SEISMIC RESPONSE OF MULTI-SIMPLE SPAN HIGHWAY BRIDGES

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SYNOPSIS

Multiple single span bridges have become very popular on freeways and turnpikes. The independence of the spans has many design advantages, but they can have a higher seismic risk because of lack of continuity in the longitudinal direction. The study focused on modeling techniques using available program features on ICES-STRUDL-II. Seismic loads were simulated using undamped sinusoidal forcing functions and checked using an El Centrol earthquake package. A three span bridge structure was modeled as a fixed support and roller support and the techniques and results compared. The latter support was developed such that traveling wave effects could be determined on long bridges. A welded rocker type connection was modeled as a multi-stage linear structure. Computed quantities included resonant frequencies, mode shapes and motions of various structural components. Results indicated that bridges that have high rocker type supports have relatively low natural periods and can be shown to be susceptible to failures by impacting between adjacent spans or excessive relative motion between span ends and pier caps.

RESUME

Les ponts à plusieurs travées avec poutres simplement appuyées sont fréquemment utilisés sur les autoroutes. Le fait que toutes les travées soient indépendantes présente des avantages du point de vue calcul mais, à cause du manque de continuité, les risques de dommages lors d'un séisme sont plus élevés. Cette étude porte principalement sur les techniques de schématisation du comportement de la structure en utilisant certaines particularités du programme ICES-STRUDL-II. On a simulé les charges sismiques en utilisant une excitation sinusoidale forcée non amortie et on a vérifié les charges avec le tremblement de terre d'El Centro. On a étudié le comportement d'un pont à trois travées avec appuis fixes et appuis mobiles sur rouleaux. Les quantités calculées comprennent les fréquences de résonnance, les modes propres et les déplacements des diverses composantes de la structure. Les résultats montrent que les ponts avec portées simples et appuis sur rouleaux ont des fréquences propres relativement faibles et qu'ils sont sensibles à la rupture par collision entre les travées adjacentes au cours d'un séisme ou par des déplacements relatifs trop importants du bout des travées par rapport aux têtes de piliers.

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INTRODUCTION

The 1971 earthquake in San Fernando, California brought to focus the need to study the responses of bridges under seismic loadings. This earthquake brought out the susceptibility of bridges to failure by vibration (1). A major conclusion reached upon examination of the damaged bridges after that earthquake was that deficiencies in structural details, especially at hinged and rocker connections, played a major role in all failures. Collapse of the simply supported bridges was initiated by bridge spans falling off of their supports at the abutments and piers due to large displacements of the spans relative to their supports. Two major factors are involved in these types of failures. The first is created by the inertial forces developed in the structure as the ground moves relative to the bridge superstructure. Due to the fact that the connections within the structure are not infinitely stiff then, restoring forces are developed in the members. These restoring forces tend to return the structure to its static equilibrium position and as a result structural vibrations are induced. As the structural members deform, internal friction forces can dissipate this induced energy. This energy dissipation, or structural damping, is directly proportional to the magnitude of the elastic forces in the structure. The vibratory response of a structure is dependent upon the predominant period of the ground motion produced by an earthquake. If the predominant period of the ground motion is significantly less than the structures fundamental periods, the mass of the structure tends to move with the ground. In this case, the member deformations remain small. However, if the predominant period of the ground motion is equal to the structure's fundamental period, there occurs a condition known as "resonance,". During resonance the member deformations reach a maximum.

The second major factor that comes into play is the interaction between the soil and the foundation during an earthquake. Vibratory consolidation of soils can lead to settlement and tilting of the structure's foundation. Resulting structural deformations may be great enough to cause collapse. Dynamic soil pressures in excess of design pressures can lead to significant deformations of the substructures. Soil liquefaction occurs when a shock or vibratory load reduces the volume because of a loss in shear strength. The resulting increase in pore pressure transforms the soil into a concentrated suspension of soil particles which are not capable of carrying a load.

These two major influences on bridge failures have been shown to be prominent for structures that have multiple-simple spans. These have been reported in bridge failures in Japan, Alaska, and of course, San Fernando (2, 3, 4). In some of the failures in Japan, it was noted that the abutments moved towards the center of the bridge and tilted towards their backfills. Intermediate piers tilted considerably and fixed supports on piers were clearly dislodged. Some roller supports on these bridges failed allowing the spans to completely fall. In other cases, spans did not collapse but were twisted and permanently deformed. In the Alaskan earthquake, large substructure displacements in the longitudinal direction were noted. The abutments displaced toward the center of the bridges pushing some deck sections over the pile bents and caused some piles to penetrate the bridge decks. Other deck sections fell to the stream bed when the joints separated. Some concrete T-beam bridges with reinforced concrete decks suffered cracking and crushing at their ends. Typical damage at the supports of Alaskan bridges included bent and sheared anchor bolts, tilted and displaced rocker supports, and crushed concrete at points of bearing. Deck displacements were largely longitudinal, with displacements being greater for bridges on timber pile bents. Little evidence of transverse displacement was noted.

In the San Fernando earthquake, a number of freeway type bridge structures were severely damaged. Simple spans in many cases were supported by bearing supports on narrow ledges. Large displacements occurred between girders and abutments and these dislodged steel rocker bars from their supports. One of the major points brought out by the studies on the bridge collapses after the San Fernando earthquake was that span discontinuities in a bridge were very detrimental to its ability to handle seismic loads.

The ability to classify the relative susceptibility of bridges to collapse was clearly needed and a study was initiated (5). The objective of the study was to review and summarize the state-of-the-art pertaining to the seismic damage of highway bridges, and based upon this limited information develop a simple classification system that can be applied to bridges particularly on the Interstate system. The state-of-the-art survey included a review of reported damages to bridge structures. Next a review of analytical investigations, case studies, and experimental investigations were made in order to discern the various parametric influences that dominated. From this a seven factor classification system evolved. The major classification parameters were: span type, pier type, pier height, support details, bridge skew, bridge curvature, and foundation type. From this classification system it was found that a multi-simple span bridge, with piers formed by single columns that were quite high and that had expansion rockers for supporting connections were factors that lead to high susceptibility to seismic collapse. Further it was noted that piers supported on spread footings appeared to be the most susceptible.

The State of New Mexico has a large number of simple supported bridges. When most of these bridges were designed and built, the design was based on the AASHO (American Association of State Highway Officials) code which consisted of applying a horizontal load to the bridge based on a percentage of the dead load of the structure (6). The inadequacy of this code was illustrated by the failure of the bridges particularly during the San Fernando earthquake. AASHTO (American Association of State Highway and Transportation Officials) has since updated its seismic load specification to consider the dynamic properties of bridges (7).

The bridges that were designed under the old code are still being used and will be for years to come. New Mexico is potentially a seismically active state and it has a number of multi-simple span highway bridges (8). It was felt that an initial effort should be directed towards the modeling of existing structures and of evaluating potential seismic damages. ICES-STRUDL-II is a computer based structural analysis language that has many applications appropriate for dynamic investigations (9). Complete structures can be modeled with data input. STRUDL has the ability to accept any nonlinear loading in digital form, but this requires digitizing of these records. Sine waves and the El Centro earthquake record are available as programmed loading functions.

The review of the damages to the multi-simple span bridges indicated that longitudinal seismic waves appeared to cause more damage than transverse. This suggests that this should be the primary emphasis for an analysis of multi-simple span bridges. STRUDL has the capabilities for inputting support accelerations, however, all supports must be accelerated simultaneously with this package. The possibility exists that a traveling seismic wave could have frequency characteristics such that abutments on the opposite ends of a long bridge could be vibrating in opposite directions and these displacements would significantly increase the susceptibility to failure. Therefore, it was decided that a phase of the study should develop the capability of sequencing of support motions such that the progressive impact of a traveling longitudinal wave could be evaluated. This was not directly available on STRUDL, therefore a technique was created that allowed the independent motion of each support such that the influence of a traveling wave could be evaluated. This was achieved by creating a roller support structural model in the computer. In this case a structure was suspended in space and the supports were assigned sufficient mass such that the structure with the roller supports displayed nearly identical vibrational characteristics as the fixed support structure generated in the STRUDL package.

It was decided that methods should be developed whereby existing programs could be used and the program should be designed such that a bridge seismic susceptibilities could be evaluated with available and relatively inexpensive techniques. Dynamic loading functions available on STRUDL were available and used in the analysis. The basic structural effects could be determined by developing earthquake simulation techniques based on programmed loading functions available with the language. Sinusoidal loading functions were available and it was decided to simulate dynamic responses by inputting sinusoidal loadings having a constant amplitude and duration and then repeat this loading for different frequencies covering the spectrum of dominant frequencies normally associated with earthquakes. The simulation technique was to be verified by comparing the simulated loading structural responses obtained with the El Centro package available on STRUDL.

