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Seismic performance of R/C structures under vertical ground motion

Selcuk Bas*[1], Jong-Han Lee[2a], Mukadder Sevinc[3b] and Ilker Kalkan[3c]

[1]*Department of Civil Engineering, Faculty of Engineering, Bartin University, 74100 Bartin, Turkey*
[2]*Department of Civil Engineering, College of Engineering, Daegu University, 38453 Gyeongsan, Korea*
[3]*Graduate School of Natural and Applied Sciences, Department of Civil Engineering, Kirikkale University, 71450 Kirikkale, Turkey*

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Abstract. The effects of the vertical component of a ground motion on the earthquake performances of semi-ductile high-rise R/C structures were investigated in the present study. Linear and non-linear time-history analyses were conducted on an existing in-service R/C building for the loading scenarios including and excluding the vertical component of the ground motion. The ratio of the vertical peak acceleration to the horizontal peak acceleration (V/H) of the ground motion was adopted as the main parameter of the study. Three different near-source earthquake records with varying V/H ratio were used in the analyses. The linear time-history analyses indicated that the incorporation of the vertical component of a ground motion into analyses greatly influences the vertical deflections of a structure and the overturning moments at its base. The lateral deflections, the angles of rotation and the base shear forces were influenced to a lesser extent. Considering the key indicators of vertical deflection and overturning moments determined from the linear time-history analysis, the non-linear analyses revealed that the changes in the forces and deformations of the structure with the inclusion of the vertical ground motion are resisted by the shear-walls. The performances and damage states of the beams were not affected by the vertical ground motion. The vertical ground motion component of earthquakes is markedly concluded to be considered for design and damage estimation of the vertical load-bearing elements of the shear-walls and columns.

Keywords: vertical ground motion; seismic performance; performance level; earthquake behavior

1. Introduction

Structures are prone to not only horizontal but also vertical excitations in earthquakes. The vertical component of earthquakes has generally been disregarded in earthquake engineering and earthquake-resistant design of structures for two main reasons. First, the vertical excitations are considered to induce negligible amounts of energy to the structures compared to the horizontal components. Secondly, the structural systems of conventional buildings are designed against the dead and imposed loads, which are also oriented vertically. Consequently, the load-bearing members of a structural system are expected to easily withstand the vertical forces induced by the vertical component of an earthquake. Nevertheless, the recent near-source records indicated that the vertical components of earthquakes are not negligible in structural earthquake analyses. Furthermore, the field observations also verified the structural damage caused by this ignored component (Papazoglou and Elnashai 1996). The study of Papazoglou and Elnashai (1996) showed that the periods of structures and earthquakes play an important role in the level of seismic damage in an earthquake rather than the energy content of the earthquake. Hence the vertical component of an earthquake should not be ignored due to its low energy content.

The recent earthquake records and field observations stimulated the researchers to develop new equations and procedures for accounting for the vertical component of earthquakes in structural analyses. Ambraseys and Simpson (1996) proposed new equations for establishing the vertical ground spectra for structures, which depend on the site geology, magnitude and distance-to-source of the ground motion. The study of Elnashai and Papazoglou (1997) indicated that assuming the vertical peak ground acceleration to be 2/3 of the horizontal peak ground acceleration is not an inadequate approach in earthquake-resistant design. A simple procedure, which employs modal analysis, was proposed to estimate the forces induced to a structure by the vertical component of an earthquake. Bozorgnia *et al.* (1998) were able to measure the peak vertical structural accelerations of steel, concrete and base-isolated structures near the causative fault of the Northridge earthquake and the vertical frequencies of the structural members and the entire structural systems of these selected buildings. The measured periods indicated that high vertical spectral accelerations need to be considered in structures located in the near-source region. The analysis of single-degree-of-freedom (SDOF) systems under 186 different near-field earthquake records indicated that the effect of vertical excitations on lateral response of SDOF systems is negligible (Ambraseys and Douglas 2003). Elgamal and He (2004) developed vertical ground motion response spectra for near-field and far-field earthquakes using a total of 111 free-field strong motion and downhole array records from

*Corresponding author, Ph.D.
E-mail: selcukbas@itu.edu.tr or sbas@bartin.edu.tr
[a]Assistant Professor
[b]M.Sc. Student
[c]Associate Professor

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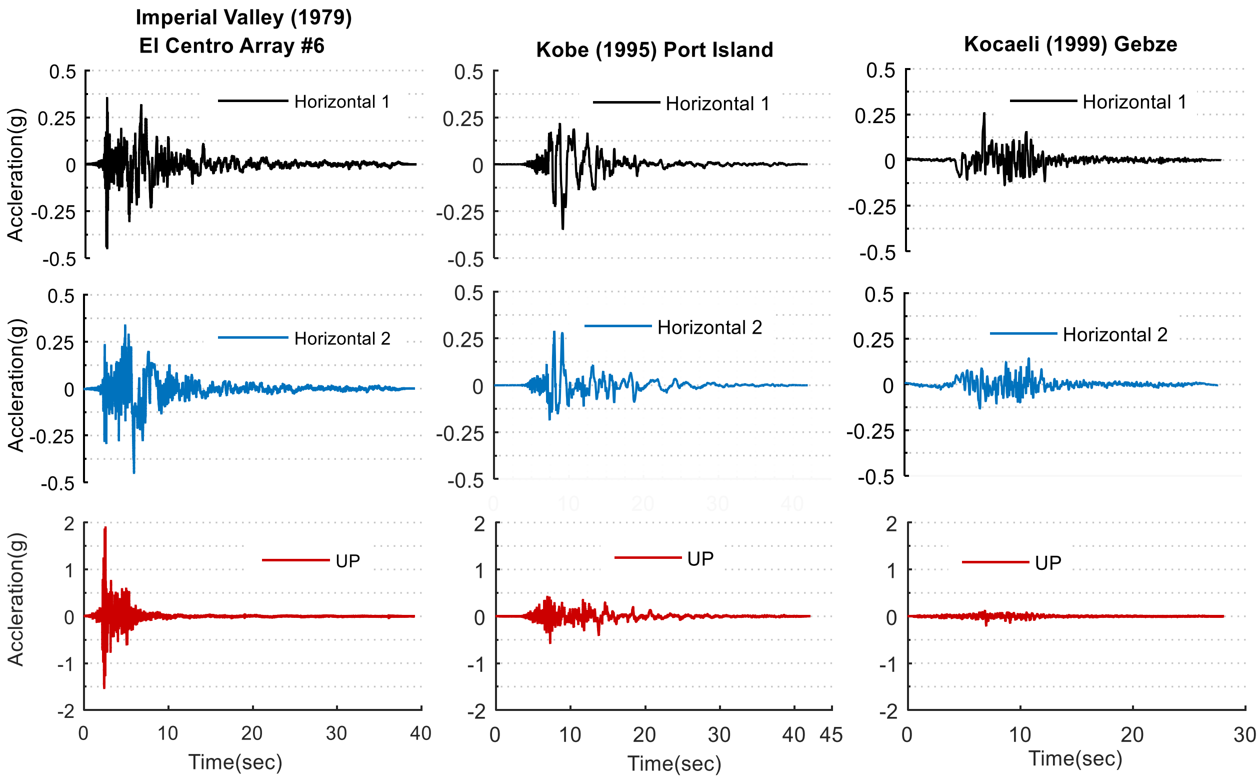
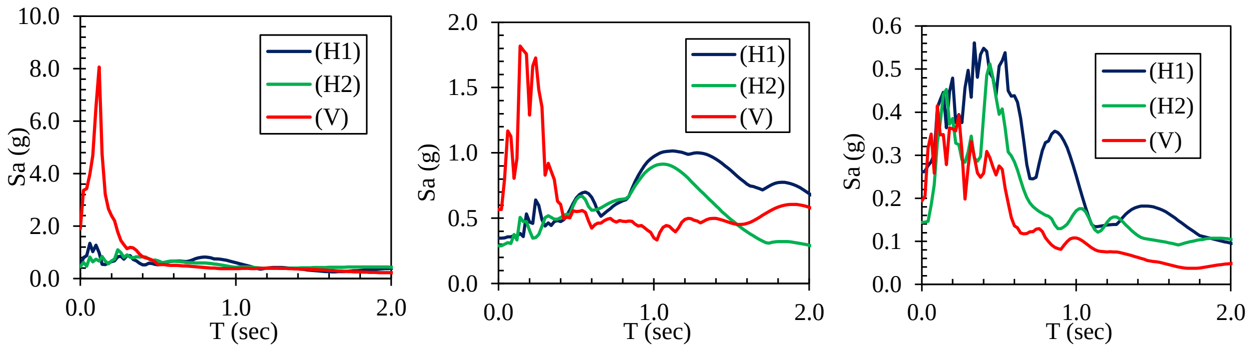


