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Seismic Responses of Concrete Building Subjected to Out-of-phase Ground Motions

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Abstract: Seismic performance of a building is commonly evaluated by applying same design ground motions at each building foundation. However, local soil conditions beneath a building likely vary, and these variations could result in out-of-phased design ground motions at each of the foundation locations. In this study, building's responses during earthquakes were analyzed and compared using same and out-of-phase ground motions. The building is 10-story, 90m-wide, reinforced concrete structure supported on isolated footings with tie beams. Dynamic time response analyses were performed using five earthquake records which were scaled to a design response spectrum for a location in Surabaya. Seismic modification factor, R, of 8 was used. The results indicate that the use of out-of-phase ground motions does not have significant impacts on building inter-story drifts; it results, however in significantly higher column base shears and tie beam axial forces compared to those calculated using same ground motions.

Keywords: Local soil effects; out-of-phase ground motions; interstory drift ratio; base shear; tie beam.

Introduction

Observations on earthquake ground motions recorded during past earthquakes have shown the effects of local soils on ground motion intensity and frequency contents. Figure 1 illustrates the effects of Holocene soil deposits on peak ground acceleration (PGA), developed based on more than 700 earthquake records, where pronounced de-amplification of rock motions at high intensity can be expected [1]. Also shown on this figure are the effects of soil deposits in the San Francisco Bay Area and Mexico City during the 1989 Loma Prieta and 1985 Mexico City earthquakes.

Saxena et al. [2] conducted a study on the effects of spatial differences in ground motions to a long bridge structure. The bridge is 492 m long and supported on multiple foundations along its length. It was analyzed by considering the variations in soil condition, foundation, and earthquake wave velocity. The results indicate higher forces and ductility requirements in structure when subject to different or out-of-phase ground motions, compared to those calculated using same ground motions along the bridge.

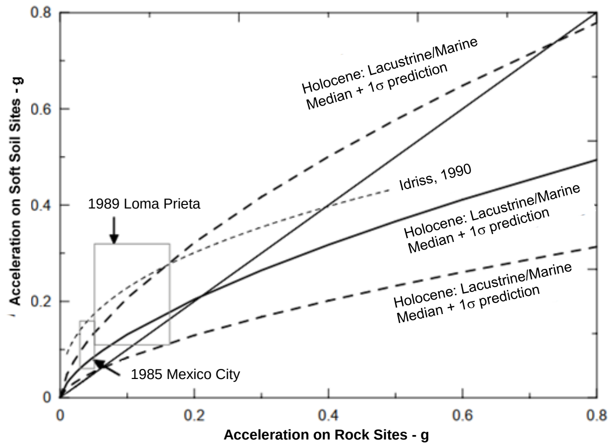

Figure 1. Deamplifications of Peak Ground Acceleration for Holocene Deposits (adapted from [1])

Other studies on the effects of soil types to earthquake ground motions and building seismic responses can be found in references [3-6].

For structures with extended dimensions, such as long buildings and bridges, it is conceivable that local soils beneath the structure will vary both in their soil types and vertical and horizontal distributions. These variations could affect the ground motions at foundation support locations and thus the behavior of the superstructure. Three factors need to be considered for accounting the spatial differences in earthquake ground motions: wave passage effect, incoherence effect, and local soil effect. For buildings with extended dimensions (width and/or length), the most influential factor is the local soil effect. In this case, local soil variations, both in terms of soil type and soil depth or thickness, can affect the ground motions

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received by building foundations. Figure 2 illustrates an example, where the presence of a soft/loose soil layer with variable thickness could produce different or out-of-phase input ground motions at the three (3) foundation support locations (i.e. different ground motions at column 1, 2 and 3 locations). The building responses subject to these out-of-phase ground motions could significantly be different than the responses when the same ground motions are applied, as commonly assumed in practices.

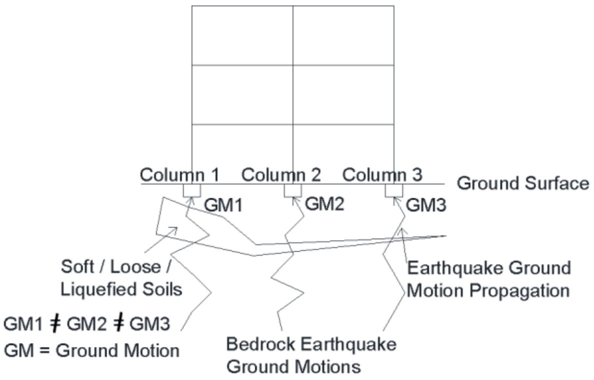


Figure 2. Effects of Local Soil Variations on Foundation Ground Motions

In this study, a 10-story, 90m-wide, reinforced concrete structure supported on isolated footings with connecting tie beams was used to evaluate the effects of out-of-phase ground motions on structural responses. Dynamic time response analyses were performed, and the inter-story drifts and column base shears calculated using same and out-of-phase ground motions were compared to determine the effects of out-of-phase motions. Axial forces induced in the connecting tie beams were also compared.