Thus the two major objectives of this study are formed. The first is in developing inexpensive modeling techniques for use on existing STRUDL-II programs such that multi-simple span bridges can be investigated. This means that the nonlinear response of the welded rocker connection must be modeled for response to longitudinal motion. Next the basic structure should be modeled such that the fixed and roller support aspects can be described and the results verified. Finally the seismic loadings are to be simulated with a variable frequency sinusoidal loading functions and the results verified with an actual earthquake record. The second objective is to apply these modeling techniques to a bridge that has a high susceptibility to seismic motion and analyze the results.

In order to exercise the model and perform the objectives of the study, a bridge was chosen that is located on Interstate 25 in Las Cruces, New Mexico. The bridge is a 3 span, simply supported bridge whose connections are of the flexible roller type design. These are shown in a photograph of the bridge in Figure 1. The girders are prestressed concrete with a composite-concrete deck. The overall length of the bridge is 131 feet, and the total width is 41 feet. The reinforced concrete deck rests on five prestressed concrete girders spaced 8.625 feet apart. Table 1 lists the pertinent structural properties. Both the abutments and the piers are perpendicular to the longitudinal axis of the bridge. Each abutment is founded on ten treated timber piles. The three columns, each 22.8 feet long, are supported on spread footings. The longitudinal girders of the two end spans are 31.5 feet long and 36 inches deep and the 68 foot center span is 45 inches deep. Concrete diaphragms are located between the girders. The deck is comprised of a $7\frac{1}{2}$ inch reinforced concrete slab. Composite action of the deck and girders is developed by shear connectors. Figure 2 shows the details of the connections. This bridge has many of the characteristics deemed undesirable in the seismic susceptibility classification study. Thus this effort complements the less quantitative approach.

MODELING

Earthquakes simulation--It was thought that the basic vibrational properties of this bridge could be analyzed if the simplified simulated earthquake loadings were applied to the structure. Sinusoidal waves were inputted without damping. Three variables had to be selected to describe the sine function. These were the amplitude, frequency and duration. The amplitude and duration were kept constant for all loadings and the frequencies were varied.

The basic criteria for the selection of the simulated loading was that the amplitude of the acceleration-time function should have a constant value of 0.09g. This is the value used for design specifications for Zone I according to AASHTO. New Mexico has both Zone I and II regions. The latter having a maximum amplitude of 0.22g. Preliminary analyses indicated that the lower value produced significant dynamic responses and this was the only one used. A review of a limited number of earthquake records indicated that their durations could vary from 0 to over 50 seconds. Many earthquakes show high intensity motion for a period of approximately 10 seconds and this value was selected as the duration for all simulated inputs. The last variable was not held constant because it was desirable to sweep a number of frequencies known to be significant in seismic studies. Frequencies were varied from 0.4 to 5.0 Hertz for horizontal motion simulations and from 1.0 to 25 Hertz for vertical motion simulations. The latter were selected over a higher range based on available records (10).

One of the secondary objectives of this study was to develop the capability to input a support displacement function. A support displacement function is more indicative of the actual support motion and is less sensitive to the high frequency effects found in acceleration-time records. A study of displacement-time records indicates that there is usually some dominant period of vibration for the records. Figure 3 shows the acceleration-time and displacement-time records for the base of the Kajima International Building for the San Fernando Earthquake (11). The displacement-time function shows a highly damped periodic motion with a period of approximately 7 seconds. Although this example is one of the better ones there still is the suggestion that earthquakes could be simulated with periodic functions particularly at the displacement-time level. Acceleration-time records for single degree of freedom systems are mathematically related to the displacement-time functions and it is reasoned that the second derivatives can be used to simulate acceleration-time earthquake records. In effect this is a filtering process for the higher frequency vibrations.

Loading functions using sine waves were used such that the displacement and acceleration time inputs were mathematically related such that they would represent the same input acceleration amplitudes, durations and frequencies. In this way the fixed and roller support models could be compared and the overall modeling process verified.

Bridge models--The model used to input this bridge into STRUDL is shown in Figure 4. This is a two dimensional model containing four separate structures identified by A, B, C and D. Joints were numbered as shown.

It is seen that the rocker type joints were modeled as rollers. The welder rocker connection, 5-8, for the middle span was modeled as a short cantilever beam that had stiffnesses assigned to it based on loading levels. Stiffnesses were assigned to five different stages of connection deformation (12). These stages are illustrated in Figure 5. A force-deflection diagram was created for this connection and then a stiffness value assigned to each stage by connecting the origin with the mid-point value for that particular stage. This procedure is similar to the use of the secant modulus of elasticity. For purposes of computations, the deflection stage of the connection was determined from a preliminary run on the computer and then a single stiffness was assigned to the cantilever beam for that run. The second run confirmed that the appropriate stiffness was selected for the loading conditions imposed. Obviously this is a critical element in evaluating longitudinal motion and can have a significant influence on the fundamental frequencies. The approach used would cause the frequencies to be on the high side. Later analyses will show that these approximations will not invalidate the overall evaluations because the placement of a relatively heavy girder on a relatively flexible connection creates geometry and mass distributions that dominate the dynamic analyses.

One of the modeling features was to use the lumped mass features of STRUDL. In a two dimensional system, each joint or lumped mass can have up to three different displacements. These are translation of motion in the x and y directions and rotation of motion about the z axis. It was found that the supporting columns and piers could be modeled as two mass structures and the girders needed to be modeled as five mass structures. The reason for the increased number of lumped masses for the horizontal girders was brought about in studying the effects of longitudinal support accelerations. It was found that STRUDL did not have sufficient flexibility such that horizontal accelerations could be inputted to a two mass structure where the masses were located at the supports.

During the period of structural evaluation and in the decision making processes for selecting the minimum number of lumped masses to be used, it was decided that the consistent mass features of STRUDL should be used. The consistent mass matrix simulates the mass being distributed along the length of the beam (13). This matrix is obtained by applying unit accelerations to each degree of freedom in succession while containing the others. The resulting inertial forces are used to make up the mass matrix. Unfortunately, the consistent mass matrix requires more computer time to run than the lumped mass matrix. In order to determine the validity the mass modeling procedures checks were made on the modes and magnitudes of the various vibrational characteristics. The structure was modeled using the lumped mass and consistent mass matrices and then theoretical modal shapes and resonant frequencies for structures A and C were computed. The results of these are compared in Table 2. The table clearly shows that the first two modal frequencies are similar with the maximum difference being 12%. Mode 3 for structure A is close to the theoretical for the lumped mass system and the consistent mass approach shows considerably higher values. The lumped mass approach was the main model used in the study since the approximations were satisfactory.

There was also the need to determine the modal shapes and resonant frequencies for the middle girder and the connecting pier that includes the welded rocker connection. A summary is shown in Table 3 along with the pertinent data from Table 2. The fundamental frequency of the center structure is considerably less than that of the other structures and this is expected due to the dependence on the welded rocker connection. It is noted that the connection is taken in its fourth stage of deformation for this analysis.

The next feature of the modeling was to model the supports. STRUDL has the capability to handle the masses and the supports if the structure is inputted with a support acceleration; however, all supports must be accelerated simultaneously. In order to determine the effect of the traveling wave it was necessary to free the supports. This makes the structure unstable. However, this can be handled in STRUDL as long as a total mass and geometry system is described in space. The releasing of the support adds a degree of freedom for each support release and creates an additional modal shape and resonant frequency. This action also changes the original modal shape and frequencies because of the changes in the mass distribution. For cases of support accelerations, the masses at the supports are ignored in STRUDL. For cases of support displacements, when the support is to be displaced by a forcing function, then an effective mass has to be created such that the desired displacement characteristics result.

The first step in developing the roller support model for released support displacements was to generate an effective mass for the support. This was initiated by having STRUDL print out the masses and stiffnesses that were being generated from the model parameters used in the support acceleration studies. In effect this established the basic reference model. Then an arbitrary mass was added to each support that had been released. This mass was selected as one billion times the existing mass, such that there would be sufficient inertia to represent that of a fixed support. This extra mass altered the mathematical characteristics of the support such that the modal shapes of the altered structure represented that of the original structure used in the support acceleration modes. The procedure was verified by comparing the modal shapes and fundamental frequencies as is shown in Table 4. It is noted that the comparisons for structure A are the same as those listed in Table 2. This is because the lumped mass model was used. The modeling for structure A is excellent and there are only slight differences in the fundamental frequencies for structure C.

