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AN ALTERNATIVE PROCEDURE FOR ACCIDENTAL ECCENTRICITY IN DYNAMIC MODAL ANALYSES OF BUILDINGS

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SUMMARY

Most building codes enforce the inclusion of accidental torsion in the equivalent static load and dynamic analyses of asymmetric buildings. The computations of accidental torsional moments in the equivalent static load procedure are straightforward and already implemented in most of building analysis tools. However, the application of accidental torsion in dynamic analysis can be performed in one of the following two basic approaches: 1) Shifting the centre of mass by the required amount of eccentricity in either direction. This will lead to change in global stiffness matrix of the system, therefore natural frequencies and modal parameters are needed to be computed for each eccentricity cases. 2) Run static analyses considering torques at each storey level to approximate accidental torsion for each eccentricity cases then combine the results with the dynamic load results. In this study an alternative procedure for the application of accidental torsion in dynamic modal superposition technique by modification of global force vectors to include the effect of accidental torsion for each modal shape separately. The proposed method has been applied to three sample multi-storey buildings and the results are compared with those obtained by using conventional approaches.

1. BACKGROUND AND INTRODUCTION

Most of the current structural design provisions require the designer to consider torsional behaviour through the use of design eccentricities, which take into account both natural and accidental sources of torsion. The natural eccentricity is generally defined as the distance between the centre of mass (CM) and the centre of rigidity (CR) at respective floors levels, while accidental eccentricity generally accounts for factors such as the rotational component of ground motion about the vertical axis, the difference between computed and actual values of the mass, stiffness or yield strength, and an unfavourable distribution of live load mass [Basu and Sudhir, 2004].

Generally, design eccentricity e_{dj} at the ith floor for static analysis of buildings can be expressed in the following general form [Goel and Chopra, 1993]

$$\begin{aligned} e_{dj} &= \alpha\, e_{si} + \beta\, b_i \\ e_{dj} &= \delta\, e_{si} - \beta\, b_i \end{aligned} \qquad (1)$$

Here the first term accounts for the natural torsional effect and the second term incorporates the accidental torsional effect and, e_{si} is the eccentricity for the ith floor defined as the distance between the floor CM and CR ; b_i is the plan dimension of the ith floor normal to the direction of ground motion; and α, β and δ are specified

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coefficients. The values of these coefficients are generally specified in the national and international design codes. Table 1 summarizes the α, β and δ coefficients that are specified in some current design codes.

For design practices, if the building codes specify $\alpha=\delta=1.0$, the location of the CR need not be explicitly calculated to implement torsional provisions. In such cases, a rigid floor diaphragm building can be analyzed by applying design lateral force at $\pm\beta b_i$ eccentricity with respect to the CM [Basu and Sudhir, 2004].

Table 1: Design torsional eccentricity coefficients in different design codes

Design Code	α	β	δ
National Earthquake Hazards Reduction Program, NEHRP [1997]	1.0	0.05	1.0
International Building Code IBC [2003]	1.0	0.05	1.0
EuroCode 8 [2003]	1.0	0.05	1.0
Turkish Code, DBYBHY [2006]	1.0	0.05	1.0
National Building Code of Canada NBCC [1995]	1.5	0.1	0.5
Mexico City Building Code MCBC [1995]	1.5	0.1	1.0

In the literature there are numerous studies about the dynamic analysis of buildings with accidental eccentricities. The concept of accidental eccentricity, to account for torsional vibrations induced by rotational excitation, was first proposed by Newmark [1969]. In their pioneering work, Rosenblueth and Elorduy [1969] defined the magnification factor for the static eccentricity, In the following years many authors studied the torsional eccentricities of rigid-base structures to review the adequacy of code torsional provisions and to propose modifications and new procedures [Dempsey and Tso, 1982; Chandler and Duan, 1997; Anastassiadis et al 1998; Calderoni et al, 2002; De-la-Colina and Almeida, 2003; Stathopoulos and Anagnostopoulos, 2005; De la Llera and Chopra, 1995].

In the current design practice, the computations of accidental torsional moments in equivalent static load procedure are straightforward and already implemented in most of building analysis codes. The accidental torsional effects are considered by applying the equivalent static lateral forces at a distance e_{di} from the centre of rigidity (CR), which includes the accidental eccentricity. The procedure can be implemented in building analysis tools in a straightforward manner and requires two static analyses of the building for each lateral direction.

