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The Effect of Overhangs on Torsional Response of Reinforced Concrete Frames under Lateral Loads

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Abstract Earthquake behavior of structural systems strongly depends on geometry of the building. Structural irregularities are one of the major causes of damage amplification under seismic action. Past earthquakes, indeed, have shown that buildings with irregular configuration or asymmetrical distribution of structural properties are subjected to an increase in seismic demand, causing greater damages.If the earthquake damages are examined, it can be seen that structures which have overhangs are damaged more than the others. In this study, the effect of overhangs of RC frame on the torsional response is evaluated. Both torsional irregularity factors and torsional moment ratio of corner column are evaluated. The model have 6 and 9-story buildings, buildings are 20 m by 20 m in plan. They have 4@5 m bays along X and Y directions, 5 different overhang alternatives and 3 different overhang lengths. Structural systems are modelled using SAP2000software and investigated under earthquake load using both response spectrum analyses and equivalent static load methods. Results show the torsional moment ratio reached approximately 9 times higher than those in the regular frame.

Keywords Earthquake, Irregularity, Overhangs, Torsional response, Torsional moment

Introduction

Earthquake field investigations repeatedly confirm that irregular structures suffer more damage than their regular counterparts. Torsional irregularity is one of the most important factors, which causes severe damage (even collapse) for the structures [1].

Earthquake load acts at the center of mass of the structure. However, resisting force acts at a point called center of rigidity on the structure, which is the center of lateral resistance. Torsional problems take place when the mass center and center of rigidity are not located at the same place. Increasing distance between center of mass and center of rigidity, building is forced to twist around the rigid structural section (rigid core) and subjected to great torsional moments [2].

Excessive torsion causes columns and concrete walls to fail or severe damage. For many asymmetrical structures, excessive torsion is the main reason of the poor seismic performance. Torsion effects increase lateral deflections on the weak direction of the structure and decrease on the strong direction. The difference between center of mass and center of rigidity locations cause torsion in structures under lateral loads. Concrete walls, slab holes, overhangs, etc. may cause eccentricity between center of mass and center of rigidity [2].

Open or closed cantilever projections are a form of irregular mass distribution commonly encountered in the most buildings to enlarge plan dimensions and create space for balconies [1].

Cantilevered facades are also fashionable for architectural and aesthetic reasons. The aim of cantilever projections is thus to maximize the gross floor area of a building by utilizing the land in the most effective manner. However, this practice can have negative effects on seismic behavior. At most, the cantilever length is commonly approximately 1.5 m, while in some cases it can be 2 m or more at the first floor level [3].

Figure 1: Overhang types: (a) Balcony, (b and c) closed one-sided and two-sided overhangs

There are two types of overhangs. First type is open overhangs Fig. (1a) and second type is closed overhangs Fig. (1b and c). Balconies are the examples of first type, whereas in second type the overhangs are closed with walls in order to be used as semi-balconies or rooms. Overhangs can be formed at any side of a building. A structural difference between these two types is that closing them with walls increases the load on the overhang,and therefore in these structures.

Columns at cantilever beam connections which intersect with overhangs are subjected to high stress. This isan important reason of damage [4] as shown in Fig (2).

Figure 2: Failure due to overhang

Literature Review

Torsional irregularity is one of the most important factors, which cause severe damage to the building structures. Many studies investigated various aspects of torsional irregularity including geometric asymmetry. Duan and Chandler (1997) [5] proposed an optimized procedure for seismic design of torsionally unbalanced structures. Ozmen (2002) [6] investigated geometric and structural aspects of torsional irregularity according to (Turkish Earthquake Code TEC 2007). Demir et al. (2010) [7] investigated torsional irregularity factors which effect multi story shear wall-frame systems according to TEC2007. Six type structures which have different story numbers, plan views, and shear wall locations were analyzed. Tezcan and Alhan (2001) [8] proposed an increase in the calculated eccentricity in order to ensure an added and inherent safety for the flexible side elements. Penelis and Kappos (2002) [9] presented a methodology for modeling the inelastic torsional response of buildings in nonlinear static (pushover) analysis, aiming to reproduce the results of inelastic dynamic time history analysis. Dogangun and Livaoglu (2006) [10] examined the differences in results from equivalent seismic load method, mode-superposition method, and analysis method in time domain. They presented some recommendations related to the usage of seismic analysis methods. Jinjie et al. (2008) [11] developed a torsion angle capacity spectrum method for the performance-based seismic evaluation of irregular framed structures. Mahdi and Gharaie (2011) [12] evaluated the seismic behavior of three intermediate moment-resisting concrete space frames with unsymmetrical plan using pushover analysis. Cosenza et al. (2000) [13] compared most of the results existing in the literature, suggested proposals of modification and underlined the importance of further studies in order to evaluate a condition of minimum torsional stiffness.

