UTILIZATION OF STRONG–MOTION DATA FROM BRIDGES AND DAMS

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ABSTRACT

Highway bridges and concrete dams are critical components of California's infrastructure. Because of the difficulty devising experiments to test the complete system response of bridges and dams in earthquakes, the design and safety evaluation of these large structures are primarily based on mathematical models and numerical simulations. Strong–motion data from bridges and dams provide the real–life laboratory for verifying the models. This paper summarizes the results from recent studies of the recorded strong–motion response data from bridges and a dam. Although the strong–motion data are limited, they have provided confirmation that models are capturing the essential features of earthquake response. Limitations of the models, however, are identified and future trends for instrumentation and data utilization for bridges and dams are discussed.

INTRODUCTION

Although very different structural systems, bridges and dams have several common characteristics. Both types of structures are critical components of the infrastructure for transportation and water resources, respectively, for which a high level of performance during an earthquake is expected by society. Considering the dynamic characteristics, interactions of the structures with the surrounding media, such as soil and water, play an important role in the earthquake response. They are large structures for which length scales of thousands of feet are common. The large size and multiple or continuous connections with the ground, over a varying topography and soil conditions, mean that the assumption of uniform free-field ground motion is seldom valid. Because of the large size of the bridges and dams, experimental testing of system performance in earthquakes presents profound difficulties with similitude, boundary conditions, and limited capacity of earthquake simulators. Consequently, mathematical modeling and numerical simulation of bridge and dams systems are the primary method for assessing earthquake performance and safety.

How accurate are the mathematical models of bridge and dam systems? One way that we can answer this question is by examining and interpreting their performance as recorded by strong–motion instrumentation during an earthquake. The recorded responses of structures in earthquakes provide the laboratory in which engineers can study characteristics, assess the validity of mathematical models, and improve understanding of earthquake performance. The instruments for the modern full-scale earthquake laboratories in the field are digital strong–motion accelerometers strategically located to capture input motion, global response characteristics, and important local response. The methodologies include direct interpretation of strong–motion records, system identification for vibration properties or system and component properties, and direct comparison numerical simulation results from models.

The goal of this paper is to examine retrospectively the use of strong–motion records for bridges and dams. The California Strong Motion Instrumentation Program (CSMIP) has instrumented bridges, and to a lesser extent dams, in addition to its basic mission and funding base for instrumenting buildings. This review will concentrate on results from bridges and one case of a concrete dam. Although numerous data utilization studies are referenced, previous studies in which the author has been involved are emphasized. The paper concludes with comments about needs for the instrumentation of bridges and dams and future trends.

INSTRUMENTATION OF BRIDGES

At the time of the 1989 Loma Prieta earthquake, CSMIP had instrumented six bridges, a segment of the BART elevated structure, and one tunnel. Three of the bridges were short with two to six spans (156/101 Separation in Hollister, Painter Street Overcrossing in Rio Dell, and Meloland Road Overcrossing in El Centro) and one was a ten span bridge retrofitted with seismic isolators (Seirra Point Overhead in South San Francisco). The other two pre–1989 instrumented bridges were the 6000 ft long Vincent Thomas suspension bridges and the 8600 ft long Dumbarton bridge with prestressed concrete and steel box girder construction. These two long bridges each had 26 accelerometers and limited free–field ground motion instruments. The Loma Prieta earthquake demonstrated the fragility of several bridge types. The only strong–motion records from bridges in Loma Prieta, as will be discussed later, were from the Sierra Point Overhead and the Dumbarton bridge. Because of the limited instrumentation a valuable opportunity to improve understanding about earthquake response of bridges was lost.

After the Loma Prieta earthquake the California Department of Transportation recognized the importance of installing strong–motion instrumentation in a variety of bridge types, and it began funding an expanded instrumentation program. Between 1989 and 1993, Caltrans and CSMIP installed strong–motion instrumentation in 14 additional bridges, including a limited number of channels (nine or less) on three toll bridges. From this era significant strong–motion records were obtained from curved connector bridges in the Landers, Big Bear, and Northridge earthquakes. Another important record was obtained at the Painter Street Overcrossing in the 1992 Petrolia earthquake, with a peak horizontal acceleration of 1.09 g.

The Caltrans–CSMIP instrumentation program accelerated after the 1994 Northridge earthquake. Through mid–1996, 24 additional bridges have been instrumented. The majority of the bridges are reinforced or prestressed concrete box girder bridges, the predominant construction type in California. Most instrumented bridges have six to twelve accelerometers, but two curved connectors have 30 to 36 channels. Two of the sites have downhole arrays with six accelerometers. The downhole arrays will provide invaluable information about the propagation of free–field motion through the site up to the bridge footings and ground surface, and provide opportunities for verifying soil models, which is an urgent need.

At this time, Caltrans plans to instrument seventeen additional freeway bridges and five downhole arrays. Also under planning are instrumentation networks for the toll bridges in conjunction with the toll bridge retrofit program. The Golden Gate Bridge District also plans to install a network as part of the seismic retrofit for that vital transportation link.

DATA UTILIZATION STUDIES FOR BRIDGES

Wilson (1984) examined the response of the first instrumented bridge in California (Route 156/101 Separation in Hollister) to the 1979 Coyote Lake earthquake. Later the two–span Meloland Road Overcrossing in the 1979 Imperial Valley earthquake was the subject of several studies (Werner, et al., 1987; Werner, et al., 1993). The Meloland studies have shown large damping ratios, in excess of 20%, for the embankment abutments. The study also showed the importance of using cracked section properties in the model for a valid comparison with the recorded response.

In the 1989 Loma Prieta earthquake, strong–motion data was recovered from the Dumbarton bridge. A study by Fenves, et al. (1992) obtained spectral estimates of vibrations properties and compared the recorded response with models using various assumptions. The results showed the sensitivity of the response to assumptions about the articulations at the hinges and also to the soil properties. Another examination of the Dumbarton bridge by McCallen (1992) using a refined finite element model made similar observations and noted the difficulty with fully reproducing the recorded response by numerical simulation.

A set of Loma Prieta records was obtained from the Sierra Point bridge. The heavily skewed, steel girder bridge was retrofit in 1985 with lead–rubber isolation bearings replacing older steel bearings. However, no seismic gap was provided for at the abutments, apparently because the primary objective of the isolators was to limit forces into the tops of the non–ductile columns, and the expectation that the abutment backwalls would fail during a large earthquake. A study of the records by Kelly, et al. (1991) showed the strains in the bearings did not exceed 4%, barely yielding the lead core, and the effective isolation vibration period was a short 0.75 sec. Although the stiff bearings provide little isolation at small shear strain, there was some reduction in short period energy transmitted from the superstructure and abutments to the columns below the isolators.

Another Loma Prieta study involved a section of the elevated BART structure in Hayward. Tseng, et al. (1992) examined the significant effects of soil–structure interaction on the response of the single column bents supporting the tracks.

