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Dynamic to static eccentricity ratio for site-specific earthquakes

P. Kamatchi[*1], G.V. Ramana[2a], A.K. Nagpal[2b], Nagesh R. Iyer[1c] and J.A. Bhat[3d]

[1]*CSIR-Structural Engineering Research Centre, Chennai, India*
[2]*Indian Institute of Technology, Delhi, India*
[3]*National Institute of Technology, Srinagar, India*

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Abstract. Damage of torsionally coupled buildings situated on soil sites has been reported in literature, however no site-specific studies are available for torsionally coupled buildings having site characteristics as a parameter. Effect of torsion is being accounted in seismic codes by the provision of design eccentricity where the dynamic to static eccentricity ratio is a parameter. In this paper, a methodology to determine dynamic to static eccentricity ratio of torsionally coupled buildings has been demonstrated for Delhi region for two torsionally coupled buildings on three soil sites. The variations of average and standard deviations of frame shears for stiff and flexible edges are studied for four eccentricity ratios for the two buildings for the three sites. From the limited studies made, it is observed that the dynamic to static eccentricity ratios observed for site-specific earthquakes are different from Indian seismic code specified value, hence a proposal is made to include a comment in Indian seismic code. Methodology proposed in this paper can be adopted for any region, for the estimation of dynamic to static eccentricity ratio for site specific earthquake.

Keywords: dynamic eccentricity; seismic code; site-specific earthquake; torsionally coupled building; Delhi

1. Introduction

Damage of torsionally coupled buildings during 1985 Mexico city earthquake due to amplification of ground motion on soil sites has been documented by Chandler (1986). Influence of local soil conditions on the intensity of shaking at the ground surface has been well recognized by researchers (Seed and Idriss 1969, Idriss and Seed 1970, Lam *et al*. 2001, Govindarajulu *et al*. 2004, Tezcan *et al*. 2002, Bakir *et al*. 2005, Kamatchi *et al*. 2007) through the damage of buildings observed in many destructive earthquakes. The time history felt at the soil surface is significantly different from that of bedrock time history due to the modification of ground motion as it passes

*Corresponding author, Principal Scientist, E-mail: kamat@serc.res.in
[a]Professor, E-mail: ramana@civil.iitd.ac.in
[b]Professor, E-mail: aknagpal@civil.iitd.ac.in
[c]Former Director, E-mail: nriyer@serc.res.in
[d]Professor, E-mail: bhat_javed@yahoo.com

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through the soil layers overlying the bedrock. Building codes are simplified tools and do not adequately represent any single earthquake event from a probable source for the site under consideration. Recently, it has been recommended (Heuze *et al.* 2004, Mammo 2005, Balendra *et al.* 2002) that, in addition to the use of provisions of seismic codes, site-specific analysis which includes generation of strong ground motion at bedrock level, propagating it through soil layers, arriving at the surface ground motions and carrying out time history analysis is also important.

On the other hand, torsional provisions in seismic codes for coupled buildings have been reviewed by many researchers (Tso and Dempsey 1980, Chandler and Hutchinson 1987, Cheung and Tso 1987, Tso and Smith 1999, Chandler *et al.* 1996, Wong and Tso 1995, Llera and Chopra 1994, Agarwal and Gupta 1994, Shakib 2004, Humar and Kumar 2000). Tso and Dempsey (1980) have reviewed torsional provisions in seismic codes of Canada, Mexico, New Zealand, United States (Applied Technology Council, ATC3) and Germany and reported that four of the five codes underestimate the torsional moment and the edge displacement significantly when the eccentricity is small and the uncoupled torsional and lateral frequencies are close. Torsional provisions in seismic codes have been reviewed by Chandler and Hutchinson (1987) by time history analyses of selected earthquakes. It has been stated that except Eurocode 8, other codes underestimate the dynamic torque especially when the static eccentricity is small and uncoupled torsional to translational frequency is close to unity. Humar and Kumar (2000) reviewed torsional provisions in National Building code of Canada (NBCC 1995), Uniform building code (UBC 1997), National Earthquake Hazard Reduction Program (NEHRP 1997), New Zealand Standards (NZS 1992) and Mexico Code (1993). According to Humar and Kumar (2000) all the five codes considered, predict un-conservative design forces for the elements in the stiff side of the building for uncoupled torsional to translational frequency ratio of less than 0.75.

Dynamic amplification factor specified in Indian Standard IS 1893-1984 (BIS, 1984) has been studied by Agarwal and Gupta (1994) using stochastic approach. It has been reported that provisions of seismic codes may lead to severely underestimated torsional moments when the eccentricity ratio (e_s/r) (r-is the radius of gyration) is small and uncoupled torsional to translational frequency ratio is close to unity. A simple equation for design eccentricity has been proposed by Shakib (2004) by taking into account the important structural parameters and soil-structure interaction effects. Recently many studies reported on the torsional provisions for base-isolated and retrofitted buildings (Tena-Colunga and Escamilla-Cruz 2007, Kilar and Koren 2009, Nakano *et al.* 2000) reemphasize the importance of looking into existing torsional provisions in seismic codes. Paglietti *et al.* (2011) have emphasized the importance of carrying out dynamic analyses for torsionally coupled buildings and commented that static methods of Eurocode 8 may lead to non-conservative forces.

Dynamic eccentricity is defined as the distance of a point from centre of stiffness, at which the uncoupled base shear has to be applied to realize the maximum dynamic torque (Tso and Dempsy, 1980). It has been highlighted in literature that most of the seismic codes except German code represent dynamic eccentricity as a linear function of static eccentricity and it is suggested that especially when the ratio of torsional to translational frequency (Ω) is equal to one, the dynamic eccentricity term has to be represented by a bilinear function of static eccentricity. The influence of the other parameters viz., aspect ratio of foundation mat and uncoupled torsional to translational frequency ratio on dynamic eccentricity have been explored by Wu and Leyendecker (1984). It has been emphasized that all the parameters contributing for rotational response have to be included in torsional provisions of seismic codes in addition to static eccentricity. In the present study site characteristics has been identified as a parameter influencing torsional response.

From the review of literature in seismic code for linear elastic systems, it is clear that no site-specific studies are available on the dynamic eccentricity provision in seismic code for torsionally coupled buildings. Hence there is a need to study the effect of local soil conditions on the dynamic to static eccentricity ratio. In this paper, a methodology has been proposed for determining dynamic to static eccentricity ratio for site-specific earthquakes for Delhi region.

The proposed methodology has been demonstrated for coupled buildings assumed to be situated on three soil sites (viz., site 1, site 2 and site 3) of Delhi capital city of India for site-specific scenario earthquakes. Artificial strong ground motions including source path effects using stochastic finite fault model (Beresnev and Atkinson 1997, 1998) are generated for a long distance scenario earthquakes of moment magnitude (M_w) 7.5 and 8.5 for a rock site at Delhi and propagated through chosen soil sites. Using the surface level time histories for three soil sites, frequency domain analyses are carried out by modelling the building with two translational and one rotational degrees of freedom at each floor. The dynamic to static eccentricity ratio has been obtained as the ratio of maximum torsional moment of coupled system to maximum translational base shear of uncoupled system for two torsionally coupled buildings and the comparison has been made with Indian seismic code specified value of 1.5. Studies indicate the necessity of determination of dynamic to static eccentricity ratio for site-specific earthquakes.

2. Dynamic eccentricity

Effect of torsion is being accounted in seismic codes by the provision of design eccentricity wherein the dynamic to static eccentricity ratio is a parameter. Asymmetric buildings (*j*th floor plan with dimensions '*l*' and '*b*' as shown in Fig. 1) undergo torsionally coupled motions under earthquake excitations. In these buildings the static eccentricity occurs due to the non-coincidence of centre of mass (CM) and centre of rigidity (CR) at a floor level and is equal to the distance between them. When the building is subjected to earthquake excitation, eccentricity is no longer remains the same and a factor of 1.5 has been suggested in seismic codes to account for dynamic effects.

