Available online at www.CivileJournal.org

Civil Engineering Journal

Vol. 3, No. 8, August, 2017

Seismic Performance of High-Rise RC Shear Wall Buildings Subjected to Ground Motions with Various Frequency Contents

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Received 19 July 2017; Accepted 31 August 2017

Abstract

Construction of tall buildings in societies is rising up for the increased population and limitation in horizontal expansion of cities. Therefore, behavior of these structures against earthquake essentially requires investigation. Recent research has shown that frequency content parameter of an earthquake has remarkable impacts on seismic response of buildings. This study aimed to investigate direct effects of frequency content on high – rise buildings. Thus, six Reinforced Concrete (RC) central core 10, 15, 20, 25, 30, and 35- story buildings were built in open source software OpenSees, and their seismic behavior under seismic records with various frequency contents were investigated. In this research, non – linear dynamic Time – History was carried out and also behavior of buildings was compared in drift, shear force of stories, and maximum displacement of stories. Results of Time – History analysis showed that low – frequency content records have the highest effects on buildings. Most of the responses of drift and displacement of stories pertained to low – frequency contents in low – rise 10 and 15-story buildings. Although the most shear force of stories was related to low – frequency contents, with increasing height of buildings, shear force of stories increased, too. So that under Kobe Japan record which has the lowest frequency content among all records in this paper. Maximum shear force of stories was 6840 ton in 10-story building, whereas it was 12332 ton in 35- story building.

Keywords: Seismic Performance; High-Rise Buildings; Reinforced Concrete Shear Walls; Frequency Content.

1. Introduction

Since the late nineteenth century, structural engineering in conjunction with high-rise buildings has greatly improved. The use of different components, various structural systems, and more height are examples of these advances. In terms of structure, tall structures require appropriate structural solutions in order to provide adequate stability and rigidity. As a matter of fact, while in designing short buildings the impacts of dead and live loads are the main factor, by increasing the height, the focus of structural engineers is on controlling horizontal displacement. Properly designed shear walls have shown acceptable seismic performance in many tests conducted [1, 2] and also, numerous analytical studies have been carried out by researchers on shear walls [3-5]. Additionally, the shear wall system seems economical for 30 to 40- story buildings. Above this height, the stresses resulting from lateral force necessitate a reduction in the required thickness for the shear walls, and this is rather cost-effective [6]. High-rise buildings with reinforced concrete shear walls and core layout are suitable for structural engineers because, comparing high-rise buildings with other systems resistant to horizontal forces, the benefits

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include lower costs, faster construction, and more open architectural space [7]. Also, the core structure with columns replaced by partition shear walls can be an efficient structural system for residential buildings, with freedom to change the internal layout [8].

Shear walls located around the elevators and escalators room can transfer lateral loads in both directions as a powerful three-dimensional system. The advantage of this approach is that, due to the three-dimensional structure, they can endure shear forces and bending moments in both directions and also resist twisting when the link beams are located between the openings [9]. The inertia moment of reinforced concrete cores is very high, and, thus, often has the ability to withstand lateral loads. Although the core is normally presented to resist lateral loads, a share of the gravity loads is also incurred [10].

Based on studies conducted on the seismic response of structures, the largest percentage of the structural response is dependent on three important seismic parameters: amplitude of PGA and PGV, frequency content, and duration of strong ground motion. Many researchers claim that the frequency content has the greatest impact on non-elastic response of structural systems of many degrees of freedom (MDOF) [11-14].

In this study, the frequency content of earthquakes was considered as the main factor targeting the high dynamic response of high-rise structures. As a result, the records of nine earthquakes, based on the selected frequency content and effect of these parameters on seismic performance of six structures, consisting of two short-rise buildings, two mid-rise buildings and two high-rise buildings, were studied.

Gutenberg and Richter (1956) [15] were the first to examine the frequency content. As previously mentioned, together with parameters, such as amplitude and continuity duration, frequency content is one of the important parameters of an earthquake which, by using the Fourier series, transforms ground motion of the time domain into the frequency. The earthquake frequency content describes the distribution of the ground motion amplitude at different frequencies. This important seismic characteristic is affected by parameters, such as magnitude of the earthquake, the distance of the site from the epicenter, rock or soil material of the site, the fracture mechanics of the fault, the fault position, the epicenter depth of the earthquake and the region topography which could be effective on the frequency content, associated phases and, subsequently, the damaging effects of the earthquake. The dynamic response of structural systems subjected to earthquake shaking was mainly affected by the frequency content of the vibration imposed on the ground by an earthquake. When the earthquake frequency content carefully complies with the structural frequency, the dynamic response of structure will be upgraded; a larger force is applied to the system and may cause serious damage to the system [16, 17]. As a result, considering the effect of the frequency content on the dynamic response of structures in examining the seismic performance becomes important. So as to detect the reason frequency content can affect the structural response, it is necessary to recognize the response of a structure with few degrees of freedom in the elastic-plastic range. The response of a structure with few degrees of freedom by a specified first mode elastic period is determined and calculated in terms of spectral acceleration. $S_a\ (T_1)$ is measured in this period. However, in addition to several parameters, the non-elastic behavior of few degrees of freedom structures under seismic vibrations is affected by two main phenomena: rigidity decline due to deterioration of structural mechanical properties resulting from the reciprocating movements and irritations of higher modes causing dynamic response.

While the first main parameter leads to increased construction period, resulting in a softer response with an increased need for location change, the second main parameter contributes to a change in the distribution of inertia force, leading to increased need for base shear. Due to the consecutive sweep of non-elastic yielding, the vibration period of a shear wall is increased to an amount equivalent to twice the initial amount [18]. Therefore, for two records with the same values of $S_a\ (T_1)$, a record with higher amount of S_a in periods other than the period T_1 causes a need for the location change to be larger than that in non-elastic systems. This indicates that the structural response of multi-degree of freedom is highly sensitive to fluctuations of acceleration response spectra. Mortezaei and Motaghi (2016) [19] have shown that records with low frequency contents produced the highest displacement and damage. It was concluded from the results that the magnitude of an earth- quake is not the only determining factor for its intensity. Moreover, a lower PGA/PGV ratio leads to larger damage. In other words, the results suggested that the destructive effect of frequency content is greater than that of magnitude and strong motion duration.

2. Methods for Determining the Frequency Content

2.1. Evaluation of the Frequency Content

It is usually more useful to describe the frequency content of earthquakes with a numeric parameter as it allows researchers to easily compare this parameter with structural frequency; therefore, the possibility of resonance or increased dynamic response can be examined [13, 20, 21]. As mentioned in the previous section, Gutenberg and Richter [15] were the first to examine the frequency content and considered the period of waves with maximum amplitude, which was assumed to be equal to T_p (the dominant spectral period). Evaluation of seismic frequency content is possible using two indices:

- The stochastic indices to describe frequency content:

The use of stochastic indices to evaluate the frequency content is based on strong phases of accelerogram's records as a stationary random process. Clough and Penzien [22] introduced the dimensionless parameter ε as a representative of the frequency content.

$$0 \leq \varepsilon = \sqrt{1 - \frac{\lambda_2^2}{\lambda_0 \lambda_4}} \leq 1 \tag{1}$$

Where:

λ_i: i-th moment of the dominant spectral density

$2/D_3 < \varepsilon < 0.85$ for widescreen frequency

$0.85 < \varepsilon < 0.90$ for medium frequencies

$\varepsilon > 0.90$ for narrow frequencies

- Algebraic indices to describe the frequency content:

Algebraic indices are associated with the maximum response of the one degree of freedom system to describe the frequency content which can be related to the seismic frequency content by the natural vibration period. Four parameters representing the frequency content are TC, TD, TM and the PGA/PGV ratio.