(a)



(b)

Fig. 1(a) selected ground motion records (b) acceleration response spectra

the state of California. Collier and Elnashai (2001) and Bozorgnia and Campbell (2004) developed simplified procedures for combining the effects of horizontal and vertical components of earthquakes. The procedure developed by Collier and Elnashai (2001) accounts for the possibility of the coincidence of peak response in horizontal and vertical directions. Wilson *et al.* (2005) found out that a vertical ground motion is capable of having a great impact on the substructure and superstructure of a skewed and curved reinforced concrete bridge with a short vibrational period in a moderate seismic zone. This impact was much more emphasized in the case of vertical-to-horizontal peak acceleration ratio exceeding unity. Jeon *et al.* (2015) specified that the variation of axial force in the bridge piers associated with the vertical ground motion has a considerable negative influence on the earthquake performances of old highway bridges with poor seismic details. The studies of Kim and Kim (2013) and Ghaffarzadeh and Nazeri (2015) showed that the horizontal spectral displacements of structures in the near-fault zone are subject to significant increase when the coupling effects of the horizontal and vertical ground motion components are considered instead of employing simple combination rules on isolated components. The increases in the horizontal spectral displacements were shown to be more pronounced for structures in near-fault zones in the presence of high vertical-to-horizontal peak acceleration ratio values. In order to develop more accurate model for R/C structures with construction joints, Yu and Liu (2016) made a numerical investigation considering vertical earthquake motion. Under both seismic excitations: horizontal (H) only and the combination of horizontal (H) and vertical (V), they performed the non-linear time-history analyses of the R/C frames in high seismic region. Based on the response parameters of the distribution of the story-drift and plastic hinge, they concluded that the vertical earthquake motion led to increase in the maximum story drift and to changing its distribution, and that the proposed

Table 1 Details of the selected earthquakes

Year	Earthquake	Station	Time (s)	Mw	Δ (km)	Soil	Mechanism	PGA (g)			V/H
								H_1	H_2	V	
1979	Imperial Valley	El Centro Array #6	39.08	6.53	1.35	Soft	strike slip	0.45	0.44	1.90	4.31
1995	Kobe	Port Island	41.98	6.90	3.31	Soft	strike slip	0.35	0.29	0.57	1.96
1999	Kocaeli	Gebze	27.99	7.51	10.92	Soft	strike slip	0.26	0.14	0.19	0.73

model was mostly compatible with the results from the field observations of the actual earthquakes according to the plastic-hinge distribution. For different aims to show the effects of the vertical earthquake motion on reinforced concrete and steel structures, a number of new studies (Mazza *et al.* 2017, Mazza and Labernarda 2017, Mazza and Alesina 2016 and Mazza and Vulcano 2012) were conducted in literature. Among these studies, the non-linear response history analysis of an R/C framed structure was made by Mazza *et al.* (2017) considering the near-fault effects that is characterized with the vertical-to-horizontal peak acceleration ratio. From the analysis, they showed that the near-fault ground motion also including vertical component of ground motion could be relatively effective on the response of the isolated R/C structures. In order to indicate the effectiveness of the High-Damping-Rubber Bearings (HDRBs) under near-fault ground motions, Mazza and Vulcano (2012) performed a numerical investigation on R/C test structures designed according to (Eurocode-8 1998). In that study, the ductility demand of the structural components under only horizontal earthquake excitation was determined relatively higher than that under the combined vertical and horizontal earthquake motions. The most important one among these studies was the investigation on intensity measure-based earthquake ground motion selection proposed by Mazza and Labernarda (2017). Based on the engineering demand parameters (EDPs), they offered an improved selection procedure for structural and non-structural intensity measures (IMs) directly pertinent to scaling earthquake ground motion records for the assessment of a six-story R/C test structure retrofitted by friction pendulum bearings (FPBs). A number of near-fault ground motions were utilized for the non-linear response history analysis. They indicated that the efficiency of the considered IMs noticeably reduced when the characterization of near-fault ground motions was made depending on the vertical-to-horizontal ratio higher than 1.0. This study also proved to be able to develop an improved intensity measures (IMs) selection procedure for the framed R/C structures with the FPBs base-isolation system. Therefore, the authors of the present study recommends to make the earthquake analysis of R/C structures subjected to near-fault seismic motions on the basis of structural and non-structural intensity measures (IMs) so as to properly estimate the engineering demand parameters of R/C structures.