The first step in checking the roller support model was to compare the modal shapes and frequencies to ensure that the basic structures were the same. The second step was to compare the dynamic responses of the two structures. For the fixed support model, a support acceleration could be used to induce the motion. For the roller support model a forcing function had to be applied to the support comprised of the effective mass such that resulting acceleration responses of the support would be the same as the fixed model. This was accomplished by making the mass of the support sufficiently high such that it dominated the calculations and then a force had to be applied such that this force divided by the effective mass would yield the desired support acceleration. The effectiveness of this technique was verified by comparing the maximum joint displacements that were computed using the two types of supports for a loading frequency of 0.8 Hertz. These are shown in Table 5 and it is seen that there are negligible differences. This technique of creating an equivalent dynamic model in space and then applying forcing functions designed to creat desired support accelerations opens up calculations for acceleration, velocity or displacement-time inputs even though the structure has many degrees of freedom. This is accomplished because the large mass at the supports allows single degree of freedom properties to dominate.

Considerable time went into the development of the roller support model to establish the technique. Since the structures used were linear and elastic and the fixed support and roller support models were developed such that they would have similar modal shapes and resonant frequencies, then the results should be duplicated and this was the case. The important factor is that this capability can be added to STRUDL and the effects of traveling waves investigated. For example, the velocity of wave propagation in shale is approximately 6000 ft/sec (14). A wave with a frequency of 3 Hertz has a wave length of 2000 ft. A long multiple-simple span bridge having a length greater than 1500 feet could be subjected to significant support displacements such that the vibrational aspects of the abutments could create situations where the abutments are moving in opposite directions at the same time and this could lead to separation of the girders from the bearing supports. Thus these techniques could be evaluated from the methods discussed.

The results show that a fixed model can be approximated with a free structure in space. With this capability the traveling wave phenomena can be applied to these structures with existing computer software. Because of the short length of the structure analyzed in this study, traveling wave results would be barely measurable and they are not included in further discussions.

BRIDGE ANALYSIS

Evaluation of Data

The dynamic responses of the bridge model due to the types of loadings described in the previous chapter were calculated using the STRUDL program. The primary responses that were determined included displacements of the span or girder ends with each other and with respect to the piers. Maximum moments in the columns were determined but are not reported. Table 6 shows a summary of the maximum theoretical relative displacements between adjacent span ends due to horizontal accelerations in the longitudinal direction. The values in the table are calculated assuming no contact between the spans, a condition which could be achieved if one of the abutments should fail. The nominal distance between adjacent spans is one inch. The table shows the theoretical responses for joints 2 and 8, which are at the left end of the middle span, and joints 9 and 14, which are at the right end of the same span. Data from a frequency sweep between 0.4 and 5.0 Hertz and the El Centro earthquake are shown.

Table 6 shows that the relative displacements at both ends of the middle span are similar and that the magnitudes become extremely large for frequencies between 0.6 and 0.8 Hertz, which is near the theoretical resonance. Relative deformations exceed 1 inch for frequencies between 0.4 and 1.25 Hertz indicating general sensitivity to motion with mean frequencies in this range. The relative responses of the El Centro earthquake showed large magnitudes and confirmed the hypothesis that a bridge of this design and construction can be susceptible to seismic loadings. The maximum El Centro magnitude is smaller than that obtained with the sinusoidal loadings for frequencies of 0.6 and 0.8 because they are near resonance and are gradually increasing with time in a linear system. If the duration of the loading were different the maximum reactive displacements would change.

Another comparison that is essential is that of the relative motion between the ends of the girder and that of the top of the pier cap. The distance from the center line of the rollers to the edge of the pier caps was 6 inches. Therefore it can be assumed that if the relative distance between the ends of the spans exceeded this amount, the girder ends might fall off of the piers. Table 7 shows a comparison of these theoretical relative displacements.

All conclusions should be tempered by the fact that the theoretical maximum displacement of the top of the right pier, joint 11, which acts as an independent structure was found to be less than 0.5 inches for all accelerations in this range, but it was over 6 inches for the El Centro earthquake. This is thought to be due to the fact that the dominant fundamental frequency for this portion of the structure was 2.7 Hertz and the sweeping frequency selected was 2.0. An analysis shows that the bridge structure has the potential for large pier displacements. A review of the maximum moments that would result at the base of the columns indicates that the columns could fail in flexure under the El Centro earthquake. Thus it is postulated that an intermediate pier that has only rocker supports between it and the girders could fail and cause a collapse of spans.

Other observations can be made in Table 7, assuming that there is no restraint at the abutments. The table shows that there would be deflections in excess of 6 inches at the left end of the center span for frequencies of 0.6 to 0.8 Hertz and for the El Centrol earthquake. It should be noted that the motion is dominated by the large mass of the middle girder and its connection to the pier cap. This is better evidenced in analyzing the relative motions at the right end. The relative displacements between joints 9 and 11 indicate the middle girder moving over the pier cap while the relatively small displacements between joints 11 and 14 indicate the independence of the pier and this large mass. The large relative motion between these two points due to the El Centro earthquake substantiates the earlier comments.

The sensitivity of the structures to the motions can be summarized with displacement-time plots for selected computer runs. Figure 6 shows a plot of the relative displacements of joints 2 and 8 due to the horizontal sinusoidal support accelerations for a frequency of 0.8 Hertz. The frequency at 0.8 Hertz was used to show the response near resonance. Figure 7 shows a similar response for a frequency of 1.25 Hertz, which is past resonance. Figure 8 shows a similar comparison for the El Centro loading.

Figure 6 shows that joint 2 remains relatively still while joint 8, which is the left end of the middle girder is exhibiting near resonance behavior. Figure 7 shows a somewhat different response, partly due to the changing of the vertical scales. For this frequency it is seen that the sinusoidal loading causes a response of the middle span that is somewhat periodic and that there is a coupling of different vibrational modes. Figure 8 is interesting because it shows that the El Centro earthquake shows periodic patterns not too unlike Figure 7 and magnitudes similar to Figure 6. It is seen that the maximum displacements for the El Centro earthquake are shown after a duration of 10 seconds, which was the longest used in this study. Presumably larger amplitudes could result at a later time. It is interesting to review the relative magnitudes of the simulated and actual earthquake motions. The sinusoidal loading has a maximum value of 0.09g while that of the El Centro is 0.33g. It is reasoned that the sinusoidal earthquake frequency sweep technique has merit and that it can be used in evaluating the seismic response of structures. Of course results would be improved with the addition of damping.

Figure 9 shows similar comparisons of the relative motion between the ends of the girders and the piers for a frequency of 1.25 Hertz. The 0.8 Hertz response was similar to Figure 6 and is not shown. Figure 9 shows that joints 4 and 8 are moving together except at the maximum amplitudes where relative motion is evident. Figure 10 shows similar quantities for the El Centro earthquake and it is seen that similar patterns result, although the relative displacements at the maximum amplitudes are greater. Figure 11 shows a similar comparison for the other end of the middle girder and its supporting pier. This figure shows that the girder end moves with the dominate frequency of the El Centro earthquake of the middle span and that the pier top vibrates at a higher frequency somewhat near its own resonance. The figure supports the previous observations.

Another aspect should be considered in discussing bridges of this type. It was noted that this three span bridge had four expansion joints each having a nominal clear distance of one inch less fixtures for sliding plates or elastomeric expansion pads. If this were a five span bridge, which is not uncommon for Interstate overpasses, there would be 6 expansion joints. If the distance between the rocker connections and the edges of the pier was approximately 6 inches, then it is conceivable that the spans could jam together such that a clear distance of 4-6 inches would be present at one place and this combined with pier movement could cause a collapse of a span without an abutment failure.

Other aspects that have not been discussed include horizontal accelerations in the transverse direction and motion in a vertical plane. Transverse motions were not studied because of attention by others. Vertical support accelerations were studied. Simulated earthquake motions having magnitudes of 0.09g and 10 sec duration were applied to the structure for frequencies ranging from 1 to 25 Hertz. The El Centro earthquake was reduced by 1/3 and induced as a vertical loading. The results showed that the maximum moments were well below the capacities of the girders and it was felt that vertical accelerations should not be a problem with this bridge.