Structural and Foundation Soil Models

In this study, the building was modeled as a 2-dimensional (2D) structural frame, 10-story high reinforced concrete structure with a total width/length of 90 meters. The story height is 3.5 meters and the span between columns is 6 meters. The structure, founded on the three (3) hypothetical soil models or profiles, as shown in Figure 3, is used to evaluate the effects of out-of-phase ground motions on structural responses. The top figure depicts a building founded on a variable soil condition, a condition most likely encountered in a real world. The bottom figures illustrate the two (2) soil conditions commonly assumed in practices, where the soft clay layer (layer 2) is taken either as a

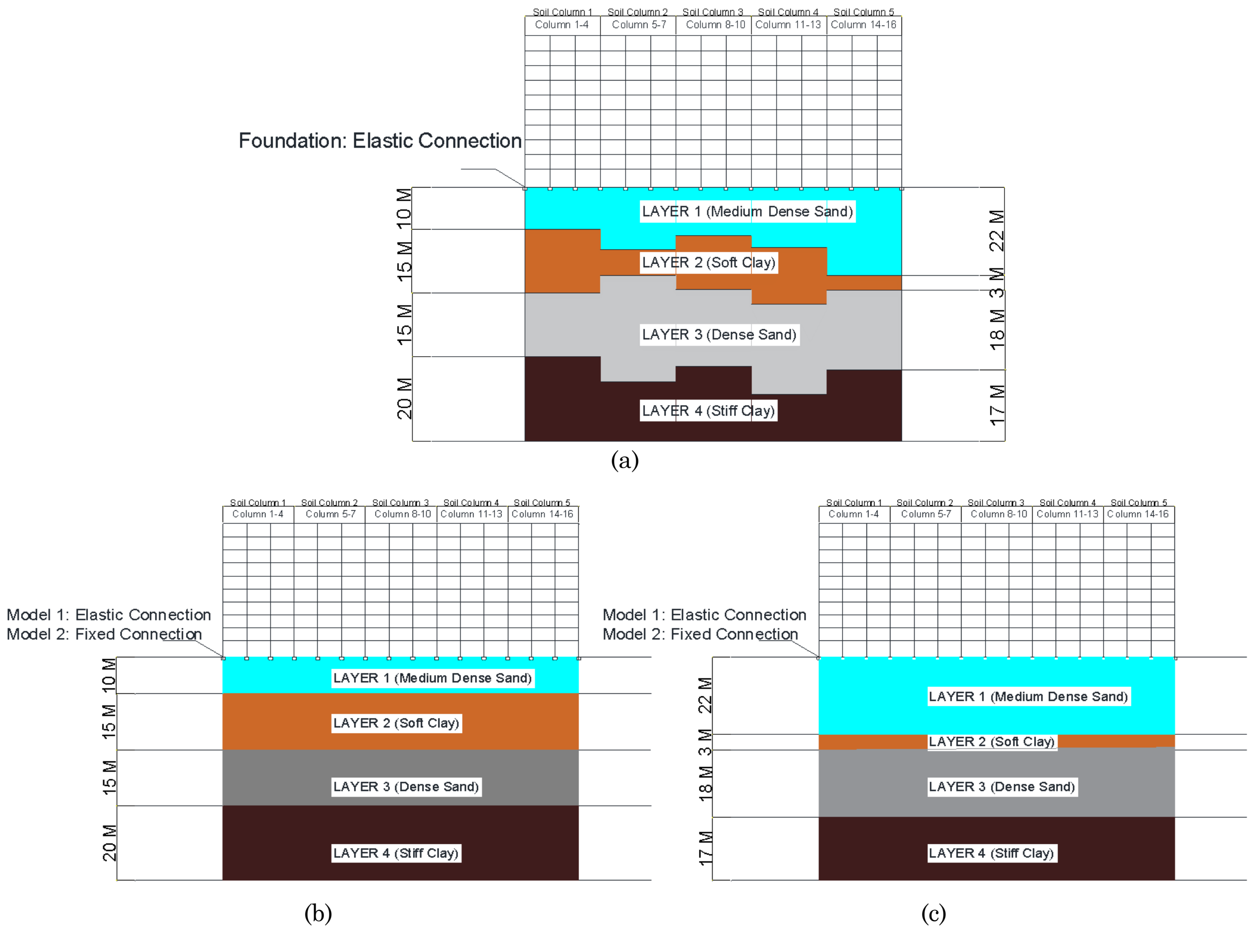


Figure 3. Structural and Foundation Soil Models used for This Study: a) Building on Variable Soil Conditions; b) Building on Uniform Soil Conditions with Thick Soft Clay; c) Building on Uniform Soil Conditions with Thin Soft Clay

uniform thick or thin soil layer across the entire building footprint. Table 1 summarizes the assumed soil engineering properties of the foundation soils.

Table 1. Soil Specifications used in Soil Profiles

Profil Tanah	Layer 1 (Medium Dense Sand)	Layer 2 (Soft Clay)	Layer 3 (Dense Sand)	Layer 4 (Stiff Clay)
γ_{dry} (kN/m3)	18	18	18	18
γ_{sat} (kN/m3)	20	20	20	20
ϕ *(degrees)*	32	0	34	0
S_u (kPa)	0	20	0	100
Vs (m/s)	250	150	500	700

* γ_{dry} = dry density, γ_{sat} = saturated density, ϕ = friction angle, S_u = undrained shear strength, V_s = shear wave velocity.