The 1992 Petrolia earthquake produced peak free–field ground acceleration of 0.52 g at the Painter Street Overcrossing, and 1.09 g at one abutment of the structure. Records from eight other earthquakes and aftershocks have been recorded at the bridge, making it one of the most studied (Maroney, et al., 1990). McCallen and Romstad (1994) developed a very refined solid nonlinear, finite element model of the bridge, embankments, and abutments. The advanced simulation showed the importance of hysteretic energy dissipation in the embankment soils. Sweet and Morrill (1993) examined a simpler nonlinear model of the soil to explain the bridge response. Using a different approach, Goel and Chopra (1997) directly estimated the time–varying abutment stiffness and torsional response from the records caused by the skew and differences in transverse stiffness of the two abutments. They concluded that the Caltrans guidelines for computing abutment stiffness based on the capacity of the abutment divided by the assumed failure displacement of 2.4 in. were reasonable. The results also show significant hysteretic energy dissipation in the abutments, which is typically not recognized in bridge design.

A CASE STUDY: CURVED CONNECTOR BRIDGE

The Northwest Connector bridge at the Interstate 10/215 interchange in Colton, Calif., was the first bridge with single column bents to undergo an extensive column and footing retrofit. As part of the 1992 retrofit, Caltrans and CSMIP installed 34 channels of accelerometers, as shown in Fig. 1. This section summarizes a study by the author (Fenves and Desroches, 1994; Desroches and Fenves, 1997) and to a lesser extent a later study by Liu, et al. (1996).

Table 1 lists the strong–motion records obtained at the Northwest Connector. The response spectra for the free–field site for 1992 Landers and Big Bear earthquakes are shown in Fig. 2. The response spectra can be compared with the smooth design spectrum used by Caltrans for a deep alluvial site and PGA of 0.70 g. Also, shown in Fig. 2 is the smoothed elastic response spectrum reduced by a factor of four, a typical value to account for inelastic behavior of single column bents. For periods less than one second, the spectra for the recorded motion exceed the reduced spectrum. For longer periods the design spectrum mostly envelopes the spectra for the two earthquakes. The spectral ordinates for the Northridge earthquake are much less than for Landers and Big Bear at periods greater than 1.5 seconds. From system identification results described later, the important vibration modes of the bridge have periods between 1.0 and 1.7 sec. In this period range there is not a large difference between the reduced design spectrum and actual spectra, so it can be expected that the forces developed in the columns approached the nominal design strengths.

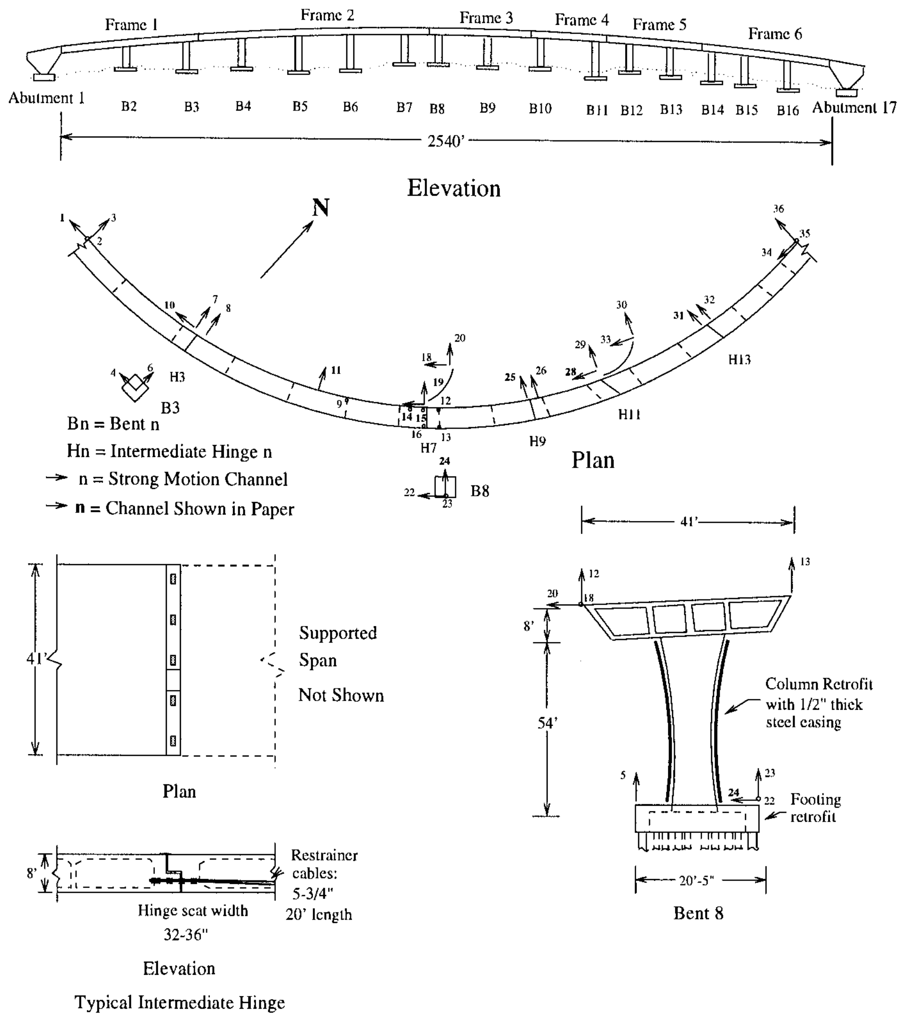

Fig. 1 General plan and instrumentation for the Northwest Connector at the Interstate 10/25 Interchange.

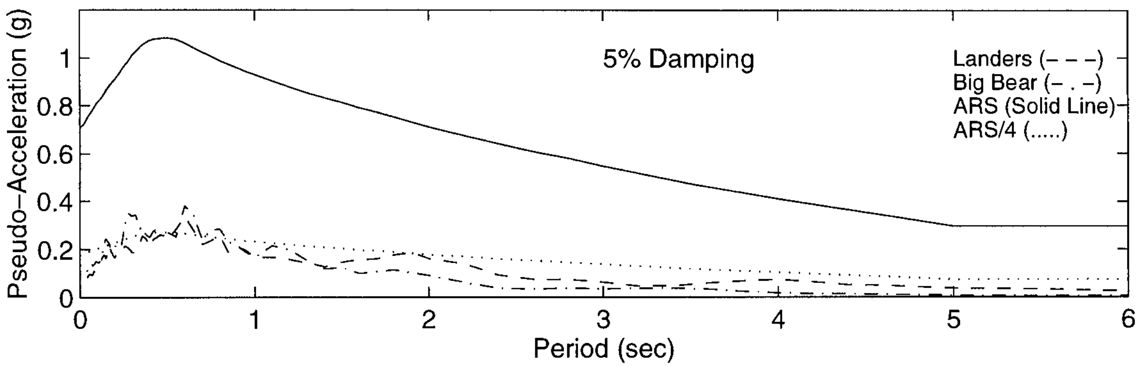

Fig. 2 Response spectra for Landers and Big Bear free–field ground motions (major axis) compared with Caltrans design spectrum.

