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Dynamic identification of soil-structure system designed by direct displacement-based method for different site conditions

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Abstract. This study mainly aims to assess the performance of soil-structure systems designed by direct displacement-based method coupled with strong column-weak beam design concept through various system identification techniques under strong ground motions. To this end, various system identification methods are employed to evaluate the dynamic characteristics of a structure (i.e., modal frequency, system damping, mode shapes, and plastic hinge formation pattern) under a strong seismic excitation considering soil-structure interaction for different site conditions as specified by ASCE 7-10. The scope of the study narrowed down to the code-complying low- to high-rise steel moment resisting frames with various heights (4, 8, 12, 16-story). The comparison of the result of soil-structure systems with fix-based support condition indicates that the modal frequencies of these systems are highly influenced by the structure heights, specifically for the softer soils. This trend is more significant for higher modes of the system which can considerably dominate the response of structures in which the higher modes have more contribution in dynamic response. Amongst all studied modes of the vibration, the damping ratio estimated for the first mode is relatively the closet to the initial assumed damping ratios. Moreover, it was found that fewer plastic hinges are developed in the structure of soil-structure systems with a softer soil which contradicts the general expectation of higher damageability of such structural systems.

Keywords: direct displacement design; dynamic structural identification; soil-structure system; steel moment resisting structures

1. Introduction

System identification generally deals with the problem of building mathematical models of dynamical systems based on recorded input and output to evaluate the dynamic system parameters. In recent years, system identification techniques have been developed and mainly used specifically in the field of civil engineering to determine dynamic characteristic of a structural system to be utilized the analytical model such as Fnite Element (FE) model to reevaluate the structural performance, damage detection, and health monitoring of a structural system under extreme dynamic events, such as seismic ground motions or high-speed winds (Mottershead and Friswell 1993, Teughels and Roeck 2005, Ebrahimian *et al.* 2017). The system identification analysis was first used in the 1940s in order tounderstand the behavior of aircraft. However, the U.S. Coast and Geodetic Survey had started examining the performance of existing buildings in the 1930s. In 1964, Crawford and Ward (1964) applied the identification methods to determine the three first vibration modes of a 19-story building using the power spectrum of signals recorded during random excitations of winds. Experimental identification of a structure aims at deriving modal parameters including modal frequencies, damping ratios, and mode shapes from the dynamic response of a structure. These parameters mostly are identified by employing a pair of recorded input and output of a system, especially when the input motion of the structure has well-defined principle directions (Safak 1991). For proper selection of recorded input and output response, dynamic modal parameters are required to be identified that depict the behavior of the structures alone (fixed-base) and the soil-structure system (Tileylioglu 2008).

Numerous studies have been addressed that the properties of underlying soil can influence the overall dynamic behavior of a building (Luco *et al.* 1988, Safak 1995, Snieder and Safak 2006, Messioud *et al.* 2016). Moreover, site-specific properties of soil, such as soil type, soil stratification and variation in depth of each layer need to be taken into account. Therefore, it would be hardly possible to have a realistic estimation of responses of a building without the inclusion of the soil in the structural model. Luco *et al.* (1998) reported one of the early works on the derivation of interactional effects from the dynamic

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response of a building. In that study, they conducted forced vibration tests for a range of frequencies using seismic exciters placed on the roof of the building. They accordingly recorded structural responses in four specific points of the building. Thereafter, they tried to determine interactional effects using recorded frequency responses. Safak (1995) conducted one of the first studies on interactional effects due to earthquake records. Following that Snieder and Safak (2006) used pulse response function obtained from the accelerograms recorded in 10-story Millikan library during Yorba Linda earthquake in 2002 to extract dynamic properties of the structure (Todorovska 2009a). Building of a typical school in Istanbul, Turkey was monitored for earthquake safety using instrumentation and system identification (Bakir 2012). In recent decades, many research has been conducted on the identification of soil-structure systems (Wolf 1985 and 1994, Wu *et al.* 2001, Todorovska 2009b, Mahmoudabadi *et al.* 2017) and it contended that the variations in identified vibration characteristics of a structure with different amplitudes are highly related to the behavior of the underlying soil in various domains of vibrations. In other words, the behavior of soil in low amplitude and high amplitude vibrations can greatly influence the system identification of structural properties. Ghahari *et al.* (2016) conducted another study on the Millikan library benchmark using a developed blind identification method (output-only data) to determine the vibration modes of the structure which were not previously identified. They used a Finite Element model in order to consider the effect of the soil-structure interaction. As experimental studies regards, Chen *et al.* (2013) developed a procedure using geotechnical centrifuge-based data for conducting seismic system identification for soil-structure interaction. In addition, Khosravikia *et al.* (2017 and 2018b) also examined the impact of the presence of soil-structure interaction in structural modeling on the building loss estimation. Ganjavi *et al.* studied the influence of soil-structure interaction on the seismic response modification factors of stiffness degrading systems (Ganjavi *et al.* 2018). Mortezaie and Rezaei studied the influence of soil-structure interaction on the seismic performance of tall building equipped with tuned mass dampers (TMS) (Mortezaie and Rezaei 2018).

The direct displacement-based design method is the main alternative for the conventional method of building seismic design. The methodology is well-documented (Priestley *et al.* 2003 and 2007) and formalized (FIB 2003). The behavior of a structure designed using direct displacement-based method was evaluated using the full-scale shake table test (Chen *et al.* 2016). Out of all research studies, a limited number of studies have been investigated the impact of soil-structure interaction on structure designed by the direct displacement-based method (Jafarieh and Ghannad 2006, Suarez and Kowalsky 2001, Mahmoudabadi 2013, Calvi *et al.* 2014), in which none of them considered the system identifications methods to determine the dynamic characteristics of the soil-structure system considering different site conditions and structure height. This is the main gap observed in the literature which the current study endeavors to bridge. This research tries to shed light on the seismic performance of moment resisting building designed using the direct displacement-based method from a different perspective. This paper investigates the dynamic characteristics of steel moment frames designed based on the direct displacement-based method located in the city of Tehran, where is considered as high seismic region due to having numerous intense earthquakes over the past decades, with different site conditions through different system identification approaches.

2. Direct displacement-based design method

Historically, design codes mainly require structures to be designed for a minimum lateral load. This approach could be traced back to early efforts for imitating wind design provisions. However, it is observed over years that ductility plays more important role than the strength. Consequently, a new displacement design approach emerged and became a viable alternative for the conventional force-based approach. This section describes the displacement-based design approach employed for the design of structural models adopted for the current course of study.

2.1 Design Methodology

The direct displacement-based design method, which is employed in this study, is originally proposed by Priestley *et al.* (2007) and then developed as implemented by Abadi and Bahar (2018) for the design of steel moment resisting frame structures. The design procedure is summarized as follows:

1- Displacements at each story of the structure should be computed using a design code specified target inter-story drift, which is assumed to be 2% for the design of studied archetypes.