T_C: Period of control spectral response that is the line between maximum acceleration and maximum spectral response.

$$T_C = 2\pi \times \frac{EPV}{EPA} \quad [23] \tag{2}$$

$$T_C = 4.89 \times \frac{PGV}{PGA} \quad [24, 25] \tag{3}$$

Where:

EPA: Maximum effective speed

EPA: Maximum effective acceleration

PGV: Maximum ground speed

PGA: Maximum ground acceleration

T_D: Period of spectral response control that is the line between the maximum speed and maximum displacement of spectral response.

$$T_C = 2\pi \times \frac{EPD}{EPV} \quad [23] \tag{4}$$

$$T_C = 5.29 \times \frac{PGD}{PGV} \quad [24, 25] \tag{5}$$

EPD: The maximum effective displacement

PGD: maximum ground displacement

T_M: average period was introduced by [13].

$$T_M = \frac{\Sigma C_i^2/f_i}{\Sigma C_i^2} \quad [13] \tag{6}$$

C_i: Fourier amplitude of accelerogram

f_i: Frequency of discrete Fourier transform

$\frac{PGA}{PGV}$: This proportion can be used as an index of earthquake frequency content and also as an indicator of damage [25].

Elnashai and Di Sarno (2008) [27] divided PGA/PGV into three categories:

1- High $\frac{PGA}{PGV}$ when $\frac{PGA}{PGV} > 1.2$

2- Medium $\frac{PGA}{PGV}$ when $0.8 \leq \frac{PGA}{PGV} \leq 1.2$

3- Low $\frac{PGA}{PGV}$ when $\frac{PGA}{PGV} < 0.8$

The aforementioned records with low, medium, and high frequency contents, respectively, created maximum displacement and stress values. In this study, the ratio of PGA/PGV was used as the frequency content indicator since this proportion correlated well with the algebraic indices of T_C and T_M, as well as the stochastic index of ε [28].

2.2. Description and Characteristics of the Models

In the present study, the behavior of six 10, 15, 20, 25, 30 and 35-storey buildings with reinforced concrete core subjected to 9 earthquakes with different frequency contents were investigated. The plans of every six buildings are similar (Figure 1) and the height of all stories and span distances is 4 meters. The shear walls are located in the middle of the plan such that it came in the form of a box. Structure plans are shown in Figure 1.Buildings are located in areas of high seismic activity and soil type 3 according to the standard 2800 (Edition 4) of Iran (2800) [29].

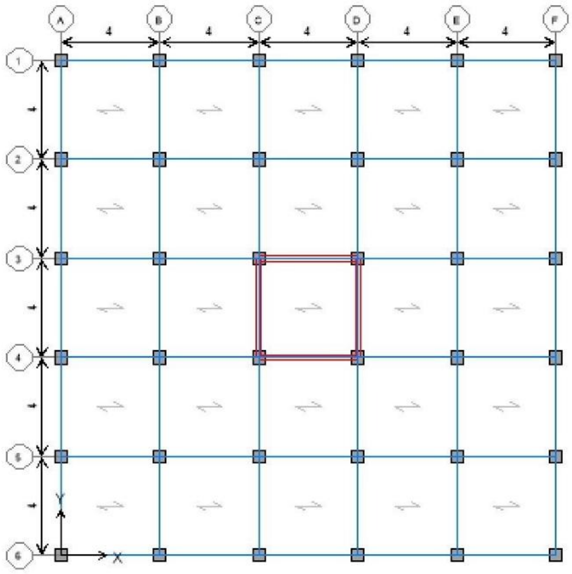


Figure 1. Architectural plan of entire buildings

2.3. Height and frequency of the buildings

2.3.1. Numerical Methods for Calculating Natural Frequency of Buildings:

Mode analysis is needed for calculating natural frequency of a vibration system in numerical method. Mode analysis is an overall identification method for dynamic features of a system. These features include special modes, natural frequencies, and mode shapes of a vibration system. The process of mode analysis using finite - element software for finding natural frequency of a model is done in direct and indirect mode analysis (using external stimulation).

- **Direct method:** The process of free vibration is simulated in the software which calculating frequencies of free vibration directly and finding shape of modes.

- **Indirect method:** After modelling in the software, system is subjected to stimulations with various frequencies, and software will calculate frequency responses. With drawing dynamic response of the system, system's basic frequency is the frequency reporting the most displacements.

Fortunately, Opensees is able to do mode analysis directly. After modelling into the Opensees, terms of free vibrations will be simulated and software will calculate frequency of free vibration of structural system and extract its modal shapes.

The height and frequency of the buildings are shown in the following table (Table 1):

Table 1. Height and frequency of the buildings

Buildings	Buildings' Height (m)	Buildings' frequency
35-Story	140	0.146
30-Story	120	0.235
25-Story	100	0.338
20-Story	80	0.395
15-Story	60	0.474
10-Story	40	0.694

2.4. Building Periods

The Periods of buildings are shown in the following table (Table 2):

Table 2. Period of the entire buildings

Modes	Buildings' Period (Sec)					
	35-Story Building	30-Story Building	25-Story Building	20-Story Building	15-Story Building	10-Story Building
Mode 1	6.8132	4.2509	2.9513	2.5295	2.1071	1.4397
Mode 2	6.1384	4.2418	2.9511	2.5294	2.1069	1.4396
Mode 3	4.1019	3.1552	2.3376	2.0236	1.7784	1.259
Mode 4	2.045	1.5402	1.1933	0.9226	0.7779	0.5235
Mode 5	2.0048	1.5331	1.1932	0.9226	0.7778	0.5234
Mode 6	1.9939	1.2947	1.0304	0.7952	0.6996	0.4912
Mode 7	1.6228	0.8898	0.6568	0.5428	0.4274	0.2915
Mode 8	1.0982	0.8873	0.6568	0.5428	0.4273	0.2915
Mode 9	1.0806	0.7863	0.5984	0.4935	0.3974	0.2719
Mode 10	0.9547	0.6114	0.4593	0.3706	0.2958	0.203

2.5. Features of Buildings' Shear Walls

The features of shear walls of all buildings are indicated in the Table 3 to 8. These features are shown respectively from 35 story building to 10 story building.

