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Sensitivity analysis for seismic response of reinforced concrete staggered wall structures

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This study investigates the sensitivity of design variables such as ultimate strength of concrete, yield stress of reinforcing steel and damping ratio to the seismic response of staggered wall structures. Two different methods for sensitivity analysis, the tornado diagram method and the first-order second-moment method, are applied to two types of model structures. Non-linear dynamic analyses are carried out using the seven maximum considered earthquake level earthquake records, which shows that the inter-storey drift is the most sensitive to the variation of modal damping ratio. It is observed in the incremental dynamic analyses that, when the earthquake intensity is relatively small, the yield stress of rebars and the concrete strength in the link beams are also important factors affecting the sensitivity of seismic response. As the intensity of seismic load increases, the strength of columns becomes another important factor.

Notation

D_U	strain at ultimate stress
D_L	streain at the beginning of stress reduction
D_X	ultimate strain at failure
E	Young's modulus
F'_c	strength of concrete
F_R	residual stress
F_u	ultimate stress
F_y	yield stress
$\boldsymbol{g}$	acceleration
g	function of a random variable
g_0	value of g at $X = x_0$
K_h	post-yield stiffness
K_0	initial stiffness
S_{DS}	spectral acceleration at short period
S_{D1}	spectral acceleration at one second period
$\boldsymbol{VC}[X]$	covariance vector
X	independent random variable
x_n	nth component of random variable X
Y	subordination variable
μ_X	mean of variable X
σ_X	standard deviation of variable X
σ_Y	standard deviation of variable Y

Introduction

In structural engineering sensitivity analysis is generally carried out for comparison of the relative importance of design variables such as structural properties and loading conditions, which helps to determine which uncertainties have the most potential impact on the performance of the structure. Sensitivity analysis is useful when a structural engineer is to determine the effect of a particular design variable if it differs from the value assumed in the design stage. Based on statistical data of the variable, the engineer can determine how changes in that variable will impact the structural response. Porter *et al.* (2002) applied the tornado diagram analysis (TDA) method to seismic sensitivity analysis for building loss estimation. Baker and Cornell (2003) applied the first-order second-moment (FOSM) method in sensitivity analysis for loss estimation of structures. Lee and Mosalam (2005) investigated the seismic demand sensitivity of reinforced concrete shear-wall buildings using the FOSM method. They also applied various sensitivity analysis methods to estimate sensitive design parameters to the seismic response of buildings (Lee and Mosalam, 2006). Kim *et al.* (2011) investigated the sensitivity of design variables of steel moment frames subjected to sudden column loss. They found that the FOSM method can be used for sensitivity analysis of structures with almost the same level of accuracy as the Monte Carlo simulation analysis.

Staggered wall structures are structural systems for reinforced concrete residential buildings in which storey-high deep beams extend across the entire width of the buildings. The floor system spans from the top of one staggered wall to the bottom of the adjacent wall serving as a diaphragm, and the staggered walls with attached slabs resist the gravity as well as the lateral loads as H-shaped storey-high deep beams. By staggering the locations of the walls on alternate floors, large, clear areas are created on each floor. A similar system, the staggered truss system, has been applied in steel structures. The system was first proposed by Fintel (1968), who conducted experiments of a half-scale staggered wall structure subjected to gravity load. He also carried out a comparative study of three different structure systems for residential buildings to investigate the cost-effectiveness of the staggered wall systems. Mee *et al.* (1975) carried out shaking table tests of 1/15 scaled models for staggered wall systems. Kim and Jun (2011) evaluated the seismic performance of partially staggered wall apartment buildings using non-linear static and dynamic analysis, and compared the results with those of conventional shear wall system apartment buildings. They found that the structure with a partially staggered wall system satisfied the collapse prevention performance objective required by FEMA-356 (FEMA, 2009) and thus is considered to have enough capacity for resisting design level seismic load. Recently Lee and Kim (2013) investigated the seismic performance of six- and 12-storey staggered wall structures with a middle corridor based on the FEMA P695 procedure. They found that the analysis model structures had enough safety margin for collapse against design level earthquakes.

The objective of this study is to identify important design parameters and structural members for reinforced concrete staggered wall buildings through sensitivity analysis. To this end two different approaches for sensitivity analysis, the TDA and the FOSM methods, are applied on six-storey staggered wall buildings without and with a middle corridor. Through the sensitivity analysis the uncertainties associated with the material properties and the member capacities are considered in order to determine the influential material properties and members for the seismic performance of the analysis model structures.

Sensitivity analysis

For sensitivity analysis of model structures, the variability of structural response due to the variability of structural properties is evaluated using the TDA and the FOSM methods. TDA is one of the sensitivity analysis tools commonly used in decision analysis. In TDA, the upper and lower bounds of a random variable are selected and the corresponding structural responses are obtained. The difference between such structural responses, referred to as swing, is presented in a bar chart and is considered as a measure of sensitivity. In this paper, tornado diagrams are developed based on the interval of mean $\pm$ 2 standard deviations of the design variables.

In the FOSM method, means and standard deviations of random variables are assumed and the mean and standard deviations of structural responses are obtained. A detailed analysis procedure of the FOSM method can be found in Lee and Mosalam (2005), the key procedure of which is described as follows. If a random variable $X = (x_1, x_2, \ldots, x_n)^{\mathrm{T}}$ has mean and co-variance vectors $\mu_X = (\mu_1, \mu_2, \ldots, \mu_n)^{\mathrm{T}}$ and $\boldsymbol{VC}[X]$, respectively, the first-order approximation of a function $Y = g(X)$ using the Taylor series expansion evaluated at x_0 can be given as

1. $$Y \approx g_0 + \left(\frac{\mathrm{d}g}{\mathrm{d}x}\right)_0 (X - x_0)$$

where $()_0$ denotes a function evaluated at x_0. In the formulation the random variable X can be considered as the structural design parameters and the function $Y = g(X)$ represents the structural analysis and the corresponding responses. For $x_0 = \mu_X$ the mean μ_Y and the standard deviation σ_Y of $Y = g(X)$ can be approximated using the FOSM method as

2. $$\mu_Y \approx g(\mu_X)$$

3. $$\sigma_Y^2 \approx \left(\frac{\mathrm{d}g}{\mathrm{d}x}\right)_0^2 \sigma_X^2$$

In this study the independent variable X corresponds to a design variable of the structures, and the dependent variable Y corresponds to a structural response obtained through seismic analysis. The gradient of g in Equation 3 is evaluated using the finite-difference approach with the perturbation size of twice the standard deviation of a design variable.