Table 1. Earthquakes and Free–Field Ground Motion Recorded at the Northwest Connector

Event	Magnitude M_S	Epicentral Distance (miles)	Peak Acceleration (g) Horizontal[a]	Peak Acceleration (g) Vertical
1992 Joshua Tree[b]	6.3	52	—	—
1992 Landers	7.6	50	0.09	0.06
1992 Big Bear	6.6	28	0.11	0.07
1994 Northridge	6.8	72	0.08	0.04

[a]Instantaneous peak horizontal acceleration.
[b]Records not processed.

The instrumentation allows examination of the spatial variation of the input motion over the length of the bridge. Figure 3 shows the displacement time histories at the free–field site and at four supports for the bridge (two abutments, bents 3 and 8) in the Landers earthquake, and the maximum displacements are plotted in Fig. 4. The coherency functions for the pairs of input acceleration histories show a general loss of coherency for frequencies greater than 2 Hz, as typically expected.

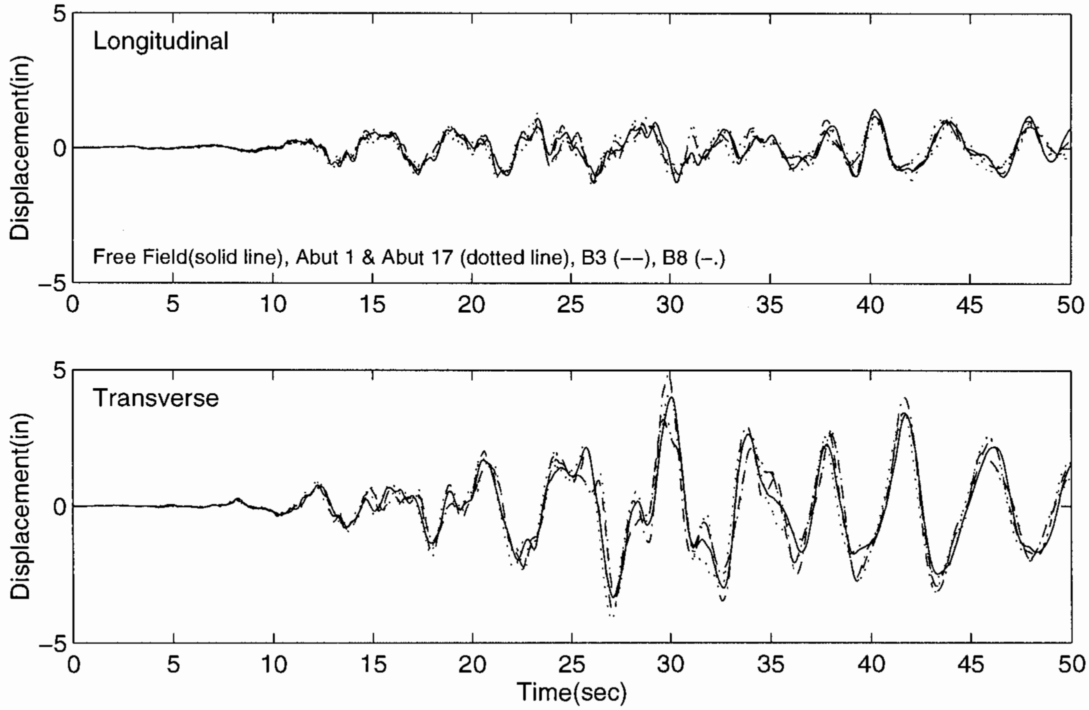

Fig. 3. Horizontal displacement at four support and free–field in global longitudinal and transverse directions for the Landers earthquake.

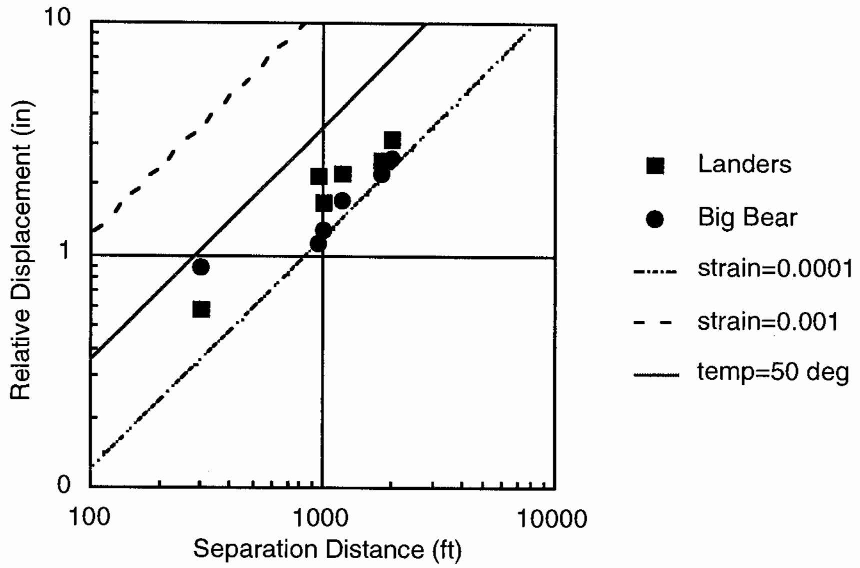

Fig. 4 Maximum relative displacement between supports as a function of distance between supports.

Evaluation of Earthquake Response

The instrumentation at bent 8 allows direct evaluation of the column deformation accounting for footing and superstructure rotation. The maximum column deformation for bent 8 near the center of bridge is 4.8 in. for Landers (0.86% drift) and 3.0 in. for Big Bear (0.54% drift). The Liu (1996) study reports bent 8 column deformations of 5.6 in. and 3.5 in. for the two earthquakes, respectively, although it is not clear if the superstructure rotation was accounted for in these calculations. Foundation flexibility through pile cap rotation are significant, contributing about 15% to the displacement of the superstructure. Based on analysis of a calibrated model and estimates of capacities, bent 8 reached 80% and 87% of the yield moment in the longitudinal and transverse direction, respectively, in the Landers earthquake. This is consistent with the post–earthquake inspection which did not reveal any evidence of cracking or other signs of deformation.

The thorough instrumentation in the bridge allowed evaluation of the opening and closing of intermediate hinges in the bridge, which are used to take temperature induced strains and simplify construction and post–tensioning. The hinge near bent 8 opens a maximum of 1.41 in. and 1.70 in. in Landers and Big Bear, respectively, which is less than the yield displacement of the cable hinge restrainers (and much less than the 32 in. hinge seat width). As a result, the cable restrainers had a small effect on the response of the bridge because of the small hinge opening. Malhotra, et al. (1994) also examined the hinge response, particularly looking at the acceleration spikes caused by pounding.

Two system identification techniques were used to identify the vibration properties of the Connector. The first method involved transmissibility functions were computed using cross–power spectral density estimators. The second method used parametric time domain identification (Safak, 1990). Table 2 summarizes the results of the identification. The periods from the two identification procedures are similar, except for the second mode for the Landers earthquake, although it is possible there are two closely spaced modes which are identified differently by the two methods. Liu (1997) study identified somewhat shorter periods of 1.67 sec, 1.47 sec, and 1.19 sec, although it is not clear for which earthquake these periods pertain.