Bosco et al. (2004) [14] described a study devoted to define the application limits of an approximated design method about non-regularly asymmetric systems. They anticipated that to define clear limits is possible in seismic codes for the simplified approaches on irregular structures. Zheng et al. (2004) [15] studied the criterion and relative regulations for torsional irregularity in UBC97 and Eurocode 8 (2004). The results through the codes were analyzed and compared from the theoretical and practical aspects.

Torsional Irregularity

Torsional Irregularity Factor η_t, is defined as; for any of the two orthogonal earthquake directions, the ratio of the maximum displacement at any story to the average displacement at the same story in the same direction Eq(1).

$$\eta_t = \left(\Delta_{\max} / \Delta_{\text{average}} \right) \tag{1}$$

During the application of earthquake lateral forces, minimum%5 eccentricities according to different code must be taken into account. In two orthogonal directions. The minimum $\Delta_{\min}$ and maximum $\Delta_{\max}$ floor displacements are calculated and the mean displacement is calculated as:

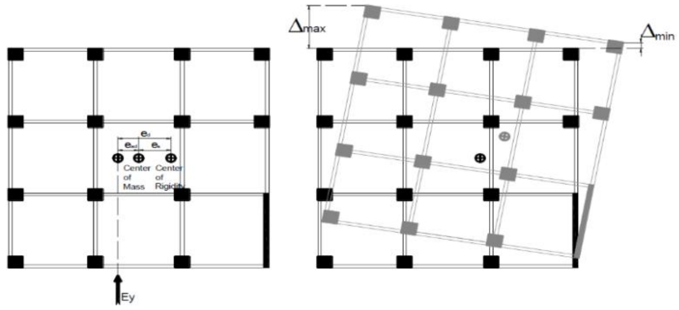

Figure 3: Calculation of maximum and minimum displacements

$$\Delta_{\text{average}} = \left(\frac{\Delta_{\max} + \Delta_{\min}}{2} \right) \tag{2}$$

The provisions of ASCE-7 (2005 & 2010) [16, 17] regarding the torsional irregularities are summarized in the following:

1- If η_t <1.2 then torsional irregularity does not exist, i.e., $A_x = 1$;

2- If 1.2 < η_t <2 then torsional irregularity exists and eccentricity amplification factor is computed by:

$$A_x = \left(\frac{\Delta_{\max}}{1.2\Delta_{average}} \right)^2 \tag{3}$$

And the design eccentricity (e_d) becomes

$$e_d = e + 0.05 * A_x \tag{4}$$

3- If η_t >1.2 take (A_x= 3.0).

Table 1 shows briefly the torsional irregularity ratio limits in the international codes and the methodology of calculation in each one. The ECP-201 (2012) and did not have any recommendation for these types of deformations, just some constrains by ECP-201 (2012) for accidental eccentricity and the projection is regarded as a percent of the building's length or area. The modern codes ASCE-7 (2005) & (2010), IS (2002), UBC (1997), TEC (2007), and NBCC (2005) are submitted allowable limits that presents the torsional irregularity deformation.

Table 1: Torsional irregularity due to different codes

Codes	Torsional irregularity	Notes
ASCE-7 (2005&2010)	$\Delta_{max} \leq 1.2\ \Delta_{average}$	Where Δ_{max} and $\Delta_{average}$ are the maximum drift computed at a particular story level, and the average of drifts computed at both sides of a structure.
IS (2002)	$\Delta_{max} \leq 1.4\ \Delta_{average}$	
UBC (97)	$\Delta_{max} \leq 1.2\ \Delta_{average}$	
TEC (2007)		
NBCC (2005)	$\Delta_{max} \leq 1.7\ \Delta_{average}$	
EC-8 (2004)	Rx> 3.33 ex Ry> 3.33 ey Rx and Ry>*ls*,	Where *Rx* and *Ry* are the torsional radius in x and y direction and *ls* is the radius of gyration.
ECP-201 (2012)	-	-

The problems related to cantilever overhangs include the followings.

- If overhangs are not located on the central axis of a building, they will create torsional irregularities and their lateral rigidity will differ from that of the floors below or above.
- The mass center of the structure is far from the ground. Heavy overhangs shift the buildings mass center upwards and remove it from the center of rigidity.

Under earthquake motion, closed projections in particular will experience critical displacements, which may lead to a partial collapse.

Description of studied buildings and used parameters

Two RC buildings, 6 and 9-storyies, are selected to represent reference low- and mid-rise buildings. The selected buildings are typical beam–column RC frame buildings with no shear walls. Both buildings have the same plan view as shown in Fig (4). The selected reference buildings are designed according to the Egyptian codes requirements [19]. The 6 and 9-story regular frame buildings are 20 m by 20 m in plan. They have 4@5 m bays along X and Y directions as shown in Fig (4), the floor plans are identical in all stories. Typical floor height is 3 m. The reference buildings do not have any irregularities.

Six different 6-storey and Six different 9-storey structural models, including the reference building, are investigated as shown in Fig (5).

