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EFFECTS OF TORSION ON THE BEHAVIOUR OF NON-STRUCTURAL COMPONENTS MOUNTED ON IRREGULAR REINFORCED CONCRETE MULTI-STOREY BUILDINGS

A.B.B. Aldeka[1,†], AHC. Chan[2,‡], and S. Dirar[1,§]

[1] School of Civil Engineering, University of Birmingham
Edgbaston, Birmingham, B15 2TT, UK
[†] ABB037@bham.ac.uk
[§] s.m.o.h.dirar@bham.ac.uk

[2] School of Science, Information Technology and Engineering, University of Ballarat
Victoria, Australia, 3350 (formerly School of Civil Engineering, University of Birmingham)
[‡] achan@ballarat.edu.au

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Abstract. *This paper considers the seismic responses of acceleration-sensitive non-structural components (NSCs) attached to irregular multi-storey reinforced concrete (RC) structures using Finite Element (FE) approach. The main objective of this study is to evaluate the behaviour of NSCs mounted on irregular RC buildings due to the effects of structural ductility and torsion which take place during earthquakes. Nonlinear dynamic analyses were used to evaluate the seismic behaviour of primary-secondary systems. The P-structures in this study include five irregular RC buildings which have the same asymmetric plan configuration, while having different number of stories, i.e., 5, 7, 10, 13, and 15-storeys. The P-structures were designed by the authors according to the full seismic provisions of Eurocode 8 (EC8). Distributed inelastic fibre elements are adopted to model the beams and columns of the P-structures. To simulate NSCs in P-structures, vertical cantilever beams fixed at their bases with masses on top are modelled with variety of lengths to adjust their natural frequencies to the natural frequencies of the P-structures. Full dynamic interaction is considered between the NSCs and P-structures during the analyses. A comparison is made between the numerical results and the EC8 formulation, and a general conclusion is drawn that the provisions of EC8 for design of NSCs are un-conservative in some cases.*

1 INTRODUCTION

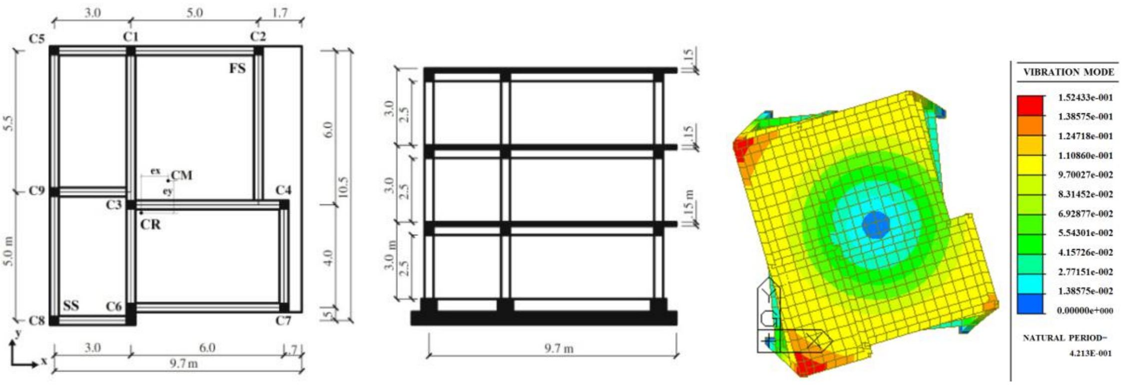
The evaluation of seismic response of non-structural components (NSCs) attached to structures began over 40 years ago with a particular focus on equipment attached to regular 2D structures [1]. Various researchers presented state of the art reviews on NSCs [1-4]. Multi-storey structures with complicated geometries have become more widespread for many reasons but mainly because of modern architectural designs and materials. In such structures with asymmetries and irregularities, strong torsional effects induced by moderate and strong base motions are usually responsible for the damage of the structure and NSCs [5]. Realizing that the torsional behaviour of a primary structure (P-structure) could have an explicit effect that may increase the seismic response of NSCs attached to the P- structure, Yang and Huang [6, 7] proposed an analytical method with and without base isolation to evaluate the seismic behaviour of light equipment in such cases. Their approach was restricted to elastic primary-secondary systems with the eccentricity of floors in one direction only.

