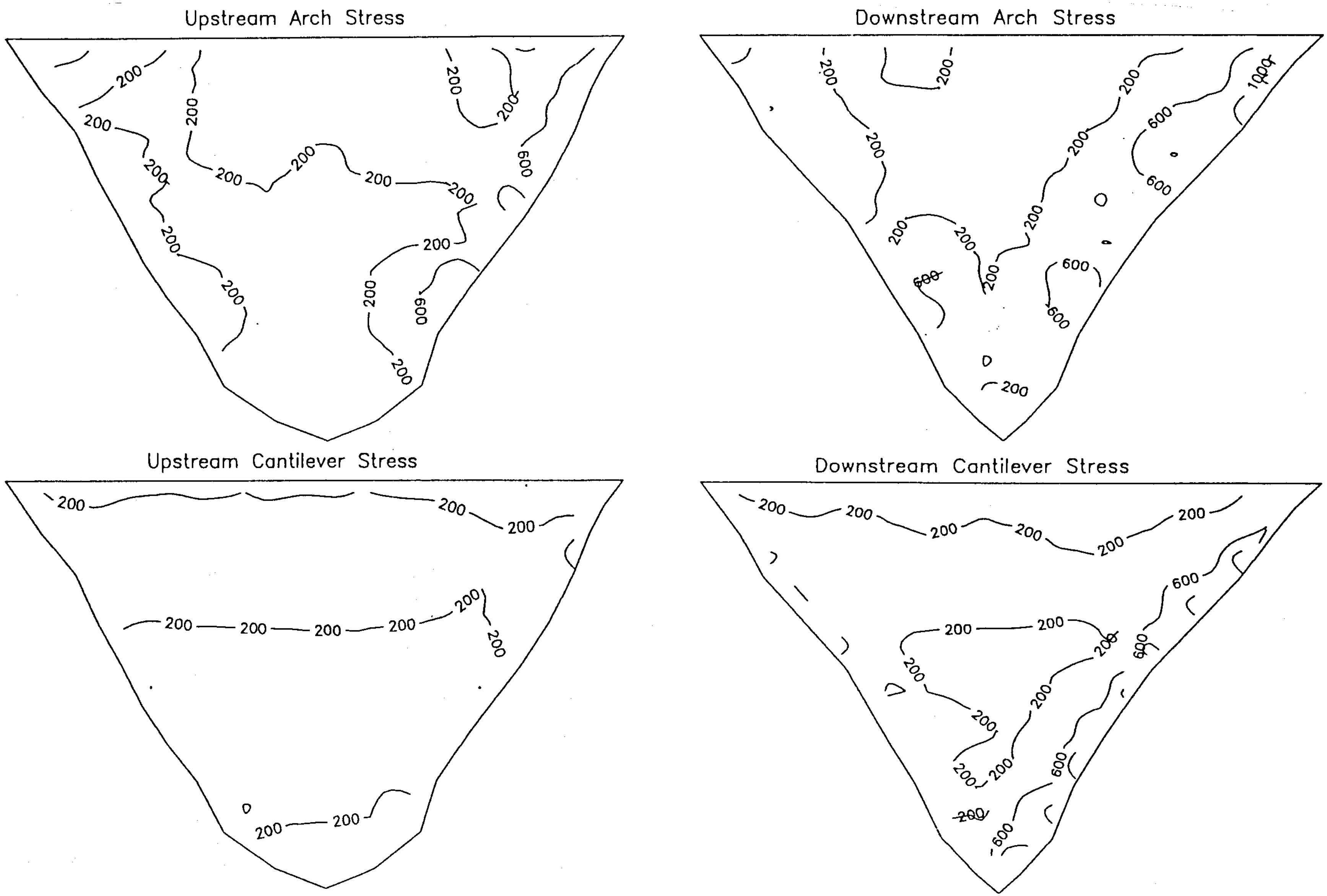


Figure 3.13 Envelopes of Maximum Stress (in psi) for Case 8: Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 233 ft Reservoir.

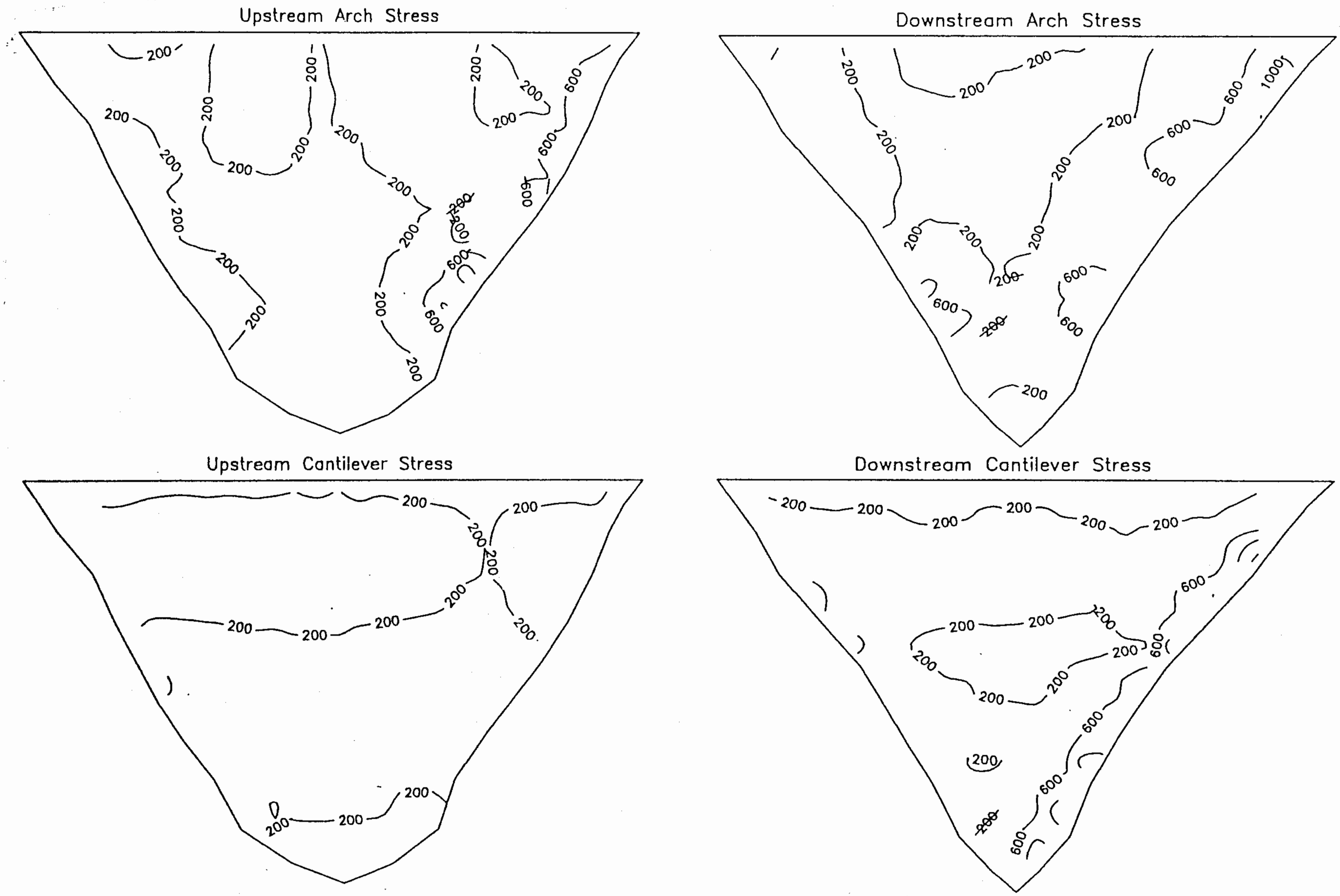


Figure 3.14 Envelopes of Maximum Stress (in psi) for Case 9: Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 300 ft Reservoir.

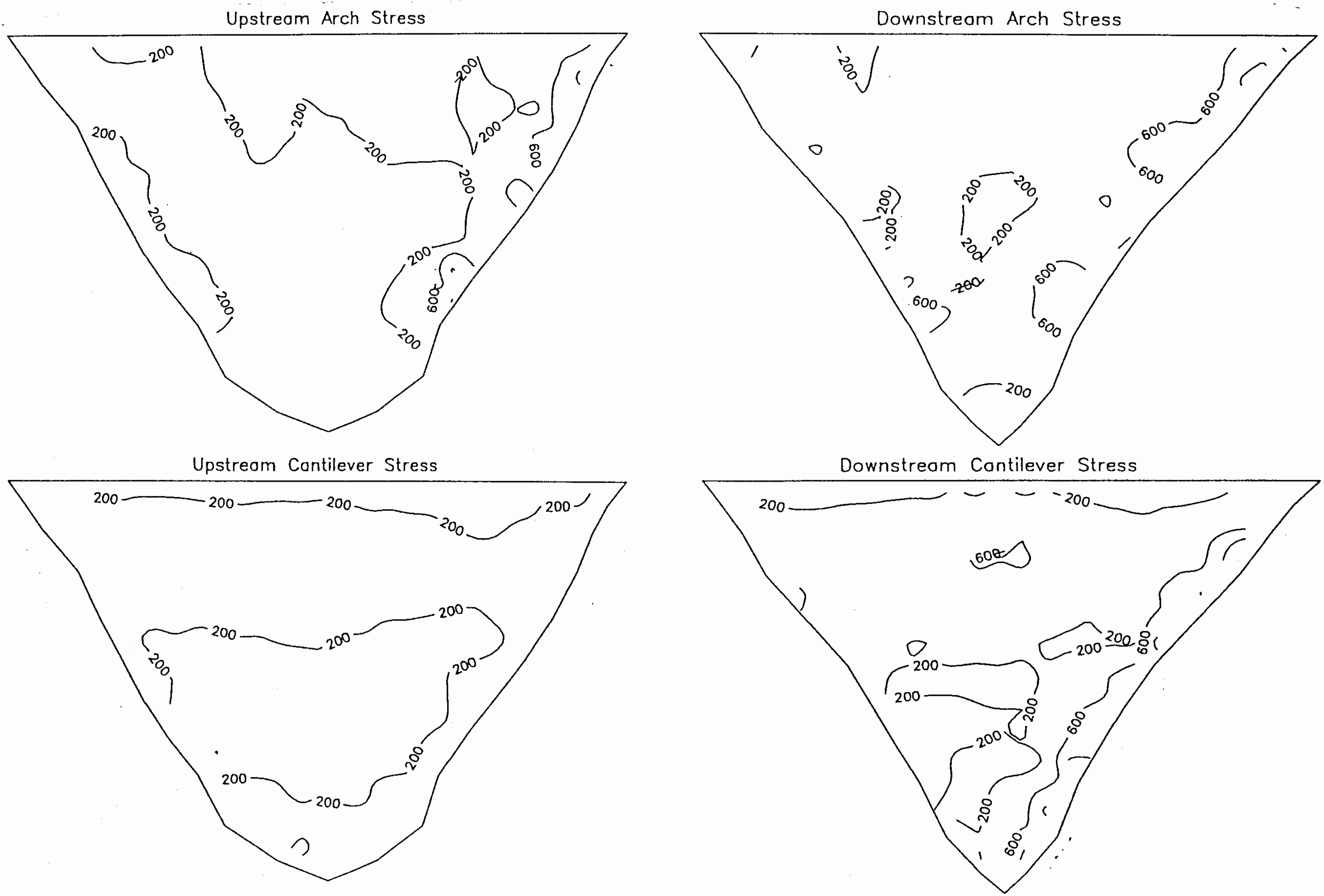


Figure 3.15 Envelopes of Maximum Stress (in psi) for Case 10: Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 360 ft Reservoir.