Various structural earthquake codes (Eurocode-8 1998, IS: 1893 2000, NZS1170.5 2003, TEC 2007, NBC105 1994, UBC 1997) recommend the incorporation of the vertical ground motion into the earthquake analyses of structures by scaling the horizontal ground motion by a certain multiplier, determined according to the studies of previous researchers (Ambraseys and Simpson 1996, Elgamal and He 2004, Bozorgnia and Campbell 2004). The vertical peak acceleration is generally assumed to be in the order of 2/3 of the horizontal peak acceleration, as also suggested by the Turkish Earthquake Code (TEC 2007). Nonetheless, fixing the vertical ground motion to a certain proportion of the horizontal component can result in inaccurate and non-conservative estimates, particularly in the case of near-field ground motions, when the vertical-to-horizontal peak acceleration ratio (V/H) can be much higher than the assumed multiplier. All of the abovementioned studies from the literature underscore the need for detailed analytical studies for the quantification of the influence of vertical ground motion in combination with the horizontal one on the lateral displacements of a structure and the internal forces in the load-bearing members of the structure. The variations in the lateral displacements and the internal forces in the members with varying V/H ratio and earthquake duration need to be established. For this purpose, non-linear time-history analyses were conducted on an actual reinforced concrete structure in the present study under three different ground motions with different V/H ratios. The damage states of the structures under these ground motion records were also determined to establish the effect of the vertical ground motion on the lateral response of the analyzed structure.

2. Earthquake ground motions

The vertical and horizontal components of an earthquake are related to the P-waves (primary waves) and S-waves (secondary waves), respectively. Consequently, the vertical component of a ground motion has higher frequency content than the horizontal one. Furthermore, a structure is subjected to the vertical component prior to the horizontal component of the ground motion due to the shorter arrival times of the P-waves compared to S-waves. In far-field earthquakes, the arrival times of the P- and S-waves to the base of a structure are rather different. In near-field earthquakes, on the other hand, the vertical and horizontal components hit structures almost simultaneously due to the approximate arrival times of P- and S-waves to the base. Accordingly, the vertical components of ground motions have more pronounced effects on the earthquake responses of structures in near-field earthquakes as a result of the combined action of both components on structures.

In addition to the distance of a structure to the source of earthquake, there are other important parameters affecting the level of influence of the vertical component of a ground motion on the structural response. For instance, the vertical ground motion becomes more influential as the natural dominant frequency/period of the vertical mode of a structure approaches the respective value of the vertical component of ground motion. The ratio of the peak vertical acceleration to the peak horizontal acceleration (V/H) is another significant factor on the influence of vertical ground motion on structural response. The V/H ratio plays an important role in the seismic damage of a structure as well.

In the present study, the V/H ratio was adopted as the main parameter. The time-history analyses were conducted by using near-fault earthquake effects. Accordingly, Imperial Valley (1979), Kobe (1995) and Kocaeli (1999) earthquakes, which have different V/H ratios, were used in

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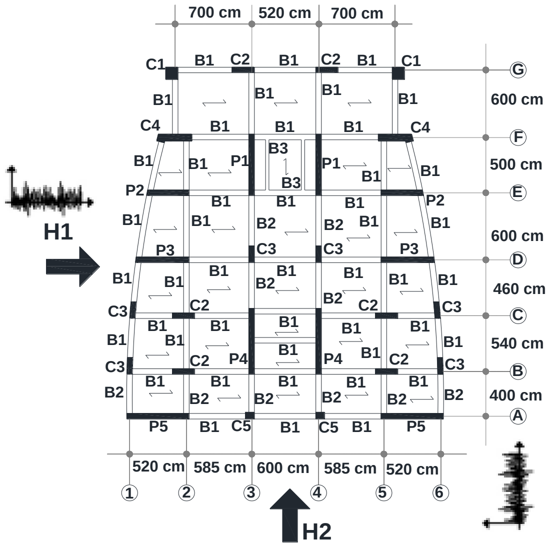

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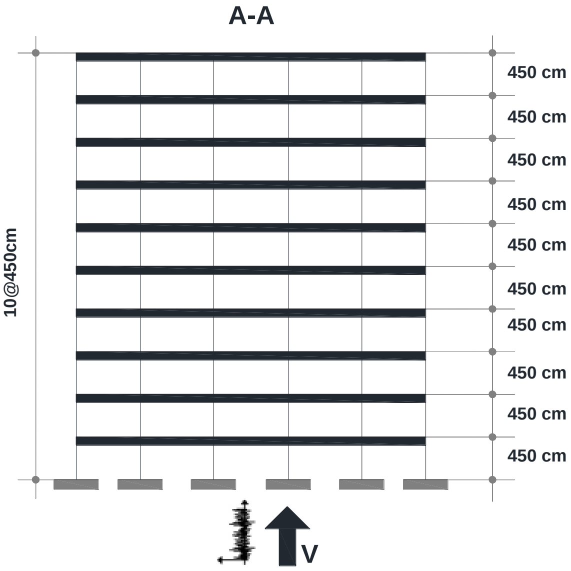


(c)

Sections of the test structure	
Beam	Width (cm) x Height (cm)
B1	50x90
B2	50x80
B3	30x90
Column	Width (cm) x Height (cm)
C1	110x110
C2	50x200
C3	50x150
C4	50x285
C5	60x80
Pier	Width (cm) x Height (cm)
P1	50x550
P2	50x380
P3	50x480
P4	50x590
P5	50x565

(d)

Fig. 2 General properties of the R/C test structure

the analyses of a high-rise RC structure. The horizontal and vertical peak accelerations, the durations, the distances to source (Δ), the magnitudes (Mw), subsoil conditions and fault mechanisms of the earthquakes are given in Table 1. The Imperial Valley (1979) earthquake had the highest V/H ratio (4.31), much higher than unity, while the V/H ratio of the Kocaeli (1999) earthquake was smaller than unity. The Kobe (1995) earthquake had a vertical peak ground acceleration about twice as great as the horizontal peak ground acceleration value. The non-linear time-history analyses of the selected R/C structure under these three ground motions, illustrated in Fig. 1(a), enabled the researchers to investigate the change in the damage of a high-rise R/C structure with changing V/H ratio in the presence of a near-source earthquake. Many design codes aforementioned also consider the acceleration response spectrum of earthquakes as an indicator to estimate the effect of earthquakes on the structure. Therefore, the response spectrums Sa(g) of the selected earthquakes for each direction were demonstrated in Fig. 1(b). Fig. 1(b) clearly shows that the Imperial Valley (1979) earthquake with the highest V/H ratio is relatively more effective in a short period range than the others due to its peak spectral acceleration value of 8.0 g. The reliability of the ground motions was also increased with the help of data processing and base-line correction.

3. Modal analyses of the selected structure

3.1 General features of the structure

A ten-story reinforced concrete (R/C) test structure with a semi-ductile frame system, composed of shear walls and an RC frame, was adopted in the analyses. As indicated in Fig. 2(a), this model corresponds to an in-service existing

Table 2 General considerations of the test structure

Number of stories:	10
Story height (m):	4.5
Occupation type:	Municipality
Structural system:	Shear-wall-frame (R=7)
Reinforcement grade:	S420 (f_{yk}=420 MPa)
Concrete grade:	C35 (fck=30 MPa)
Live load participation factor (n):	0.3
Dead and Live load:	7.50 and 5.0 kN/m^2
Total mass = 10 × floor mass:	10×1342.6 (ton)
Seismic zone:	1st (A_0=0.4)
Importance factor (I):	1.5
Type of the ground:	Z3 (T_a=0.15 s, T_b=0.60 s)

structure in Turkey. The structure was designed based on the requirements of the Turkish Earthquake Code (TEC 2007). General features of the test structure were given in Fig. 2: (b) the identical floor plan in all floors, (c) the elevation of the structure from the A-A grid and (d) the dimensions of all components of the structure according to the given abbreviated names. In addition, general considerations from the concrete and reinforcement grade, the live load participation factor, the live and dead load, each floor mass, total mass and the seismic zone, the importance factor to the type of the ground were presented in Table 2.