Design and analysis modelling of example structures

Structural design of analysis models

In this study six-storey staggered wall structures without and with a middle corridor are designed for sensitivity analysis. In the simplest form of a staggered wall structure columns and beams are located along the longitudinal perimeter of the structures, providing a full width of column-free area within the structure. Along the longitudinal direction, the column–beam combination resists lateral loads as a moment-resisting frame. Figure 1 shows the overall structural configuration of the model structure with middle corridor, and Figure 2 and Figure 3 depict the structural

Figure 1. Staggered wall system structure with middle corridor (case 2)

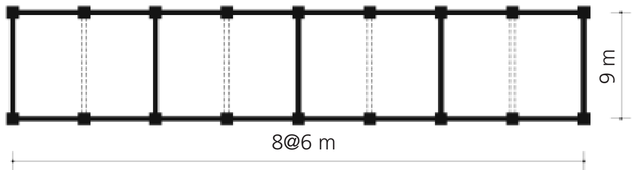
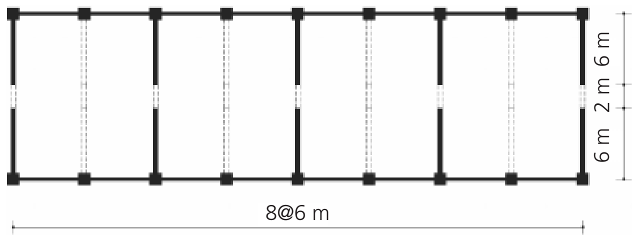
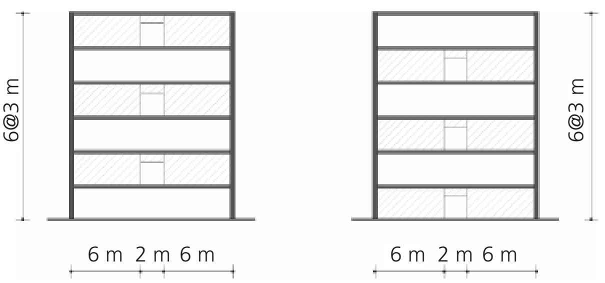


Figure 2. Floor plan of case 1 analysis model without middle corridor



(a)



(b)

Figure 3. Floor plan and elevation of case 2 analysis model with middle corridor: (a) floor plan; (b) elevation

plan and elevation view of the structures without (case 1) and with middle corridor (case 2), respectively. In the structural plan the transverse solid lines represent the locations of staggered walls, and the dotted lines represent the staggered walls in the storey above and/or below.

The model structures are designed as per the ACI 318-05 (ACI, 2005) using the seismic loads specified in the IBC 2009 (ICC, 2009). For gravity loads, the dead and live loads of 7 kN/m^2 and 2 kN/m^2 are used, respectively. The design seismic load is computed based on the design spectral response acceleration parameters $S_{\mathrm{DS}} = 0{\cdot}37\boldsymbol{g}$ and $S_{\mathrm{D1}} = 0{\cdot}15\boldsymbol{g}$ with the short and long period site coefficients of 1·0. This loading condition is equivalent to the design seismic load in the Los Angeles (LA) area with the site class B, which is a rock site. As the response modification factor for a staggered wall system is not specified in the current design codes, the response modification factor of 3·0 is used to compute the design base shear of staggered wall systems; this is generally used for structures to be designed without consideration of seismic detailing. Along the longitudinal direction the structures are designed as ordinary moment-resisting frames. The ultimate strength of concrete is 27 MPa and the tensile strength of rebars is 400 MPa. The thickness of the staggered walls is 20 cm throughout the stories, and the link beams have the dimensions of 600 × 200 mm. D13 rebars (deformed rebars with $d = 13$ mm and $A = 1{\cdot}267$ cm^2) are placed in the staggered walls at the interval of 200 mm in both horizontal and vertical directions. The thickness of the floor slabs is 210 mm, which is the minimum thickness required for shear wall apartment buildings in Korea to prevent transmission of excessive noise and vibration through the floors. The sizes of corner columns in the structure without middle corridor vary from 400 × 400 mm in the first storey to 300 × 300 mm in the top storey. The sizes of other exterior columns are 600 × 600 mm in the first storey, reducing to 500 × 500 mm in the top storey. The column sizes in the structure with middle corridor vary from 500 × 500 mm to 400 × 400 mm in the corner and from 600 × 600mm to 500 × 500 mm in the exterior columns. Tables 1 and 2 show the size of columns and reinforcing bars used to design model structures. The fundamental natural periods of the two model structures are presented in Table 3. It can be observed that the natural periods along the transverse direction, where the staggered walls are placed, are significantly smaller than those along the longitudinal direction in both structures. It also can be noticed that the natural period increases significantly when the middle corridor is inserted along the longitudinal direction and the staggered walls are separated by connection beams.

Corner columns	Size: mm	Rebar	Exterior columns	Size: mm	Rebar
C1_1F	400 × 400	6-D19	C2_1F	600 × 600	6-D29
C1_2F	380 × 380	6-D19	C2_2F	580 × 580	6-D29
C1_3F	360 × 360	6-D19	C2_3F	560 × 560	6-D29
C1_4F	340 × 340	6-D19	C2_4F	540 × 540	6-D25
C1_5F	320 × 320	6-D16	C2_5F	520 × 520	6-D25
C1_6F	300 × 300	6-D16	C2_6F	500 × 500	6-D25

Table 1. Sectional properties of columns in the structure without middle corridor (case 1)

Corner columns	Size: mm	Rebar	Exterior columns	Size: mm	Rebar
C1_1F	500 × 500	6-D25	C2_1F	600 × 600	6-D32
C1_2F	480 × 480	6-D22	C2_2F	580 × 580	6-D29
C1_3F	460 × 460	6-D22	C2_3F	560 × 560	6-D29
C1_4F	440 × 440	6-D22	C2_4F	540 × 540	6-D25
C1_5F	420 × 420	6-D22	C2_5F	520 × 520	6-D25
C1_6F	400 × 400	6-D19	C2_6F	500 × 500	6-D25

Table 2. Sectional properties of columns in the structure with middle corridor (case 2)

Direction	Case 1	Case 2
Longitudinal	0·87	1·02
Transverse	0·16	0·28

Table 3. Fundamental periods of the model structures (s)

Modelling for analysis

Non-linear analyses of the model structures are carried out using the program code Perform 3D (2006). Figure 4 shows the stress–strain relationships of concrete and reinforcing bars based on Paulay and Priestley (1992) without confinement effect. The ascending branch of concrete is modelled with trilinear lines, and the yield stress and the residual stress are taken to be 60% and 20% of the ultimate strength, respectively. In the model the ultimate strength and the yield strength of concrete are 27 MPa and 16MPa, respectively, and the residual strength is 5·5 MPa. The strain at the ultimate strength is 0·002, and the ultimate strain is defined as 0·004. The reinforcing steel is modelled with bilinear lines. Overstrength factors of 1·5 and 1·25 are used for concrete and reinforcing steel, respectively, in the non-linear static and dynamic analyses.

In Perform 3D, shear wall elements have shear and axial-bending properties. The shear property is generally assumed to be elastic, and the axial-bending property is modelled by fibre cross-sections to represent inelastic behaviour. In this study the staggered walls were modelled by the shear wall fibre elements. Each shear wall element was modelled using eight fibres with 0·3175% reinforcement in each fibre. In the model the yield and the ultimate strength of concrete are 27 MPa and 18 MPa, respectively, and the residual strength is defined as 20% of the ultimate strength. The strain at the ultimate strength is 0·002, and the ultimate strain is defined as 0·004. The reinforcing steel is modelled with bi-linear lines, and the overstrength factors of 1·5 and 1·25 are used for concrete and reinforcing steel, respectively. The hysteresis loops of concrete and rebar in each fibre element are shown in Figure 5. As the shear wall element has no in-plane rotational stiffness at its nodes, a beam element is embedded in the wall to specify a moment-resisting connection between a beam and a wall. The beams and columns are modelled by the concrete type

