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Equivalent lateral force method for buildings with setback: adequacy in elastic range

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Abstract. Static torsional provisions employing equivalent lateral force method (ELF) require that the earthquake-induced lateral force at each story be applied at a distance equal to design eccentricity (e_d) from a reference resistance centre of the corresponding story. Such code torsional provisions, albeit not explicitly stated, are generally believed to be applicable to the regularly asymmetric buildings. Examined herein is the applicability of such code-torsional provisions to buildings with set-back using rigid as well as flexible diaphragm model. Response of a number of set-back systems computed through ELF with static torsional provisions is compared to that by response spectrum based procedure. Influence of infill wall with a range of opening is also investigated. Results of comprehensive parametric studies suggest that the ELF may, with rational engineering judgment, be used for practical purposes taking some care of the surroundings of the setback for stiff systems in particular.

Keywords: irregular; torsion; seismic; code-provisions; elastic

1. Introduction

Geometry of the structure is often dictated by the architectural and functional requirements whereas the safety of the structure with optimum economy - the key design aim - is ensured by structural engineers. For instance, a stepped form (setback systems) of buildings is often adopted by the architects for adequate daylight and ventilation in the lower stories of the buildings in an urban locality where closely spaced tall buildings are expected. Such setback structures form an important sub-class of irregular structures wherein irregularities are characterized by discontinuities in the distribution of mass, stiffness and strength along the height of the building.

Research progress for systems with irregularity in elevation is scarce primarily owing to the relative difficulty to characterize such systems (Kusumastuti *et al*. 1998). Studies (e.g., Humar and Wright 1977, Aranda 1984, Moehle and Alarcon 1986) up to mid-1980 on seismic response and relevant code provisions of systems with symmetric setback have been reviewed in the literature (Wood 1986). A simple definition to measure irregularity of such systems has been proposed and used in the recent works (Mazzolani and Piluso 1996, Karavasilis *et al.* 2008). Simplified method

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to obtain lateral load distribution in symmetric and eccentric set-back systems has been developed using the concept of compatible load profile (Cheung and Tso 1987). Illustrations therein demonstrate the possibility of higher damage potential in members near the set-back. Subsequent analytical and experimental studies (Shahrooz and Moehle 1990) also corroborate such observation. It is reported elsewhere (Wood 1992, Pinto and Costa 1995, Mazzolani and Piluso 1996, Kappos and Scott 1998, Romao *et al.* 2004) that the seismic response of setback systems is not significantly different from regular systems. While the effectiveness of the first mode of vibration to represent displacement response is observed in some study (Wong and Tso 1994), significant participation of higher modes is also noted elsewhere (Athanassiadou 2008, Karavasilis *et al*. 2008). The relative vulnerability associated to mass, strength and stiffness irregularities is examined in the literature (Al-Ali and Krawinkler 1998). Thus, contradictions exist and the progress in understanding seismic behavior of set-back buildings is rather slow. Although relatively simple method for the analysis of setback buildings is pursued (Basu and Gopalakrishnan 2008), major building codes (IS 1893-1984 2002, ASCE 7 2005, Eurocode 8 2004), to date, recommend for dynamic analysis for the design of setback buildings. The codes further recommend that the base shear obtained from the dynamic analysis (and thereby, other response quantities) to be scaled up to that from the code specified empirical formula.

Seismic codes permit equivalent static procedure (ELF) usually for regular buildings. In equivalent static analysis, the design base shear is estimated as a product of seismic weight and codified seismic coefficient associated to fundamental period of vibration. Such seismic coefficient takes into account the importance and ductility capacity of the structure as well as the type of soil and seismic activity of the region. For asymmetric system, building codes (e.g., IAEE 1997) specify that the earthquake-induced lateral force so computed be statically applied with an eccentricity equal to design eccentricity (e_d) relative to some reference center of resistance. Such design eccentricities are outlined in the forms of primary design eccentricity, e_{d1j} and secondary design eccentricity, e_{d2j}, at any typical j-th story, as given below

$$\begin{aligned} e_{d1j} &= \alpha\, e_j + \beta D \\ e_{d2j} &= \delta\, e_j - \beta D \end{aligned} \tag{1}$$

where D is the plan dimension of the building normal to the direction of ground motion and e_j is the static eccentricity at j^{th} story. The first part is a function of static eccentricity - real distance between center of mass and center of resistance. Dynamic amplification factor α in e_{d1} is intended to compensate for the dynamic effect of torsional response through static analysis. Factor δ in e_{d2} specifies the portion of the torsion-induced so-called negative shear that can be reduced for the design of stiff-side elements. The second part, referred to as accidental eccentricity, is expressed as a fraction of plan dimension, i.e., βD (normal to the direction of ground motion and is introduced to account for the imponderables). For each element, the value of e_d yielding greater force should be used in design.

However, a lack of unanimously acceptable definition of reference centre of resistance for multistory buildings often appears to be a major setback to implement such static procedure. A search for proper resistance centre reveals a number of alternatives (e.g., Poole 1977, Humar 1984, Riddell and Vasquez 1984, Smith and Vezina 1985, Cheung and Tso 1986, Hejal and Chopra 1987, Tso 1990, Goel and Chopra 1993, Jiang *et al*. 1993, Makarios and Anastassiadis 1998). Such alternative reference centres, despite being placed at differing locations, often lead to similar response (Harasimowicz and Goel 1998). This observation fundamentally implies that the traditional notion of applicability of code-torsional provisions to regularly asymmetric systems (where centre of mass and centre of resistance are aligned along two vertical lines separated by a constant distance) may be over-restrictive. Limited studies (Das and Nau 2003, Tremblay and Poncet 2005) covering systems with mass and some specific form of vertical irregularity, in fact, suggest the conservativeness of code-imposed limitation to ELF.

With this backdrop, the goal of the present investigation is set to explore the applicability of equivalent lateral force method (ELF) to buildings with setback where resistance centres may dramatically vary storey-wise and thus to avoid the complexities of the dynamic analysis recommended for these systems. In this context, buildings are modeled as rigid diaphragm system in general. Moreover, the influence of floor flexibility is also examined and compared.

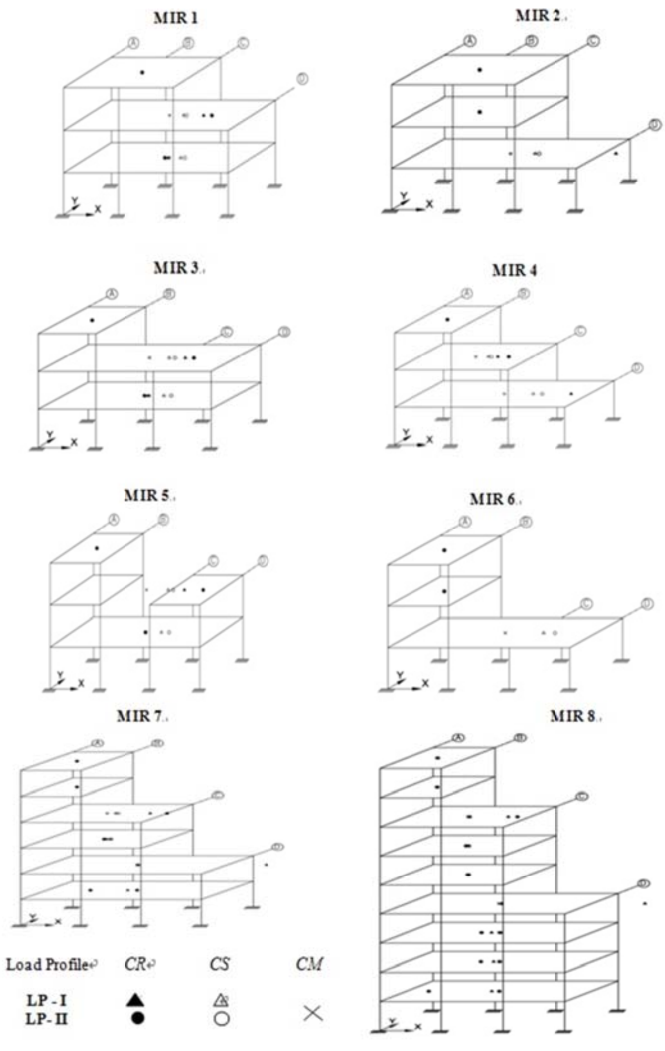


Fig. 1 Configuration of structural models showing centre of mass (CM), centre of rigidity (CR) and shear centre (CS)

Table 1 Dynamic characteristics of buildings with associated irregularity indices

Sl. No.	Maximum no. of story	Model Identification	Irregularity Index			Dynamic characteristics					
						Mode 1		Mode 2		Mode 3	
			Φ_b	Φ_s	$\Phi_{avg.}$	T (sec.)	Γ	T (sec.)	Γ	T (sec.)	Γ
1	3	M-IR1	1.25	1.25	1.25	0.367	0.858	0.244	0.057	0.135	0.068
2	3	M-IR2	2.00	1.25	1.63	0.362	0.828	0.217	0.028	0.140	0.136
3	3	M-IR3	1.25	2.00	1.63	0.333	0.814	0.234	0.118	0.122	0.050
4	3	M-IR4	1.75	1.75	1.75	0.326	0.770	0.183	0.161	0.121	0.055
5	3	M-IR5	1.75	1.75	1.75	0.303	0.727	0.218	0.207	0.120	0.057
6	3	M-IR6	2.00	2.00	2.00	0.306	0.701	0.158	0.217	0.134	0.079
7	6	M-IR7	1.75	1.30	1.53	0.623	0.708	0.562	0.167	0.326	0.058
8	9	M-IR8	1.52	1.19	1.36	0.957	0.706	0.860	0.144	0.529	0.076

*Γ represents participating mass ratio for excitation in Y-direction (Refer to Fig. 1) and
EI for all columns = $8.54 \times 10^7\ Nm^2$

Table 2 Eccentricities (distance between CM and shear centre in metre) in representative setback buildings with and without floor flexibility corresponding to LP-I