The first model, or reference model, does not contain a cantilever projection and is named the "Reference Frame". The second model (Model D) contains an overhang along the one side, while (Model E) have were two projections on adjacent sides. (Model F) also contains two projections, but on two opposite sides. Three and four cantilevers are attached to the regular frames in (Models G and H), respectively. The model identifiers are provided in Fig (5). Three-dimensional mathematical models are created using the SAP2000 finite element program [20] to carry out separate linear static (ESL) and dynamic analyses (RS).

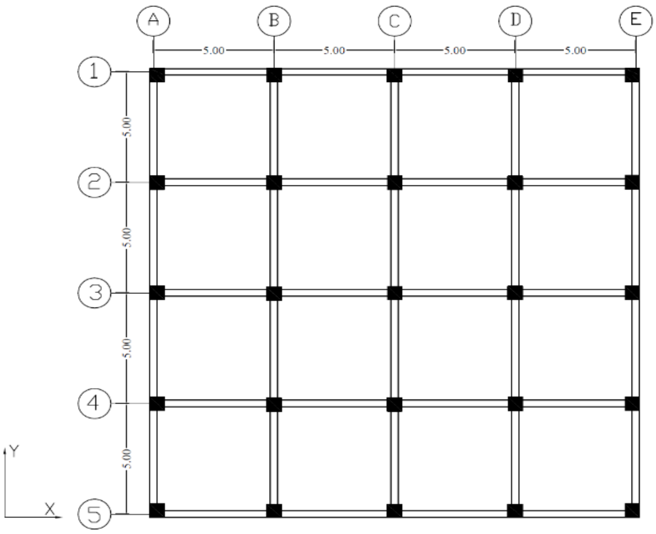

Figure 4: Plan of reference Model (Regular)

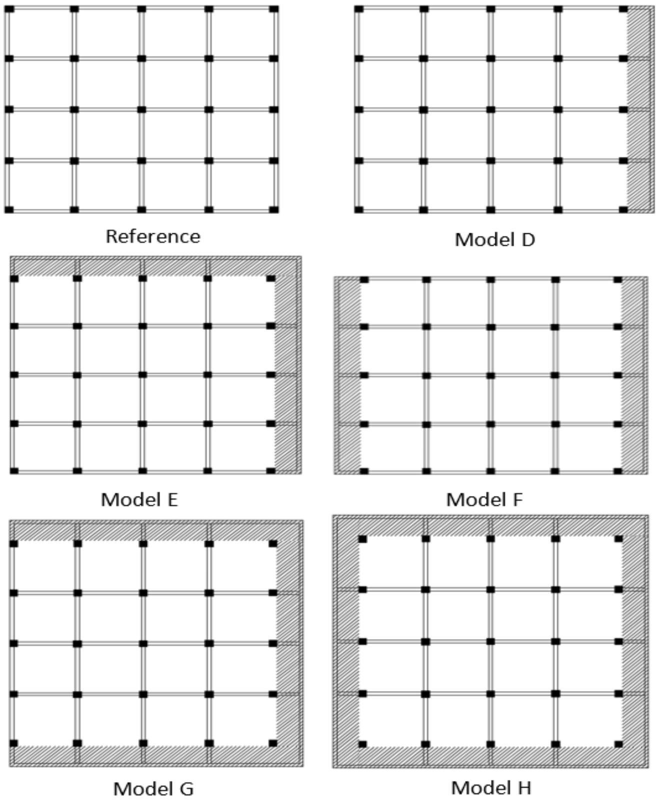

Figure 5: Plan view of building models

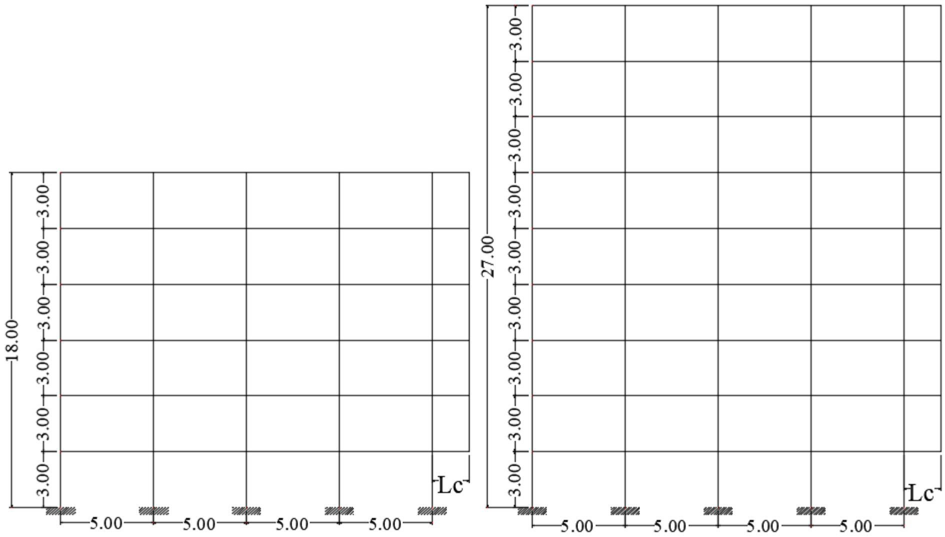

Figure 6: Elevation view of 6- and 9-story model D

The infill wall loadings were relocated on the beams surrounding the overhang portion. The overhang length was also a parameter and set at L=1 m, 2 m or 3 m at the first floor level as shown in Fig (6).