The practical inclusion of the torsional eccentricity in dynamic analyses can be made by using one of the following two basic approaches: 1) Moving centre of mass (CM) of each floor from its nominal position by the required amount of eccentricity, e_{di} , in either direction. This will lead to a change in the global stiffness matrix of the system, therefore natural frequencies and modal parameters have to be computed for each eccentricity cases 2) Run static analyses considering torques at each storey level to approximate accidental torsion for each eccentricity cases then combine the results with the dynamic load results.

The main drawback of the first method is that the natural characteristics of the system are inevitably changed in each eccentricity case. A direct implementation of the method, therefore, does not exist in the majority of the building analysis softwares. Consequently, considerable computational effort is necessary to come up with an envelope solution for four independent runs of each eccentricity cases.

It is also important to note that the static and dynamic analyses methods predict significantly different increases in design displacements and forces under a given accidental eccentricity [Lin et al, 2001]. Besides , the code static analyses are not consistent with the analytical predictions of accidental torsion [De la Llera and Chopra 1995].

In this paper a simple modification to the modal analysis procedure is proposed to include the accidental torsion effects for multi-mode response of the multi-degree of freedom systems. The methodology is based on dynamic modal superposition technique by applying of accidental torsion to global force vectors related to each modal shape. For this purpose, the global displacement vectors are computed for each modal shape at the first step. Then, corresponding global force vectors are computed by using the global mass matrix, the eigenvalues of the system and the global displacement vectors. For each mode shape, the accidental torsional moments of the global force vector are updated by the required amount of eccentricity in either direction. Static analysis is carried out to find the modified global modal displacement vectors and the internal forces for each member for each modal shape. The nodal displacements and the internal force resultants can be combined by using standard modal combination techniques.

The proposed procedure can be applied for building with rigid and/or flexible diaphragms and it can easily be implemented in building analysis tools. In addition the proposed procedure has the following advantgages over the existing methodologies:
1) Only one eigenvalue analysis is required for all torsional eccentricity cases.
2) Easier combination and envelope computation of the four each torsional eccentricity cases.
3) It offers a unified procedure for torsional eccentricity applied to static equivalent and modal dynamic analyses
4) It include the effects of torsional eccentricity for all modal shapes considered in the analysis.

2. THEORETICAL BACKGROUND

The equation of motion of the for a system subjected to ground motion acceleration, $\ddot{\boldsymbol{u}}_{\boldsymbol{g}}(t)$, is

$$\boldsymbol{M}\ddot{\boldsymbol{u}}(t) + \boldsymbol{C}\dot{\boldsymbol{u}}(t) + \boldsymbol{K}\boldsymbol{u}(t) = -\boldsymbol{M}\boldsymbol{T}_x\ddot{\boldsymbol{u}}_g(t) \tag{2}$$

Here, $\boldsymbol{M}$, $\boldsymbol{C}$, $\boldsymbol{K}$ are the mass, damping and stiffness matrices respectively for the system; **u** is the vector of nodal displacements above the base, relative to the free ground motion. The related nodal velocity and acceleration are denoted by the vectors $\dot{\boldsymbol{u}}$ and $\ddot{\boldsymbol{u}}$. $\boldsymbol{T}_x$ is the influence vector associated with the components of ground motion acceleration, $\ddot{\boldsymbol{u}}_g(t)$ in the x direction.

The natural vibration mode shapes, $\boldsymbol{\Phi}_n$ and corresponding natural frequencies,ω_n, are the solutions of the free vibration eigenvalue problem of the system

$$\boldsymbol{K\Phi} = \omega_n^2\boldsymbol{M\Phi} \tag{3}$$

The nodal displacement vector can be rewritten by applying modal coordinate transformation in time domain as;

$$\boldsymbol{u}(t) = \boldsymbol{\Phi}\,\boldsymbol{Z}(t) = \sum_{n=1}^{Y}\boldsymbol{\Phi}_n Z_n(t) \tag{4}$$