Sr. No.	No of story	M IR 4		MIR 7		MIR 8	
		Rigid	Flexible	Rigid	Flexible	Rigid	Flexible
1	St-1	2.54	2.50	2.66*2	2.65	2.13	2.11
2	St-2	1.10	1.01	3.00	2.92	2.23	2.18
3	St-3	0.00*1	-0.06	1.13	1.04	2.44	2.32
4	St-4	-	-	1.52	1.45	2.87	2.41
5	St-5	-	-	0.01	-0.06*3	0.84	0.87
6	St-6	-	-	0.01	-0.01	1.05	0.98
7	St-7	-	-	-	-	1.48	1.27
8	St-8	-	-	-	-	0.02	-0.06
9	St-9	-	-	-	-	0.05	-0.03

*1At CM; *2To the right of CM; *3To the left of CM

Table 3 Uncoupled dynamic characteristics of fundamental mode of vibration

Sl. No.	Model Identification	Type of Diaphragm	Irregularity Index		Mode 1			Mode 2			Mode 3		
			Φ_b	Φ_s	T_L (sec.)	T_θ (sec.)	T_θ/T_L	T_L (sec.)	T_θ (sec.)	T_θ/T_L	T_L (sec.)	T_θ (sec.)	T_θ/T_L
1	MIR 4	Rigid	1.75	1.75	0.309	0.177	0.572	0.136	0.096	0.705	0.098	0.069	0.706
		Flexible			0.314	0.266	0.848	0.227	0.153	0.674	0.139	0.091	0.650
2	MIR 7	Rigid	1.75	1.30	0.574	0.311	0.542	0.240	0.160	0.667	0.221	0.110	0.497
		Flexible			0.575	0.350	0.609	0.243	0.242	0.999	0.227	0.226	0.996
3	MIR 8	Rigid	1.52	1.19	0.892	0.506	0.567	0.353	0.236	0.669	0.221	0.136	0.616
		Flexible			0.893	0.533	0.597	0.355	0.289	0.814	0.227	0.254	1.117

*1At CM; *2To the right of CM; *3To the left of CM

2. Details of structural systems

Three, six and nine story systems are considered as representatives of low, medium and high-rise buildings. Three story models (annotated as MIR 1 through MIR 6) with different feasible forms of set-back are considered. Further, medium and high-rise buildings are examined in the sample form considering one six story (MIR 7) and one nine story system (MIR 8). Such systems are schematically presented in Fig. 1. Irregularity Indices (Φ_b, Φ_s) of the systems proposed and utilized elsewhere (Mazzolani and Piluso 1996, Karavasilis *et al*. 2008, Sarkar *et al*. 2010, Mahato *et al*. 2012), are computed as follows andfurnished in Table 1 to recognize the nature of elevation irregularity.

$$\Phi_b = \frac{1}{n_b - 1}\sum_{i=1}^{i=n_b-1}\frac{H_i}{H_{i+1}} \text{ and } \Phi_s = \frac{1}{n_s - 1}\sum_{i=1}^{i=n_s-1}\frac{L_i}{L_{i+1}} \tag{2}$$

where n_s is the number of story, n_b is the number of bay in the first story, L is length of bay and H is height of story. L and H are chosen as 5.0m and 3.5m unless otherwise specified. Length of the bay in the direction normal to set-back is also kept equal to 5.0m.

Location of centre of resistance varies as per different definitions and is also known to be dependent on the distribution of lateral load. Height-wise distribution of lateral load is assumed as $\frac{w_i H_i^k}{\sum_{i=1}^{n_s} w_i H_i^k}$ where w_i and H_i are the weight and height of ith story and k is an exponent. Values of k are chosen as 1.0 and 2.0 in load profile LP-I and LP-II, respectively. Generalized centre of rigidity (CR) and shear centre (CS) in each story as defined in the literature (Tso 1990) are presented in Fig. 1. Such centres are computed assuming the floor diaphragm as rigid. However, CS is also computed for typical low, medium and high rise buildings accounting floor flexibility as outlined in the literature (Basu and Jain 2004). Such study considers thickness of the floor slab as 150 mm and a height-wise distribution of lateral load conforming to LP-I. It will be apparent in the following sections that such reference points are computed only to gauge relative irregularity of the buildings and bear no relevance to implement code-static procedure.

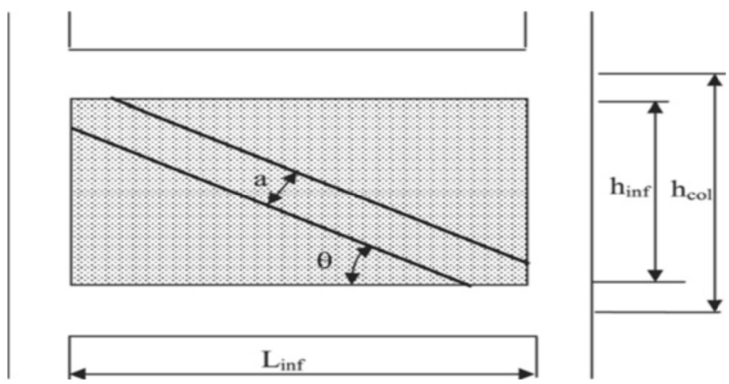


Fig. 2 Equivalent diagonal compressive strut model to represent infill walls under lateral load (extracted from Kose 2009)

2.1 Modeling of infill wall

Buildings are usually analyzed as bare frames in practice. However, lateral force induced shear causes in-plane lateral deformation in the infill wall. Such mode of deformation tends to elongate one diagonal and shorten the other of each panel of a building frame. However, the brick infill within the panel resists against the shortening of the diagonals only. Thus the effect of infill wall, in the linear elastic range, may be modeled using truss member connected to beam-column joints through hinges. Such "equivalent strut" (Smith 1962, Smith and Carter 1969, Mainstone and Weeks 1970, Mainstone 1971) is introduced along one diagonal only with similar attributes in both tension and compression. This, from the view point of mechanics, is analogous to the inclusion of two 'compression only' truss member along two diagonals of the panel in linear elastic range. The effective width (a) of such equivalent struts having actual diagonal length (r_{inf}) and wall thickness (t_{inf}) is determined following the recommendation given in FEMA 306 (FEMA 306 1998). The equivalent width of a diagonal compressive strut, a, is given by

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{\inf} \tag{3}$$

where, $\lambda_1 = \left[\frac{E_{me} t_{\inf} \sin 2\theta}{4 E_{fe} I_{col} h_{\inf}}\right]^{\frac{1}{4}}$ in which $\theta = \tan^{-1}\left(\frac{h_{\inf}}{L_{\inf}}\right)$, $h_{col.}$ and $I_{col.}$ respectively stand for centre to centre height and moment of inertia of column (m^4); h_{inf} and L_{inf} represent height and length of infill wall (also refer to Fig. 2). Modulus of elasticity of infill wall (E_{me}) and the modulus of elasticity of frame elements (E_{fe}) are assumed as 6300 MPa and 25,000 MPa respectively. Thicknesses of outer and inner infill wall are taken as 230 mm and 115 mm respectively.

To account for the effect of opening due to doors and windows, width of the compressive struts so estimated is modified by stiffness reduction co-efficient $\lambda_{graphic}$ as outlined in the literature (Asterris 2003, Kose 2009). In the parametric study, values of such opening percentage are taken as 0, 10, 25, 50 and 100 respectively. It may be mentioned that the case with 100% opening represents popularly used bare frames, while the first one (0% opening) corresponds to no opening at all. Representative systems (MIR-4, MIR-7 and MIR-8) with three, six and nine stories are analyzed to realize the impact of infill.

2.2 Dynamic characteristics

Free vibration characteristics of bare frames modeled as rigid diaphragm are presented in Table 1. Natural periods, mode shapes are computed corresponding to translational (Y) and torsional (rotation about Z) degrees of freedom. The participating mass ratio (Γ), defined for n[th] mode as $\frac{(f_{yn})^2}{M_y}$, is also computed. f_{yn} ($f_{yn} = \varphi_n{}^T m_y$) is the participation factor where m_y is the load corresponding to unit acceleration and M_y is the total unrestrained mass in Y-direction. The mode shapes (φ) are normalized such that $\varphi_n^T M \varphi_n = 1$ in which M is the global mass matrix (SAP 2000, Sarkar *et al*. 2010). For torsionally coupled systems, relative proximity of the uncoupled lateral and torsional periods of the systems is known to be a useful indicator of torsional vulnerability. Thus, uncoupled lateral (T_L) and torsional (T_θ) periods are also computed for the systems chosen. Uncoupled fundamental lateral periods are computed by standard eigen-value analysis constraining the stories to translate in Y-direction only. To assess uncoupled torsional periods, mass moment of inertia at each floor is specified only (with no translational mass). Subsequent eigen-value analysis leads to uncoupled torsional mode of vibration about some torsion axis depending on the relative distribution of mass in various stories. Relative proximity of such uncoupled torsional to lateral periods, quantified as $\tau = T_\theta/T_L$, appears indicative of the likely coupling between lateral and torsional modes of the systems. Such quantities, for representative cases, are presented in Table 3 for both rigid and flexible floor (slab thickness: 150 mm) systems. It is observed that the systems chosen are torsionally stiff (τ <1.0) and hence code-torsional provisions may be relevant. Low torsional stiffness in an asymmetric building causes the rotational modes to have a more important role in the deformations of the elements. The corresponding change in dynamics of torsionally flexible (τ >1.0) buildings is such that the pattern of seismic demand in the elements is not in agreement with the strength distribution suggested by static torsional provisions (Mogadham and Tso 2000). A careful scrutiny reveals that the influence of floor flexibility may increase the lateral period marginally. However, the corresponding increase of torsional period may be as high as 50% particularly for low-rise systems. Thus, the parameter τ may significantly increase (about 48% in MIR-4) and hence may alter the seismic behaviour of coupled systems (refer to Table 3).