Characteristics of Model Buildings

Materials properties

Concrete having a characteristic strength f_{cu} of 25 N/mm^2 after 28-days, and high grade steel with yield strength for longitudinal rebars f_y= 360 N/mm^2 and mild steel with f_y= 240 N/mm^2 for transverse rebars (stirrups) are used for analysis and design. The specific weight of reinforcement concrete is taken as γ_c= 25 kN/m^3, modulus of elasticity *Ec* is determined using the formula $E_c = 4400\sqrt{f_{cu}}$ =22000000 KN/mm^2 (ECP-203, 2007) [19].

The elastic modulus of steel is taken as 200 KN/mm^2. Poisson's ratios υ of concrete and steel are taken equal to 0.2 and 0.3, respectively.

Table 2: Columns size and total longitudinal reinforcement

Building	Cross section (cm)	Reinforcement
6 stories	60 × 60	20 Φ16
9 stories	70 × 70	22 Φ16

Load analysis

Gravity loads

The loads that act on the RC building are categorized as gravity loads, which include dead and live loads, and +lateral loads, which include earthquake loads. The assigned values for the dead loa4851ds in terms of the weight of flooring cover and the weight of partitions (walls) of load distributed on the beam are 1.5 kN/m^2, and 6.48 kN/m respectively. The own weight of the structural elements, as a part of the dead loads is automatically computed by the used structural software package. According to the Egyptian code, The live load value for residential RC building has been assigned to be 2 kN/m^2.

Seismic loads

Analysis methods according Egyptian code are characterized as linear and nonlinear static and dynamic. The main difference between the equivalent static procedure (ESL) and dynamic analysis procedure (RS) lies in the magnitude and distribution of lateral forces over the height of the buildings. In the dynamic analysis procedure, the lateral forces are based on properties of the natural vibration modes of the building, which are determined by the distribution of mass and stiffness over height. In the equivalent lateral force procedure, the magnitude of forces is based on an estimation of the fundamental period and on the distribution of forces as given by a simple formula that is appropriate only for regular buildings.

A total seismic mass including dead loads (DL) plus 25% of live load (LL) is considered [18]. Semi-ductile moment resisting frame system is considered to carry the seismic load, therefore the response modification factor (R) is taken 5 as (Limited-ductile moment resisting frame)according to ECP-201 (2012) [18]. The seismic analysis has been carried out with the assumption of soil class 'C' as per referring to moderate dense/stiff soil; Importance factor (γ) is equal to 1.00; Seismic zone factor (Z) = 0.25g for building location (zone) (5A), Damping correction factor η =1, C_t (period factor) = 0.05 and the shape of the spectrum is a type (1).

For response spectrum method, square root of sum of squares (SSRS) is used as directional combination method and complete quadratic combination (CQC) for modal combination method. Use Ritz vector and number of mode shape to achieve more than 90% from mass participation as response spectrum condition, damping ratio (ζ) = 5% as for RC moment resisting frame building. The response spectrum curve shown in Fig (7).

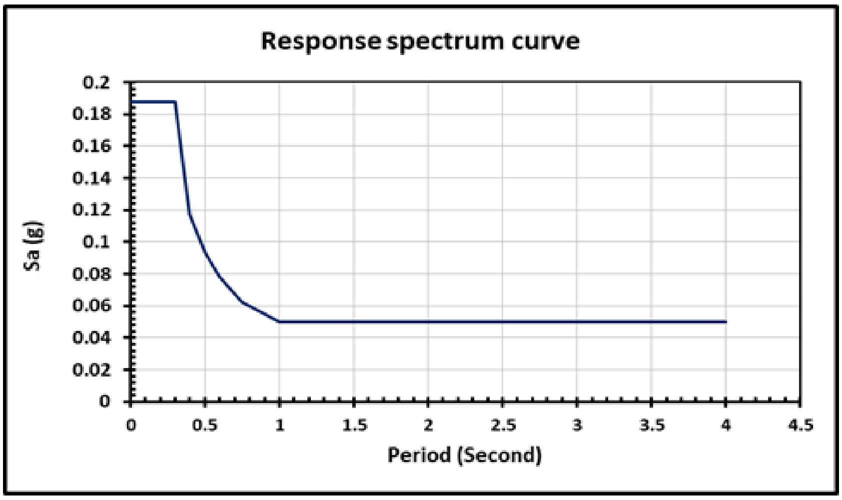

Figure 7: Response spectrum curve

Results and Discussions

Figure (8) shows the torsional irregularity factors of all 6-story building models for overhang lengths 1m, 2m and 3m. In this figure Equivalent static load analysis method was performed and the joint displacement were determined.

