Influence of Soil-Structure Interaction on Seismic Behaviour of Reinforced Concrete Integral Bridge Piers

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SUMMARY:

The current study is focused on the effect of Soil-structure Interaction (SSI) on seismic behaviour of Reinforced Concrete bridge piers. A symmetric RC bridge with piles, underlying soil is modelled using 4-noded quadrilateral elements with the use of OpenSees. Lysmer-Kuhlemeyer boundary conditions were imposed along soil boundaries. Two ground motions, Loma Prieta (broadband) & Northridge (narrowband) earthquakes are applied at the base of soil domain and the response of structure is studied. Based on the analysis, the influence of Peak Ground Acceleration (PGA) on response of bridge is obtained by varying the PGA of the ground motions. It was found that the vertical reaction and inertia force from superstructure significantly influence the forces in piers. The forces in the pier adjacent to the abutment were higher are due to the presence of abutment. The response of the pier was observed maximum at the time of PGA of ground motion.

Keywords: soil structure interaction, integral bridge, bridge design, absorbing boundary, OpenSees

1. INTRODUCTION

Seismic soil-structure interaction is an essential part of realistic seismic analysis of structures. During strong earthquake shaking, the underlying soil response influences the motion of the structure and structural response, in turn, influences the motion of the soil (Kramer, 1996). Ground motions that are not influenced by the presence of structures are referred to as free-field motions. When a rocky stratum at the ground surface is subjected to earthquake shaking, the extremely high stiffness of the rock constrains the rock motion to be very close to the free-field motion. Structures founded directly on rock are considered to be fixed-base structures. On the other hand, the same structure would respond differently if supported on a soft soil deposit. Although soil-structure interaction alters the seismic response of a structure from the fixed base action, most of the current analyses tend to ignore this aspect due to scare pertinent experimental data, computational intensiveness and lack of easy application to design.

In the past research, nonlinear finite element analyses of integral bridges, with soil-structure interaction, have been carried out and the observations include rocking at the base of piers (Zhang et al., 2008), significant longitudinal and transverse displacements of the bridge systems due to permanent ground deformations (Elgamal et al. 2008) and influence of liquefaction induced lateral spreading on seismic demands (Conte et al. 2002).

In the present study, a symmetric long span integral abutment bridge is modeled with the underlying soil domain using the computer program OpenSees (Opensees, 2010). Two recorded earthquake ground motions, having broadband and narrowband characteristics, are applied at the base of the soil domain. The effect PGA and characteristics of ground motion on the response of structure are studied by using linear time history analysis. Effect of liquefaction on pier response was not considered.

2. COMPUTATIONAL MODELING

The Bridge Pier Soil (BPS) system was modeled using the graphical pre-processor user interface of GID version 10.0.4 (GID, 2010). The assigned geometric and material properties, along with the relevant boundary conditions, are discussed in the subsequent sections.

2.1 Modeling of Bridge Structure

In the present study, the Humboldt Bay Middle Channel (HBMC) Bridge near Eureka in northern California, USA is modeled. The bridge is 330m long, 10m wide and 12m high (average height over the mean water level). The superstructure consists of four precast prestressed concrete I-girders and cast-in-place concrete slabs (Fig.2.1). Two seat-type abutments and eight bents, each bent consisting of a single column and hammer head cap beam, form the supporting part. The heights of the piers (numbered #1 to #8 from left to right) range from 11m to 14m. The deep foundations consist of driven precast prestressed concrete pile groups supporting pile caps (Zhang et al., 2008).

All the members of the superstructure, substructure and foundation were discretized using 2-noded linear beam column elements with 3 Degrees of Freedom (DOFs) at each node, namely two translational DOFs and one rotational DOF. Each span was discretized into 5 elements. The average height of beam column element in piers was taken as 12m. The pile elements were assumed to extend up to 5.2m below ground level (Gentela, 2011).

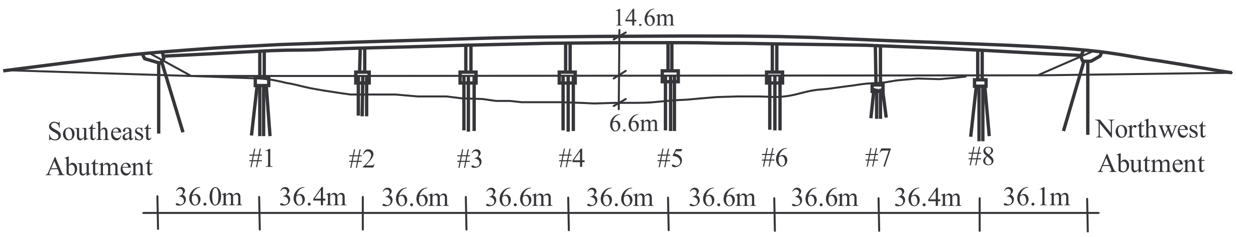

Figure 2.1. Piers and pile groups in elevational view of bridge (Elgamal et al., 2010)

2.2 Modeling of Foundation soil

The entire soil domain was broadly divided into two parts, namely (i) backfill soil above the natural ground level, and (ii) foundation soil lying below the natural ground level. Any soil deposit, on top of the footing level, has not been considered for modeling. Backfill-abutment interaction, due to possible abutment movement, has also not been considered in the present study.