Table 2. Identified Vibration Periods and Damping Ratios for the Northwest Connector

Mode	Landers Earthquake: Spectral Analysis Period (sec)	Landers Earthquake: Parametric Identification Period (sec)	Landers Earthquake: Parametric Identification Damping Ratio (%)	Big Bear Earthquake: Spectral Analysis Period (sec)	Big Bear Earthquake: Parametric Identification Period (sec)	Big Bear Earthquake: Parametric Identification Damping Ratio (%)
1	1.51	1.56	3.1	1.78	1.75	8.2
2	1.19	1.30	11.0	1.24	1.29	2.1
3	1.02	0.98	5.0	1.14	1.09	15.0
4	0.79	0.83	7.0	0.95	0.96	7.0

The most significant finding in Table 2 is the difference in the fundamental mode period of the bridge in the Landers and Big Bear earthquakes. Although there is little change in the second mode, the third and fourth mode period are long in the Big Bear earthquake. The change in the fundamental mode implies a 20% reduction in stiffness of the bridge, and the damping increases from 3.1% to 8.2%. The lengthening of the vibration periods and generally increased damping indicate that the bridge softened during the Landers earthquake. Since the column forces were less than the yield strength, the increased flexibility is due to column cracking and softening of the pile footings and soil. The identified results show the footing rotational stiffness was smaller in the Big Bear earthquake than in Landers, possibly due to compaction and gapping of soil around the piles or a change in the ground water table between the two earthquakes. The Liu (1996) study tracked the decrease in bent 8 lateral stiffness as a function of displacement and only a slight increase in flexibility due to soil–structure interaction in Big Bear.

Verification of Mathematical Models

The major objective of the Fenves and Desroches study (1994) was to examine the efficacy of typical lumped–parameter models used in bridge design. Since the structural components did not experience inelastic deformation, it was appropriate to use a linear elastic model. However, the nonlinear behavior of hinge opening and closing are included in the model with gap elements. Generally, the modeling followed guidelines in the recent ATC–32 report (ATC, 1996). The effective stiffness of the steel jacketed columns was selected to match identified vibration properties. The best fit model give an effective stiffness of $1.20EI_g$ and $0.95EI_g$ for the Landers and Big Bear earthquakes, respectively, for the columns with full–length steel jackets, where EI_g is based on the gross section properties. A factor of $1.20\ EI_g$ is consistent with a cracked stiffness of $0.75EI_g$ and an increase in stiffness from the steel jackets of 60%. The calibration represents the reduction in stiffness apparent in the lengthening of the vibration periods between the earthquakes.

The comparison of total displacements for Landers is shown in Fig. 5 assuming uniform free–field ground motion. Overall, the comparison is reasonable although some of the peak displacements are underestimated by the model, particularly during the strong–motion response. The model does not produce as accurate accelerations because of the greater participation of the high frequency modes, which are not represented well in the coarse lumped parameter model. The model captures the captures the acceleration spikes characteristic of the impact between adjacent frames as the hinges pound closed. Figure 5 shows that the model represents adequately the response of the hinge. In addition the model gives the propagation of acceleration pulses from impact at the hinge.

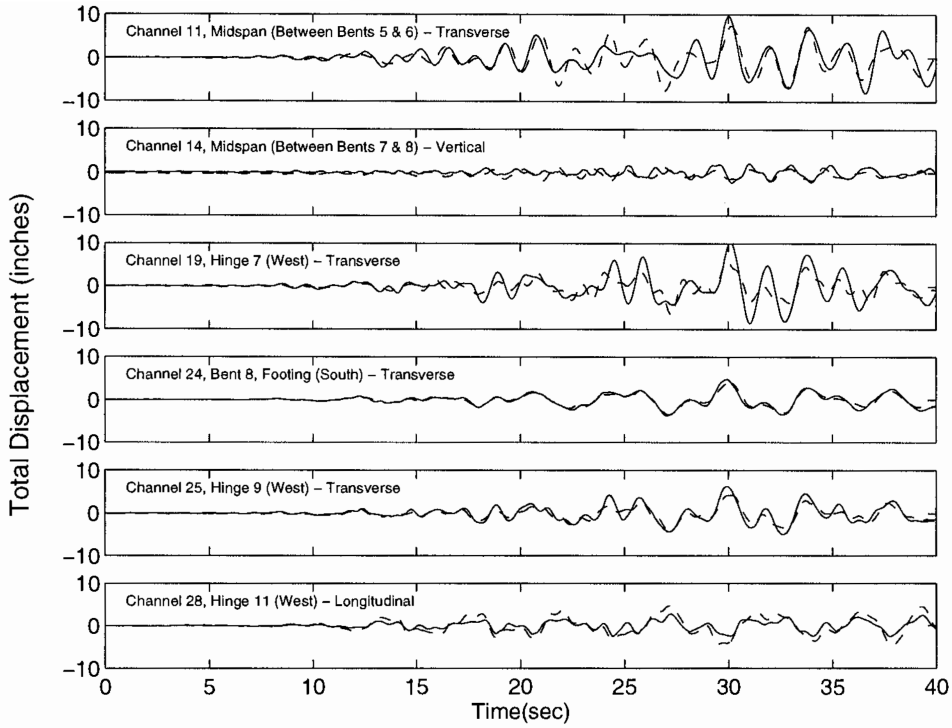

Fig. 5 Comparison of recorded displacement (solid line) with computed displacement (dashed line) using uniform free–field Landers earthquake ground motion.