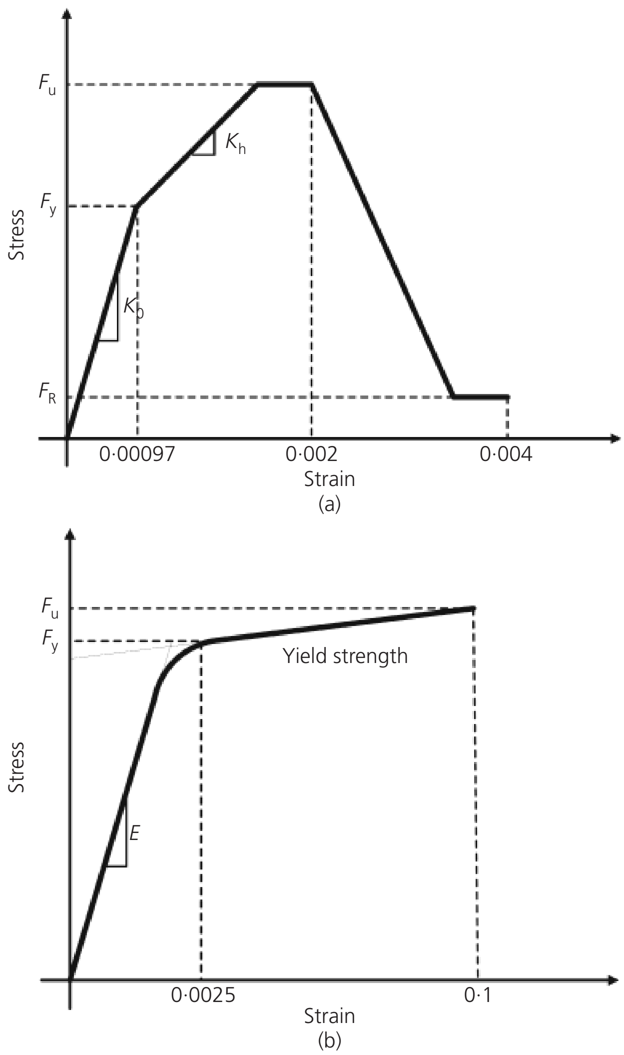


Figure 4. Stress–strain relationships of structural materials: (a) concrete; (b) rebar

FEMA beam and FEMA column elements, respectively, provided in Perform 3D. The moment–rotation hysteresis loops of beams and columns are depicted in Figure 6. The analysis model for link beams located between two staggered walls is composed of two

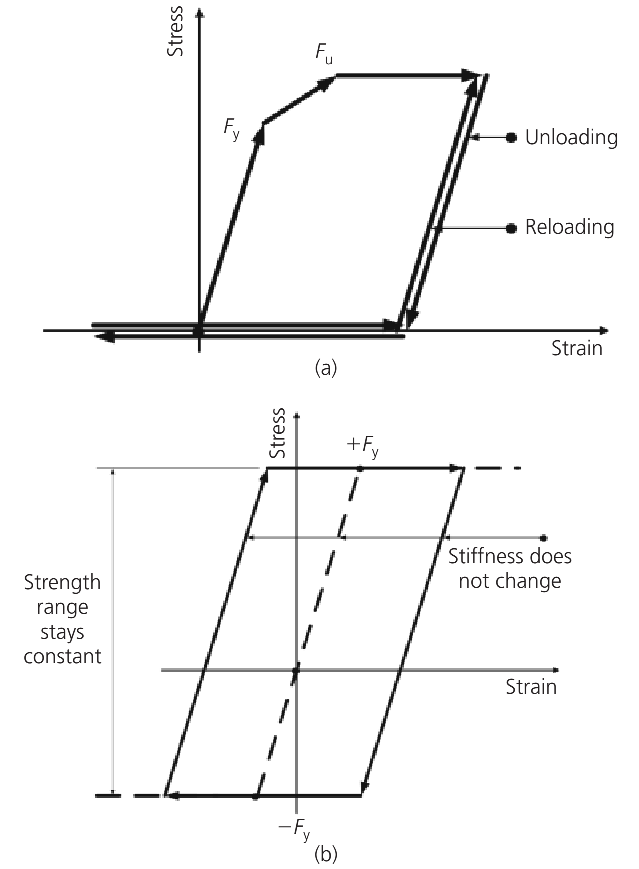


Figure 5. Hysteresis loop of staggered wall fibre elements: (a) concrete; (b) rebar

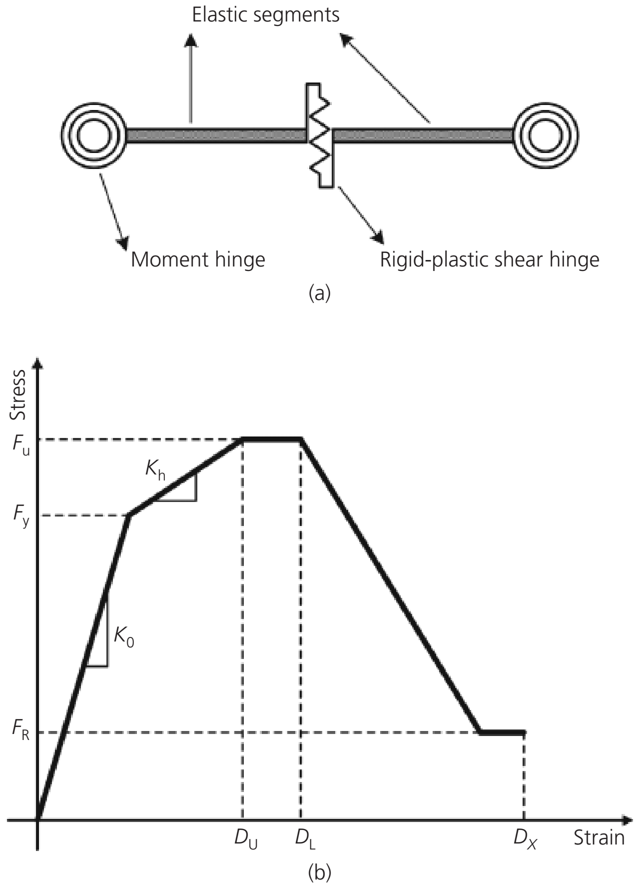


Figure 7. Analysis modelling of link beams: (a) modelling; (b) moment–rotation relationships