The analysis consists of two (2) major steps: 1) development of input earthquake ground motions for structural analysis by considering variable soil conditions and 2) dynamic time response analysis for evaluating the building responses during earthquakes. These analysis steps are discussed as follows.

Earthquake Ground Motions

One-dimensional (1D) site response analysis was conducted to develop the ground motions at the ground surface that were used as inputs to the structural time response analysis. The analysis was performed using the computer program Deepsoil [7] and the non-linear, modified hyperbolic, MKZ (Modified Kondner-Zelasko) soil model calibrated to the published back-bone curves (G/G_{max} and damping versus shear strains curves) recommended by Darendali [8].

Design Response Spectrum

Site response analysis requires application of earthquake time histories at a depth corresponding to a reference site (typically, a rock or stiff/dense soil site). A depth of 60 m below ground at the bottom of soil layer 4 (see Figure 3) was selected as the reference depth with an average shear wave velocity to a depth of 30 meters (V_{s30}) of 700 m/s. The design response spectrum at the reference depth was then developed using the SNI 1726:2019 [9] requirements for a site class C (stiff soils) and a location in Surabaya, Indonesia. Figure 4 depicts the 5%-damped design response spectrum at the reference depth.

Earthquake Time Histories

Table 2 summarizes the five (5) earthquake time histories selected for the site response analysis. These time histories were selected from records of past earthquakes and scaled to match reasonably well the design response spectrum in the range of 0.5T to 1.5T, where T is the building fundamental period. The building fundamental period, T, was calculated to be 2.39 seconds, and therefore, the scaling was performed in the range from 1.2 to 3.6 seconds. Figure 5 plots the average response spectrum of the scaled time histories and the target design spectrum, showing good agreement in the period range of interest (1.2 to 3.6 seconds).

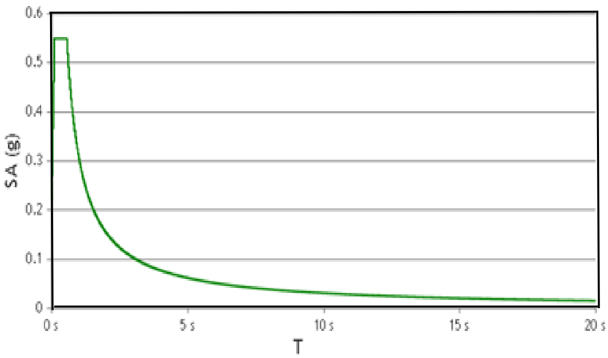


Figure 4. Design Response Spectrum for a Surabaya Location and Site Class C [10]

Surface Ground Motions

To determine the effects of varying local soil conditions on ground motions, the soil profile directly beneath the building were sub-divided into five (5) 1D soil columns, as shown in Figure 3 (top figure – soil columns 1 through 5). Soil layer 2 (soft clay) and layer 3 (dense sand) are expected to affect the ground motions the most, and hence, they were varied across the building to capture the variation of foundation soils.

The scaled time histories developed above were inputted at a depth of 60 m on a stiff soil reference site, and then propagated upward to obtain ground motions at the ground surface. The wave propagation analysis was performed on the five (5) soil columns discussed above. Since these soil columns have different soil layer thicknesses, the surface ground motions calculated for the five (5) soil columns will be different or out-of-phase, simulating the effects of local soil variations on ground motions. Figure 6 shows the calculated acceleration response spectra and displacement time histories at ground surface for the 1940 Imperial Valley earthquake. Similar results were obtained for the other earthquakes. From Figure 6, it can be observed that soil columns with thicker soft clay (i.e., soil columns 1, 3 and 4) generally produce lower spectral values for high-frequent (or low-period) motions, in line with expectations where the presence of thick soft soils tends to de-amplify high-frequency motions at high intensity.

Table 2. Selected Earthquake Time Histories

Earthquake	Year	Station	Direction	Magnitude	PGA (g)
Imperial Valley	1940	El-Centro Array	E-W	6.95	0.28
Kobe	1955	Takatori	E-W	6.90	0.62
Kocaeli	1999	Terkidag	E-W	7.51	0.13
Northridge	1994	Alhambra	E-W	6.69	1.67
Chi-Chi	1999	CHY002	E-W	7.62	0.34