The depth and horizontal length of foundation soil domain were 16m from the ground level (up to the bedrock level) and 1050m respectively (Zhang et al., 2003). Plane strain condition of the foundation soil was mobilised by using four-noded, bilinear, isoparametric finite elements (2 translational DOFs at each node) for modeling. A uniform mesh size of 0.4m×0.4m was used throughout the domain to obtain the response with sufficient accuracy. The mesh size was decided based on the restriction of element size to $1/8^{th}$-$1/10^{th}$ of the shortest wave length in the direction of wave propagation (Kuhlemeyer and Lysmer, 1973). The thickness of the domain was taken as 6.10m (Gentela, 2011).

2.3 Modeling of Soil-Structure Interfaces

Proper modeling of the soil-structure interface is very much important when permanent displacement and debonding at the interface influence the response of the structure significantly. Therefore, newly developed zero-length node-to-node contact elements were employed at the soil-pile interfaces to simulate the earthquake induced actions at the base of the piers (Fig. 2.2) (Gentela, 2011). Modulus of elasticity of each element was taken to be equal to that of soil for obtaining better interaction. The contact elements are connected between nodes having the same number of DOFs. As each soil element node has 2 DOFs and each pile node 3 DOFs, a set of dummy nodes with 2 DOFs at each node, were introduced at all three soil-pile interfaces. Thus, the contact elements were connected between the soil nodes and the corresponding dummy nodes, and the dummy nodes were connected to the corresponding pile nodes by equal DOF constraints along both length and depth of domain. Along the interfaces, all these nodes (soil, pile and dummy) created at the same physical location.

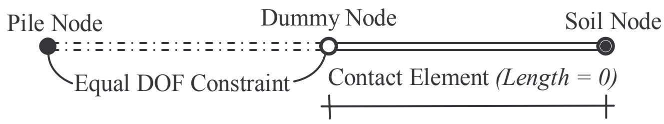

Figure 2.2. Connectivity of nodes at soil-pile interface

2.4 Boundary Conditions

In a finite element (FE) model of bounded soil domain, wave energy tends to get trapped inside the computational domain without propagating away. But in reality, part of the energy propagates beyond the domain of the computational model and gets dissipated in the soil domain outside; that energy never returns back into the domain. Thus, the modeling of semi-infinite soil domain using a fixed or spring boundary fails to simulate the actually observed radiation boundary conditions and results in spurious oscillations caused by the reflection of waves along the boundaries (Zhang et al., 2008).

To eliminate the spurious reflections of radiating waves, Lysmer-Kuhlemeyer (L-K) (Lysmer and Kuhlemeyer, 1969) boundary conditions were invoked; these ensured the transmission of incident wave transmitted entirely into the computational domain without any distortion and no spurious reflection from the boundary (Zhang et al., 2003). At each boundary node of the bottom domain, dashpots were attached along horizontal and vertical directions to absorb the shear waves and the compression waves respectively. The coefficients of the horizontal and vertical dashpots are obtained as $C_s = \rho V_s A$ and $C_p = \rho V_p A$, respectively, where V_s and V_p are the velocities of secondary and primary waves respectively, and A the tributary area of the node. Similarly, at each node along the lateral boundaries, horizontal and vertical dashpots of coefficients C_p and C_s are attached to transmit the compression and shear waves respectively (Kolay, 2009).

At every boundary nodal location, two nodes each having 2 translational DOFs were created. The DOFs were restrained along both directions in one of the nodes and the other node had displacement constraints along the two directions. These two nodes were connected using zero-length elements to simulate the Lysmer dashpot. Modulus of elasticity of soil was used to simulate the linear behaviour of the zero-length elements. After creating these elements, constrained nodes were connected to soil nodes using equal DOF constraint as mentioned in Section 2.3 (Gentela, 2011).

3. ANALYSIS AND RESULTS

The effects of foundation soil on the seismic response of the system were investigated through linear dynamic analysis under ground motions taken from PEER strong ground motion records (PEER, 2010). Before performing dynamic analysis, eigenvalue analysis was carried out as part of free vibration study.

3.1 Free Vibration Analysis

The natural periods and natural modes shapes of vibration of the soil domain were obtained in OpenSees. In that study, the soil material model was set as linear elastic and the bottom boundary nodes were restrained against horizontal and vertical displacements, whereas, the nodes at the lateral boundaries were kept free against displacements. These modifications were required to remove the singularity of global stiffness matrix under L-K boundary conditions. The free-field model was analyzed to get the natural frequencies and mode shapes (Table 3.1). The fundamental period (T_1) of any soil deposit of infinite horizontal extent can be approximately obtained as (Kramer, 1996),

$$T_1 = \frac{4H_{soil}}{V_s} \tag{3.1}$$

where, H_{soil} is the depth of soil deposit. Considering H_{soil} = 16m and V_s as 234.41 m/sec., T_1 was calculated as 0.273 sec which was only 2.41% more than that obtained from the eigenvalue analysis. Thus the considered horizontal extent of the soil domain was sufficient to simulate the response under vertically propagating plane shear waves.