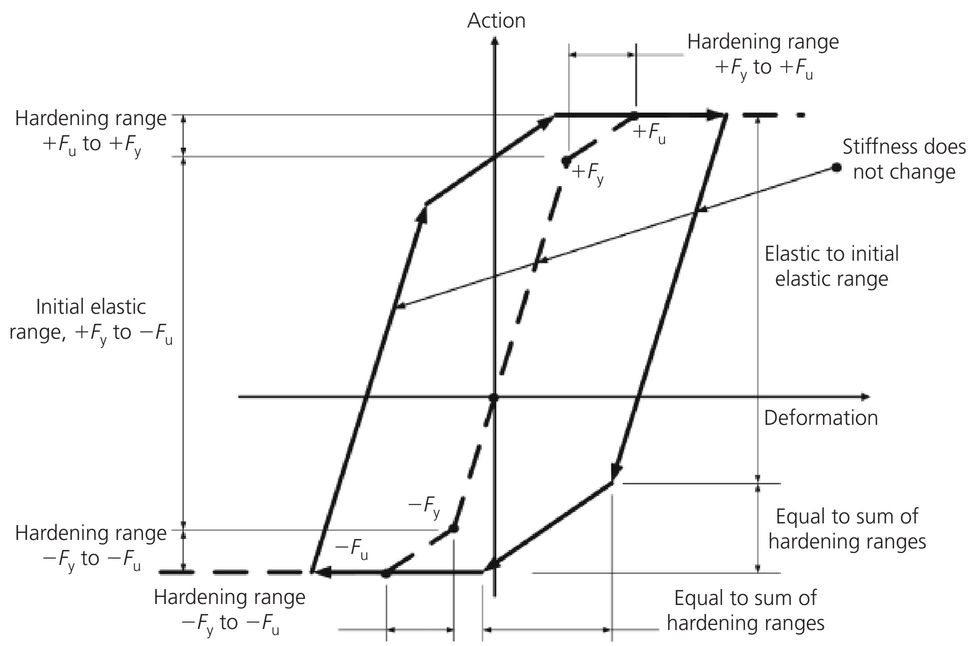


Figure 6. Hysteresis loop of columns and beams

end rotation type moment hinges and a middle shear hinge as shown in Figure 7(a). The behaviour of the moment hinge shown in Figure 7(b) is defined based on the ASCE/SEI 41-06 (ASCE, 2007), and the shear hinge is defined based on Englekirk (2003). As the shear wall element has no in-plane rotational stiffness at its nodes, a beam element is embedded in the wall to specify a moment-resisting connection between the beam and a wall.

Seismic performance of model structures

Before conducting sensitivity analysis, non-linear static pushover analyses are carried out first to investigate non-linear behaviour and collapse mode of model structures which are designed using the mean values of design variables. The lateral load for pushover analysis is applied proportionally to the fundamental mode shape of the model structures, and the analyses are carried out until the roof displacement reaches 2% of the building height. The base shear–roof displacement relationships of model structures obtained from the pushover analysis are shown in Figure 8, and the plastic hinge formation at the two points of sudden

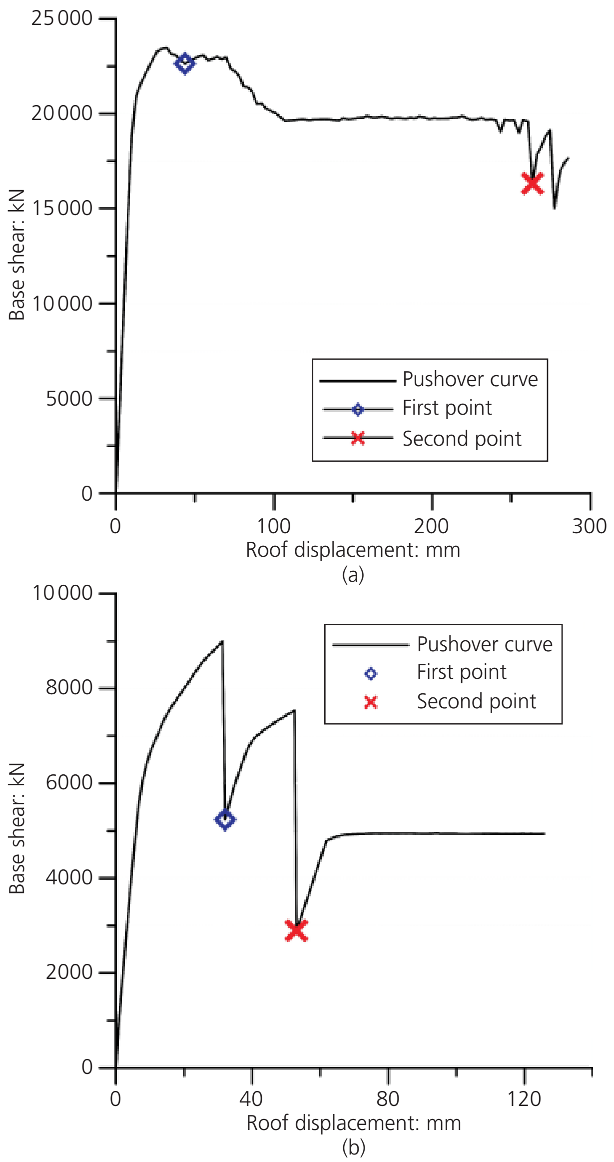


Figure 8. Pushover curves of analysis model structures: (a) case 1; (b) case 2

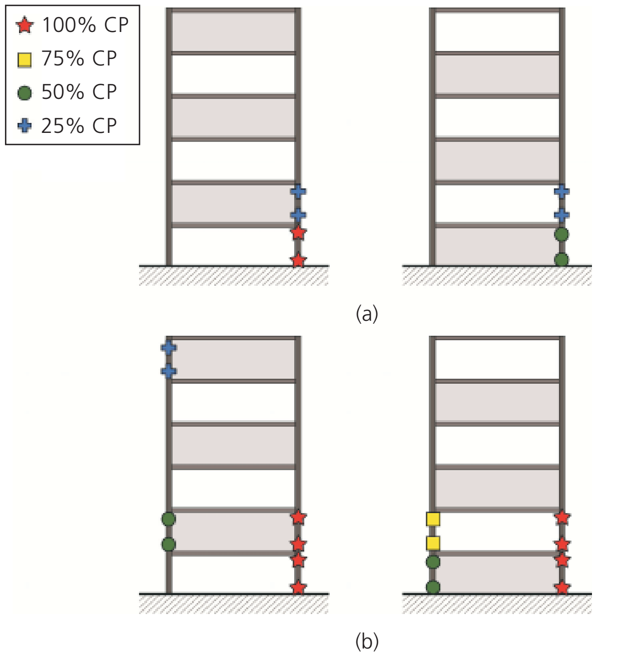


Figure 9. Plastic hinge formation in the case 1 model structure at the two strength drop points marked on the pushover curve: (a) at first point; (b) at second point

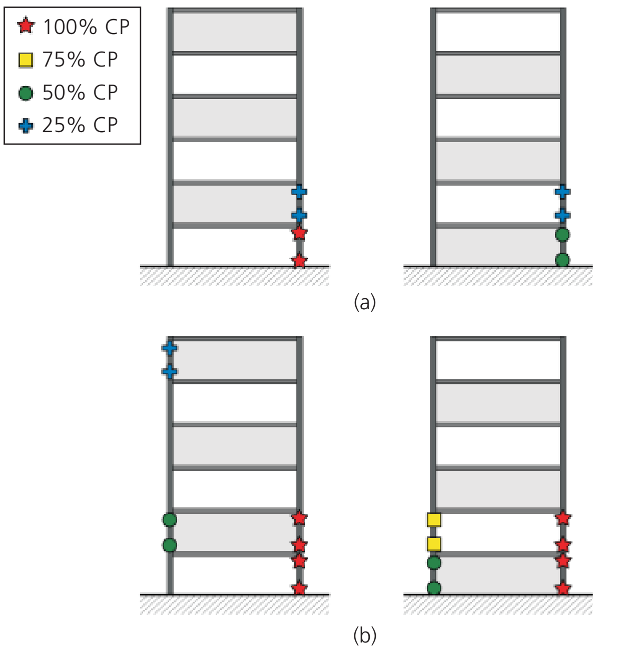


Figure 10. Plastic hinge formation in the case 2 model structure at the two strength drop points marked on the pushover curve: (a) at first point; (b) at second point