3.2 Finite element modeling (FEM)

The three-dimensional finite element (FE) model of the R/C structure (Fig. 3) was developed with the help of the SAP2000 FE program (CSI 2014). The beams and columns of the structural frame were modeled with 3-D frame elements, while the shear walls and the slabs were modeled with thin shell elements. Since the mat foundation was extremely rigid, it was not included in the model and the structure rested on fixed supports at the base instead. The slab elements were ensured to safely transmit the lateral earthquake forces with the help of rigid diaphragm constraints.

Non-linear modeling of the structure was accomplished with the lumped plastic approach named plastic hinge assumption. As illustrated in Fig. 4, potential plastic regions were considered to be concentrated end points of the beams, columns and piers. Inelastic relationship was represented by the moment-curvature (M-Φ) curve for these elements. For this objective, only moment (M1-M2) interaction was utilized for moment-curvature relation of the beam as shown in Fig. 4 due to approximately no axial force on the beam. For the columns and shear walls, moment-curvature curves were obtained depending on the level of the axial force. Therefore, as depicted in Fig. 4, the interaction surface was defined due to high axial load on these elements. Upon assigning the M1-M2 and P-M1-M2 plastic hinges to the beams and columns/shear walls, respectively, bi-linear moment-curvature curve was adopted according to the law that requires to be equal of the area under the original moment-curvature curve to that under the bi-

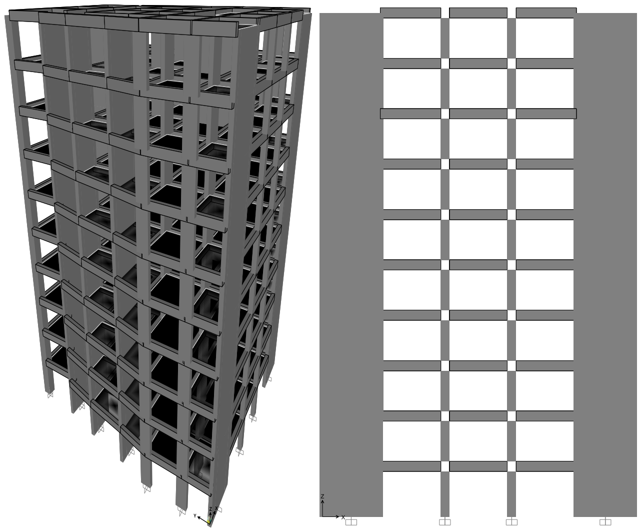


Fig. 3 FE model of the test structure

linearized curve. Shear force deformations were not considered in these analyses. Bi-linear laws for cyclic behavior of the beams, columns and shear walls were taken into account. This aim was achieved by the idealized M-Φ curve that consists of the elastic branch with the slope of EI1 and linearly hardening inelastic branch with the slope of EI2 as shown in Fig. 4. Strength degradation was not considered for cyclic behavior. All these considerations are schematically summarized in Fig. 4.

In the non-linear response history analyses of the R/C test structure, viscous damping was also considered as the proportional damping proposed by Rayleigh (1945). In this approach, as shown in Eq. (1), the damping matrix ($\boldsymbol{c}$) of a structural system is presented as linear combination of the mass ($\boldsymbol{m}$) and stiffness ($\boldsymbol{k}$) matrices with the constants of a0 and a1. However, the most common use of the proportional damping in the non-linear direct-integration response history analysis is the formulation given in Eq. (2) even though certain drawbacks of this approach that leads to "spurious" damping force due to the consideration of specific number of modes were drawn by Chopra and McKenna (2016). The constants of a_0 and a_1 can be easily determined by substituting the specific two modal angular frequencies (ω_i and ω_j) and the corresponding modal damping ratios (ζ_i and ζ_j) into Eq. (2).

$$[c] = a_0[\boldsymbol{m}] + a_1[\boldsymbol{k}] \tag{1}$$

$$\xi = \frac{a_0}{2}\frac{1}{\omega_n} + \frac{a_1}{2}\omega_n \tag{2}$$

According to Table 3, the modal frequencies (ω_i and ω_j) to define proportional damping in the time-history analyses were selected as the dominant modes of the test structure in the horizontal and vertical directions: the 1st dominant horizontal mode (f_1=1.414 s^{-1}) and the 1st dominant vertical mode (f_{20}=10.847 s^{-1}). Corresponding modal damping ratio for these specified modes were considered as ζ_1=ζ_{20}=5% that is well-known value for R/C frame structures. Thus, calculated proportional damping coefficients of a_0=0.7860 and a_1=0.0013 were defined to the SAP2000.