Torsional irregularity factors were calculated for every floor, the maximum values are represented in this figure, which also illustrates the code ASCE-7 (2005 & 2010) limit of 1.2 [16, 17]. This figure clearly shows that the code limit was exceeded for models D, G and E at every cantilever length. The torsional irregularity factors were below the code limit in models Regular, F and H. This result was quite logic because the overhangs on the opposite sides balanced the structure.

From this figure, it can be seen that as the overhang (cantilever) length increases the torsional irregularity factors increases.

Additionally, it is observed that the model G gives the highest torsional irregularity factor while the regular building gives the lowest torsional irregularity factor.

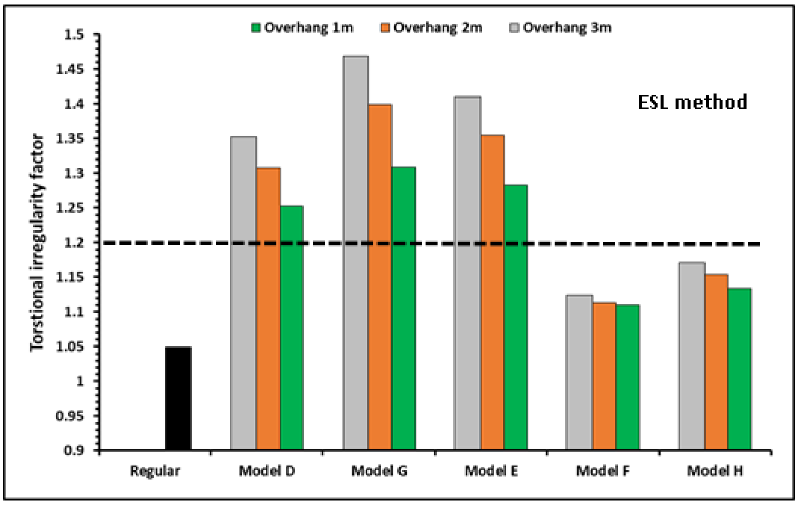

Figure 8: Torsional irregularity factor for 6-story buildings in case of ESL method

Figure (9) shows the torsional irregularity factors of all 6-story building models for overhang lengths 1m, 2m and 3m. In this figure, Response spectrum analysis method is applied and the torsional irregularity factors were determined. This figure clearly shows that the code limit was exceeded for models D at every cantilever length. In model G and E, on the other hand, the limit value was exceeded for the 2m and 3m cantilever length. The torsional irregularity factors were below the code limit in models Regular, F and H. This result was quite logic because the overhangs on the opposite sides balanced the structure.

Additionally, it is observed that model D gives the biggest torsional irregularity factor while the regular buildings the lowest torsional irregularity factor.

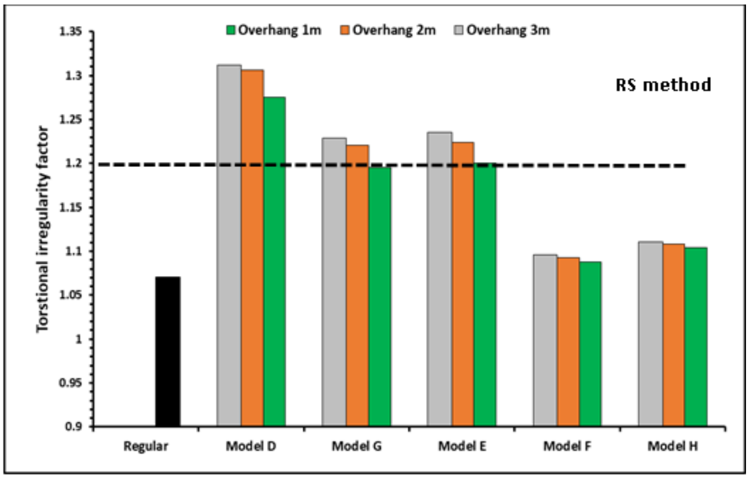

Figure 9: Torsional irregularity factor for 6-story buildings in case of RS method

Figure (10) shows the torsional irregularity factors of all 9-story building models for overhang lengths 1m, 2m and 3m. In this figure, Equivalent static load analysis method is applied and the joint drift were determined.

This figure clearly shows that the code limit was exceeded for models D, G and E at every cantilever length. The torsional irregularity factors were below the code limit in models Regular, F and H. This result was quite logical because the overhangs on the opposite sides balanced the structure.

From this figure, it can be seen that as the overhang (cantilever) length increases the torsional irregularity factors increases. Additionally, it is observed that the model G gives the biggest torsional irregularity factor while the regular building give the lowest torsional irregularity factor.