2- The target displacement is obtained using the following Eq. 1,

$$\Delta_d = \frac{\sum_{i=1}^{n} m_i \Delta_i^2}{\sum_{i=1}^{n} m_i \Delta_i} \tag{1}$$

where m_i is the mass of the ith floor, Δ_i is displacement of the ith floor, and n is the number of stories.

3- Mass and height of the equivalent single degree of freedom system (SDOF) can be calculated using Eqs. 2 and 3 as follows,

$$M_{eq} = \frac{\sum_{i=1}^{n} m_i \Delta_i}{\Delta_d} \tag{2}$$

$$h_{eq} = \frac{\sum_{i=1}^{n} m_i \Delta_i H_i}{\sum_{i=1}^{n} m_i \Delta_i} \tag{3}$$

Table 1 Steel frame archetypes designed based on Direct Displacement method

Story No.	16-Story		12-Story		8-Story		4-Story	
	Column	Beam	Column	Beam	Column	Beam	Column	Beam
1	W18x130	W18x35	W18x119	W18x35	W18x97	W18x46	W18x40	W18x35
2	W18x106	W18x40	W18x86	W18x50	W18x71	W18x55	W18x35	W18x35
3	W18x97	W18x46	W18x76	W18x50	W18x65	W18x55	W18x35	W18x35
4	W18x86	W18x46	W18x76	W18x50	W18x60	W18x50	W18x35	W18x35
5	W18x86	W18x46	W18x76	W18x46	W18x50	W18x46		
6	W18x76	W18x46	W18x65	W18x46	W18x46	W18x35		
7	W18x76	W18x46	W18x60	W18x46	W18x35	W18x35		
8	W18x71	W18x46	W18x50	W18x40	W18x35	W18x35		
9	W18x65	W18x46	W18x46	W18x35				
10	W18x60	W18x40	W18x40	W18x35				
11	W18x55	W18x40	W18x35	W18x35				
12	W18x50	W18x40	W18x35	W18x35				
13	W18x40	W18x35						
14	W18x35	W18x35						
15	W18x35	W18x35						
16	W18x35	W18x35						

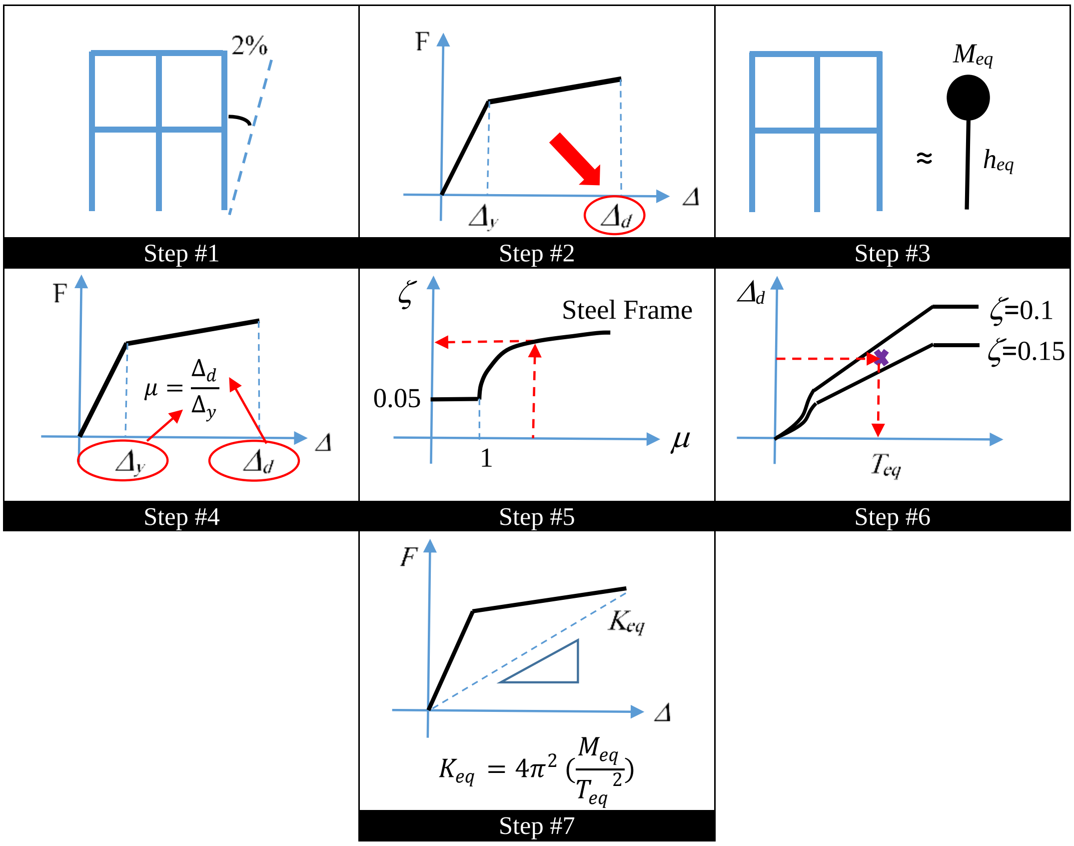

Fig. 1 Geometry and coordinates of a functionally graded beam resting on the elastic foundation

where H_i is the total structure height.

4- Design ductility ($\mu\Delta$) then is assessed using Eq. 4,

$$\mu_\Delta = \frac{\Delta_d}{\Delta_y} \tag{4}$$

Yield displacement (Δ_y) can be estimated in different ways depending on the structural systems.

5- Equivalent viscous damping (ζ_{eq}) can be obtained by summation of elastic damping (i.e., 5%) and hysteretic damping. Hysteretic damping needs to be predicted using an empirical relationship (Abadi and Bahar 2018).

Table 2 Cone model parameters for different soil categories

Cone Model Parameter	Equation	Vs = 50 m/s	Vs = 150 m/s	Vs = 350 m/s	Vs = 700 m/s
K	$(\rho V_s^2 A_0)/z_0$	$5.14\text{x}10^7$	$4.63\text{x}10^8$	$2.52\text{x}10^9$	$1.01\ \text{x}10^{10}$
C	$\rho V_s A_0$	$2.32\text{x}10^5$	$6.96\text{x}10^5$	$1.62\text{x}10^6$	$3.25\text{x}10^6$
C'	$(2\xi_0 K)/\omega_0$	$1.52\text{x}10^6$	$4.56\text{x}10^6$	$1.06\text{x}10^7$	$2.13\text{x}10^7$
m'	$(\xi_0 C)/\omega_0$	$3.43\text{x}10^3$	$3.43\text{x}10^3$	$3.43\text{x}10^3$	$3.43\text{x}10^3$
$K\varphi$	$(3\rho V_s^2 I_0)/z_0$	$3.16\text{x}10^8$	$2.84\text{x}10^9$	$1.55\text{x}10^{10}$	$6.19\text{x}10^{10}$
$C\varphi$	$\rho V_s I_0$	$3.79\text{x}10^6$	$9.12\text{x}10^6$	$2.65\text{x}10^7$	$4.26\text{x}10^7$
$C'\varphi$	$(2\xi_0 K_\varphi)/\omega_0$	$1.43\text{x}10^6$	$4.28\text{x}10^6$	$9.98\text{x}10^6$	$2.00\text{x}10^7$
$m'\varphi$	$(\xi_0 C_\varphi)/\omega_0$	$6.86\text{x}10^3$	$6.86\text{x}10^3$	$6.86\text{x}10^3$	$6.86\text{x}10^3$

