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Influence of Ground Type on the Seismic Response of Non-structural Components Integrated on Asymmetrical Reinforced Concrete Buildings

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Abstract

This paper studies the effect of ground type (namely ground types A, B, C, D and E as defined in Eurocode 8 (EC8)) on the seismic response of lightweight acceleration-sensitive non-structural components (NSCs) integrated on irregular multi-storey reinforced concrete (RC) structures. Dynamic nonlinear finite element (FE) analyses of the primary-secondary (P-S) systems were conducted to provide insight into the seismic response of the NSCs and to evaluate the accuracy of EC8 predictions when the NSCs are attached to the flexible sides of the primary structures (P-structures). Representative constitutive models were adopted to represent the behaviour of the RC P-structures. The NSCs were modelled as vertical cantilevers fixed at their bases with masses on the free ends and varying lengths so as to match the frequencies of the P-structures. Full dynamic interaction is considered between the NSCs and P-structures. Different sets of natural and artificial earthquake records consisting of 63 accelerograms were utilised. The results of this paper confirm the outcome of previous studies suggesting that the current EC8 provisions underestimate the dynamic response of NSCs influenced by the torsional modes of RC P-structures designed on different ground types.

Keywords: dynamic analysis, Eurocode 8, finite element, non-structural components, reinforced concrete buildings, torsion, irregular building

1. INTRODUCTION

The term "non-structural components" (NSCs) refers to secondary systems (S-systems) integrated on a structure but are not considered as part of the main structural elements of the structure. As NSCs are affected by the seismic response of the primary structure (P-structure) under the effect of earthquakes (Whittaker and Soong, 2003), it is important for safety and economic purposes to investigate the effects of seismic loadings and dynamic interaction of the primary-secondary systems (P-S systems) on the dynamic response of the NSCs. The review of the literature on the response of the S-systems (Chen and Soong, 1988; Phan and Taylor, 1996; Villaverde, 1997; Whittaker and Soong, 2003) reveals that very limited numerical studies have focused on the response of NSCs attached to inelastic multi-storey irregular reinforced concrete (RC) structures experiencing significant torsional modes due to irregularities in the plan and/or vertical mass.

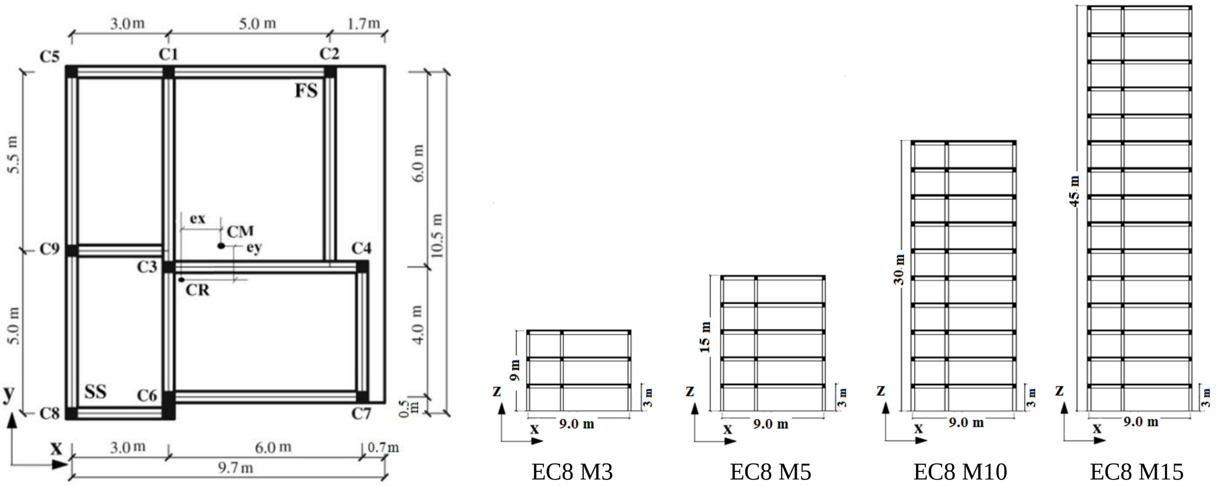
Evaluation of the primary systems (P-systems) seismic performance requires either experimental work or numerical modelling. Attributable to the large dimension of civil engineering structures such as multi-storey structures, nuclear power stations, and long-span bridges, physical tests are not usually feasible. Hence, analytical and numerical simulations are most often used for seismic structural assessment. However, due to novel materials, advanced construction techniques and modern architectural requirements, the structural layouts and composite materials used in present-day P-structures are too complicated for an analytical solution to be available. Therefore, one viable solution to bridge the knowledge gap in the area of seismic response of S-systems mounted on large civil structures is to use advanced numerical methods such as finite element (FE) analysis. This paper presents three-dimensional nonlinear dynamic FE analyses of NSCs mounted on inelastic irregular RC multi-storey structures designed on different types of ground. The NSCs considered in this study are lightweight acceleration-sensitive equipment. In order to increase the confidence in the results of a previous study performed by Aldeka et al. (2014), twenty cases of asymmetrical RC P-structures designed for construction on ground types A, B, C, D, and E (EC8 2004) were adopted. The main aim of this paper is to investigate the influence of ground type on the seismic response of non-structural components integrated on the selected buildings. The FE computer code MIDAS Gen Ver. 3.1 (2012) was used for implementation of the dynamic analyses of the P-S systems. In Aldeka et al. (2014), a validation of this code was conducted for the dynamic analysis of irregular RC frame structures with significant torsional behaviour. Furthermore, the influences of NSC to P-structure vibration period ratio, peak ground acceleration (PGA), NSC to P-structure height ratio, and P-structure torsional behaviour on the seismic response of NSCs were investigated in the study by Aldeka et al. (2014).