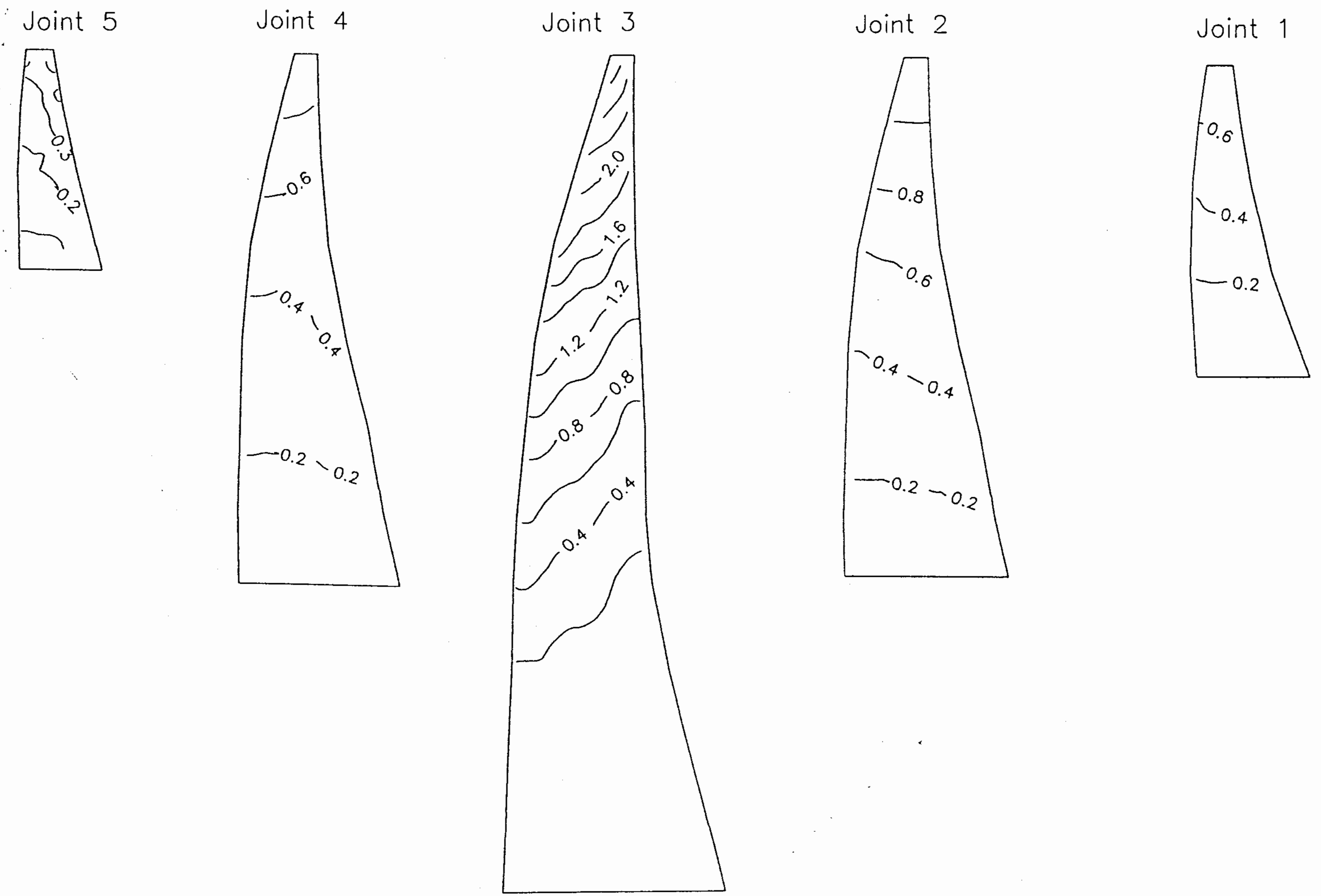


Figure 3.16 Envelopes of Maximum Joint Opening (in inches) for Case 3: Uniform Free-field Motion with 2/3-Upper Left Abutment Record, Vertical Joints Open, 233 ft Reservoir.

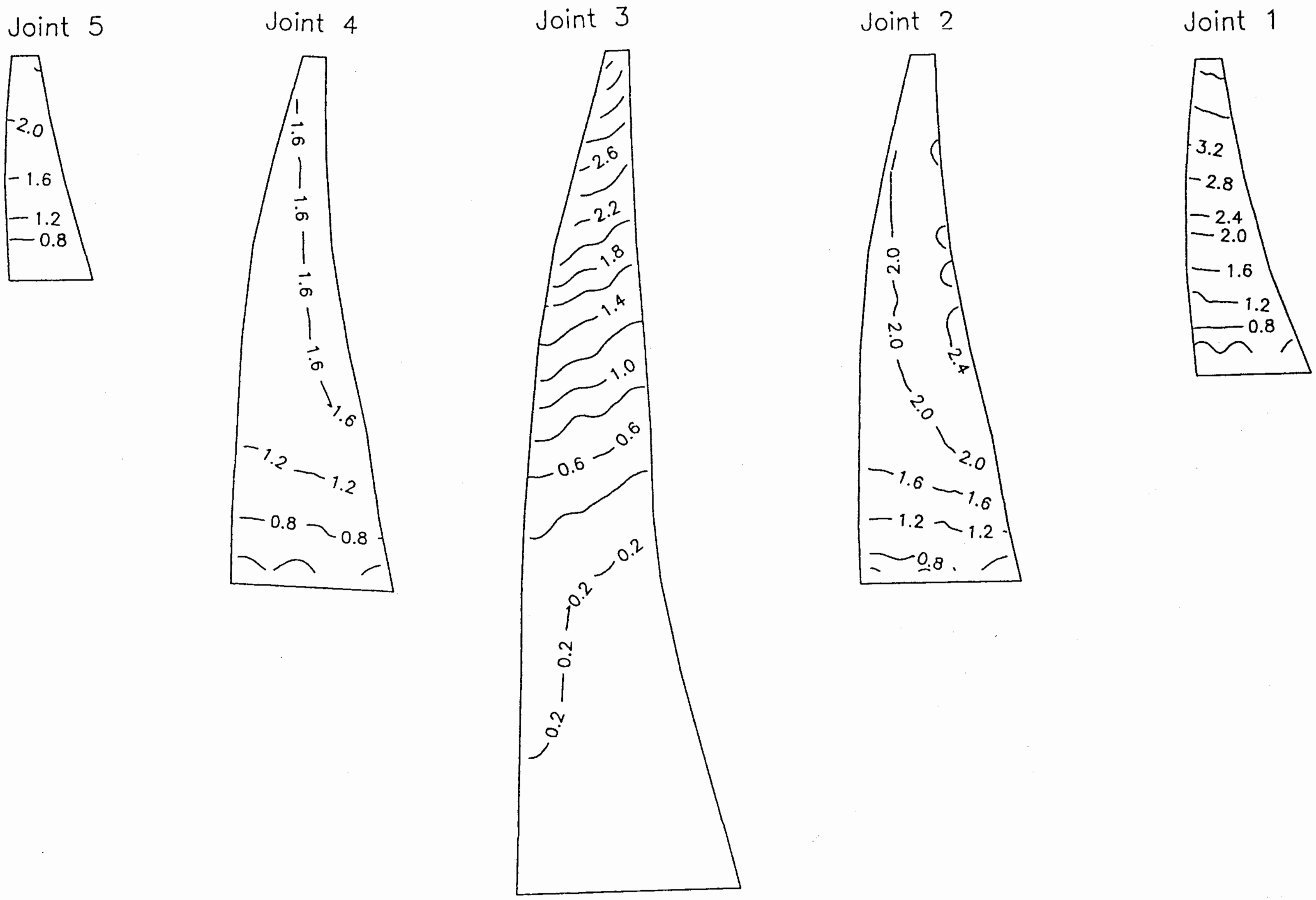


Figure 3.17 Envelopes of Maximum Joint Opening (in inches) for Case 5: Non-uniform Free-field Motion, Vertical Joints Open, 233 ft Reservoir.

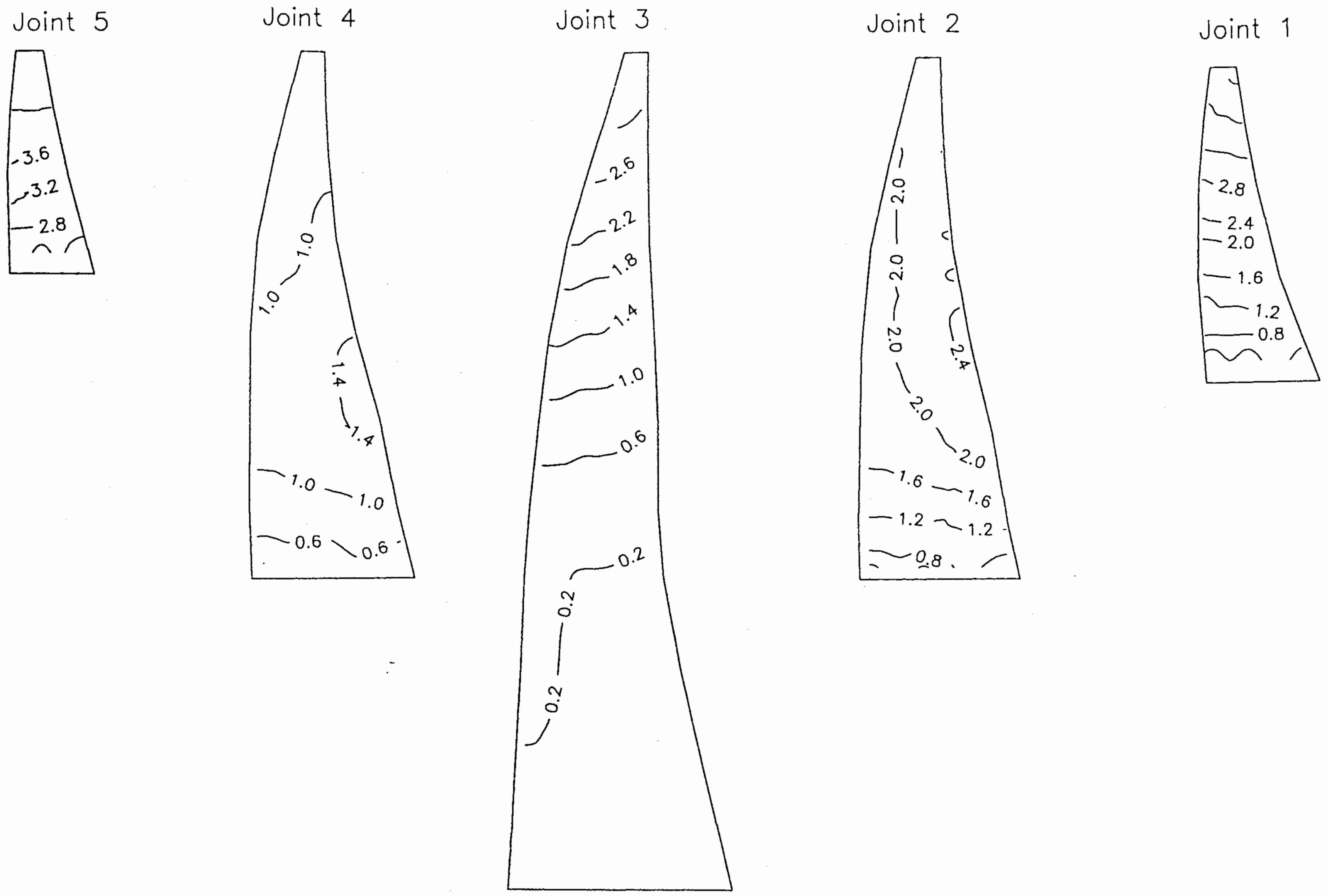


Figure 3.18 Envelopes of Maximum Joint Opening (in inches) for Case 7: Non-uniform Free-field Motion, Vertical Joints Open, Thrust Block Omitted, 233 ft Reservoir.