2.6. 35-Storey Building

Table 3. Features of shear walls of 35-story buildings

Story	Dimension (cm)		Reinforcement		Reinforcement	
	B [(a)]	H [(b)]	# [(c)]	φ_H [(d)]	#	φ_V [(e)]
1-5	270	30	10	12	8	20
6-10	270	30	10	12	8	20
11-15	270	30	10	12	8	18
16-20	270	30	10	12	8	18
21-25	270	25	10	12	8	16
25-30	270	20	10	12	8	14
30-35	270	20	10	12	8	14

[(a)]Shear wall length [(b)]Shear wall thickness

2.7. 30-Storey Building

Table 4. Features of shear walls of 30-story buildings

Story	Dimension (cm)		Reinforcement		Reinforcement	
	B	h	#	φ_H	#	φ_V
1-5	270	30	10	12	8	20
6-10	270	30	10	12	8	18
11-15	270	30	10	12	8	18
16-20	270	25	10	12	8	16
21-25	270	20	10	12	8	14
25-30	270	20	10	12	8	14

2.8. 25-Storey Building

Table 5. Features of shear walls of 25-story buildings

Story	Dimension (cm)		Reinforcement		Reinforcement	
	B	h	#	φ_H	#	φ_V
1-5	270	30	10	12	8	18
6-10	270	30	10	12	8	18
11-15	270	25	10	12	8	16
16-20	270	20	10	12	8	14
21-25	270	20	10	12	8	14

2.9. 20-Storey Building

Table 6. Features of shear walls of 20-story buildings

Story	Dimension (cm)		Reinforcement		Reinforcement	
	b	h	#	φ_H	#	φ_V
1-5	270	30	10	12	8	18
6-10	270	25	10	12	8	16
11-15	270	20	10	12	8	14
16-20	270	20	10	12	8	14

2.10. 15-Storey Building

Table 7. Features of shear walls of 15-story buildings

Story	Dimension (cm)		Reinforcement		Reinforcement	
	b	h	#	φ_H	#	φ_V
1-5	270	25	7	12	8	16
6-10	270	20	10	12	8	14
11-15	270	20	10	12	8	14

2.11. 10-Storey Building

Table 8. Features of shear walls of 10-story buildings

Story	Dimension (cm)		Reinforcement		Reinforcement	
	b	h	#	φ_H	#	φ_V
1-5	270	25	7	12	8	16
6-10	270	20	10	12	8	14

3. Earthquake Records

The selected records are based on the frequency contents and peak ground acceleration (PGA). In this paper, so as to obviate the need for coordinating the records, only earthquakes with PGA of 0.6 to 0.7 in the X direction were selected. Hence, the results were analysed only in the X direction. The categories of earthquakes are based on the frequency content of the PGA / PGV:

$\frac{PGA}{PGV} > 1.2$, $0.8 < \frac{PGA}{PGV} < 1.2$ and $\frac{PGA}{PGV} < 0.8$ [11, 12]

In this study, nine records, divided into three groups, were used, which, in terms of PGA/PGV, the first is larger than 1.2, the second category is between 0.8 and 1.2, and third was less than 0.8. Every map contained two components, one perpendicular to the fault and the other one parallel to it. Table 9 shows characteristics of all ground motions used:

Table 9. Records features

#	Earthquake Name	Station Name	Record Sequence Number*	Year	Magnitude	Time Duration (Sec)	Effective Time (Sec)	PGA* in X (g)	PGA in Y (g)	PGV* in X (m/sec)	PGV in Y (m/sec)	PGA/PGV* in X	PGA/PGV in Y
1	Chuetsu-oki_ Japan	JoetsuOshimaku Oka	4845	2007	6.8	59.99	4.8	0.613	0.655	0.209	0.21	2.93	3.11
2	Imperial Valley-06	Bonds Corner	160	1979	6.53	37.785	9.7	0.6	0.777	0.468	0.449	1.28	1.73
3	Victoria Mexico	Cerro Prieto	265	1980	6.33	24.52	8.2	0.645	0.633	0.336	0.176	1.91	3.59
4	Chi-Chi_ Taiwan	CHY028	1197	1999	7.62	89.995	8.7	0.636	0.76	0.614	0.858	1.03	0.88
5	Kobe Japan	Takarazuka	1119	1995	6.9	40.95	4.6	0.697	0.614	0.684	0.863	1.01	0.71
6	Loma Prieta	Corralitos	753	1989	6.93	39.98	7.9	0.645	0.483	0.56	0.476	1.15	1.01
7	Chuetsu-oki_ Japan	Kashiwazaki City Center	4856	2007	6.8	59.99	7.5	0.65	0.482	0.955	0.947	0.68	0.5
8	Chuetsu-oki_ Japan	OguniNagaoka	4874	2007	6.8	59.99	12.2	0.625	0.513	0.791	0.502	0.79	1.02
9	Kobe Japan	Takatori	1120	1995	6.9	40.95	11.3	0.618	0.671	1.207	1.23	0.51	0.54

4. Non-Linear Modelling, Structural Analysis, and Openssee

For three-dimensional modelling and structural analysis, open source software (Opensees) was used. OpenSees, the Open System for Earthquake Engineering Simulation, is an object - oriented, open source software framework. It allows users to create both serial and parallel finite element computer applications for simulating the response of structural and geotechnical systems subjected to earthquakes and other hazards.

The method of this software is based on the finite element method. Modelling in this software is in coding and there is no limit on the building and analysis of the models. In Opensees, roofs were not simulated and loads of roofs applied as distributed loads on beams. All supports were fixed and shear walls were modelled as equivalent columns, and non-linear beam column element command considering the element as a broad plasticity was used. Combination of dead load and 20% of live load was 896 kg/m^2 and also, for simulating of beams and columns, the command of beam-column element based on the displacement and also a shear wall with wide plasticity was used. To model a section of columns and beams in the software Opensees, two types of concrete were used for the core and cover in which, the FC or the 28-day strength of core concrete was equal to 220 kg/cm^2 and FC or 28-day strength of cover concrete was equal to 210 kg/cm^2. Also, the core concrete strain and concrete cover strain were considered as 0.00219 and 0.00175, respectively. To build the core and cover concrete, the material Concrete01, with zero tensile strength, was used.

Rebars yield strength or Fy was equal to 2353 kg/cm^2 and E or elasticity modulus of steel was placed at 1.99×10^6. Rebars in Opensees were made from Steel02 material which is an isotropic hardening steel. Combination of dead load and 0.2 of live load (Seismic load) is 896 kg/cm^2.

The type of analysis used in this research is a non-linear time history analysis and also, the time step to conduct the analysis was set at 0.001. Structures were compared in terms of the amount of drift, shear force of stories, and maximum displacement of the structure. It is important to note that the 10-storey building subjected to earthquake No. 7 (Kashiwazaki City Center, 2007; Chuetsu-oki_ Japan) after 25.7 seconds was lost as a result of conformity between the frequency content of the earthquake and the natural frequency of the structure. All supports in these buildings are fixed and features of used materials are shown in Table 10.

Table 10. Mechanical properties of materials

Concrete for core of columns & beams	#	Concrete for cover of columns & beams	#	Features of Steel (Rebars)	#
Compressive Strength (fpcc)	220 kg/cm^2	Compressive Strength (fpcU)	210 kg/cm^2	Yield Strength (Fy)	2353 kg/cm^2
Strain at compressive Strength (eps0c)	0.00219	Strain at compressive Strength (eps0U)	0.00175	Strain hardening ratio (b)	0.03
Crushing Strength (fpcuC)	128.15 kg/cm^2	Crushing Strength (fpcuU)	0.0	Control the transition from Elastic to Plastic Branches (R0, cR1, cR2)	15, 0.925, 0.15
Strain at Crushing Strength (epsuC)	0.00759	Strain at Crushing Strength (epsuU)	0.004	Isotropic hardening parameters (a1, a2, a3, a4)	0,1,0,1
Young Modulus (E)	200 Gpa	Young Modulus (E)	200 Gpa	Initial elastic tangent (E)	1999.347 Gpa

5. Time-History Analysis

To reproduce the energy dissipation processes developed in Reinforced concrete buildings during a seismic event, performing the nonlinear time-history analyses is indispensable. For this purpose, a nonlinear time history dynamic analysis of the buildings was performed based on the explicit dynamic method under the nine selected earthquakes. The time-history nonlinear analysis method foresees a direct step- by-step integration of the equations of motion in the time domain. In this regard, a reliable nonlinear numerical model of the buildings and proper modelling of the seismic input must be assured.