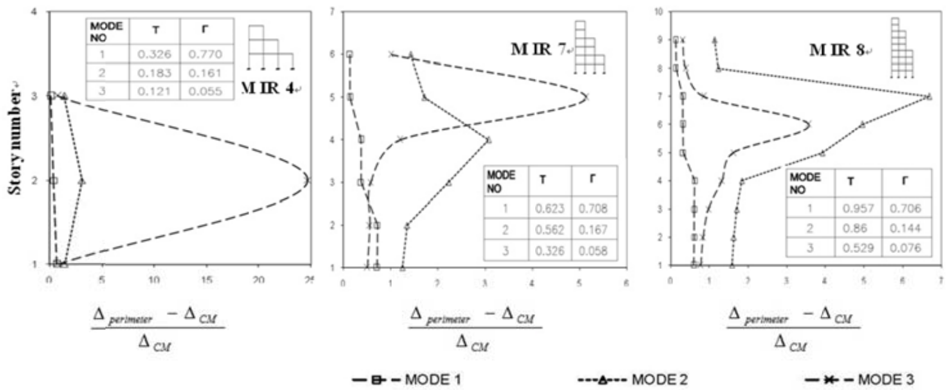


Fig. 3(a) Torsional to lateral coupling in different participating modes of vibration of sample building model with rigid diaphragm

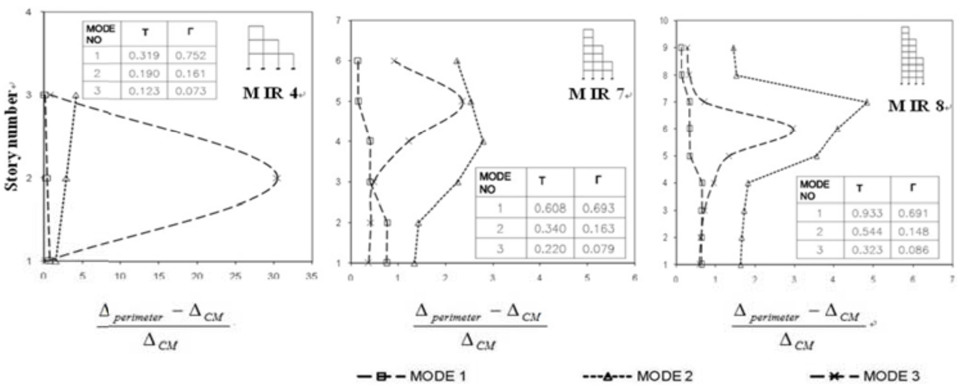


Fig. 3(b) Torsional to lateral coupling in different participating modes of vibration of sample building model with flexible diaphragm (150 mm thk floor slab)

To achieve further insight into the mode coupling phenomenon, $\left|\frac{\Delta_p}{\Delta_c}-1\right|$, where of Δ_p and Δ_c are displacements of the perimeter frame and centroid of the deck respectively, is graphically presented in Fig. 3 along with the coupled natural periods and participating mass ratio in each mode. Δ_p is recorded on the edge where translational and torsional displacements are additive. Fig. 3 shows that, for fundamental mode of vibration, influence of torsion relative to translation is subdued. Dominance of translational vibration in the first coupled mode is also confirmed from associated Γ (in the range of 70% to 86%) and hence the systems are torsioanlly stiff. It may further be noticed that, although both the coupled periods and corresponding Γ closely remain stable, order of coupling in higher modes may potentially change due to floor flexibility.

It seems apparent from a thoughtful observation to Table 1 that, as arithmetic average of Φ_b and Φ_s decreases implying a tendency towards regularity in configuration, contribution of torsion dominated second mode usually diminishes and the participation of the translation dominated fundamental mode increases. Thus, the simple irregularity index ($\Phi_{b,}$ Φ_s) appears to be compatible with the important dynamic characteristics of the systems at least qualitatively. In this context, it may also be interesting to asses fundamental building period using codified formula such as $T_l = 0.0731h^{3/4}$ (UBC 1997), where h is the overall height of the building. Fundamental period of buildings without infill is evaluated as 0.43 sec, 0.72 sec. and 0.92 sec. for three, six and nine story systems respectively. This shows that the building period of this class of low to medium-rise systems may generally be shorter than what by code-specified empirical formula. As further evidence, an authoritative study (Goel and Chopra 1997) developing formula to estimate fundamental period of vibration of moment resisting frames may be referred. Such investigation, on the basis of 'measured' data on vibration period of a large number of buildings during real earthquakes, identified the similar limitation of empirical formula outlined in the code. Inadequacy of code-based empirical formulae is also pointed out in another illuminating study (Harasimowicz and Goel 1998). Thus, the codified formula for building periods need be re-evaluated since a higher estimate of period may often result in underestimating the design force.

Dynamic characteristics of buildings with infill (50% opening assumed) are assessed and compared to those of the bare frames using rigid diaphragm model. Results, though not presented herein (but available in Mahato 2012), show that fundamental period reduces by around 20% due to the stiffening effect of infill wall for low to high rise systems. Systems are, however, observed to be torsionally stiff and hence the application of code-torsional provisions may be warranted.

3. Method of analysis

Response of the structures excited in Y-direction is first calculated by equivalent lateral force (ELF) method. To this end, fundamental period of the system is estimated employing empirical formula outlined in the code (UBC 1997). Subsequently, base shear (V_0) is computed through multiplying the relevant spectral ordinate by seismic weight (refer to Table 4). Zone factor Z is assumed as 0.2, while seismic co-efficients C_a and C_v are chosen as 0.24 and 0.32 respectively. Considering occupancy importance factor as unity and response reduction factor for ordinary moment resisting frames (OMRF) as 3.5, design base shear is computed as per relevant guideline of UBC 97 (UBC 1997). Design base shear so calculated is distributed over the building height

Table 4 Basic seismic design parameters

Sl. No.	Maximum no. of story	Model Identification	Seismic weight (kN)						Design base shear (kN) [U BC 97]
			Story 1	Story 2	Story 3	Story 4	Story 5 to 7	Story 8 & 9	
1	3	M-IR1	1350	1350	900	-	-	-	620
2		M-IR2	1350	900	900	-	-	-	540
3		M-IR3	1350	1350	450	-	-	-	540
4		M-IR4	1350	900	450	-	-	-	465
5		M-IR5	1350	900	450	-	-	-	465
6		M-IR6	1350	450	450	-	-	-	385
7	6	M-IR7	1350	1350	900	900	450	-	690
8	9	M-IR8	1350	1350	1350	1350	900	450	850

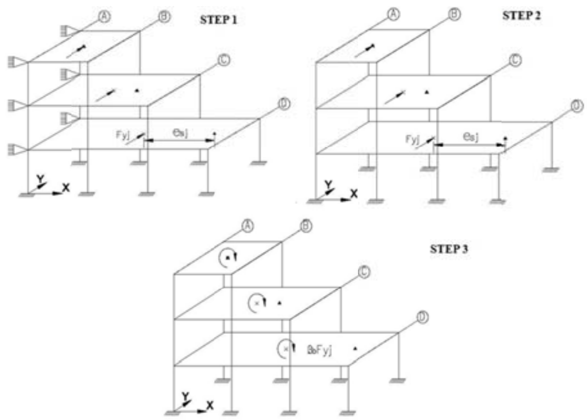
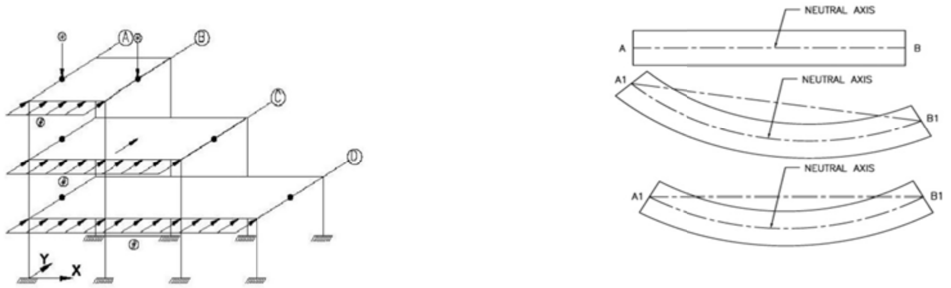


Fig. 4(a) Different Steps of analysis for ELF method without locating center of resistance (after Goel and Chopra 1993)



(i)

(ii)

(iii)

*CENTRAL NODES OF BOTH ENDS OF THE DIAPHRAGMS ARE CONSTRAINED TO ENSURE EQUAL HORIZONTAL DISPLACEMENT

LATERAL LOAD PROPORTION TO THE MASS DISTRIBUTION ALONG THE FLOOR LENGTH

No-torsion condition in buildings with flexible floor diaphragm:
(i) Un-deformed floor diaphragm;
(ii) Deflected shape of floor slab under in-plane loading with torsion;
(iii) Deflected shape of floor slab under in-plane loading without torsion.

Fig. 4(b) Procedure for analysis in flexible diaphragm system (after Basu and Jain 2004) according to LP-I and LP-II. Following major seismic codes, three combinations of α and δ are chosen. Static lateral load analysis is conducted utilizing the procedure developed elsewhere (Goel and Chopra 1993, Basu and Jain 2004) and is summarized below for convenience.

ELF is implemented by combining the results of three sets of analyses performed through standard frame analysis software (ETABS; SAP 2000) as described below.

Step 1: The asymmetric buildings are restricted to deform only in the Y-direction by constraining the floor rotations. Such restriction is ensured by introducing hinges at each story in case of rigid diaphragm system (refer to Fig.4(a)). On the other hand, for flexible floor system, since the floor can translate, bend and twist under lateral load, 'no-torsional rotation of floor' is redefined as identical horizontal displacement of centre nodes of both ends of the diaphragm (refer to Fig. 4(b)). This condition is achieved by setting equal constraints (SAP 2000) in Y-translation to centre nodes of both ends of each floor. Systems so modeled are analyzed with the code-specified lateral forces applied at the floor CM for rigid diaphragm system and as a distributed force (proportional to mass distribution) for flexible floor system. The response quantities of such restrained systems are denoted as R_r. It is evident that the procedure outlined for flexible floor model is generic and may also be applied to rigid floor system.

