

## NONLINEAR RESPONSE OF STEEL GIRDER BRIDGES UNDER VARIOUS DIRECTIONS AND INTENSITIES OF EARTHQUAKE MOTIONS

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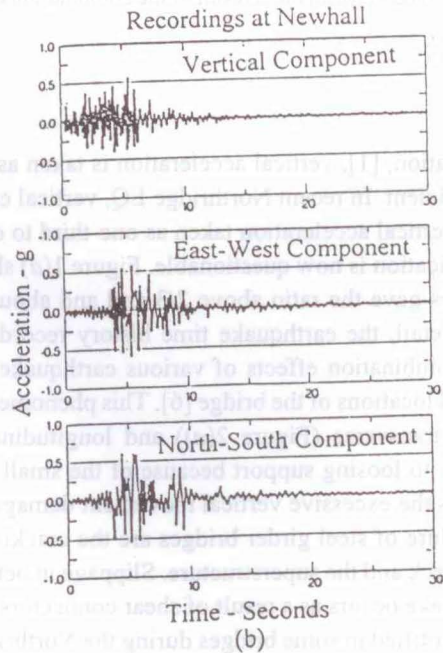
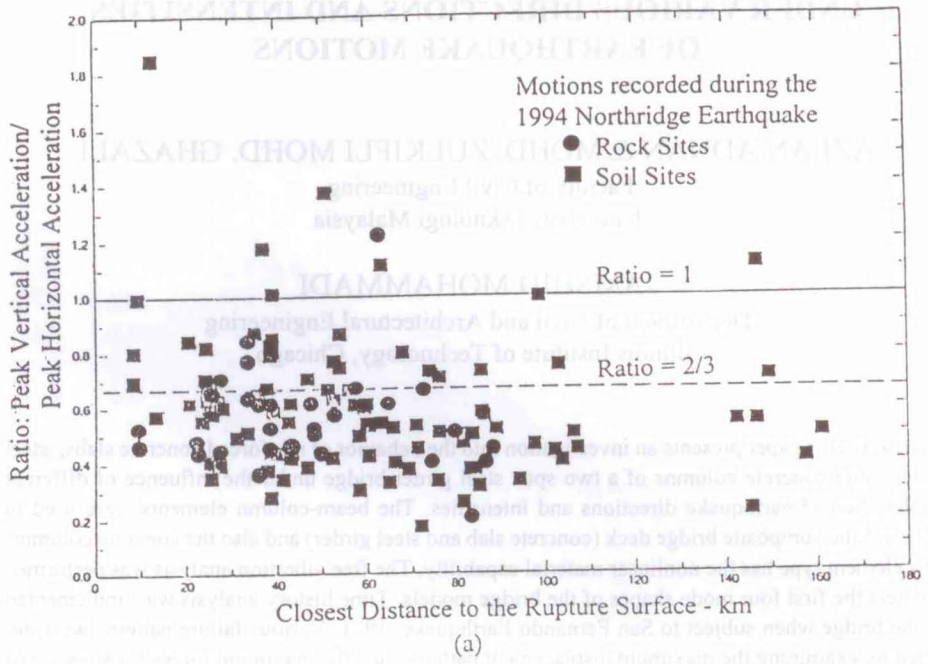
**Abstract.** This paper presents an investigation into the behavior of reinforced concrete slabs, steel girders, and concrete columns of a two span steel girder bridge under the influence of different combination of earthquake directions and intensities. The beam-column elements were used to represent the composite bridge deck (concrete slab and steel girder) and also the concrete columns. This element type has the nonlinear material capability. The free vibration analysis was performed to detect the first four mode shapes of the bridge models. Time history analysis was implemented on the bridge when subject to San Fernando Earthquake, 1971. Various failure patterns were detected by examining the maximum displacement patterns, and the maximum forces (or stresses) of the bridge components. Significant amplification of the bridge response occurred in all directions, longitudinally, transversely, and vertically as a result of the combination effect of earthquake in the other directions.

### 1 INTRODUCTION

In the current design specification, [1], vertical acceleration is taken as 1/3 to 1/2 of the horizontal acceleration, but this is insufficient. In recent Northridge EQ, vertical component is higher than the horizontal components [5]. Vertical acceleration taken as one-third to one-half of the horizontal as specified in the current specification is now questionable. Figure 1(a) shows that almost 30% of the records taken at different sites gave the ratio above 2/3 and about 10% above the ratio of 1. Figure 1(b) shows in more detail, the earthquake time history records in vertical and horizontal directions. Sometimes, the combination effects of various earthquake directions can amplify the displacement at certain critical locations of the bridge [6]. This phenomenon may cause failure of the steel girder bridges in both transverse (Figure 2(a)) and longitudinal directions (Figure 2(b)). Expansion joints open up due to loosing support because of the small seat width is also shown in Figure 3 [3]. Figure 4 exhibits the excessive vertical movement damaged the support system .

Some other possible failure of steel girder bridges are the cracking of the concrete deck and loss of integrity between the deck and the superstructure. Slippage in between slab and steel girder is expected if very large earthquake occurs as a result of shear connectors failure. Cracks at web plate near stiffeners were recently notified in some bridges during the Northridge Earthquake in 1994 [2].

All of the above mentioned failures could occur as a result of several factors. In this study, the effect of various directions of earthquake and intensities is studied and the nonlinear finite element analysis is performed.



**Figure 1** Record from the Northridge Earthquake on the January, 17, 1997. (a) Vertical to Horizontal Acceleration Ratio, (b) Vertical and Horizontal Component. (From Seismis Advisory Board, 1994)



The general objectives of this study are to determine whether:

- (a) the combination of earthquake directions produce amplification of bridge nonlinear response;
- (b) 1/2 to 2/3 vertical earthquake load considered in design is sufficient;
- (c) the intensity level where the bridge response linearly and non-linearly;
- (d) the three different level of earthquake intensity (low, moderate and high) affect the bridge; and,
- (e) whether intensities and combination of EQ directions contribute to the seismic deficiencies.

The research work investigates the behavior of the reinforced concrete slabs, the steel girders and the concrete columns under the influence of various earthquake directions (horizontal, vertical and any combination of them). The beam models consisting of beam-column elements, were used to simulate a 2-span steel girder bridge located in Southern Illinois. A non-linear finite element software, DRAIN D2DX, was fully utilized for the analysis purposes. Free vibration analysis was performed to detect the first four mode shapes of the bridge models. Time history analysis of the bridge is also studied when the bridge is subject to San Fernando Earthquake, 1971. Various failure patterns were detected by examining the maximum displacement patterns, and the maximum forces (or stresses) of the structural components.