In this equation $\boldsymbol{Z}(t)$ is the modal coordinate vector, Y is the number modes considered, $\boldsymbol{\Phi}_n$ and $Z_n(t)$ are the mode shape vector and modal coordinate for the nth mode respectively. The nodal velocities and accelerations are expressed respectively, as

$$\dot{\boldsymbol{u}}(t) = \sum_{n=1}^{Y}\boldsymbol{\Phi}_n\dot{Z}_n(t)\ ,\ \ddot{\boldsymbol{u}}(t) = \sum_{n=1}^{Y}\boldsymbol{\Phi}_n\ddot{Z}_n(t) \tag{5}$$

Applying the above transformations to Eq (2) and considering the orthogonality properties of the mode shapes, one may obtain a set of equations for the generalized modal displacement, $Z_n(t)$, as

$$\ddot{Z}_n(t) + 2\,\xi_n\omega_n\,\dot{Z}_n(t) + \omega_n^2\,Z_n(t) = -\left(\frac{L_{xn}}{M_n}\right)\ddot{\boldsymbol{u}}_{gx}(t) \tag{6}$$

In Eq. (6) ξ_n is the damping ratio of the nth mode. L_{xn} and M_n are defined as:

$$L_{xn} = \boldsymbol{\Phi}_n^T\,\boldsymbol{M}\,\boldsymbol{T}_x = \sum_1^N\left(m_i\Phi_{xin}\right) \tag{7a}$$

$$M_n = \boldsymbol{\Phi}_n^T\,\boldsymbol{M}\,\boldsymbol{\Phi}_n = \sum_1^N\left(m_i\Phi_{xin}^2 + m_i\Phi_{yin}^2 + m_{\theta i}\Phi_{\theta\,in}^2\right) \tag{7b}$$

The ratio L_{xn}/M_n is a dimensionless parameter defined as “the participation factor of the nth mode in x earthquake direction”. In the spectral response analysis the solution of the Eq. (3) for the maximum modal coordinate of the rth mode can be written as;

$$Z_{n,max} = \left(L_{xn}/M_n\right)\frac{S_a(T_n)}{\omega_n^2} \tag{8}$$

where, $S_a\,(T_n)$ is the spectral acceleration for the nth mode obtained by using the code spectrum.

By using Eq (2) for the x directional earthquake the maximum displacement for the nth mode can be written as;

$$\boldsymbol{u}_{n,max} = \boldsymbol{\Phi}_n\, Z_{n,max} = \boldsymbol{\Phi}_n\left(L_{xn}/M_n\right)\frac{S_a(T_n)}{\omega_n^2} \tag{9}$$

Consequently, the analysis earthquake force vector for the nth mode in x direction can be written as;

$$\boldsymbol{F}_{n,max} = \boldsymbol{K}\ \boldsymbol{u}_{n,max} \tag{10a}$$

The same equation can be written as by using Eq. (7) and free vibration properties;

$$\boldsymbol{F}_{n,max} = \omega_n^2\ \boldsymbol{M}\ \boldsymbol{u}_{n,max} = \boldsymbol{M}\ \boldsymbol{\Phi}_n\left(L_{xn}/M_n\right) S_a(T_n) \tag{10b}$$

In a similar manner the forces acting on the ith storey of a structure in the nth mode x and y direction and the torsional moment about z axis can be written as;

$$F_{xin,max} = m_i \Phi_{xin}\left(L_{xn}/M_n\right) S_a(T_n) \tag{11a}$$

$$F_{yin,max} = m_i \Phi_{yin}\left(L_{xn}/M_n\right) S_a(T_n) \tag{11b}$$

$$F_{\theta\, in,max} = m_{\theta i} \Phi_{\theta in}\left(L_{xn}/M_n\right) S_a(T_n) \tag{11c}$$

3. ACCIDENTAL TORSION UPDATE TO MULTI-MODAL DYNAMIC ANALYSES

The accidental torsional effects are considered at the ith storey of a structure in the nth mode by updating $F_{\theta \mathrm{in,max}}$ to include the additional torsional moment results. The additional torsional moment are computed by applying $F_{\mathrm{xin,max}}$ and $F_{\mathrm{yin,max}}$ forces at a distance e_{di} from the centre of mass (CM).

$$F^*_{\theta\, in,max} = F_{\theta\, in,max} + F_{yin,max} \times ex_{di} \tag{12a}$$

$$F^*_{\theta\, in,max} = F_{\theta\, in,max} + F_{xin,max} \times ey_{di} \tag{12b}$$

In Eqs. 12a and 12b ex_{di}, ey_{di} are the design eccentricities at the ith floor in x and y directions respectively. Eq (10a) represents the torsional moment modification for x direction motion eccentricity cases while Eq (10b) stand for the eccentricity cases in y direction, Figure 1.