Analytical research studies were carried out by Agrawal and Datta [8-10]. They studied the response of a secondary element mounted over inelastic one-storey primary system (P-system) with torsional effect and subjected to base motion in one direction. Their model was 2-D model with the eccentricity of the P-system in one direction [8]. They also studied the dynamic response of a secondary element mounted on a torsionally inelastic P-system under base motions in two directions. Sets of coupled inelastic differential equations were used to evaluate the hysteretic force deformation behaviour of the inelastic P-system [9, 10]. Dynamic responses of acceleration-sensitive NSCs mounted on a multi-storey RC building with torsional effect were studied using a finite element (FE) approach by the present authors [11]. A single-bay three storey reinforced concrete (RC) frame was selected as the P-structure in order to investigate the behaviour of the NSCs under different eccentricities of the P-structure. Both acceleration and displacement responses of NSCs attached to the flexible side were investigated. In the majority of the above-mentioned studies, the NSCs were mounted over either a linear P-structure with eccentricity in one direction or an inelastic single-bay torsional building with eccentricity in both directions. Usually, the P-structures are too complicated in their dimensions and the use of composite materials in their elements for an analytical solution to be available; therefore, numerical analysis has to be employed.

The main objective of this paper is to evaluate the effects of structural ductility and torsion on the behaviour of NSCs mounted on irregular RC multi-storey buildings. Numerical results of NSCs attached to multi-storey buildings with different heights are presented and compared to the provisions of Eurocode 8 (EC8) [12] for the design of NSCs.

2 RC P-STRUCTURS CHARACTERISTICS AND MODELLING

In order to represent buildings in the range of low- and medium-rise, five irregular RC moment-resisting buildings having different number of storeys, i.e., 5, 7, 10, 13, and 15-storeys, are chosen to study the behaviour of NSCs. The plan configuration of a three-storey building, namely SPEAR building [13], was used as a reference plan for designing the multi-storey buildings in this study. The SPEAR building is asymmetric in the plan in the two horizontal directions, and characterized so as to have significant torsional behaviour as shown in Figures 1(a) [13] and 1(c) respectively. The multi-storey buildings in this study were designed by the authors according to the guidelines of the modern seismic provisions of EC8 [12] considering the Medium Ductility Class (DCM) rules. The buildings are labelled as "EC8 M#"; the term "EC8 M" refers to buildings designed as per EC8 and for Medium ductility. The symbol "#" is a number and indicates the number of stories (i.e., #=5, 7, 10, 13, and 15) respectively. All buildings were assumed to be constructed on ground soil type C. However, the conclusions of this paper should apply to other soil types. A design peak-ground acceleration (a_g) of 0.25 g for moderate seismic zones is considered. Concrete Class C25/30 and steel reinforcement Class A, S400 were adopted in the design process of the five buildings. The characteristic values of loads are taken to be 2.7 and 2.0 kN/m^2 for permanent and variable actions respectively. The resulting members dimensions and the amount of longitudinal and shear steel reinforcements are shown in Table 1.



(a) (b) (c)

Figure 1: The SPEAR structure: (a) plan; (b) elevation [13]; and (c) torsional mode.

Building	Storey	Columns C1,C2,C3,C4, C5,C7,C8,C9 Cross section	Long. steel	C6 Cross section	Long. steel	Shear Steel	Beams Cross section	Long. steel	Shear steel
EC8 M5	1~2F	450x450	18Ø20	450x1000	20Ø20	Ø8 @120	350x500	9Ø16	Ø8 @90
	3~5F	400x400	14Ø20	400x850	20Ø20				
EC8 M7	1~2F	550x550	20Ø20	550x1000	22Ø20	Ø8 @110	350x500	10Ø16	Ø8 @90
	3~4F	500x500	20Ø20	500x1000	22Ø20				
	5~7F	450x450	18Ø20	450x850	20Ø20				
EC8 M10	1~2F	650x650	30Ø20	600x1000	30Ø20	Ø8 @110	350x500	13Ø16	Ø8 @90
	3~4F	600x600	30Ø20	550x1000	30Ø20				
	5~7F	550x550	28Ø20	550x1000	28Ø20				
	8~10F	500x500	22Ø20	500x850	22Ø20				
EC8 M13	1~2F	750x750	24Ø25	750x1000	26Ø25	Ø8 @90	400x600	17Ø16	Ø10 @110
	3~4F	650x650	24Ø25	650x1000	24Ø25				
	5~7F	650x650	24Ø25	650x1000	24Ø25				
	8~10F	600x600	20Ø25	600x1000	22Ø25				
	11~13F	500x500	24Ø20	500x850	26Ø20				
EC8 M15	1~2F	850x850	30Ø25	850x1000	32Ø25	Ø8 @90	450x650	13Ø20	Ø10 @100
	3~4F	750x750	30Ø25	750x1000	32Ø25				
	5~6F	700x700	30Ø25	700x1000	32Ø25				
	7~9F	650x650	28Ø25	650x1000	30Ø25				
	10~12F	600x600	24Ø25	600x1000	26Ø25				
	13~15F	500x500	18Ø25	500x850	20Ø25				