Variables	Nominal values	Mean	Standard deviation	COV: %
F'_c_Link beam	27·6 MPa	34·0 MPa	4·93	14·5
F_y_Link beam	420 MPa	483 MPa	24·15	5·0
F'_c_Wall	27·6 MPa	34·0 MPa	4·93	14·5
F_y_Wall	420 MPa	483 MPa	24·15	5·0
F'_c_Column	27·6 MPa	34·0 MPa	4·93	14·5
F_y_Column	420 MPa	483 MPa	24·15	5·0
Damping ratio	—	5·0%	2·0%	40·0

Table 4. Statistical parameters of variables used in the sensitivity analysis

Record number	Ground motion record	Duration: s	PGA:[a] *g*	Scale factor
953	Northridge, USA	29·99	0·52	0·739753
1602	Duzce, Turkey	55·90	0·82	0·422518
1111	Kobe, Japan	40·96	0·51	0·435937
1158	Kocaeli, Turkey	27·18	0·36	1·184186
1633	Manjil, Iran	53·52	0·51	0·604537
725	Superstition Hills, USA	22·30	0·45	0·973875
68	San Fernando, USA	28·00	0·21	0·391271

[a] PGA: peak ground acceleration.

Table 5. List of earthquake ground motions used in the dynamic analysis

strength drop marked on the pushover curves are depicted in Figure 9. It can be observed in the pushover curves that the maximum strength of the model structure with middle corridor (case 2) is significantly lower than that of the model structure without the middle corridor (case 1). The reason for the lower strength in the case 2 structure is the existence of the link beams right above the middle corridor, which yield first, well before the other members start to yield. In the case 1 structure with no link beams, plastic hinges first form at the lower storey columns, especially in the columns which are not attached to staggered walls, as can be observed in Figure 9(a). At the loading stage of the second strength drop (Figure 9(b)), all columns under compression in the first and the second stories reached collapse prevention (CP) damage state specified in the ASCE/SEI 41-06 (ASCE, 2007), which defines CP damage state of columns as plastic rotation angle of 0·002∼0·02 depending on the variables such as axial force, spacing of tie bar and shear force. At the first strength drop point of the case 2 structure, plastic hinges formed only in the link beams, as can be observed in Figure 10(a). At the second point of strength drop, the plastic hinges spread to columns (Figure 10(b)); however, the damage states of columns are much less than those of the link beams. No plastic hinge is observed in the staggered walls.

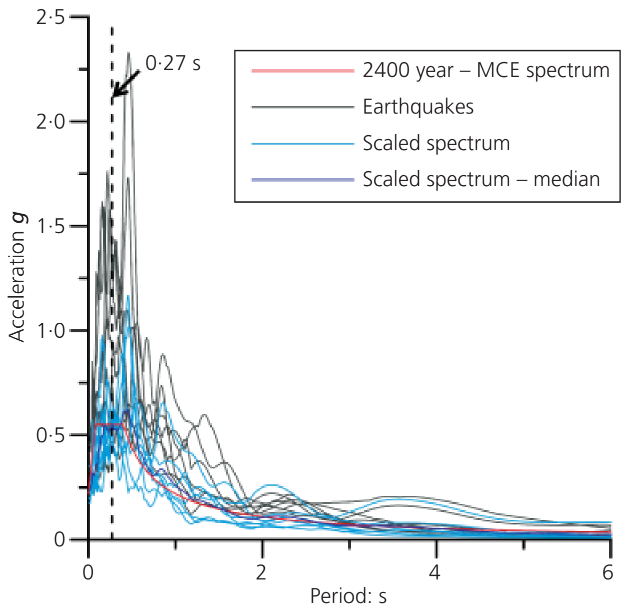


Figure 11. Response spectra of seven earthquake records and maximum considered earthquake (MCE) spectrum

Sensitivity analysis results of analysis models

For sensitivity analysis of the case 1 structure designed without middle corridor, the yield strength of reinforcing bars and the compressive strength of concrete in the structural members, such as shear walls and columns, are selected as the design variables to be investigated. For case 2 structure with middle corridor, the strength of the link beams is included. As the responses are obtained by dynamic analysis, the effect of the damping ratio is also considered for investigation. Table 4 shows the statistical properties of the selected design variables obtained from Nowak and Szerszen (2003).

Sensitivity of the design parameters on seismic responses are studied using the TDA and the FOSM methods. In the process of computing the sensitivity of a certain random variable, the other random variables are fixed at their mean values. Non-linear dynamic analyses of model structures are carried out using the seven earthquake ground motions listed in Table 5, which are selected from the database of the Pacific Earthquake Engineering Research Center (PEER, 2011). The earthquake records are scaled in such a way that the spectral accelerations at the fundamental natural frequencies of the model structures become equal to the design spectra for the maximum considered