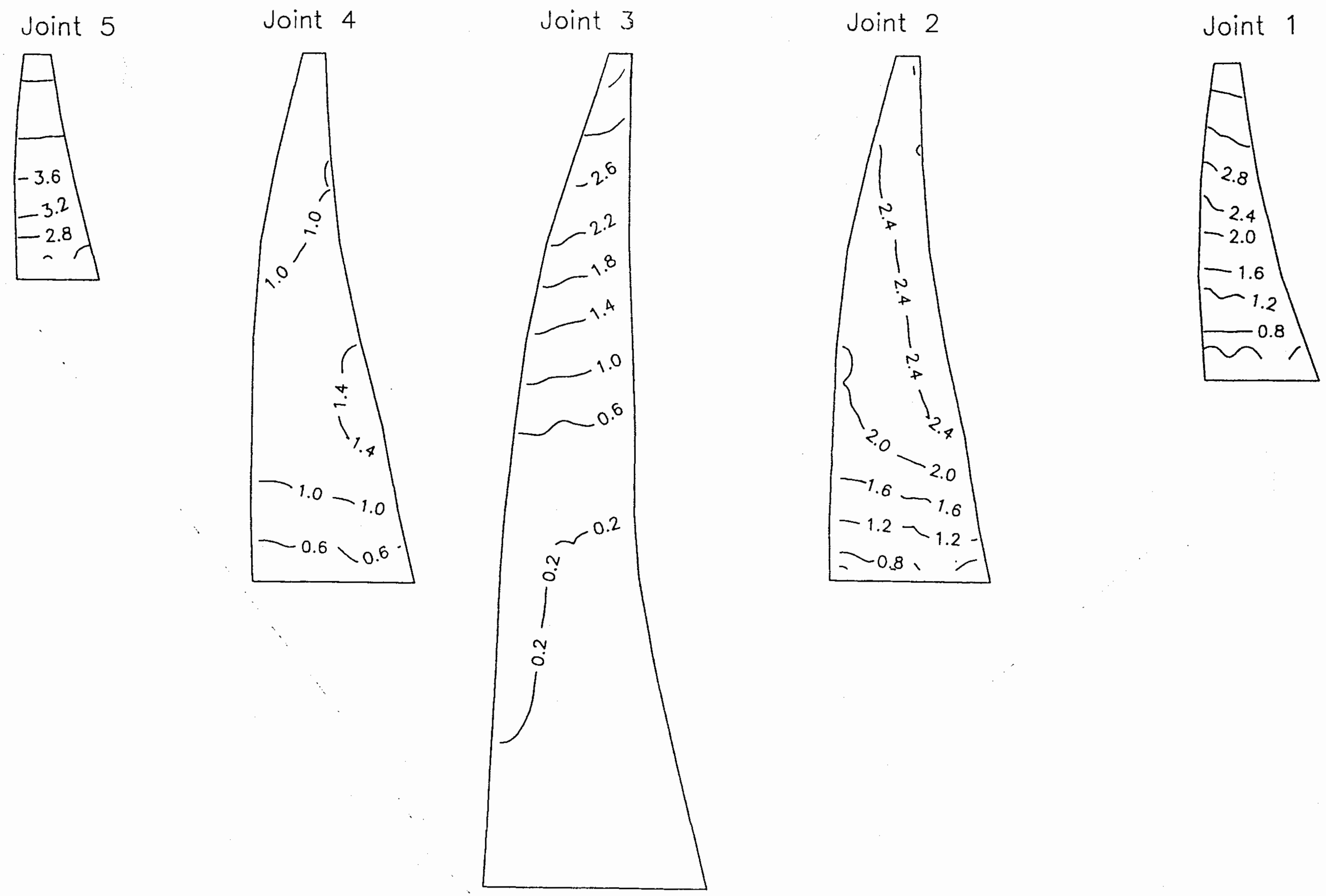


Figure 3.19 Envelopes of Maximum Joint Opening (in inches) for Case 8: Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 233 ft Reservoir.

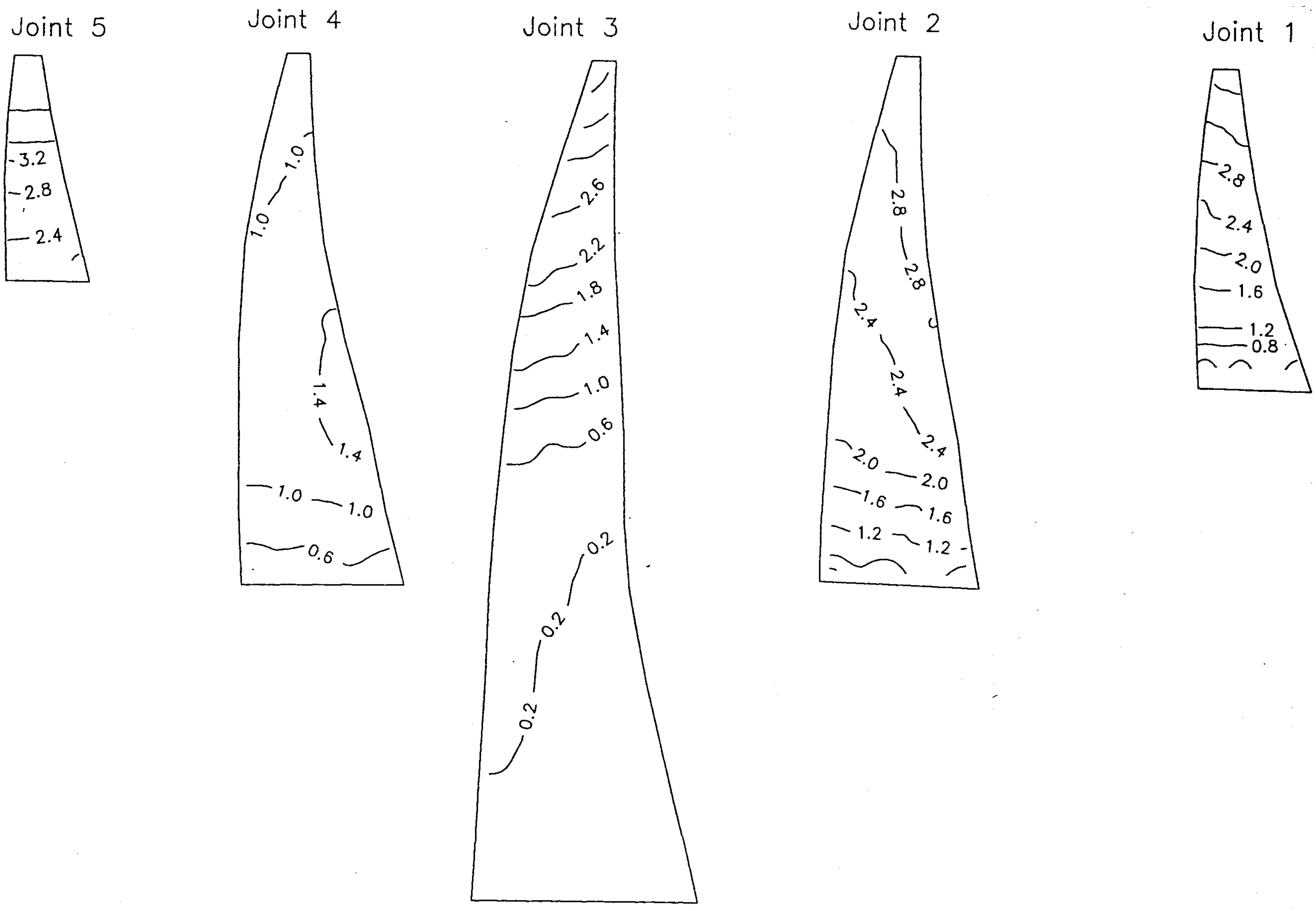


Figure 3.20 Envelopes of Maximum Joint Opening (in inches) for Case 9: Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 300 ft Reservoir.

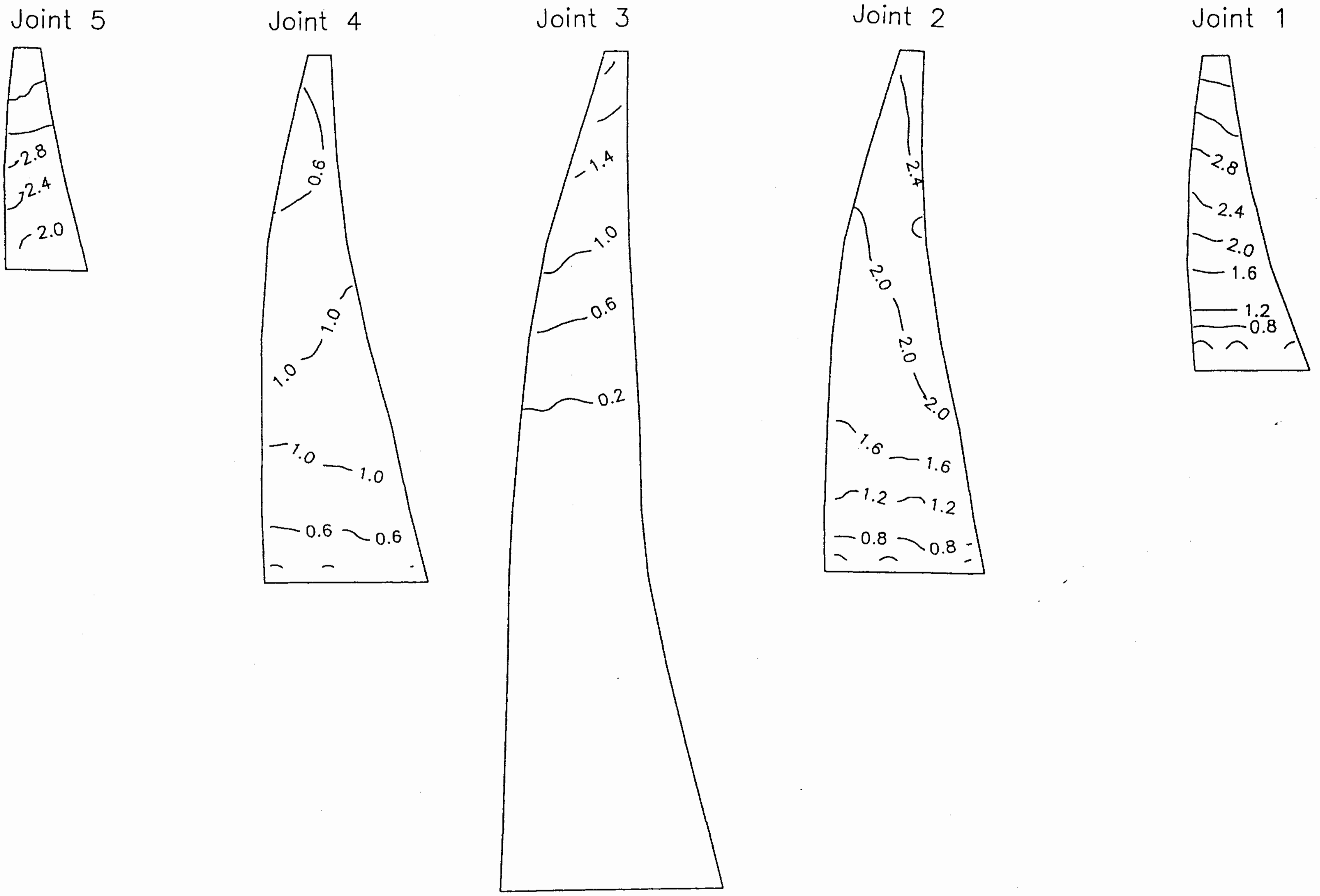


Figure 3.21 Envelopes of Maximum Joint Opening (in inches) for Case 10: Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 360 ft Reservoir.