## 2 THE FINITE ELEMENT MODELS

Figure 5 shows the configuration of the bridge under study. It is a 2-span steel girder bridge located at Southern Illinois and built in 1936. Since the finite element program can only analyse object in a two dimensional plane, the bridge has to be modelled both longitudinally and transversely with pinned and roller support system as described in Figures 6, 7, and 8. Beam-column element from type 02 is used for elements in the models. The San Fernando Earthquake record was taken at Pacoima Dam. All values are the maximum values taken at any joints along the superstructure (deck) and/or along the column.

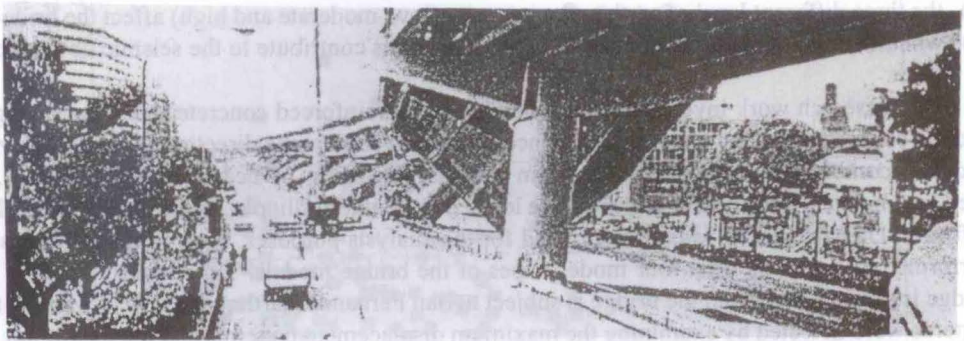
**Model No.1: 2-Span bridge with pin-ended supports:** At end supports, the boundary conditions are issued by restraining the x and y translational directions (Figure 6). There are 9 nodes on the deck and 2 nodes at the column. A total of 10 elements are used. The column is fixed on the ground. This single column elements actually represents 3 columns. Therefore, the properties are adjusted accordingly.

**Model No.2: 2-Span bridge with roller-ended supports:** The same model as in model no.1, except for roller supports are used instead of pin supports (Figure 7).

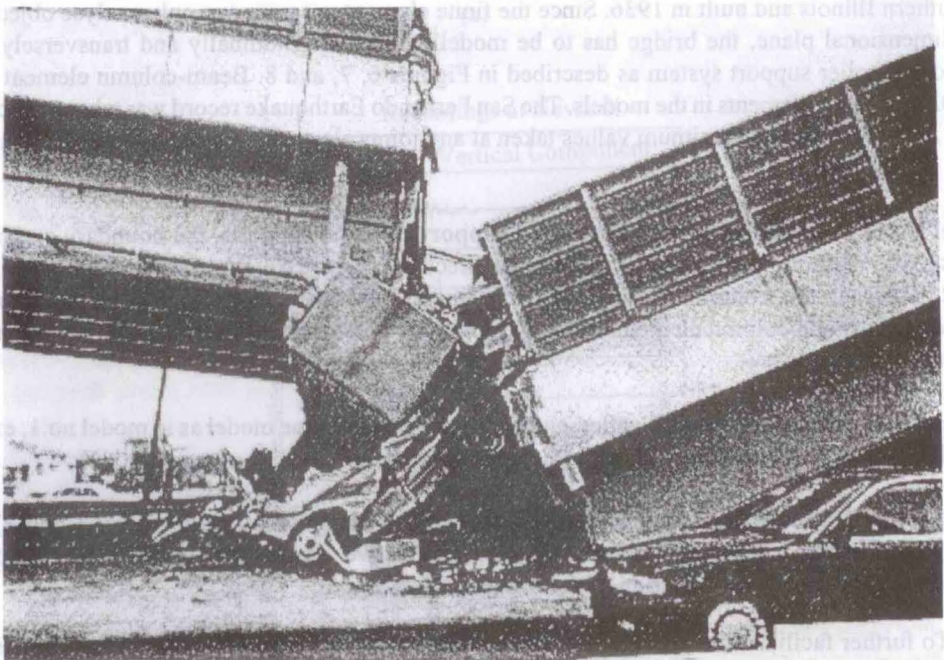
**Model No.3: 2-Span bridge in transverse direction:** In the previous models (No: 1 and 2), the transverse direction cannot be obtained because it is a two dimensional model. This model is representing the transverse direction of the bridge (Figure 8).

To further facilitate the reference to the direction of seismic motion for the three models, the following notations are used:

- x-EQ = longitudinal direction of earthquake for model no.1 and 2, and transverse direction of earthquake for model no.3;
- y-EQ = vertical direction of earthquake (for all models); and,
- xy-EQ = longitudinal and vertical directions of earthquake (for model no. 1 and 2), and transverse and vertical directions of earthquake (for model no.3).



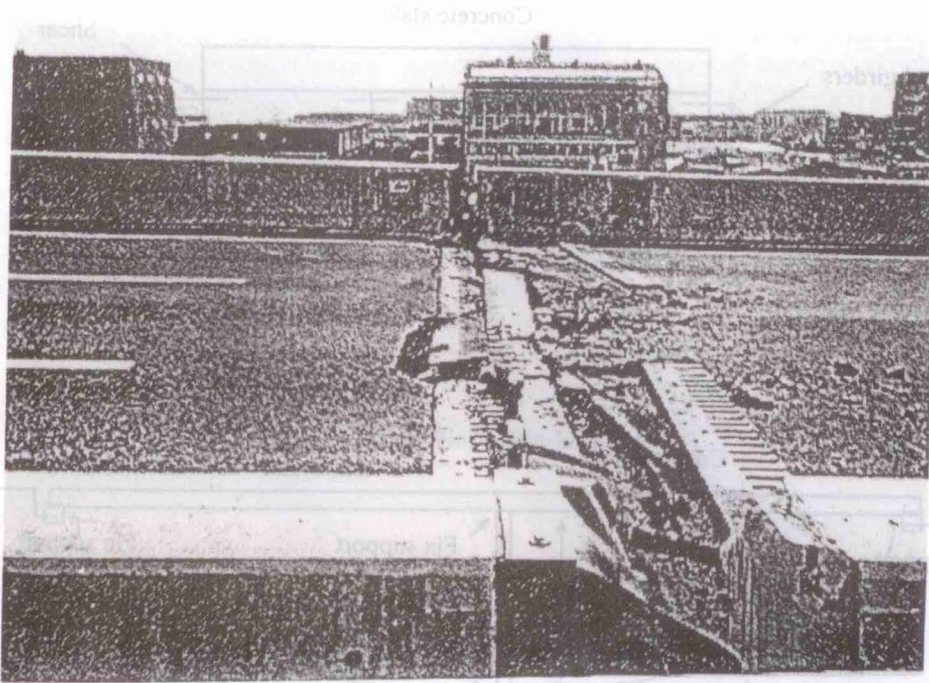
(a)