Table 1: Details of longitudinal and shear steel reinforcements of the buildings (all dimensions in mm).

The P-structures were modelled with a distributed inelastic fibre element using a FE computer code, MIDAS Ver. 3.1 [14], which leads to a very accurate description of the geometrical and mechanical properties of each element. Confined and unconfined concrete models proposed by Mander et al. [15] and the reinforcement model by Menegotto and Pinto [16] were used. Newmark method was used to integrate the equation of motion of the system performing full Newton-Raphson iterations until convergence was achieved. Constant acceleration method was adopted with the Newmark time integration parameters, γ and β, equal to 0.5 and 0.25 respectively. Modal analyses were carried out for these buildings to estimate their natural fundamental periods as shown in Table 2.

		Building				
		EC8 M5	EC8 M7	EC8 M10	EC8 M13	EC8 M15
Period [s]	T1	0.66	0.84	1.17	1.29	1.39
	T2	0.64	0.83	1.15	1.26	1.31
	T3	0.51	0.66	0.92	1.02	1.12

Table 2: Fundamental periods of the buildings under consideration.

3 NON-STRUCTURAL COMPONENTS MODELLING

In the experimental studies reported in [17], in order to simulate the NSCs in the P-structures, single-degree-of-freedom (SDOF) mechanical oscillators were used and attached to the floors and adjusted to correspond to the natural frequencies of the P-structures. In this study, the NSCs were similarly modelled as vertical cantilever beams with variety of lengths and fixed at their bases with masses on the top. Each mass was modelled as a lump mass made of steel 152x152x51mm weighing about 9.2 kg. The arms of the cantilevers were modelled as circular sections, 40 mm in diameter, with varying lengths to match the frequencies of the P-structures. Full dynamic interaction was considered between the NSCs and P-structures during the analyses.

A damping ratio of 5% [18] was used for the P-structures. However, a damping ratio of 3% [19] and frequencies matching the first (transitional in x dir.) and third (torsional) fundamental periods of the P-structures, as well as rigid NSCs (Tc=0 s.), were adopted for the NSCs in the analyses.

4 THE EARTHQUAKE RECORDS

In this study, in order to evaluate the seismic responses of NSCs during the nonlinear dynamic analyses, a set of seven artificial base motion records were used. These artificial records were generated by using SIMQKE-I code [20]. This code generates a spectral density function from response spectrum (RS) data input and then obtains sinusoidal signals having random phase angles. Response spectrum type C–EC8 with a peak ground acceleration (PGA) of 0.25 g was used as input to generate seven accelerograms with duration of 30 s. Figure 2 displays the RS of the seven artificial records and their average, which match quite well with the elastic RS of EC8 for 5% damping.

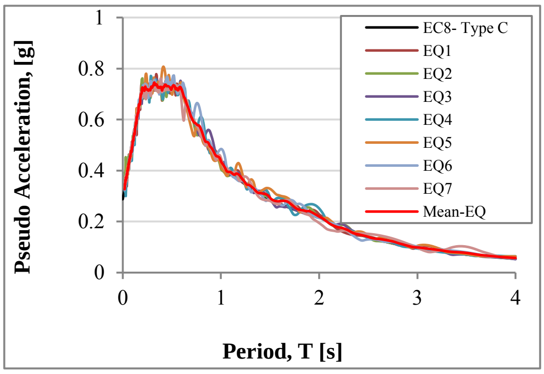


Figure 2: Artificial ground motions.