V_s: Shear wave velocity
ρ: Soil specific density
z_0: Apex height of cone model
ζ_0: Soil damping ratio
ω_0: Fundamental frequency of soil-structure system
A_0: Foundation area
I_0: Foundation moment of inertia

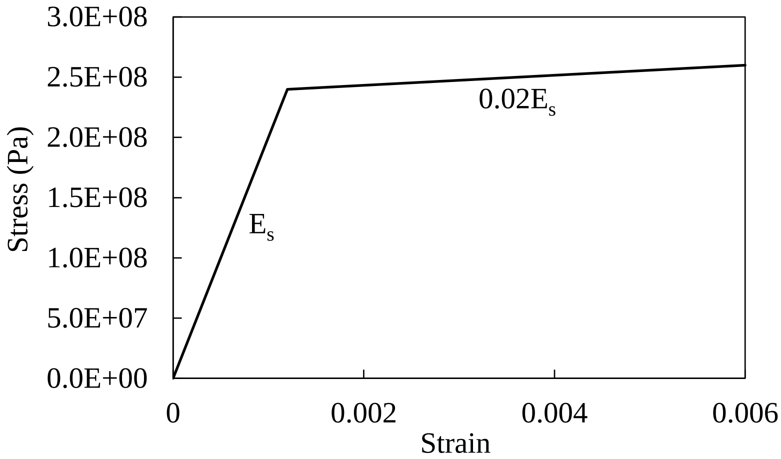

Fig. 2 Bilinear steel strain-stress curve with 2% strain hardening ratio

6- The equivalent period of vibration (T_{eq}) or a given Δ_d and ζ_{eq} that are obtained using a graph available in Abadi and Bahar (2018).

7- Stiffness of the equivalent SDOF and the design base shear (F) can be obtained as follows (Eqs. 5 and 6),

$$k_{eq} = 4\pi^2 \left(\frac{M_{eq}}{T_{eq}^{\ 2}}\right) \tag{5}$$

$$F = k_{eq} \Delta_d \tag{6}$$

Finally, after all, each steel frame is designed for the base shear obtained from the above procedure. Additionally, the sizes of beams and columns are determined in a way to observe the strong column-weak beam rule using the stiffness ratio of these members (Eq. 7),

$$\frac{\sum(\frac{I}{l})_{Beam}}{\sum(\frac{I}{l})_{Column}} < 0.5 \tag{7}$$

where I is the moment of inertia and l is the length. The application process of a structure design using the direct displacement-based method is illustrated in Fig. 1. Please note Eq. 7 is not the same of strong column-weak beam rule commonly utilized in the most codes which is based on the strength ratio.

2.2 Structure Archetype and Geometry

Four 2-D steel moment resisting frames with different heights (4, 8, 12 and 16-story) are considered in this study. The structures are designed using direct displacement-based method considering the strong column-weak beam concept as discussed in details in the previous section. It is assumed that these structures are located in a very high seismic region. The typical story height is 3 m. There are three bays in the frame with an equal span of 6 m. The details of designed frames including column and beam sizes are listed in Table 1. It should be noted that the Wide-Flange sections (W-Section) are taken into account for designing all structure beams and columns.

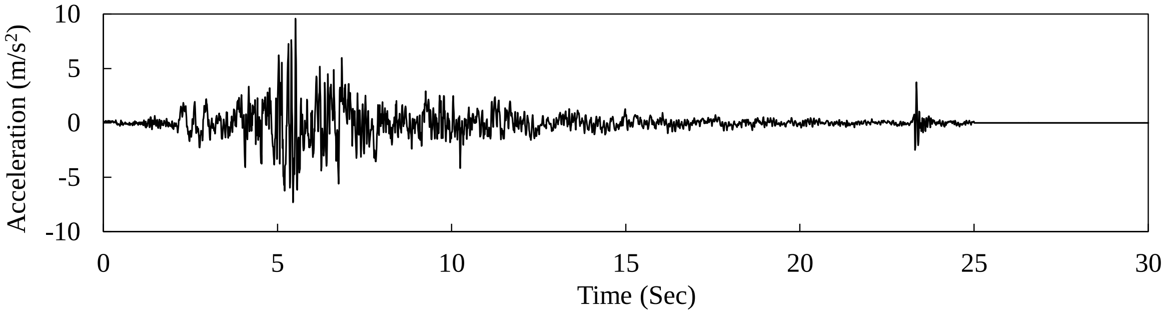

Fig. 3 Acceleration recorded in strong Taiwan earthquake, Chi-Chi 1999

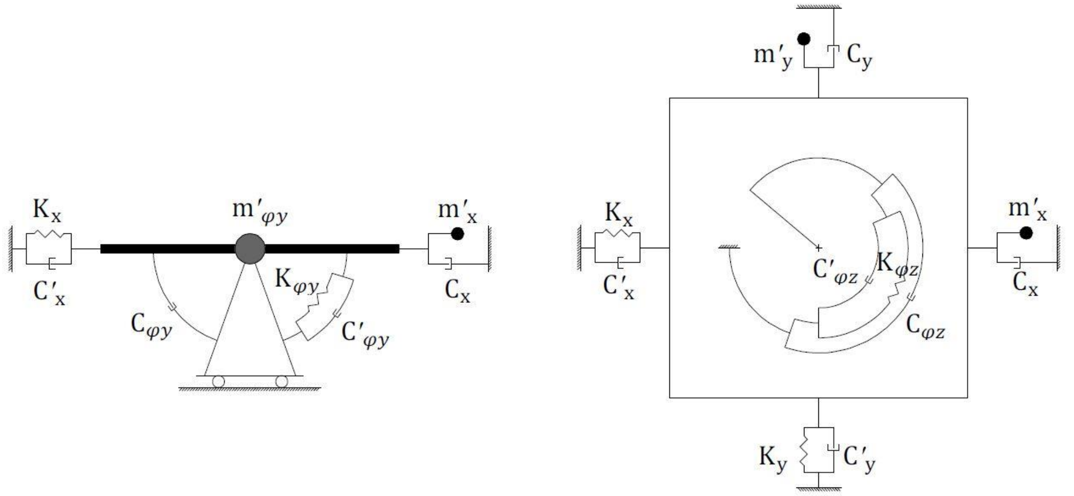

Fig. 4 Cone model for soil-structure interaction effect (Kenarangi and Rofooei 2010)

2.3 Computer Modeling and Material Properties

In order to have a more accurate and faster nonlinear dynamic analysis of studied structures under seismic excitations, OpenSEES is chosen in this study to predict nonlinear dynamic responses of the structure (Mazzoni 2006). All structural members including beams and columns are modeled using the fiber elements which are discretized in 10 segments. The material behavior of the steel is assumed to be a bilinear model with 2% strain hardening ratio and the initial modulus of elasticity (E_s) and yield stress (F_y) equal to $2.04\text{x}10^{11}$ N/m^2 and $2.5\text{x}10^8$ N/m^2, respectively (Fig. 2). To evaluate the structure under seismic excitation. Each structure is subjected to a strong component of the Chi-Chi (Taiwan) 1999 earthquake which has the peak acceleration of motion close to 1g and categorized as near-field earthquake with shallow epicenter (Fig. 3). It should be noted that the damping ratio of the structure is considered to be constant for all vibration modes and equal to 5% using the Rayleigh damping method (Rayleigh 1954).