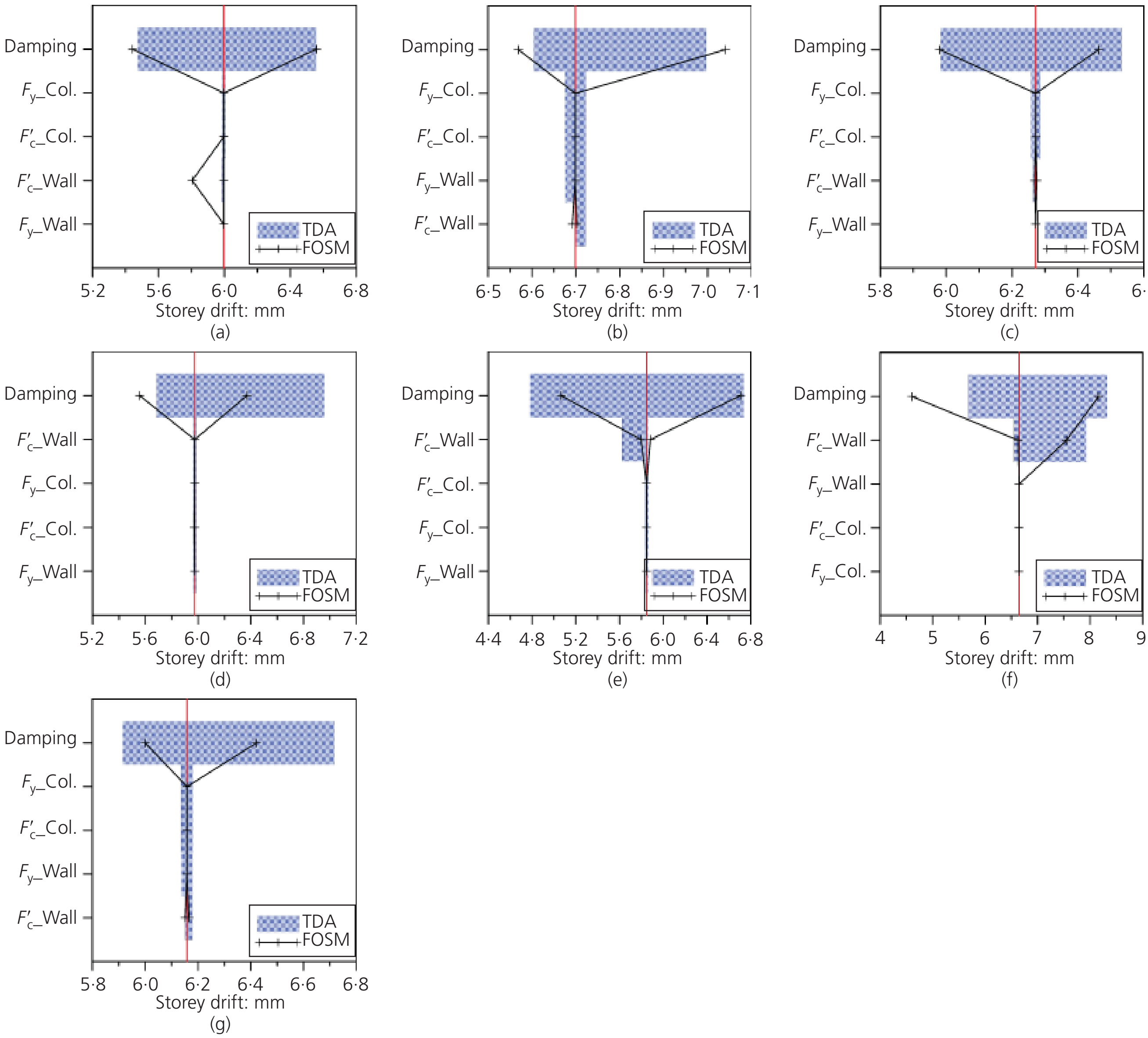


Figure 12. Swings of maximum inter-storey drifts obtained from non-linear dynamic analyses of case 1 structure using MCE-level earthquakes: (a) Northridge, USA; (b) Duzce, Turkey; (c) Kobe, Japan; (d) Kocaeli, Turkey; (e) Manjil, Iran; (f) Superstition Hills, USA; (g) San Fernando, USA

earthquakes (MCE) with return period of 2400 years in the Seoul area. The scale factor for each record is shown in Table 5. Figure 11 depicts the response spectra of the seven selected earthquake records along with their mean spectrum and the MCE design spectrum.

Figure 12 depicts the swing of the maximum inter-storey drifts of case 1 model structure without middle corridor obtained from non-linear dynamic analyses using the seven earthquakes scaled to the MCE-level design spectrum. The size of swing is determined by both the TDA and the FOSM methods, and the mean values of the seven analysis results are plotted. To compare the sensitivity according to the two different methods, results from the TDA are presented with bar graphs and those from the FOSM method are presented with solid lines in the same figure. In these diagrams, tornado diagrams are developed based on the

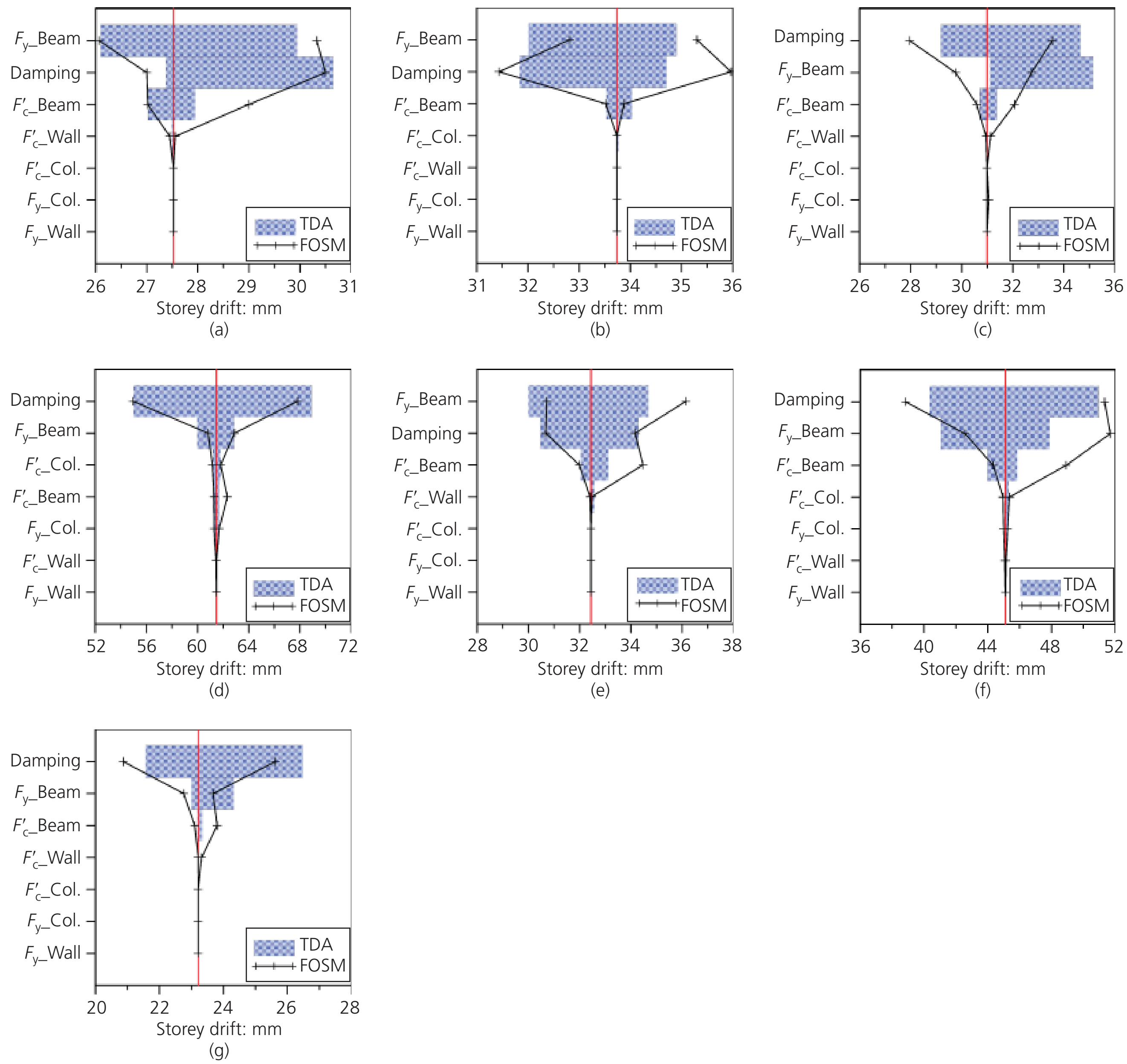


Figure 13. Swings of maximum inter-storey drifts obtained from non-linear dynamic analyses of case 2 structure using MCE-level earthquakes: (a) Northridge, USA; (b) Duzce, Turkey; (c) Kobe, Japan; (d) Kocaeli, Turkey; (e) Manjil, Iran; (f) Superstition Hills, USA; (g) San Fernando, USA

mean ± twice standard deviation of the design variables, which requires three analyses for each earthquake record. In the tornado diagram, swings for various random variables are displayed in the descending order of the swing size from top to bottom. A larger swing size implies a larger effect of the corresponding random variable on the inter-storey displacement. According to the analysis results of both TDA and FOSM, the damping ratio turns out to be the most sensitive design variable for inter-storey drift of the case 1 model structure subjected to the MCE-level ground excitation. For earthquake records such as the Manjil and the Superstition Hills earthquakes, the inter-storey drift is somewhat sensitive to the variation of the strength of staggered walls. The effects of the other variables are minor in comparison with that of damping ratio. The trend of swings in most design variables is similar in both TDA and FOSM methods. Figure 13 depicts the swing of the maximum inter-storey drifts of case 2 model structure with middle corridor. According to the analysis results, damping ratio causes the largest sensitivity in the inter-storey drift of the case 2 model structure subjected to the MCE-level ground excitation. In addition, the yield strength of the link beams turns out to be another sensitive design variable followed by the compressive strength of the link beams. The effects of the other variables are insignificant in comparison with those of the three variables. Both the TDA and the FOSM methods produce similar sensitivity in most cases.