The explicit dynamics analysis procedure is based on the employment of an explicit integration rule incorporated with the use of diagonal or lumped element mass matrices. The motion differential equations are represented in terms of matrices, partitioned in constrained u (the submatrices have S as a subscript), and q (the submatrices have D as a subscript). Using time discretization, at the time step of i + 1, the motion equation can be written as:

$$\begin{bmatrix} M_{SS} & 0 \\ 0 & 0 \end{bmatrix} \begin{Bmatrix} \ddot{q}_{i+1} \\ \ddot{u}_{i+1} \end{Bmatrix} + \begin{bmatrix} C_{SS} & C_{SD} \\ C_{DS} & C_{DD} \end{bmatrix} \begin{Bmatrix} \dot{q}_{i+1} \\ \dot{u}_{i+1} \end{Bmatrix} + \begin{bmatrix} K_{SS} & K_{SD} \\ K_{DS} & K_{DD} \end{bmatrix} \begin{Bmatrix} q_{i+1} \\ u_{i+1} \end{Bmatrix} = \begin{Bmatrix} 0 \\ r_{i+1} \end{Bmatrix} \tag{7}$$

Where M, C, and K = mass, damping, and stiffness matrices, respectively.

The equations of motion for the body are integrated using the explicit central difference integration rule and can be expressed as follows:

$$\dot{u}^{(i+0.5)} = \dot{u}^{(i+0.5)} + \frac{\Delta t^{(i+1)} + \Delta t^{i}}{2} \ddot{u}^{i} \tag{8}$$

$$u^{(i+1)} = u^{(i)} + \Delta t^{(i+1)} \dot{u}^{(i+0.5)} \tag{9}$$

Where $\dot{u}$= velocity; and $\ddot{u}$= acceleration. The superscript (i) refers to the increment number and $i - 0.5$ and i + 0.5 refer to midincrement values. The central difference integration operator is explicit in that the kinematic state can be advanced using known values of $\dot{u}^{(i+0.5)}$ and $u^{(i)}$ from the previous increment.

The explicit procedure integrates through time by using many small time increments. The central difference operator is conditionally stable, and the stability limit for the operator is given in terms of the highest eigenvalue in the system. In this procedure, the NewtonLineSearch method is also used as a numerical technique for solving the nonlinear equilibrium equations. This method is usually used in large finite-element models.

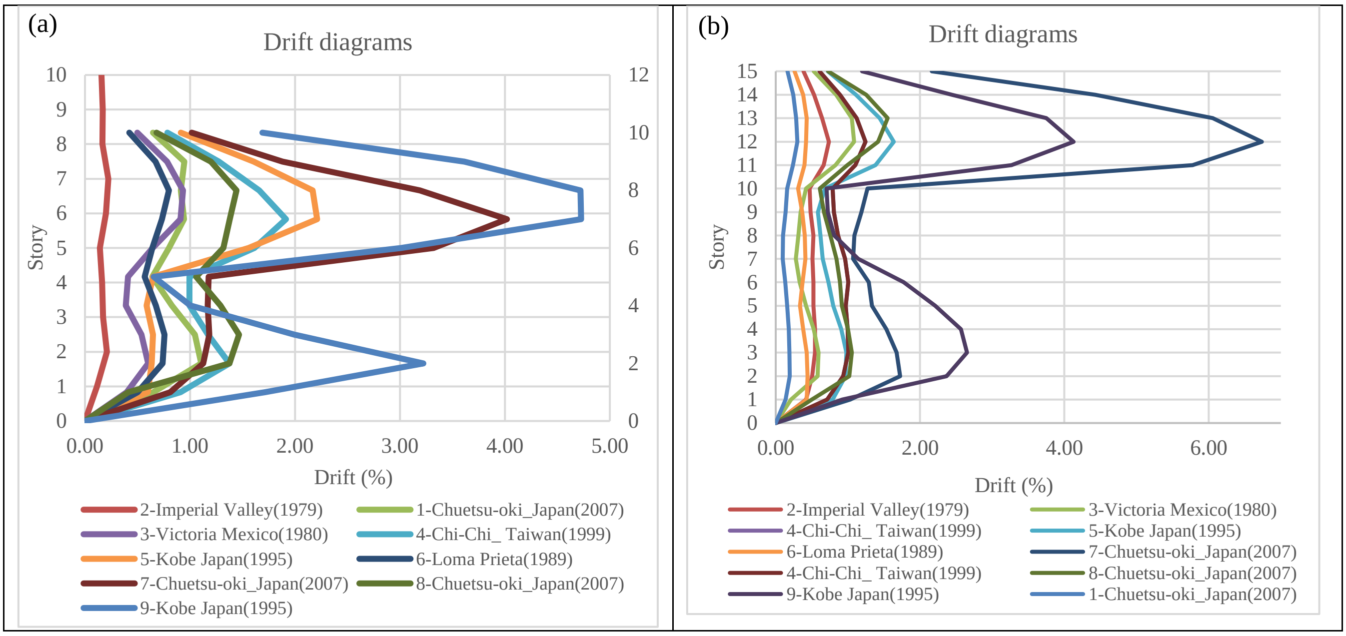
6. Results of Non-Linear Dynamic Time-History Analysis Structural Drift

Table 11. presents the maximum drift of structures. As mentioned in the selected records section and as can be seen in Table 11, all results were discussed only in the X direction.

Table 11. Maximum drift of buildings subjected to entire ground motions

#	Earthquake Name	Station Name	PGA in X axis (g)	PGA/PGV in X axis	Maximum Drift (%)					
					High Rise Buildings		Mid Rise Buildings		Low Rise Buildings	
					35 Story	30 Story	25 Story	20 Story	15 Story	10 Story
1	Chuetsu-oki_ Japan	JoetsuOshimaku Oka	0.613	2.93	0.1985	0.2117	0.2193	0.3944	0.372	0.3408
2	Imperial Valley-06	Bonds Corner	0.6	1.28	0.8033	0.9079	1.1226	1.0122	1.0058	1.274
3	Victoria Mexico	Cerro Prieto	0.645	1.91	0.4672	0.4833	0.6281	0.8981	1.0681	0.9339
4	Chi-Chi_ Taiwan	CHY028	0.636	1.03	1.5941	1.6613	2.2609	1.3663	1.4975	2.8624
5	Kobe Japan	Takarazuka	0.697	1.01	1.6942	1.7885	2.3019	2.5935	2.5275	2.2115
6	Loma Prieta	Corralitos	0.645	1.15	0.6037	0.7277	0.8156	0.927	0.945	1.0827
7	Chuetsu-oki_ Japan	Kashiwazaki City Center	0.65	0.68	3.1912	3.0981	3.391	6.641	6.7367	4.0199
8	Chuetsu-oki_ Japan	OguniNagaoka	0.625	0.79	1.107	1.3085	1.0158	1.1438	1.551	1.4675
9	Kobe Japan	Takatori	0.618	0.51	2.4373	2.159	3.2092	4.8843	4.128	5.5009

The following diagrams show the drift of every six structures subjected to all earthquakes (Figure2)