For nth mode in x direction, the updated torsional moments $F^*_{\theta \mathrm{in,max}}$ are used to modify the force vector, $\boldsymbol{F}_{\mathrm{n,max}}$, defined in Eq (8). Therefore, the effects of respective design eccentricity will be included in the resulting force vector, $\boldsymbol{F}^*_{\mathrm{n,max}}$.

The maximum nodal displacement $\boldsymbol{u}_{\mathrm{n,max}}$ can be updated to include the torsional eccentricity by solving the following linear system of equations;

$$\boldsymbol{K}\ \boldsymbol{u}^*_{n,max} = \boldsymbol{F}^*_{n,max} \tag{13}$$

The updated maximum nodal displacement for each n^{th} mode, $\boldsymbol{u}^*_{n,max}$, can directly be used in the modal combination procedures as proposed by the design codes to calculate the maximum response values of nodal displacements and internal force vectors.

The procedure requires to solve the linear system given in Eq (13) for each of the n^{th} modes in the required eccentricity case. Even though the proposed procedure appears to be numerically cumbersome, practically it is not the case. For simple solvers, like Gauss elimination method or LU decomposition, once the time consuming elimination process for $\boldsymbol{K}$ matrix is done, only back-substitution for each $\boldsymbol{F}^*_{n,max}$ is required. This makes the modification procedure simple and effective for almost all multi-storey building structures.

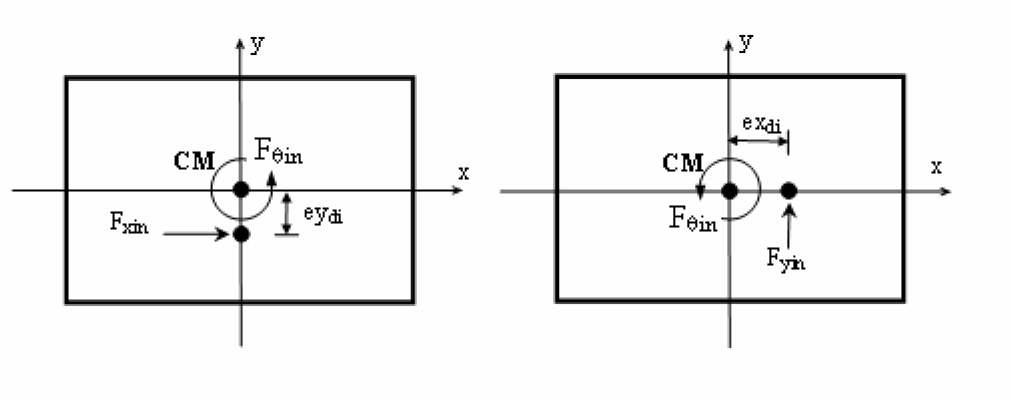

Figure 1: Updating torsional moment for specified vibration mode

4. NUMERICAL EXAMPLES

The proposed method has been applied to three sample multi-storey buildings. As the importance of choosing real building configuration for torsional eccentricity studies have been emphasized by Stathopoulos and Anagnostopoulos [2005], one- , five- and ten-storey reinforced concrete space frames with real configurations have been considered. Typical floor configuration of the buildings considered are formed by three and four plane frames in x and y directions as shown in Figure 2. Columns and beam cross-section dimensions and the slab thickness are given in Table 2. The cross-section of the vertical structural members for each individual building are assumed constant over the entire height of the building. All buildings have a typical storey height of 3.0 m and a ground storey height of 4.0 m.

The storey masses are formed using appropriate distribution of the slabs loads, and brick partition walls. The slab loads composed of self weight (G) and 30% of the live load (Q), where, G = own weight + 1 kN/m^2, Q = 3.5 kN/m^2. The mass centres of the building floors are assumed to be at the geometrical centres of the floor plans.