2.4 Soil-structure interaction model

Methods for considering and modeling a soil profile in soil-structure systems can be classified into two general categories; direct method and substructure method. In the direct method, a subsection of the underlying soil appears to be integrated by the structure is modeled, and soil free-field excitation is applied on the model boundaries (Wolf 1994).

In the substructure method, the soil-structure system is divided into two parts; the first part is the structure itself located on the foundation and the second part is the soil part with a common border with the foundation. To apply the substructure method, first, force-soil displacement relationships (dynamic rigidity) needs to be determined for the nodes residing on the common border, which can be idealized in a physical form by a number of masses, springs, and dampers which their properties depend on the soil shear wave velocity and foundation type and geometry. Then, the soil-structure system is analyzed by exerting seismic excitation on the interface nodes. Foundation and subsurface soil model consists of a set of frequency-independent masses (m), springs (K) and dampers (C). Therefore, even the most complex soil-structure system can be broken down into two manageable parts resulting in analyses with a lower computational time cost. In this study, the underlying soil is considered as homogeneous half-space and is modeled by a discrete model based on the concept of substructure method using the Voigt Viscoelastic Cone Models (Wolf 1985, Wolf and Meek 1992 and 1993). In this model, the soil under the foundation is modeled as a divergent cone, and displacement in soil is exerted through the soil-structure interface nodes. Principles used in

Table 3 Verification of OpenSEES model with SAP2000

Frame Fixity		Frequency (Hz)		Displacement (m)	
		OpenSEES	Sap2000	OpenSEES	Sap2000
Fix	Mode #1	1.645	1.661	0.087	0.087
	Mode #2	5.348	5.405		
	Mode #3	9.901	10.00		
	Mode #4	14.085	14.286		
Vs = 50 m/s	Mode #1	1.634	1.637	0.086	0.086
	Mode #2	5.236	5.291		
	Mode #3	9.615	9.709		
	Mode #4	13.889	14.085		

obtaining equations, which are dominant in these models, are based on the beam theory. The foundation model in the study was assumed to rest on the ground surface for all structure models. The presented soil model has four degrees of freedom for sway and rocking motions about x and y directions and one degree of freedom for the torsion about the z-axis. Sway in the z-direction is not allowed. In order to assess the effect of different site conditions, four different soil models with shear velocities of 50, 150, 350, and 700 m/s are considered in this study. The values of soil shear wave velocities are selected somehow to represents the site classes of soft to dense soils based on the ASCE 7-10 (ASCE 7, 2010). The soil Poisson's ratio (υ), density (ρ), and initial damping ratio (ζ_0) are set to 0.3, 2000 kg/m^3, and 0.05, respectively. The soil model that has been used in this study is shown in Fig. 4. The soil model parameters shown in Fig. 4 then are calculated as summarized in Table 2.

2.5 Model verification

In order to validate the OpenSees modeling, the result obtained from OpenSees is compared with SAP2000 (CSI 2017). The verification is conducted for a 4-story 2-D steel moment resisting frame with two different support boundary conditions subjected to El-Centro (1940) ground motion. In the first model, the soil-structure interaction is entirely ignored, and the structure support condition is assumed to be fully restrained at the base (fixed-based). For the second one, the structure was assumed to rest on a soft soil condition with V_s of 50 m/s. In order to verify the models, the results of modal frequencies of the system and displacement of the roof floor from OpenSees and SAP2000 (CSI 2017) are compared for both support conditions. Noteworthy, modal frequencies of each model tend to vary throughout nonlinear dynamic analyses, and therefore, frequencies reported in Table 3 are obtained at the end of each analysis.

3. System identification for soil-structure model

Determination of dynamic parameters of a structure (i.e., damping ratio, mode shapes, and structural modal frequencies) using the system identification methods are widely used to update the structure finite element model to have a better understanding of the performance of structures under different types of dynamic excitation. The system identification methods rely on the structure input (i.e., the foundation motion) and output (i.e., the motion of the structure) data which highly depend on the soil-structure interaction. Generally, this effect is ignored throughout the dynamic analysis of structure, and it is assumed that the structure is fully restrained at the base. This assumption sounds reasonable when a structure is founded on the bedrock, but soil-structure interaction can have a major impact on the seismic response of a structure founded on soft soils, which can significantly alter the vibration characteristics and, consequently, the characteristics of recorded motions. To accurately identify the soil-structure interaction, in addition to the records from the structures, the free-field record, not influenced by the structure, is also required. Due to the scarcity of such a record, the identification method needs to be employed for assessing the impact of soil-structure interaction on the dynamic response of the structure. Each identification method, parametric and nonparametric, in either the time domain or the frequency domain has its own advantages and disadvantages. Generally, parametric methods such as Ibrahim Time Domain, Eigen-system Realization Algorithm, Random Decrement Technique or Half Power-Bandwidth Method (Safak 1988 and 1991, Stewart 1996) are preferable for estimating modal damping but not for natural frequencies, mode shapes. On the other hand, nonparametric methods such as Peak-Picking, Frequency Domain Decomposition, Enhanced Frequency Domain Decomposition, Transfer Function or Fourier Transform (or Inverse Fourier Transform) (Ljung 1987, Pandit 1991, Fenves and DesRoches 1994) are proper methods for predicting the natural frequencies and mode shapes. In this study, to better estimate the dynamic parameters of the structure under seismic excitation, the various identification methods including Half-Power Bandwidth Method, Transfer Function, and Inverse Fourier Transform are adopted as presented in details below.

3.1 Transfer function

One of the most favorable identification methods in determination of soil-structural system characteristics under dynamic excitation is Transfer Function which can be calculated by the ratio of structure response to based-input motions. The outcome of the transfer function highly depends on the dynamic characteristics of the structure. The key dynamic parameters which influence transfer function are structure frequency and damping of different vibration modes of the system (Todorovska 2009). In this study, first, transfer function plots are compared for different soil-structure systems considering different site conditions with different soil shear wave velocities. After determining the system frequencies from transfer functions, then the determined frequencies are compared with ones obtained from the models using eigenvalue analysis. In order to assess the structural damping, the Half-Power Bandwidth Method is employed based on the system transfer function considering the structure roof response. This method is discussed in detail in the next section.