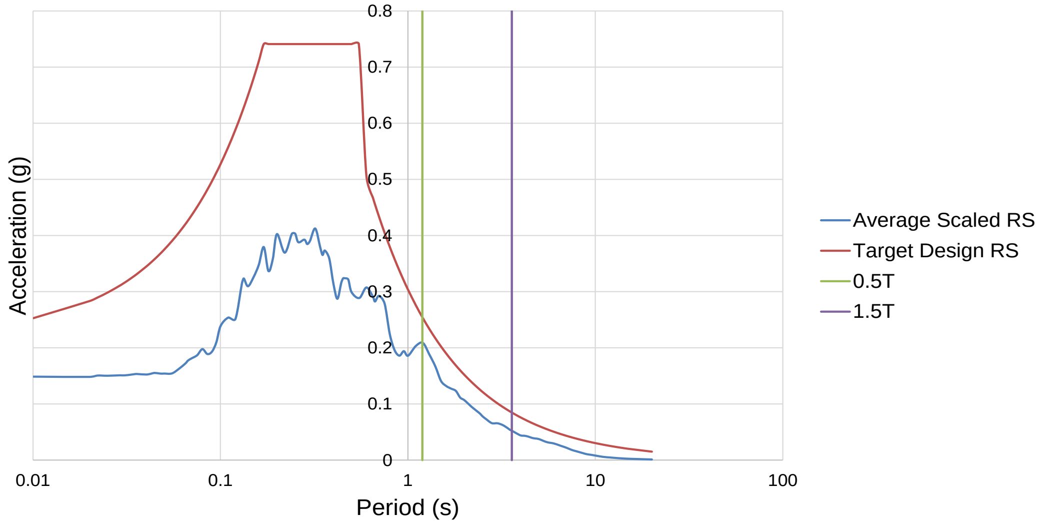
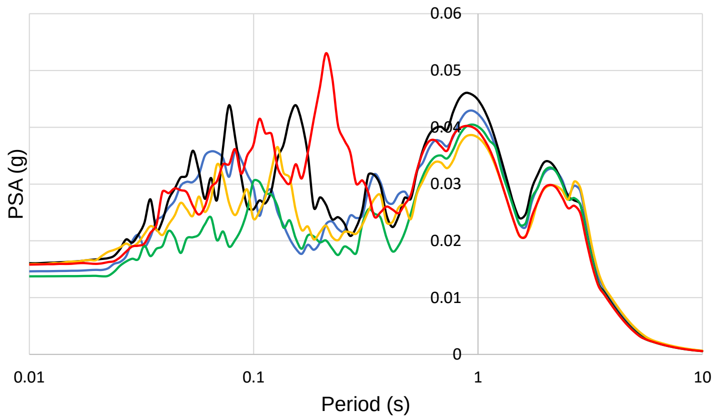
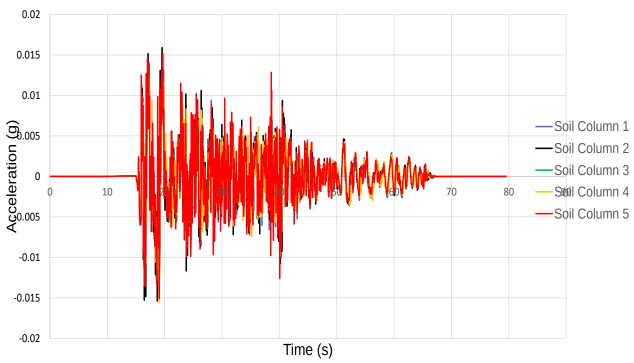


Figure 5. Comparison Between Average Scaled Response Spectrum and Target Design Spectrum



(a)



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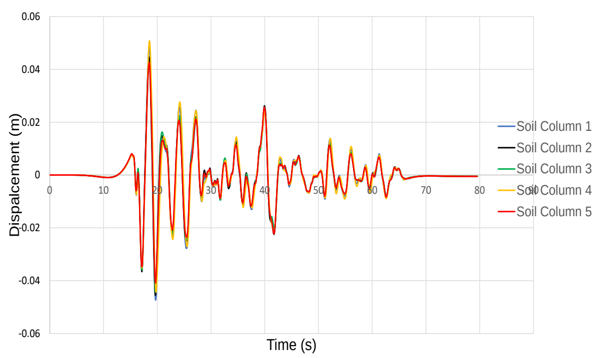


(c)

Figure 6. a) Calculated Ground Surface Response Spectra, b) Acceleration, and c) Displacement Time Histories for the 1940 Imperial Valley earthquake

Dynamic Time Response Analysis

Dynamic time response analysis was conducted using the computer program SAP2000 [11] to evaluate the building seismic responses under the same and out-of-phase earthquake ground motions. The building responses were assessed in terms of inter-story drift and column base shear.

The building was modeled as a 2D reinforced concrete frame and the calculated surface ground motions developed above were inputted at the column foundations. The structural elements (i.e., beams and columns) were modeled as linear elastic elements, with no hinges or plastic deformations allowed to develop during shaking. In reality, plastic hinges will likely develop in columns and/or beams during strong ground shakings, which in turn, will absorb the earthquake energy due to material ductility and will reduce the element forces. To account for this reduction in forces due to ductility in reinforced concrete, the input ground motions for the structural analysis were reduced by a Modification Factor, R-factor, of 8. A total of five (5) analysis case were considered for this study, as summarized in Table 3. The first three (3) cases consider a flexible foundation connection to the soils, while the other two (2) cases use a fixed foundation connection, as commonly assumed in practices.

Two sets of input earthquake ground motions were used: the same and out-of-phase ground motions. For the out-of-phase ground motions, the ground surface displacement time histories calculated for the five (5) soil columns were applied to the column foundations located within each soil column limits (i.e., ground motions for soil column 1 were applied to structure columns 1 through 4, ground motions for soil column 2 were applied to structure columns 5 through 7, etc., see Figure 3). For the same ground motions, the ground surface displacement time histories calculated for soil column 1 or 5 were inputted to all building footings. Soil column 1 or 5 was chosen because it represents the profile with the thickest or thinnest soft clay layer (soil layer 2).