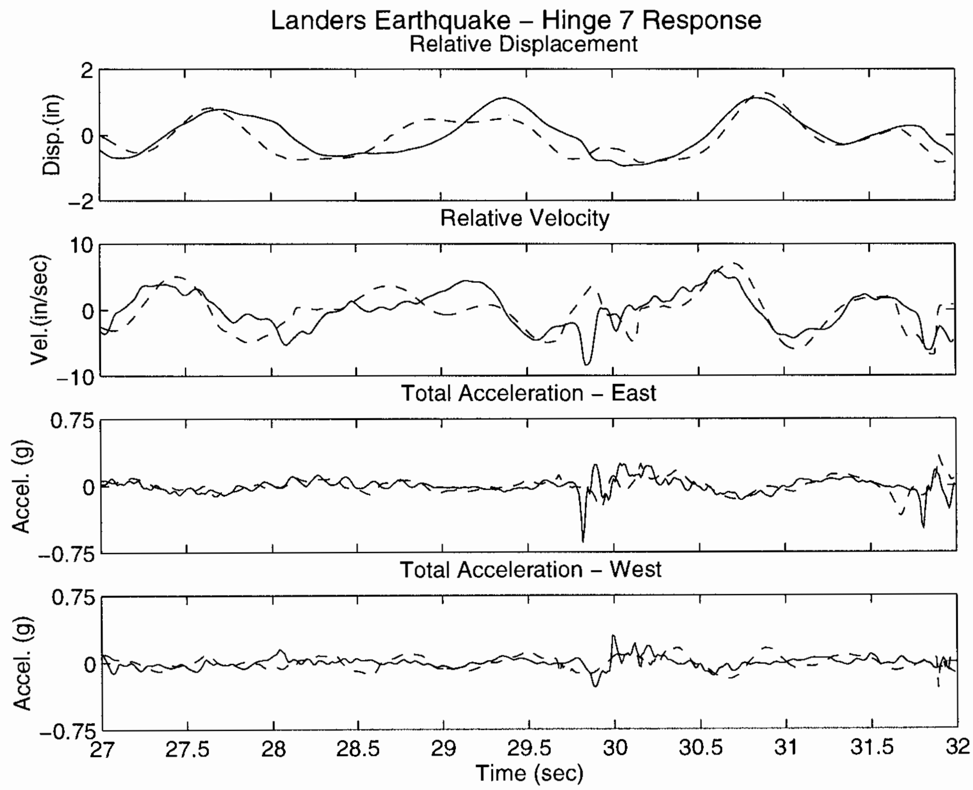

Fig. 6 Comparison of recorded longitudinal hinge motion (solid line) and response from model (dashed) for the Landers earthquake.

To examine the effect of non–uniform ground motion on the response of the bridge, the recorded displacements at the four supports were imposed using a simple interpolation scheme. Figure 7 shows the comparison for the Landers earthquake, which is a considerable improvement over the results in Fig. 5 for uniform free–field input motion. These results indicate that the apparently small differences in input displacement (see Fig. 3) have a fairly important effect on the displacement histories of the bridge. To examine the effect on column moments, Fig. 8 shows the non–uniform input motion has a more important effect on the transverse bending moment than on the longitudinal moment. Most of the difference is caused by dynamic response, because the pseudo–static forces are relatively small.

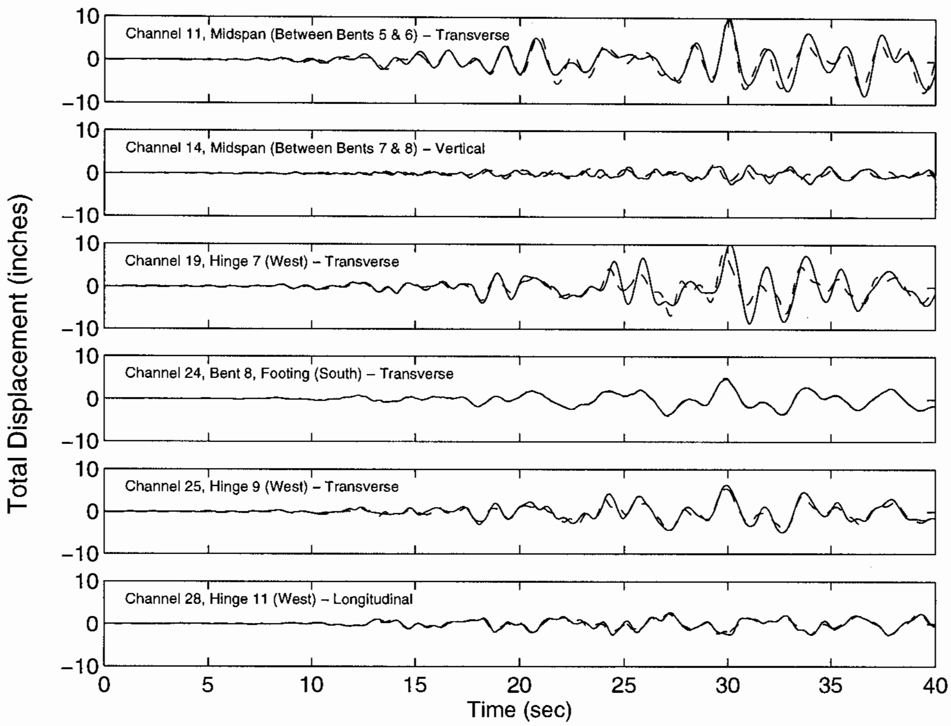

Fig. 7 Comparison of recorded displacement (solid line) with computed displacement (dashed line) using non–uniform input motion from the Landers earthquake.

Evaluation of Design Models

Bridge design is generally based on a linear global model to compute the displacement demands and elastic forces. To account for the effects of the hinge opening and closing, it is common to bound the response by two linear models: a tension model and compression model. The tension model essentially represents the hinges as open and compression model as hinges closed. Using the calibrated model, Fig. 9 plots the column moments for the design model with uniform free–field motion compared with the moments from the nonlinear model with non–uniform input motion. Considering the longitudinal response, the compression model gives larger moments than the tension model for the short columns, particularly at the end frames. The tension model gives larger moments for the taller, flexible columns. Generally the two models bound the nonlinear response within about 10%. However, in the transverse direction the nonlinear model produces up to 50% larger moments for five columns. This is primarily due to larger response for the non–uniform input motion compared with the uniform motion in the design models.

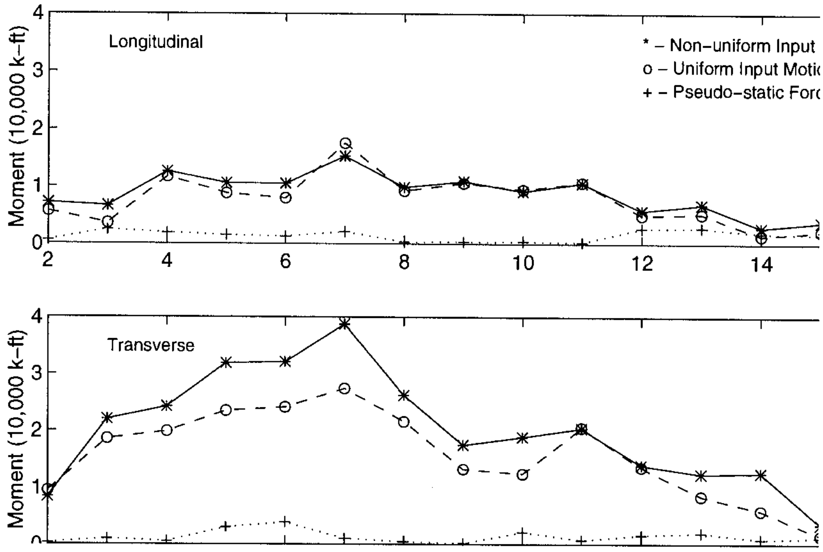

Fig. 8 Maximum column moments for uniform and non–uniform input motion in the Landers earthquake.