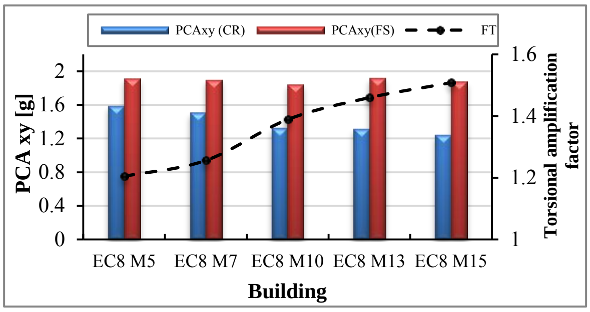
5 DYNAMIC RESPONSES OF NSCs

Eurocode 8 [12] (section 4.3.3.4.3) allows the use of the mean of the effects of seven or more base motion records on the structures, rather than the maximum effects, for analysis and design purposes. Hence, the results of this study are based on averages of seven ground motion inputs in the two horizontal directions. Dynamic analyses have been conducted to study the responses of NSCs attached to the floors of multi-storey buildings. Three periods of NSCs, T_C=0 s., T_C=T_1, and T_C=T_3 were considered in the analyses.

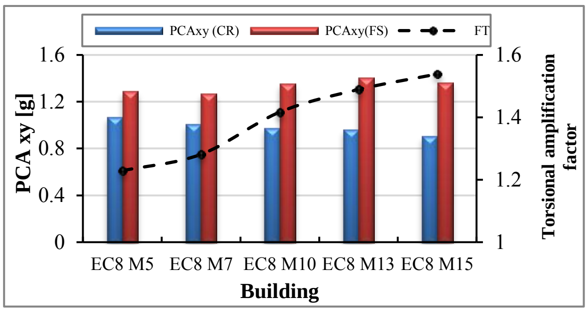
During earthquakes, in order to pick up the values of accelerations due to transitional modes only of P-structures, the values of accelerations at the centre of rigidity are recorded. Recorded values of accelerations at the flexible side of a building give accelerations due to lateral modes and any torsional modes [21]. Therefore, a relationship between these two recorded accelerations (i.e., records at centre of rigidity and flexible side of a building) gives the amplification in accelerations due to torsional modes only.

To evaluate the maximum response of NSCs attached to the P-structures, average values of peak component accelerations (PCA) and top floor rotations are calculated under the effect of base motions having a PGA that gives a value of structural ductility factor (μ) equal to 1.0. Therefore, a PGA value of 0.16 g was used as input for artificial ground motions so that the EC8 M5, EC8 M7, and EC8 M10 buildings attain their maximum elastic response (i.e., μ=1.0). A PGA value of 0.17 g was used for EC8 M13 and EC8 M15 buildings.

Figures 3(a) and 3(b) show respectively the average values of PCA_{xy} at the flexible sides (FS) and the corresponding values at centre of rigidity (CR), as well as the torsional amplification factors (F_T) for NSCs attached to the top floors of the buildings. The NSCs had periods equal to T1 and T3 of the P-structures. It can be seen form these Figures that the torsional amplification factor increases with the increase in total height of the buildings. This trend is affected by the top floor rotation patterns of the P-structures at μ equal to 1.0, as shown in Figure 4. It can be observed that the higher the value of top floor rotation, the higher the torsional amplification factor.



(a)



(b)

Figure 3: Average values of PCA_{xy} at flexible side and centre of rigidity of top floor of the buildings, and torsional amplification factors for NSCs having :(a) Tc=T1; and (b) Tc=T3.

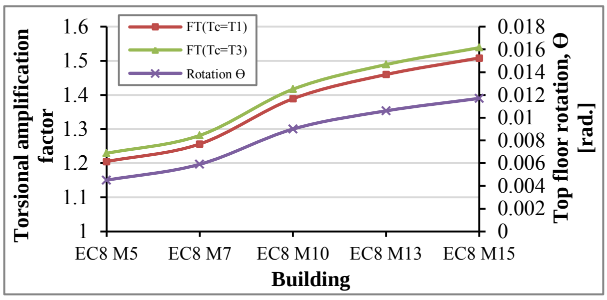


Figure 4: Torsional amplification factors of NSCs and top floor rotation values of the buildings at μ equal to 1.0.

Another trend that can be noticed in Figure 4 is that for a given building, the torsional amplification values are approximately equal for NSCs having periods equal to T1 and T3.