Table 4 Comparison of modal frequencies of different soil-structure interaction (SSI) models with fixed-based condition

Site Condition \ Frame		4-Story			8-Story			12-Story			16-Story		
		Fix (Hz)	SSI (Hz)	Diff. (%)	Fix (Hz)	SSI (Hz)	Diff. (%)	Fix (Hz)	SSI (Hz)	Diff. (%)	Fix (Hz)	SSI (Hz)	Diff. (%)
Vs = 50 m/s	Mode #1	1.65	1.60	2.90	0.85	0.83	2.08	0.67	0.65	3.00	0.49	0.47	3.27
	Mode #2	5.35	3.26	39.10	2.60	2.53	2.78	1.90	1.88	1.31	1.40	1.39	0.83
	Mode #3	9.90	4.95	50.00	4.65	3.29	29.28	3.37	3.08	8.62	2.45	2.42	1.45
	Mode #4	14.09	5.41	61.60	7.04	4.67	33.64	4.98	3.53	28.98	3.58	3.17	11.43
Vs = 150 m/s	Mode #1	1.65	1.64	0.30	0.85	0.85	0.25	0.67	0.67	0.33	0.49	0.49	0.39
	Mode #2	5.35	5.32	0.50	2.60	2.60	0.00	1.90	1.90	0.00	1.40	1.40	0.00
	Mode #3	9.90	9.43	4.70	4.65	4.65	0.00	3.37	3.36	0.34	2.45	2.45	0.00
	Mode #4	14.00	10.31	26.80	7.04	6.99	0.70	4.98	4.98	0.00	3.58	3.57	0.36
Vs = 350 m/s	Mode #1	1.65	1.64	0.20	0.85	0.85	0.09	0.67	0.67	0.07	0.49	0.49	0.05
	Mode #2	5.35	5.35	0.00	2.60	2.60	0.00	1.90	1.90	0.00	1.40	1.40	0.00
	Mode #3	9.90	9.90	0.00	4.65	4.65	0.00	3.37	3.37	0.00	2.45	2.45	0.00
	Mode #4	14.09	14.09	0.00	7.04	7.04	0.00	4.98	4.98	0.00	3.58	3.58	0.00
Vs = 700 m/s	Mode #1	1.65	1.65	0.00	0.85	0.85	0.09	0.67	0.67	0.00	0.49	0.49	0.00
	Mode #2	5.35	5.35	0.00	2.60	2.60	0.00	1.90	1.90	0.00	1.40	1.40	0.00
	Mode #3	9.90	9.90	0.00	4.65	4.65	0.00	3.37	3.37	0.00	2.45	2.45	0.00
	Mode #4	14.09	14.09	0.00	7.04	7.04	0.00	4.98	4.98	0.00	3.58	3.58	0.00

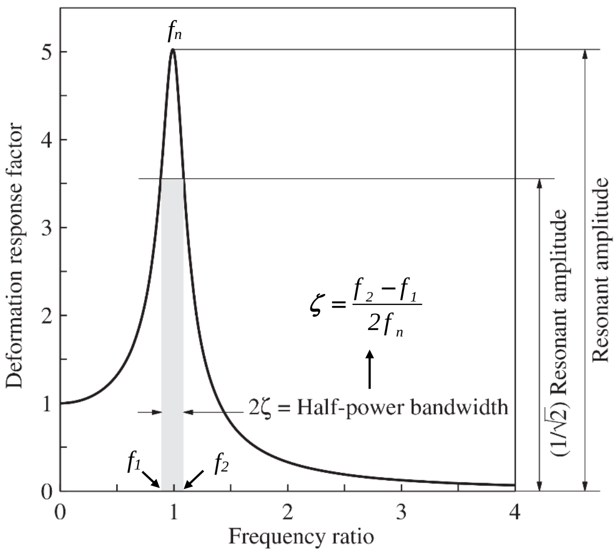

Fig. 5 System damping ratio calculation through the half-power bandwidth method (Chopra 2007)

3.2 Half-power bandwidth method

The half-power bandwidth method is commonly used for estimating damping in multi-degree freedom of freedom (MDOF) systems, although it was originally derived from the frequency response of an SDOF system. In this study, after constructing the TF of acceleration frequency response for different soil-structure systems, the primary modes of systems are identified and extracted. Then, in order to evaluate the damping of systems, the half-power method is employed to calculate damping ratio from the first four modes of the systems to compare with the 5% damping ratio which is added to the system through Rayleigh method. This way of assessment helps to identify the accuracy of the method to predict the system damping and also shows which one of the structure modes is more reliable for determining the characteristics of a structure. The process of calculating the damping ratio using the half-power method is illustrated in Fig. 5.

3.3 Inverse fourier transform

The mode shapes are one of the main dynamic characteristics of a structure which are mainly used in dynamic design analysis. Generally, mode shapes are computed through eigenvalue analysis by most of the structural analysis software packages based on structural mass and stiffness matrices. Most of these software packages are not able to account for the soil effect in the structural modeling; or even if so, the computed stiffness matrix of the system is not quite accurate. This problem mostly occurs when the stiffness matrix of the system is not updated at the end of each analysis iteration due to incorporation with the not updated soil stiffness matrix. Thus, modes shapes are not accurate and do not represent the actual soil-structural system. In order to prevent this issue and determine the accurate mode shapes of a soil-structure system, the inverse Fourier Transform, as one of the prevalent method in system identification, is employed in this study. By applying the inverse Fourier Transform, the mode shapes of the structure can be easily found using only the soil-structure system response under any dynamic excitation instead of using eigenvalue analysis (Richardson and McHargue 1993).