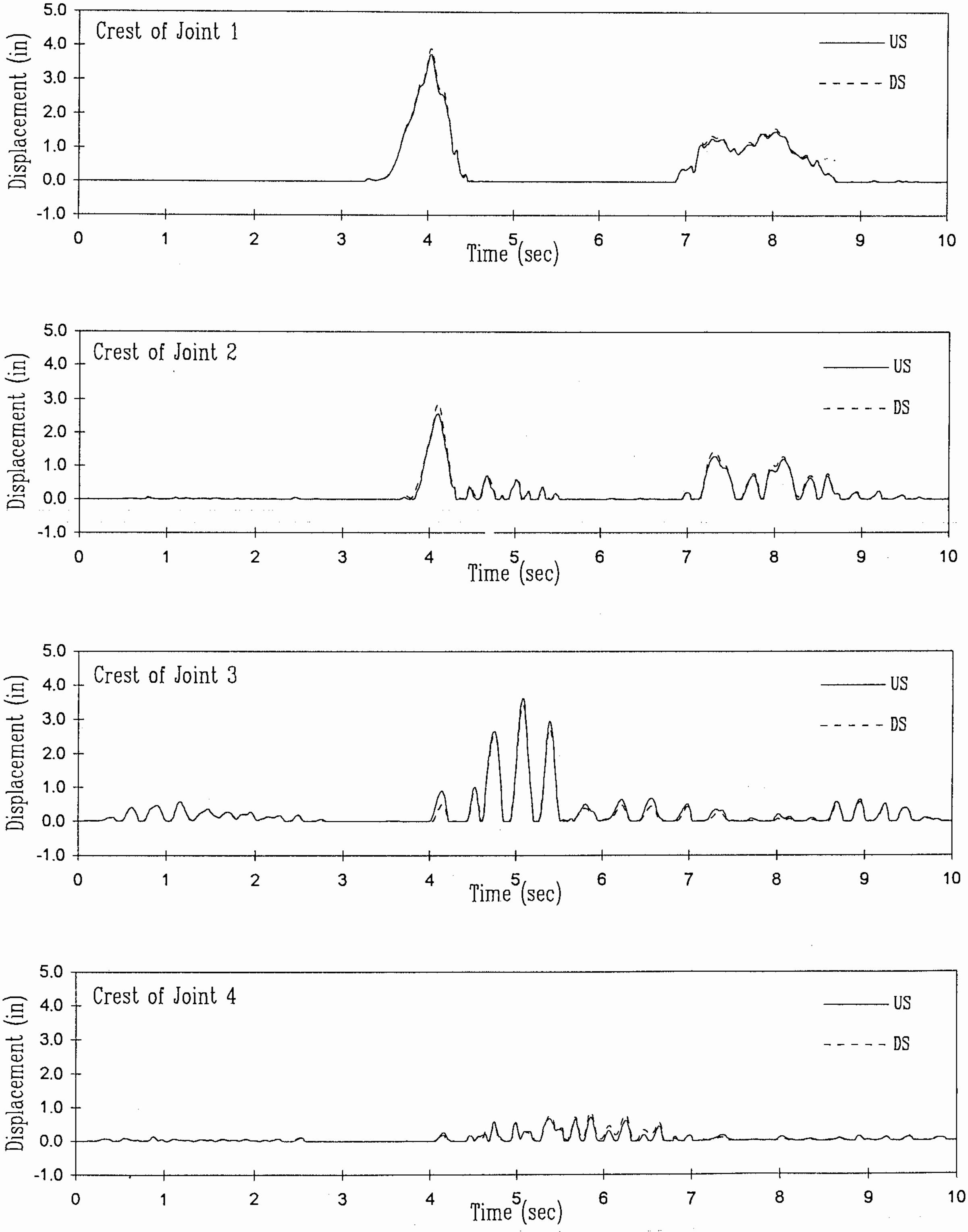


Figure 3.22 Joint Opening History for Case 8: Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 233 ft Reservoir.

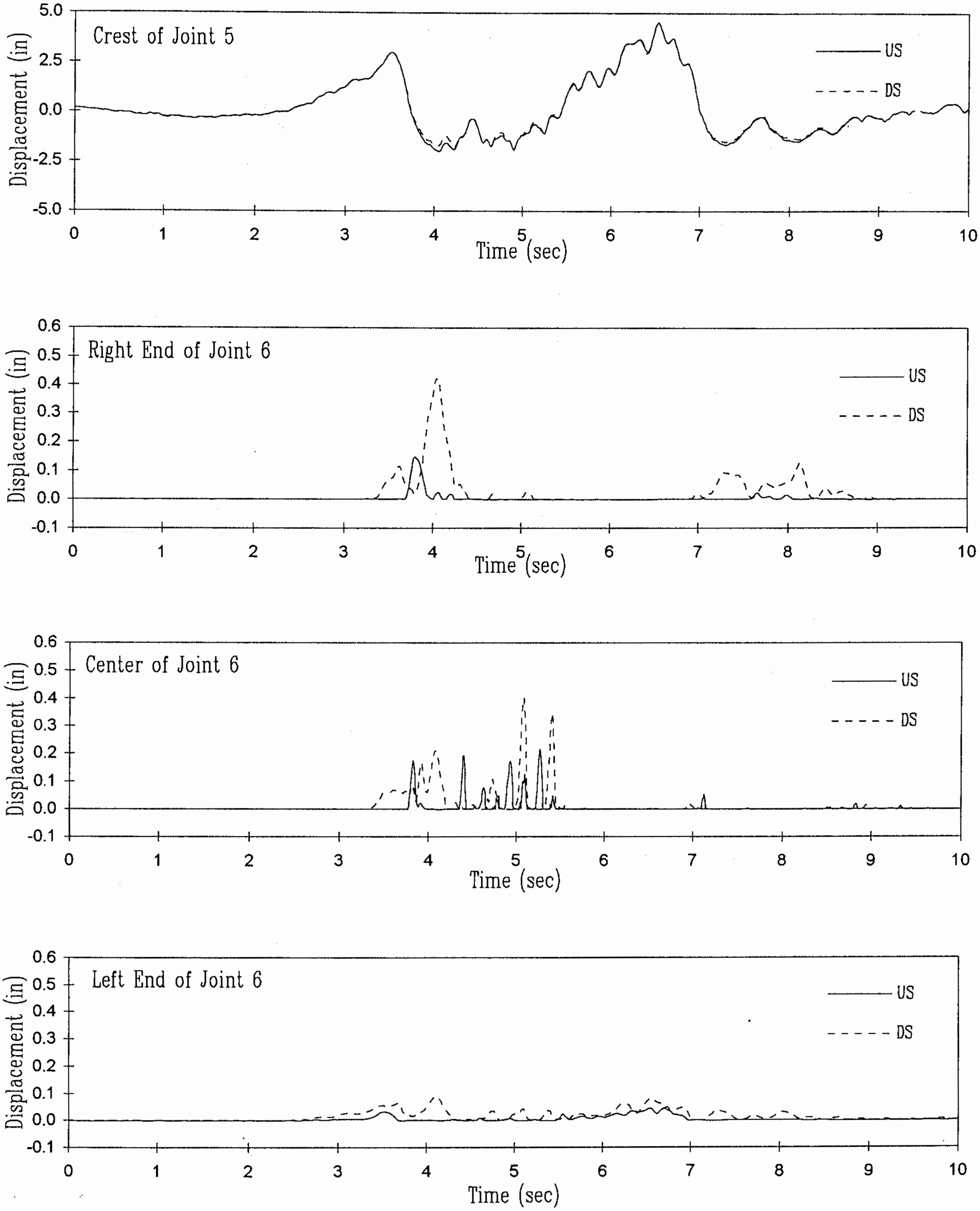
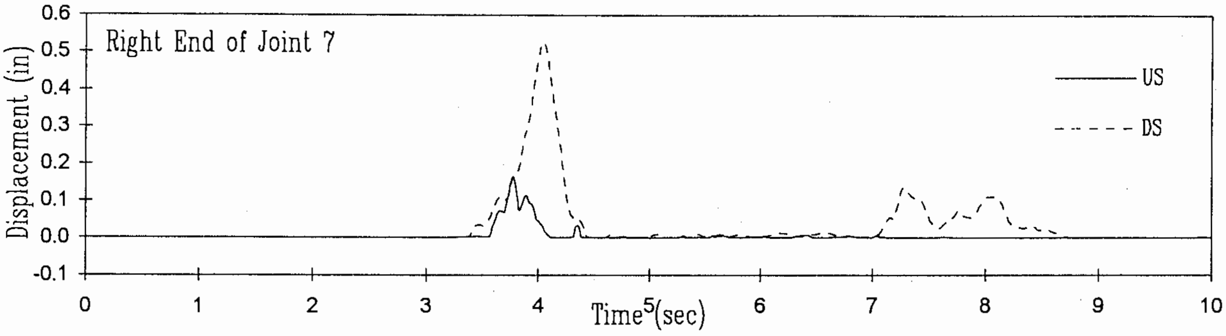
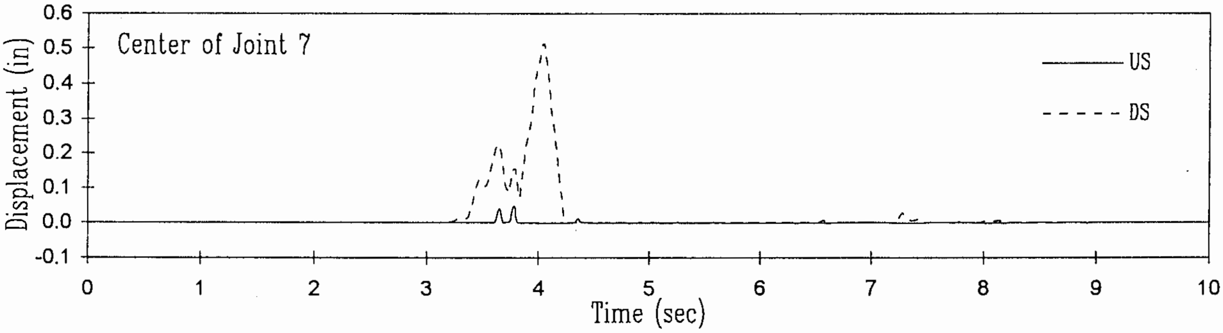