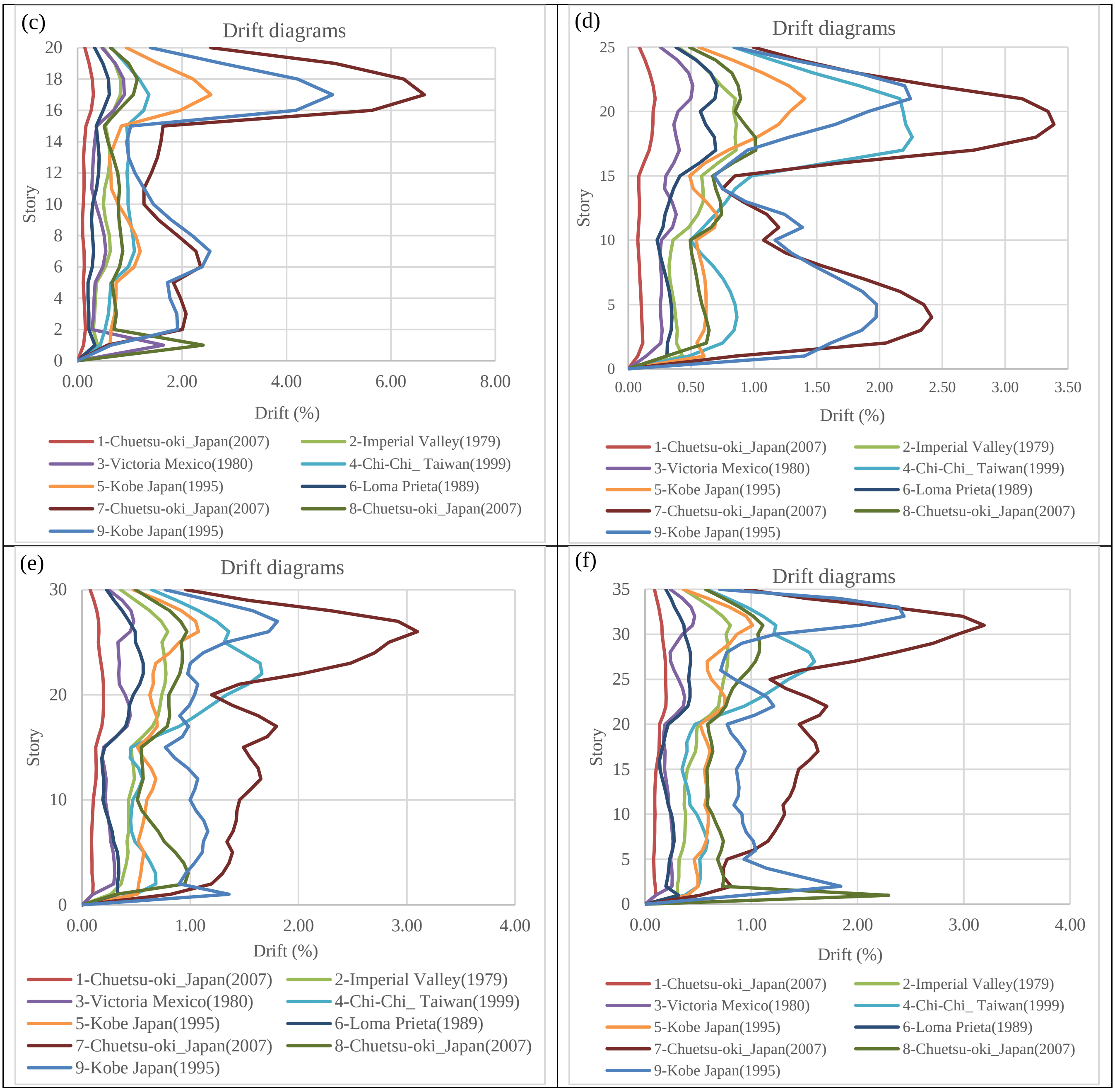


Figure 2. Drift diagrams of entire buildings subjected to all earthquakes (a) 10-Story; (b) 15-Story; (c) 20-Story; (d) 25-Story; (e) 30-Story; (f) 35-Story

Investigation of the drift results of the time history analysis showed that the maximum drift amount was 6.73% which is related to the short-rise 15-storey structures under earthquake of Chetsu-oki JapanKashiwazaki City Center 2007 which had a frequency content less than 0.8, and in Row 7 of the records table. However, it should be noted that, as mentioned already, under the effect of this quake, the 10-storey building collapsed in 25 seconds and if this had not occurred, the maximum drift would have been associated with this structure. As is clear from the graphs, the maximum drift of all structures was in the 5th storey. Given the amount of structural drifts, it becomes clear that the drift of taller structures is smaller than that of shorter structures. Therefore, the structural drift can be examined from the standpoint of frequency content in the following. In general, the amount of drift in the category of earthquakes with content less than 0.8 is more than the other two categories, and, also, the drift under the effect of the frequency content less than 1.2 and greater than 0.8 is more than the 0.8 category. But the drift subjected to the earthquake Chuetsu-oki_JapanOguniNagaoka 2007, an earthquake with a frequency content of 0.68, is less than the earthquake Kobe Japan Takarazuka1995 with a frequency content of 1.01 and Chi-Chi_ Taiwan CHY028 1999 Bamhtvay with a frequency content of 1.03. Also, the drift under the earthquake Loma PrietaCorralitos 1989 with frequency content of 1.15 is less than the Imperial Valley Bonds Corner 1979 earthquake with a frequency content of 1.28.

6.1. Shear Force of the Stories

The maximum shear force of the classes is presented in Table 12. As before in this part, the study is only in the X direction.

Table 12. The maximum shear force of stories subjected to entire ground motions

#	Earthquake Name	Station Name	PGA in X axis (g)	PGA/PGV in X axis	Stories' Shear force (ton)					
					Hig Rise Buildings		Mid Rise Buildings		Low Rise Buildings	
					35 Story	30 Story	25 Story	20 Story	15 Story	10 Story
1	Chuetsu-oki_ Japan	Joetsu Oshimaku Oka	0.613	2.93	3371.7	2814	3052.2	3698.6	2445.1	2526.1
2	Imperial Valley-06	Bonds Corner	0.6	1.28	7963.6	5950.6	9039.7	8567.54	5113.8	6265.6
3	Victoria Mexico	Cerro Prieto	0.645	1.91	4981.9	4537.5	4884.1	4411.17	4622.7	4815.4
4	Chi-Chi_ Taiwan	CHY028	0.636	1.03	8322.4	8169.84	9111.58	8913.75	6515.9	6053.74
5	Kobe Japan	Takarazuka	0.697	1.01	9676.5	7838.66	9195.6	9876	6677.43	5647.1
6	Loma Prieta	Corralitos	0.645	1.15	8391.32	7191.3	7397	7717.22	5201.2	4979.6
7	Chuetsu-oki_ Japan	Kashiwazaki City Center	0.65	0.68	10621.44	8912.1	10151.88	10515.47	6993.33	5162.5
8	Chuetsu-oki_ Japan	Oguni Nagaoka	0.625	0.79	11068.78	10173.18	8662.1	8879.2	6758.07	6534.8
9	Kobe Japan	Takatori	0.618	0.51	12332	10746.27	11421.74	11567.69	6857.7	6804.6

At first, as it stands, the greatest amount of classes' shear force subjected to all three earthquake categories is associated with a high-rise 35-storey building. The amount of shear force in the 10-storey structure subjected to 3earthquakes with frequency content higher than 1.2 is more than that of the 15-storey structure, but this was not so in the other categories.

As was predicted, in a 35-storey structure the shear force in earthquake categories with low frequency content was higher than the other categories, and as the amount of frequency content increased, the amount of this force decreased. As was expected, by reducing the height of structures, the shear force of the storeys was reduced and a direct impact of frequency content on the structural response was visible because the effect of fluctuations in the frequency content on this force was evident in all structures. Accordingly, it can be concluded the higher frequency content, the lower shear force and vice versa. However, there are two things that need to be mentioned:

1- Short-rise 10-storey buildings collapsed when subjected to Chetsu-oki_JapanKashiwazaki City Center 2007 earthquake, but if it had not happened, then, certainly, the 10-storey structure under this earthquake would have the maximum shear force than other earthquakes due to the equality of earthquake frequency content and structural frequency.