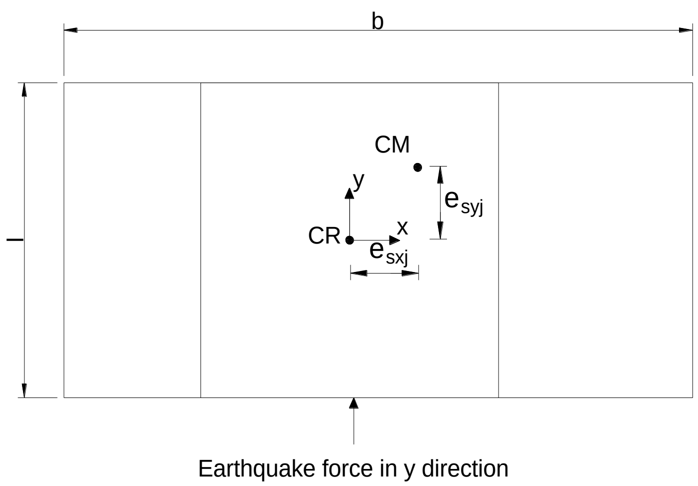


Fig. 1 Plan of asymmetric framed building

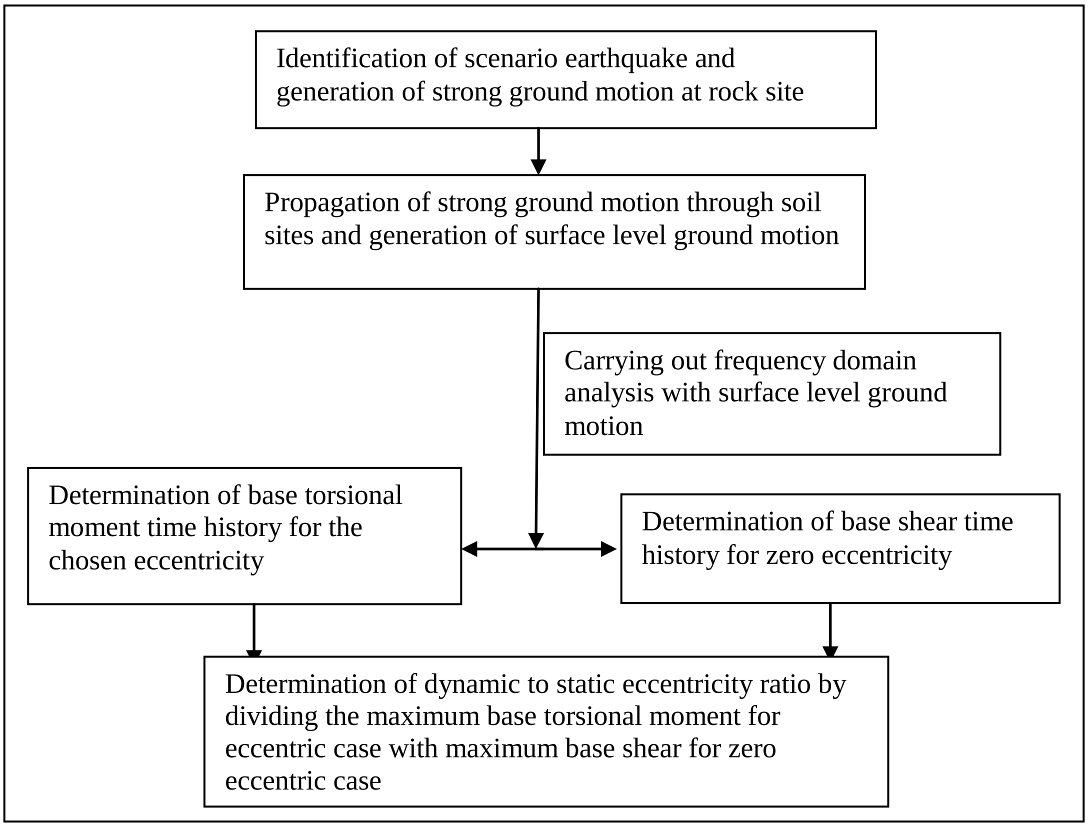


Fig. 2 Methodology to determine dynamic to static eccentricity ratio for site-specific earthquake (Kamatchi *et al*. 2015)

3. Methodology to determine dynamic to static eccentricity ratio for site-specific earthquake

Methodology to determine dynamic to static eccentricity ratio for site-specific earthquake is shown in Fig. 2. Major steps involved are identification of scenario earthquake, generation of strong ground motion for a rock site, propagation of ground motion through soil stratum and carrying out three dimensional frequency domain analyses to arrive at the base shear and torsional moment time histories. Dynamic to static eccentricity ratio is obtained as the ratio of maximum base torque for the chosen static eccentricity to maximum base shear for zero eccentricity. Proposed methodology has been demonstrated for Delhi city for two torsionally coupled buildings.

3.1 Identification of scenario earthquake

Seismologists (Bilham *et al*. 1998, Khattri 1999) have reported that three major thrust planes viz., Main Central Thrust (MCT), Main Boundary Thrust (MBT) and Main Frontal Thrust (MFT) exist in Himalayas due to the relative movement of Indian plate by 5 cm/year with respect to Eurasian plate. Khattri (1999) has estimated the probability of occurrence of a great earthquake of moment magnitude 8.5 from the large unbroken segment called central seismic gap (Fig. 3)

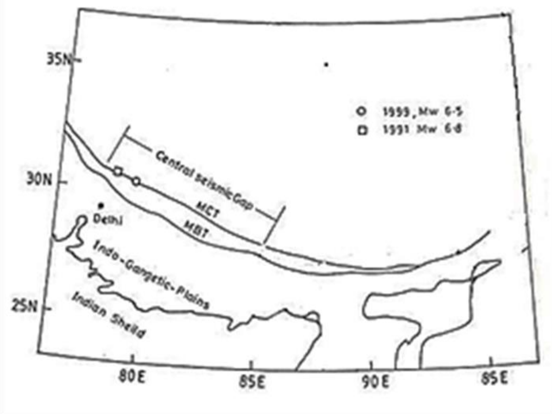
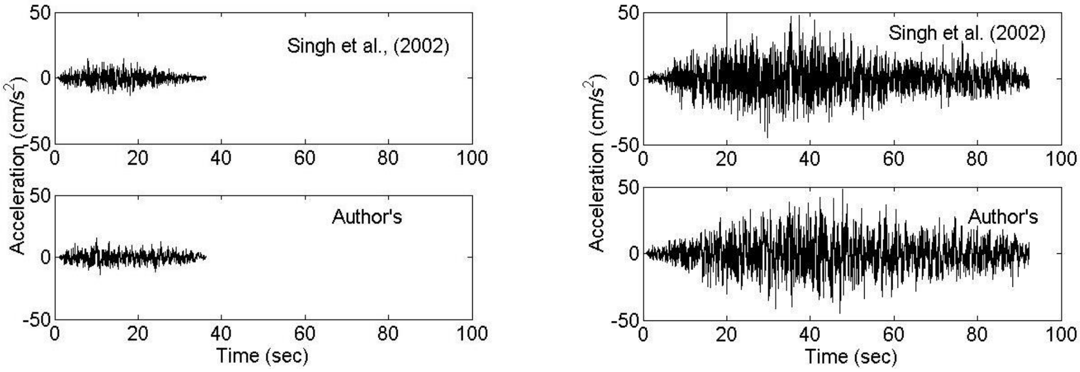


Fig. 3 Central seismic gap of Himalayan region

between MBT and MCT in the next 100 years to be 0.59. Delhi is situated at a distance of roughly 200 km from MBT and 300 km from MCT. Hence in the present study scenario earthquakes for Delhi city are assumed to be originated from central seismic gap of Himalayan region.

3.2 Generation of strong ground motion

Generation of artificial strong motions using stochastic models by identifying major fault zones and propagating seismic waves generated at these potential sources to the sites of interest is well accepted in literature (Boore 1983, 2003, Beresnev and Atkinson 2002). In this process, path effects and anelastic attenuation effects predicted by the empirical and theoretical models (Beresnev and Atkinson 2002) are used. For source representation, point source models (Boore and Atkinson 1987) or finite source models (Hartzell 1978) are widely used. Finite fault simulation program (FINSIM) has been widely used for the generation of ground motions of large size earthquakes (Beresnev and Atkinson 1998, Atkinson and Beresnev 2002, Roumelioti and Beresnev 2003) and hence has been adopted in the present study.



(a) (b)

Fig. 4 Comparison of artificial ground motions generated for a rock site at Delhi for earthquakes from central seismic gap with similar generations from literature (a) M_w=7.5 (b) M_w=8.5

Seismological parameters used in the generation of rock outcrop motions for Delhi region have been broadly adopted from Singh *et al.* (2002) and has been given elsewhere (Kamatchi 2008, Kamatchi *et al.* 2008). In order to minimize the noise due to random fault rupture in the simulation, 15 ground motions have been generated for each earthquake magnitude. One simulation of the time histories generated in the present study for magnitudes 7.5 and 8.5 for rock outcrop (Ridge observatory) have been compared (Fig. 4), with one simulation obtained from Singh (2005) through personal communication.

3.3 Propagation of strong ground motion through soil layer using one dimensional equivalent linear analysis

One dimensional equivalent linear vertical wave propagation analysis is the widely used numerical procedure for modeling soil amplification problem (Idriss 1990, Yoshida *et al.* 2002). In one dimensional wave propagation analysis, soil deposit is assumed to be having number of horizontal layers with different shear modulus (G), damping (ζ) and unit weight (ρ). In the linear analysis, G and ζ are assumed to be constant in each layer. Since the soil will be subjected to nonlinear strain (Yoshida *et al.* 2002) even under small earthquake excitation, equivalent linear analysis is preferred over linear analysis and the equivalent linear analysis program SHAKE (Schnabel *et al*. 1972, Ordonez 2000) is used in the present study. Equivalent linear modulus reduction ($G/G_{\max}$) and damping ratio (ξ) curves generated from laboratory test results are adopted from Vucetic and Dobry (Vucetic and Dobry 1991) depending on the plasticity index of different soil layers. Since SHAKE is a total stress analysis program (Yoshida *et al*. 2002) depth of water table has not been considered in the analysis.