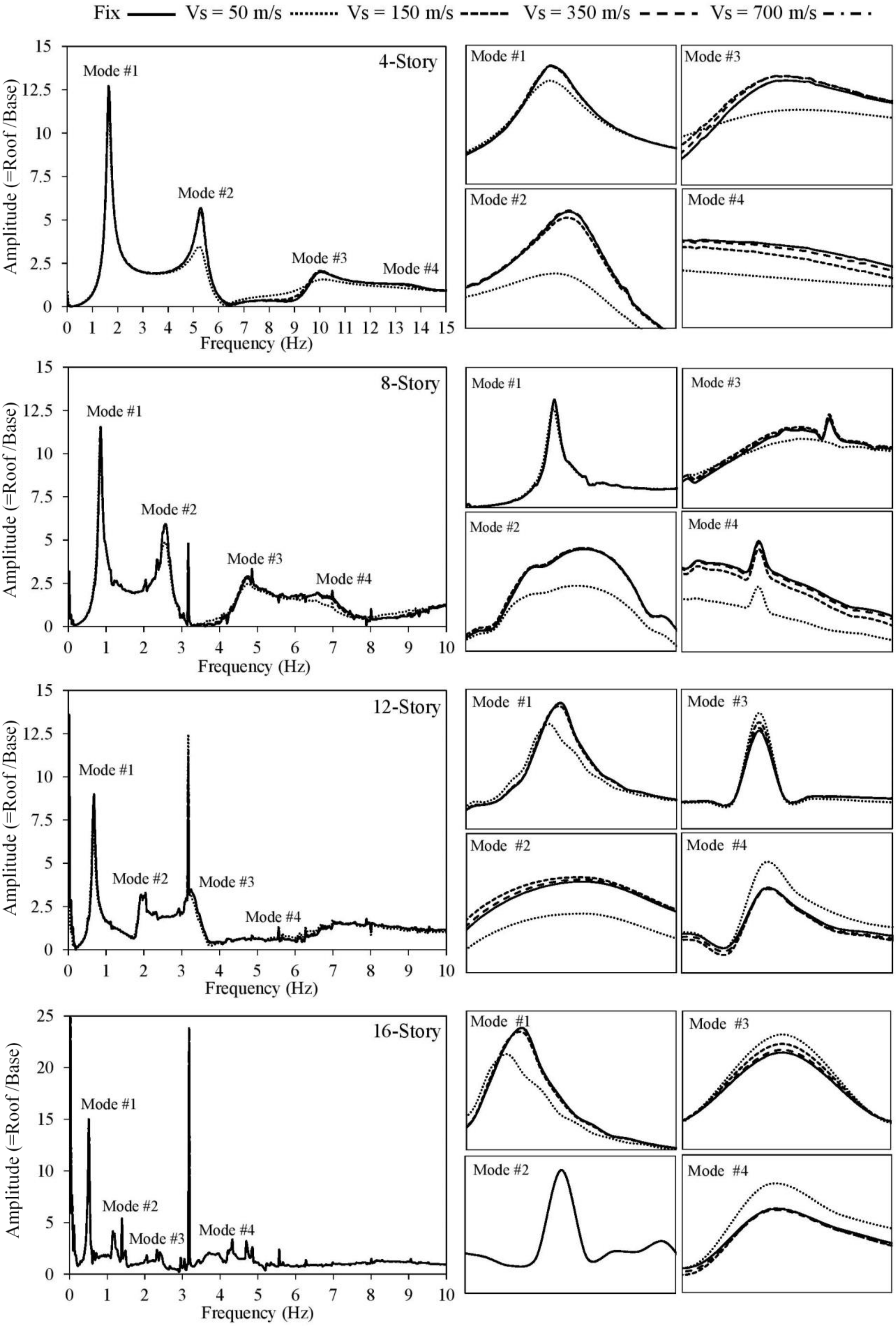

Fig. 6 Transfer functions of designed soil-structure systems for four first modes

Table 5 Soil-structure system damping derived from different modes based on half-power bandwidth

Site Condition / Frame	*Vs* = 50 m/s				*Vs* = 150 m/s				*Vs* = 350 m/s				*Vs* = 700 m/s			
	Mode #1	Mode #2	Mode #3	Mode #4	Mode #1	Mode #2	Mode #3	Mode #4	Mode #1	Mode #2	Mode #3	Mode #4	Mode #1	Mode #2	Mode #3	Mode #4
4-Story	3.78	0.57	0.56	0.39	4.21	0.33	0.41	0.28	4.26	0.33	0.41	0.28	4.26	0.33	0.41	0.28
8-Story	2.33	1.75	1.10	0.56	2.61	2.14	1.10	0.28	2.61	2.14	1.10	0.28	2.61	2.14	1.10	0.28
12-Story	3.08	0.92	0.90	0.66	2.29	1.05	0.75	0.66	2.29	1.05	0.75	0.66	2.29	1.05	0.75	0.66
16-Story	3.78	1.05	1.37	0.82	4.21	0.87	1.67	0.67	4.26	0.87	1.67	0.67	4.26	0.87	1.67	0.67

4. Results and discussions

4.1 Transfer function

After modeling the soil-structure systems, nonlinear dynamic analyses are conducted for all site conditions under Chi-Chi (1999) earthquake excitation. In the first step of the identification of dynamic structural characteristics and understand the structural performance, the transfer function of the structures are derived based on the acceleration response of roof story (highest story of each structure) and the base input motions which are recorded at the base support (Fig. 6). It should be noted that in order to investigate the impact of site conditions on the conventional structural design, another analysis also is performed for the same structures with fixed-based support condition. Worth mentioning that in the nonlinear dynamic analysis, the structure stiffness can vary instantaneously, thus the analysis outputs are different from the initial conditions. To this end, in order to have a better assessment, the recoded outputs in this study are captured at the end of each analysis which also considers the effect of plastic hinges occurred in the structural elements including beams and columns. As shown in the figure, the amplitude of the first mode depicts a slight difference for all cases in which the softer soil (Vs=50 m/s) has lower amplitude compared to the other site conditions. These trends in the transfer functions amplitude are more significant for higher modes for all site conditions. Also, it can be observed that the nonlinearity of transfer functions is increased by the increase of the height of the structure. This observation can be attributed to the increase in the number of the plastic hinges in the structural elements (beams and columns) for taller buildings as will be discussed in details in “Plastic Hinge Formation” section. It should be noted that the increase of number of structural elements also could increase the chance of introducing the frequency noise into the frequency content of structural response in the frequency-domain analysis. These noises can induce errors in the identification of the system response which needs to be addressed by filtering the frequency response to cancel out the noise.

4.2 Modal frequency

Regarding modal frequency, it is expected that inclusion of the underlying soil in structural modeling and analysis increase the flexibility of the system compared to the fixed-based system, and subsequently affecting the frequency response of the system. As shown in transfer function (Fig. 6), the modal frequencies of the soil-structure system are affected for all structure heights specifically for the softer soils (V_s =50 and 150 m/s) in comparison with the fix-based condition.

This difference is more meaningful for higher vibration modes of the system. The first mode, which significantly dominates the response of the structure, exhibits less than 2% variation. As a result, it can be concluded that the soil-structure interaction has greater impacts on higher modes of the vibration which can be more crucial for structural systems with significant participation of higher modes in their seismic performance such as tall buildings. However, the results show that the modal frequency is not changing for stiffer soils (V_s =350 and 700 m/s) in higher modes of the soil-structure systems in particular for high-rise structure archetypes (12- and 16-story). The modal frequency of the first four modes of the system for all structure heights and site conditions are summarized and compared with the fix-based condition in Table 4.