SUMMARY AND CONCLUSIONS

This analytical study has been initiated to investigate and develop relatively inexpensive modeling techniques suitable for use on long multi-simple span highway bridges such that longitudinal seismic effects might be evaluated. Techniques included using a staged linear stiffness to represent the behavior of a nonlinear welded rocker connection. A typical three span bridge was described in a STRUDL-II program and the pier columns described as first a fixed support structure and then as a support having longitudinal independence of the supports such that the influence of a traveling wave could be determined. The technique used in describing the roller support structure was to increase the mass and apply a forcing function using a single degree of freedom approximation. The method was verified by comparing the modal shapes and natural frequencies of the fixed and roller support structures. Finally the third modeling technique was to apply undamped sinusoidal loadings having a fixed magnitude of 0.09g and a duration of 10 seconds. This same pulse was applied with different frequencies and the dynamic responses evaluated and then compared with results obtained by using the El Centro program package. This simulation technique did produce responses similar to the El Centro earthquake and shows promise of being a useful tool in making preliminary seismic evaluations.

The modeling techniques were applied to a single bridge in an effort to bring out the principal factors associated with multi-simple span bridges. The study shows that the welded rocker type connection in combination with unwelded rocker connections, a design feature used to free girders from longitudinal temperature effects, creates a structure that has a low fundamental frequency and is sensitive to low level seismic influences in the longitudinal direction. In particular two features are noted. The first is that there is a tendency for the spans to impact against each other and the abutments. Failure that any point in the chain could lead to structural collapse. Second, the relative motion between the ends of the spans and the pier cap can be severe, particularly when there are only roller connections. In this case the compressing of expansion joints could result in sufficient accumulation of girder translation such that one end of a span could fall from a narrow pier cap. This action could be accentuated over a long bridge where abutments could be excited to motion in opposite directions due to traveling wave effects.

The net conclusion is not that this is the first time that this phenomena has been observed, because retrofit procedures have been recommended for similar types of bridges (15). The main point that has evolved in this study is that such structures can be studied and evaluated with relatively inexpensive computer programming packages and this can lead to economy in selecting those bridges that might need either revised designs or retrofit applications.

ACKNOWLEDGEMENTS

The authors are indebted to the Civil Engineering Department and the Engineering Experiment Station at New Mexico State University for their support in this study.

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Table 1

Summary of Bridge Properties

Design Specifications

fc' = 5000 psi - Girders, 3000 psi - Columns, deck
E = 4.3×10^6 psi - Girders, 3.3×10^6 psi - Columns

Structural Properties

Part	Number	Effective Area in^3	Effective Moment of Inertia in^4	Average Densities lb/in^3
End Girders	2	3877	785,930	0.1536
Middle Girders	1	5000	1,691,070	0.1294
Columns	3	576	32,710	0.0868

Welded Rocker Connection Properties

Deformation Stage	Average Stiffness lb/in^3	Maximum Horiz. Defl.
I	340,190	0.013
II	297,860	0.031
III	152,290	0.129
IV	44,790	4.39

Table 2

Comparison of Modal Characteristics

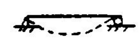
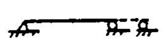
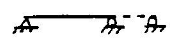
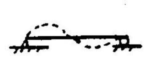
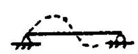
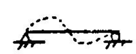
Structure	Mode	Frequency (Hertz) Lumped-Mass	Consistent-Mass	Theoretical
A	1	15.82	17.89	16.24
	2	68.22	67.72	61.85
	3	58.88	80.20	64.95
C	1	2.66	2.64	2.46
	2	18.86	18.85	18.69
			Modal Shapes	
A	1			
	2			
	3			
C	1			
	2			

Table 1

Summary of Bridge Properties

Design Specifications

fc' = 5000 psi - Girders, 3000 psi - Columns, deck
E = 4.3×10^6 psi - Girders, 3.3×10^6 psi - Columns

Structural Properties

Part	Number	Effective Area in^3	Effective Moment of Inertia in^4	Average Densities lb/in^3
End Girders	2	3877	785,930	0.1536
Middle Girders	1	5000	1,691,070	0.1294
Columns	3	576	32,710	0.0868

Welded Rocker Connection Properties

Deformation Stage	Average Stiffness lb/in^3	Maximum Horiz. Defl.
I	340,190	0.013
II	297,860	0.031
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Table 2

Comparison of Modal Characteristics

Structure	Mode	Frequency (Hertz) Lumped-Mass	Consistent-Mass	Theoretical
A	1	15.82	17.89	16.24
	2	68.22	67.72	61.85
	3	58.88	80.20	64.95
C	1	2.66	2.64	2.46
	2	18.86	18.85	18.69

Modal Shapes

Structure	Mode
A	1
	2
	3
C	1
	2

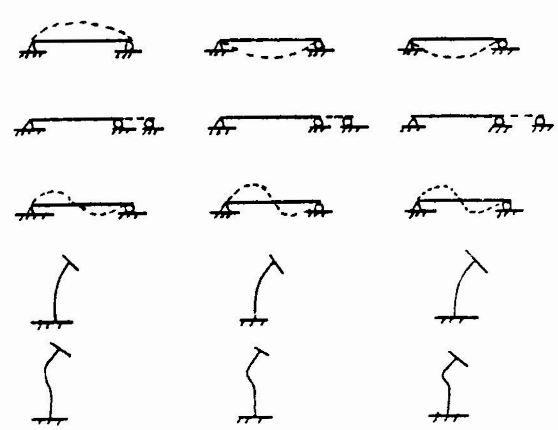


Table 3

Frequencies and Mode Shapes

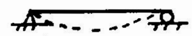
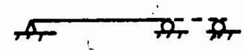
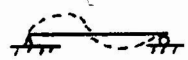
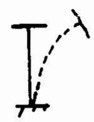
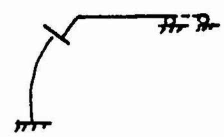
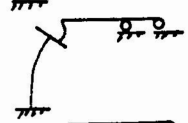
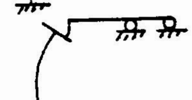
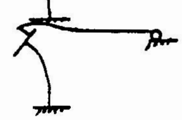
Structure	Mode	Frequency (Hertz)	Mode Shape
A and B	1	17.9	
	2	67.7	
	3	80.2	
C	1	2.7	
	2	18.9	
	3	57.6	
D (Connection Stage 4)	1	0.77	
	2	5.34	
	3	6.63	
	4	23.0	

Table 4

Comparison of Modal Characteristics - Fixed-Support and Roller-Support Models

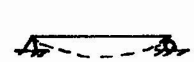
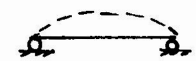
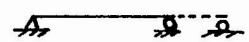
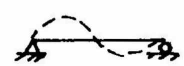
Structure	Mode	Frequency (Fixed Support Model)	Frequency (Roller-Support Model)	Shape (Fixed-Support Model)	Shape (Roller-Support Model)
A	1	17.89	15.82		
	2	67.72	68.23		
	3	80.20	58.88		
C	1	2.66	2.56		
	2	18.86	18.86		

Table 5

Comparison of Maximum Joint Displacements

Joints	2	4	8	9	11	14
Roller Support	+0.0002 -0.0002	+17.5 -17.3	+24.6 -24.1	+24.6 -24.1	+0.20 -0.20	+0.0002 -0.0002
Fixed Support	+0.0002 -0.0002	+19.6 -20.5	+26.5 -27.8	+26.5 -27.8	+0.17 -0.17	+0.0002 -0.0002

Table 6

Maximum Relative Displacements Between Spans (inches)

		Frequencies							
		0.4	0.6	0.8	1.0	1.25	2.0	5.0	El Centro
Joints	2-8	2.25	21.26	27.79	4.54	2.23	0.83	0.23	17.73
Joints	9-14	1.79	21.38	26.51	4.60	2.24	0.84	0.23	17.40

Table 7

Maximum Relative Displacements Between Spans and Piers (inches)

		Frequencies							
		0.4	0.6	0.8	1.0	1.25	2.0	5.0	El Centro
Joints	2-4	1.59	7.12	19.57	3.39	1.65	0.77	0.21	6.50
Joints	4-8	0.25	14.26	7.27	1.27	0.59	0.06	0.02	10.48
Joints	9-11	2.39	21.53	27.97	4.79	2.46	1.31	0.36	23.93
Joints	11-14	0.14	0.15	0.17	0.19	0.22	0.47	0.13	6.50