3.3.1 Typical soil strata for Delhi region

Three actual soil sites designated as site 1, site 2 and site 3 located in Delhi as shown in Fig. 5 are chosen for the present study. Layer wise soil characteristics (medium type) and the depth to the base of the layer from the surface are available elsewhere (Kamatchi 2008, Kamatchi *et al.* 2008). Shear wave velocity (V_s) measurements are not available for the sites chosen. However the variations of standard penetration test (SPT) 'N' values with depth are available as shown in Fig. 6. Variation of shear wave velocity along the depth in the present study is obtained by using the correlations suggested for Delhi region by Rao and Ramana (2004) as given in Eqs. (1)-(2).

Fig. 5 Location of three soil sites of Delhi region

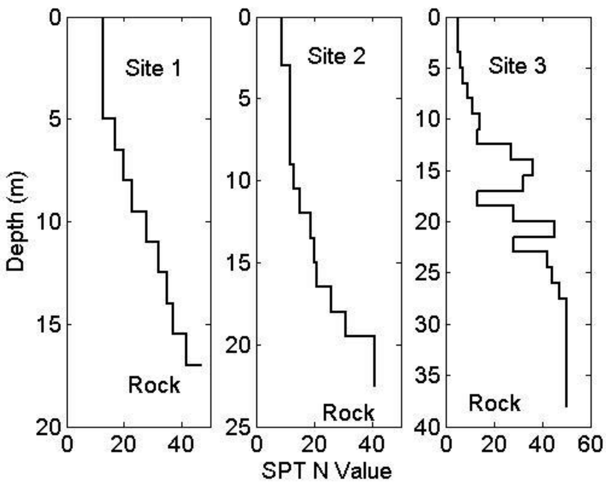


Fig. 6 Variation of SPT '*N*' value along depth of soil strata for Site 1, Site 2 and Site 3

$$V_s=79N^{0.43} \quad \text{(sand)} \tag{1}$$

$$V_s=86N^{0.42} \text{ (silty sand/sandy silt)} \tag{2}$$

3.4 Structural model and analysis procedure for determining dynamic to static eccentricity ratio

Model for three dimensional analysis of coupled building in the present study is assumed to be consisting of a floor slab with infinite in-plane rigidity with no bending stiffness (rigid diaphragm). Movement of a floor of a framed building system is described by three global degrees of freedom, viz., two translational components along two orthogonal axes (x and y) and one rotational component about a vertical (z) axis with number of stories equal to N. Degrees of freedom (DOF) in x direction is from 1 to N, DOF in y direction is from N+1 to 2N and DOF in z direction is from 2N+1 to 3N as shown in Fig. 7(a). A 3-D frame model (MDOF) is considered, in which all the stiffening elements are assumed to act independently. Lateral stiffness matrix of each frame is obtained in local coordinate system and then transformed into global coordinate system for the whole building. The global stiffness matrix (3N×3N; N being the number of storeys considered) of the building is obtained by summing stiffness matrices of all frames/stiffening elements (Bhat 2003). The equation of motion of the building system is expressed as

$$[M]\{\ddot{q}^t(t)\} + [C]\{\dot{q}(t)\} + [K]\{q(t)\} = 0 \tag{3}$$

where [M], [K] and [C] are the mass, stiffness and damping matrices in global coordinates respectively. Global stiffness matrix [K] is assembled from condensed stiffness matrices of individual frames. $\{\ddot{q}^t(t)\}$ is the total acceleration which includes acceleration of ground and structure and $\{\dot{q}(t)\}$, $\{q(t)\}$ represent velocity and displacement of the structure only. In Fig. 8 displacement of ground in x direction and y direction are represented by q_{gx} and $q_{gy.}$ Displacement of building in x and y direction for different storey levels are represented by $q(1)$ to $q(N)$ and

$q(N+1)$ to $q(2N)$ respectively. Rotation of first to Nth storey is represented as $q(2N+1)$ to $q(3N)$. Vector $\{\ddot{q}^t(t)\}$ is expressed as

$$\{\ddot{q}^t(t)\} = [A]\{\ddot{q}_g(t)\} + \{\ddot{q}(t)\} \tag{4}$$

In Eq. (4) transpose of vector $\{\ddot{q}_g(t)\}$ is $\{\ddot{q}_g(t)\}^T = \{\{\ddot{q}_{gx}(t)\}\ \{\ddot{q}_{gy}(t)\}\}$ where $\{\ddot{q}_{gx}(t)\}$ and $\{\ddot{q}_{gy}(t)\}$ are ground motions in x and y directions respectively, [A] is the transformation matrix of the form given below

$$[A]^T = \begin{bmatrix} 1 & 1 & 1 \ldots\ldots 1 & 0 & 0 & 0 \ldots\ldots 0 & 0 & 0 \ldots\ldots 0 \\ 0 & 0 & 0 \ldots\ldots 0 & 1 & 1 & 1 \ldots\ldots 1 & 0 & 0 \ldots\ldots 0 \end{bmatrix} \tag{5}$$

On making use of Eq. (4) in Eq. (3), Eq. (6) is obtained as follows

$$[M]\{\ddot{q}(t)\} + [C]\{\dot{q}(t)\} + [K]\{q(t)\} = [M][A]\{\ddot{q}_g(t)\} \tag{6}$$

Eq. (6) is solved using frequency domain analysis procedure implemented in an in-house computer program developed at Indian Institute of Technology, Delhi (Nagpal 1976, Bhat 2003, Kamatchi 2008), and the solution yields $\{q(t)\}$, the vector of global structural displacement. For the building, floor load time histories $\{Q(t)\}$ are obtained by pre-multiplying $\{q(t)\}$, with global stiffness matrix [K].

The building storey shear time histories designated as $V_{jx}(t)$ and $V_{jy}(t)$ at level j in x and y-direction respectively are obtained by summing the lateral floor loads in x and y directions above level j respectively as

$$V_{jx}(t) = \sum_{k=j}^{N} Q_k(t) \tag{7}$$

$$V_{jy}(t) = \sum_{k=N+j}^{2N} Q_k(t) \tag{8}$$

Similarly, the floor torque time history at a level j designated as $T_{jz}(t)$ is obtained by summing the torsional moments about z-axis above the level j

$$T_{jz}(t) = \sum_{k=2N+j}^{3N} Q_k(t) \tag{9}$$

Absolute maximum value of the storey shears in x, y direction and maximum storey torque about z axis are designated as $|V_{jx,\max}|$, $|V_{jy,\max}|$ and $|T_{jz,\max}|$ respectively. Maximum value of the storey shears in x, y direction for the building undergoing pure translation ($e_{sxj}=e_{syj}=0$) are designated as $|V^{tr}_{jx,\max}|$, and $|V^{tr}_{jy,\max}|$ respectively. Static eccentricity value is given as an input to the program as the coordinates of center of mass with respect to centre of rigidity or geometric centre of building. For a symmetric building the coordinates of centre of mass are (0, 0) at each floor and for eccentric building the coordinates of the centre of mass are taken as (e_{sxi}, e_{syi}) at ith floor level. For a torsionally coupled building with static eccentricities, e_{sxj} in x direction (Fig. 1) and e_{syj} in y direction, the dynamic eccentricities e_{dxj} and e_{dyj} in x and y direction respectively are obtained (Tso and Dempsey 1980) as the ratio of maximum storey torques and the maximum storey shears as given below

$$e_{dxj} = \frac{\left|T_{jz,max}\right|}{\left|V^{tr}{}_{jy,max}\right|} \tag{10}$$

$$e_{dyj} = \frac{\left|T_{jz,max}\right|}{\left|V^{tr}{}_{jx,max}\right|} \tag{11}$$

Ratio of dynamic to static eccentricity ratios in x and y directions designated as R_{xj} and R_{yj} are obtained as follows

$$R_{xj} = \frac{e_{dxj}}{e_{sxj}} \tag{12}$$

$$R_{yj} = \frac{e_{dyj}}{e_{syj}} \tag{13}$$

Maximum storey shears of the individual frames designated as $|V^{e}_{j,\max}|$, which are of design interest are also evaluated by pre-multiplying the displacement vectors of each frame with the condensed stiffness matrix of the frame (Nagpal 1976, Bhat 2003, Kamatchi 2008).