To investigate the effect of earthquake intensity on the sensitivity of a structural response, non-linear dynamic analyses of the case 1 structure are carried out using the Superstition Hills earthquakes scaled to the spectral acceleration of 2·0g at the fundamental natural frequency, and the results are presented in Figure 14. It is observed that at the earthquake intensity higher than the MCE level intensity, the influence of the yield stress of columns increases due to the formation of plastic hinges in the columns. Figures 15 and 16 show the relative sensitivity of maximum inter-storey drifts of the case 1 and case 2 structures, respectively, obtained from incremental dynamic analyses using the seven earthquakes used above. A series of non-linear dynamic analyses are conducted with the intensity of the earthquakes varied in such a way that the spectral acceleration of a model structure corresponding to the natural frequency increases from 0·1g to 2·0g. The mean values of the seven analysis results are plotted in the figures for each earthquake intensity level. Figure 15(a) shows the relative sensitivities of the selected design variables in case 1 structure obtained by TDA method. It can be observed that when the seismic intensity is lower than a certain level, the damping ratio is the dominant factor affecting the sensitivity of the inter-storey drift. The effect of the other variables is almost negligible. As the earthquake intensity increases higher than the spectral acceleration of 1·0g at the natural period, however, the relative sensitivity of damping ratio keeps decreasing and that of the column yield strength keeps increasing as a result of plastic hinge formation in columns. The analysis results of the FOSM method, shown in Figure 15(b), indicate similar results. However, in this case, the effect of damping ratio decreases more rapidly as the earthquake intensity increases and more variables participate in affecting the sensitivity of the response. Figure 16 depicts the relative sensitivity of the design variables in case 2 structure for various intensity of earthquake records. It can be observed that the overall trend of the relative sensitivity with respect to the varying earthquake intensity is similar to that of the case 1 structure. As in the previous case, the relative sensitivity of the damping ratio decreases as the earthquake intensity increases. However, the effect of the yield and compressive strength of the link beams is quite significant in relatively lower intensity of earthquakes. As the response spectrum at the fundamental period increases over 1·0g, the effects of yield and compressive strength of columns also become important in the sensitivity of the response. The variations in the yield and compressive strength of staggered walls are not significant in either the TDA or the FOSM sensitivity analyses.

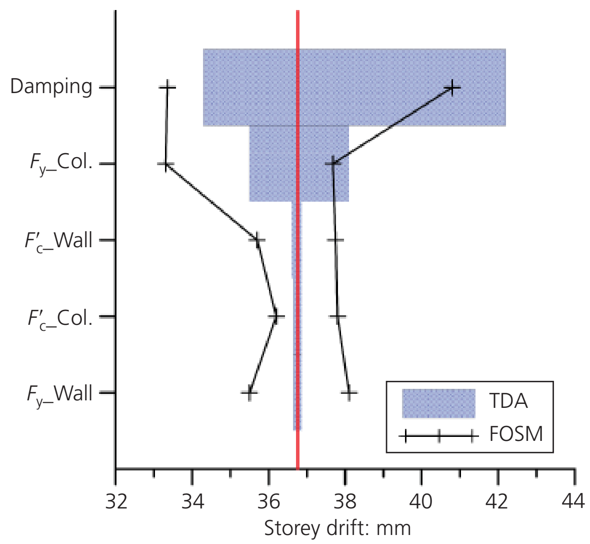


Figure 14. Swings of maximum inter-storey drifts obtained from non-linear dynamic analyses of case 1 structure using Superstition Hills earthquakes with spectral acceleration of 2·0g

Summary

In this study the sensitivity of various design variables to the seismic response of six-storey staggered wall model structures is investigated using the TDA method and the FOSM method. Sensitivity analysis is carried out considering the probabilistic distribution of design variables, such as ultimate strength of concrete, yield stress of reinforcing steel and damping ratio. Seven earthquake records scaled to the MCE-level design spectrum are used for incremental dynamic analysis of the model structures designed with (case 2) and without (case 1) middle corridor. Based on the analysis results, the following observations are made.

(*a*) According to the non-linear dynamic analysis results of the model structures subjected to the seven MCE-level

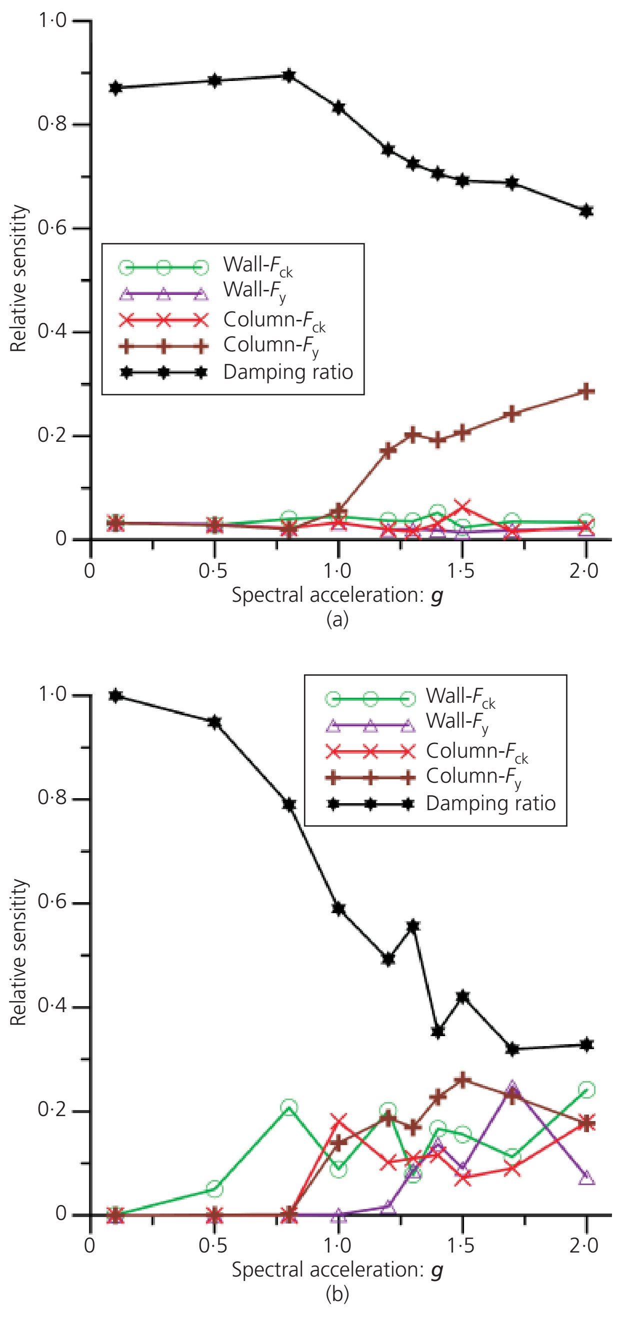


Figure 15. Relative sensitivity of maximum inter-storey drifts obtained from incremental dynamic analysis (case 1): (a) TDA; (b) FOSM