Table 3.1 Natural frequencies and periods of vibration of soil domain

Mode	Circular Frequency (rad/sec)	Natural Frequency (Hz)	Natural Period (sec)
Mode 1	22.443391	3.571976	0.2799570
Mode 2	22.443404	3.571987	0.2799568
Mode 3	23.057983	3.669792	0.2724950
Mode 4	23.192339	3.691127	0.2709164
Mode 5	23.405080	3.725034	0.2684539
Mode 6	23.688007	3.770063	0.2652475

3.2 Analysis Procedure

The step-wise staged analysis procedure (Zhang et al., 2008) for seismic analysis of the BPS system is discussed as follows:

1) The base nodes of the BPS system were restrained against translations along both horizontal and vertical directions; the lateral nodes were restrained against translation along longitudinal direction only. The soil material was set as linear elastic and the self weights of soil and bridge were applied statically in one single step. The static equilibrium was achieved through iterations, and the lateral support reactions were obtained along bottom and lateral boundaries.
2) Next, the displacement restraints at both the boundaries were removed. Reactions, obtained in Step 1, were applied at the corresponding boundary nodes (which were restrained earlier) along the appropriate directions.
3) The L-K boundary conditions were invoked by attaching horizontal and vertical dashpots at the boundary nodes. The static equilibrium was again achieved through iteration.
4) Finally from the static equilibrium position under gravity loads, the seismic excitation was applied as equivalent nodal shear forces along the base nodes.

Two response parameters of the BPS system are presented, namely (i) displacement at the top of pier and (ii) forces at the critical sections of pier.

3.2.1 Horizontal displacements

In the case of Loma Prieta earthquake, two PGAs namely 0.18g and 0.66g were considered. The maximum displacements at the top of pier were found to vary from 5.4mm to 5.1mm and 16.4mm to 16.5mm for PGAs of 0.18g and 0.66g respectively, from end to middle pier. The maximum displacement occurred at around 5.64sec of the motion for both the PGAs. In case of Northridge ground motion, the same two PGAs (0.18g and 0.66g) were considered. The maximum displacement at the top of pier was found to vary from 2.7mm to 3.2mm and 8.4mm to 8.9mm for PGAs of 0.18g and 0.66g respectively, from end to middle pier. These were less than those obtained for Loma Prieta earthquake. These maximum displacements and the PGAs occurred at the time instants of 14.82 sec and 9.74sec respectively for both the motions.

For the Loma Prieta ground motion, the range of frequency for the maximum energy input was 1.1Hz - 1.3Hz. The soil domain and the bridge structure had natural frequencies around 3.57Hz and 1.97Hz respectively. Thus, a significantly high response of the integrated system was observed. At the time instant of the occurrence of PGA, marginally less response was observed. Due to the presence of abutment, the displacement of pier near to that was found to be less as compared to other pier.

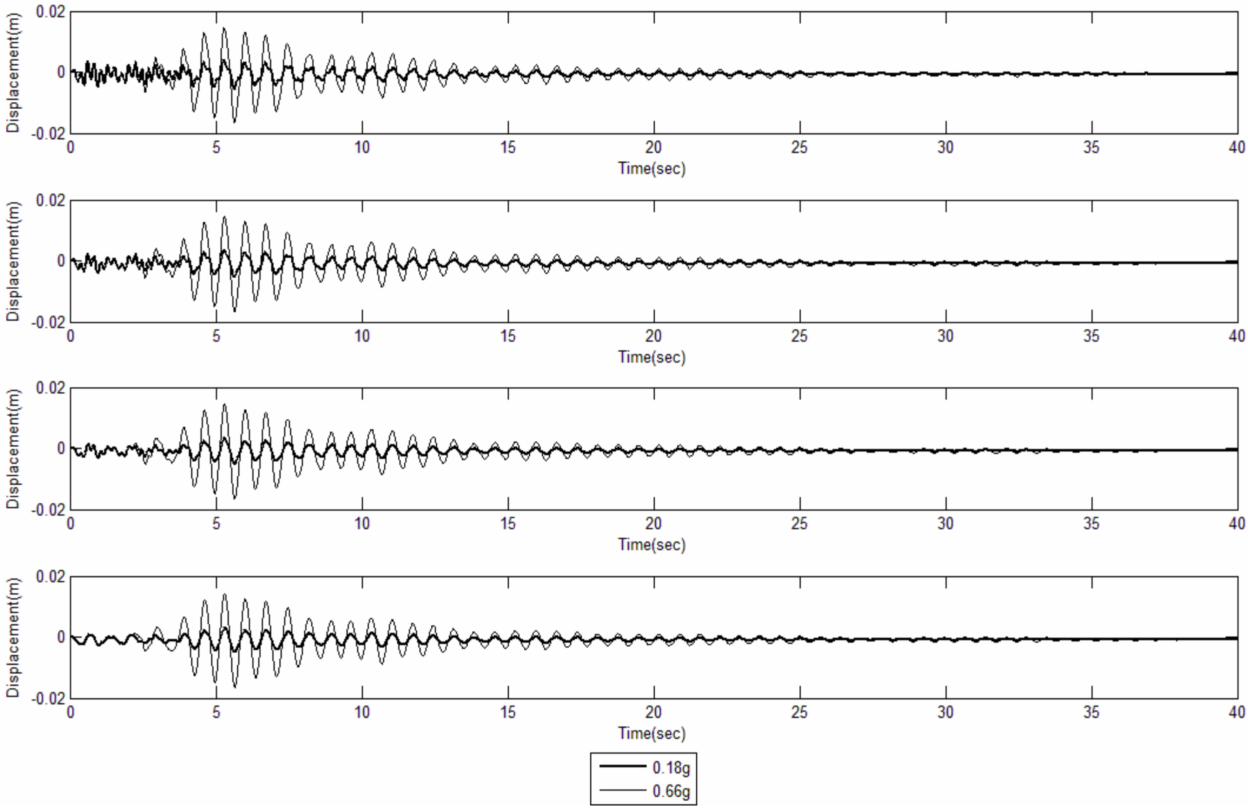

Figure 3.1. Variation of displacements in Piers #1, #2, #3 and #4 for Loma Prieta ground motion with PGAs