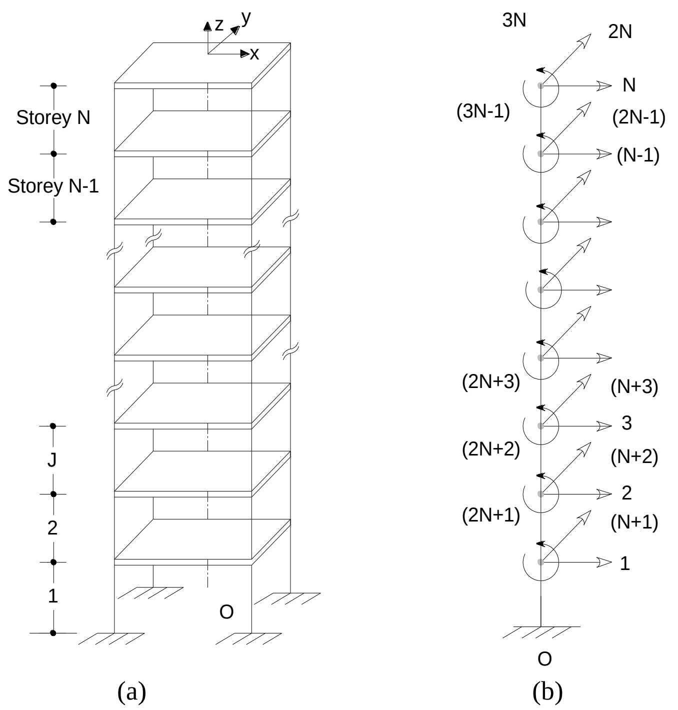


Fig. 7 Multi-storey building system: (a) schematic 3-D model; and (b) degrees of freedom

4. Numerical study

Scenario earthquakes simulated for Delhi Ridge observatory for earthquake magnitudes M_w=7.5 and 8.5 are used as rock motions and the free field time histories are obtained on three soil sites. A three storey and a ten storey torsionally coupled reinforced concrete framed building designated as TC1 and TC2 respectively with plan details as given in Fig. 9 are chosen for the present study. Plan dimensions of TC1 and TC2 are same and the properties of TC1 and TC2 are given in Table 1. Elastic modulus of beam and column elements of the buildings are assumed as 2.549e7 kN/m^2. Buildings TC1 and TC2 are assumed to have the same static eccentricity e_{sxj} for all the stories. For the present study, four values of e_{sxj}/b (j=1, 2, 3) from 0.05 to 0.2 in steps of 0.05 have been considered. First uncoupled (with $e_{sxj}=e_{syj}=0$) translational frequency (ω_y) and uncoupled torsional frequency (ω_θ) of TC1 are 20.93 radians/s and 22.428 radians/s respectively. Uncoupled torsional to translational frequency ratio (Ω) of TC1 is thus equal to 1.07. It may be noted that the properties of TC1 are chosen in such a way that Ω=1.07 which is closer to Ω=1.0 for which effect of eccentricity is reported to be the maximum (Tso and Dempsey 1980, Agarwal and Gupta 1994, Wu and Leyendecker 1984). For this purpose several trials were made to arrive at the structural properties of the building.

Structural properties of TC2 are varied along the height of the building as given in Table 1. First uncoupled translational frequency (ω_y) and uncoupled torsional frequency (ω_θ) of TC2 (with $e_{sxj}=e_{syj}=0$) are 7.21 radians/s and 8.45 radians/s respectively. Uncoupled torsional to translational frequency ratio (Ω) of TC2 is thus equal to 1.17. Buildings are analysed for the fifteen simulations of ground motions which are applied in y direction.

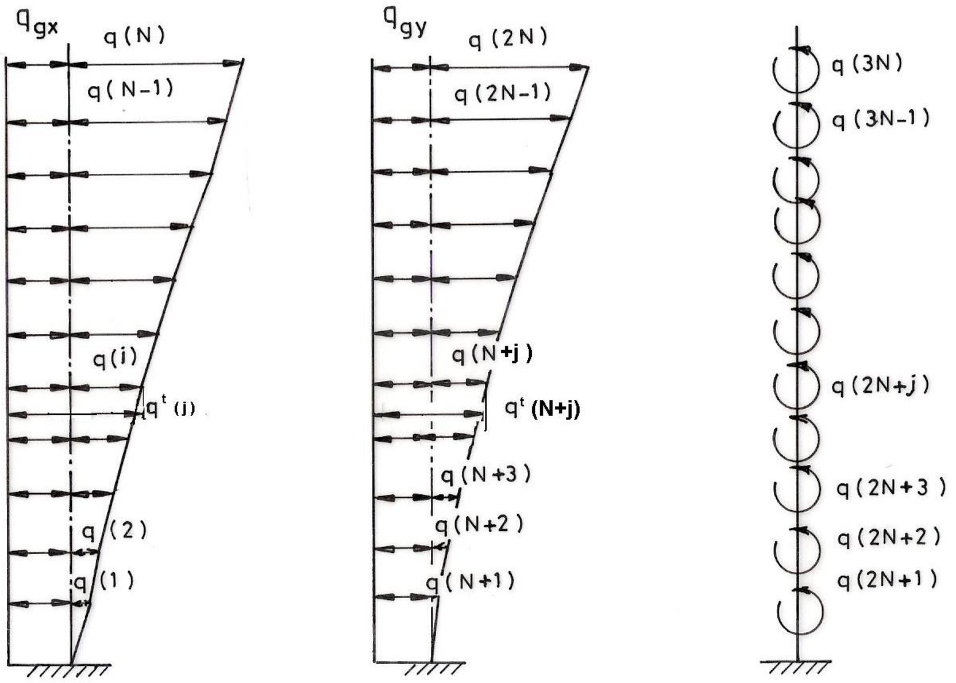


Fig. 8 Displacements of the building: (a) x-displacement (b) y-displacement and (c) z-rotation

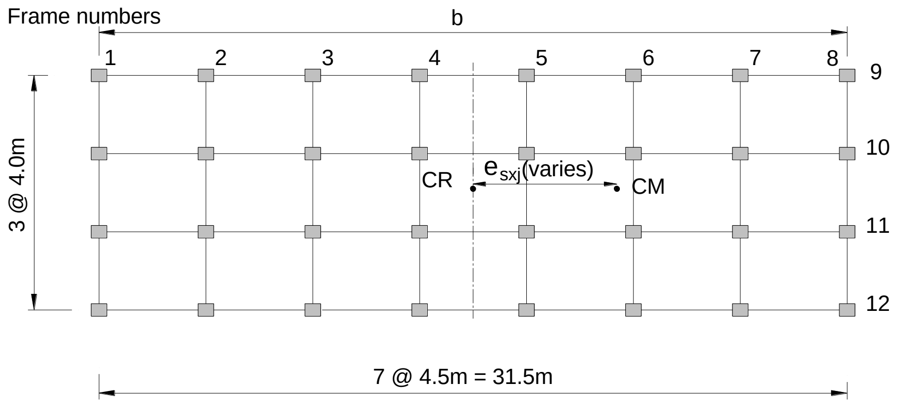


Fig. 9 Plan of TC1 and TC2

4.1 Response of three storey building, TC1

Objective of the present study is to bring out the importance carrying out site-specific analysis of torsionally coupled buildings. The time period of the coupled building is different from the time period of the uncoupled building. Fundamental translational time period of the building gets modified for different eccentricities, which leads to variation of structural response of building when it is situated at different sites. Typical plots of base shear time history and base torque time history of TC1 for the three sites for 5, 10, 15 and 20 percent eccentricity ratios are shown in Figs. 10 and 11 for one simulation of M_w=7.5 earthquake.

Absolute peak value of symmetric storey shears and absolute peak value of storey torques are given in Tables 2 and 3 respectively for M_w=8.5 for TC1 for the three sites. From Tables 2 and 3 considerable variations in symmetric storey shears and storey torque can be observed for the three sites. Variations in storey torques are more for higher eccentricities compared to lower eccentricities. One way of quantifying the effect is to find the variation of dynamic eccentricity of building for different static eccentricities and for different sites.

Dynamic to static eccentricity ratio, R_{xj} has been worked out for each simulation using the absolute maximum values of storey shears, and the maximum values of storey torques. Values of R_{xj} for the three sites are given in Table 4 for 5, 10, 15 and 20 percent eccentricity ratios for earthquake of moment magnitude, M_w=7.5. It is seen from the results that R_{xj} values are quite different from code specified value of 1.5 for lower e_{sxj}/b values upto a maximum of 3.76 and 2.68 for 5 and 10 percent eccentricity ratios respectively. Though the variation of R_{xj} values with 1.5 is lesser for 15 and 20 percent eccentricity ratio it is seen that, R_{xj} values of three sites are different. Similarly R_{xj} values for earthquake magnitude M_w=8.5 are also worked out as given in Table 5 for TC1 and the comparison has been made with the Indian seismic code IS 1893(Part 1), 2002 (BIS, 2002) specified value of 1.5 for different eccentricity ratios. Even for the stronger earthquake shaking M_w=8.5, similar deviations with respect to seismic code specified value of 1.5 are observed.