Step 2: Buildings modeled as three-dimensional frame are then analyzed. Code-specified lateral forces are applied as stated in Step 1 to compute the corresponding response R_0.

Step 3: Buildings are re-analyzed for the code-specified floor torques equal to $\beta D_j F_{yj}$ to obtain R_{ac}, i.e., the contribution of accidental eccentricity on the desired response (F_{yj} is the lateral load in the i^{th} story). In flexible floor model, such floor torque is simulated by application of a compatible lateral load (refer to Basu and Jain 2004).

Finally, the responses $R_d^{(1)}$ and $R_d^{(2)}$ are obtained by combining R_r, R_0 and R_{ac} as follows

$$R_d^{(1)} = (1-\alpha)R_r + \alpha R_0 \pm R_{ac} \tag{4a}$$

$$R_d^{(2)} = (1-\delta)R_r + \delta R_0 \pm R_{ac} \tag{4b}$$

The algebraic sign of R_{ac} should be the one that increases the magnitude obtained from the sum of the first two terms. The design value of the desired response is taken as the larger of two obtained from $R_d^{(1)}$ and $R_d^{(2)}$. In case of restriction to reduce the response due to torsion-induced negative shear, the design value is the highest of $R_d^{(1)}$, $R_d^{(2)}$ and R_r.

The above approach is preferred in view of (a) the variability of the location of resistance centres with the distribution of lateral load and (b) the difference in torsional response due to floor forces applied at CR and story shears acting at CS for setback buildings (with unequal deck dimensions) when accidental eccentricity is accounted (Basu and Jain 2006). Simultaneously, responses of all the buildings are computed by dynamic response spectrum analysis (using design spectrum of UBC 97) combining modal responses by complete quadratic combination (CQC) (Chopra 2007). Adequate numbers of modes are considered so that at least 95 percent of the total seismic mass is captured. Following codal recommendation, response quantities obtained from dynamic analysis is scaled by a factor equal to V_0/V_{dyna} where V_{dyna} is the base shear from dynamic analysis. Thus, the trend in results presented herein is generic and does not depend on the choice of code and other related factors such as Z, C_a and C_v etc.

4. Results and discussions

4.1 Rigid floor system

Maximum response in terms of frame shear, maximum inter-story drift is computed through ELF employing lateral load conforming to both LP-I and LP-II. Three sets of α and δ combinations, *viz.*, α = 1.0, δ = 0.5 (NBCC 1990); α = 1.5, δ =1.0 (IS 1893-1984 (2002, Mexico 1990) and α = 1.0, δ =1.0 (NZS 4203 19984) are used. Such response is normalized by the companion quantities obtained from response spectrum based analysis and is presented through Fig. 5 to Fig.10 for rigid floor systems.

Fig. 5(a) presents the height-wise variation of normalized frames shear in the perimeter frames (as the effect of torsion is maximum in the edge) of three story buildings corresponding to the distribution of design base shear as per LP-I (in ELF). Response of flexible side considering α = 1.0 (NZS 4203 19984) is observed to consistently underestimate the response. However, it appears that the response of the flexible side may often be reasonably predicted by taking α = 1.5, although such response may be somewhat underestimated in the higher stories near the set-back in particular. Such concentration of force in the upper story elements in the surroundings of the set-back indicates significant participation of higher modes. This observation is in line with those of a few earlier works (e.g., Cheung and Tso 1987, Shahrooz and Moehle 1990). Response of stiff side may, however, be estimated with an error limit of around ±22-25% for the values of δ specified in the the codes. Results of MIR 7 and MIR 8 presented in Fig. 5(b) displays a similar trend. Fig. 6, on the other hand, describing representative results corresponding to a load profile compatible with LP-II (in ELF), substantially overestimate the response particularly in higher stories. It may be recalled that the value of the exponent k involved in the definition of load distribution has been recommended as unity (as chosen in LP-I) in IBC 2003 (IBC 2003) for buildings with fundamental period lesser than 0.5 sec. Thus, such distribution (LP-I) appears to be useful also for setback buildings and is adopted in rest of the study along with the values of α and δ as 1.5 and 0.5 respectively (unless otherwise specified)

Fig. 7(a) describes the variation of similar response parameter as a function of change of bay length, while such response with change of story heights is presented in Fig. 7(b). Bay length is considered to vary in the range of 4.0m to 6.0m whereas the story height is ranging between 3.0 m to 5.0 m to cover the practical range of interest. This includes a panel aspect ratio of 0.58 to 1.0. Values of α and δ are assumed as 1.5 and 0.5 respectively. It is observed that the variation of normalized response is relatively insensitive to the aspect ratio of panel excepting in the neighbourhood of 0.7.

Infill wall is observed to substantially alter the dynamic characteristics of the system. Thus, the performance of ELF is re-examined considering the effect of infill wall. Influence of opening due to doors and windows is taken into account through considering an opening of 0%, 10%, 25%, 50 % and 100% in the infill wall. Normalized frame shear in perimeter frames at different stories of MIR 4, MIR 7 and MIR 8 are presented in Fig. 8 for a height-wise distribution of lateral load as per LP-I and LP-II, respectively. It seems that beyond 25% to 30% of opening, influence of infill wall on the response of flexible side may be marginal. Stiff side, however, does not show any systematic trend.

It may be stated that, while frame shear may be used directly in design of frame elements, maximum inter-story drift may also be useful to envisage the seismic performance for both

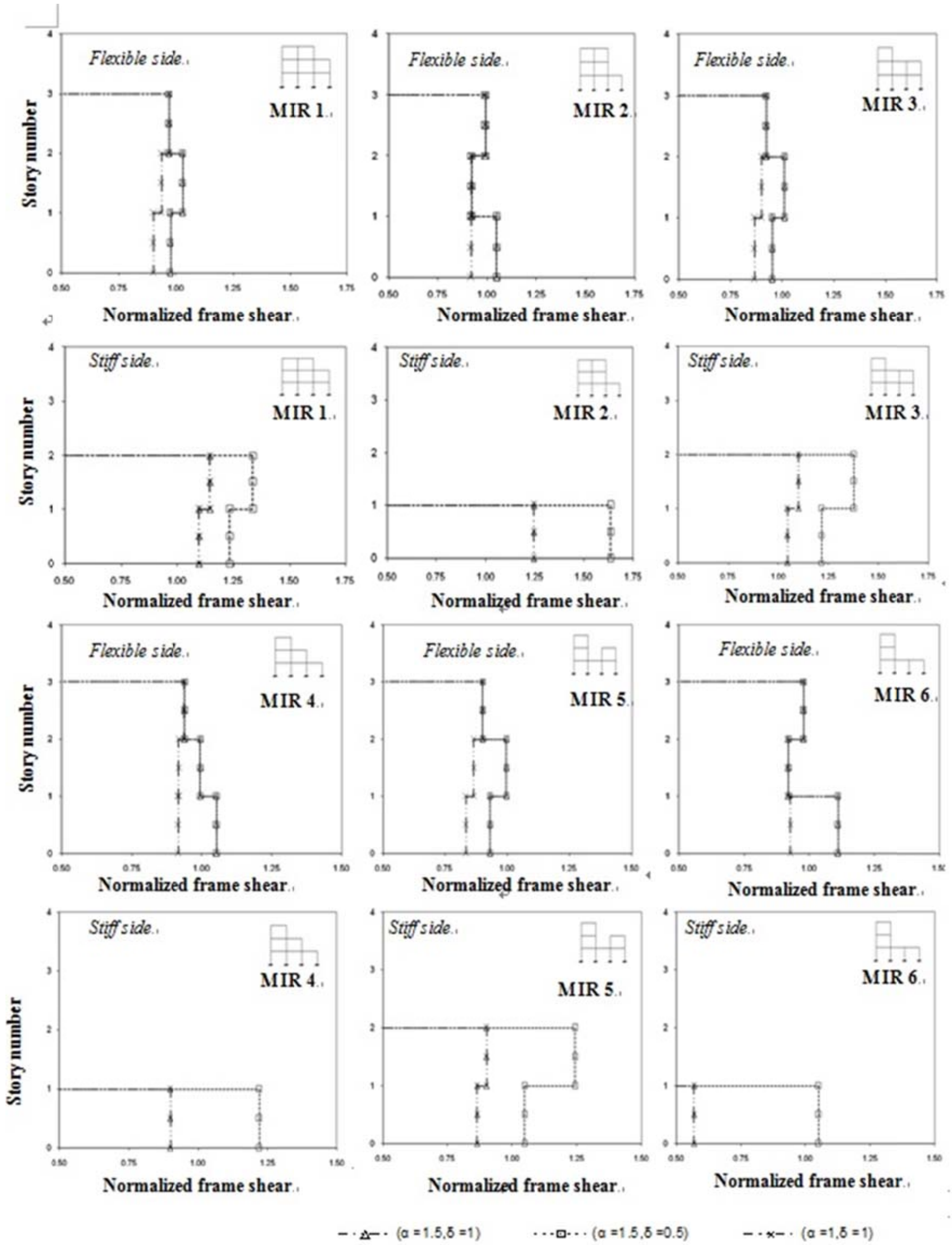


Fig. 5(a) Variation of normalized frame shear in perimeter frames of low-rise buildings (lateral load profile: LP-I)