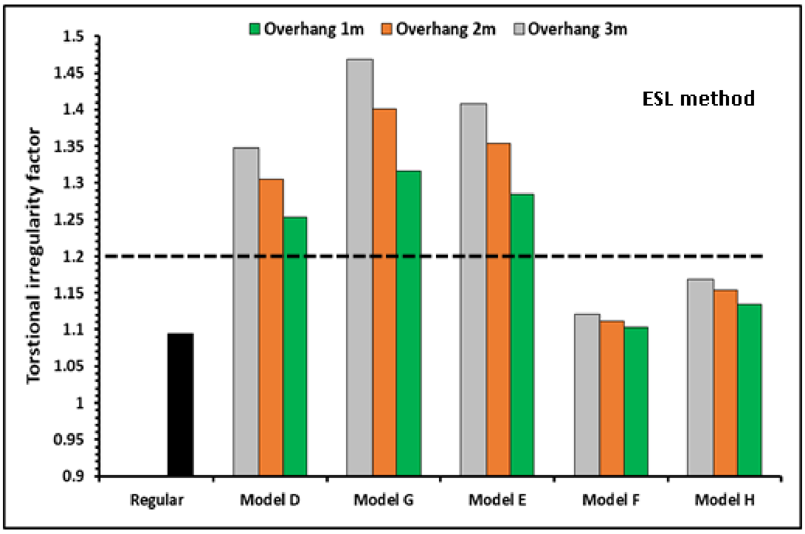

Figure 10: Torsional irregularity factor for 9-story buildings in case of ESL method

Figure (11) shows the torsional irregularity factors of all 9-story building models for overhang lengths 1m, 2m and 3m. In this figure, Response spectrum analysis method is applied and the torsional irregularity factors were determined.

This figure clearly shows that the code limit was exceeded for models D, G and E at every cantilever length. The torsional irregularity factors were below the code limit in models Regular, F and H. This result was quite logical because the overhangs on the opposite sides balanced the structure.

Additionally, it is observed that the model D give the biggest torsional irregularity factor while the regular building give the lowest torsional irregularity factor.

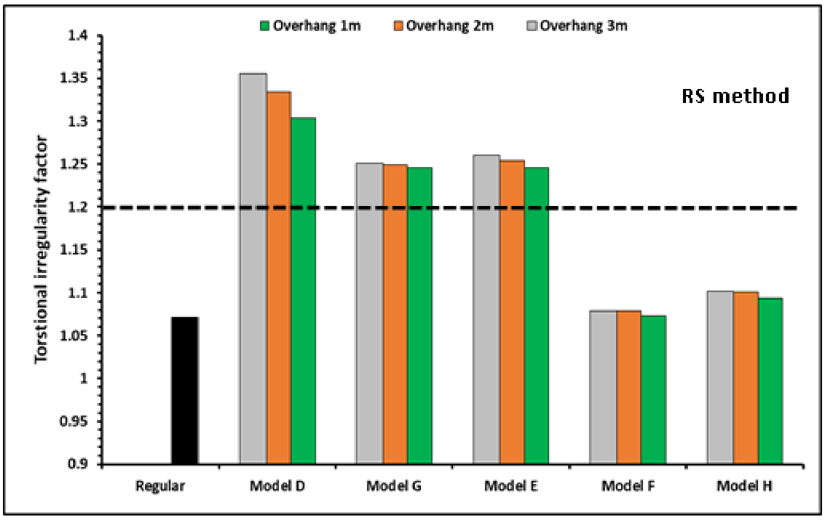

Figure 11: Torsional irregularity factor for 9-story buildings in case of RS method

The maximum torsional moment of all 6-story building models for overhang lengths 1m, 2m and 3m due to Equivalent static load analysis method occurred in the corner column on the 1-E axes, and is illustrated in Fig (12). The obtained torsional moments were normalized by dividing them by the corresponding torsional moment obtained from the regular building. The torsional moments were increased in all cases. Cantilever length had a significant effect on torsional moments. As the cantilever length increased, the torsional moments also increased. The torsional moments in model G were approximately 6 times higher than those in the regular case, while those in models D, E, H and F were 4, 5, 3 and 2 times higher than the corresponding value of reference frame, respectively.

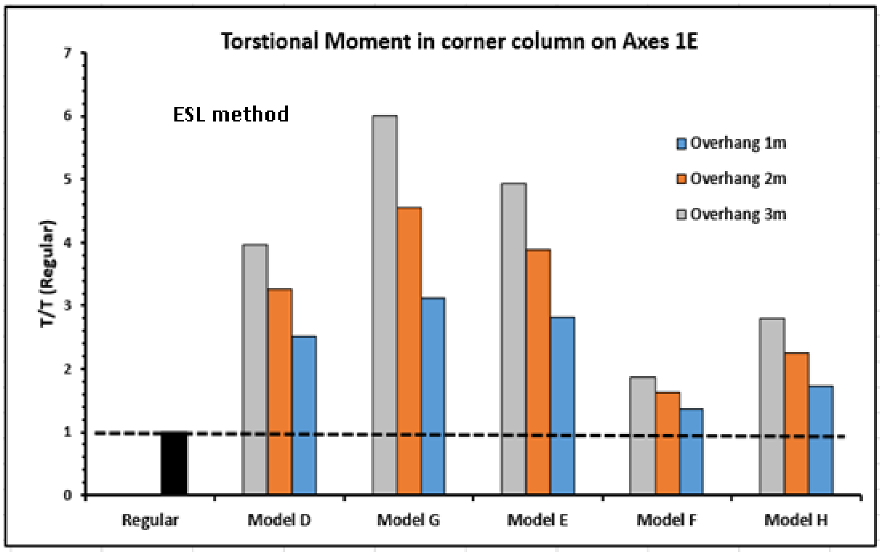

Figure 12: Torsional moment ratios for 6-story buildings with respect to the regular model in case of ESL method