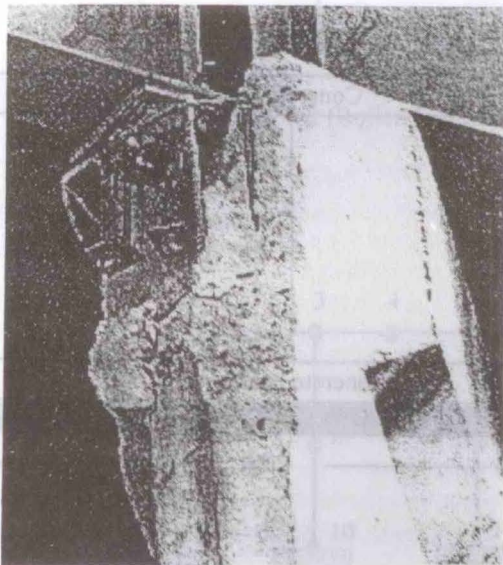
(b)

**Figure 2** Bridge Damages Caused by (a) Transverse Movement, and (b) Longitudinal Movement. (From Earthquake Engineering Research Institute, 1995)

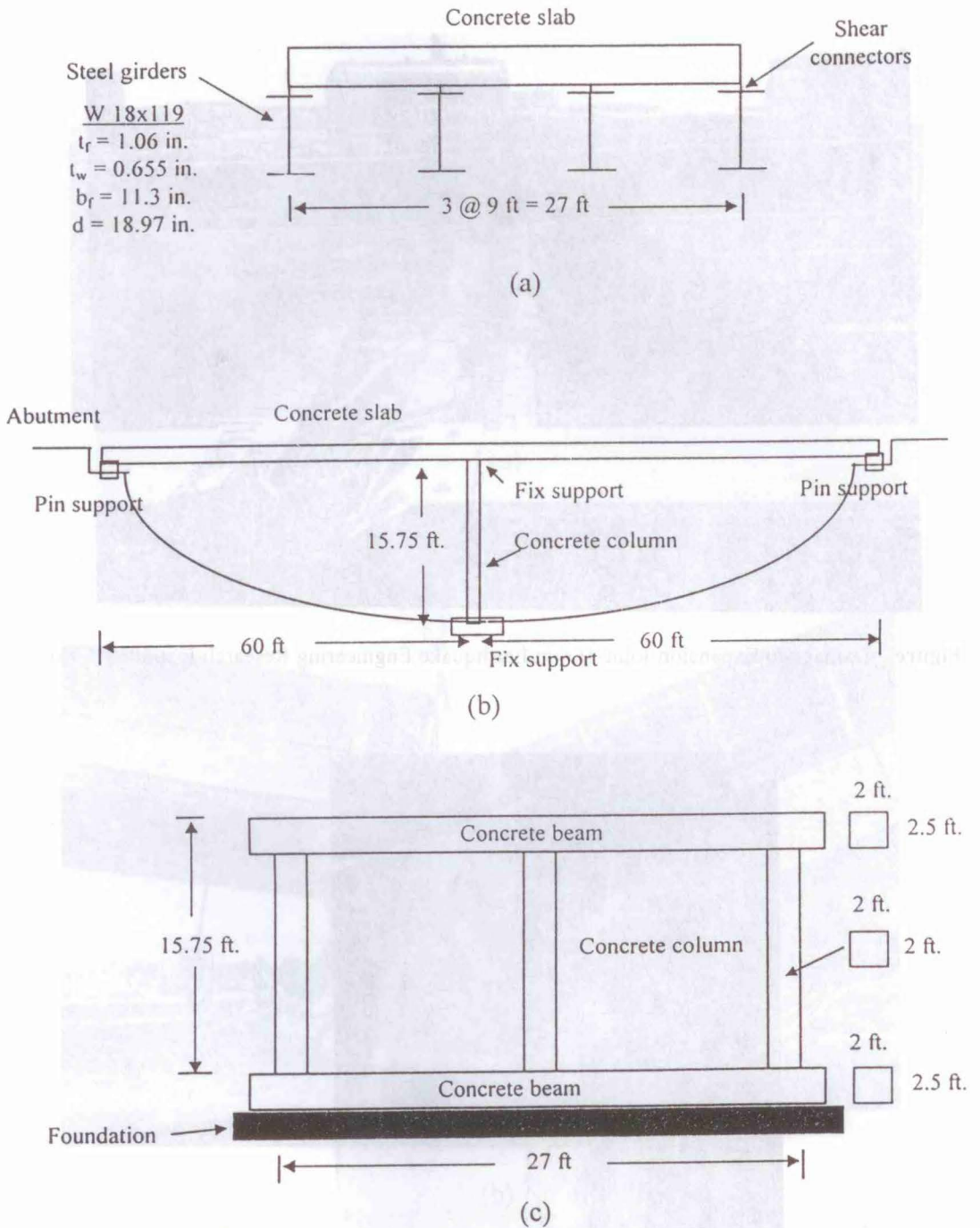




**Figure 3** Damage to Expansion Joint. (From Earthquake Engineering Research Institute, 1995)



**Figure 4** Damage to Support System. (From Seismic Advisory Board, 1994)



**Figure 5** The 2-Span Bridge. (a) The Concrete Slabs and Steel Girders (b) The Bridge Superstructure (c) The Concrete Column Bent.

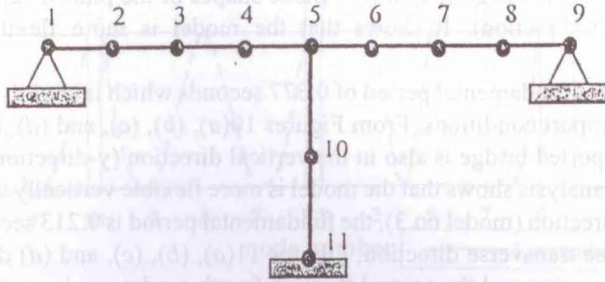


Figure 6 Model No. 1 (Pin-End Support).