The results in Figures 5 to 7 illustrate the seismic response of NSCs attached along the heights of the buildings. These Figures compare the values of numerically-predicted acceleration amplification factors to the corresponding predictions of the current EC8 provisions (Eq. 4.25-EC8) for the design of NSCs. It can be seen from Figure 5 that the majority of acceleration amplification factors for rigid NSCs, T_C=0.0 s., attached along the heights of the buildings at centres of rigidity are within the range predicted by EC8. However, the acceleration amplification factors for NSCs attached to the flexible sides at the lower thirds of the buildings that have 10, 13, and 15-storey exceed the EC8 predictions. This result suggests that rigid NSCs should not be attached to such locations of the P-structures in order to avoid their failures.

For NSCs having periods equal to the first or third modes of P-structures, T_C=T1 or Tc=T3, the amplification values are higher than the corresponding values calculated using the EC8 provisions as shown in Figures 6 and 7 respectively. Figure 7 shows that, for NSCs having periods equal to the torsional modes of P-structures (T_C=T3), the acceleration amplification factors increase gradually from the lower to the higher floors. A similar trend can be observed for the floors rotation patterns of all the P-structures under consideration at μ equal to 1.0 as shown in Figure 8. This suggests that a correlation exists between the values of the NSCs amplification factors and the values of floor rotations. Further research is being carried out to investigate such a correlation.

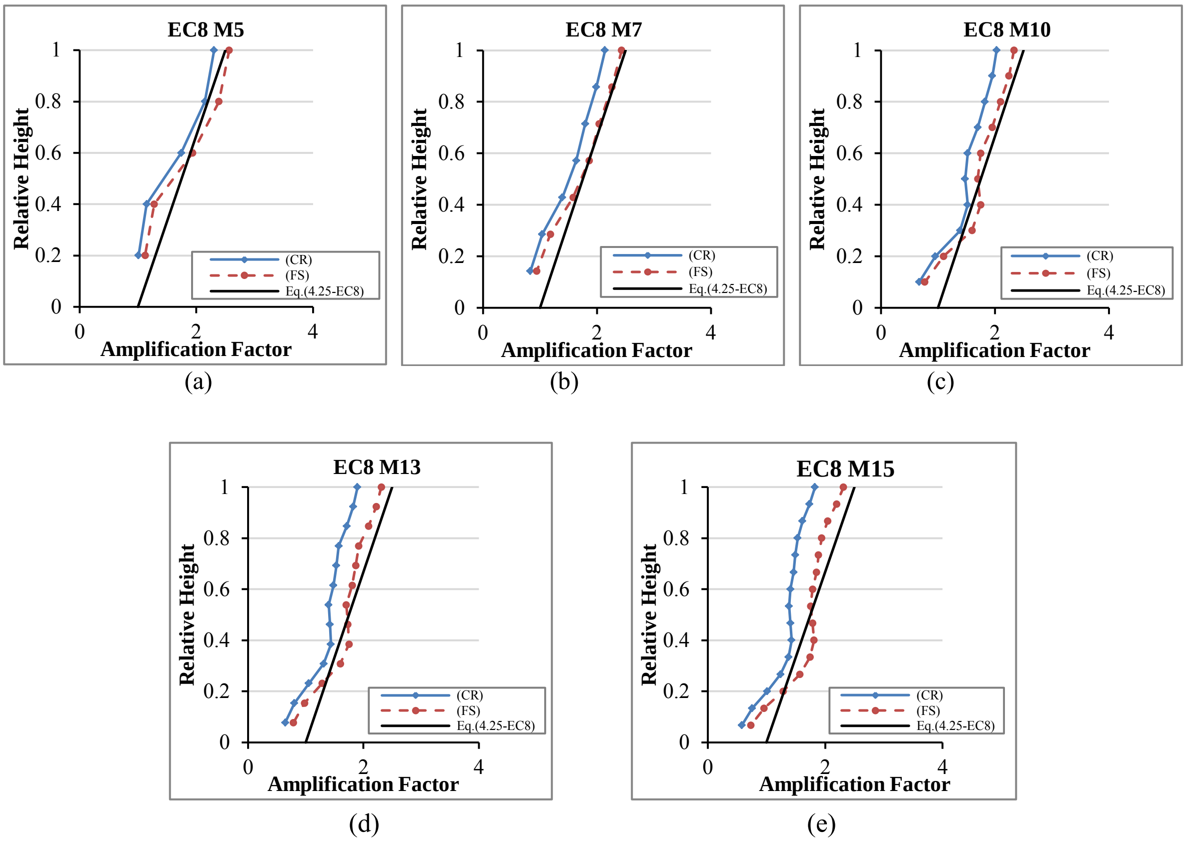


Figure 5: Comparison of the numerical results for acceleration amplification factors of NSCs with Tc=0 s to the corresponding values of EC8: (a) EC8 M5; (b) EC8 M7; (c) EC8 M10; (d) EC8 M13; and (e) EC8 M15.