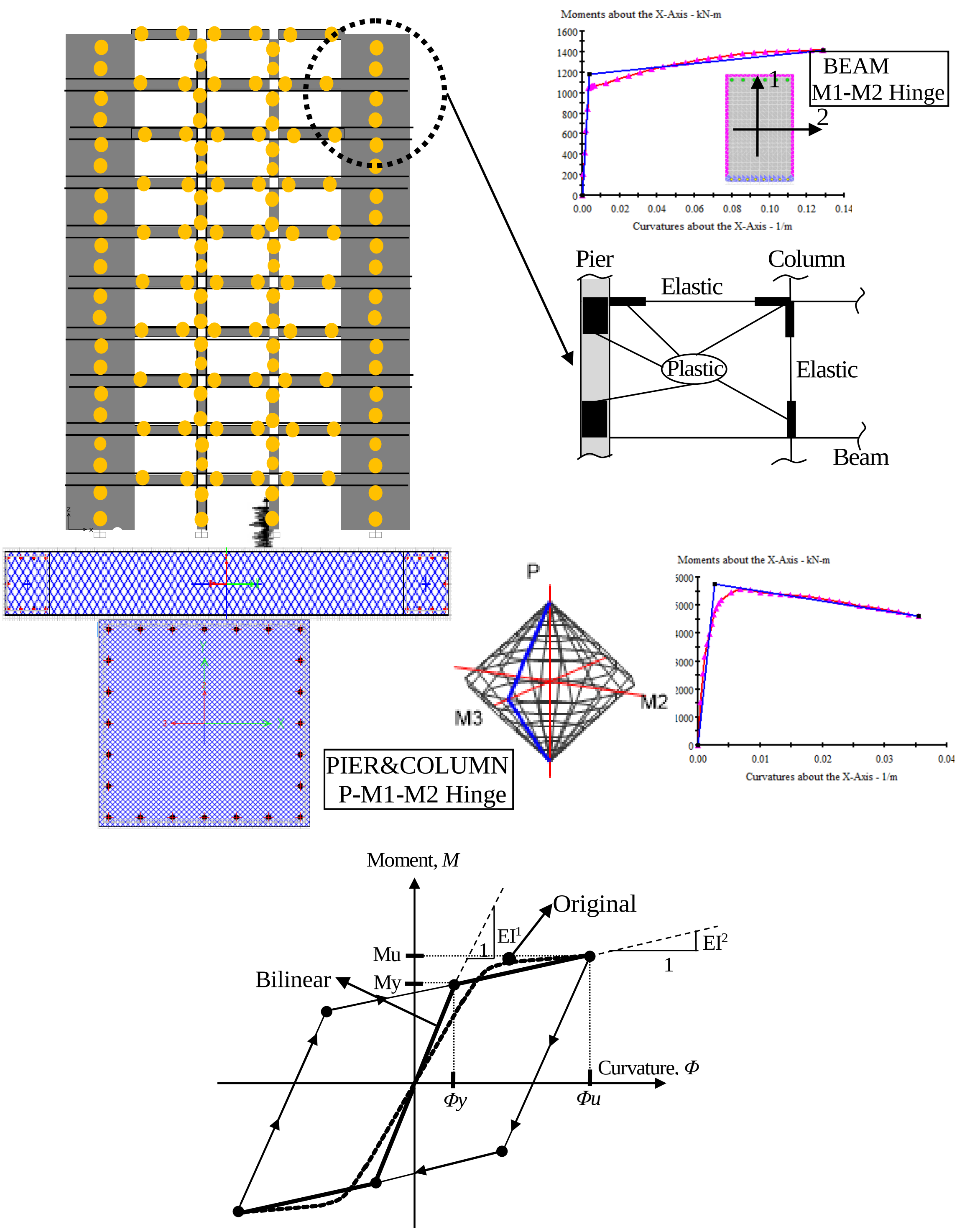


Fig. 4 Non-linear modeling considerations of the test structure

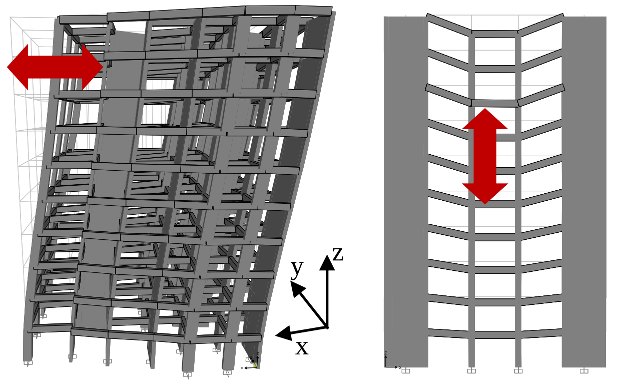
3.3 Modal analysis

Modal analyses were conducted on the model to determine the natural dominant frequencies of the structure in horizontal and vertical directions. The dominant frequencies of the vertical component of the earthquakes were compared to the natural dominant frequency of the structure in vertical direction to investigate the influence of the vertical component as its frequency approaches to the natural frequency of the structure.

The first mode shape of the structure in horizontal direction and the vertical dominant mode are illustrated as representative in Figs. 5(a)-(b), respectively. According to Table 3, the first two dominant horizontal (Mode 1 and Mode 2) and the vertical mode shape (Mode 20), and the dominant torsional modes shape (Mode 3) were obtained as 1.41 s^{-1}, 1.52 s^{-1}, 10.85 s^{-1} and 1.604 s^{-1}, respectively. These mode shapes were readily estimated depending on the mass participation percent given in Table 3 for the first 20 modes. The modes between the first four modes and the mode 20

Table 3 Dynamic characteristics of the structure

Mode	Period (s)	Frequency (Hz)	Mass Participation Ratio (%)					
			X-Direction		Y-Direction		Z-Direction	
			m_x	r_x	m_y	r_y	m_z	r_z
1	**0.707**	**1.414**	**0.394**	0.081	0.325	0.122	0.000	0.020
2	**0.659**	**1.517**	0.258	0.106	**0.417**	0.084	0.000	0.063
3	**0.623**	**1.604**	0.068	0.003	0.010	0.022	0.000	**0.674**
4	0.205	4.869	0.025	0.264	0.107	0.049	0.000	0.002
5	0.190	5.268	0.033	0.034	0.014	0.063	0.000	0.088
6	0.181	5.528	0.098	0.023	0.012	0.185	0.000	0.041
7	0.112	8.949	0.000	0.001	0.000	0.004	0.019	0.000
8	0.109	9.193	0.000	0.000	0.000	0.001	0.006	0.000
9	0.108	9.230	0.000	0.000	0.000	0.000	0.000	0.000
10	0.108	9.266	0.000	0.000	0.000	0.000	0.000	0.000
11	0.107	9.308	0.000	0.003	0.003	0.001	0.001	0.000
12	0.107	9.327	0.001	0.014	0.013	0.001	0.000	0.002
13	0.107	9.359	0.000	0.000	0.000	0.000	0.000	0.000
14	0.106	9.405	0.000	0.000	0.000	0.000	0.000	0.000
15	0.106	9.436	0.000	0.000	0.000	0.000	0.000	0.000
16	0.105	9.481	0.000	0.001	0.000	0.000	0.000	0.000
17	0.103	9.671	0.002	0.029	0.030	0.003	0.000	0.001
18	0.099	10.089	0.000	0.002	0.003	0.001	0.000	0.044
19	0.093	10.719	0.049	0.001	0.003	0.052	0.003	0.001
20	**0.092**	**10.847**	0.000	0.020	0.000	0.003	**0.676**	0.000
Total mass (%)=			0.929	0.583	0.937	0.590	0.706	0.936



(a) The 1st horizontal mode (b) The 1st vertical mode

Fig. 5 Mode shapes

were obtained not to be effective due to relatively less mass participation.

4. Linear and non-linear time-history analyses of the R/C structure

4.1 Linear time-history analysis

Linear time-history analyses of the structure were conducted to determine the influence of the vertical component of ground motion on the base shear force, overturning moment, lateral and vertical deflections and angle of rotation of the top story with respect to the base. For each ground motion record (Imperial Valley, Kobe and Kocaeli), the structure was subjected to two different loading scenarios. In the first scenario (H), the structure was only subjected to the two horizontal components of the ground motion, while both the vertical and horizontal components were applied to the structure in the second scenario (H+V). The differences between the force, moment, displacement and rotation values for the two loading scenarios enabled the researchers to figure out the effects of the vertical component of a near-field earthquake on the structural performance.