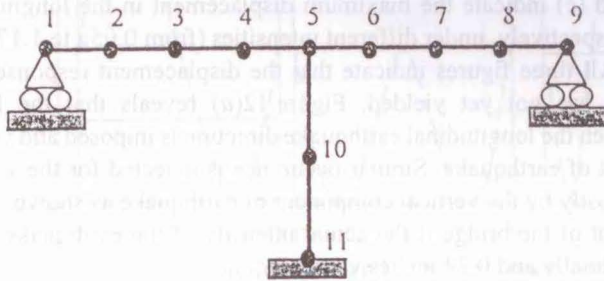


Figure 7 Model No. 2 (Roller-End Support).

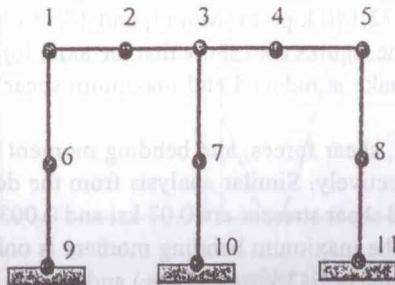


Figure 8 Model No. 3 (Transverse Direction).



### 3 RESULTS AND DISCUSSIONS

#### 3.1 Free Vibration Analysis

The fundamental period for model no.1 is 0.253 seconds which is common to a short span bridge. From Figures 9(a), (b), (c), and (d), the first four mode shapes of the pinned supported bridge is in the vertical direction (y-direction). It shows that the model is more flexible vertically than longitudinally.

Model no.2 produces fundamental period of 0.377 seconds which is slightly higher than model no.1 due to the roller support conditions. From Figures 10(a), (b), (c), and (d), the first four mode shape of the pinned supported bridge is also in the vertical direction (y-direction) similar to model no.1. The free vibration analysis shows that the model is more flexible vertically than longitudinally.

In the transverse direction (model no.3), the fundamental period is 0.213 seconds, meaning that the bridge is stiffer in the transverse direction. Figures 11(a), (b), (c), and (d) denote that the first mode is in transverse direction and the second through fourth modes are in vertical direction. This particular model is more flexible transversely than vertically because of its longer natural period in the transverse direction.

#### 3.2 Time History Analysis

##### 3.2.1 Model No.1: 2-Span bridge with pin-end supports

Figures 12(a), (b), and (c) indicate the maximum displacement in the longitudinal, vertical, and rotational directions respectively, under different intensities (from 0.05g to 1.17g) and directions of earthquake motions. All three figures indicate that the displacement responses linearly and may show that the bridge has not yet yielded. Figure 12(a) reveals that the largest longitudinal displacement occur when the longitudinal earthquake direction is imposed and very little effect from the vertical component of earthquake. Similar occurrence is detected for the vertical displacement which is influenced mostly by the vertical component of earthquake as shown in Figure 12(b). The maximum displacement of the bridge if the actual intensity of the earthquake (1.17g) occurred is 0.0025 inches longitudinally and 0.22 inches vertically.

Figures 13(a), (b), and (c) depict the maximum axial and shear forces, and bending moments of the superstructure (deck). These values can be converted to axial and shear stresses by dividing the values with the composite cross section area of 432.776 in<sup>2</sup> which has been converted to steel section. The bending moment capacity of the superstructure or column can be obtained from the DRAIN-2DX input data for Mp(+) and Mp(-). The 1.17g intensity earthquake will provide the maximum stresses of 0.17 kips/in<sup>2</sup> (axial) and 0.1 kips/in<sup>2</sup> (shear). These values are well below the allowable tensile and shear stresses of steel which are 36 ksi and 4 ksi respectively. The yield moment issued in the program was 72,140 kips-in [My(+)] and 37,584 ksi [My(-)] and the maximum response is only 7400 kips-in. The figures also show that the axial forces are at the maximum when longitudinal direction of earthquake is induced and maximum shear forces are due to the vertical earthquake component.

The maximum axial forces, shear forces, and bending moment for the columns are shown in Figures 14(a), (b), and (c), respectively. Similar analysis from the deck results can be done to the column. The maximum axial and shear stresses are 0.07 ksi and 0.003 ksi compare to the allowable stress of 0.44 ksi for concrete. The maximum bending moment is only 300 kips-in, lower than the yield moment used in the program, 14,643 kips-in [M(+) and M(-)]. From the figures, we can also notice that the maximum axial forces are due to the vertical earthquake motion and the maximum shear forces are due to the longitudinal earthquake motion.



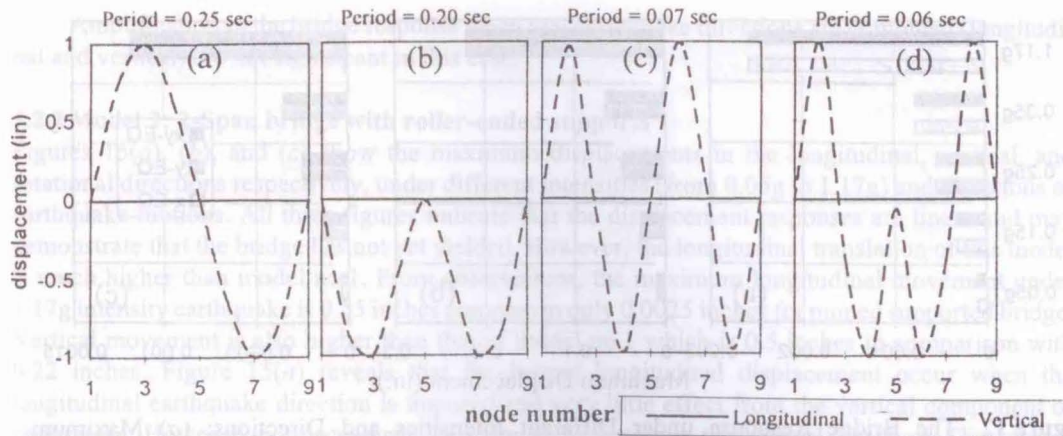


Figure 9 Free Vibration Analysis for the 2 Span Bridge in Pin-End Support; (a) Mode Shape 1, (b) Mode Shape 2, (c) Mode Shape 3, and (d) Mode Shape 4.