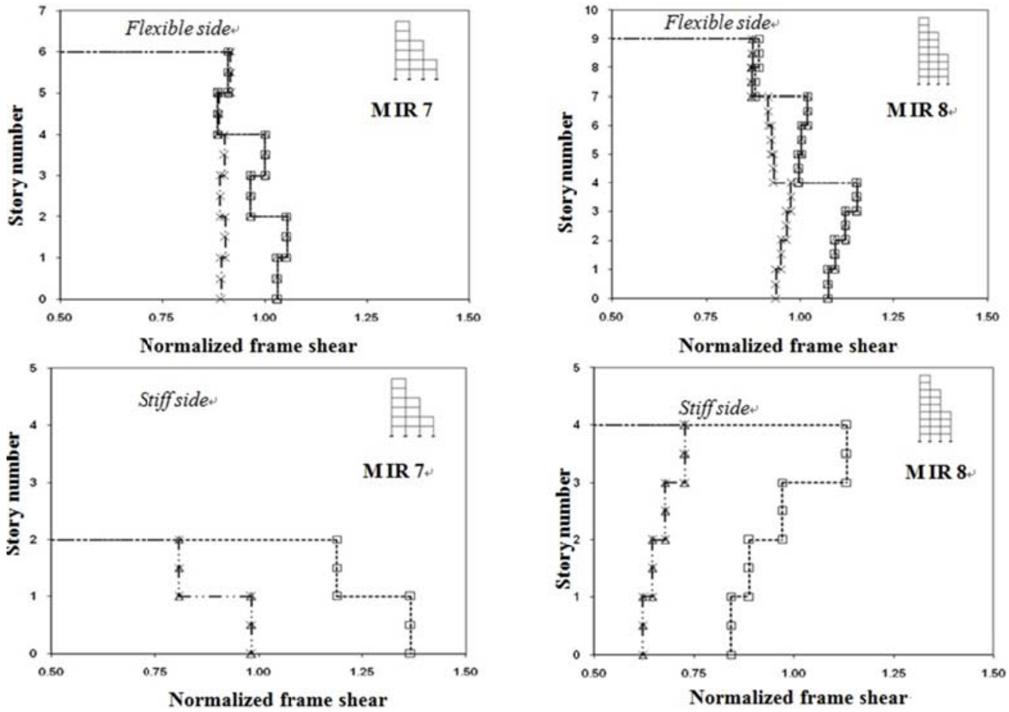


Fig. 5(b) Variation of normalized frame shear in perimeter frames of medium & high-rise buildings (lateral load profile: LP-I

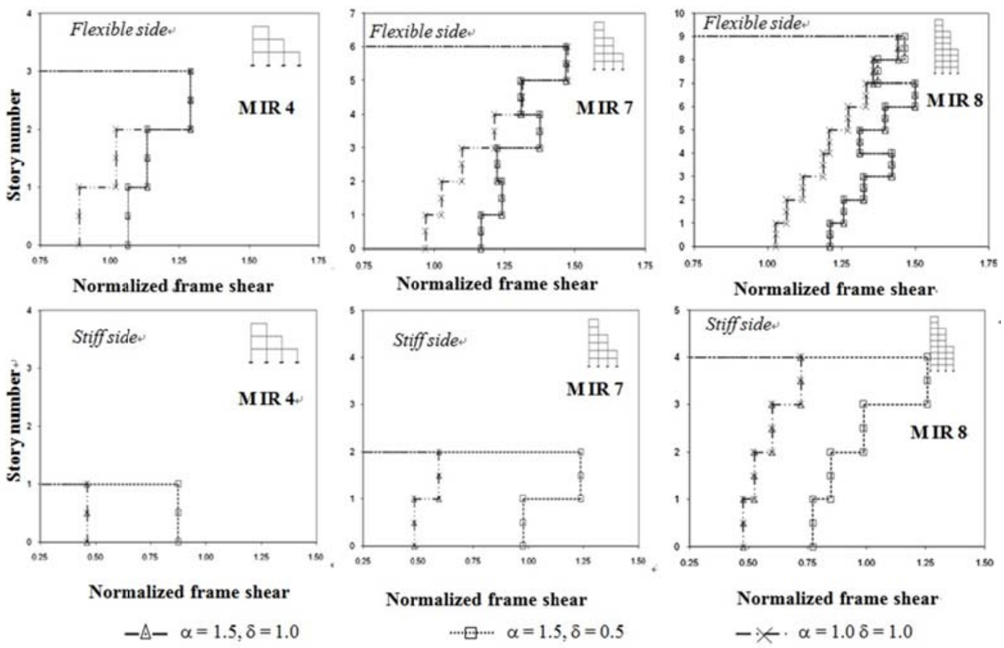


Fig. 6 Variation of normalized frame shear in perimeter frames of low, medium & high-rise buildings (lateral load profile: LP-II)

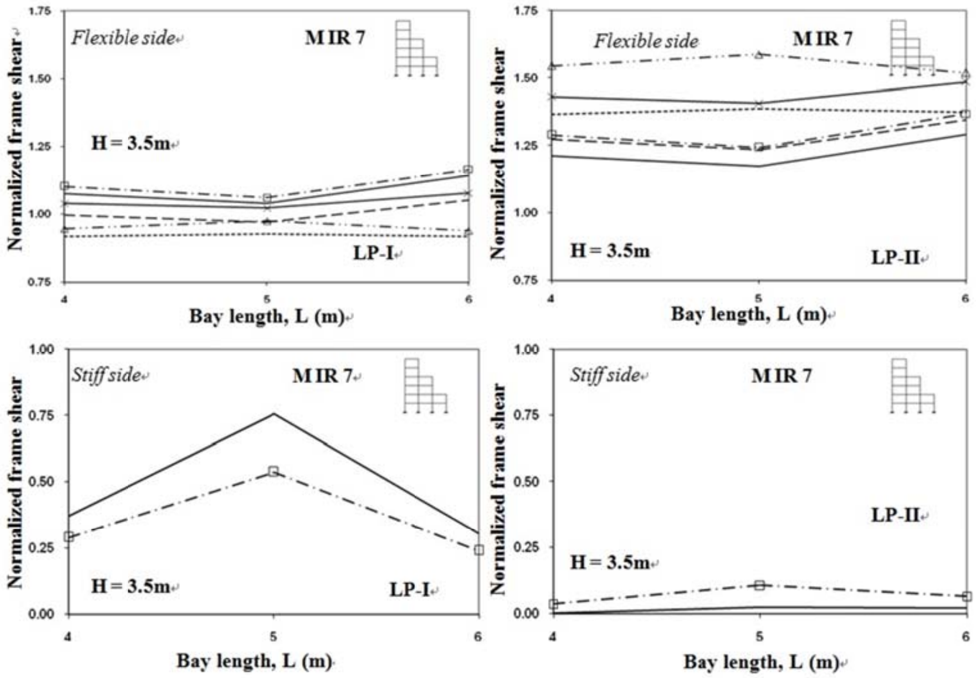


Fig. 7(a) Variation of normalized frame shear in perimeter frames of medium-rise buildings with change of bay length

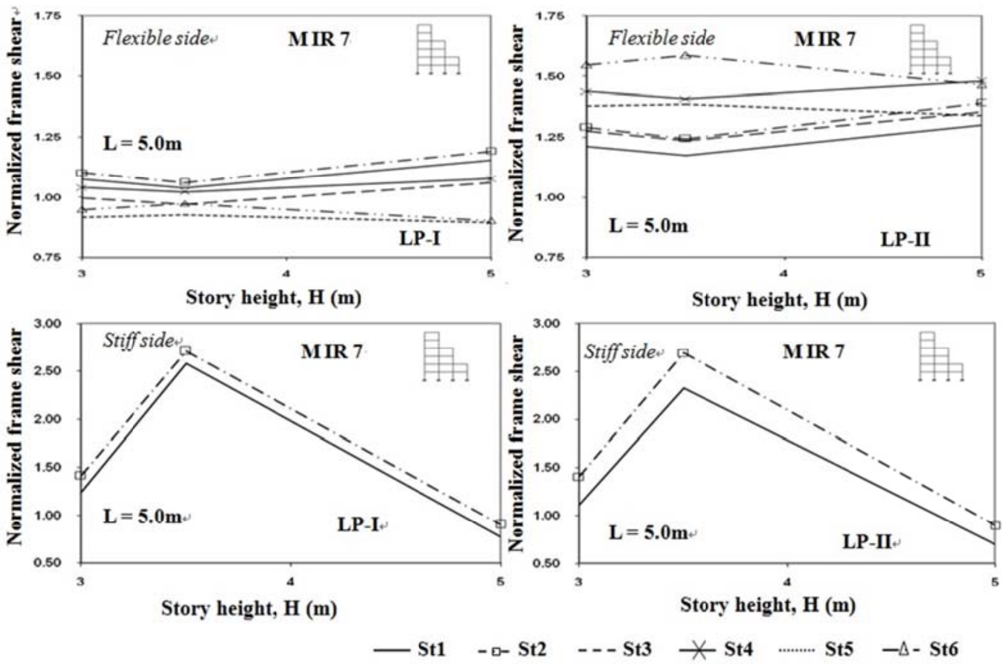


Fig. 7(b) Variation of normalized frame shear in perimeter frames of medium-rise buildings with change of bay length