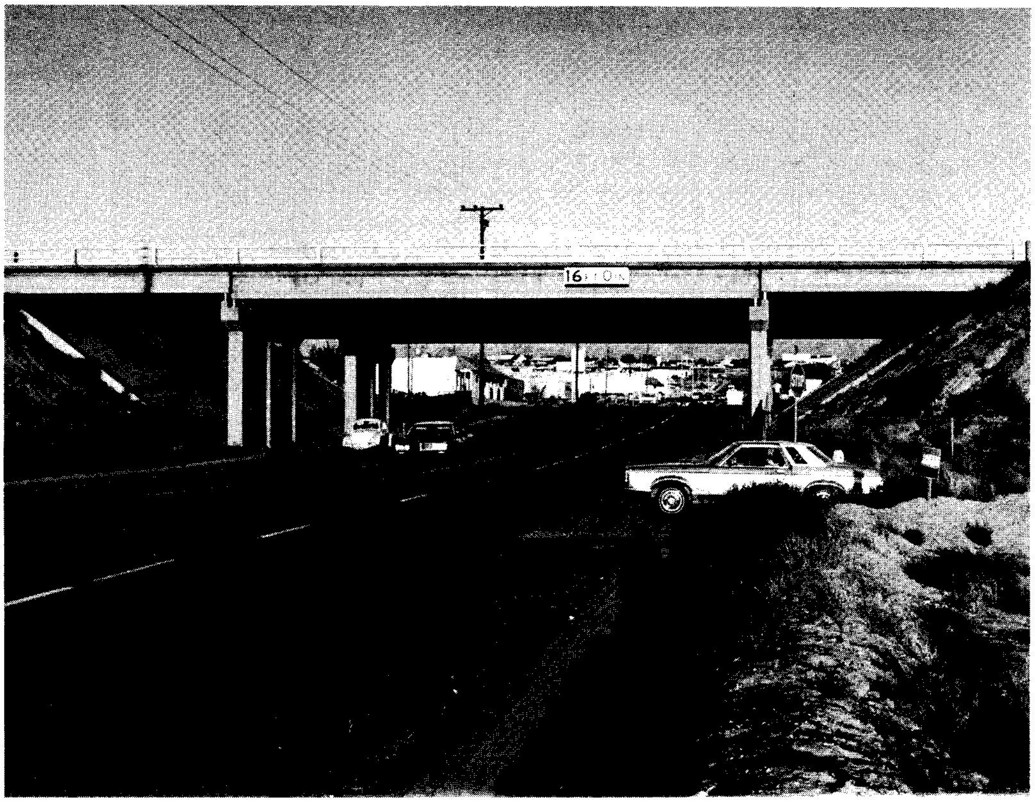
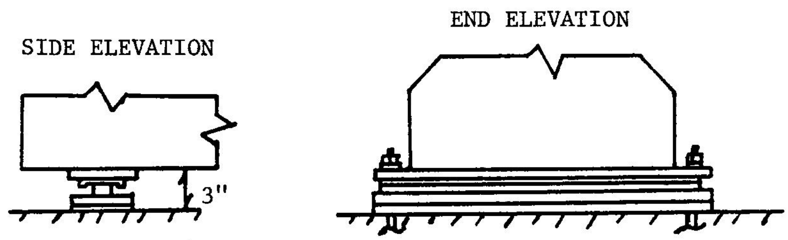
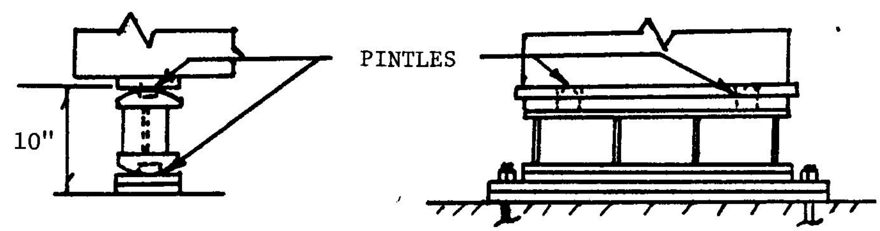


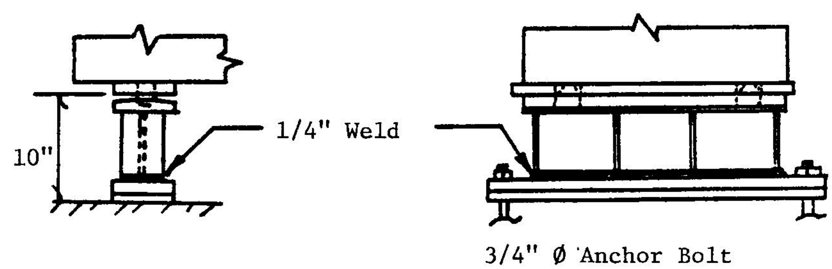
Figure 1 Southbound bridge crossing Missouri Street on I-25 in Las Cruces, New Mexico.



a. Hinged Connection



b. Rocker Connection



c. Welded Rocker Connection

Figure 2 Connection Details

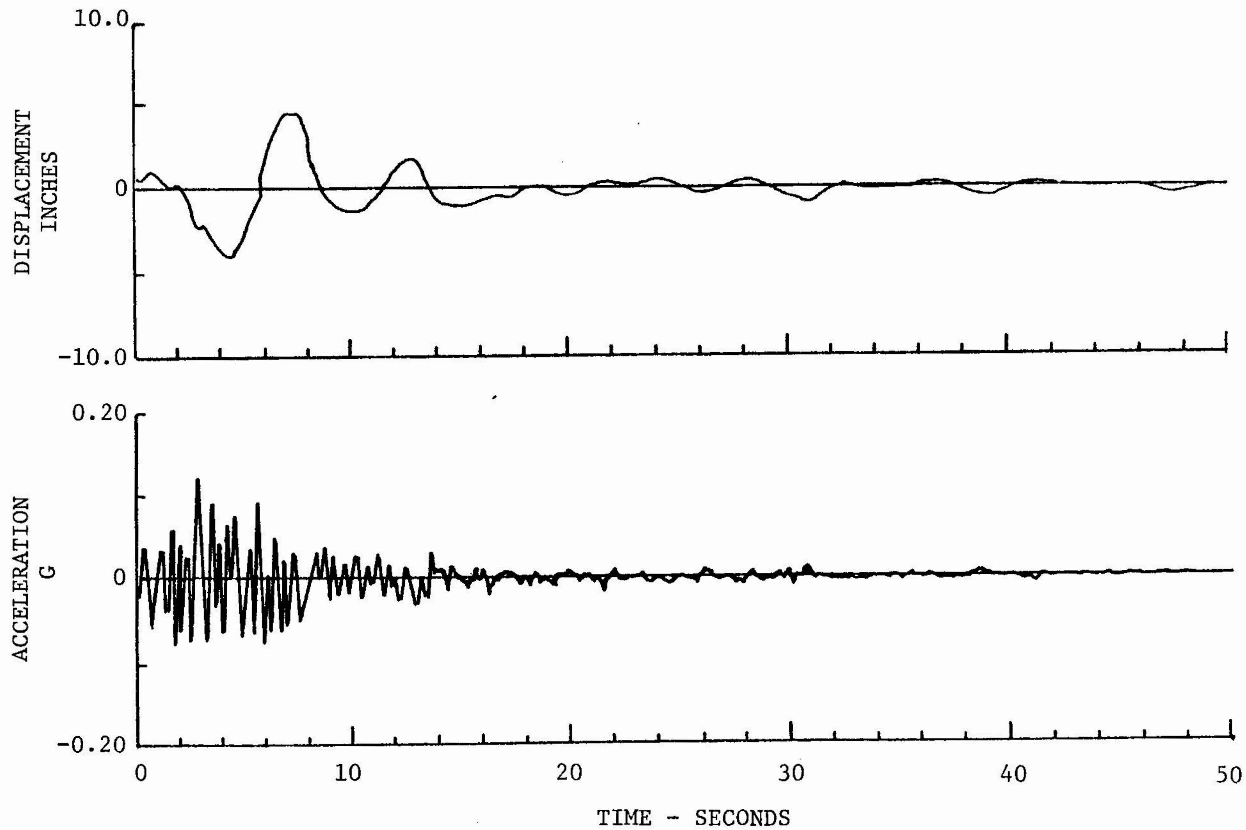


Figure 3 Support acceleration and displacement for the Kajima International Building due to the San Fernando earthquake.