Frame shears are of design interest and the average maximum frame shears for the stiff edge frame (1) and flexible edge frame (8) are worked out for different eccentricity ratios as given in Fig. 12 for M_w=8.5. The standard deviations of the frame shears for stiff edge and flexible edge are given in Fig. 13. It is seen that, the variations of frame shear for the flexible edge are more compared to stiff edge for the three sites for different eccentricity ratios. The variation of frame shears for the three sites considered clearly brings out the difference in shears experienced by frames for the three sites considered.

4.2 Response of ten storey building, TC2

Typically results for earthquake magnitude M_w=7.5 only are presented in this paper for building TC2. Absolute peak value of symmetric storey shears and absolute peak value of storey torques are given in Tables 6 and 7 respectively for M_w=7.5 for TC2 for the three sites. R_{xj} values are evaluated from the maximum values of storey shears, $|V_{jy,\max}|$ and the maximum values of storey torques, $|T_{jz,\max}|$ for each simulation and the average of 15 simulations are obtained. Variation of R_{xj} for the three sites are given in Table 8 for e_{sxj}/b=0.05; e_{sxj}/b=0.1; e_{sxj}/b=0.15; and e_{sxj}/b=0.2. As has been observed for TC1, R_{xj} values are quite different from code specified value of 1.5 for lower e_{sxj}/b values viz., 0.05 and 0.1. For TC2 also, average maximum frame shears for the stiff edge frame (1) and flexible edge frame (8) are worked out for different eccentricity ratios as given in Fig. 14 for M_w=7.5. The standard deviations of the frame shears for stiff edge and flexible edge are given in Fig. 15. Unlike TC1, the variations of frame shear are more for both stiff edge and flexible edge for TC2 for the three sites and for different eccentricity ratios.

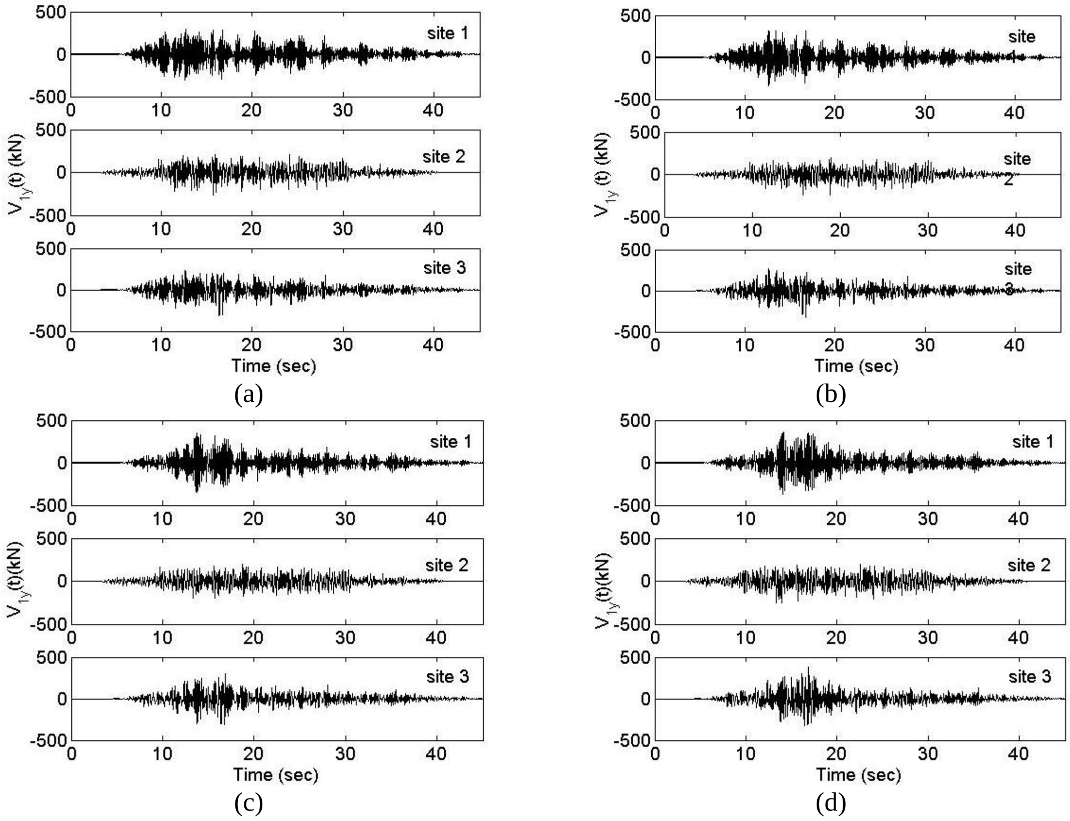
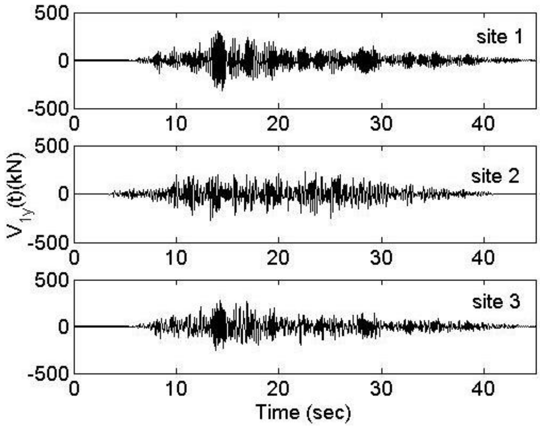
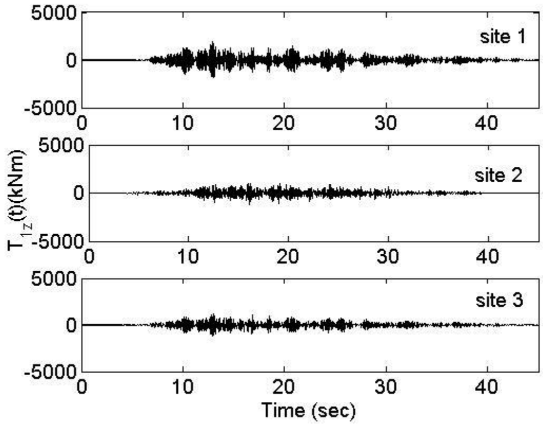


Fig. 10 Base shear time history, $V_{1y}(t)$ of TC1 for the three sites, M_w=7.5 (a) e_{sxj}/b=0.0 (b) e_{sxj}/b=0.05 (c) e_{sxj}/b=0.1 (d) e_{sxj}/b=0.15 (e) e_{sxj}/b=0.2

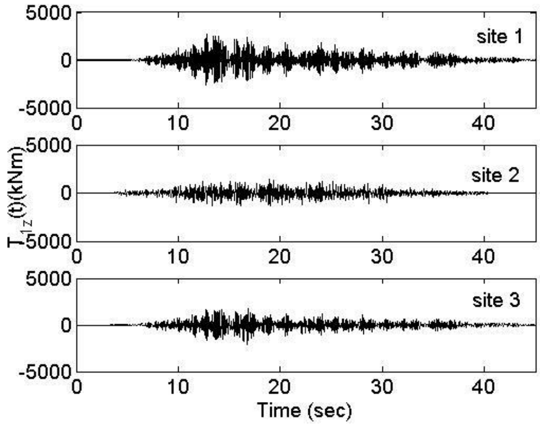


(e)

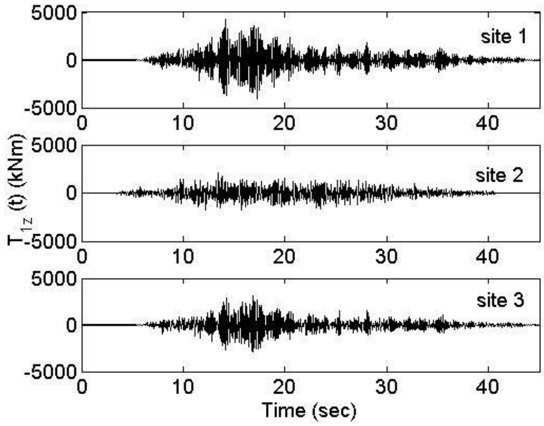
Fig. 10 Continued



(a)



(b)



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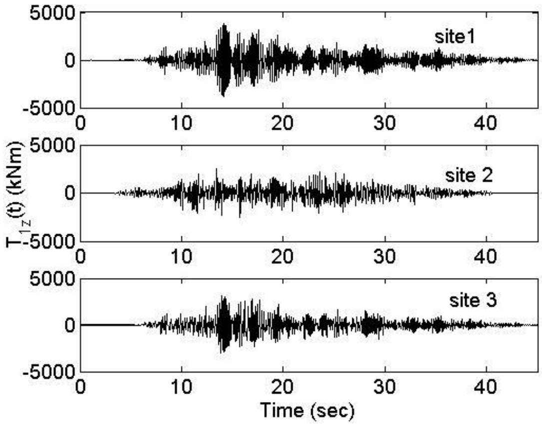