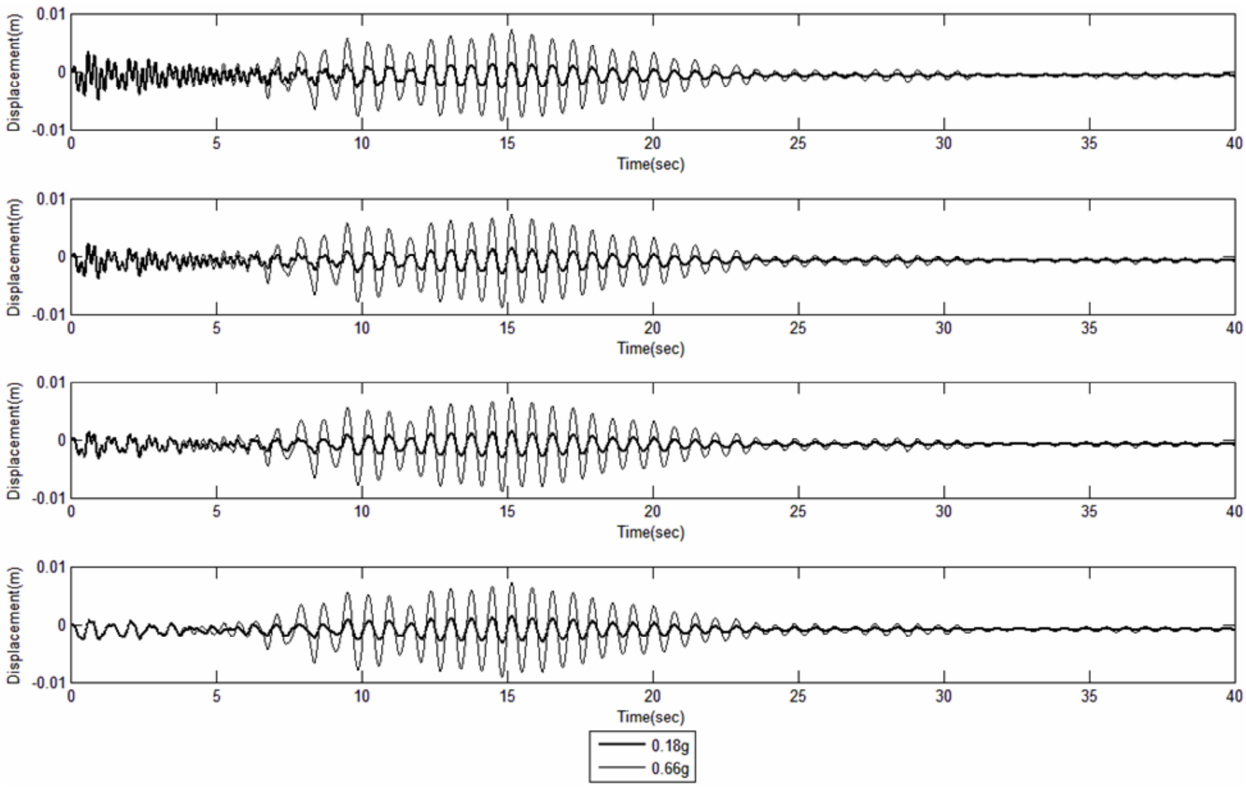

Figure 3.2. Variation of displacements in Piers #1, #2, #3 and #4 for Northridge ground motion with PGAs

Abutment dimensions were more than the pier so it could be able to provide resist against displacement. The displacement of middle pier was obtained more because of superstructure lateral deformation. The span of super structure is about 36m so that may deform in lateral direction also.

In the case of narrow band motion (Northridge earthquake), response was found to be less compared to the broadband motion (Loma Prieta earthquake) because of their frequency contents. Finally, the response for severe shaking was found to be 3 times more that obtained for low level of shaking and PGA plays a major role in response of structure. In Loma Prieta earthquake, the wave frequencies were more than those for Northridge. The natural frequency of the soil was also more than that of structure. Thus, during Loma Prieta earthquake, soil might vibrate more due to possible frequency matching of waves and soil. For Northridge earthquake, wave frequencies were observed to be less than those for Loma Prieta earthquake. But the wave frequencies of Northridge ground motion could have more chances of matching with the natural frequency of the bridge structure. This might lead to significant vibration of the structure during Northridge earthquake. Fig. 3.3 shows the variation of displacements for both earthquakes of same PGA. Severe level of shaking was considered for comparative study. In the case of broadband motion, the dominant frequencies of the motion are widely spaced with the possibility of the soil natural frequency lying in that domain. At certain time instants, these two frequencies might come closer and this may lead to higher response in case of broadband motion. In presence of soil, the response of the bridge structure is governed primarily by the soil response.