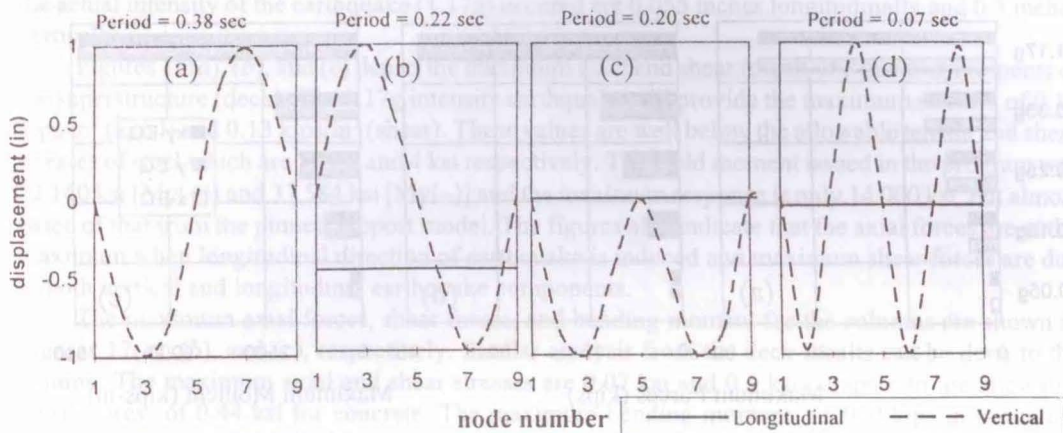


Figure 10 Free Vibration Analysis for the 2 Span Bridge with Roller-End Support; (a) Mode Shape 1, (b) Mode Shape 2, (c) Mode Shape 3, and (d) Mode Shape 4.

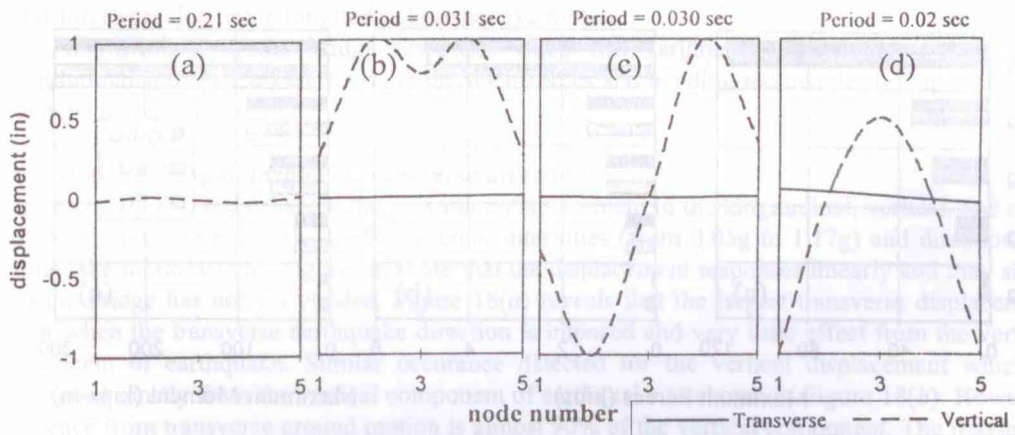
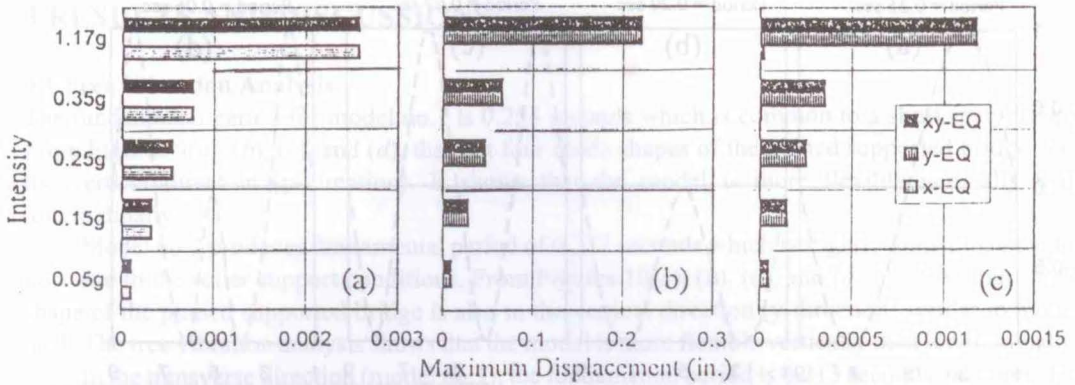
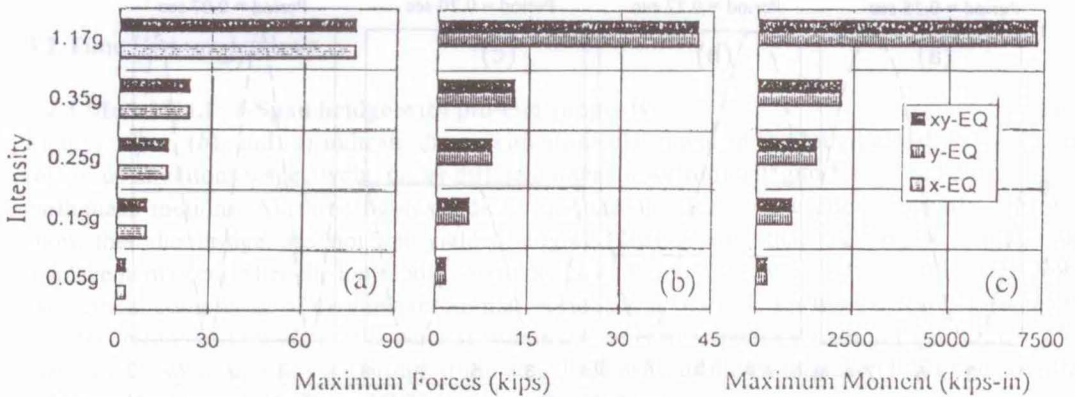


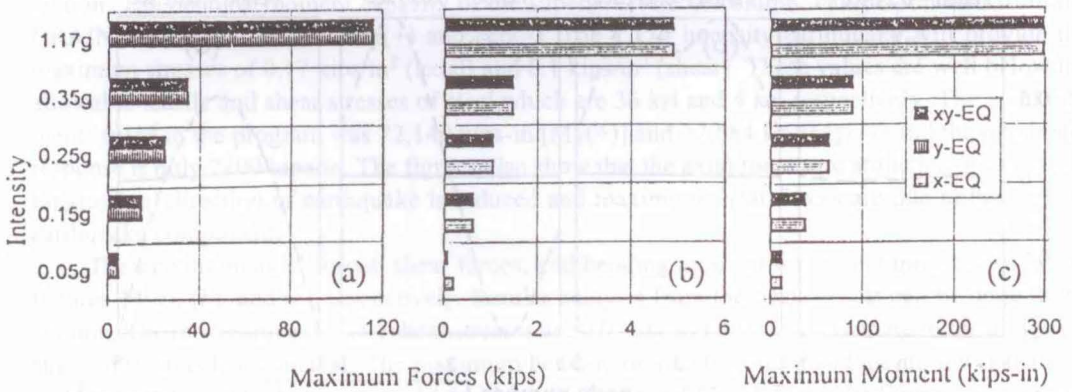
Figure 11 Free Vibration Analysis for the 2 Span Bridge in Transverse Direction; (a) Mode Shape 1, (b) Mode Shape 2, (c) Mode Shape 3, and (d) Mode Shape 4.