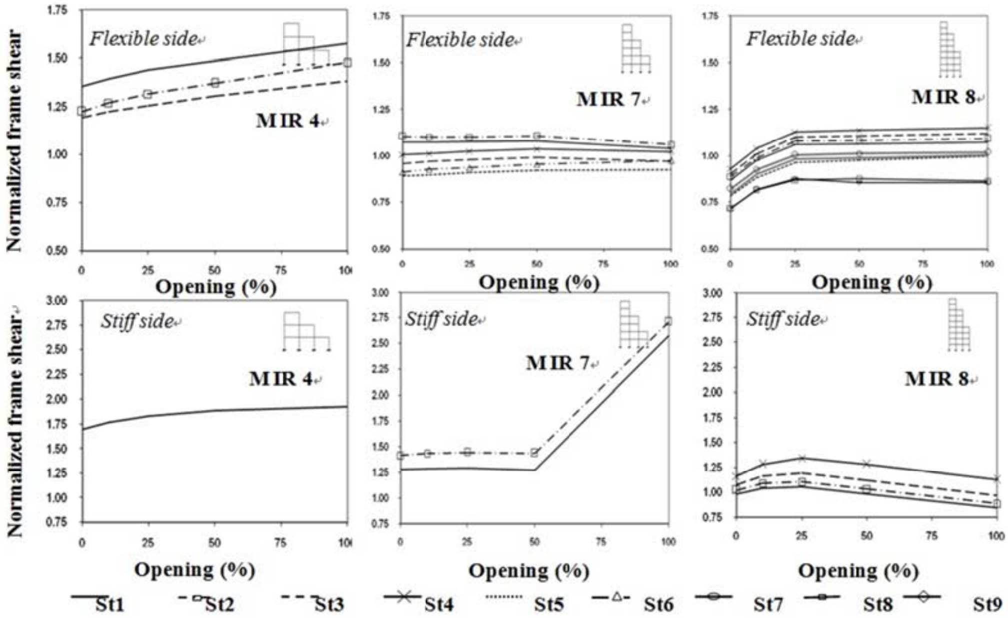


Fig. 8 Variation of normalized frame shear in perimeter frames with change of opening percentage in infill wall (Load profile: LP-I)

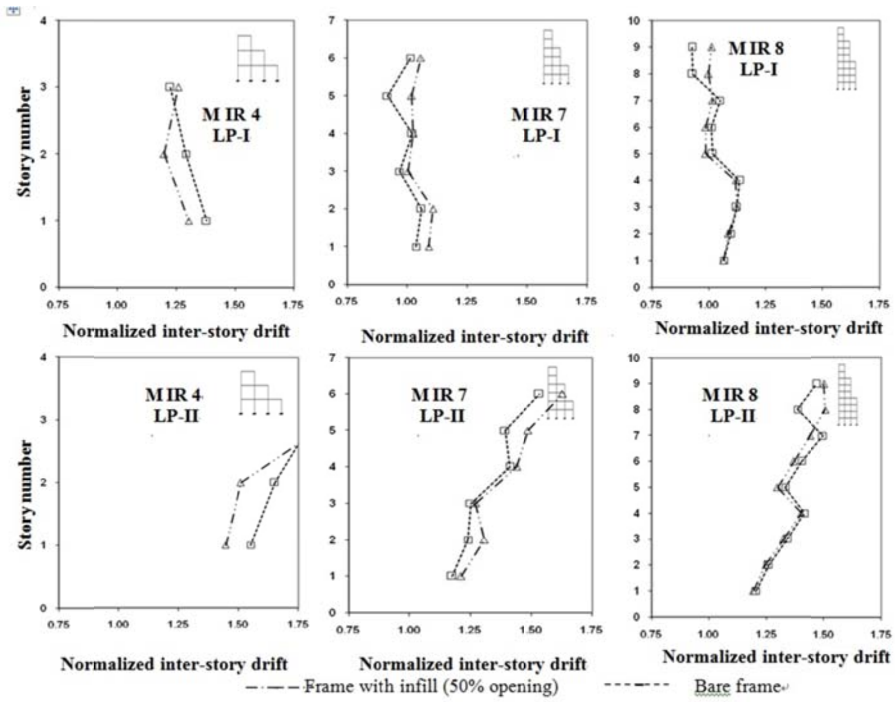


Fig. 9 Variation of maximum inter-story drift obtained from ELF (lateral load profile: LP-I and LP-II) and normalized to spectrum based response

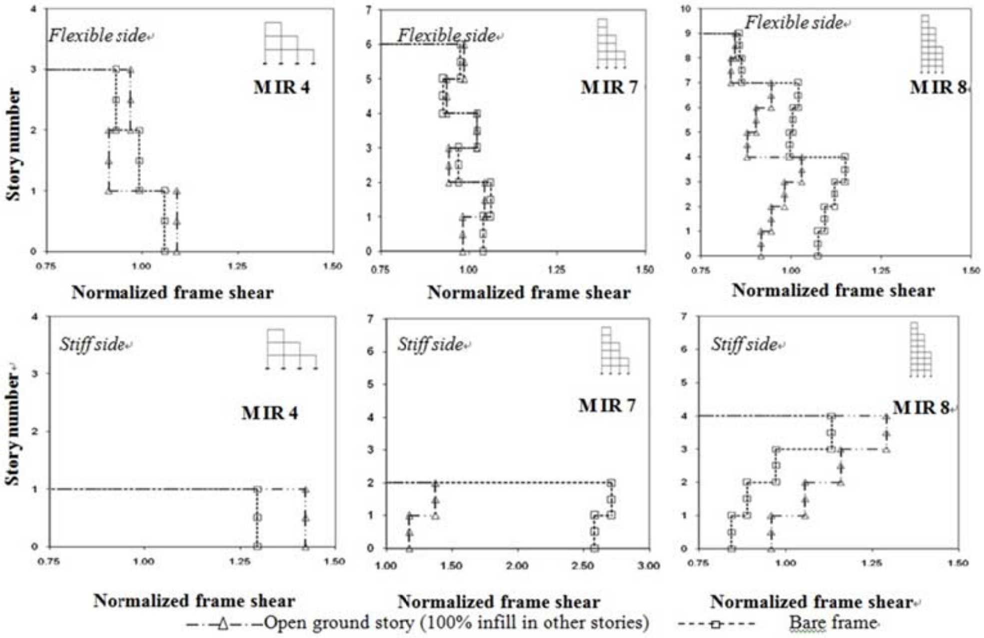


Fig. 10 Variation of normalized frame shear in perimeter frames of buildings with open ground story and bare frame model (Load Profile: LP-I)

structural and non-structural elements. Variation of normalized maximum inter-story drift over the height of the buildings (MIR 4, MIR 7 and MIR 8) is shown in Fig. 9. Results are computed considering bare frame model and also with infill effect (50% opening). Results of ELF are found to be in fair agreement with dynamic analysis particularly for medium to high-rise systems. However, normalized inter-story drift appears to be overestimated in case of LP-II (Fig. 9(b)). This is in line with the earlier response scenario in terms of frame shear parameter (Fig. 6).

Buildings are often found to be open in ground story in order to accommodate garages, shops etc. Response of such system may be critical at the soft ground story level with no infill wall. In this context, it may be interesting to examine the response of these systems in the backdrop of the bare frame behavior. Soft story buildings are assumed to have 100% infill wall in all higher story panels. Normalized frame shear of buildings (MIR 4, MIR 7 and MIR 8) for two above-stated cases, as furnished in Fig. 10, suggests close resemblance particularly for flexible side. However, this observation obtained from elastic models deserves further scrutiny in view of the limitation of the elastic method to account for the likely effect of localization on story displacement to significant stiffness irregularity.

4.2 Flexible floor system

MIR4, MIR7 and MIR8 are analyzed considering floor flexibility (diaphragm thickness of 100 mm, 150 mm and 250 mm assumed) excluding the effect of infill wall. Height-wise distribution of lateral load complying with LP-I is chosen considering the effectiveness of the same for the class of buildings chosen. Response quantities calculated by ELF are normalized by those from dynamic analysis. In view of lack of systematic trend on the influence of diaphragm flexibility, variation in frame shear is enveloped by mean plus and mean minus standard deviation curves. Variation of such normalized frame shear quantity computed using rigid floor model is also superimposed for comparison (refer to Fig. 11). Moreover, to recognize the order of dispersion due to diaphragm flexibility, co-efficient of variation (COV) of the normalized frame shear parameter is computed (Fig. 12). Such quantity is observed to be not more than about 0.02 for low and high rise buildings while the same may be around 0.06 to 0.09 in medium-rise system. Further investigation on medium-rise building (Fig. 13) reveals that, with change of aspect ratio of panel, variation in response is relatively stable in flexible side. This observation is in line with the similar cases in rigid floor model (Fig. 7). By and large, the normalized frame shear parameter obtained from flexible floor model may be at variance of around (-) 5% to (+) 15% relative to such response computed through rigid floor model (Fig. 11).

To achieve further insight into the impact of floor flexibility, frame shear obtained from dynamic analysis (response spectrum) using rigid and flexible floor model is compared. Frame shear obtained from flexible floor diaphragm model (for varying thickness of floor slabs) is normalized to that of rigid floor system and presented in Fig. 14 for MIR-7 and MIR-8. Comparison suggests that the seismic response may alter by around 10% due to floor flexibility in practical range of interest. Order of such change in response is generally similar in all stories and reduces in high-rise flexible systems.