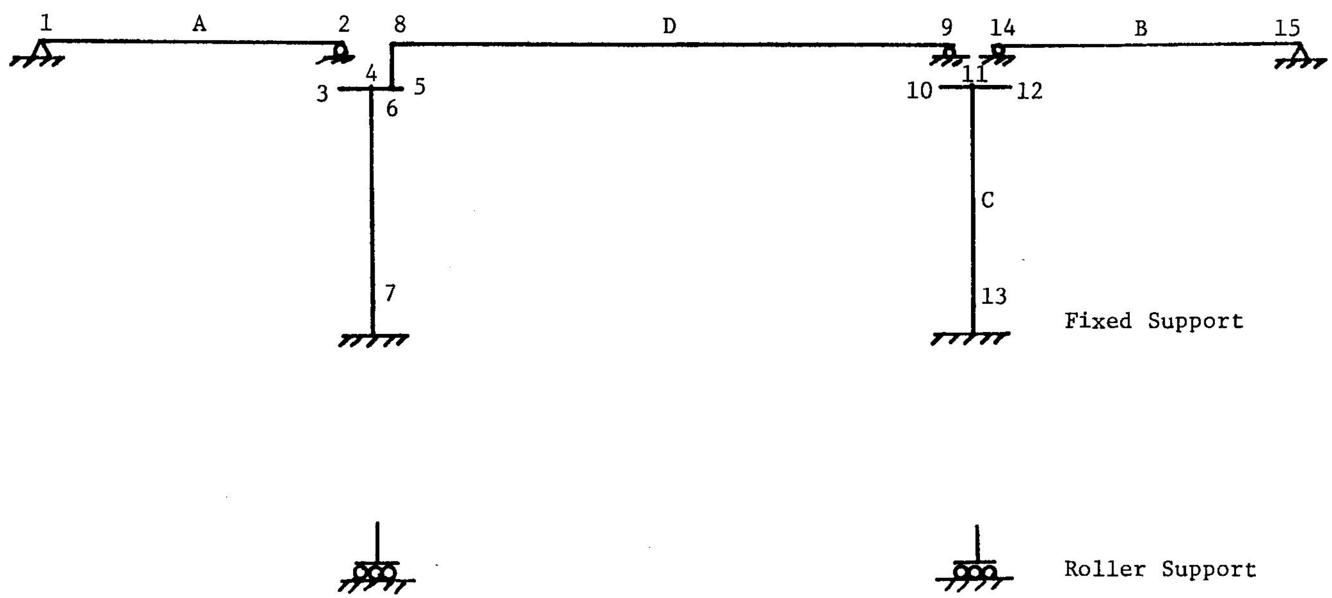


Figure 4 Joint numbering system for bridge model.

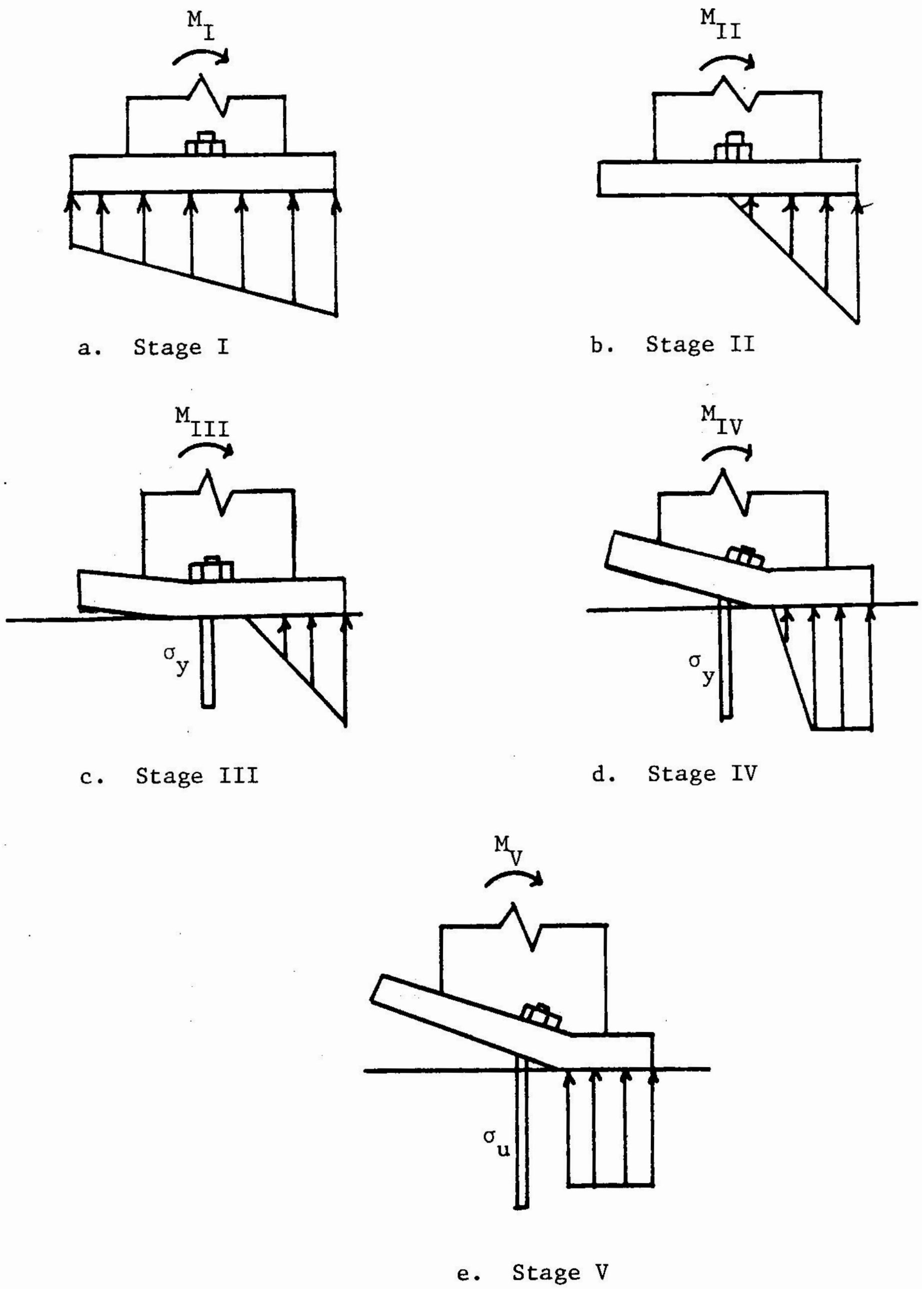


Figure 5 Stress distribution for the five stages of the welded-rocker connection

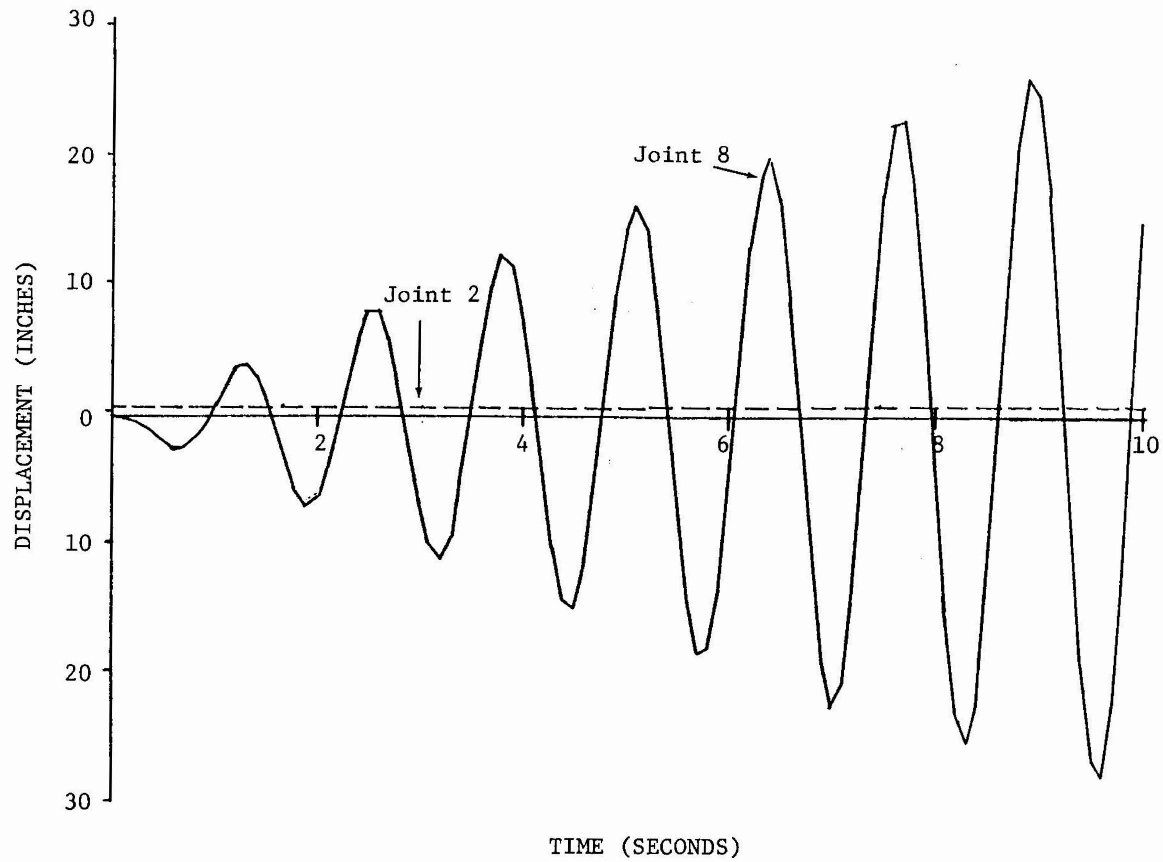


Figure 6 Response of joints 2 and 8 due to a horizontal support acceleration with a frequency of 0.8 hertz.