Structure Dimensions

The dimensions of structural beams and columns were determined from the column spacings and tributary areas for dead and live loads. The dead loads include self-weights of columns, beams, slabs, tie beams, and walls, and live loads of 2.4 kN/m^2 and 0.96 kN/m^2 were used for floors and roof, respectively [12]. Based on these loads, beam size (including tie beams) of 300x500 mm was estimated. For columns, the dimension varies from 800x800 mm at the base to 300x300 mm at the roof.

Foundation Models

As stated previously, there are two types of foundation model used in the analysis: the flexible two-joint link foundation and the fixed foundation. The use of flexible or spring foundation is intended to capture the interaction between soils and footings during earthquakes. The springs were modeled to represent the responses of soils against vertical (K_z), lateral (K_x), and rotational (K_{ry}) forces acting on footings. The following formulas recommended by Gazetas [13] were utilized:

$$\boldsymbol{K_z} = \frac{2GL}{1-v}\,(0.73 + 1.54\,X^{0.75})\ with\ X = Ab/4L^2 \qquad (1)$$

$$\boldsymbol{K_y} = \frac{2GL}{2-v}(2 + 2.5\,X^{0.85}) \qquad (2)$$

$$\boldsymbol{K_{ry}} = \frac{3G}{1-v}\,I_{by}^{0.75}\left(\frac{L}{B}\right)^{0.15} \qquad (3)$$

where G is soil shear modulus, v *is* Poisson's ratio, B and L are half the width and half the length of foundation, respectively, Ab is foundation surface area, and I_{by} is footing moment of inertia with respect to the y-axis.

The above static stiffness values were then multiplied by a coefficient to account for the dynamic stiffness of soils, as a function of loading force circular frequency, frequency at the highest spectral acceleration and soil shear wave velocity (see Table 1 and Figure 2 of [13]). A footing size of 2 m by 3 m was used.

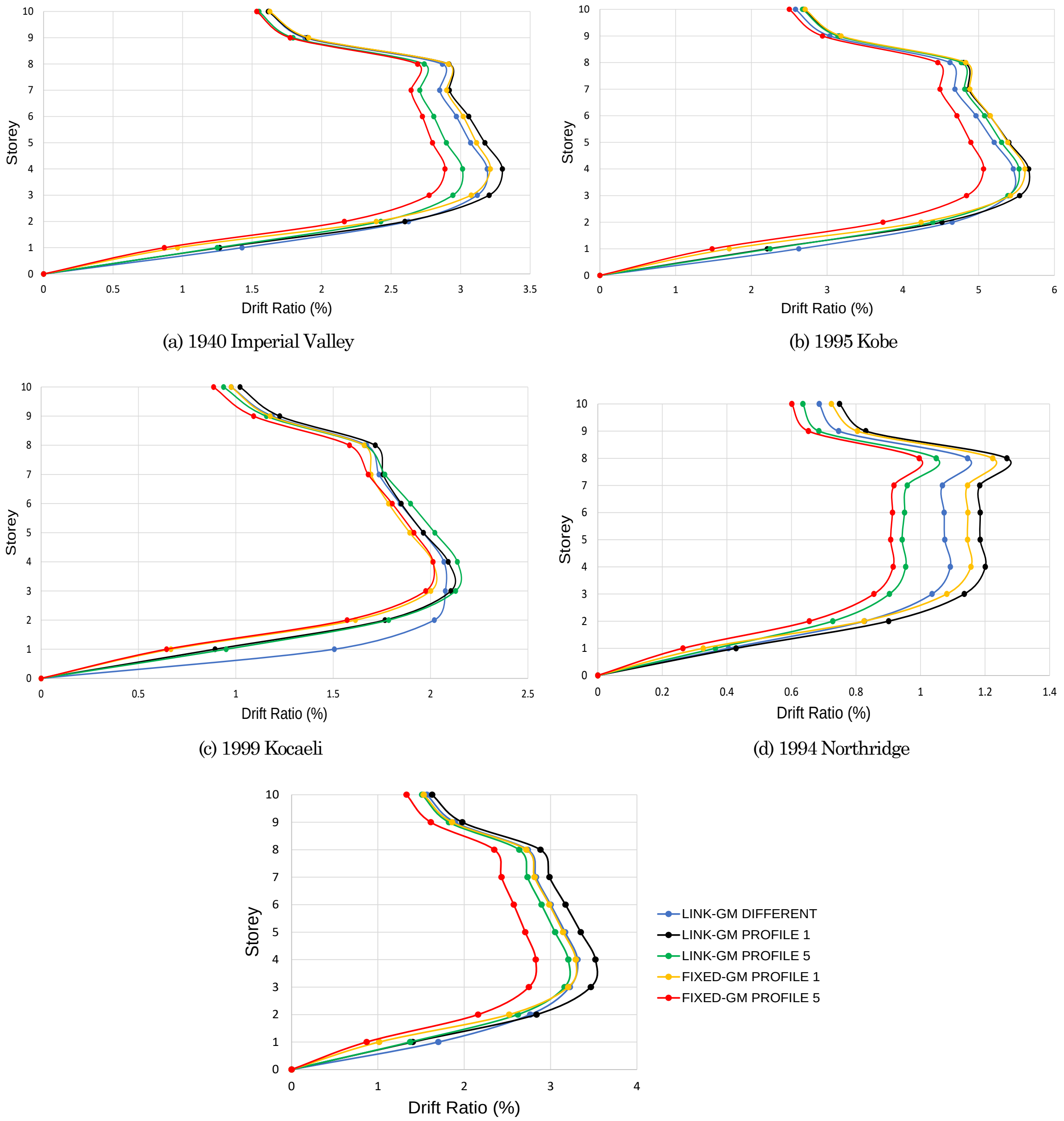
These springs were modeled as a two-joint link in SAP 2000, and the input earthquake loads (ground displacements) were applied at the base point of the link. For the fixed foundation model, the foundations and tie beams were assumed to move in unison with the soils.