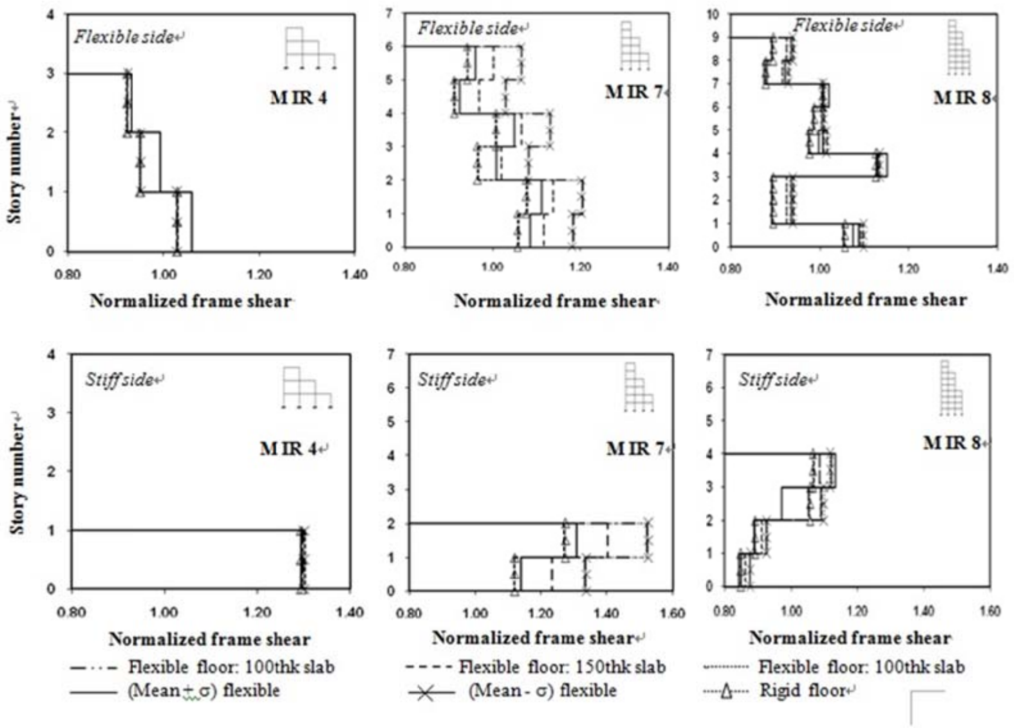


Fig. 11 Variation of normalized frame shear in perimeter frames

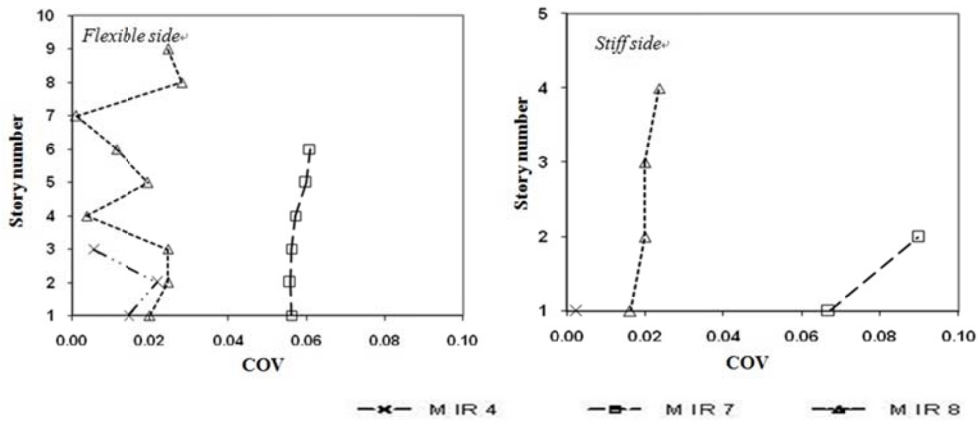


Fig. 12 Co-efficient of variation of normalized perimeter frame shear due to change of floor flexibility (slab thickness = 100, 150 & 250 mm) for sample buildings

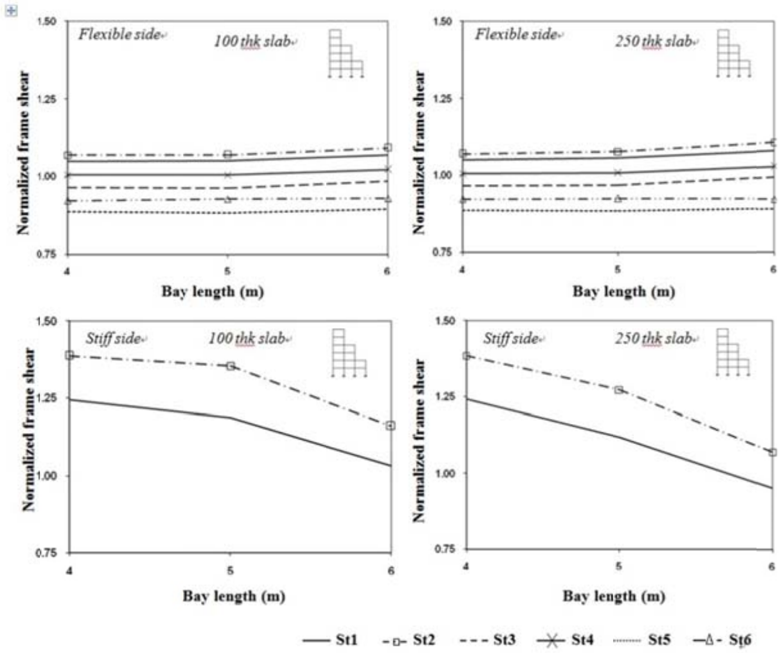


Fig. 13 Variation of normalized frame shear in perimeter frames of medium-rise buildings with change of bay length (lateral load profile: LP I)

5. Influence of accidental torsion

In view of the prevailing controversy on the implication of accidental torsional provisions, impact of the same on both rigid and flexible floor buildings with setback are separately evaluated. Procedure to include the effect of accidental torsion through ELF is already explained. It may be noted that there exists no general census as to how such accidental eccentricity be accounted in

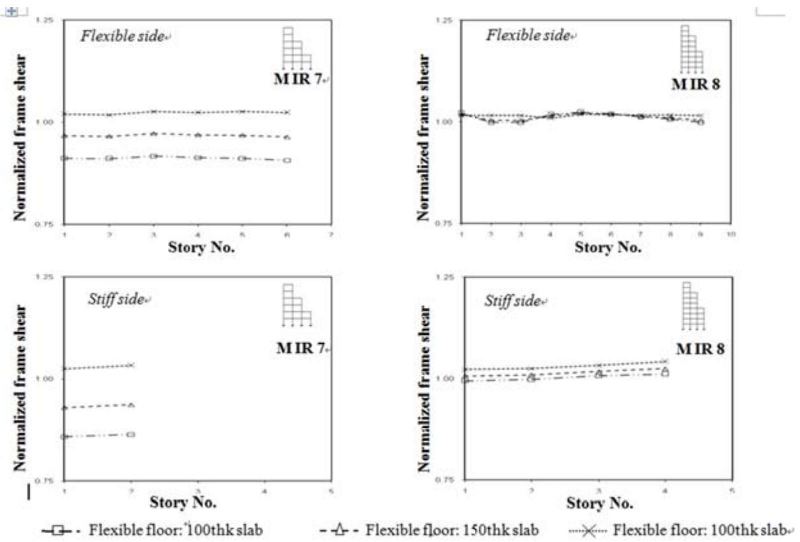


Fig. 14 Variation of normalized frame shear (CQCflexible/CQCrigid) in perimeter frames of medium & high rise buildings (lateral load profile: LP I)

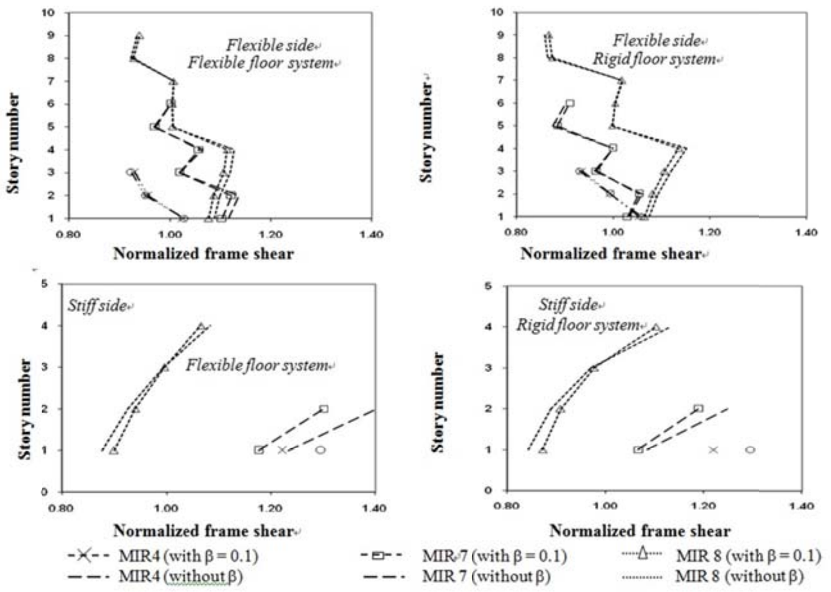


Fig. 15 Variation of normalized frame shear in perimeter frames of buildings modeled as rigid and flexible floor (150thk.) system with and without including accidental eccentricity (LP-I)

dynamic analysis. It is proposed, on the one hand, to perform dynamic analysis by mathematically shifting CM at each floor by an amount of accidental eccentricity from original CM. On the other hand, superposition of results due to application of static torsional moments equal to lateral force times the accidental eccentricity to the results from dynamic analysis of original system is also

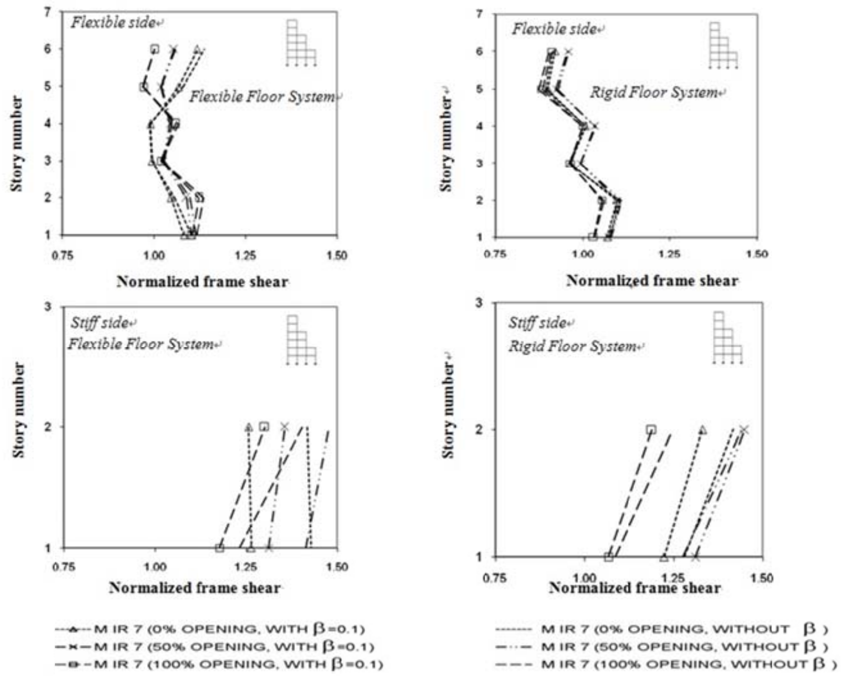


Fig. 16 Variation of normalized frame shear in perimeter frames of buildings modeled as rigid and flexible floor (150thk.) system with and without including accidental eccentricity along with infill wall (opening percentages: 0%, 50% & 100% and LP-I)

permitted. Unfortunately, it is well-established that these two approaches do not yield concordant results (De Le Llera and Chopra 1994). In this backdrop, the second approach is adopted herein following NBCC-95 (User's Guide- NBC 1995). Such approach has also been used elsewhere (Harasimowicz and Goel 1998).