The variations of the base shear force, overturning moment, top lateral and vertical deflections and angle of rotation are illustrated in Figs. 6-8. The seismic performances of structures are generally expressed in terms of the lateral deflections in two perpendicular directions and the torsional rotations of the story. The structure exhibited similar response in both orthogonal directions, and therefore, only the lateral deflections in one direction were considered. Similarly, the internal forces in a structure during an earthquake are generally given in terms of the base overturning moments about two orthogonal axes and the base shear forces in two orthogonal directions. Due to the similar seismic performances in the two orthogonal directions, only the base shear and overturning moment values in one direction were taken into consideration.

Among the considered measures (base shear, overturning moment, lateral and vertical top deflections, angle of rotation at the top), the inclusion of the vertical component of ground motion had the greatest impact on the vertical top-story deflections and the overturning moments at the base of the structure. The increase in the base shear force and top lateral deflection with the inclusion of the vertical component was not as pronounced as the increase in the vertical top deflection and overturning moment. All five measures increased more significantly with increasing V/H ratio in the presence of the vertical component, implying that the ratio of the vertical peak acceleration to the horizontal one is a rather significant factor on the deformations of a structure and the forces in a structure during an earthquake. The highest increases in all measures with the inclusion of the vertical component were obtained in the case of the Imperial Valley (1979) ground motion, which had the highest V/H ratio among the adopted ground motion records.

The increased top-story vertical deflections in the H+V loading scenario constituted one of the most significant outcomes of the linear time-history analyses of the present study. The increase in these deflections result in the slabs to cease their ability to act as rigid diaphragms during a ground motion. Furthermore, the significant overturning moments at the base of the structure induce significant second-order moments to the vertical load-bearing members (shear walls and columns) of the system. The fail of the slabs to act as rigid diaphragms and the increased overturning moments in the columns and shear walls might completely change the earthquake performance of a structure. The equivalent force methods, which are the most commonly preferred methods in the seismic analysis and

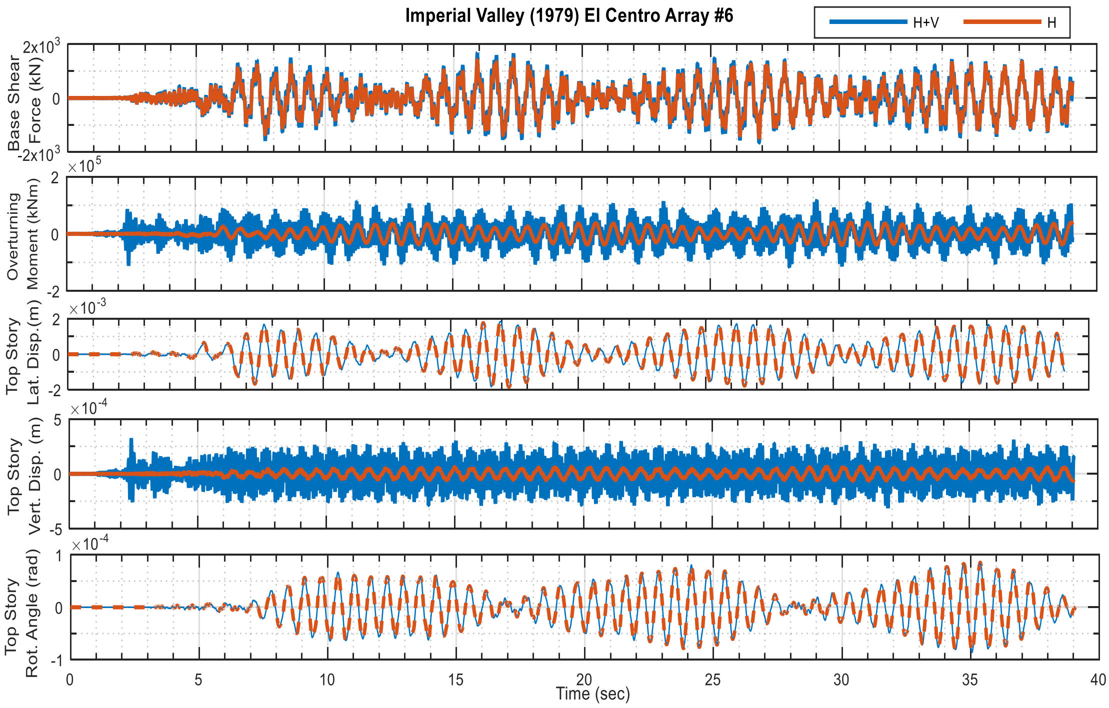


Fig. 6 The results from Imperial Valley (1979)

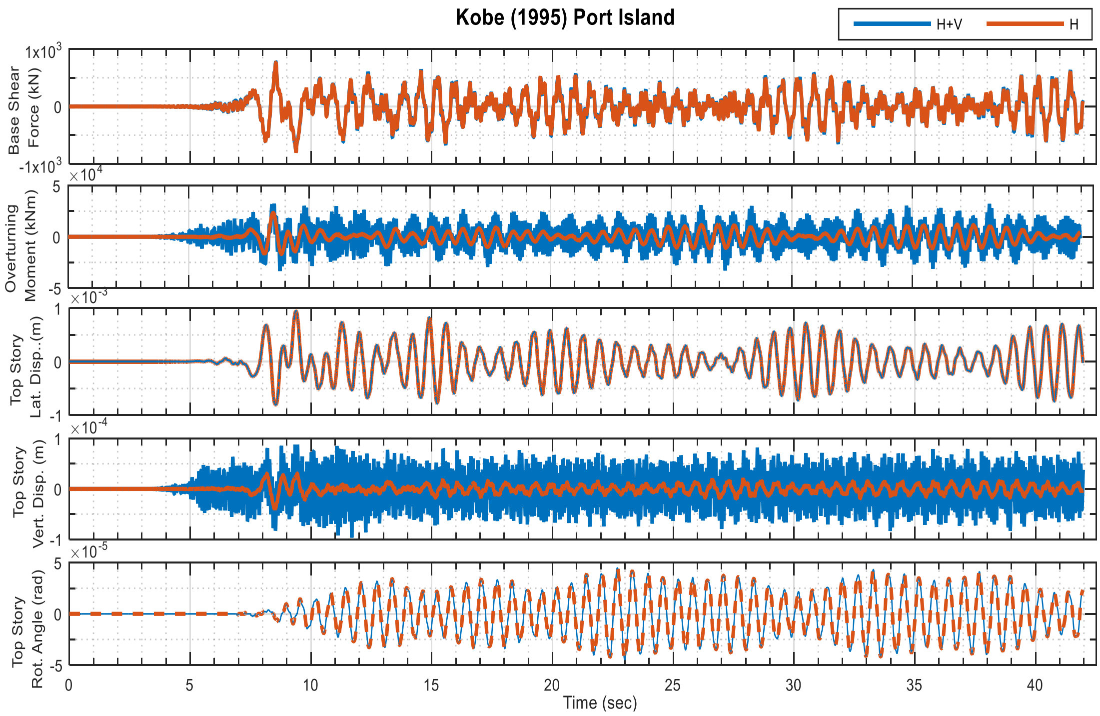


Fig. 7 The results from Imperial Valley (1979)

design of ordinary low- and medium-rise buildings, might yield to non-conservative and inaccurate estimates due to these changes in the earthquake behavior of structures in the presence of the vertical component of a ground motion. The base shear force, whose increase in the presence of the vertical component remained much more limited, will play a much less important role in the change of the structural seismic performance compared to the base overturning moment. The equivalent force methods, which are the most commonly preferred methods in the seismic analysis and design of ordinary low- and medium-rise buildings, might yield to non-conservative and inaccurate estimates due to these changes in the earthquake behavior of structures in the presence of the vertical component of a ground motion. The base shear force, whose increase in the presence of the vertical component remained much more limited, will play a much less important role in the change of the structural seismic performance compared to the base overturning