(d)

Fig. 11 Base torque time history, $T_{1z}(t)$ of TC1 for the three sites, M_w=7.5; (a) e_{sxj}/b=0.05 (b) e_{sxj}/b=0.1 (c) e_{sxj}/b=0.15 (d) e_{sxj}/b=0.2

Table 1 Structural properties of TC1 and TC2

Building	Storey No	Moment of inertia of square columns 10^{-4} (m^4) (Frames 1,8)	Moment of inertia of square columns 10^{-4} (m^4) (Frames 2,3,4,5,6,7)	Area of columns 10^{-2} (m^2) All frames	Moment of Inertia of beams about axis of bending 10^{-4} (m^4) All frames	Storey height (m)	Mass of all the floors except top floor (kN-s^2/m)	Mass of the top floor (kN-s^2/m)
TC1	1 to 3	11.1	21.0	infinite	infinite	3.0	410	205
TC2	1 to 5	108	108	36	200	3.0	600	600
	6 to 10	40	40	20	200	3.0	600	

Table 2 $\left|V^{tr}_{jy,max}\right|$ in kN of TC1 for the three sites, M_w=8.5, e_{sxj}/b=0.0

Storey No.	Site 1	Site 2	Site 3
3	539.48	950.93	803.06
2	1403.23	2608.22	2209.84
1	1830.27	3654.39	3118.78

Table 3 $\left|T_{jz,max}\right|$ in kN-m of TC1 for the three sites, M_w=8.5

Storey No.	e_{sxj}/b=0.05			e_{sxj}/b=0.10			e_{sxj}/b=0.15			e_{sxj}/b=0.20		
	Site 1	**Site 2**	**Site 3**	**Site 1**	**Site 2**	**Site 3**	**Site 1**	**Site 2**	**Site 3**	**Site 1**	**Site 2**	**Site 3**
3	2819	5591	4320	3687	7843	6424	4047	8909	8588	4342	8572	10283
2	7497	15012	11762	9681	21109	17530	10419	23892	23339	11039	22874	28081
1	10318	20353	16195	13238	28928	24193	14284	32605	31994	15117	30754	38457

Table 4 R_{xj} of TC1 for the three sites, M_w=7.5

Storey No.	e_{sxj}/b=0.05			e_{sxj}/b=0.10			e_{sxj}/b=0.15			e_{sxj}/b=0.20		
	Site 1	**Site 2**	**Site 3**	**Site 1**	**Site 2**	**Site 3**	**Site 1**	**Site 2**	**Site 3**	**Site 1**	**Site 2**	**Site 3**
3	3.52	3.76	3.55	2.36	2.14	2.67	1.74	1.30	2.22	1.49	0.98	1.76
2	3.51	3.73	3.52	2.32	2.11	2.68	1.73	1.28	2.20	1.39	0.96	1.74
1	3.61	3.71	3.47	2.39	2.07	2.65	1.78	1.26	2.15	1.45	0.95	1.69

Table 5 R_{xj} of TC1 for the three sites, M_w=8.5

Storey No.	e_{sxj}/b=0.05			e_{sxj}/b=0.10			e_{sxj}/b=0.15			e_{sxj}/b=0.20		
	Site 1	**Site 2**	**Site 3**	**Site 1**	**Site 2**	**Site 3**	**Site 1**	**Site 2**	**Site 3**	**Site 1**	**Site 2**	**Site 3**
3	3.33	3.73	3.42	2.19	2.62	1.71	1.65	1.98	2.26	1.39	1.43	2.03
2	3.39	3.65	3.38	2.22	2.57	1.69	1.65	1.94	2.24	1.36	1.39	2.02
1	3.57	3.54	3.30	2.33	2.51	1.65	1.74	1.89	2.17	1.42	1.34	1.96

Table 6 $\left|V^{tr}_{jy,\max}\right|$ in kN of TC2 for the three sites, M_w=7.5, e_{sxj}/b=0.0

Storey No.	**Site 1**	**Site 2**	**Site 3**
10	275.81	353.25	305.78
9	716.38	947.59	838.45
8	988.37	1379.37	1255.25
7	1130.37	1719.69	1576.88
6	1148.37	2010.39	1801.70
5	1127.69	2206.36	1965.76
4	1255.60	2330.92	2097.23
3	1450.81	2401.91	2226.85
2	1611.85	2492.16	2314.75
1	1681.84	2570.74	2334.40

Table 7 $\left|T_{\mathrm{jz,max}}\right|$ in kN-m of TC2 for the three sites, M_w=7.5

Storey No.	**e_{sxj}/b = 0.05**			**e_{sxj}/b = 0.10**			**e_{sxj}/b = 0.15**			**e_{sxj}/b = 0.20**		
	Site 1	**Site 2**	**Site 3**	**Site 1**	**Site 2**	**Site 3**	**Site 1**	**Site 2**	**Site 3**	**Site 1**	**Site 2**	**Site 3**
10	1382.9	1658.3	1496.0	1921.7	2378.4	2219.4	1996.5	2899.8	2423.6	2040.5	3251.1	2643.3
9	3526.5	4434.7	4042.3	4993.3	6314.5	6121.0	5119.4	7687.8	6625.0	5123.0	8485.3	7224.0
8	4882.6	6568.0	5850.3	6838.4	9197.4	9121.4	7189.1	11032.5	9930.1	6967.0	12216.0	10832.2
7	5392.8	8411.8	7239.5	7724.5	11641.0	11431.7	8231.1	13494.9	12661.1	8036.1	14670.0	13856.8
6	5259.6	10023.2	8218.0	7744.7	13799.6	13209.1	8600.8	15548.5	14878.6	8665.0	16617.8	16428.6
5	5195.3	11326.6	9112.5	7831.8	15456.2	14712.5	8929.1	17339.6	16759.9	9479.9	18358.0	18517.4
4	5881.4	12512.3	10154.1	9037.7	17081.8	16321.2	9623.3	19104.0	18652.8	10401.7	20237.7	20138.9
3	7019.9	13506.2	11232.3	10481.4	18659.1	17790.2	10629.6	20874.3	20206.8	11423.8	22499.7	21521.2
2	7958.2	14394.2	12053.5	11562.7	19933.3	18764.2	11414.5	22153.3	21267.8	12127.4	24494.7	22593.4
1	8344.3	14819.5	12415.7	11968.5	20650.0	19202.5	11670.6	22896.3	21792.4	12402.0	25619.5	23144.9

Table 8 R_{xj} of TC2 for the three sites, M_w=7.5

Storey No.	**e_{sxj}/b=0.05**			**e_{sxj}/b=0.10**			**e_{sxj}/b=0.15**			**e_{sxj}/b=0.20**		
	Site 1	**Site 2**	**Site 3**	**Site 1**	**Site 2**	**Site 3**	**Site 1**	**Site 2**	**Site 3**	**Site 1**	**Site 2**	**Site 3**
10	3.18	2.98	3.11	2.21	2.14	2.30	1.53	1.74	1.68	1.17	1.46	1.37
9	3.13	2.97	3.06	2.21	2.12	2.32	1.51	1.72	1.67	1.14	1.42	1.37
8	3.14	3.02	2.96	2.20	2.12	2.31	1.54	1.69	1.67	1.12	1.41	1.37
7	3.03	3.11	2.91	2.17	2.15	2.30	1.54	1.66	1.70	1.13	1.35	1.39
6	2.91	3.17	2.90	2.14	2.18	2.33	1.59	1.64	1.75	1.20	1.31	1.45
5	2.93	3.26	2.94	2.20	2.22	2.38	1.68	1.66	1.80	1.33	1.32	1.50
4	2.97	3.41	3.07	2.29	2.33	2.47	1.62	1.73	1.88	1.31	1.38	1.52
3	3.07	3.57	3.20	2.29	2.47	2.54	1.55	1.84	1.92	1.25	1.49	1.53
2	3.13	3.67	3.31	2.28	2.54	2.57	1.50	1.88	1.94	1.19	1.56	1.55
1	3.15	3.66	3.38	2.26	2.55	2.61	1.47	1.88	1.98	1.17	1.58	1.57