Orion [2006], structural design software is utilized for three dimensional modelling, analyses, and design of the sample buildings. The lumped mass models with masses lumped at the joints were considered in the analysis and complete quadratic combination (CQC) with 5% damping were used for modal combination procedure. Modal periods and associated cumulative sum participating mass ratios in lateral x, lateral y and rotation z directions (Ux, Uy, Rz) have been computed and tabulated in Table 3.

The design seismic loading is calculated using the response spectrum based on the Turkish Earthquake Code, [2006]. The buildings are assumed to be located in the first seismic zone, with local soil profile Z2 and building importance coefficient (I) equal to 1. As it is suggested by the code, the structural behaviour factor (R) is considered to be 7. The spectrum coefficient and the design acceleration spectra curves are given in Figure 3.

The buildings have been analyzed under seismic loading with 5% accidental eccentricity (β=0.05). The seismic loads that include the effects of accidental torsion have been computed using the proposed method (Multi-Modal Eccentricity) in both x and y directions. For comparison purposes, the analyses have also been carried out by using equivalent static method (EQ Static) and dynamic analysis based on displacing the centre of masses, CM, of each floor from its nominal position to a distance equal to the accidental eccentricity (Dynamic CM Shift).

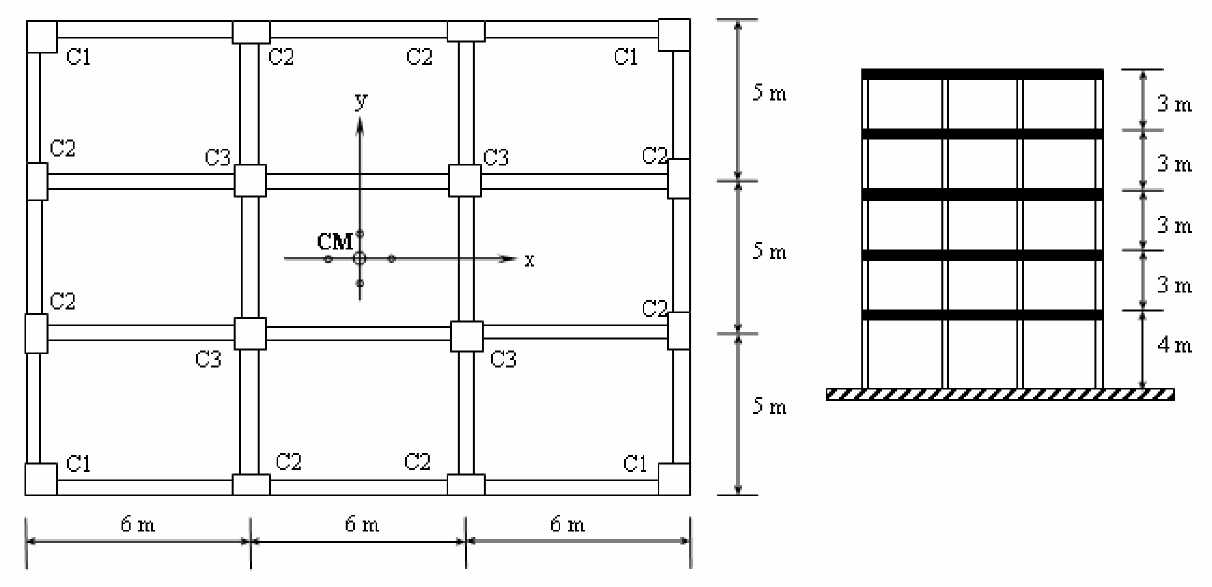

Figure 2: Floor plan and cross-section of reinforced concrete frames

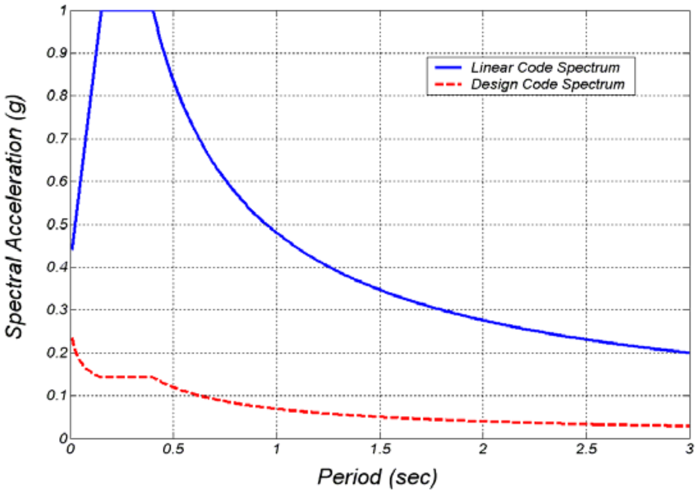

Figure 3: Design response spectrum used in the analysis

Table 2: Members dimensions of the sample buildings

	One Storey Frame	*Five Storey Frame*	*Ten Storey Frame*
Column C1 (cm)	30x30	50x50	70x70
Column C2 (cm)	25x30	30x50	70x70
Column C3 (cm)	30x30	50x50	60x50
Beams in x Direction (cm)	25x60	25x60	25x60
Beams in y Direction (cm)	25x60	25x60	25x60
Slab Thickness (cm)	15	15	15

Table 3: Modal periods and cumulative sum of modal participating mass ratios of the frames