Figure (13) shows the maximum torsional moment of all 6-story building models for overhang length 1m, 2m and 3m due to Response spectrum analysis method. The torsional moments were increased in all cases. Cantilever length had a significant effect on torsional moments. As the cantilever length increased, the torsional moments also increased.

The torsional moments in model G were approximately 9 times higher than those in the regular case, while those in models D, E, H and F were 7, 7.5, 2 and 1.5 times higher than the corresponding value of reference frame, respectively.

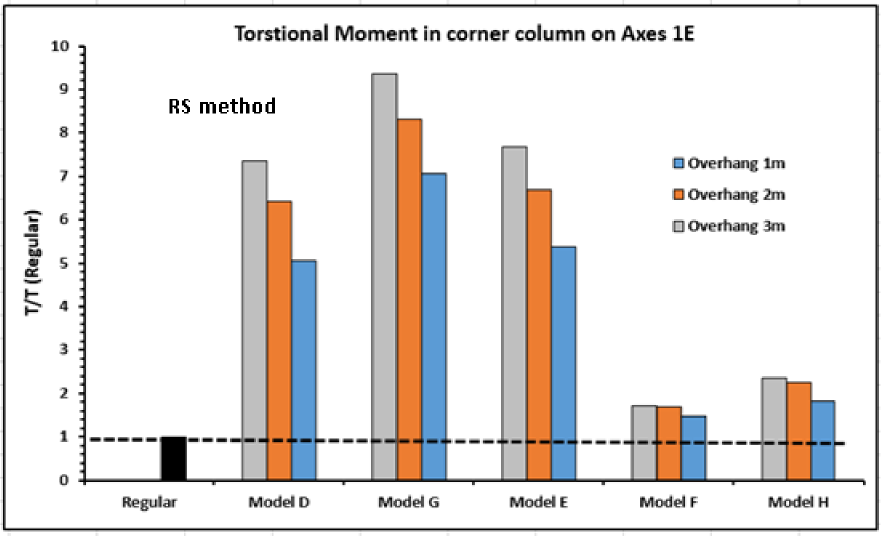

Figure 13: Torsional moment ratios for 6-story buildings with respect to the regular model in case of RS method

Figure (14) shows the maximum torsional moment of all 9-story building models for overhang length 1m, 2m and 3m due to Equivalent static load analysis method. The torsional moments were increased in all cases. Cantilever length had a significant effect on torsional moments. As the cantilever length increased, the torsional moments also increased.

The torsional moments in model **G** were approximately 6 times higher than those in the regular case, while those in models D, E, H and F were 4, 5, 2.5 and 2 times higher than the corresponding value of reference frame, respectively.

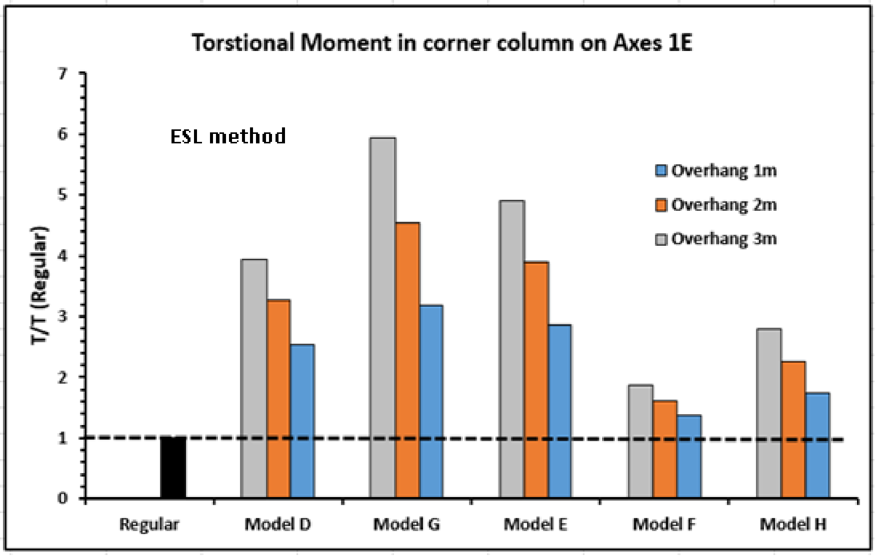

Figure 14: Torsional moment ratios for 9-story buildings with respect to the regular model in case of ESL method

Figure (15) shows the maximum torsional moment of all 9-story building models for overhang length 1m, 2m and 3m due to Response spectrum analysis method. The torsional moments were increased in all cases. Cantilever length had a significant effect on torsional moments. As the cantilever length increased, the torsional moments also increased.