2- In the 15-storey building, the highest amount of shear of the stories under Chetsu-oki_JapanKashiwazaki City Center 2007 earthquake occurred. Although this earthquake did not have the largest frequency content, due to the approaching overhead earthquake frequency content to the structural frequency, the maximum shear force affected by the earthquake was generated.

For better understanding of Table 12, the aspect ratio diagram (Figure 3) including all aspect ratios for all structures subjected to all earthquakes is presented.

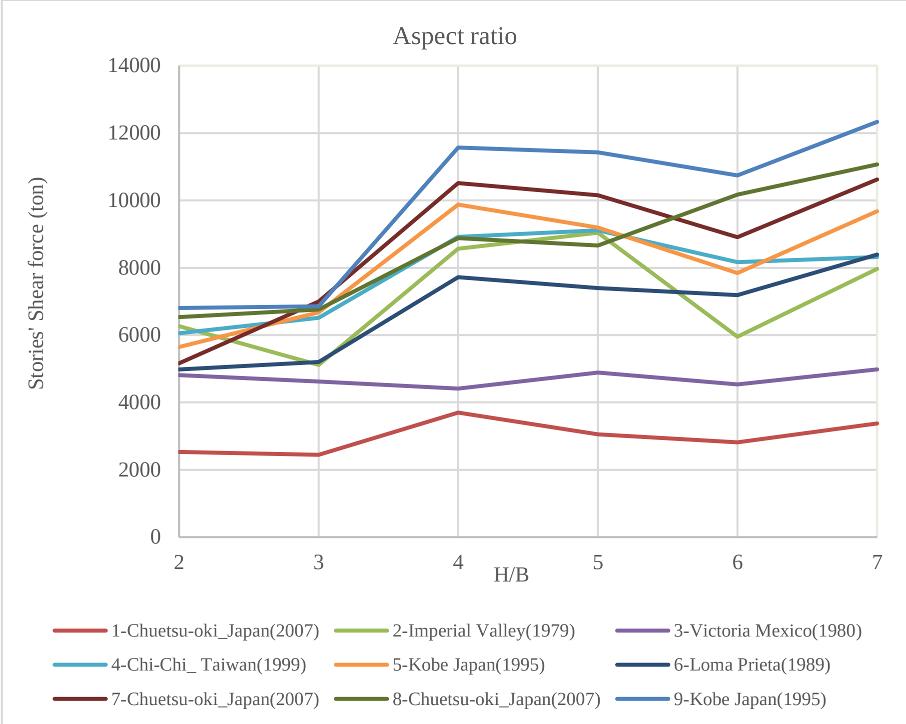


Figure 3. Aspect ratio of shear force of stories subjected to entire ground motions

6.2. The Maximum Displacement of Stories

The maximum displacement of the classes is shown in Table 13.

Table 13. The maximum displacement of stories subjected to entire ground motions

#	Earthquake Name	Station Name	PGA in X axis (g)	PGA/PGV in X axis	Maximum Displacement (m)					
					Hig Rise Buildings		Mid Rise Buildings		Low Rise Buildings	
					35 Story	30 Story	25 Story	20 Story	15 Story	10 Story
1	Chuetsu-oki_ Japan	Joetsu Oshimaku Oka	0.613	2.93	0.15	0.15	0.09	0.07	0.07	0.05
2	Imperial Valley-06	Bonds Corner	0.6	1.28	0.55	0.47	0.43	0.37	0.34	0.30
3	Victoria Mexico	Cerro Prieto	0.645	1.91	0.13	0.14	0.16	0.17	0.15	0.17
4	Chi-Chi_ Taiwan	CHY028	0.636	1.03	0.45	0.48	0.80	0.61	0.50	0.58
5	Kobe Japan	Takarazuka	0.697	1.01	0.54	0.52	0.67	0.68	0.65	0.42
6	Loma Prieta	Corralitos	0.645	1.15	0.21	0.26	0.29	0.27	0.23	0.26
7	Chuetsu-oki_ Japan	Kashiwazaki City Center	0.65	0.68	0.66	0.59	0.85	0.95	0.84	0.70
8	Chuetsu-oki_ Japan	Oguni Nagaoka	0.625	0.79	0.75	0.83	0.44	0.30	0.30	0.31
9	Kobe Japan	Takatori	0.618	0.51	0.48	0.62	0.53	0.83	0.68	0.70

The greatest displacement according to Table 13 is related to the 20-storey structure, and, then, 25, 15, 30, 35 and 10-storey structures had the highest displacements, respectively. The 10-storey structure under earthquake Chetsu-oki_JapanKashiwazaki City Center 2007 had the greatest displacement, and due to the collapse of the structure, the displacement can be compared with other structures. As a result, the maximum displacement in the 35-storey structure was lower than that of other structures'. In the following, the aspect ratio diagram of maximum displacement under earthquake (Figure 4) is presented to better describe changes in the displacements.