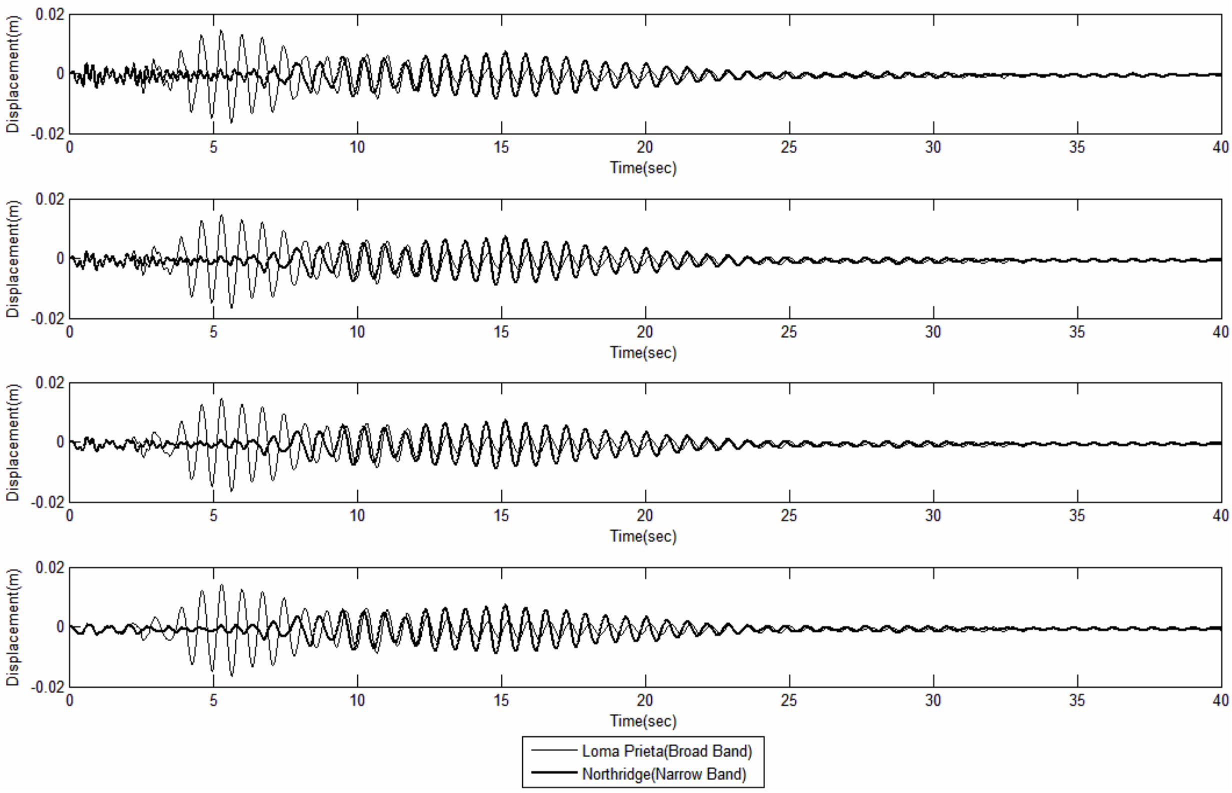

Figure 3.3. Variation of displacements at top of Piers #1, #2, #3 and #4 with frequencies of ground motions

3.2.2 Forces at the Critical Section of Pier

In the case of Loma Prieta Earthquake for 0.18g PGA, the maximum shear force (SF) and bending moments (BM) were found to vary between 1545kN to 287.4kN and 14840kNm to 2455kNm respectively from end to middle pier (Figs. 3.4 and 3.5). These peak values were observed at the time instant of 0.72sec during the transient part of the response. At the instant of PGA, i.e., around 5.34sec, these values varied from 1421kN to 239kN and 13160kNm to 1719kNm respectively which were less than the values observed in the transient part. In case of 0.66g, the maximum SF and BM were found to vary between 1545kN to 290.7kN and 14850kNm to 2478kNm respectively from end to middle pier in transient part of the response; these were around 1.5 to 2 times more than those observed for 0.18g case. So the maximum response was considered at the time instant of PGA, i.e., around 5.32sec. At this instant, these values varied from 1850kN to 545.4kN and 15950kNm to 4747kNm for Pier #1 to #4 respectively. These were 1.5 to 2 times more than those observed in case of 0.18g PGA. Generally, the transient response is not considered in design.

In the case of Northridge earthquake, for 0.18g the maximum SF and BM were found to vary between 1546kN to 281.5kN and 14840kNm to 2401kNm from end to middle pier in transient part of the response (Figs. 3.6 and 3.7). In case 0.66g, the maximum SF was found to be 1645kN to 280.5kN from end to middle pier during transient response which were marginally lesser than those observed for Loma Prieta earthquake. However the maximum SF and BM were observed at the PGA. The maximum SF and BM were obtained as follows. For 0.18g PGA, the SF and BM varied from 1213kN to 89.77kN and 5425kNm to 614.5kNm respectively at the time instant of 14.86 sec.

As observed, the pier next to the abutment drew more shear force and bending moment as compared to other piers. For Pier #1, the observed SF and BM were attained at initial time only and kept on deceasing up to 20sec, possibly due to the presence of abutment next to it and large inertia force transferred from superstructure at the abutments. Due to large dimensions of abutment as compared to the piers, the abutments tend to vibrate much in isolation and that possibly affects the response of the adjacent pier. Rayleigh damping was also applied to the system which might have led to the gradual decrement in the response of Pier #1. Superstructure vertical reaction would also increase the SFs and BMs in that pier.