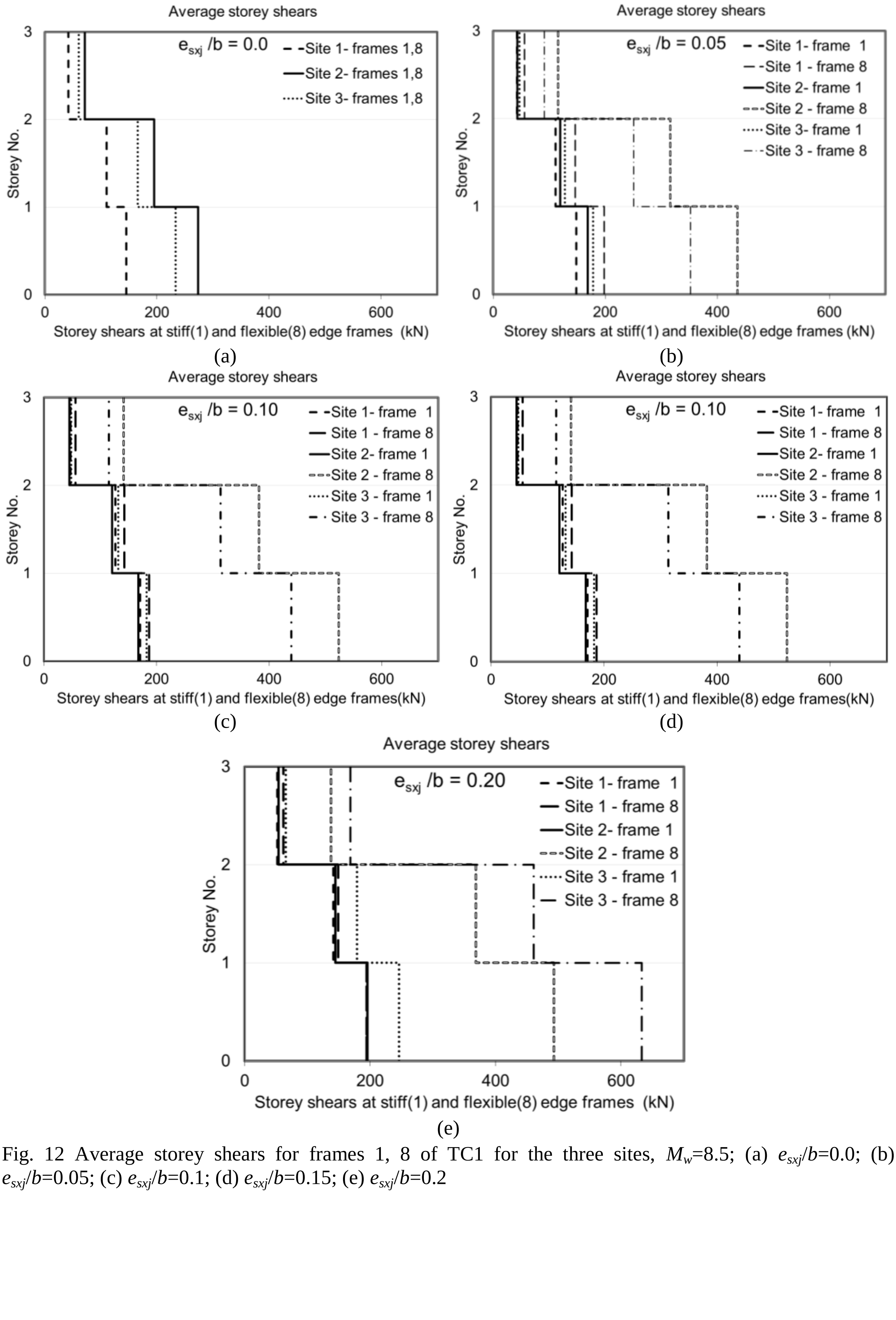

Fig. 12 Average storey shears for frames 1, 8 of TC1 for the three sites, M_w=8.5; (a) e_{sxj}/b=0.0; (b) e_{sxj}/b=0.05; (c) e_{sxj}/b=0.1; (d) e_{sxj}/b=0.15; (e) e_{sxj}/b=0.2

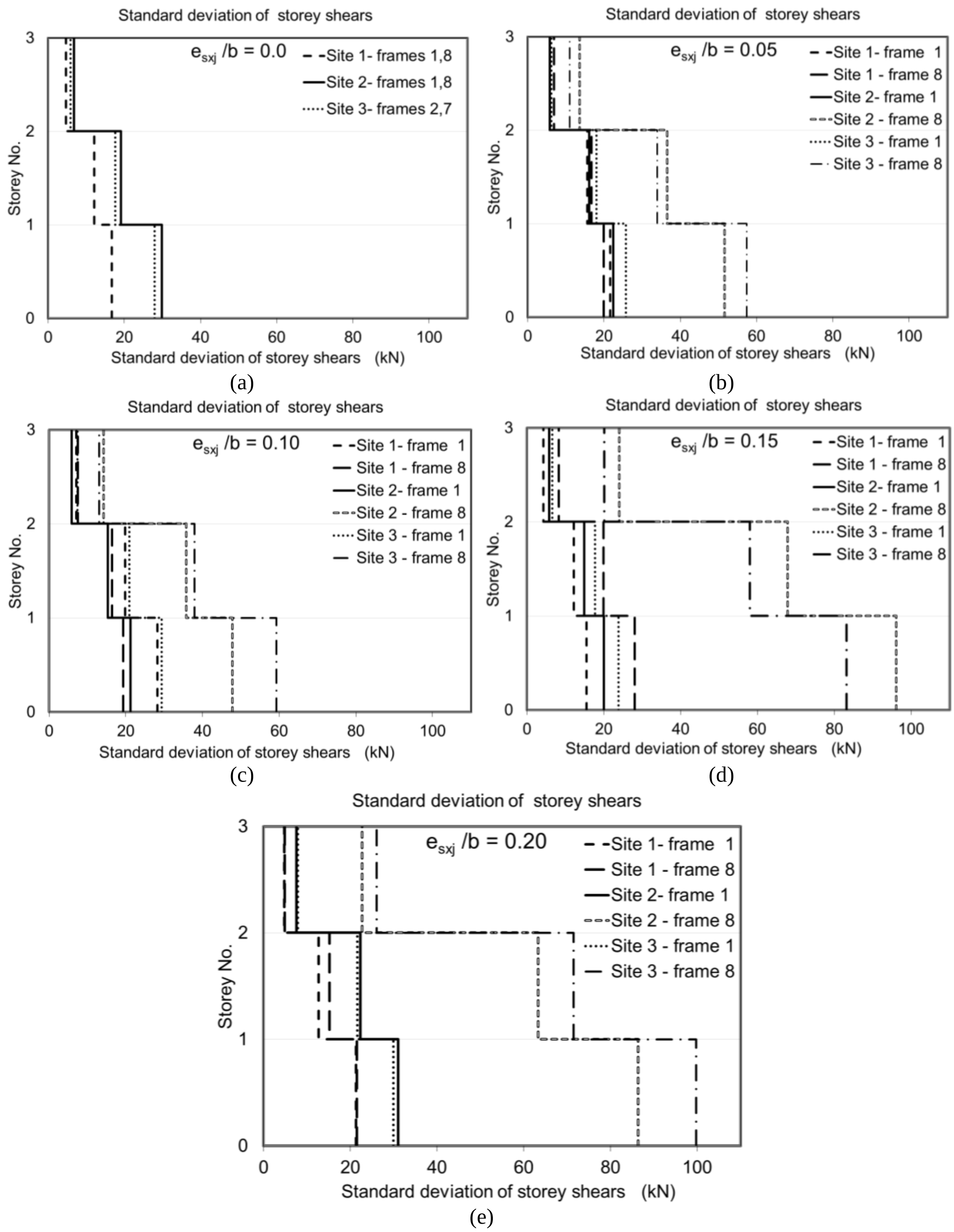

Fig. 13 Standard deviations of storey shears for frames 1, 8 of TC1 for the three sites, M_w=8.5; (a) e_{sxj}/b=0.0; (b) e_{sxj}/b=0.05; (c) e_{sxj}/b=0.1; (d) e_{sxj}/b=0.15; (e) e_{sxj}/b=0.2