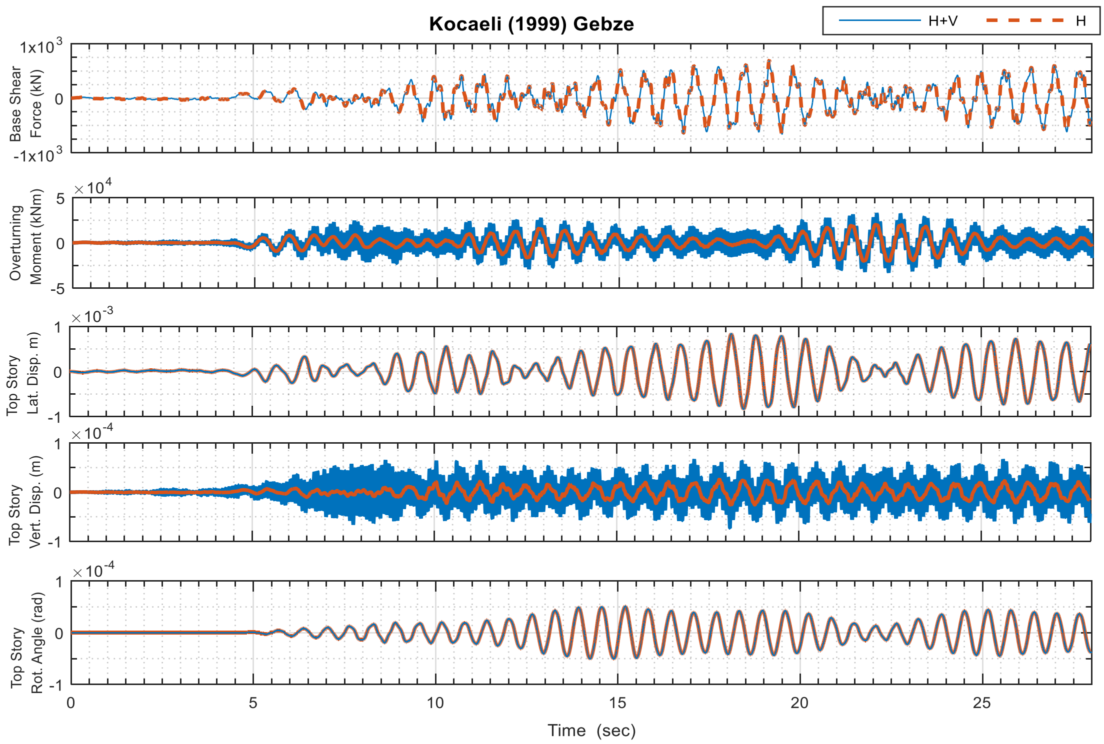


Fig. 8 The results from Kocaeli (1999)

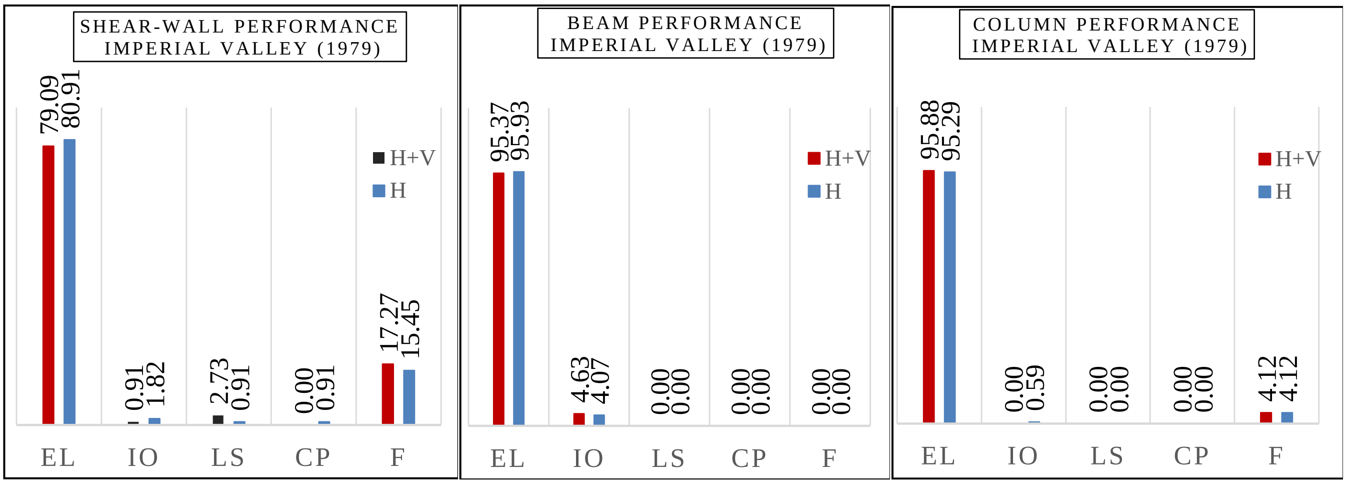


Fig. 9 Performance levels for the Imperial Valley (1979) earthquake

moment.

4.2 Non-linear time-history analysis

The earthquake performance levels and the damage states of the members of the analyzed R/C building could not be evaluated with the help of the linear time-history analyses. For this purpose, non-linear time-history analyses were conducted for each ground motion record and for each loading scenario (H and H+V). The provisions of the ASCE/SEI 41-13 (ASCE 2014) code, which takes four different performance levels into account, were used in the analyses. These performance levels are the Elastic (EL), Immediate Occupancy (IO), Life Safety (LS), Collapse Prevention (CP) and Failure (F) levels. The Elastic (EL), or the Operational Level, corresponds to very light damage with no permanent drift and minor cracking in non-structural elements (ASCE 2014). The Immediate Occupancy (IO) level corresponds to light damage with no permanent drift and continuation of the occupancy of the structure. Minor cracks in the non-structural elements and low risk of safety are the other indicators of this level. In EL and IO levels, all stories are required to maintain their original stiffness and strength. The Collapse Prevention (CP) level corresponds to moderate damage with some residual strength and stiffness in all stories. The vertical load-bearing elements are expected to function while significant damage in non-structural elements (no out-of-plane failure) are possible. Finally, the Failure (F) level corresponds to severe damage with little residual strength