4.3 System damping

As discussed in the previous section, the damping is one of the main dynamic characteristic of the system for the seismic design for various types of structures. Thus, determining the actual structural damping with respect to the underlying soil has a significant role in the real-time assessment of the performance of the structure under future seismic events. To identify the damping of the soil-structure system, the half-power bandwidth method is employed and applied to the vibration modes which are derived from the transfer function of the systems. Among all considered modes, the damping derived for the first mode of the vibration is the closest to the 5% as the initial damping ratio assumption of the analysis (see Table 5). It is worth mentioning that, as it is expected, since the softer soil increases the flexibility of the system, the soil-structure system laid on the soft soil (V_s =50 m/s) shows lower damping ratio compared to the rest of site conditions. With respect to these findings, it can be concluded that the first mode of the structure is more reliable for determining the system damping ratio in the case of blind system identification which purely entails the output-only data. Also, it seems that inclusion of the soil impact into the analysis results in a decrease of the system stiffness which accordingly leads to the decline of developed internal forces in the structural components due to the ground shaking and subsequently relatively fewer damage within structural members.

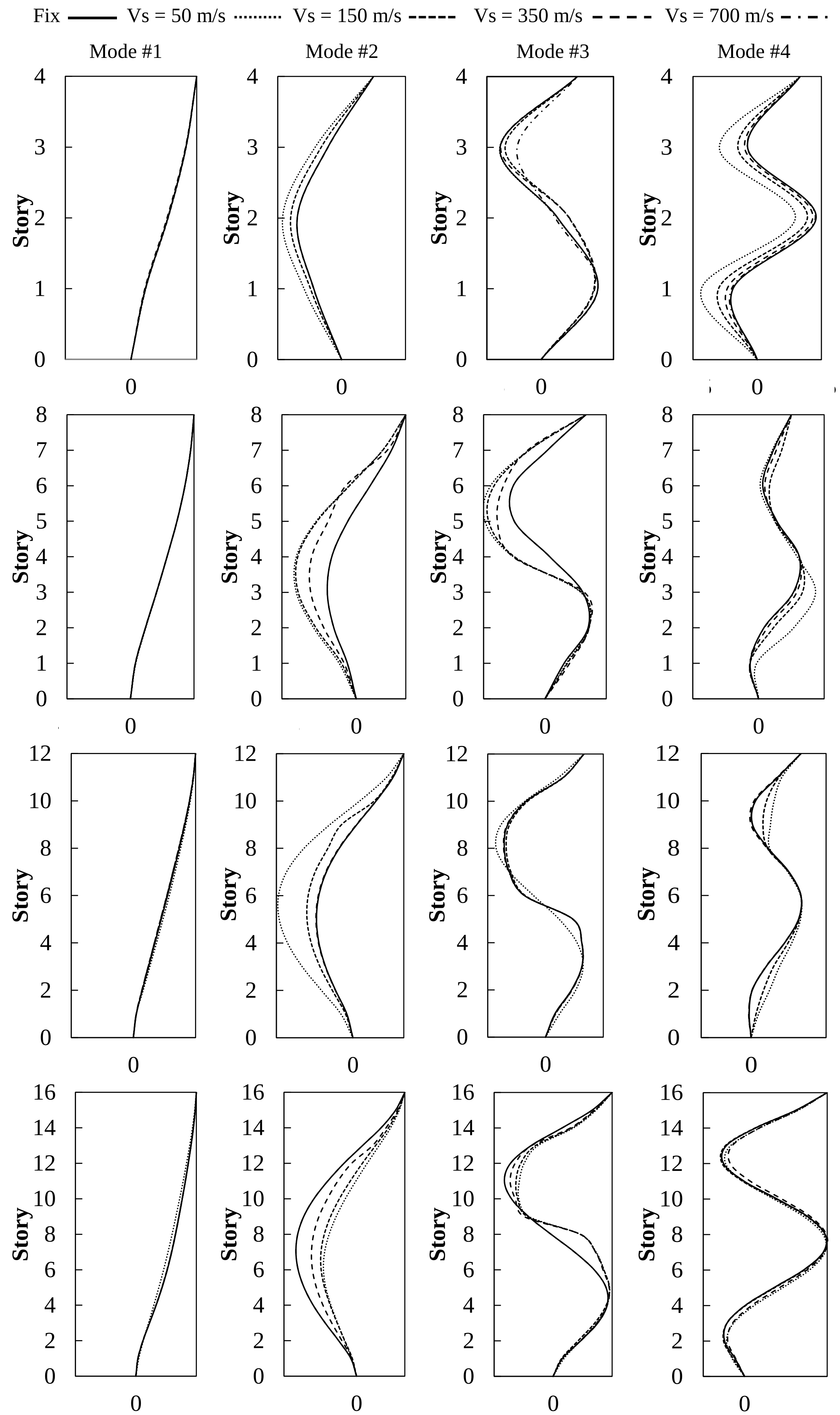

Fig. 7 The modal shape of soil-structure systems considering various site conditions for different structure height