Figure 3.22 Joint Opening History for Case 8: Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 233 ft Reservoir. (con'td)





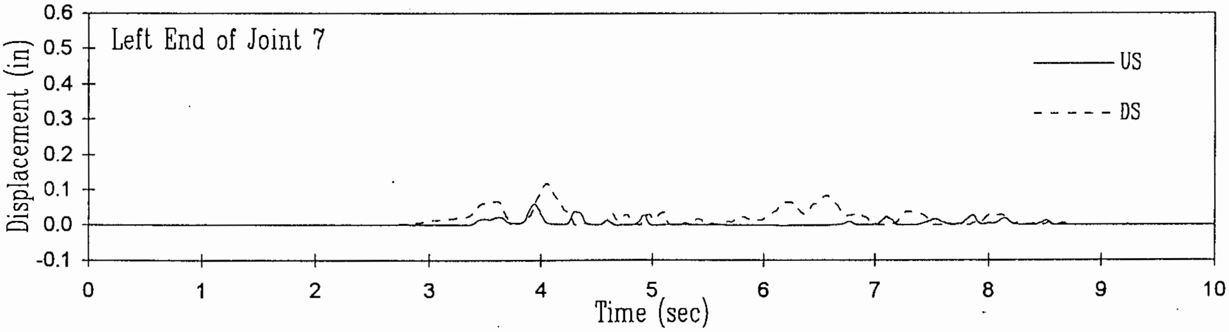


Figure 3.22 **Joint Opening History for Case 8: Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 233 ft Reservoir. (con'td)**

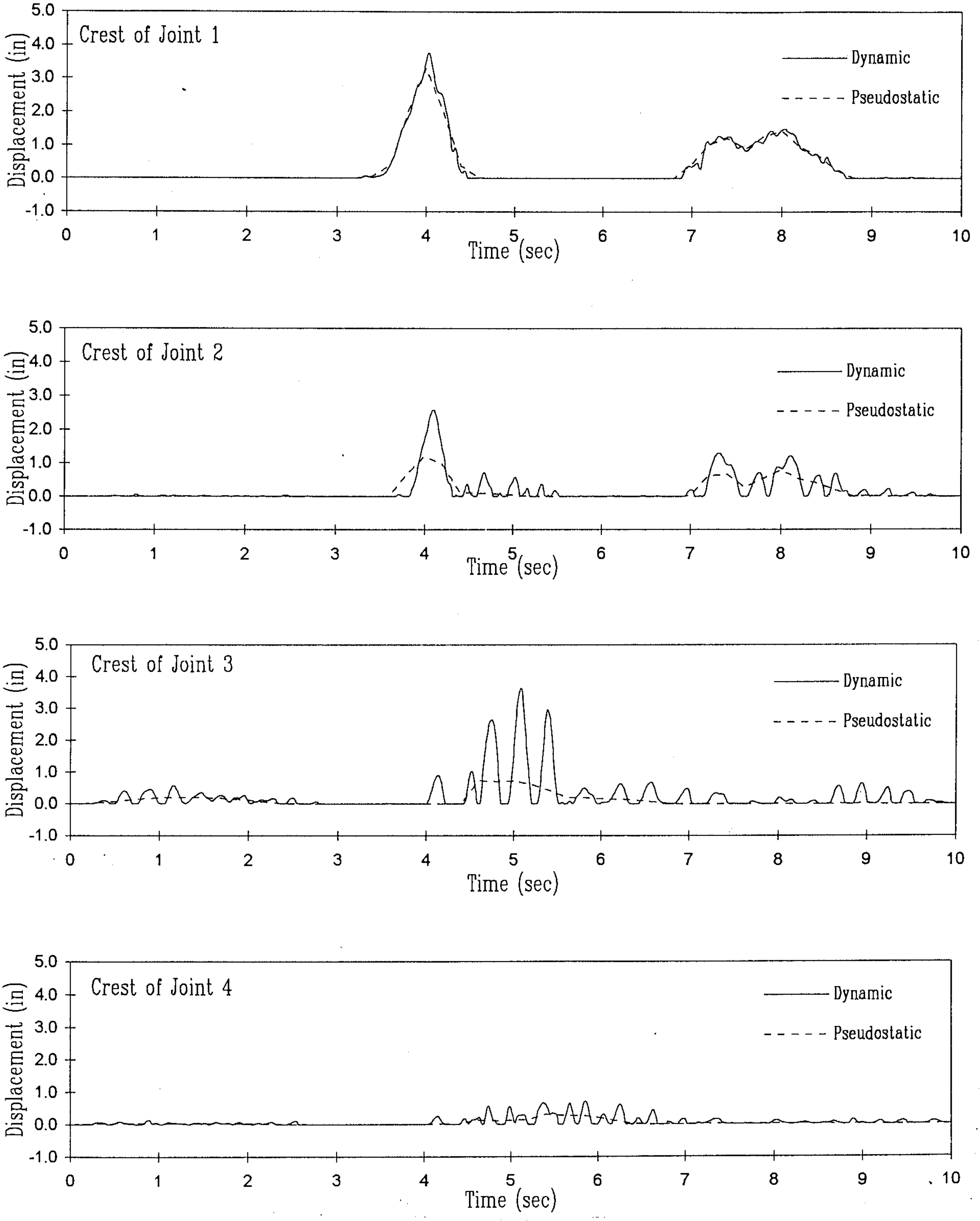


Figure 3.23 Joint Opening History for Case 8 (Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 233 ft Reservoir) Compared with Pseudo-static Joint Opening.

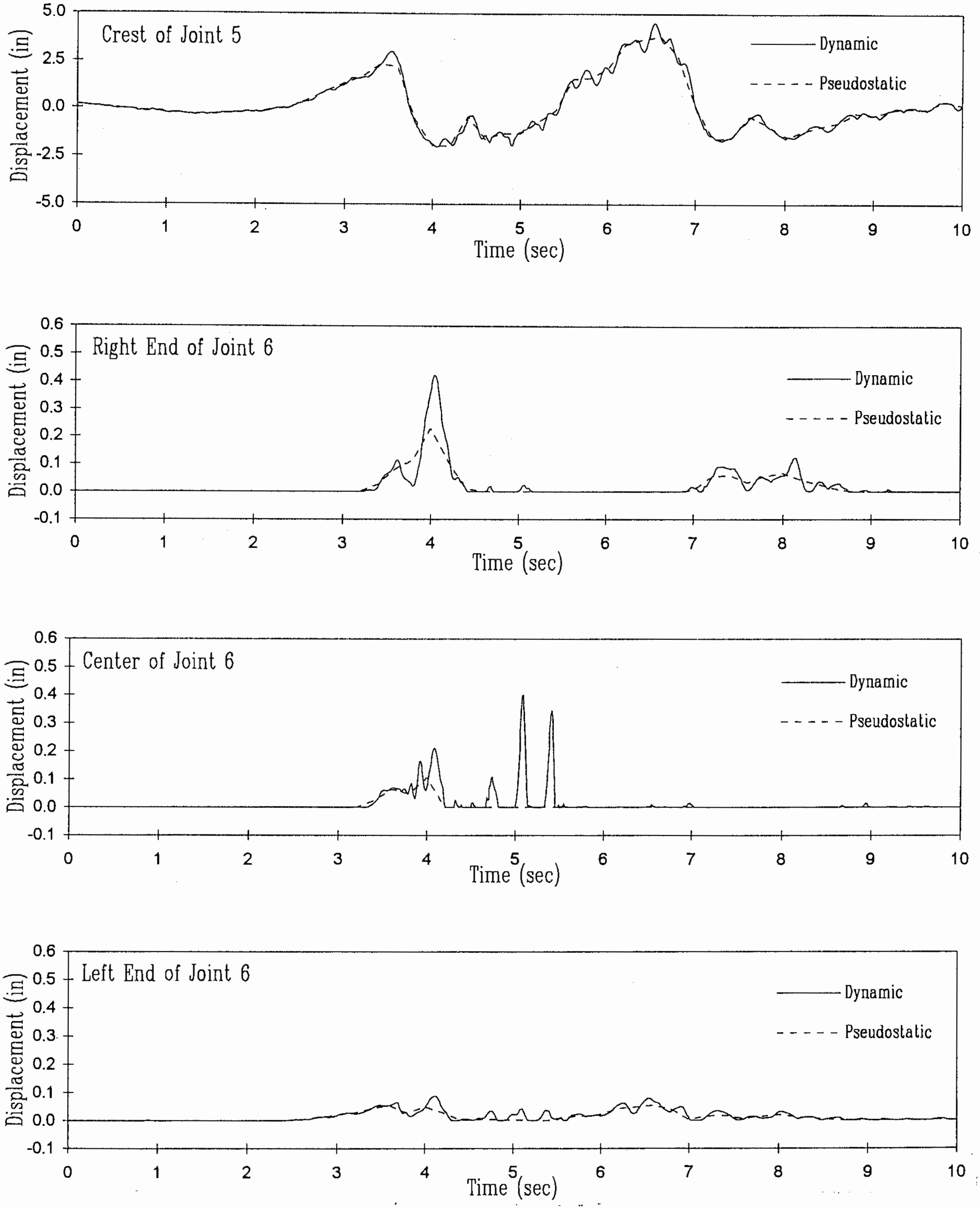


Figure 3.23 Joint Opening History for Case 8 (Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 233 ft Reservoir) Compared with Pseudo-static Joint Opening. (cont'd)