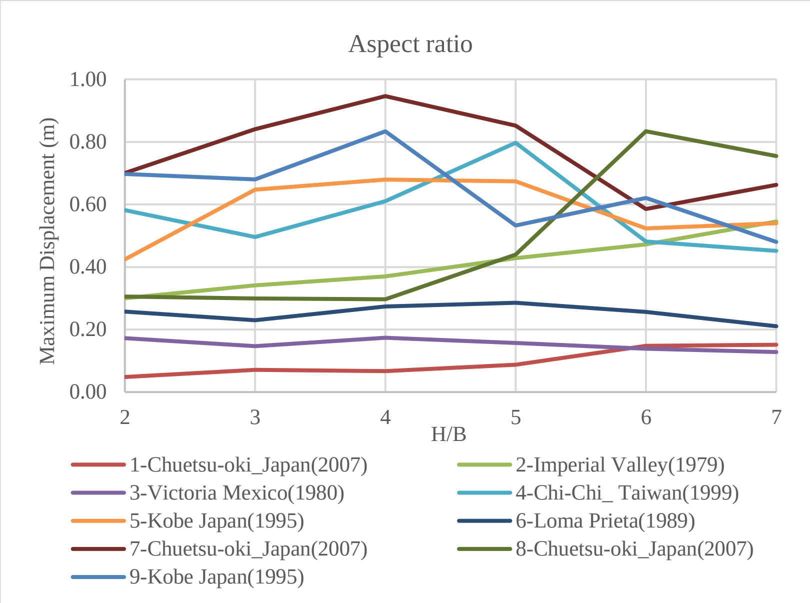
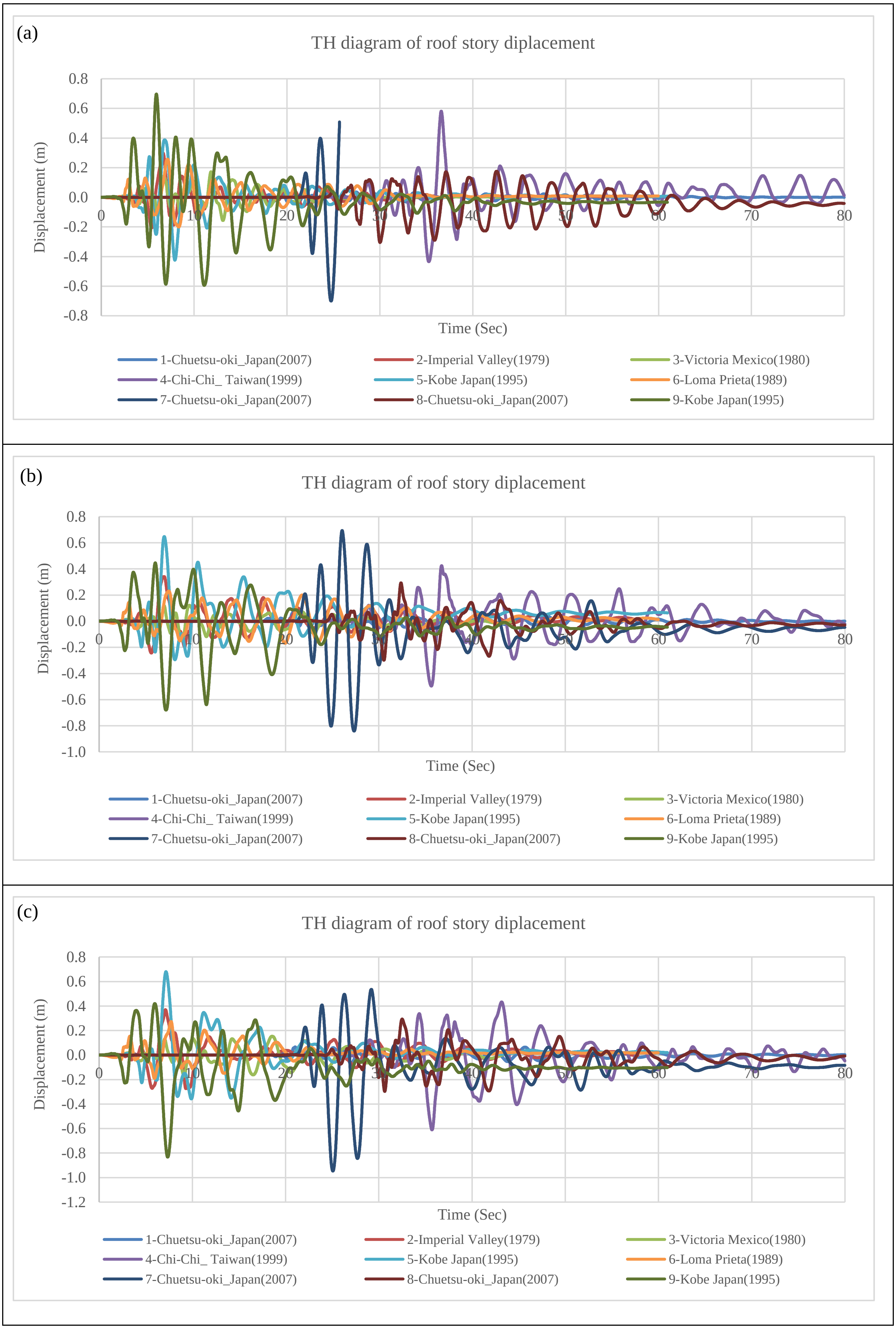


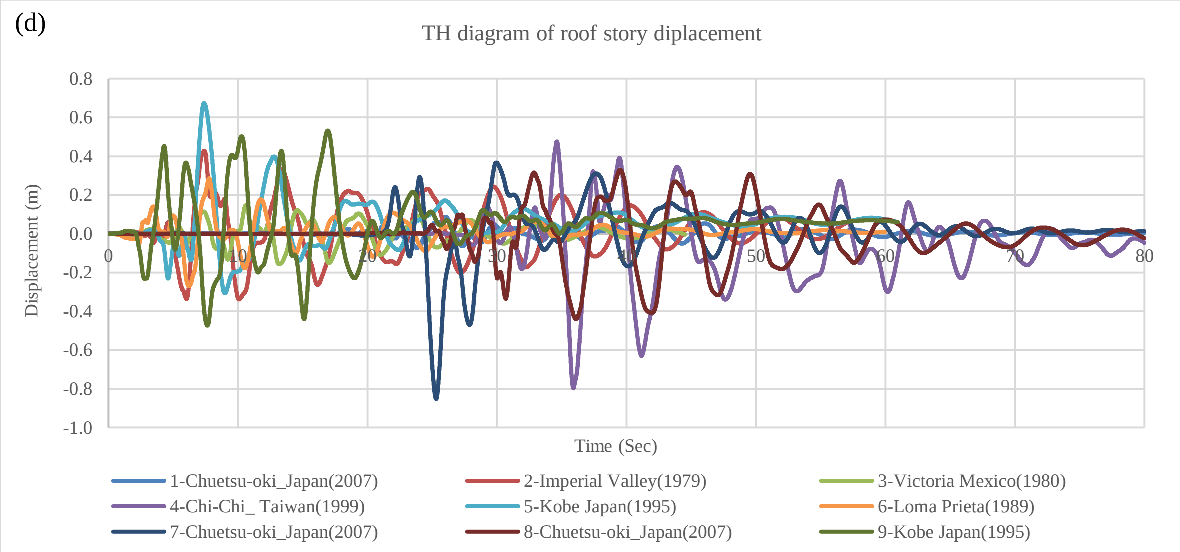
Figure 4. Aspect ratio of maximum displacement of stories subjected to entire ground motions

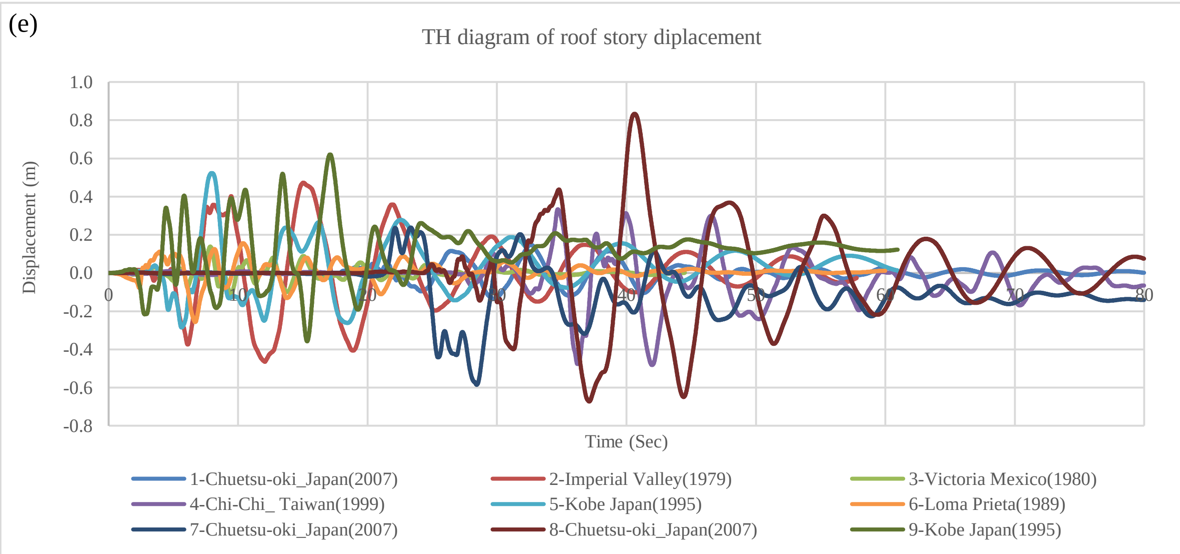
In relation to the frequency content in this section, it is not possible to express conclusively if the displacement will be decreased or increased by changing the frequency content. This will be discussed in the following section:

- 35-storey building: the maximum displacement of this building under earthquake Kobe Japan Takatori 1995, which recorded the lowest frequency content, was less than the maximum displacement under the earthquakes Chuetsu-oki_JapanOguniNagaoka 2007, the Chuetsu-oki_JapanKashiwazaki City Center 2007, the Kobe Japan Takarazuka 1995 and Imperial Valley Bonds Corner 1979.
- 30-storey building: the maximum displacement of this building under the earthquake Chuetsu-oki_JapanOguniNagaoka 2007 was more than the maximum displacement under the earthquake Kobe Japan Takatori 1995.
- 25-storey building: the maximum displacement of this building under the earthquakes Chuetsu-oki_JapanKashiwazaki City Center 2007 with a frequency content of 0.68 and CHY028 Chi-Chi_ Taiwan 1999 with a frequency content of 1.03 was higher than the other earthquakes, even Kobe Japan Takatori 1995, with a content frequency of 0.51.
- 20-storey building: in this building, as expected, a maximum displacement in earthquakes with a frequency content of less than 0.8 was more than the other categories, but, still, under earthquake Chetsu-oki_JapanKashiwazaki City Center 2007 with frequency content of 0.68, it was more than the earthquake in Kobe Japan Takatori 1995 with frequency content of 0.51. Conversely, under the effect of earthquake Chuetsu-oki_JapanOguniNagaoka 2007 with frequency content of 0.79, it was less than some earthquakes in other categories.
- 15-storey and 10-storey buildings: the maximum displacement in both structures was the same as that of the 20-storey structure.

In the following, displacement time history graphs of the roof level for six structures subjected to all earthquakes are displayed (Figure 5):


(a)
TH diagram of roof story diplacement
Displacement (m)
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
-0.8
0
10
20
30
40
50
60
70
80
Time (Sec)
1-Chuetsu-oki_Japan(2007)
2-Imperial Valley(1979)
3-Victoria Mexico(1980)
4-Chi-Chi_ Taiwan(1999)
5-Kobe Japan(1995)
6-Loma Prieta(1989)
7-Chuetsu-oki_Japan(2007)
8-Chuetsu-oki_Japan(2007)
9-Kobe Japan(1995)
(b)
TH diagram of roof story diplacement
Displacement (m)
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
-0.8
-1.0
0
10
20
30
40
50
60
70
80
Time (Sec)
1-Chuetsu-oki_Japan(2007)
2-Imperial Valley(1979)
3-Victoria Mexico(1980)
4-Chi-Chi_ Taiwan(1999)
5-Kobe Japan(1995)
6-Loma Prieta(1989)
7-Chuetsu-oki_Japan(2007)
8-Chuetsu-oki_Japan(2007)
9-Kobe Japan(1995)
(c)
TH diagram of roof story diplacement
Displacement (m)
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
-0.8
-1.0
-1.2
0
10
20
30
40
50
60
70
80
Time (Sec)
1-Chuetsu-oki_Japan(2007)
2-Imperial Valley(1979)
3-Victoria Mexico(1980)
4-Chi-Chi_ Taiwan(1999)
5-Kobe Japan(1995)
6-Loma Prieta(1989)
7-Chuetsu-oki_Japan(2007)
8-Chuetsu-oki_Japan(2007)
9-Kobe Japan(1995)





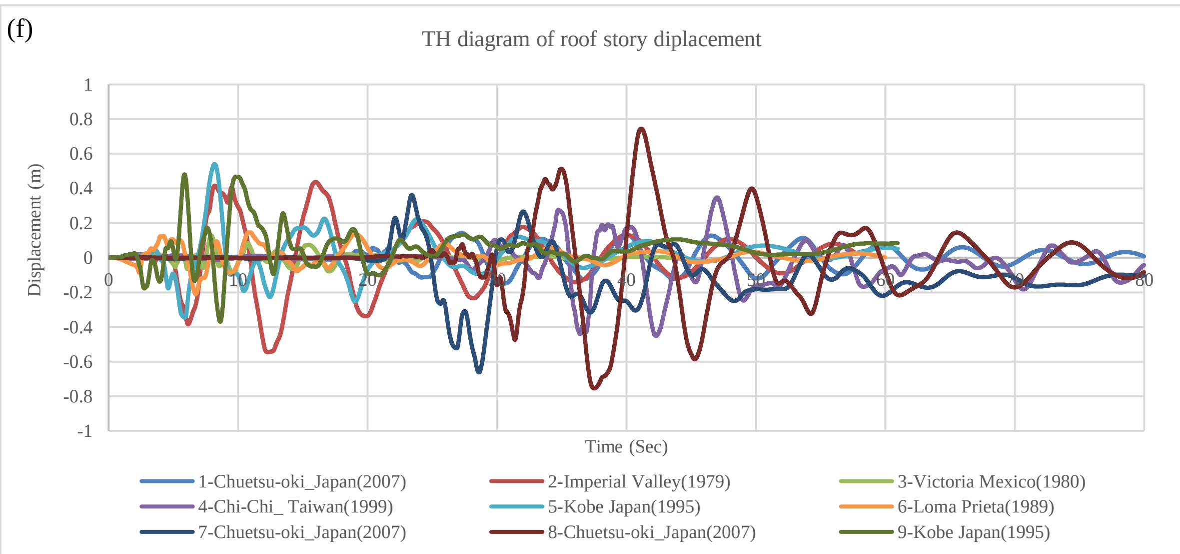


Figure 5. Time-History diagram of roofs displacements of entire building (a) 10-Story; (b) 15-Story; (c) 20-Story; (d) 25-Story; (e) 30-Story; (f) 35-Story

7. Conclusion

In this study, the seismic performance of six reinforced concrete 10, 15, 20, 25, 30, and 35-storey structures with reinforced concrete core subjected to different frequency contents of earthquakes were studied. To evaluate the performance of the mentioned structures, dynamic nonlinear time-history analysis was used, and the following results were obtained:

1. Reviewing the drift of the structures, the 10, 15, and 20-storey structures drifted more than the other structures. For example, under the effect of earthquakes with frequency content of 0.51, the drifts of the 10-storey structure were, respectively 25, 11, 42, 62, and 56 percent more than that of the 15, 20, 25, 30, 35-storey structures. It can be predicted that with increasing height, the drift lowers. As expected, the greatest drifts occurred in structures subjected to earthquakes with low frequency content.

2. In all three categories of frequency contents of 20, 25, and 35-storey structures, shear force were close to one another, but the 30-storey building had less shear force than the others. In other words, in terms of shear force, it can be concluded that with regard to height, high-rise buildings performed better than shorter structures.

3. In examining the maximum displacement of structures, generally by reducing the frequency content, structural displacements were increased, but this theory is not always correct. For example, in the 35-storey structure the maximum displacement under earthquake with frequency content of 0.51 was 14.5% and 12.5% less than earthquakes with frequency content of 1.28 and 1.01. This trend was also found in other structures.

4. In this study, it was found that changes which resulted from earthquakes in two high-rise structures had better coordination to earthquake frequency content changes as compared to the shorter structures, and this can be applied in designing high-rise structures for the prediction of seismic performance.

The effects of frequency content of earthquakes on the structures were very important and impressive, so that in the 10-storey building with the natural structural frequency, approximately equal to the frequency content of the earthquake Chuetsu-oki_Japan Kashiwazaki City Center 2007, the building collapsed.

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