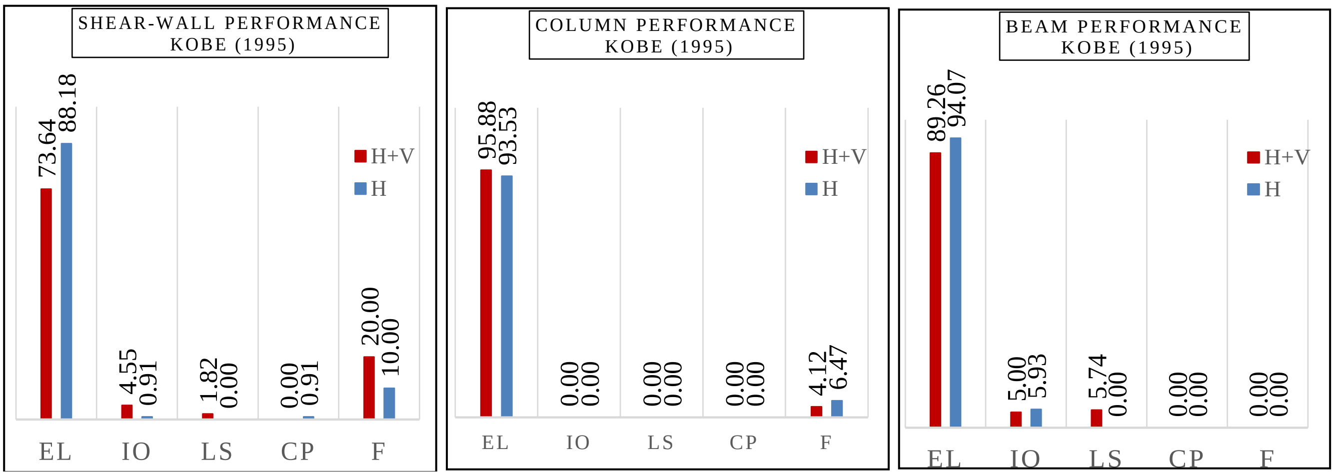


Fig. 10 Performance levels for the Kobe (1995) earthquake

and stiffness in all stories. The vertical load-bearing elements are expected to function with large permanent drifts and high risk of safety. These performance levels correspond to different amounts of rotation of the hinges along the moment-curvature plot. The performances of the beams, columns and shear walls were only investigated for the Imperial Valley (1979) and Kobe (1995) earthquakes. The Kocaeli (1999) earthquake was not included in this discussion, since the non-linear time-history analyses indicated that the inclusion of the vertical ground motion of this earthquake had little influence on the performances of the members due to the relatively low V/H ratio of the Kocaeli (1999) record compared to the other two earthquakes. Plastic hinges obtained from the XTRACT (Imbsen 2004) program were assigned to the ends of the beams, columns and shear walls in each analysis. The performance levels of the members for the two ground motions are illustrated in Figs. 9-10.

The figures indicate that the performances of the shear walls of the structural system increased by 2% and 10% with the incorporation of the vertical ground motion into the analyses for the Failure (F) performance level in Imperial Valley (1979) and Kobe (1995) earthquakes, respectively. The failure performance levels of the columns, on the other hand, remained unchanged in the Imperial Valley (1979) earthquake, while undergoing a 2% decrease in the Kobe (1995) earthquake. The changes in the performances of the vertical load members indicate that the changes in the forces and moments throughout the structural system as a result of the inclusion of the vertical ground motion were directly resisted by the shear walls at Failure (F) level. Nevertheless, the resistance of the additional forces and moments in the H+V scenario by the shear walls means that the structural system exhibits a less ductile behavior in the presence of the vertical ground motion, since the columns are much ductile members than the shear walls in an earthquake if designed properly.

The higher changes in the performances of the shear walls and columns in the Kobe (1995) earthquake are associated with the longer duration of this ground motion compared to the Imperial Valley (1979) earthquake. Accordingly, the duration of an earthquake has as significant impact on the performances of the vertical members of the structural system as the V/H ratio.

The beams did not undergo any major change in all performance levels in the Imperial Valley (1979) earthquake. In the Kobe (1995) earthquake, however, the beams underwent a 6% increase with the inclusion of the vertical component in the Life Safety (LS) performance level, while having an about 5% decrease in the Elastic (EL) performance level. In general, the vertical ground motion can be concluded to have little or no influence on the performances of the beams of the structural system. The vertical component of a ground motion affects the performances of the vertical load-bearing members of the structural system to a considerable extent. The relatively unchanged performances of the beams in both loading scenarios can be attributed to the fact that the changes in the forces and moments of the system with the incorporation of the vertical ground motion into the analyses are compensated by the increased performances of the shear walls, particularly at the failure level.

5. Conclusions

The present study pertains to the influence of the ratio of the vertical peak acceleration to the horizontal peak acceleration (V/H) of near-source earthquakes on the structural performances, deflections and forces of semi-ductile R/C frame systems. Linear and non-linear time-history analyses were conducted on an existing R/C structural system for three different ground motion records. Elastic (EL), Immediate Occupancy (IO), Life Safety (LS), Collapse Prevention (CP) and Failure (F) performance levels were used in the non-linear time-history analyses. The analyses for two loading scenarios, including and excluding the vertical component of the ground motion, of each record yielded to the following conclusions:

• The incorporation of the vertical component of a ground motion results in significant increase in the vertical deflections at the top and the overturning moments at the base of a structure. The lateral top deflections, base shear forces and angles of rotation do not undergo as great increases as the vertical top deflections and base overturning moments in the presence of the vertical component. The increase in all deflections, forces and moments becomes more pronounced with increasing V/H ratio of the earthquake.

• The effects of the inclusion of the vertical component of a ground motion into the time-history analysis are compensated by the shear walls in the structural system. Since the contribution of the shear walls to the system response increases in the presence of the vertical component, the structure exhibits a less ductile behavior during an earthquake in this scenario. The shear walls reach a much more damaged state at the Failure (F) performance level.

• Unlike the vertical load-bearing members (shear walls and columns) of the structural system, the performances and the damage states of the beams are not influenced to a major degree in the presence of the vertical ground motion.

• The effects of the inclusion of the vertical component of a ground motion into the analyses depend on the duration of the earthquake as well as the V/H ratio. The performances of the vertical members are affected to greater degrees with increasing duration. The present analyses indicated that the duration of an earthquake can be more influential on these performances compared to the V/H ratio.

• In general, the performances and the damage states of the members of a structural system are not affected by the inclusion of the vertical component of a ground motion considerably if the V/H ratio of the earthquake is less than unity.

• The vertical component of a ground motion should be taken into account particularly in the design of the vertical load members of a structural system, since this component plays an important role in the seismic performances of these elements.

The conclusion drawn in the present study are based on a limited number of analyses of the structure under near-fault ground motions. Further analysis are needed for verification of the results and for drawing firm conclusions.

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