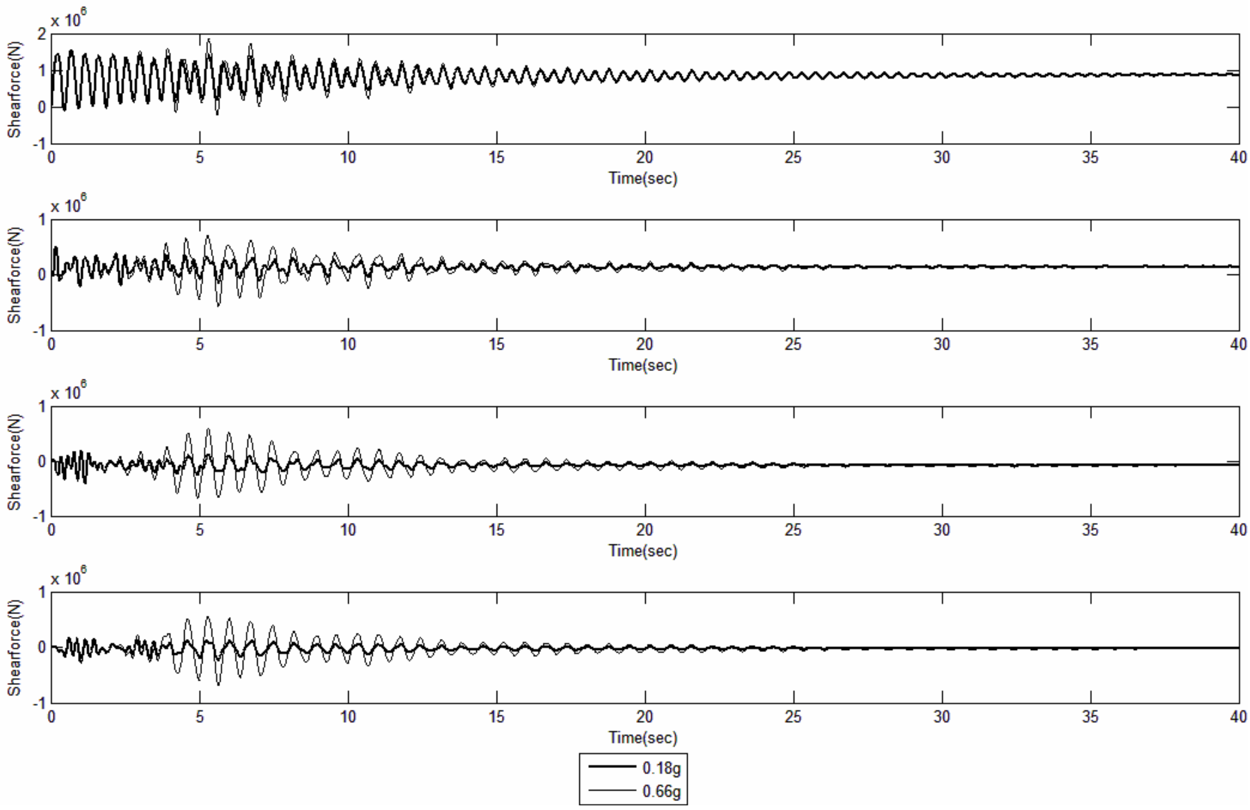

Figure 3.4. Variation of shear forces in Piers #1 to #4 for Loma Prieta ground motion with varying PGAs

In the case of narrow band motion, response was found to be less as compared to broadband motion as obtained earlier. In the case of broadband motion, the maximum value of response was attained earlier as compared to narrow band motion because of their frequency contents as discussed for bridge without soil. Fig. 3.8 to Fig. 3.9 show the variation of SFs and BMs with given ground motions. In the case of Loma Prieta ground motion, there was a likely possibility of wave frequencies close to the

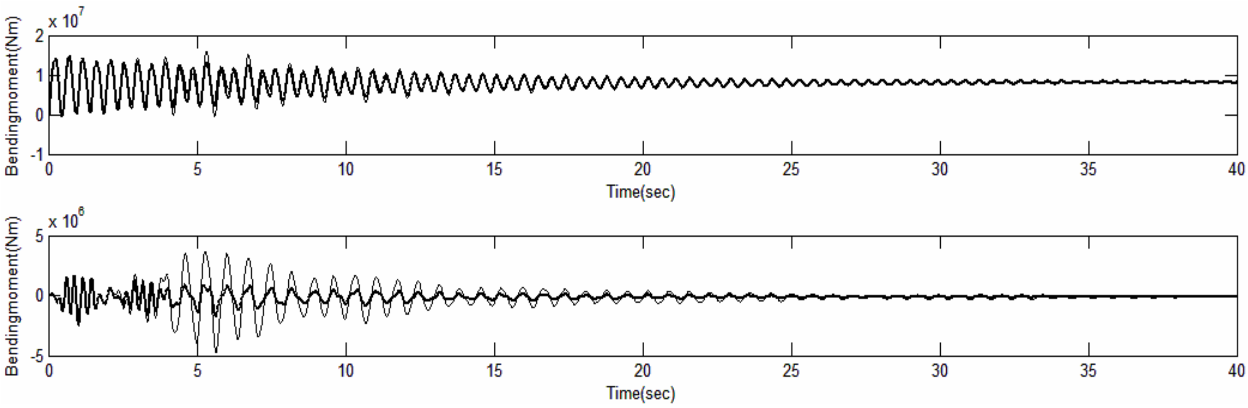

Figure 3.5. Variation of bending moments in Piers #1, #2, #3 and #4 for Loma Prieta earthquake with PGAs