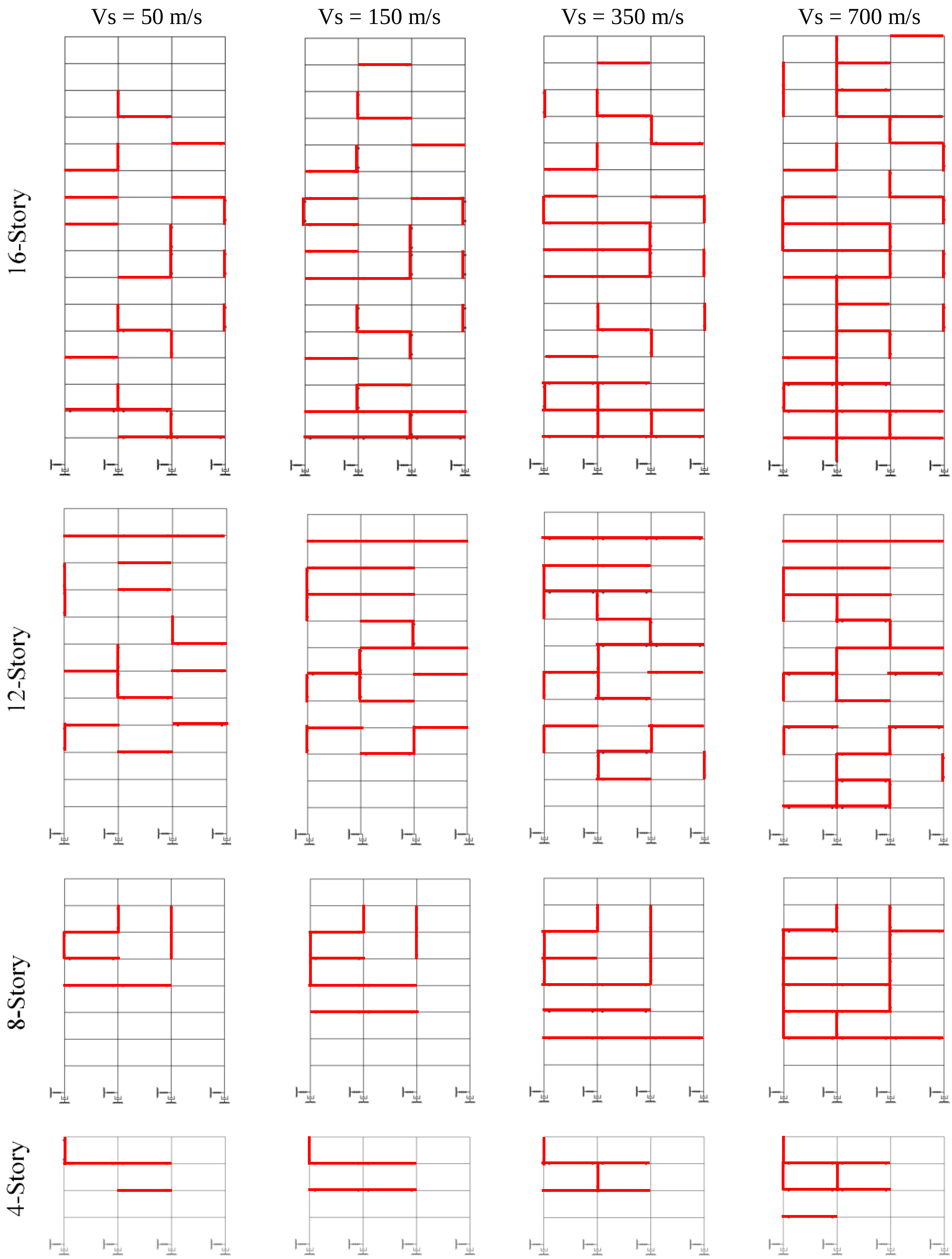

Fig. 8 Plastic hinge formation pattern of different soil-structure systems considering various site conditions under Chi-Chi (1999) earthquake excitation (Members with plastic hinge are marked with solid red color)

4.4 Mode shape

Mode shapes of the soil-structure system are derived based on the results obtained from the Inverse Fourier Transform analysis applied to the acceleration frequency response of roof story for each structure model. First, four mode shapes of the soil-structure systems obtained from the nonlinear dynamic analysis are presented in Fig. 7. As shown in the figure, mode shapes exhibit small changes in the first mode and significant changes in higher modes. It can be seen from the result that applying site condition impacts the mode shapes of the structure, especially in higher modes compared to the structure with a fixed-base support condition. This finding is in a good agreement with the result of modal frequency which is discussed previously in Table 5. Also, it appears that the softer soils with a shear velocity of 50 and 150 m/s show more difference among other site conditions in comparison with the fixed-based condition. However, this difference decreases while the height of the structure is increased.

4.5 Plastic Hinge Formation Pattern

One of the most important purposes of system identification is the prediction of failure mode as well as damages within a structural system. Formation of plastic hinges in structural elements such as beams and columns not only influence the dynamic structural parameters such as modal frequency and damping ratio but also change the system stiffness which leads to the impairment of the structure performance. Thus, determining the plastic hinges in structural elements under dynamic excitation is required for damage detection in any types of structure for investigating the performance of structures in future seismic events. To this end, in this study, the plastic hinges are determined by evaluating the stress-strain behavior of each structural element with regard to the adopted material behavior for all soil-structure systems to identify which one of the elements have been entered in the material plastic zone. As shown in Fig. 8, the structural elements in which the plastic hinges are formed, are identified for each soil-structure systems. It can be seen from the figure that the number of elements with plastic hinge increases as the structure height raises. This finding sounds reasonable since the structural elements and the applied lateral load due to seismic load are increased with increase of the structure height. Also, it is found that the soil-structure system with softer soil develops less number of the plastic hinge while it is supposed to observe more damages for such systems in comparison to the other site conditions. The predicted behavior shows a significant improvement in the performance of the structure located on the soft soil.

This observation can be attributed to the direct displacement-based method used for designing of the structure by which a target story drift is set to limit building drifts to prevent the large displacements in the structure and have the applied load among the structural components properly distributed. Worth mentioning that the soft story condition does not occur in any of the structure models investigated in this study over all site conditions under a strong ground motion such as Chi-Chi (1999). This finding proves that the strong column-weak beam theory which is embodied in the direct displacement method is well-satisfied in the overall behavior of designed structures. Following that, as it was expected the plastic hinges are mainly and pervasively formed in the beams rather than columns which have less influence on the overall performance of the structure. It should be noted that the plastic hinge formation pattern of fixed-based condition was the same as the site condition with V_s of 700 m/s.

5. Summary and conclusions

Identification of dynamic characteristics of a structure is a beneficial tool to monitor and assess the performance of a structural system under upcoming seismic events. By finding the updated dynamic characteristics of a structure, then it can be reanalyzed to detect the damage in the structure. In order to have a better estimate of these characteristics, the studied structure needs to be modeled precisely considering not only the structure but also the soil-structure interaction. Thus, in this study, different identification techniques were employed to determine the dynamic parameters of a soil-structure system considering different site conditions through changing the soil shear wave velocity. Chi-Chi earthquake (1999), which is a near-field earthquake with shallow epicenter, is selected as an input excitation in this study. The site conditions are selected based on the ASCE 7-10 to account a range of soft to stiff soils. The structure model used in this study was designed for different heights (4, 8, 12, and 16-story) based on the direct displacement method to control the maximum story drift. Also, the applied design method prevents the damage caused by a soft story in the structure by applying the strong column-weak beam theory.

Based on the results, after deriving the transfer functions for all soil-structure systems, the modal frequencies of the systems are changed for all structure heights specifically for the softer soils (V_s =50 and 150 m/s), around 2%, in comparison with the fix-based condition. This change is greater for higher modes of the systems in comparison with the first mode, which significantly dominates the structure response. To identify the damping of the soil-structure system, the half-power bandwidth method is employed and applied to the modes which are derived from the transfer function of the systems. Out of all modes, the damping calculated from the first mode is the closest to the initial damping ratio which was set for the model at the beginning of the analysis. By using the Inverse Fourier Transform, as another system identification technique, on the soil-structures response, the mode shapes of the structure were determined for the first four modes. The mode shapes represent small changes in the first mode and significant changes in higher modes. Also, it was found that the site condition impacts the structure mode shapes, especially in higher modes compared to the structure with the fixed-base support condition. Finally, the plastic hinge formation pattern of studied soil-structure systems was identified by evaluating the stress-strain behavior of each structural element with regard to the adopted material behavior. Results show that the number of elements with plastic hinge increases as the structure height raises. Also, it is found that the soil-structure system with softer soil shows the formation of less number of plastic hinges while it was supposed to cause more damage in comparison to the other site conditions. This happens due to the employment of the direct displacement method that was used for designing the structure in which the story drift is limited to an allowable drift to prevent occurring a large displacement and distribute properly the applied load among the structural components. In addition, the soft story condition does not occur in any of the structural models investigated in this study for all site conditions. This finding proves that the weak-beam strong-column theory which is embodied in the direct displacement method is well-satisfied in the dynamic behavior of all designed structures.

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