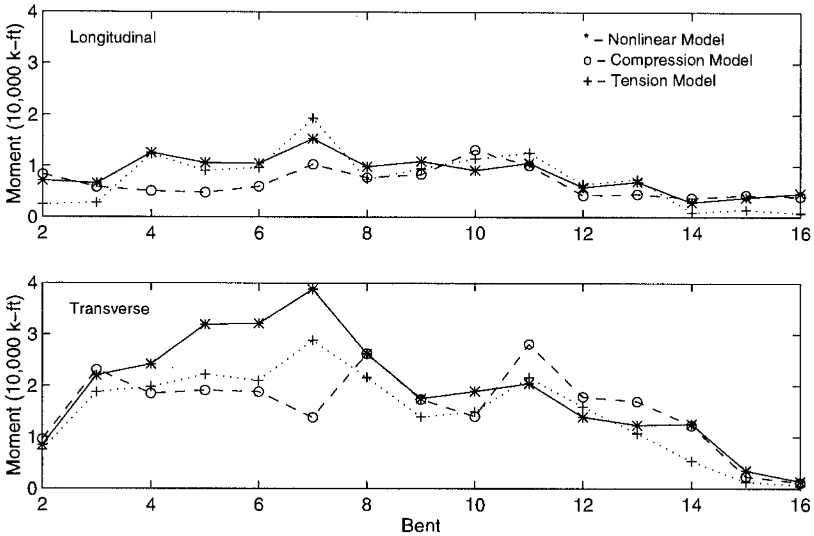

Fig. 9 Comparison of column moments from nonlinear model with moments from design models.

DATA UTILIZATION FOR CONCRETE DAMS

Compared with bridges, few dams have been instrumented with strong–motion accelerometers. Strong–motion records from several earth dams in the Loma Prieta and Northridge earthquakes are available and have been studied by geotechnical engineers. However, there is only one set of strong–motion records for a concrete dam. This section summarizes the results of a recent study of Pacoima dam in the 1994 Northridge earthquake (Mojtahedi and Fenves, 1996).

Pacoima dam is illustrated in Fig. 10. It is a 365 ft tall, 589 ft long flood control arch dam located 4.5 miles northeast of San Fernando, California. Uniformly spaced contraction joints with 12 in. deep beveled keys divide the dam into eleven cantilevers. The left abutment is supported by a concrete thrust block through a 60 ft tall joint. The dam was subjected to severe shaking in the 1994 Northridge earthquake, with the epicenter 11 miles from the dam. The peak accelerations of 1.6 g and 1.2 g, at the left abutment in the horizontal and vertical directions, respectively, are among the largest ever measured during an earthquake. The reservoir level was 233 ft above the base (about two–thirds full including the silt).

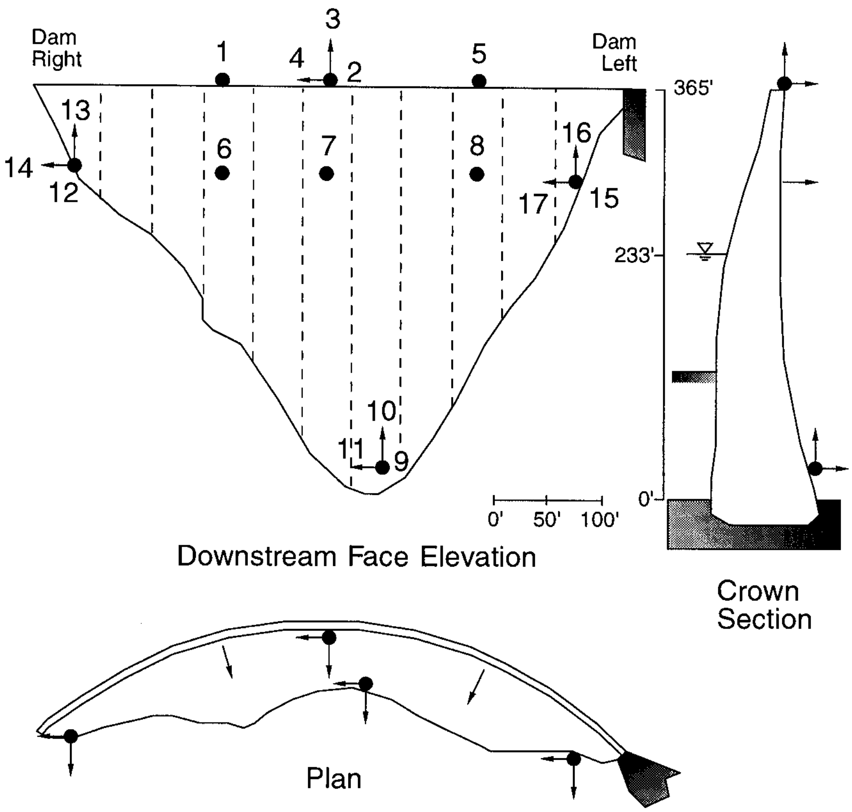

Fig. 10 Pacoima dam and strong–motion instrumentation plan.

Over–stressing of the dam occurred during the earthquake, as indicated by cracks and permanent movements of the concrete blocks. Most of the damage, however, can be attributed to the movement of the thrust block due to a failure in the supporting foundation rock. The contraction joint between the dam and the thrust block opened and remained open after the earthquake: about 2 in. at the crest and 1/4 in. at the bottom. Some permanent differential movement in the vertical direction also occurred at the joint, with the thrust block lower with respect to the dam. A diagonal crack occurred in the thrust block near the bottom of the thrust block joint. The dam body suffered less damage, with minor cracking permanent offsets of some blocks. There was evidence that the vertical contraction joints opened because of their subsequent clean appearance. This was the first evidence of contraction joint opening in a concrete dam during an earthquake.

The strong–motion records provided a unique opportunity to examine the contraction joint behavior in an arch dam subjected to a large earthquake. The locations of the accelerometers on the dam are shown in Fig. 10. Several of the records could not be digitized completely because large acceleration peaks, which exceeded the range of instruments, were intertwined on the film. Processed records are available for two locations in the canyon, one at the downstream and the other 50 ft above the left abutment, and channels 8 to 11 on the dam. The peak accelerations recorded in the canyon were 0.43 g and 1.58 g for the downstream and the upper left abutment (ULA) instruments, respectively, indicating the amplification of ground motion by the canyon topography. The stream component of these motions are shown in Fig. 11. Channel 8 recorded the radial acceleration, with a peak of 1.31 g, at the left quarter point at 80% height of the dam. Figure 12 shows the radial accelerations of the dam at three different elevations, the base, 80% height and the crest. The partially digitized but unprocessed acceleration records from the crest of crown section are plotted in Fig. 13,

Development of the Model

The finite element computer program ADAP–88 (Fenves, Mojtahedi, et al., 1989; Fenves, Mojtahedi, et al., 1992) was used for the earthquake analysis of the dam. Nonlinear joint elements simulate opening–closing of the contraction joints, lift joints, and the dam–foundation interface. Spatially non–uniform seismic input, can be specified through displacement histories at the dam–foundation interface (Mojtahedi and Tseng, 1994).

Two types of earthquake analysis were performed. The first type used a uniform free–field motion, whereas non–uniform free–field motion was considered in the second type. The free–field motions for both cases were derived from the motions recorded in the canyon during the Northridge earthquake. For determining the non–uniform free–field motion, it was not possible to separate dam–foundation interaction effects from the spatial variation of the canyon motion. Hence, those dam–foundation effects were neglected and the recorded motion at the interface was assumed to be the free–field motion. Due to the profound difficulties in determining the motion of canyon and also lack of sufficient acceleration records from the earthquake, a simple approach was adopted for specification of the non–uniform free–field motion. The same motion was specified for the right and the left abutments. The ULA and the dam base records were specified for the crest and the base of the dam, respectively. The motion at intermediate elevations was computed by linear interpolation from these records, as illustrated in Fig. 14.