Three, six and nine storey buildings (MIR 4, MIR 7 and MIR 8) are analyzed assuming accidental eccentricity β equals to 0.1 (step 3 of section 3). Height-wise distribution of lateral load is assumed to conform to LP-I. Buildings are modeled as both rigid and flexible floor systems (slab thickness assumed 150 mm). Response due to accidental eccentricity is superimposed with those from ELF based analysis conducted at the exclusion of accidental eccentricity as already presented. Such effect is also suitably included in the results of dynamic analysis. Subsequently, response quantities obtained from ELF is normalized by those from response spectrum based analysis.

Fig. 15 presents such normalized shear of perimeter frames of MIR 4, MIR 7 and MIR 8 accounting the effect of accidental eccentricity. Similar quantities without the inclusion of accidental eccentricity are also superimposed therein. On close scrutiny, it transpires that the impact of accidental eccentricity is marginal for both flexible and stiff side elements. Influence of accidental eccentricity in presence of infill wall is also reviewed in the limited form (model no. MIR 7 is considered). Effect of opening in the infill owing to the doors and windows are accounted in such study through considering an opening of 0%, 50% and 100% in the infill wall. Results of such case studies (Fig. 16) corroborate the earlier trend.

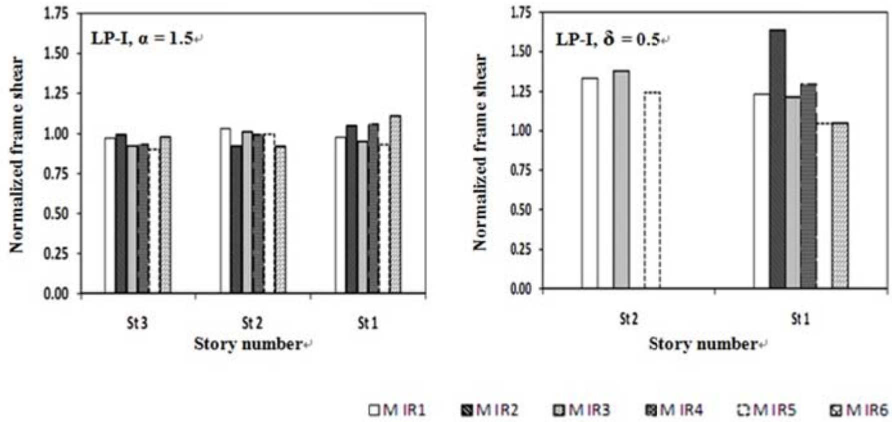


Fig. 17 Variation of normalized frame shear in perimeter frames of low-rise building (Load profile: LP-I)

6. Proposals for design

In order to recognize the adequacy of code torsional provisions with ELF in the context of set-back buildings, salient results are re-formatted in the form of a bar chart (Fig. 17). In view of the response summarized, it is perceived that the ELF in conjunction with static torsional provisions (considering α= 1.5) may be adequately used to estimate frame shear in flexible side. However, δ equals to 0.5 may often overestimate the response by around 25%. Lateral load distribution over the height of the building should conform to LP-I for design. Maximum inter-story drift may also be reasonably assessed by ELF for systems with and without infill.

Performance of code-torsional provisions through ELF is examined hitherto in terms of the compliance with the critical perimeter frame response only. However, for the purpose of satisfactory design, forces in all other frames are also needed to be estimated with reasonable accuracy. Thus, the normalized frame shear computed from ELF and dynamic analysis for all frames are reviewed. For this purpose, results of ELF conducted on MIR4, MIR7 and MIR8 modeled as rigid floor bare frames are compared to the those from dynamic analysis. Values of α, δ and β are respectively chosen as 1.5, 0.5 and 0.1 while lateral load profile is assumed as LP-I in ELF analysis. Variation in response expressed in percentage for different frames are furnished in Fig. 18 in a comprehensible format. Variation in results beyond ±10%, as observed only at a few locations, are encircled while the same between ±[5-10]% are highlighted for further scrutiny. This reveals that ELF can reasonably predict dynamic response of setback systems with certain exceptions in the vicinity of the setback in particular. Seismic force may be estimated with an upper bound error of around +25% (for low to medium-rise systems) while such deviation may be on the order of -15% (in lower story of stiff side). Further, in the upper story levels of flexible perimeter frames, response may be underestimated by around 6% (for low-rise) to 11% (for high-rise). It may, however, be noted that design force is regulated by an appropriate combination of dead load, live load and seismic loads and in that context, difference in predicted seismic force even on the order of 25% may typically alter the design force by only 9.7% (as observed from sample case study on MIR-7).

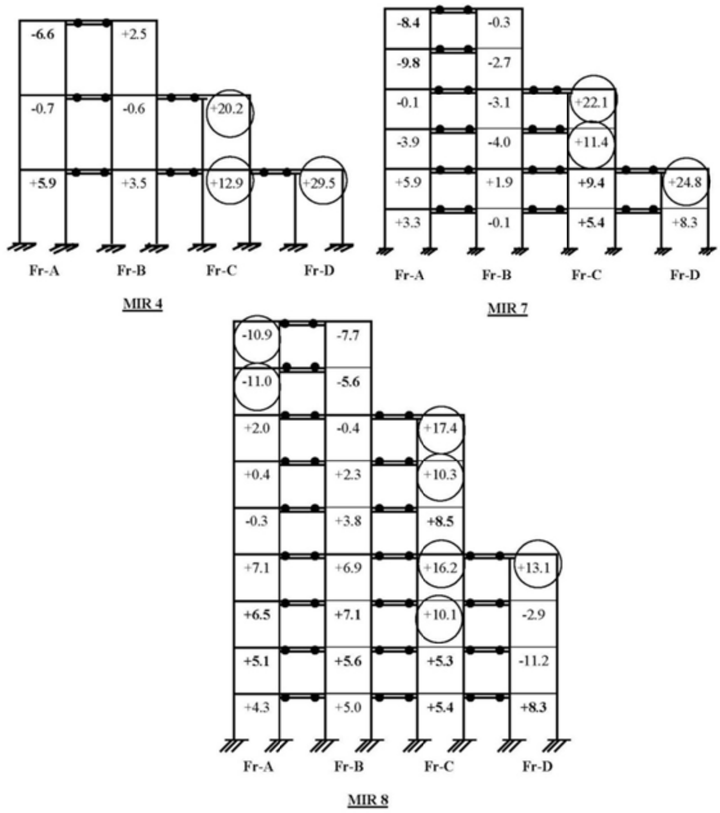


Fig. 18 Overall performance of ELF in typical setback building frames

In view of the relatively insignificant impact of floor flexibility and considering the associated rigor to incorporate such effect, rigid diaphragm model seems to be a pragmatic choice for real-life design at least for the class of systems studied. Limited study also reflects that the 'design for accidental eccentricity is likely to be ineffectual' and may not be considered at all. This observation is akin to the recommendations made elsewhere (Paulay 2001, Priestley *et al*. 2007).

7. Conclusions

In the context of relative complexity to carry out dynamic analysis in routine seismic design, codified torsional provisions with ELF may be useful. Such provisions, although not explicitly stated, are generally believed to be applicable to buildings with regular asymmetry. The present study examines the applicability of such codified standards for systems with set-back. To this end, the current investigation systematically examines the response of a number of systems covering representative configuration of elevation irregularity through ELF and response spectrum based methods. A comparison of the response reveals the following broad conclusions:

1. The study, through comprehensive case studies, observes that the code-specified ELF along with torsional provisions may be used, contrary to the conventional notion, for seismic design of setback buildings. Values of dynamic amplification factors α and δ may be adopted respectively as 1.5 and 0.5 along with a height-wise distribution of lateral load conforming to LP-I. Influence of accidental torsion and diaphragm flexibility may be ignored in practice. This code-static procedure may yield some concentration of seismic forces in the surroundings of the setback. However, such concentration may generally be acceptable keeping in view the overall design force (combination of dead, live, seismic loads etc.) and inherent uncertainties of seismic design.

2. Inter-story drift can be reasonably estimated for set-back buildings through employing code torsional provisions with ELF assuming a lateral load profile as per LP-I.

3. The observations outlined above are applicable for buildings with and without infill. It seems from the limited study that, beyond 25% to 30% opening in infill wall, response of the flexible side tends to be similar to those for bare frames particularly in medium to high-rise systems. Response of stiff side, in contrast, appears to be relatively sensitive to the infill percentage. However, pending further investigation confirming the generality of such observation, modeling of infill wall, as it exists, is desirable.

4. Simple irregularity indices proposed elsewhere (Karavasilis *et al*. 2008) appears to be in some compliance with the dynamic characteristics of the system. This justifies the characterization of set-back buildings in terms of such simple parameters from a more conceptual standpoint. Further it seems that the code-specified empirical formulae for building period need introspection.

In sum, the present study establishes that the ELF may be used for the design of vertically irregular systems, with certain experience and judgment, particularly in the vicinity of the setback. Seismic design strategy inherently relies on ductile response and hence the performance of such systems designed by both the approaches (ELF and response spectrum based) need be evaluated in inelastic range in future course of studies.

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