**Figure 12** The Bridge Response under Different Intensities and Directions; (a) Maximum Longitudinal Displacement, (b) Maximum Vertical Displacement, and (c) Maximum Rotational Displacement.



**Figure 13** The Bridge Deck Response under Different Intensities and Directions; (a) Maximum Axial Forces (kips), (b) Maximum Shear Forces (kips), and (c) Maximum Bending Moment (kips-in).



**Figure 14** The Bridge Column Response under Different Intensities and Directions of Earthquake; (a) Maximum Axial Forces (kips), (b) Maximum Shear Forces (kips), and (c) Maximum Bending Moment (kips-in).



Amplification in the bridge response when two earthquake directions are combined (longitudinal and vertical) are not significant in this case.

### 3.2.2 Model 2: 2-Span bridge with roller-ended supports

Figures 15(a), (b), and (c) show the maximum displacements in the longitudinal, vertical, and rotational directions respectively, under different intensities (from 0.05g to 1.17g) and directions of earthquake motions. All three figures indicate that the displacement responses are linear and may demonstrate that the bridge has not yet yielded. However, the longitudinal translation of this model is much higher than model no.1. From observations, the maximum longitudinal movement under 1.17g intensity earthquake is 0.55 inches compare to only 0.0025 inches for pinned supported bridge. Vertical movement is also higher than that of model no.1 which is 0.5 inches in comparison with 0.22 inches. Figure 15(a) reveals that the largest longitudinal displacement occur when the longitudinal earthquake direction is imposed and very little effect from the vertical component of earthquake. However, not necessarily the same trend occurs in the vertical direction. The vertical displacement is now influenced mostly by the longitudinal ground movement and the combination effect is far more influential, as shown in Figure 15(b). The maximum displacements of the bridge if the actual intensity of the earthquake (1.17g) occurred are 0.055 inches longitudinally and 0.5 inches vertically.

Figures 16(a), (b), and (c) depict the maximum axial and shear forces, and bending moments of the superstructure (deck). The 1.17g intensity earthquake will provide the maximum stresses of 0.17 kips/in<sup>2</sup> (axial) and 0.13 kips/in<sup>2</sup> (shear). These values are well below the allowable tensile and shear stresses of steel which are 36 ksi and 4 ksi respectively. The yield moment issued in the program was 72 140 ksi [My(+)] and 37 584 ksi [My(-)] and the maximum response is only 14 000 ksi, but almost twice of that from the pinned support model. The figures also indicate that the axial forces are at the maximum when longitudinal direction of earthquake is induced and maximum shear forces are due to both vertical and longitudinal earthquake components.

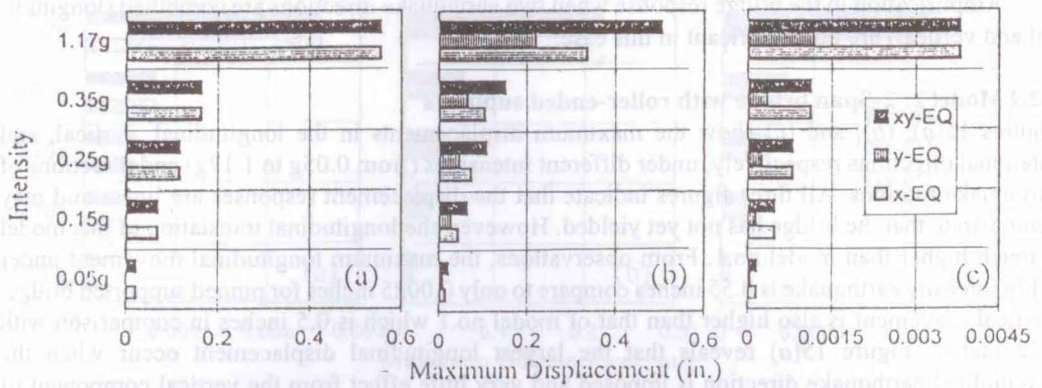
The maximum axial forces, shear forces, and bending moment for the columns are shown in Figures 17(a), (b), and (c), respectively. Similar analysis from the deck results can be done to the column. The maximum axial and shear stresses are 0.07 ksi and 0.1 ksi compare to the allowable tensile stress of 0.44 ksi for concrete. The maximum bending moment, 15 000 kips-in is slightly more than the yield moment used in the program, 14 643 kips-in [M(+) and M(-)]. This results indicate that the column might fail first before the superstructure fail. From the figures, we can also notice that the maximum axial forces are due to the vertical earthquake motion and the maximum shear forces are due to the longitudinal earthquake motion.

Amplification in the bridge responses when two earthquake directions are combined (longitudinal and vertical) are found in the shear forces and bending moment in the superstructure and none in the column.

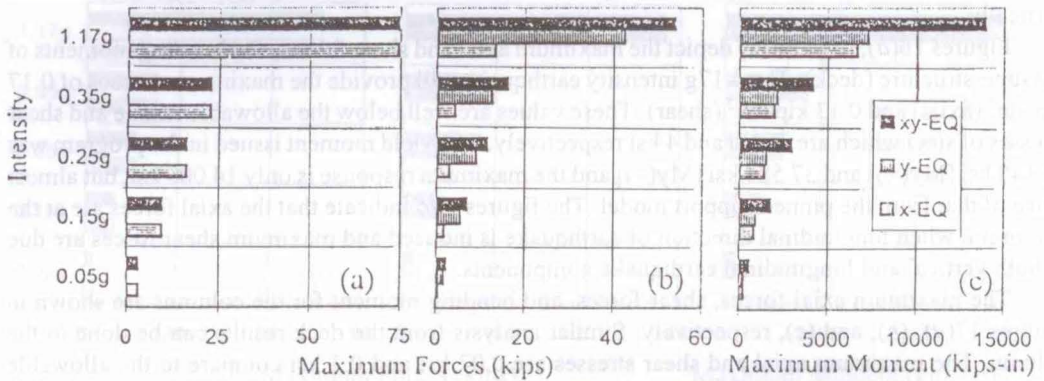
### 3.2.3 Model 3: 2-Span bridge in transverse direction

Figures 18(a), (b), and (c) show the maximum displacement in the longitudinal, vertical, and rotational directions respectively, under different intensities (from 0.05g to 1.17g) and directions of earthquake motions. All 3 figures indicate that the displacement responses linearly and may show that the bridge has not yet yielded. Figure 18(a) reveals that the largest transverse displacement occur when the transverse earthquake direction is imposed and very little effect from the vertical component of earthquake. Similar occurrence detected for the vertical displacement which is influenced mostly from the vertical component of earthquake as shown in Figure 18(b). However, influence from transverse ground motion is almost 90% of the vertical component. The maximum displacement of the bridge if the actual intensity of the earthquake (1.17g) occurred is 0.23 inches transversely and about 0.01 inches vertically.