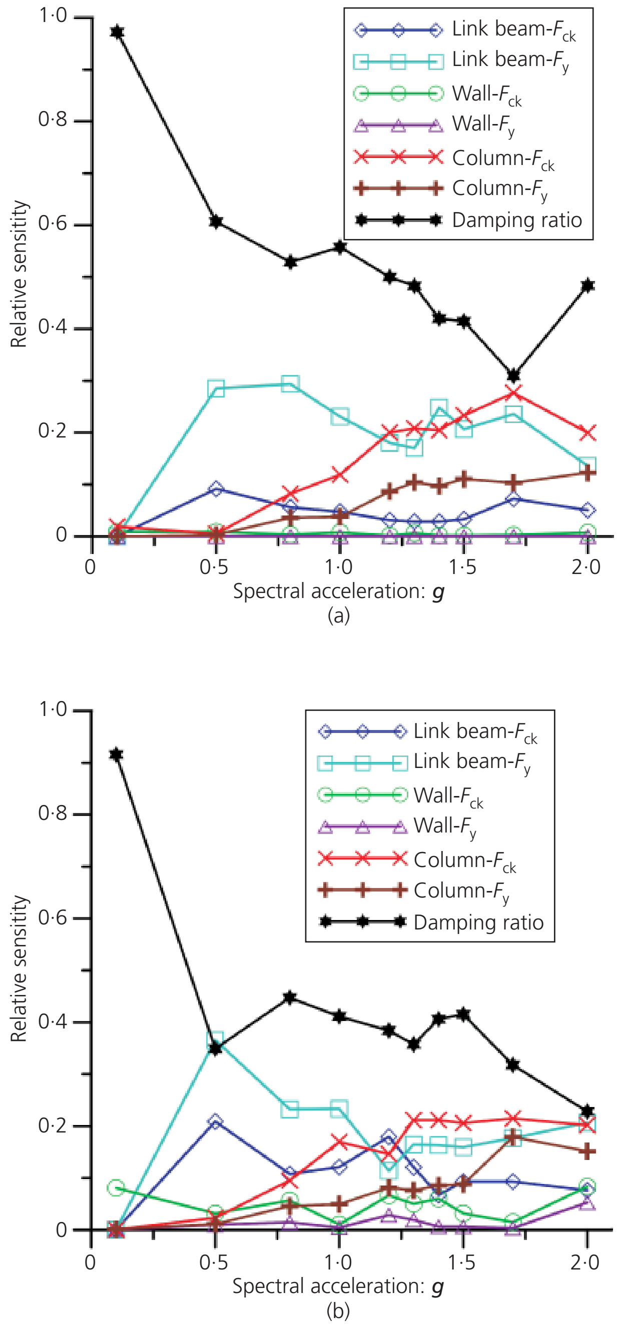


Figure 16. Relative sensitivity of maximum drifts obtained from incremental dynamic analysis (case 2): (a) TDA; (b) FOSM

earthquake ground excitations, the inter-storey drift was the most sensitive to the statistical variation of the modal damping ratio in both the case 1 and the case 2 structures. The variation in the strength of link beams also affected the sensitivity of seismic response of the case 2 structure significantly. The effects of other variables were relatively small or negligible.

(*b*) The incremental dynamic analysis results of the case 1 structure designed without middle corridor showed that when the seismic intensity was relatively low, the damping ratio was the dominant factor affecting the sensitivity of the inter-storey drift. As the earthquake intensity increased, however, the relative sensitivity of the damping ratio kept decreasing and that of the column yield strength kept increasing as a result of plastic hinge formation in columns. Compared with the results of the TDA method, the FOSM method resulted in higher sensitivity in the variables associated with staggered walls at high earthquake intensity.

(*c*) In the case 2 structure with middle corridor, the yield stress of rebars and the concrete strength in the link beams also turned out to be important factors affecting the sensitivity of seismic response when the earthquake intensity is relatively small. As the intensity of earthquake ground motions increased, the relative importance of the strength of columns gradually increased. The effect of the strength of staggered walls on the sensitivity of the seismic response was not significant in both the TDA and the FOSM methods.

(*d*) The sensitivity analysis results of the tornado diagram method and the first-order second-moment method showed a similar trend in most cases.

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REFERENCES

ACI (American Concrete Institute) (2005) *Building Code Requirements for Structural Concrete*. ACI, Farmington Hills, MI, USA, ACI 318-05.

ASCE (American Society of Civil Engineers) (2007) ASCE Standard ASCE/SEI 41-06: Seismic rehabilitation of existing buildings. American Society of Civil Engineers, Reston, VA, USA.

Baker KA and Cornell CA (2003) Uncertainty specification and propagation for loss estimation using FOSM methods. Dept. of Civil and Environmental Eng., Standford University, CA, USA, Report 142, Feb. 2003.

Englekirk R (2003) *Seismic Design of Reinforced and Precast Concrete Building*. Wiley, Hoboken, NJ, USA.

FEMA (Federal Emergency Management Agency) (2009) *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*. FEMA, US Department of Homeland Security, Washington DC, USA, FEMA-356.

Fintel M (1968) Staggered transverse wall beams for multistory concrete buildings. *ACI Journal* **65(5)**: 366–378.

ICC (International Code Council) (2009) *International Building Code*. International Code Council, Inc., Falls Church, VA, USA.

Kim J and Jun Y (2011) Seismic performance evaluation of partially staggered-wall apartment buildings. *Magazine of Concrete Research* **63(12)**: 927–939.

Kim J, Park JH and Lee TH (2011) Sensitivity analysis of steel buildings subjected to column loss. *Engineering Structures* **33(2)**: 421–432.

Lee J and Kim J (2013) Seismic performance evaluation of staggered wall structures using Fema P695 procedure. *Magazine of Concrete Research* **65(17)**: 1023–1033.

Lee TH and Mosalam KM (2005) Seismic demand sensitivity of reinforced concrete shear-wall building using FOSM method. *Earthquake Engineering and Structural Dynamics* **34(14)**: 1719–1736.

Lee TH and Mosalam KM (2006) *Probabilistic Seismic Evaluation of Reinforced Concrete Structural Components and Systems*. University of California, Berkeley, CA, USA, PEER Technical Report 2006/04.

Mee AL, Jordaan IJ and Ward MA (1975) Dynamic response of a staggered wall-beam structure. *Earthquake Engineering and Structural Dynamics* **3(4)**: 353–364.

Nowak AS and Szerszen MM (2003) Calibration of design code for buildings (ACI 318): Part 1 – Statistical models for resistance. *ACI Structural Journal* **100(3)**: 377–382.

Paulay T and Priestley MJN (1992) *Seismic Design of Reinforced Concrete and Masonry Buildings*. Wiley, Hoboken, NJ, USA.

PEER (Pacific Earthquake Engineering Research Centre) (2011) *Users Manual for the PEER Ground Motion Database Web Application*. PEER, University of California, Berkeley, CA, USA. See http://peer.berkeley.edu/nga/ (accessed 03/02/2012).

Perform 3D (2006) Nonlinear analysis and performance assessment for 3D structures. In *User Guide, Computers&Structures*. Computers&Structures Inc., Berkeley, CA, USA.

Porter KA, Beck JL and Shaikhutdinov RV (2002) Sensitivity of building loss estimates to major uncertain variables. *Earthquake Spectra* **18(4)**: 719–743.

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