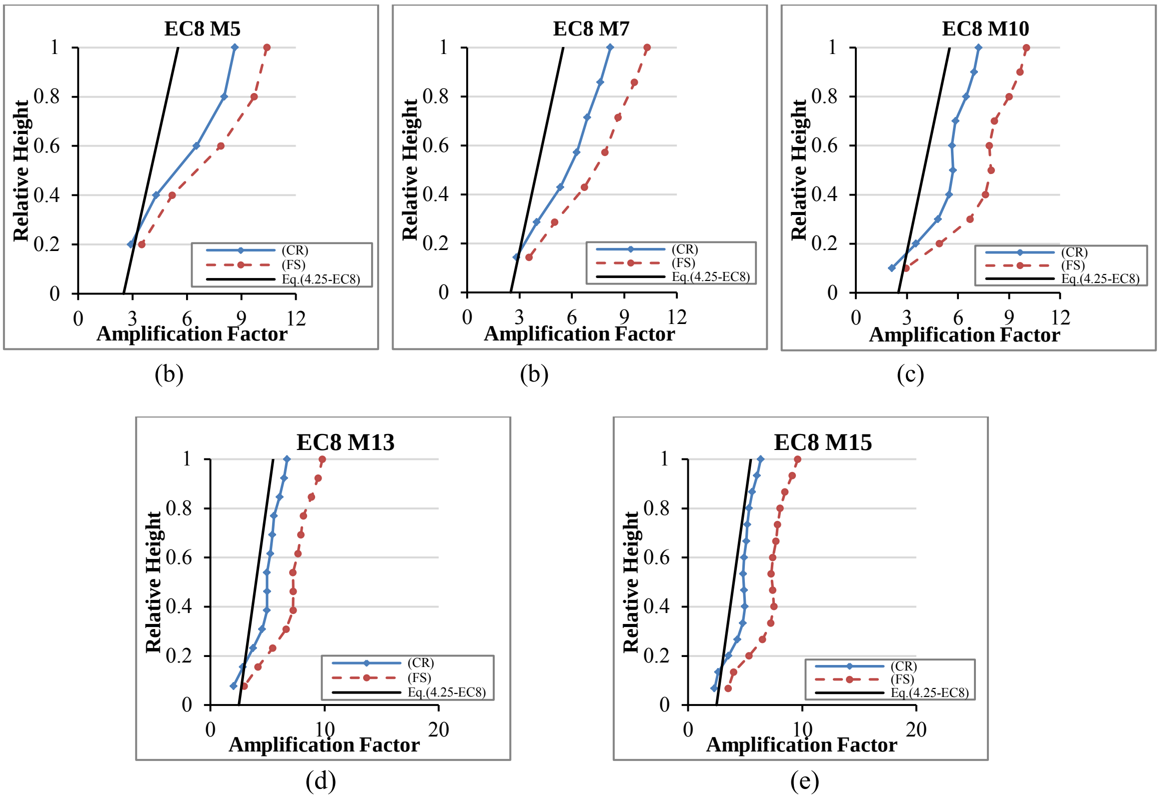


Figure 6: Comparison of the numerical results for acceleration amplification factors of NSCs with Tc=T_1 to the corresponding values of EC8: (a) EC8 M5; (b) EC8 M7; (c) EC8 M10; (d) EC8 M13; and (e) EC8 M15.