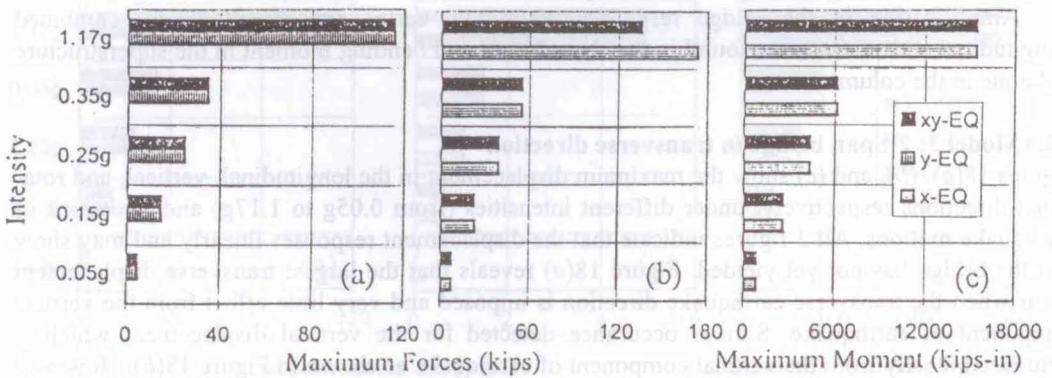




**Figure 15** The Bridge Response under Different Intensities and Directions; (a) Maximum Longitudinal Displacement, (b) Maximum Vertical Displacement, and (c) Maximum Rotational Displacement.



**Figure 16** The Bridge Deck Response under Different Intensities and Directions; (a) Maximum Axial Forces (kips), (b) Maximum Shear Forces (kips), and (c) Maximum Bending Moment (kips-in).



**Figure 17** The Bridge Column Response under Different Intensities and Directions of Earthquake; (a) Maximum Axial Forces (kips), (b) Maximum Shear Forces (kips), and (c) Maximum Bending Moment (kips-in).



Figures 19(a), (b), and (c) depict the maximum axial and shear forces, and bending moments of the superstructure (deck). These values can be converted to axial and shear stresses by dividing with the composite cross section area of  $720 \text{ in}^2$  which has been converted to steel section. The bending moment capacity of the superstructure or column can be obtained from the DRAIN-2DX input data for  $M_p(+)$  and  $M_p(-)$ . The 1.17g intensity earthquake will provide the maximum stresses of  $0.017 \text{ kips/in}^2$  (axial) and  $0.065 \text{ kips/in}^2$  (shear). These values are well below the allowable tensile and shear stresses of steel which are 36 ksi and 4 ksi respectively. The yield moment issued in the program was 3328 kips-in [ $M_y(+)$  and  $M_y(-)$ ] and the maximum response is only 2750 kips-in. The figures also show that the axial and shear forces are at the maximum when transverse direction of earthquake is induced.

The maximum axial forces, shear forces, and bending moment for the columns are shown in Figures 20(a), (b), and (c), respectively. Similar analysis from the deck results can be done to the column. The maximum axial and shear stresses are 0.11 ksi and 0.07 ksi compare to the allowable stress of 0.44 ksi for concrete. The maximum bending moment is about 3700 kips-in compare to the yield moment used in the program, 4,881 kips-in [ $M(+)$  and  $M(-)$ ]. From the figures, we can also notice that the maximum axial forces are due to both transverse and vertical earthquake motions and the maximum shear forces are due to the transverse earthquake motion.

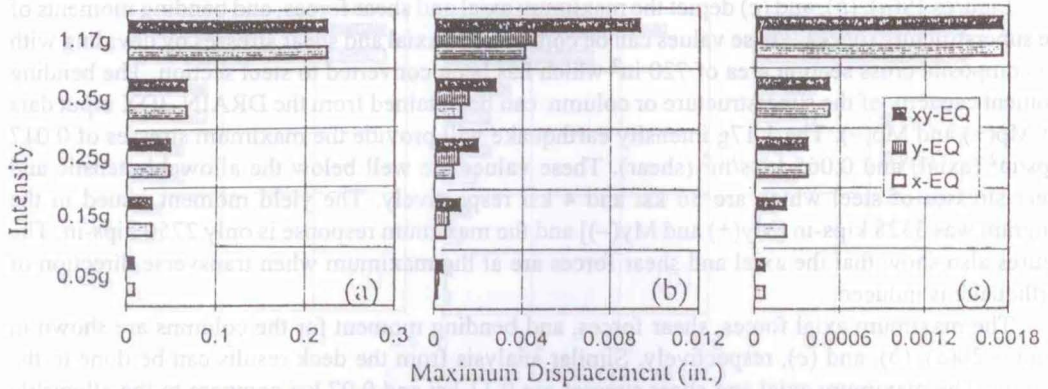
Amplification in the bridge responses when two earthquake directions are combined (transverse and vertical) are found in the maximum vertical displacement, maximum shear forces in deck, and maximum axial forces in the column.