The torsional moments in model G were approximately 9.5 times higher than those in the regular case, while those in models D, E, H and F were 7, 8 , 2.5 and 1.5 times higher than the corresponding value of reference frame, respectively.

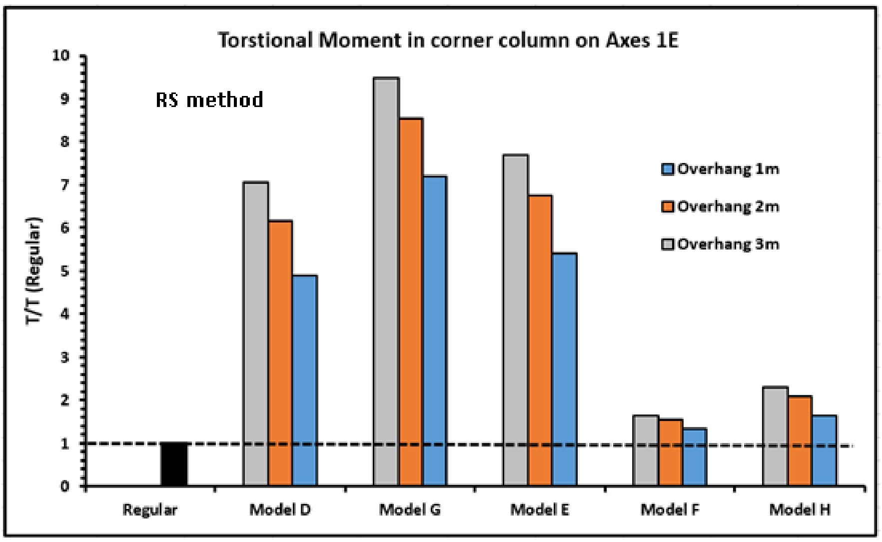

Figure 15: Torsional moment ratios for 9-story buildings with respect to the regular model in case of RS method

Table (3) shows and summarize the Torsional moment values in corner column on axis 1-E for all case.

Table 3: Torsional moment in corner column on axis 1-E

Method	Frame Height	Overhang Length	Torsional moment in corner column on axis 1-E (KN.m)					
			Regular	Model D	Model G	Model E	Model F	Model H
Equivalent static load	6-stories	1m	1.93	4.84	6.02	5.42	2.62	3.31
		2m		6.31	8.76	7.5	3.12	4.34
		3m		7.66	11.58	9.53	3.6	5.4
Equivalent static load	9-stories	1m	4.2	10.66	13.35	11.97	5.75	7.3
		2m		13.69	19.11	16.33	6.08	9.45
		3m		16.5	24.99	20.58	7.81	11.76
Response spectrum analysis	6-stories	1m	2.73	13.84	19.24	14.68	3.98	4.99
		2m		17.49	22.65	18.23	4.55	6.14
		3m		20.09	25.52	20.96	4.69	6.41
Response spectrum analysis	9-stories	1m	6.16	30.18	44.41	33.26	8.19	10.16
		2m		37.94	52.54	41.51	9.6	12.81
		3m		43.48	58.39	47.37	10.16	14.22

Conclusion

In this study a parametric investigation is performed on different types of typical structures by considering different story numbers, overhang length and overhang direction. Findings under lateral load are evaluated and the following conclusions are summarized:

- For all the investigated structures, torsional irregularity factors and torsional moment ratio increase as the overhang length increase, i.e., maximum irregularity factors occur for structures with overhang length 3m.
- The torsional irregularity factors for Case D (one overhang), Case G (three overhangs) and Case E (two overhangs on adjacent sides) exceeded the code limit of 1.2 for all overhang lengths. In Case H (four overhangs), the torsional irregularity factors fell below the limit of 1.2. A superior response was obtained in Case F (two overhangs on opposite sides) and its torsional irregularity factors values were nearly equal to those obtained for the reference model. The code upper limit of 2 was never exceeded in any case.
- For all the investigated structures, the highest torsional moment ratios was found maximum for model G. For Response spectrum analysis method the torsional moments in model **G** were approximately 9.5 times higher than those in the regular case while those in models D,E were approximately 8 times higher than the corresponding value of reference frame. If the designer does not take this large values, it will lead to a collapse the buildings.

Notation and Abbreviations

η_t torsional Irregularity Factor
Δ_{min} minimum floor displacement
Δ_{max} maximum floor displacement
$\Delta_{average}$ average floor displacement
A_x eccentricity amplification factor
e_d design eccentricity
ESL equivalent static load
RS response spectrum
CQC complete quadratic combination
SSRS square root of sum of squares
ξ damping ratio

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