Inter-story Drifts

Figure 7 plots the maximum inter-story drifts calculated for the five (5) selected earthquake records and five (5) cases analyzed for this study (see Table 3).

Table 3. Analysis Cases Considered for This Study

Foundation	Ground Motion	Soil Profile	Label
Flexible Connection	Out-of-phase Ground Motion	Using Figure 3 Model	LINK-GM DIFFERENT
Flexible Connection	Same Ground Motion	Soil Column 1	LINK-GM PROFIL 1
Flexible Connection	Same Ground Motion	Soil Column 5	LINK-GM PROFIL 5
Fixed Connection	Same Ground Motion	Soil Column 1	LINK-GM PROFIL 1
Fixed Connection	Same Ground Motion	Soil Column 5	LINK-GM PROFIL 5



(a) 1940 Imperial Valley

(b) 1995 Kobe

(c) 1999 Kocaeli

(d) 1994 Northridge

(e) 1999 Chi-Chi

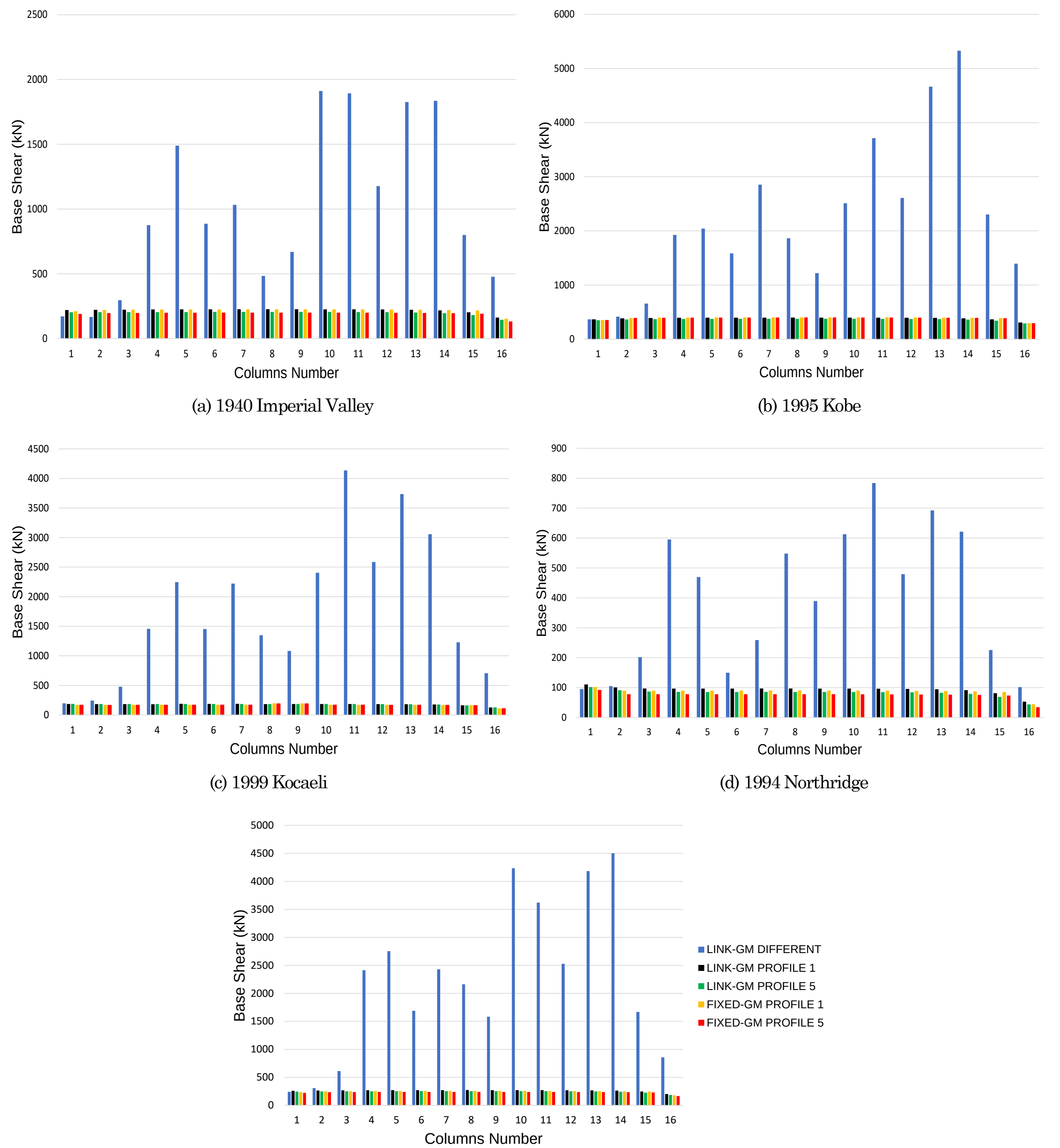
*LINK = spring-footed building, FIXED = fixed-footed building, GM DIFFERENT = testing with different ground motions, GM PROFILE 1 = testing with ground motion soil column 1, GM PROFILE 5 = testing with ground motion soil column 5.