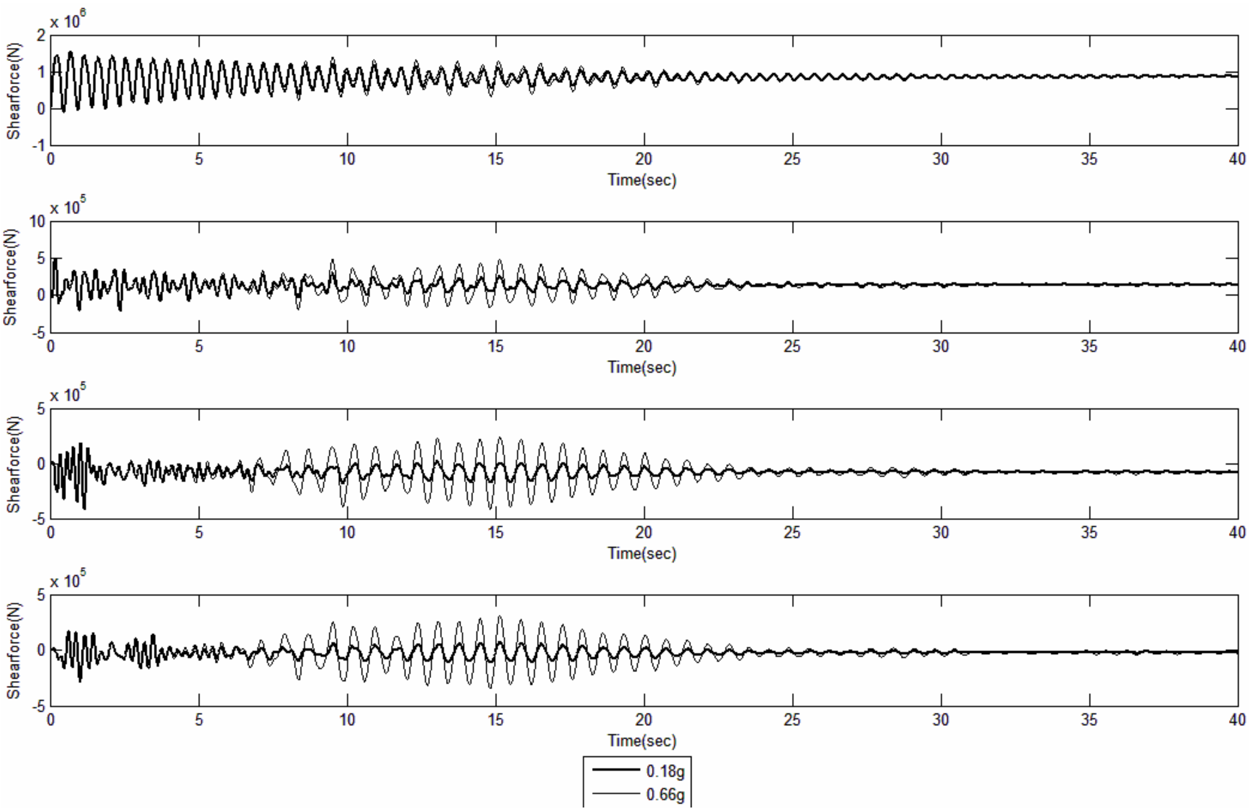

Figure 3.6. Variation of shear forces in Piers #1, #2, #3 and #4 for Northridge ground motion with PGAs

natural frequency of the soil as obtained from eigenvalue analysis. Natural frequency of soil was observed higher than structure. If high frequency waves compared to Northridge motion were present, there could be more chances of matching the frequencies with soil. In case of broadband motion, the dominant frequencies are widely spaced as mentioned earlier. Section forces under broadband motion are observed to be more than those experienced for narrow band motion.

4. CONCLUSIONS

The following salient conclusions were drawn based on the present study:

1) Variation of PGA of a ground motion plays a major role in the response of structure. In severe level of PGA of ground motion would be more than the low level of shaking.

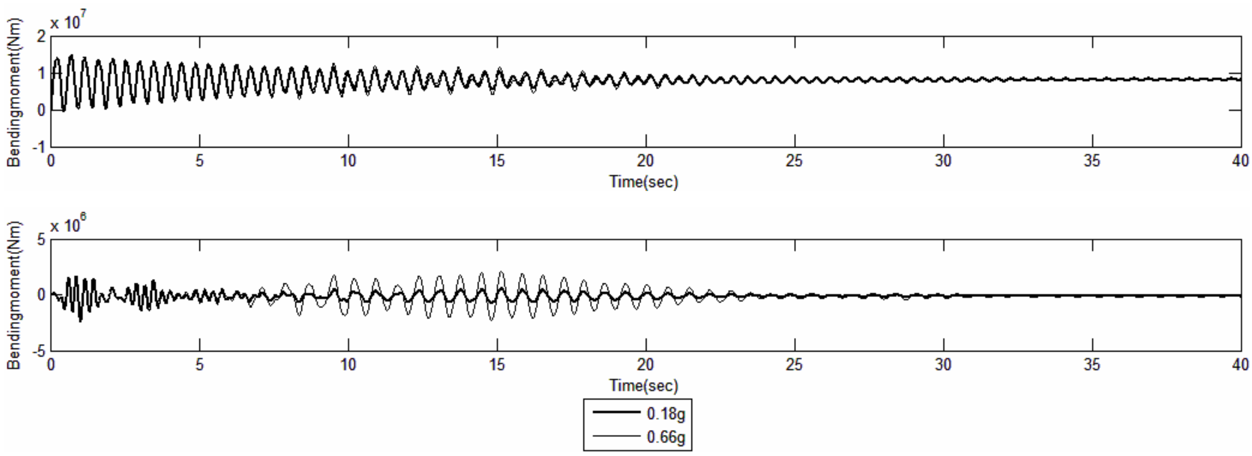

Figure 3.7. Variation of bending moments in Piers #1 and #4 for Northridge ground motion with PGA.