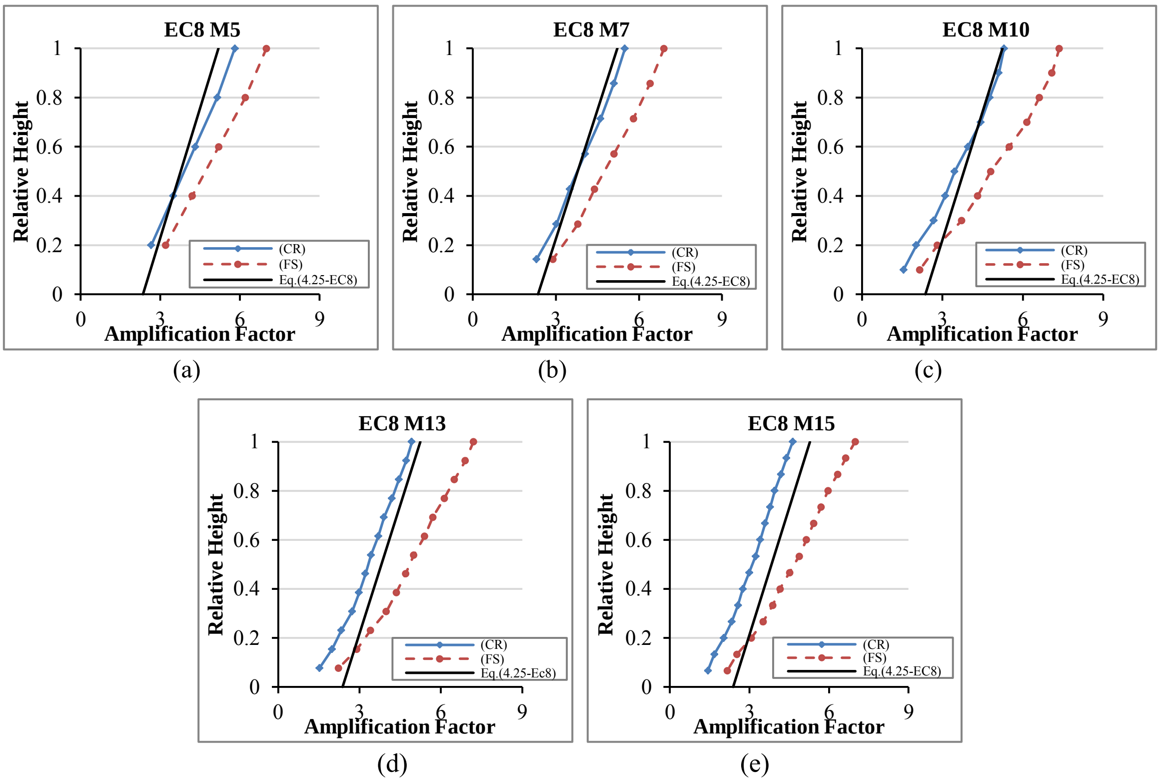


Figure 7: Comparison of the numerical results for acceleration amplification factors of NSCs with Tc=T_3 to the corresponding values of EC8: (a) EC8 M5; (b) EC8 M7; (c) EC8 M10; (d) EC8 M13; and (e) EC8 M15.

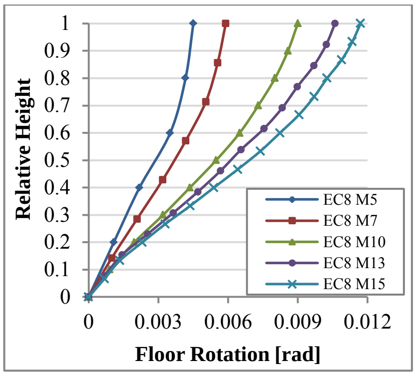


Figure 8. Average values of floor rotations of the buildings at μ equal to 1.0.

6 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the current study, the following conclusions can be drawn:

- The torsional amplification factors of NSCs increase with the increase in height of the buildings. This trend seems to be affected by the floor rotation patterns of the P-structures; the higher the value of top floor rotation, the higher the torsional amplification factor.
- For a given building, the torsional amplification values are approximately equal for NSCs having periods equal to T1 or T3 of the P-structures.
- The majority of amplification factors for rigid NSCs, T_C=0.0 s., attached along the heights of the buildings at centre of rigidity are within the range of the EC8 predictions. However, the values of amplification factors for NSCs attached to the flexible sides at the lower thirds of the buildings that have 10, 13, and 15-storeys exceed the EC8 predictions.
- For NSCs having periods equal to the first and third modes of P-structures, the NSCs amplification valuess are higher than the values calculated using the EC8 provisions.