Figure 7. Calculated Inter-story Drifts for the Selected Earthquakes and Analyses Cases

The maximum value was obtained at the column line and was taken at the time when the inter-story drift ratio reached the largest percentage at a certain story at the column line. From these plots, the following can be observed: 1) drift ratios calculated using the link foundation are larger than those calculated using the fixed connection (i.e., drifts for the "LINK-GM SOIL COLUMN 1/ SOIL COLUMN 5" cases are larger than those for the "FIXED-GM SOIL COLUMN 1/ SOIL COLUMN 5" cases) and 2) drift ratios calculated using the out-of-phase ground motions (LINK-GM DIFFERENT case) are not that much different than those calculated using the same ground motion.

Base Shears

Figure 8 shows the maximum (absolute) column base shears calculated for the five (5) selected earthquake records and five (5) cases analyzed for this study. The absolute base shear was taken for each column individually, and hence, the column base shears shown in Figure 8 did not necessary occur at the same time step. These figures indicate the following: 1) the use of out-of-phase ground motions produces much larger base shears than those estimated using the same ground motions. This is due to foundation movements that are not always in the same direction when



(a) 1940 Imperial Valley

(b) 1995 Kobe

(c) 1999 Kocaeli

(d) 1994 Northridge

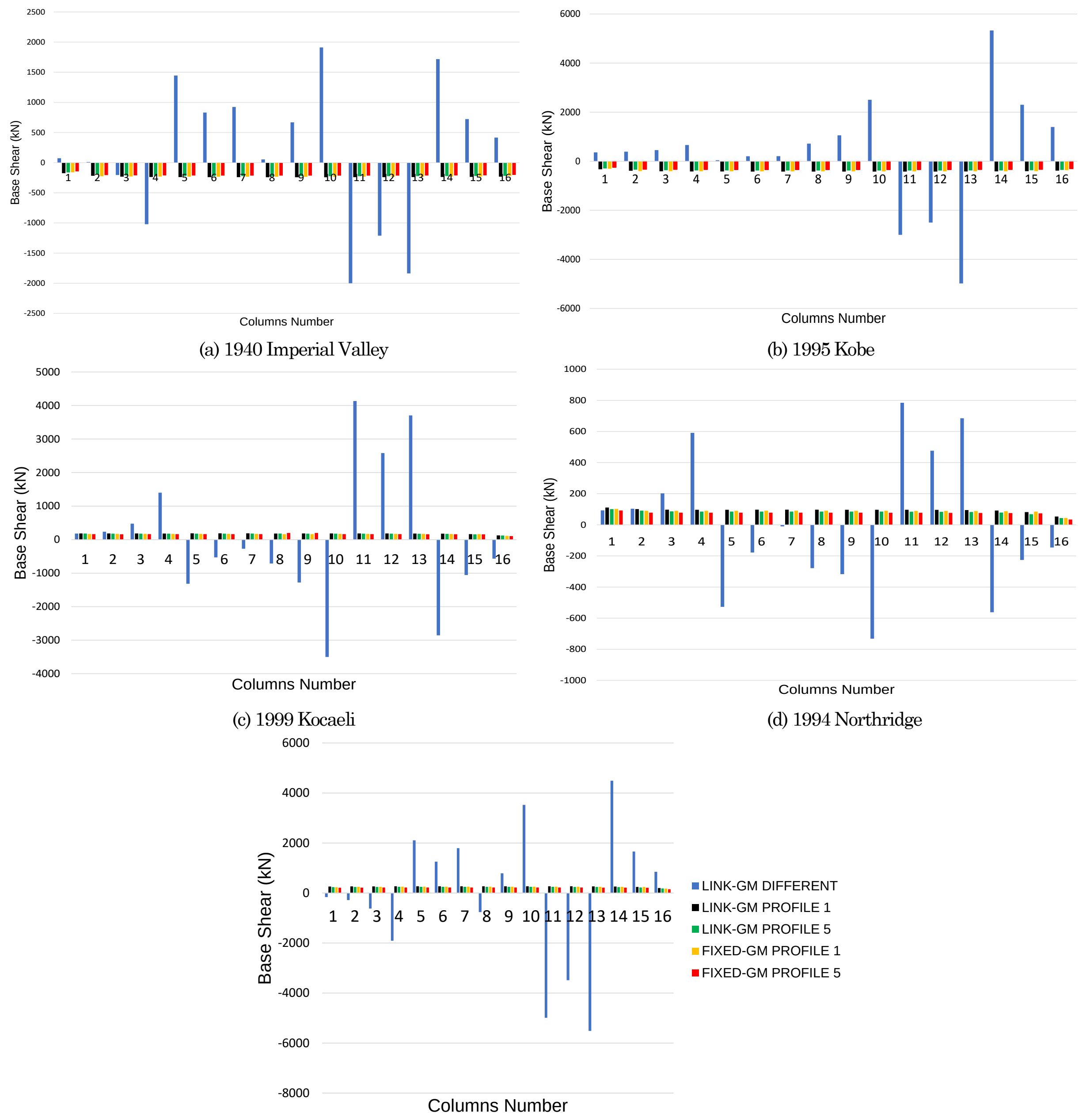
(e) 1999 Chi-Chi

*LINK = spring-footed building, FIXED = fixed-footed building, GM DIFFERENT = testing with different ground motions, GM PROFILE 1 = testing with ground motion soil column 1, GM PROFILE 5 = testing with ground motion soil column 5.

Figure 8. Calculated Maximum Base Shears for the Selected Earthquakes and Cases

subject to out-of-phase ground motions and 2) similar base shears were calculated for the same ground motions, regardless of the foundation connection model used.

The magnitude and direction of base shear in each column are expected to change at each time step during earthquake. Figure 9 compares the instantaneous (i.e., not the absolute value) column base shears at the time when the maximum total base shear occurred. Similar to the previous results, the column base shears calculated using the out-of-phase ground motions are much larger than those estimated using the same ground motion, and more importantly, these base shears act in different directions, compared to the base shears for the same ground motion that always act in the same direction. Adjacent base shears acting in opposite directions could potentially result in much larger alternating compressive and tensile forces in beams and slabs.



(a) 1940 Imperial Valley

(b) 1995 Kobe

(c) 1999 Kocaeli

(d) 1994 Northridge

(e) 1999 Chi-Chi

*LINK = spring-footed building, FIXED = fixed-footed building, GM DIFFERENT = testing with different ground motions, GM PROFILE 1 = testing with ground motion soil column 1, GM PROFILE 5 = testing with ground motion soil column 5.

Figure 9. Calculated Base Shears for the Selected Earthquakes and Cases

Tie Beam Axial Force