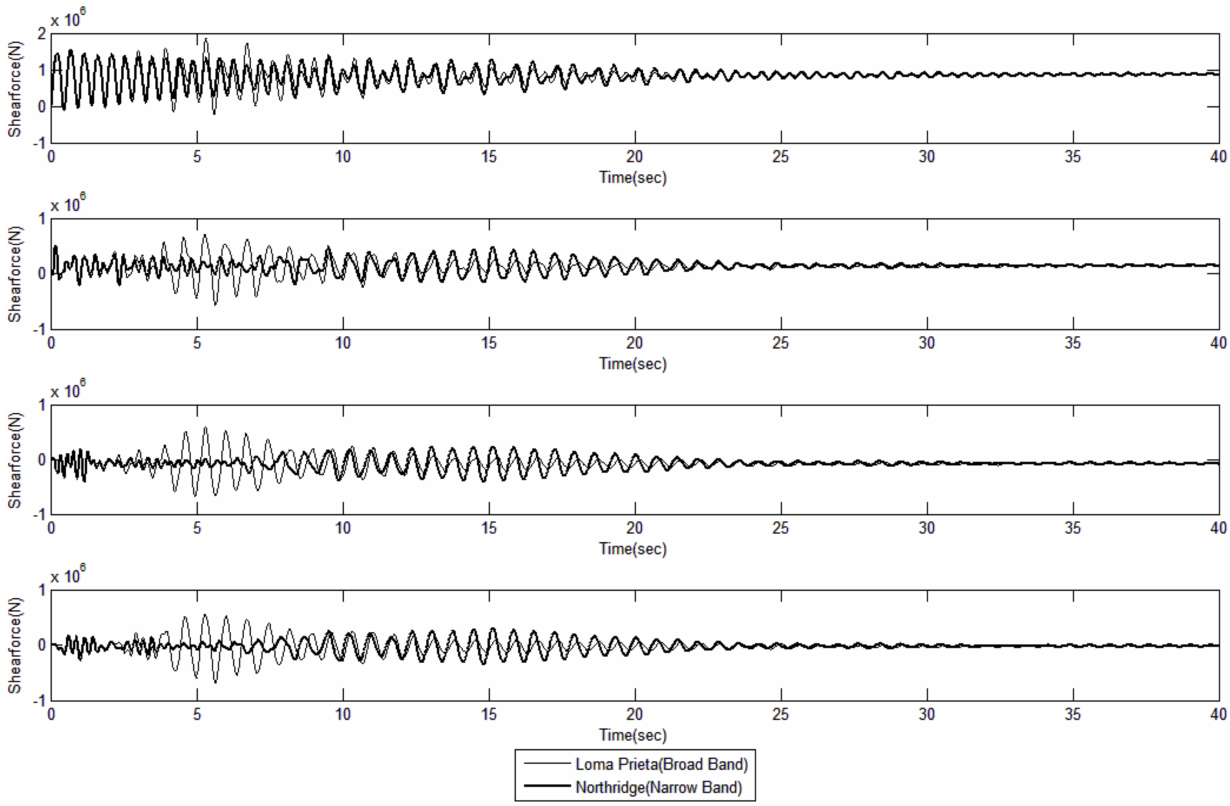

Figure 3.8. Variation of shear forces in Piers #1, #2, #3 and #4 with ground motions of different frequencies

2) The response of the pier adjacent to abutment was found to be more due to the presence of abutment.
3) Vertical reaction and inertia force from superstructure plays a significant role in increasing the forces at the critical section of piers.
4) Response of the structure was found maximum at the time where the ground motion attaining maximum value, *i.e.*, PGA.
5) The characteristics of ground motion were shown considerable influence on structure response.
6) Broadband motion was shown more response than narrowband due to the widely spaced dominating frequency.
7) Frequency of ground motion was also an important parameter to be considered in response of structure.

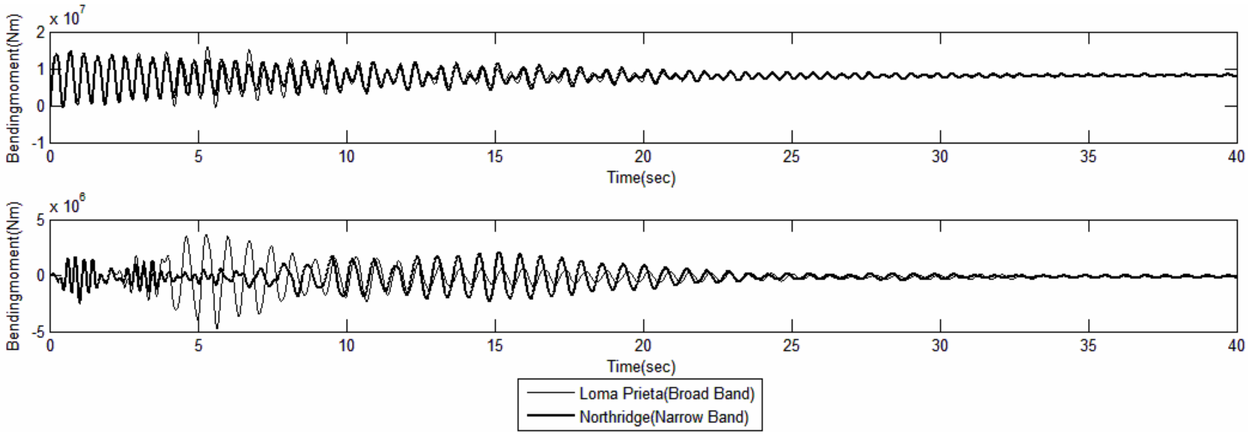

Figure 3.9. Variation of bending moments in Piers #1 and #4 with frequencies of ground motion

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