#### 4 CONCLUSIONS

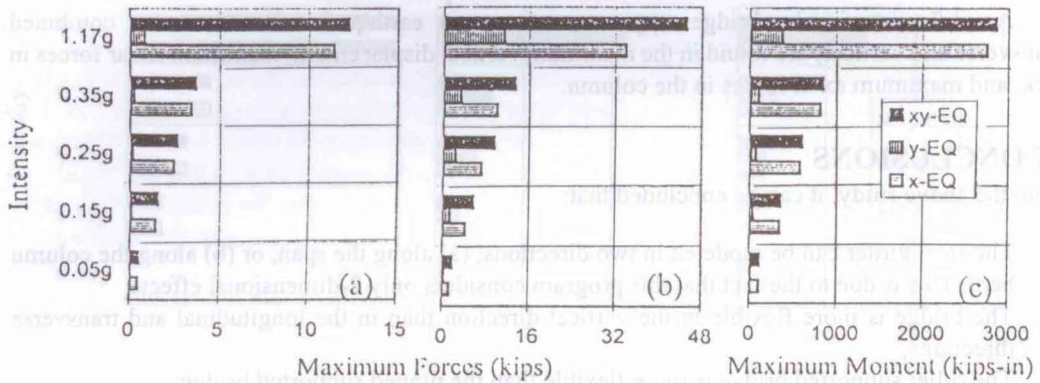
From the above study, it can be concluded that:

- (1) The steel girder can be modeled in two directions; (a) along the span, or (b) along the column bent. This is due to the fact that this program considers only 2-dimensional effects;
- (2) The bridge is more flexible in the vertical direction than in the longitudinal and transverse directions;
- (3) The roller supported bridge is more flexible than the pinned supported bridge;
- (4) Combination of earthquake directions produce amplification of bridge response. In nonlinear analysis, significant amplification of bridge response occur in all directions, longitudinally, transversely, and vertically as a result of the combination effect of earthquake in the other directions;
- (5) Considering only 1/2 to 2/3 of the horizontal component earthquake as the vertical component in design is not sufficient because from the 1994 Northridge Earthquake record, it is possible to have earthquake that has the vertical to horizontal ratio of one. The combination effect of vertical component with the lateral component of earthquakes to all types of responses are significant especially for the displacement response where the amplification is at the most;
- (6) The intensity level increment can determine when and where the bridge response linearly and nonlinearly. This procedure is sometime called "Push-Over Analysis". For this type of bridge and model, we found that the earthquake intensity and bridge responses vary linearly until 1.17g. It is possible to detect at which particular component and location of the bridge where the response could be acting non-linearly. The non-linear analysis is important for engineers to check whether the structure has gone through nonlinear response. For example, in the study of a 2-span bridge, with intensity level of 1.17g, none of the components of the deck (slab and girder) has passed the yielded stress except for the model with roller support system where the bending moment capacity of the column has exceeded to about 7%. However, the maximum displacement of about 0.55 inches in the x-direction (longitudinal) somewhere along the girder

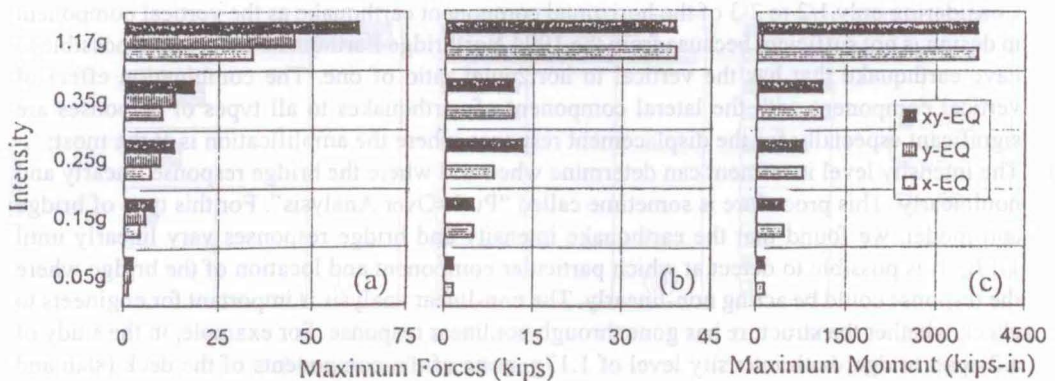




**Figure 18** The Bridge Response under Different Intensities and Directions; (a) Maximum Longitudinal Displacement, (b) Maximum Vertical Displacement, and (c) Maximum Rotational Displacement.



**Figure 19** The Bridge Deck Response under Different Intensities and Directions; (a) Maximum Axial Forces (kips), (b) Maximum Shear Forces (kips), and (c) Maximum Bending Moment (kips-in).



**Figure 20** The Bridge Column Response under Different Intensities and Directions of Earthquake; (a) Maximum Axial Forces (kips), (b) Maximum Shear Forces (kips), and (c) Maximum Bending Moment (kips-in).



might produce possibilities of the superstructure to displace from the seat width at the expansion joint and end joints;

- 7) The low intensity earthquake with the combination of earthquake directions will not affect the bridge. Low intensity earthquake is defined as earthquake with intensity level less than 0.15g, moderate intensity, between 0.15g to 0.35g and high intensity, above 0.35g. The bridge under study is within the high intensity earthquake level which is 1.17g. From the observation, the low intensity earthquake together with the different combination of earthquakes will not damage the bridge; and,
- 8) The moderate and high intensity earthquake and combination of the earthquake directions contribute to the bridge seismic deficiencies. Intensities and combination of different earthquake directions may contribute to the seismic deficiencies especially for bridges in the moderate to high intensity regions where the columns suffered the most damage. In this case study, the decks are still below the yield point, but the columns may fail in bending.

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Lumpur penggerudian yang memenuhi lubang telaga menggunakan satu ukuran hidrostatik, keros hidrostatik ini biasanya 250 g/cm<sup>3</sup> (124 kPa) hingga 490 g/cm<sup>3</sup> (218 kPa) lebih tinggi dari pada air tawar. Tekanan hidrostatik yang lebih tinggi ini mengialang berakibat merosotnya lapisan pasir di lubang telaga. Secara umum, jika tekanan hidrostatik kurang daripada ukuran formasi, maka formasi akan masuk ke dalam lubang telaga. Frekuensi yang dikenal sebagai tendangan 4-6 m jika gagal dikawal dengan segera dan berkesan, boleh menyebabkan berlakunya sebarang jenis kerosakan yang boleh mempercepatkan alam sekitar, kebiliargan sumber petroleum, memalukan hari-honda dan kehilangan nyawa pekerja (Adam et al., 1980).

Seperti industri hujam petroleum negara lain, industri hujam petroleum di Malaysia juga menggunakan barit (nama kimia barium sulfat) sebagai bahan pemberat untuk melindungi kemampuan lumpur penggerudian kepada nilai yang dikehendaki. Barit yang diguna diperolehi dari sumber-sumber tempatan. Secara umum, barit merupakan bahan lengai dan mempunyai keunjutan rata-rata antara 4.2 dan 4.3, nilai 4.2 kali lebih berat daripada air. Kekerasannya ialah antara 2.5 dan 3.5 mengikut skala Mohs (10). Lenggokan barit boleh didapati di beberapa kawasan (12), seperti di Sungai Terengganu, Tasik Cuni, Pahang, Sungai Pedeh, Krantan, Bukit Panchor, Kelantan dan sebagainya. (Fahmy).