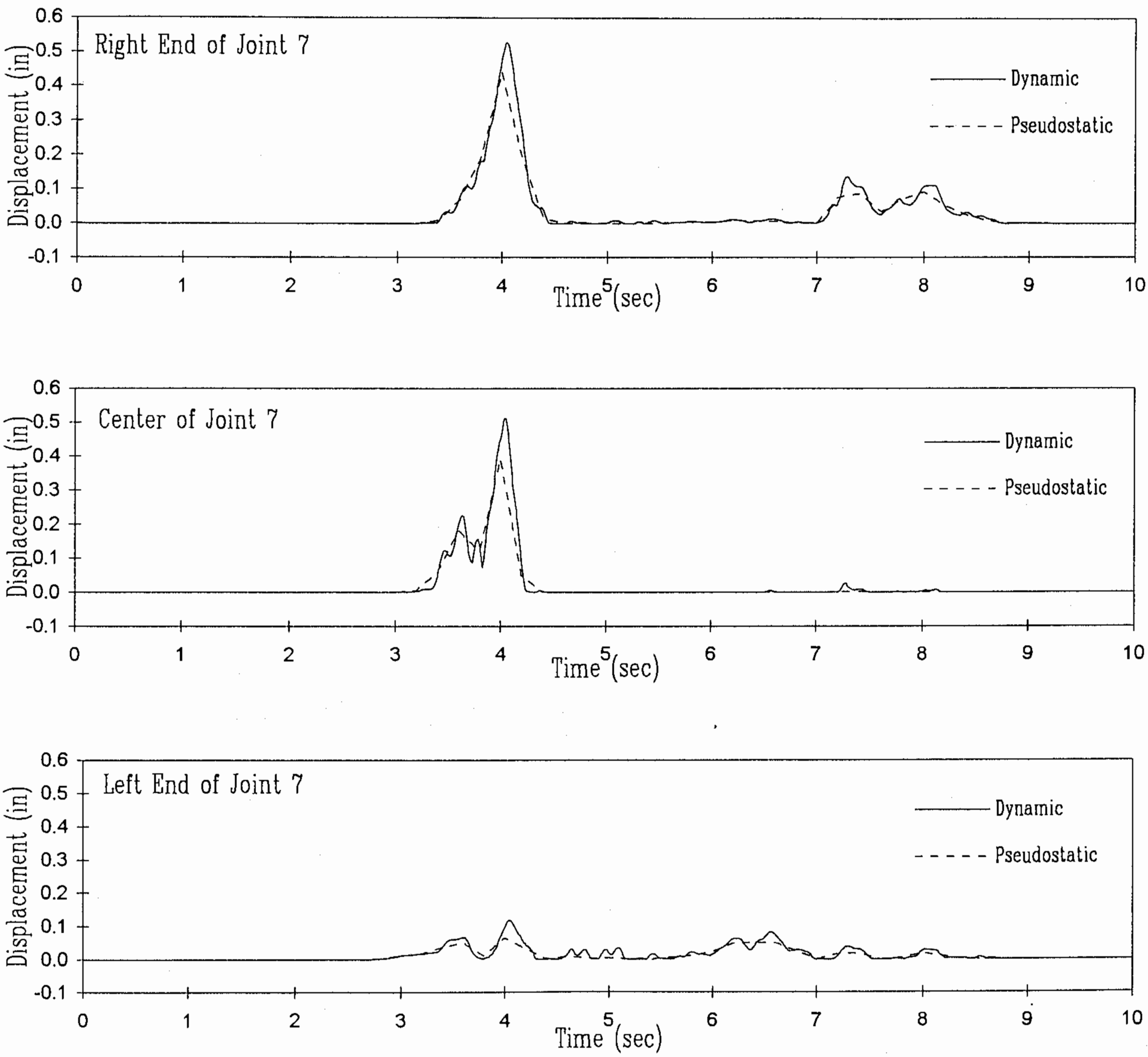


Figure 3.23 Joint Opening History for Case 8 (Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 233 ft Reservoir) Compared with Pseudo-static Joint Opening. (cont'd)

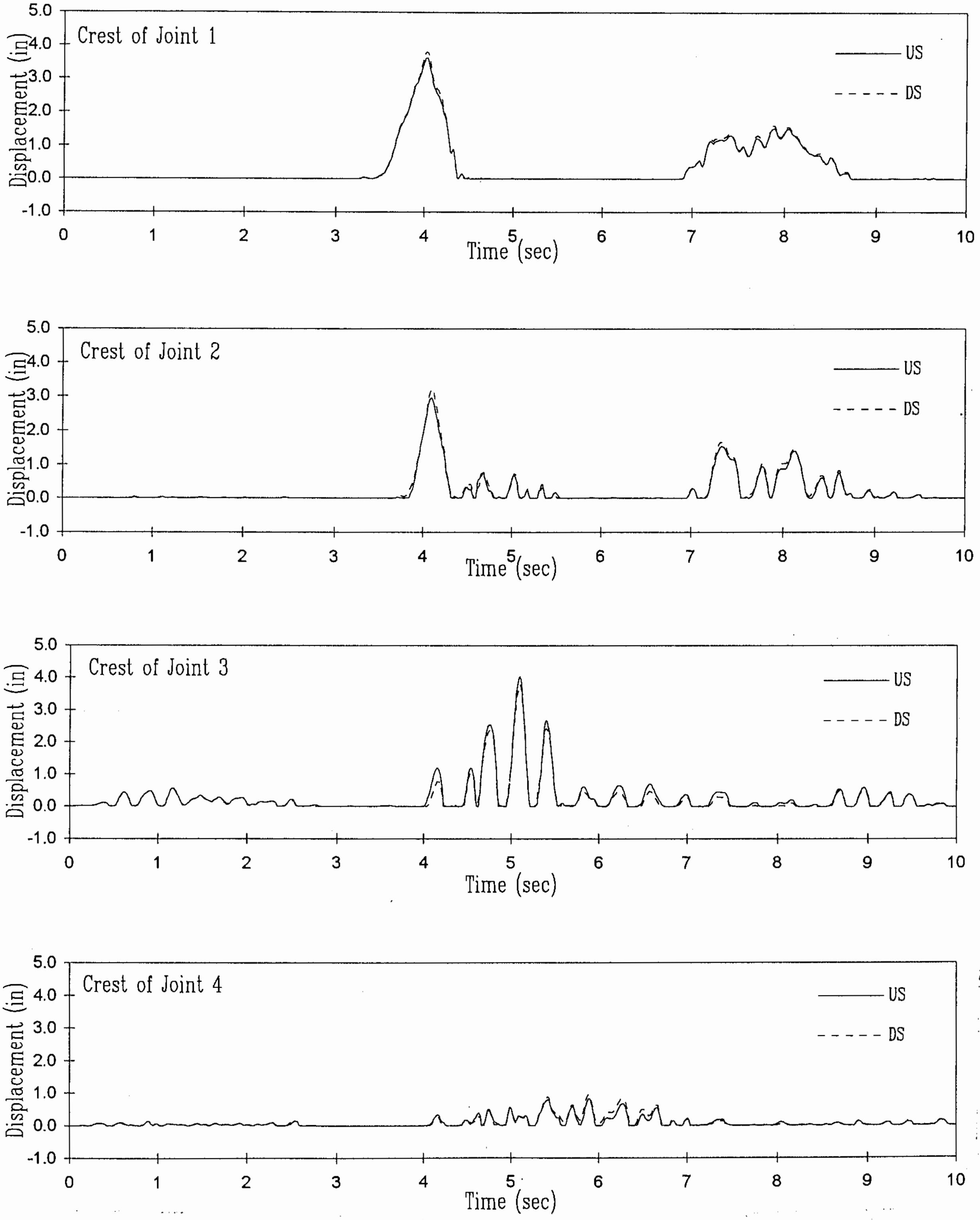


Figure 3.24 Joint Opening History for Case 9: Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 300 ft Reservoir.

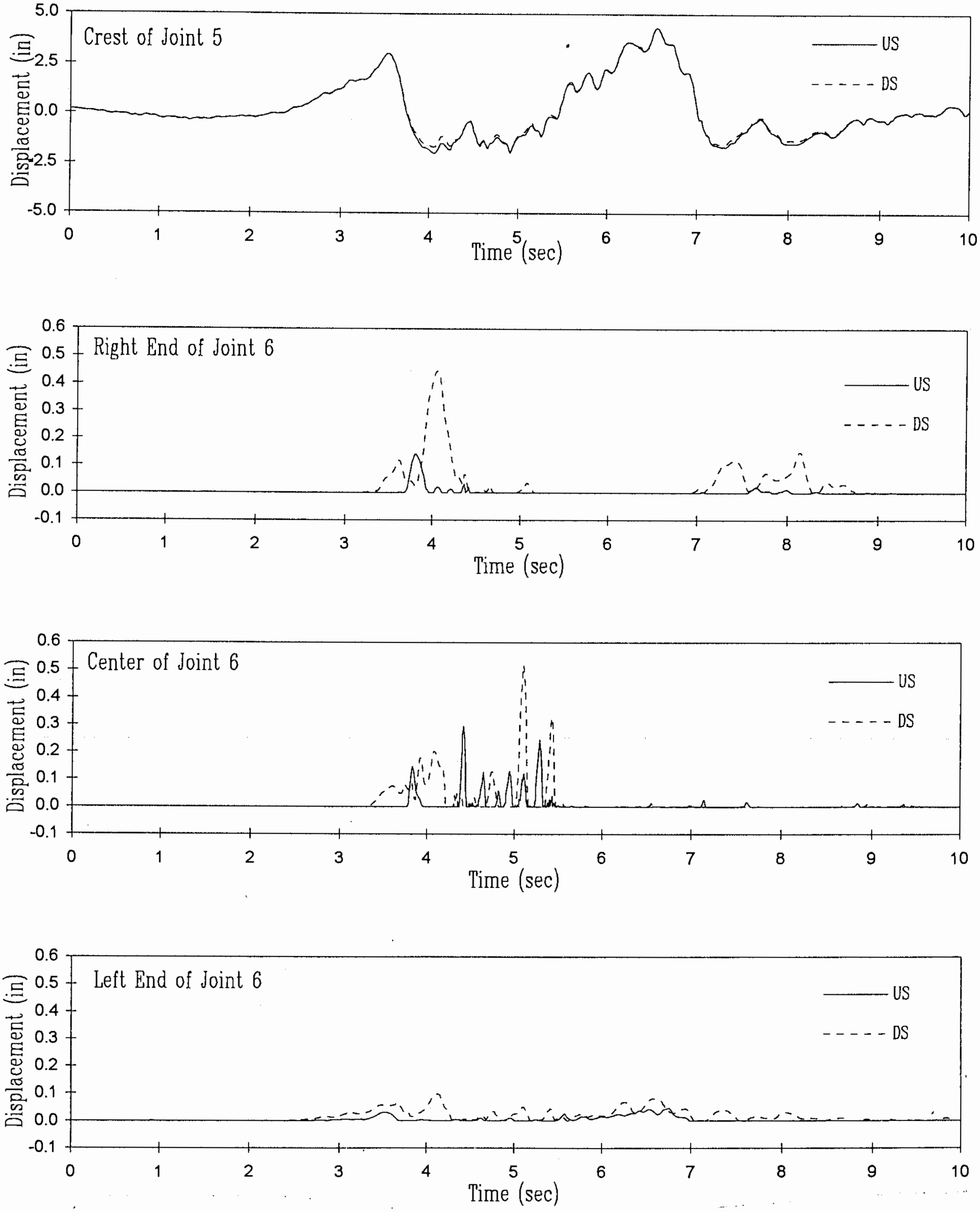


Figure 3.24 Joint Opening History for Case 9: Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 300 ft Reservoir. (cont'd)