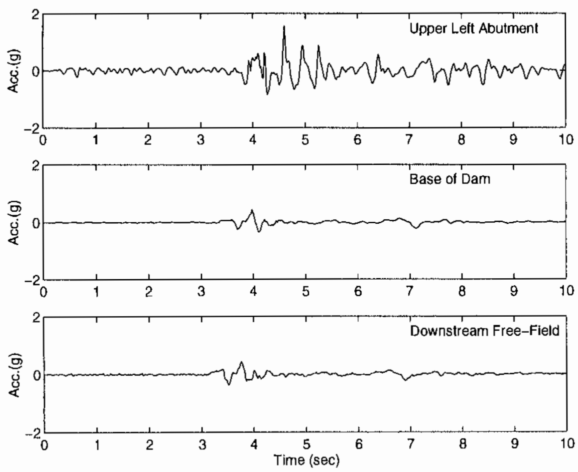

Fig. 11 Recorded acceleration histories for the canyon of Pacoima dam.

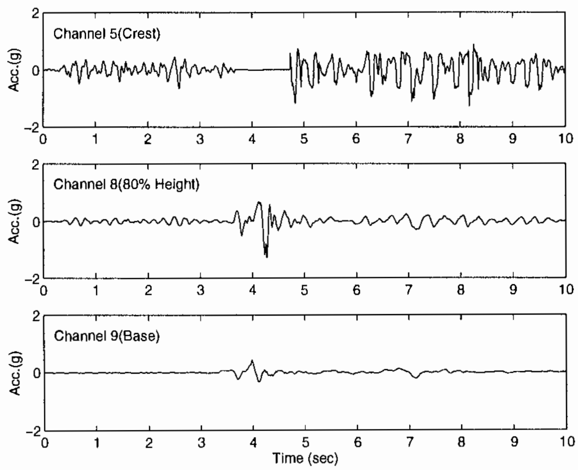

Fig. 12 Recorded radial acceleration history of dam. Zero acceleration shown for gaps in digitization of Channel 5.

For the finite element analysis, a total of 588 eight–node 3–D elements model the dam body. The number of contraction joints and lift joints in the dam model was selected from trial analyses.

Of the twelve vertical joints in the dam, only five were included in the model. In addition, five horizontal joints, at 50, 97, 135, 202, and 282 ft above the base, were included to represent lift joints. A foundation rock region with a depth approximately equal to the height of the dam was included in the model to account for dam–foundation interaction effects. Although the foundation rock geometry and material properties are complicated, a prismatic shape was assumed for the canyon using a coarse mesh of the foundation rock region with 220 3–D solid elements. To suppress the propagation of seismic waves, the foundation rock was assumed to be massless. Rayleigh damping was assumed for the dam–foundation system with parameters selected to produce 10% damping at 5 Hz and 20 Hz. The assumed damping is relatively high, but is justified considering that radiation damping in the massless foundation is not explicitly included in the model. For analysis with uniform free–field motion, the acceleration was specified at the rigid base of the foundation model. For analyses with non–uniform free–field motion, the displacement was specified at the nodes on the dam–foundation interface.

The first natural frequency computed for the dam using these properties, with the 233 ft reservoir level, was 4.3 Hz with an anti–symmetric mode shape. To confirm the selected properties, the transmissibility function was computed for the radial motions recorded at the base and channel 8. The fundamental frequency of the dam from the transmissibility function was 4.0 Hz which, considering the limited data, is in reasonable agreement with the model frequency.

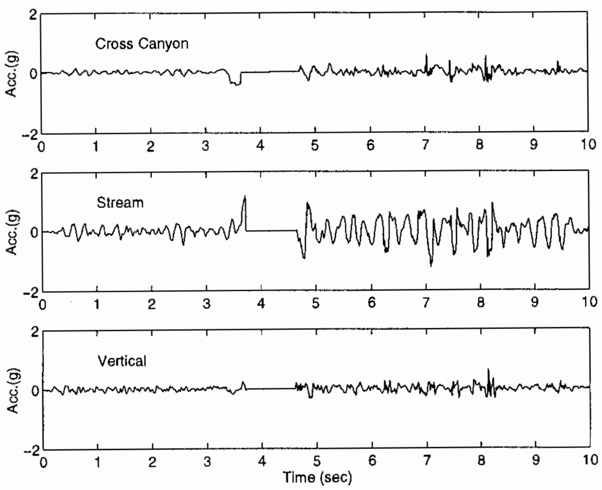

Fig. 13 Unprocessed acceleration histories at crest (channels 2–4). Zero acceleration shown for gaps in digitizing.

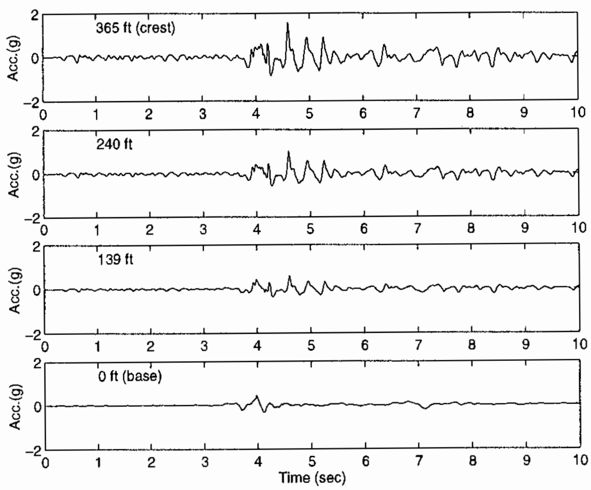

Fig. 14 Assumed non–uniform acceleration histories at the dam–canyon interface in stream direction.

Evaluation of the Model

The amplification of seismic waves in the canyon was indicated by variation in the acceleration amplitudes recorded at the downstream, base, and upper left abutment (Fig. 11). To assess the effects of the ground motion input on the response of the dam, two cases were analyzed with all joints closed. Following previous studies of the dam (Dowling and Hall, 1989), two–thirds of the ULA motion was considered as the average motion of the canyon and was specified as uniform input motion for the first case. The previously described non–uniform free–field motion was used for the second case.

The recorded and computed accelerations for channel 8 are shown in Fig. 15. The accelerations computed for the two closed joint cases differ in amplitude and phase. The case with non–uniform input agrees better with the recorded acceleration than does the uniform input case. However, the responses computed for both cases contain a large amplitude 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 model. The sliding of the foundation rock mass near the left abutment, not accounted for in the model, may also be responsible for the discrepancy.