	One Storey Frame				*Five Storey Frame*				*Ten Storey Frame*			
	Period	**ΣUx%**	**ΣUy%**	**ΣRz%**	**Period**	**ΣUx%**	**ΣUy%**	**ΣRz%**	**Period**	**ΣUx%**	**ΣUy%**	**ΣRz%**
1	0.330	99.8	0.0	0.2	0.872	84.0	0.0	0.0	1.440	79.6	0.0	0.0
2	0.327	99.8	99.9	0.3	0.830	84.0	83.3	1.0	1.331	79.6	78.7	1.0
3	0.287	100.0	100.0	100.0	0.755	84.0	84.2	84.3	1.221	79.6	79.7	80.4
4					0.281	94.2	84.2	84.3	0.457	89.9	79.7	80.4
5					0.269	94.2	94.3	84.3	0.424	89.9	90.1	80.5
6					0.244	94.2	94.4	94.3	0.391	89.9	90.2	90.2
7									0.252	93.9	90.2	90.2
8									0.235	93.9	94.0	90.3
9									0.219	93.9	94.1	94.1

Table 4: Comparison of maximum displacements

Building	**Disp. (m)**	**Dynamic (No Eccentricity)**		**EQ Static (with Eccentricity)**		**Dynamic CM Shift**		**Multi-Modal Eccentricity**	
		+X	**+Y**	**+X**	**+Y**	**+X**	**+Y**	**+X**	**+Y**
One Storey	Ux	0.00396	0.00015	0.00413	0.00037	0.00467	0.00145	0.00408	0.00045
	Uy	0.00014	0.00392	0.00030	0.00425	0.00124	0.00505	0.00026	0.00432
Five Storey	Ux	0.01863	0.00159	0.02394	0.00215	0.02627	0.00946	0.02298	0.00318
	Uy	0.00009	0.01871	0.00173	0.02393	0.00691	0.02795	0.00173	0.02387
Ten Storey	Ux	0.03572	0.00286	0.04995	0.00409	0.04850	0.01754	0.04474	0.00576
	Uy	0.00033	0.03401	0.00336	0.04747	0.01157	0.05175	0.00325	0.04412

Table 5: comparison of maximum shear forces (+X direction)

Building	**Shear Force (kN)**	**Dynamic (No Eccentricity)**			**Equivalent Static (with Eccentricity)**			**Dynamic CM Shift**			**Multi-Modal Eccentricity**		
		C1	**C2**	**C3**	**C1**	**C2**	**C3**	**C1**	**C2**	**C3**	**C1**	**C2**	**C3**
One Storey	Fx	17.62	19.93	19.65	18.39	20.81	19.95	20.75	23.47	20.77	18.15	20.53	19.86
	Fy	0.63	0.71	0.25	1.39	1.55	0.52	5.81	6.46	2.24	1.18	1.32	0.44
Five Storey	Fx	62.39	61.56	79.26	76.67	75.99	93.79	88.56	87.36	100.0	77.07	76.05	93.90
	Fy	0.50	0.47	0.33	6.34	6.21	2.81	28.03	27.46	12.77	6.64	6.47	2.92
Ten Storey	Fx	87.64	101.6	109.0	109.7	127.7	131.7	121.2	140.4	134.7	110.5	128.0	131.8
	Fy	1.09	1.26	0.50	9.15	10.72	3.83	38.04	44.51	16.87	9.91	11.59	4.16