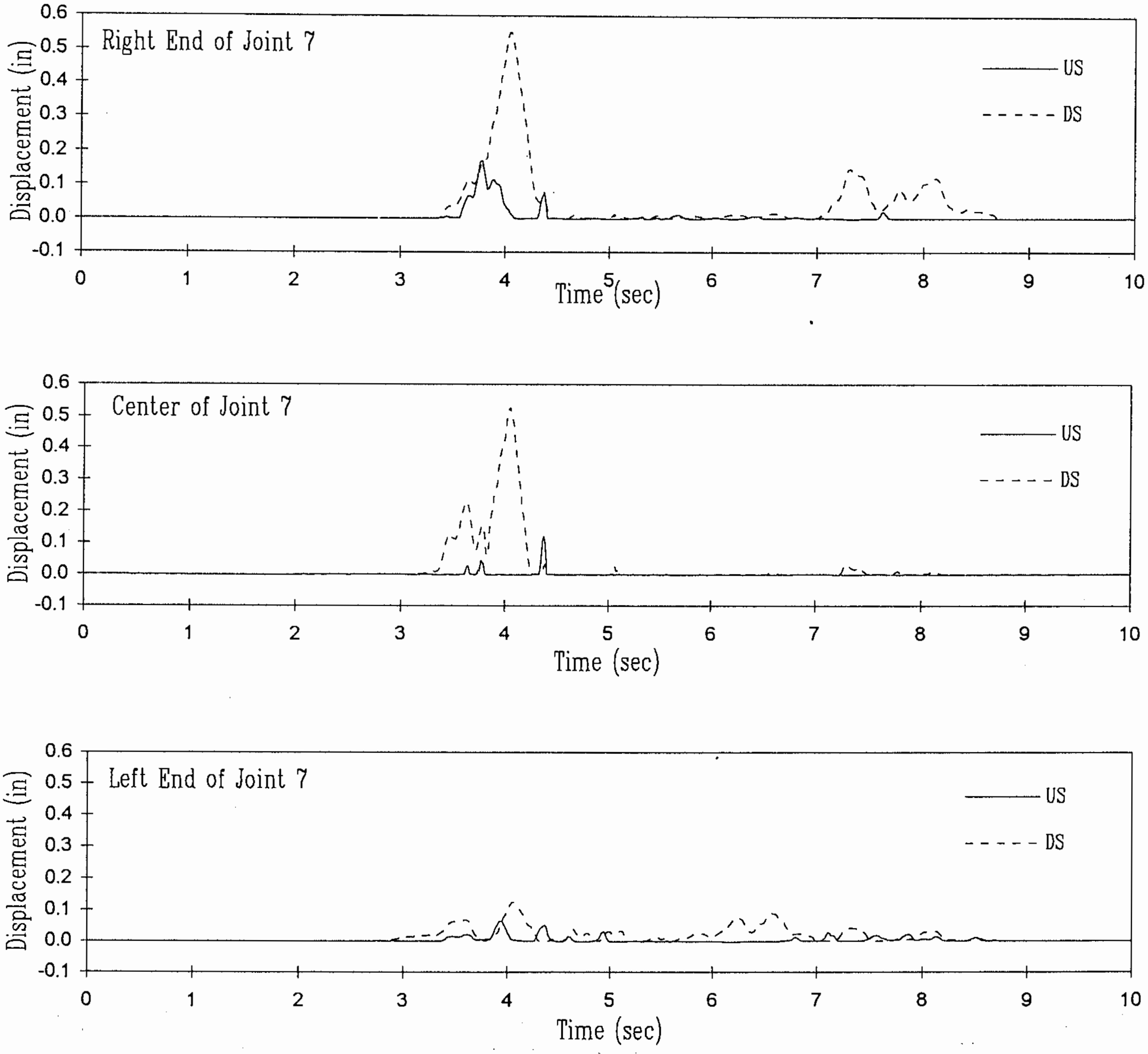


Figure 3.24 Joint Opening History for Case 9: Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 300 ft Reservoir. (cont'd)

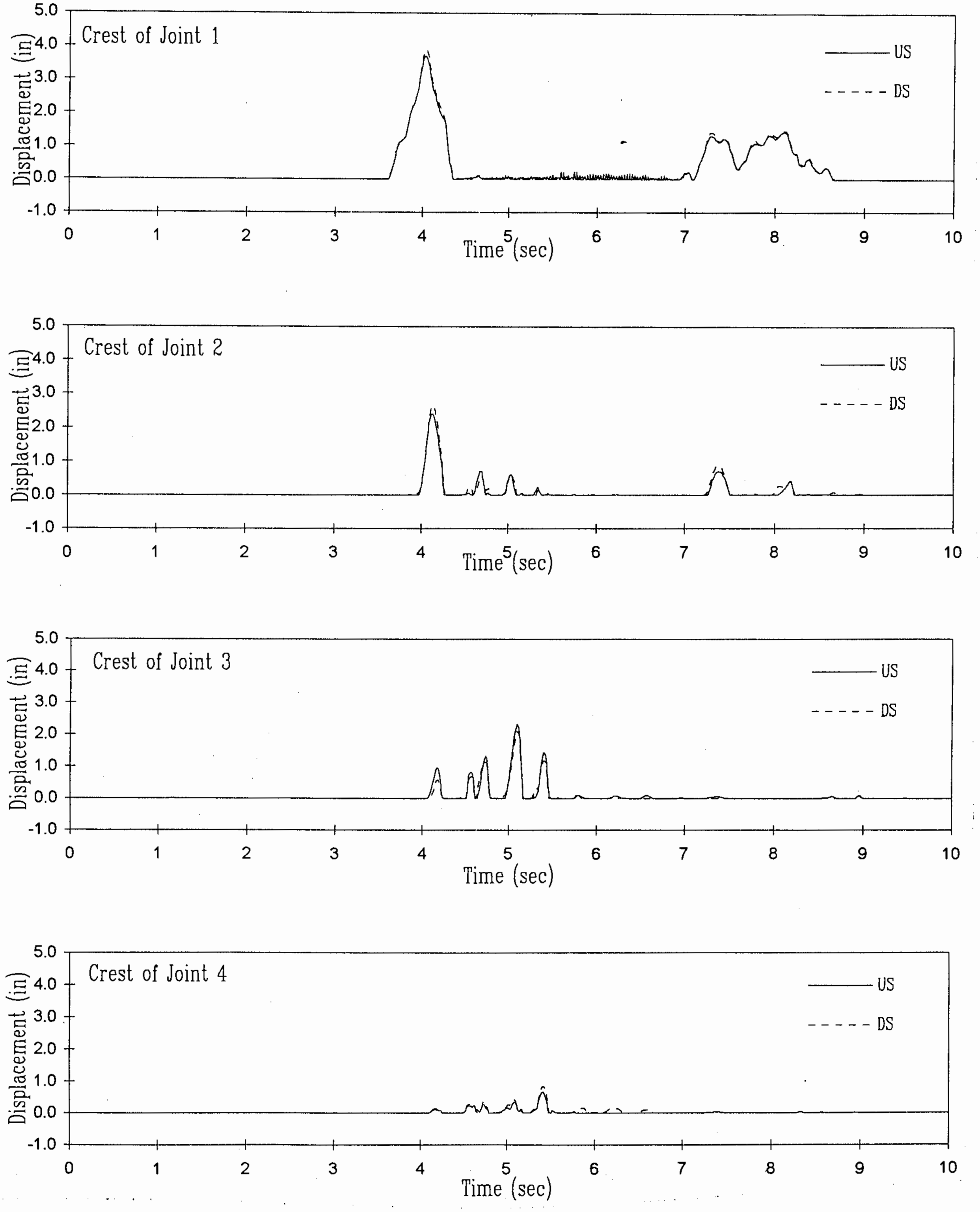


Figure 3.25 Joint Opening History for Case 10: Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 360 ft Reservoir.

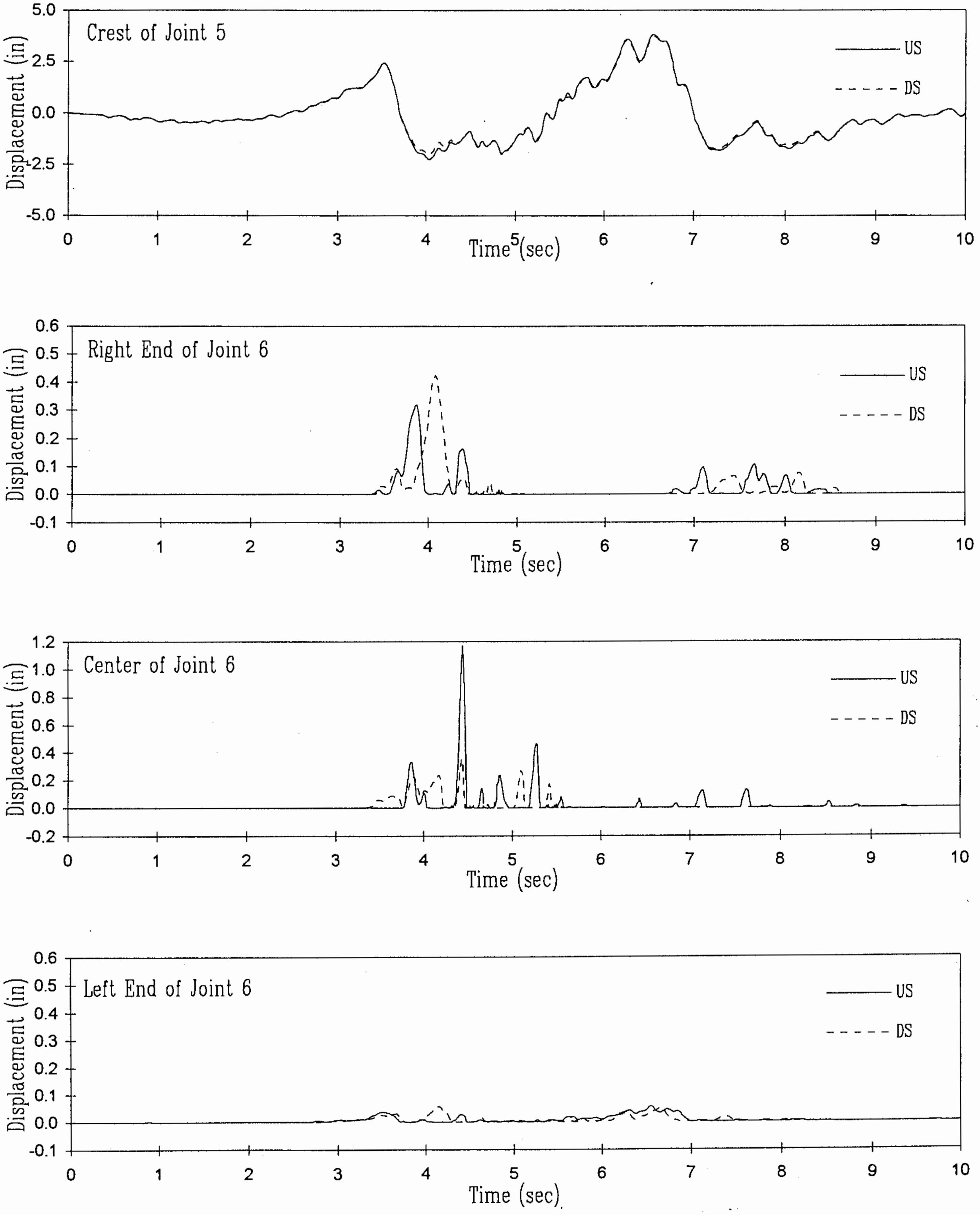


Figure 3.25 Joint Opening History for Case 10: Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 360 ft Reservoir. (cont'd)