According to the preceding numerical results and conclusions , the EC8 design provisions for NSCs need to be improved by taking into account the torsional effects of the P-structures.

REFERENCES

[1] Y.Q. Chen, T.T. Soong, Seismic Response of Secondary Systems, *Engineering Structures,* 10, 4:218-228, 1988.

[2] R. Villaverde, Seismic Design of Secondary Structures: State of the Art, *Journal of Structural Engineering-ASCE*, 123, 8:1011-1019, 1997.

[3] A. Filiatrault, C. Christopoulos, C. Stearns, Guidelines, Specifications, and Seismic Performance Characterization of Non-structural Building Components and Equipment. *Pacific Earthquake Engineering Research Centre, College of Engineering*, University of California, Berkeley, 102, 2002.

[4] A.S. Whittaker, T.T. Soong, An Overview of Nonstructural Components Research at Three U. S. Earthquake Engineering Research Centres, *Proceedings of ATC-29-2 Seminar on Seismic Design*, Performance, and Retrofit of Nonstructural Components in Critical Facilities, ATC-29-2 Report, Applied Technology Council, Redwood City, California, 271-280, 2003.

[5] A.M. Chandler, Building damage in Mexico City earthquake, *Nature*, 320, pp. 497-501, 1986.
[6] Y.B. Yang, W.H. Huang, Seismic Response of Light Equipment in Torsional Building, *Earthquake Engineering and Structural Dynamics,* 22:113-128, 1993.
[7] Y.B. Yang, W.H. Huang, Equipment-structure interaction considering the effect of torsion and base isolation, *Earthquake Engineering and Structural Dynamics,* 27:155-171, 1998.
[8] A.K. Agrawal, T.K. Datta, Behaviour of Secondary System Attached to a Torsionally Coupled Primary System, *Journal of European Earthquake Engineering,* 2:47-53, 1997.
[9] A.K. Agrawal, T.K. Datta, Seismic Response of a Secondary System Mounted on a Torsionally Coupled Non-linear Primary System, *Journal of Earthquake Engineering,* 2:339-356, 1998.
[10] A.K. Agrawal, T.K. Datta, Seismic Behaviour of a Secondary System on a Yielding Torsionally Coupled Primary System, *JSEE,* 2, 1:35-46, 1999.
[11] A.B. Aldeka, AHC. Chan, S. Dirar, Finite Element Modelling of Non-structural Components Mounted on a Torsionally Multi-storey Building, *Proceedings of the 20th UK conference of the Association for Computational Mechanics in Engineering (ACME2012)*, Manchester, United Kingdom, 67-70, 2012.
[12] EC8 Eurocode 8 EN 1998-1. Design of structures for earthquake resistance, 2004.
[13] P. Negro, E. Mola, J. Molina, and G. Magonette, Full-scale PSD testing of a torsionally unbalanced three-storey non-seismic RC Frame, *13th World conference on Earthquake Engineering*, Vancouver, Canada, No. 968, 2004.
[14] Midas Gen. Analysis Manual, version 3.1, 2012, http://www.MidasUser.com/.
[15] J.B. Mander, M.J.N. Priestley, R. Park, Theoretical stress-strain model for confined concrete, *Journal of Structural Engineering- ASCE,* 114, 8:1804-1826, 1988.
[16] M. Menegotto, P.E. Pinto, Method of analysis for cyclically loaded RC plane frames including changes in geometry and non-elastic behaviour of elements under combined normal force and bending. Symposium on the Resistance and Ultimate Deformability of Structures acted on by well defined loads, *International Association for Bridge and Structural Engineering*, Zurich, Switzerland, 15-22, 1973.
[17] J.L. Sackman, J.M. Kelly, Seismic analysis of internal equipment and components in structures, *Engineering Structure,* 1:179-190, 1979.
[18] M. Paz, International handbook of earthquake engineering: codes, programs, and examples, Springer, 1994.
[19] H. Graves, R. Morante, Recommendations for Revision of Seismic Damping Values in Regulatory Guide 1.61, 2006.
[20] SIMQKE User Manual. NISEE Software Library, University of California, Berkeley, USA, 1976.
[21] R.M. DiJulio, G.C. Hart, Torsional Response and Design of High-rise Buildings, *ULCA-ENG-7373*, Los Angeles, Calif. Univ. (L.A.) Sch. of Engineering & Appl. Sci. [Report], 1974.