In practices, tie beams are used to connect columns and foundations (footings or pile caps). The main purpose of tying up columns and foundations is to reduce or minimize the adverse impacts to structure due to differential movements of foundations. Design axial force for a tie beam is often taken as 10-percent of the adjacent maximum column axial force. Table 4 lists the axial forces generated in the tie beam for the five (5) analyses case during the 1940 Imperial Valley record (the 4th column). Also listed are the axial forces in the adjacent columns that are connected by the tie beam (Axial K13 and Axial K14). These columns were selected because they received ground motion in opposite directions during earthquake and produced larger axial forces in the tie beam. The last two (2) columns show the tie beam axial forces, as percentages of column axial forces. Two observations can be made: 1) the axial force generated in the tie beam is significantly larger when the out-of-phase ground motions are used (about 59% of the column axial force) and 2) when the foundations are modeled using springs or links, the axial force in the tie beam is about 9% of the column axial force, consistent with current practices.

Table 4. Calculated Tie Beam Axial Forces (TB) as Percentages of Column Base Shears

Imperial Valley Earthquake (kN)					
Models	K13 Axial Force	K14 Axial Force	TB Axial Force	%TB to K13	%TB to K14
LINK-GM DIFFERENT	-3648.08	-3677.77	-2142.51	58.73	58.73
LINK-GM PROFILE 1	-2645.52	-2523.39	-239.35	9.04	9.49
LINK-GM PROFILE 5	-2123.85	-2298.23	-19121	9	8.32
FIXED-GM PROFILE 1	-2329.36	-2452.23	0	0	0
FIXED-GM PROFILE 5	-1892.31	-1727.23	0	0	0

Conclusions

A 10-story, 90m-wide, reinforced concrete structure supported on isolated footings with tie beams was used to evaluate the effects of variable soil conditions on seismic ground motions and building responses. To assess the effects of variable soil conditions on seismic ground motions, a hypothetical soil profile with varying soft clay thickness was sub-divided into five (5) soil columns and 1D dynamic site response analyses were then performed on these soil columns to generate the out-of-phase ground motions at ground surface. Dynamic time response analyses were conducted using the out-of-phase and same input ground motions to evaluate the building seismic responses in terms of inter-story drift, column base shear and tie beam axial force. Both flexible and fixed foundation-soil connections were considered.

The results of the analyses indicate the following:
1. Building inter-story drifts are not significantly affected by the use of out-of-phase ground motions. This may be due to the use of tie beams connecting the columns that results in a horizontally rigid structure.
2. Column base shears become very large when the out-of-phase ground motions are used, as compared to the base shears calculated using the same or uniform ground motions. Moreover, the base shears on adjacent columns could act in opposite directions during shaking, potentially resulting in much larger alternating compressive and tensile forces in beams and slabs.
3. Tie beam axial force could be much larger than the current practice recommends (10% of column axial force) when the out-of-phase ground motions are used.

The results observed in this study should be considered preliminary, and further studies are needed to confirm these findings. For future studies, it is recommended that improvements could be made to better assess the effects of variable soil conditions on building seismic performances, including the use of nonlinear structural elements, 2D site response analysis and greater variation in soil conditions with the presence of liquefiable soils.

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