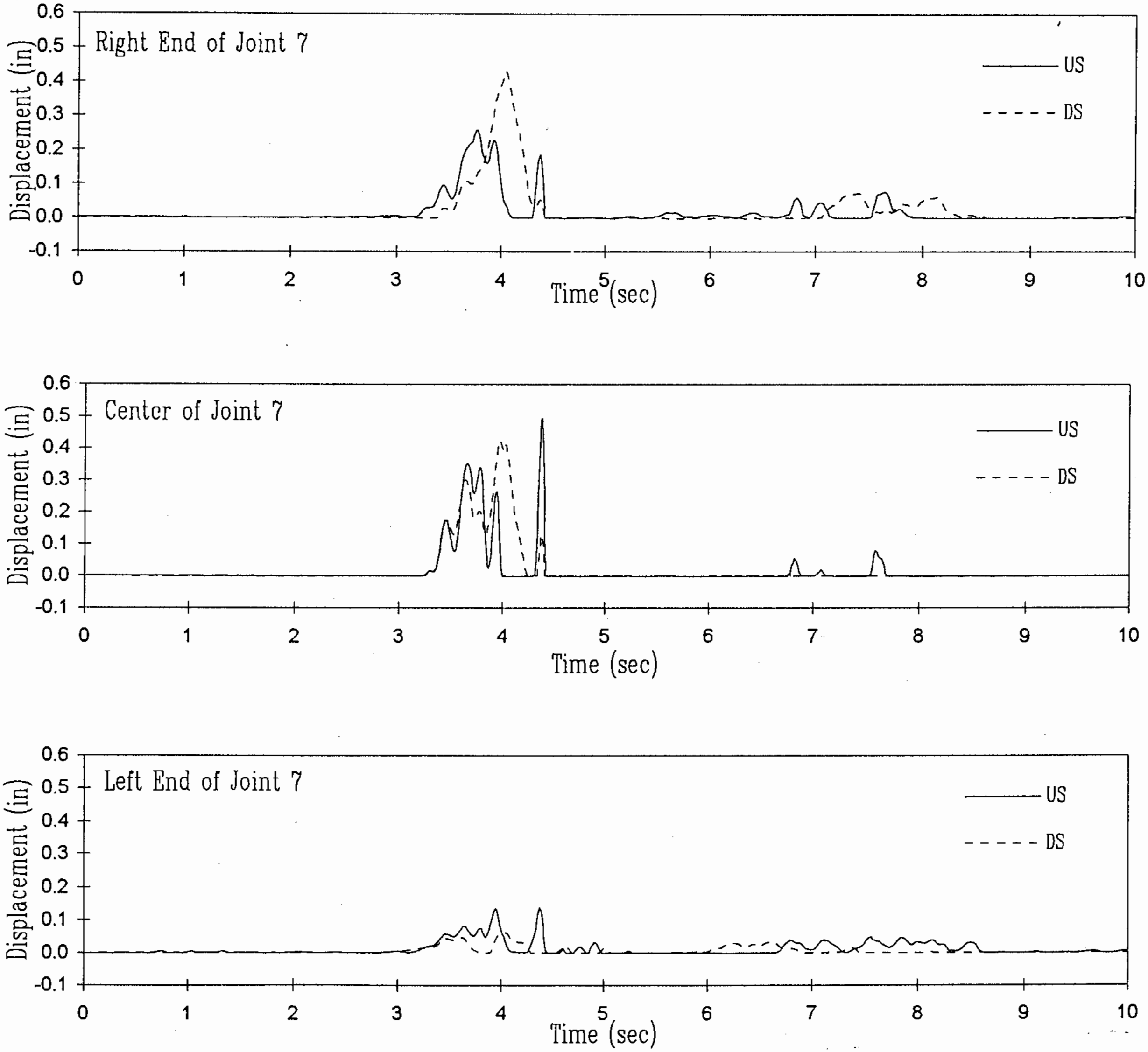


Figure 3.25 Joint Opening History for Case 10: Non-uniform Free-field Motion, Vertical and Lift Joints Open, Thrust Block Omitted, 360 ft Reservoir. (cont'd)

4. CONCLUSIONS AND RECOMMENDATIONS

The earthquake response of the Pacoima dam to the 1994 Northridge earthquake has been evaluated in this study. The limited processed strong ground motion records obtained from CDMG accelerometers for the dam and canyon were used to study issues in seismic response and analysis of arch dams in general, and the seismic response of Pacoima dam in particular.

The computer program ADAP–88 was used in this study to examine the effects of contraction joint opening on the earthquake response of the dam. The acceleration recorded in the radial direction at 80% of the dam height near the left quarter point was compared with the corresponding acceleration computed with the model of the dam. The recorded and computed accelerations were comparable for the early part of the response history, including the first large pulse. However, the computed response had several acceleration spikes that were not in the recorded acceleration. The analytical model of the dam overestimated the recorded peak accelerations by about 50%. The discrepancy could be due to inadequate representation of the free-field motions, since a deconvolution analysis was not performed, or possibly due in part to the absence of radiation damping in the model.

Major observations, conclusions and recommendations are summarized in the following subsections.

Amplification Effects Due to Canyon Topography

The effect of the canyon topography on the seismic response of the dam was studied by analyzing two linear (closed–joint) cases assuming a uniform free–field motion as input. The accelerations recorded at a location downstream of the dam were used in one case while two–thirds of the accelerations recorded at the upper–left abutment were used in the second. Important differences were found between the responses obtained for two cases. The maximum stresses for the case using downstream records were about one–half of the corresponding stresses for the case using the upper left abutment record.

Since the free–field motion is not uniform, the effects of non–uniform motion were examined by assuming a variation of motion along the dam–foundation rock interface based on the limited processed records on the interface. The accelerations recorded at the upper left abutment and at the base were specified for the crest and base elevations, respectively. The accelerations for other elevations were obtained by linear interpolation of the accelerations at the crest and base elevations. The comparison of stresses caused by the non–uniform motion with the stresses obtained for the uniform–input case using 2/3 of the upper left abutment records indicated significant differences both in magnitude and distribution of the stresses. Therefore, the assumption of uniform free–field motion for an arch dam does not provide an accurate estimate of the dam response.

Effects of Contraction Joint Opening

To assess the effects of opening of contraction joints and abutment joints on stresses of the dam, five contraction joints were used in the model to represent eleven joints of the dam. Zero tensile strength was assumed for the joints with slippage of the joints prevented. Analysis of the model with joints using uniform and non–uniform free–field motions showed a significant reduction of maximum tensile arch stresses due to the opening of the joints. This comparison demonstrates the deficiency of linear analysis procedures in computing response of an arch dam to strong ground motion. The opening of the joints transfers arch loads to the cantilevers and therefore increases the maximum tensile cantilever stresses compared with the closed joint case. However, the joint at the dam–foundation rock interface limits the cantilever stresses near the base.

Of particular importance in the study was the magnitude of the joint opening because of the implications in the selection of depth of the keys and the validity of no–slip assumption used in the analyses. The opening of the contraction joints at the upstream and downstream faces was found to be in–phase, indicating complete separation through the dam thickness. The maximum opening for the case of uniform input (2/3 upper left abutment record) and the case of non–uniform input was 2.6 in. and 4.0 in., respectively. These opening displacements are less than the 12 in. depth of the keys in Pacoima dam; therefore, it can be presumed that no major slippage between the monoliths occurred during the earthquake and the no–slip assumption can be justified for the vertical joints.

Effects of Opening of Lift Joints

The maximum tensile cantilever stress of the dam for the case of non–uniform input with contractions joints allowed to open was found to be about 1800 psi occurring at the downstream face. Since this level of tensile stress cannot be resisted by the lift joints or the concrete, it was concluded that some opening of the lift joints and/or horizontal cracking of concrete has occurred during the earthquake resulting in release of cantilever stresses. To account for this effect, nonlinear analysis of the dam was performed using two horizontal joints, at 97 ft and 202 ft elevations, with zero tensile strength assumed for the joints and joint slippage prevented. The opening of the horizontal joints resulted in reduction of the downstream cantilever stresses, while the other stresses were unaffected. The reduced maximum stress of 1000 psi still exceeds the tensile strength of concrete indicating more horizontal joints should be included in the model to represent the effects of cantilever cracking and the opening of lift joints.

As for the vertical joints, the magnitude and extent of opening of the horizontal joints were considered to be of interest due to implications in the stability of the dam and the validity of no–slip assumption. The opening of these joints at the upstream and downstream faces was found to be out–of–phase, in general, indicating rocking motion of the cantilevers. Complete opening through the dam thickness was also indicated at some locations along the joints. The maximum opening, however, was only about 0.5 inch which is too small to be of concern for the stability of the dam or to question the validity of the no–slip assumption used in the analysis.

The Importance of the Pseudo–static Component

As mentioned previously, the spatial variation of the input motion significantly influences the magnitude and distribution of the stresses of an arch dam. For a better understanding of the response to non–uniform input, the dam was analyzed with inertial effects neglected. The resulting response, termed the pseudo–static component, was compared to the total response including the dynamic effects. The stresses as well as joint openings computed for the two cases were found to be remarkably similar. It can be concluded that a large part of the dam response to non–uniform input is the pseudo–static response. The contribution from dam vibration has a smaller effect except near the upper center portion of the dam.

Dam–Water Interaction Effects

The reservoir level at the time of the Northridge earthquake was 233 ft (64% full). The first ten natural frequencies of the dam for this level of the reservoir were found to be close to the frequencies of the dam with empty reservoir. It can be concluded that no major dam–water interaction effects were involved in the response of the dam to the Northridge earthquake.

To assess the effects of dam–water interaction, the seismic response of the dam was computed for two other water levels: 300 ft (82% full), corresponding to normal pool condition; and 360 ft (98% full), corresponding to hypothetical flood and earthquake conditions. The non–uniform free–field motion was used for all three cases with opening allowed for all joints. From the stress results, the dam–water interaction effects in the response was found to be still small for the case with the normal pool condition. However, with water level increased to 360 ft, the cantilever stresses of the downstream face increased significantly, indicating the importance of the dam–water interaction effects for reservoir levels above the crest of the spillway.

Structural Stability of the Dam

Based on the summary in Section 2.2, the major damage suffered by the dam during Northridge earthquake consists of permanent opening of the left thrust block joint, two major diagonal cracks, one in the arch section and the other in the thrust block, and several horizontal and vertical offsets along the joints. Apart from such effects, which have been found to be related to the failure of the foundation rock, the dam remained intact.

The seismic response modeled for the dam in this study does not indicate any of the observed damages because the foundation failure, which resulted in the permanent effects of the dam, was not accounted for in the modeling and analysis. Nevertheless, the integrity of the dam under the severe ground shaking was simulated by a nonlinear seismic analysis in which a non–uniform free–field input was used and all joints were allowed to open.

Strong Motion Instrumentation

The accelerograms obtained by the CDMG instruments for the body and the canyon of the Pacoima dam proved very useful for evaluating the seismic response of the dam to the Northridge earthquake. However, the lack of sufficient processed data caused some difficulties in the course of the study. For example, due to lack of data, the same free–field motion was specified for the left and right sections of the dam-foundation interface. Also, the correlation of the computed and recorded motions was carried out for a single point on the dam body since only one processed record from the dam was available. For the same reason, it was not possible to perform a more comprehensive system identification of vibration properties.

For a better description of the response of the dam and canyon to an earthquake, modern digital accelerographs with ranges larger than those of the existing instruments should be redeployed. To help understand the effects of canyon topography on seismic response of the dam, it is recommended that additional free–field instruments be deployed in the canyon downstream of the dam. Canyon instruments would provide information about the free–field motion, without the effects of dam–foundation interaction, and improve the understanding of topographic effects on the free–field motion.

Recommendations for Seismic Analysis and Design

Seismic analysis and design of arch dams should be performed with proper attention given to the effects of canyon topography on the free–field motions. To the extent possible, an appropriate seismic wave type should be selected and the response of the canyon to the selected incident wave should be investigated. The effects of the spatial variation of the free–field motion must be accounted for in the analysis.

A nonlinear analysis of arch dams, including joint opening effects, should be performed for estimating the response to strong earthquake ground motion. Based on results of the present study and in view of the performance of Pacoima dam during the Northridge earthquake, a nonlinear analysis procedure considering opening–closing of contraction joints, lift joints, and dam–foundation interface only can be used. A complete nonlinear analysis including plasticity models may also be useful especially for very unusual cases, but validated analysis procedures are not available at this time.

The energy loss due to propagation of seismic waves into the foundation must be accounted for in the seismic analysis. To this end, an appropriate radiation damping should be developed and be included in the model.

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