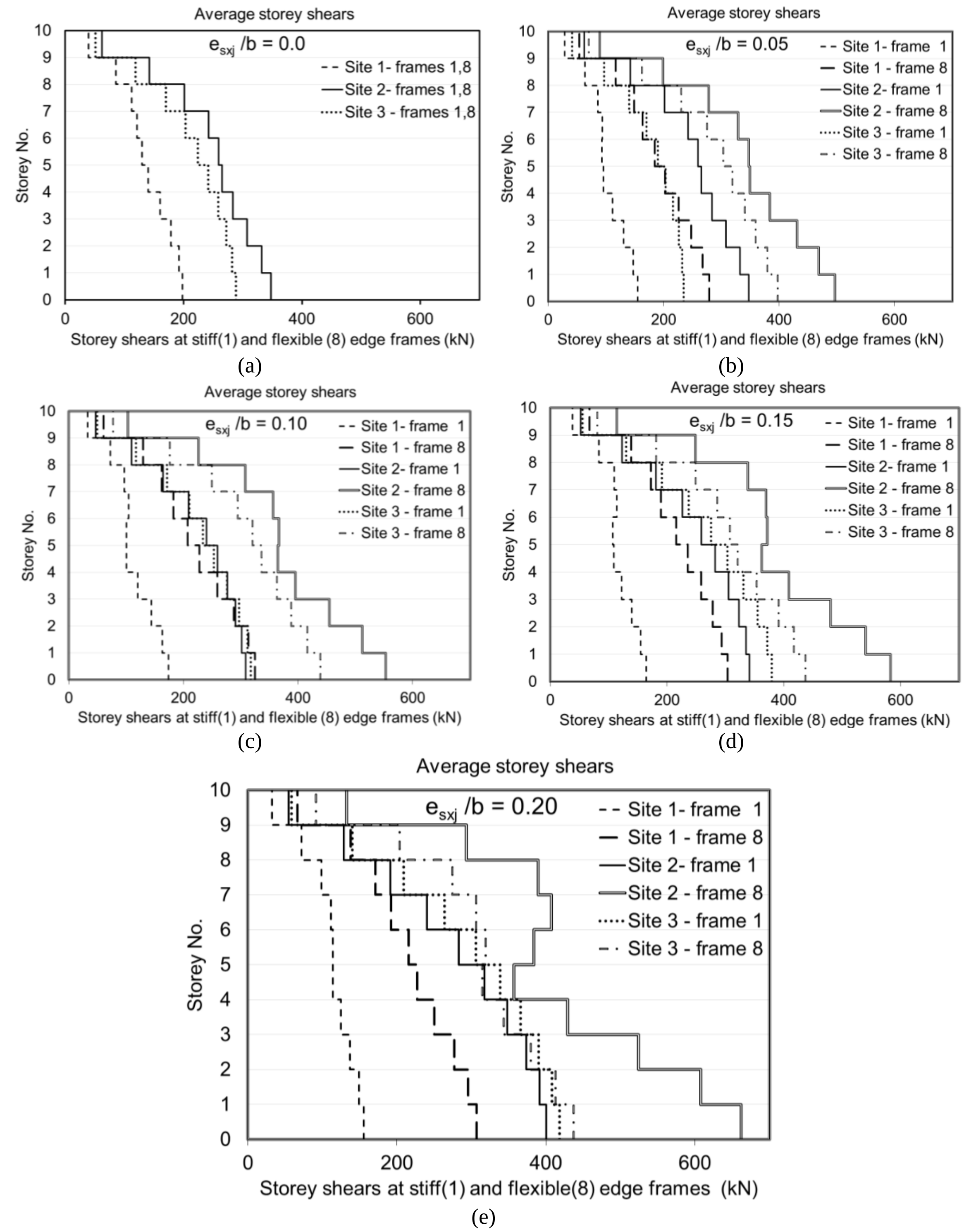

Fig. 14 Average storey shears for frames 1,8 of TC2 for the three sites, M_w=7.5; (a) e_{sxj}/b=0.0; (b) e_{sxj}/b=0.05; (c) e_{sxj}/b=0.1; (d) e_{sxj}/b=0.15; (e) e_{sxj}/b=0.2

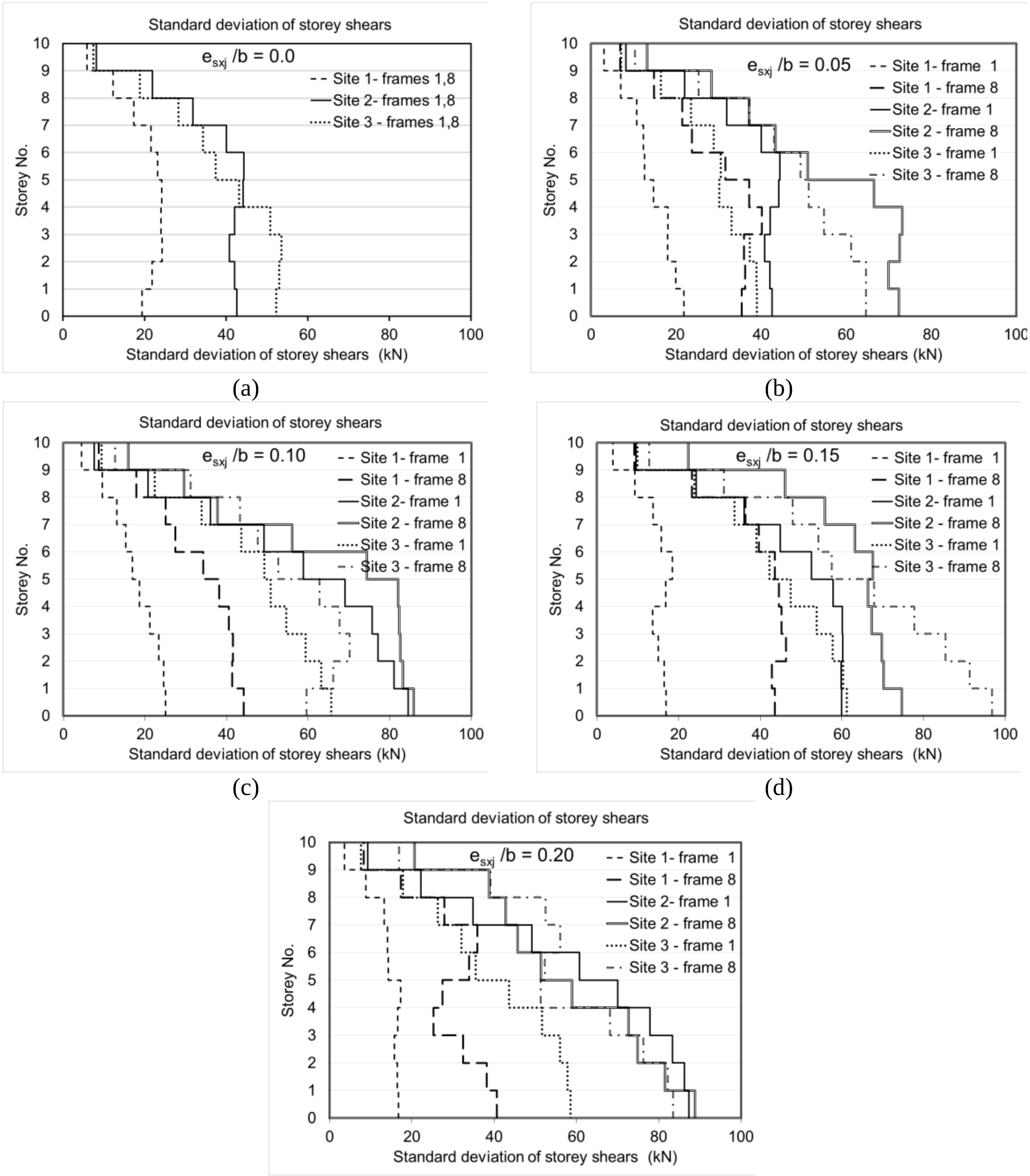

Fig. 15 Standard deviations of storey shears for frames 1,8 of TC2 for the three sites, M_w=7.5; (a) e_{sxj}/b=0.0; (b) e_{sxj}/b=0.05; (c) e_{sxj}/b=0.1; (d) e_{sxj}/b=0.15; (e) e_{sxj}/b=0.2

5. Conclusions

In this paper, site characteristics has been identified as a parameter that influences the dynamic to static eccentricity ratio of torsionally coupled buildings and a methodology to determine the dynamic to static eccentricity ratio for site-specific earthquake has been proposed. Proposed methodology has been demonstrated for two torsionally coupled buildings on three soil sites at Delhi, for scenario earthquakes of magnitude M_w=7.5 and M_w=8.5. From the results obtained it is observed that dynamic to static eccentricity ratio for the three sites for both the buildings and two earthquake scenarios are different from code specified values of 1.5. Difference between the dynamic to static eccentricity ratio for site-specific earthquake and code specified value is more for lower eccentricity ratios and the difference is less for higher eccentricity ratios. Variations in average and standard deviations of frame shears for the stiff and flexible edges of the buildings for the three sites are also studied for different eccentricities. It is seen that the variations of flexible edge are more compared to stiff edge for the three storey building. However, considerable variations are observed for both stiff and flexible edges for the ten storey building. From the limited studies made it is seen that dynamic to static eccentricity ratio provision in existing Indian seismic code of practice may not be adequate to include the effect of site-specific earthquake, for buildings having torsional to translational frequency ratio closer to one.

In existing Indian seismic code it is mentioned that in case, design spectrum is specifically prepared for a structure for a particular project site, the same may be used for design at the discretion of the project authorities. In a similar manner, a statement "in case, dynamic to static eccentricity ratio is obtained for a structure for a particular project site, the same may be used for computing design eccentricity at the discretion of the project authorities" may be added in Indian seismic code. The methodology proposed in this paper can be adopted to carry out site-specific analyses for torsionally coupled buildings for any chosen region.

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