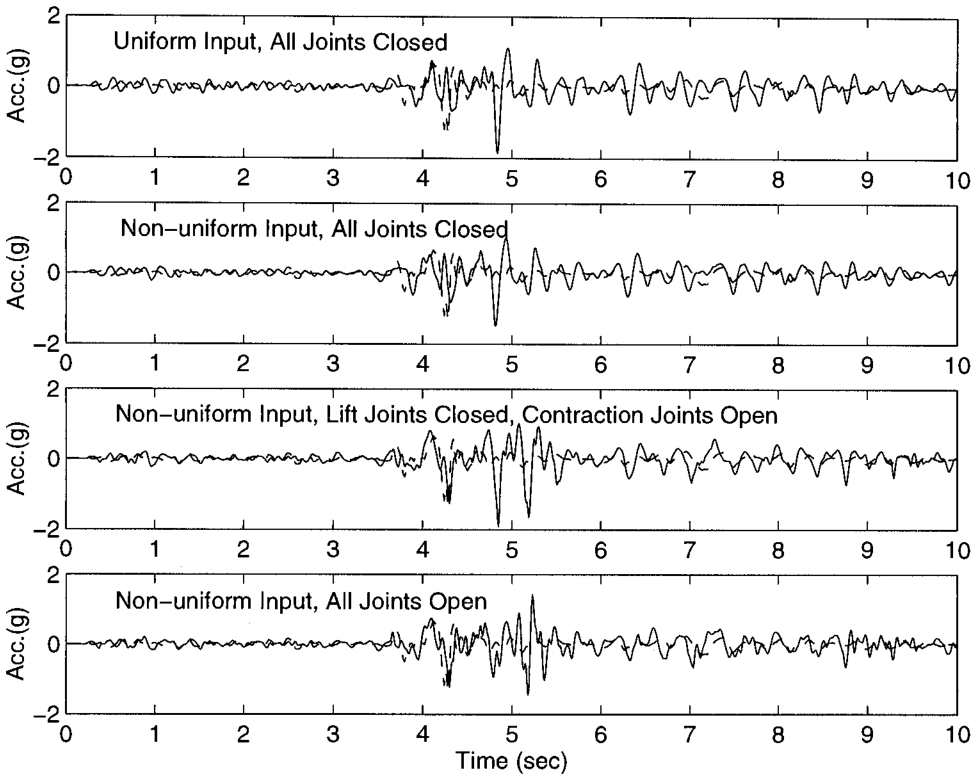

Fig. 15 Comparison of recorded acceleration for channel 8 (dashed line) with computed acceleration (solid line) at the same location.

To examine the effects of contraction joints, as well as opening at the dam–foundation interface due to tensile stresses, the analysis with non–uniform input motion was repeated with these joints allowed to open. From the channel 8 acceleration comparison in Fig. 15, the agreement between computed and recorded acceleration improves significantly when the joints are allowed to open. However, the computed response again contains spikes not present in the recorded motion for the reasons mentioned previously. The cantilever stresses computed for the case with contraction joint opening exceed the tensile strength of concrete and the lift joints. To account for the opening of lift joints, the dam was analyzed with all vertical, horizontal and abutment joints allowed to open, and considering the non–uniform input motion. In view of the permanent opening of the thrust block joint, the tangential stiffness of the joint was omitted. The last history in Fig. 15 for non–uniform input and all joints open shows the best comparison between the computed and recorded acceleration at channel 8.

Mojtahedi and Fenves (1996) present contours of peak tensile stresses from the various models. The maximum cantilever stress of downstream face reduces to 1000 psi from 1800 psi for lift joints assumed closed. Significant opening occurs at the top lift joint with a maximum value of

2.6 in. at the base of thrust block on the downstream face. The opening for all other locations is less than 0.5 in. The simulations show complete separation through the thickness over a short length occurs at a several locations.

While this study has shown that the non–uniform canyon motion affected the response of Pacoima dam, and the opening of contraction joints was confirmed by the model and simulation, the lack of processed strong–motion data limited the quality of information from the Northridge earthquake. Although further attempts at digitizing and processing the film records may be futile, there is an important need for a thorough study of the records and deconvolution analysis to identify the free–field motion in the canyon.

FUTURE NEEDS

Strong–motion instrumentation of structures always involves a tradeoff between the number of instruments (i.e. the cost) and the richness of the data for one structure or over an expected epicentral region. As with most endeavors, cost is the controlling factor. However, the costs of not recording data in a major earthquake are rarely quantified. Increasing the instrumentation program for bridges and dams will require well–reasoned arguments that the current costs are justified by the future costs avoided.

There has been tremendous progress for bridges in the past four years. The next northern or southern California urban earthquake should produce information about the response of bridges including the effects of structural yielding and near field effects. The performance of a retrofitted bridge or a post–1990 designed bridge in a near field event has not yet been recorded. The collection of such data will provide essential information to improve understanding of system response in the nonlinear range. The plans for instrumenting the toll bridges is a welcome development. However, the relatively light instrumentation on large flexible bridges will not provide a complete picture of system response. Careful studies on how to deploy instruments optimally should be undertaken before finalizing the instrumentation plans for the toll bridges.

The studies of strong–motion records from bridges have shown the importance of capturing the non–uniform free–field and input motions at multiple supports. The recent deployment of downhole arrays will help in understanding the propagation and reflection of earthquake waves to the surface and the distribution of input motion over footings and kinematic soil–structure interaction effects. Although inertial soil–structure interaction has always been considered in the design and analysis of bridges, the recent studies are increasing are appreciation of the importance of interaction and recognizing nonlinearities of the footings and abutments. The hysteretic behavior of footings and abutments may be providing significant energy dissipation that is not recognized when assessing the inelastic demands on columns. Although there have been limited studies for abutments, there has not been a similar study for footings because it requires a dense array. Critical bridges on soft soils should be provided instrumentation at the footings to capture nonlinear soil–structure interaction effects. This should be done at sites with downhole arrays.

The instrumentation of concrete dams is inadequate considering the importance of dams for water supply and the strict review of dams required by the State. Unlike bridges, for which the owner (Caltrans) funds the instrumentation, there is no requirement or incentive for dam owners to instrument their dams. Instrumentation for Pacoima dam should be upgraded with a modern digital system. It is strongly recommended that the canyon downstream of the dam be instrumented to capture the free–field motion of the canyon without the effects of the dam.

Collecting strong–motion data is only the first step. System identification for linear, time invariant systems is fairly mature. System identification for nonlinear systems has primarily considered simple systems with simple nonlinearities. However, improvements are needed for identifying the behavior of large structures such as bridges as they enter the nonlinear range. Identification procedures that provide changes in mode shapes, vibration periods, and damping over time are useful for understanding global changes in behavior. However, the time–varying procedures do not provide information about the underlying nonlinear component behavior that is leading to the change in vibration properties. There is a critical need to validate the nonlinear models that are beginning to be used in bridge design.

A final comment is that particularly for bridges, we are often more interested in deformation, that is displacements between two points, rather than forces (i.e. acceleration). Deformations can be determined from double integrating differences of accelerations but these requires enough accelerometers and entails error because of random noise and other factors. Direct dynamic measurement of deformation and even strain would be very useful for validating models. Rugged and reliable displacement instruments with a wide dynamic range, possibly using fiber optics or lasers, should be developed.

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