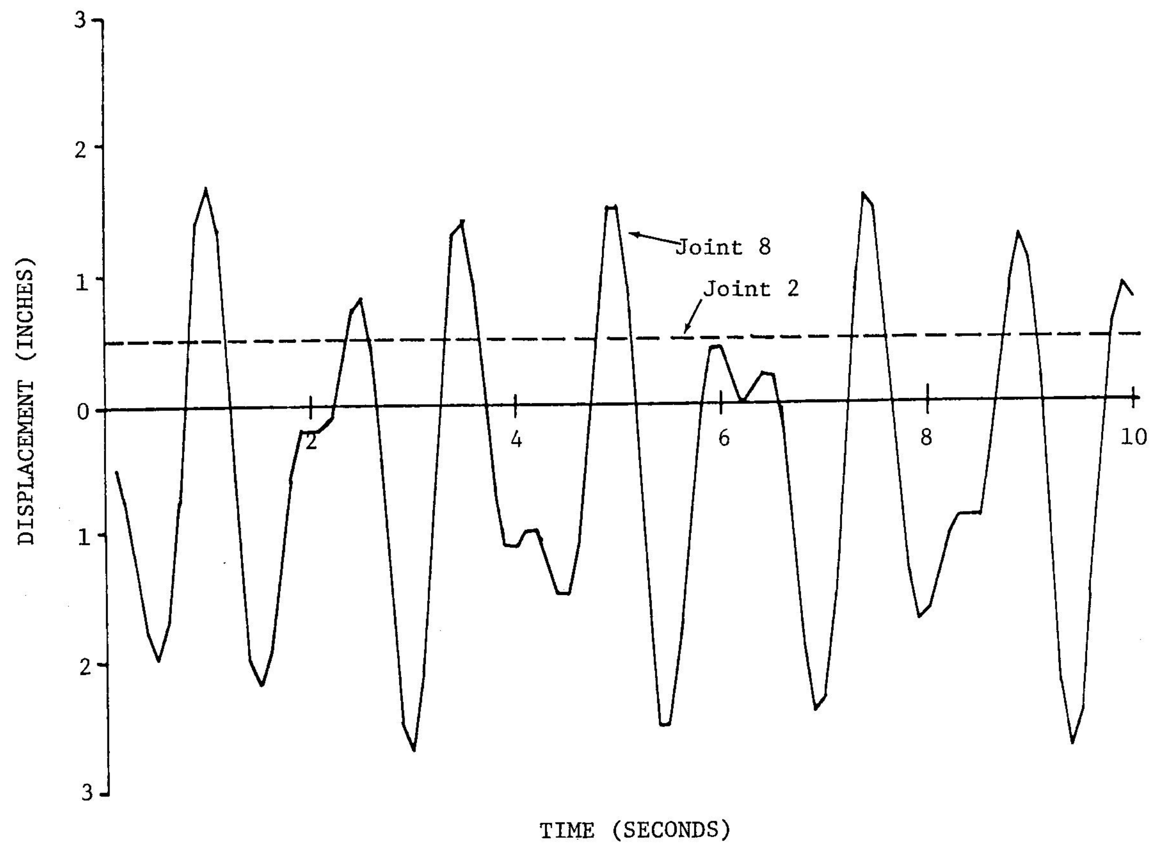


Figure 7 Response of joints 2 and 8 due to a horizontal support acceleration with a frequency of 1.25 hertz.

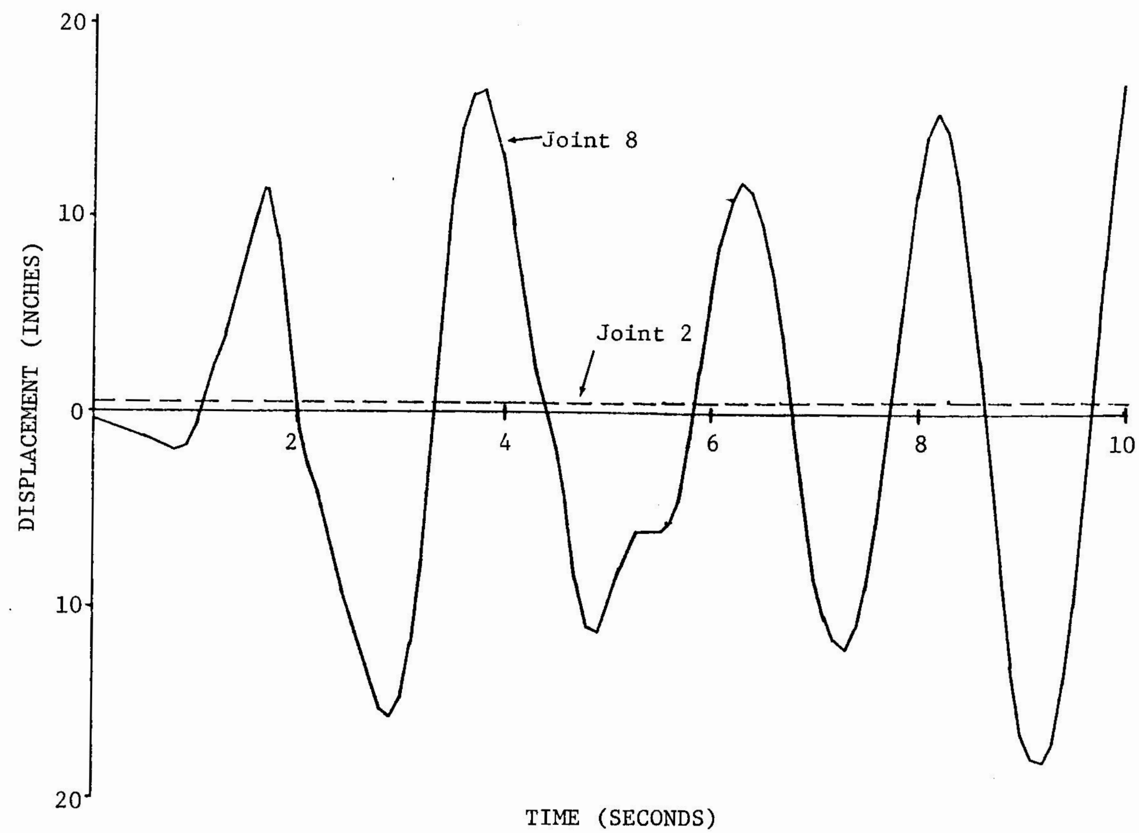


Figure 8 Response of joints 2 and 8 due to the El Centro earthquake.