Table 6: Comparison of maximum shear forces (+Y direction)

Building	Shear Force (kN)	Dynamic (No Eccentricity)			Equivalent Static (with Eccentricity)			Dynamic CM Shift			Multi-Modal Eccentricity		
		C1	**C2**	**C3**	**C1**	**C2**	**C3**	**C1**	**C2**	**C3**	**C1**	**C2**	**C3**
One Storey	Fx	0.65	0.73	0.26	1.64	1.85	0.62	6.30	7.13	2.47	2.01	2.27	0.77
	Fy	17.96	19.97	19.56	19.44	21.63	20.12	23.16	25.76	21.08	19.81	22.04	20.25
Five Storey	Fx	5.59	5.51	2.46	7.26	7.17	3.23	31.02	30.61	13.24	11.07	10.91	4.82
	Fy	70.08	68.18	84.71	85.56	83.63	100.44	104.6	102.0	106.7	89.25	86.89	101.85
Ten Storey	Fx	8.00	9.20	3.37	10.30	11.87	4.26	43.9	50.54	18.41	16.00	18.40	6.68
	Fy	97.51	113.8	116.9	121.5	142.5	141.5	150.2	175.5	150.7	126.9	148.1	143.4

The static equivalent analyses have been computed using the procedure defined in the Turkish Earthquake Code, [2006]. Total equivalent seismic loads (base shears), have been determined by multiplying the total weight of the structure by the design spectral acceleration corresponding to the first natural vibration period of the building Lateral load distribution is triangular shape action at each storey level. Additional equivalent seismic load has been assigned to the top floor of the ten storey building as the building height was over 25m.

Sufficient number of vibration modes is considered to assure that that the sum of effective participating masses calculated for each mode in x and y earthquake directions are more than 90% of the total building mass. All internal force and displacement quantities determined by mode superposition method are amplified by a scaling factor. For each earthquake direction, the scale factor is defined as the ratio of the base shear obtained from modal combination to the base shear obtained from equivalent seismic load method.

The three buildings have been analyzed under two lateral seismic loads (x and y directions). First, modal dynamic analyses without any accidental eccentricity have been performed. Then, the two procedures to apply the accidental eccentricity in dynamic analyses (Dynamic CM Shift, Multi-Modal Eccentricity) have been applied. A positive 5% accidental eccentricity is considered, and the analyses have been carried out using two procedure described above. For comparison, the analyses of the buildings under equivalent static load with accidental eccentricity have been performed.

The maximum x and y displacements at the upper floor of each building are given in Table 4. The maximum ground floor column shears, F_x and F_y (for column types C1,C2 and C3) are presented in Tables 5 and 6 for dynamic forces in x and y directions respectively.

The results show very good agreement between the results of the conventional techniques for one and five storey buildings. As far as the nodal displacements and the member shear forces are concerned , the Dynamic CM Shift method give higher values, with maximum relative difference of 18% for nodal displacements and 22% for members shear forces.

For ten storey building, on the other hand, the maximum relative difference between the results of the Static Equivalent method and the proposed Multi-Modal Eccentricity method was 7% for displacement and 4% for members shear forces. In this case, the maximum relative difference for Dynamic CM Shift method was 9% for nodal displacements and 23% for members shear forces.

5. CONCLUSION

In this study, the accidental torsion code requirements for equivalent static load procedure and dynamic analyses are discussed. The current design approaches for the computation of accidental torsional moment effects in equivalent static load and dynamic analysis are summarized. Alternatively a new procedure for the application of accidental torsion in dynamic modal superposition technique by modification of global force vectors to include the effect of accidental torsion for each modal shapes separately have been proposed. The procedure of the proposed method have been applied to three sample multi-story buildings and the results are compared with the conventional approaches. There are good agreement between the results of Static Equivalent method and the proposed Multi-Modal Eccentricity method for one and five storey buildings especially for low and moderate high buildings. As far as nodal displacements and member shear forces were concerned the conventional dynamic analysis consistently gave slightly higher results than the static equivalent method and the proposed Multi-Modal Eccentricity method.

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