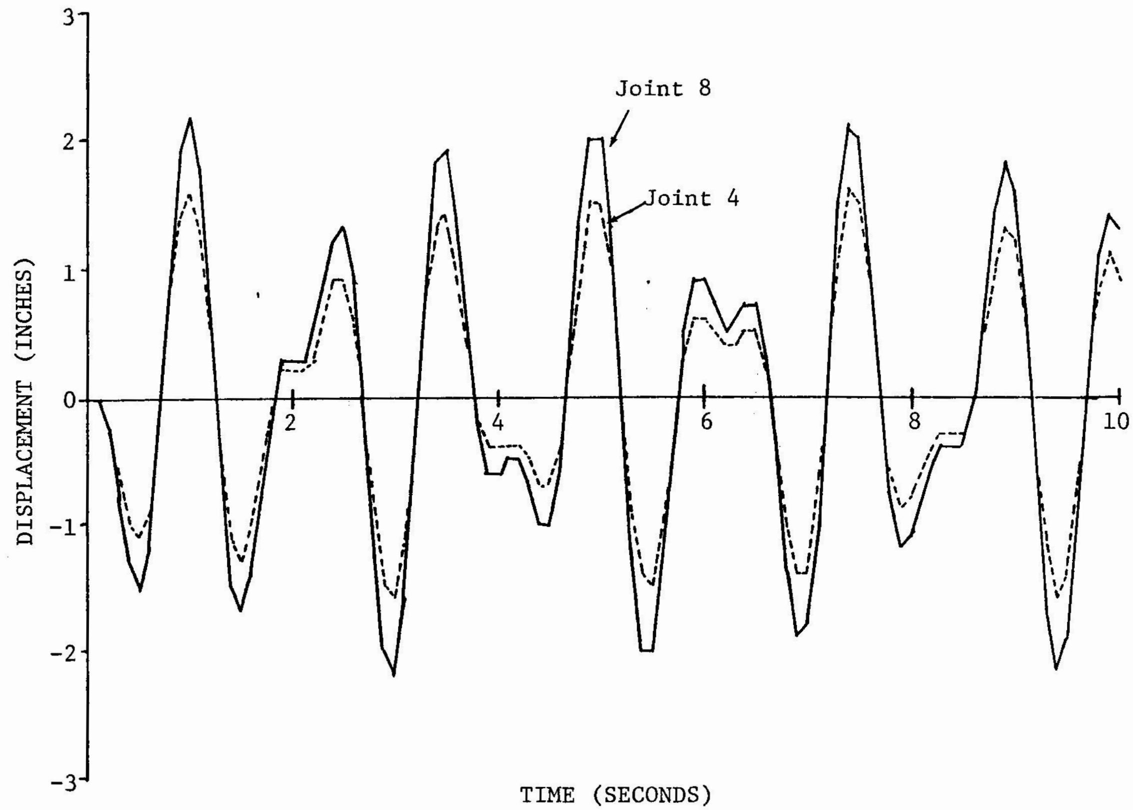


Figure 9 Response of joints 4 and 8 due to a horizontal support acceleration with a frequency of 1.25 hertz.

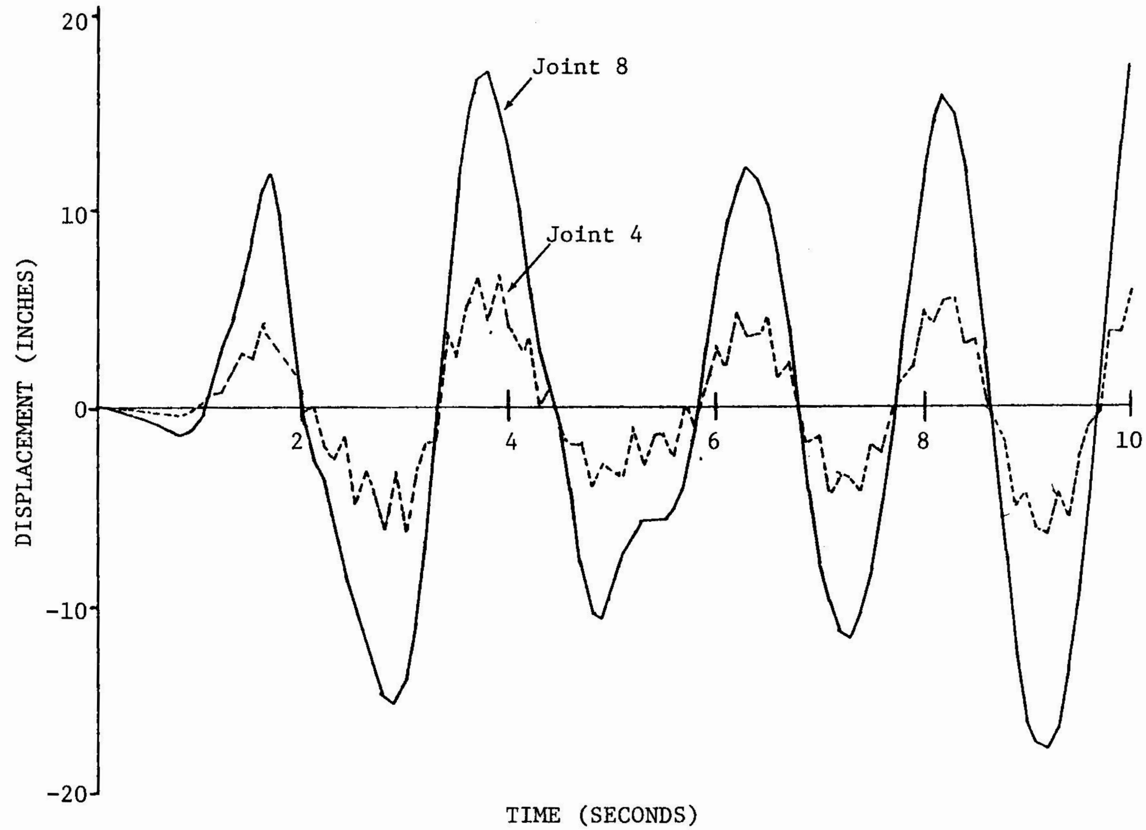


Figure 10 Response of joints 4 and 8 due to the El Centro earthquake.

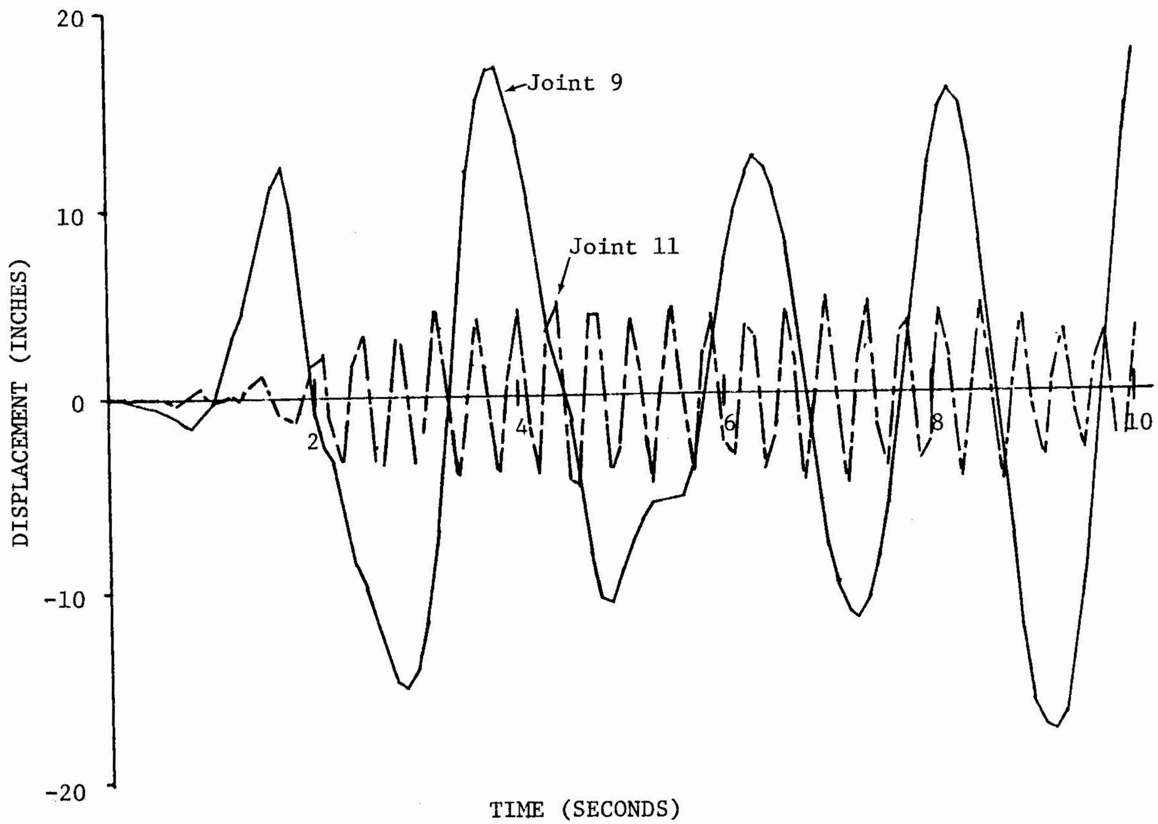


Figure 11 Response of joints 9 and 11 due to the El Centro earthquake.