2. CHARACTERISTICS AND MODELLING OF RC P-STRUCTURES

Four irregular RC P-structures (EC8 M3, EC8 M5, EC8 M10 and EC8 M15) were chosen in this paper. These buildings have the same plan layout of "SPEAR" structure (Negro et al., 2004) but differ in the total height and type of ground assumed for construction. Figure 1(a) shows the plan of the RC buildings where the eccentricities between their centres of mass (CM) and centres of rigidity (CR) are in two directions. The maximum values of the eccentricity ratios in the X and Y directions of the studied buildings were found equal to 0.157 and 0.145 respectively. The terms SS and FS presented in Figure 1(a) refer respectively to the stiff and flexible sides of the P-structures. The typical floor height is 3 m and the total heights of these buildings are in the range of 9-45 m as shown in Figure 1(b). The buildings are labelled as "EC8 M#". The term "EC8 M" refers to buildings designed as per Eurocode 8 (2004) Ductility Class M (DCM). The symbol "#" indicates the number of stories (i.e. # = 3, 5, 10, or 15). Five types of ground; namely A, B, C, D, and E as defined in EC8 (2004); were adopted. Therefore, a total of 20 RC P-structures were considered. Elastic response spectra consistent with EC8 Type 1 Response Spectrum (RS) for the above-mentioned ground types were used in the design. Based on EC8 (2004), Table 1 shows description of each type of ground that used in the design of the P-structures.

The design of the selected RC buildings satisfied the provisions of EC8 (2004). It can be summarised as follows: DCM with value of the behaviour factor (q) equal to 3.45; design acceleration (a_g) on Type A ground equal to 0.25 g; concrete class C25/30 for beams and columns and steel class C S500 for steel reinforcement. The characteristic values used for the floor loads were 2.7 kN/m^2 and 2.0 kN/m^2 for permanent and variable actions respectively. Considering the soil factors of 1.0, 1.2, 1.15, 1.35, and 1.4 for ground types A, B, C, D, and E respectively, the design ground accelerations on these types of ground were equal to 0.25 g, 0.30 g, 0.29 g, 0.34 g, and 0.35 g respectively. The resulting member dimensions and the amount of longitudinal and shear steel reinforcements of the P-structures can be found in Appendix A (see Tables A1 to A5). For each studied building in this paper, Table 2 shows the maximum seismic capacity determined according to Annex B of EC8 (2004).

A distributed inelastic fibre element was used to model the structural members. This modelling approach produces a very precise simulation of the geometrical and mechanical characteristics of the structural elements. The concrete and steel models proposed by Mander et al. (1988) and Menegotto and Pinto (1973) respectively were used. A damping ratio of 5% (Paz, 1994) is adopted for the P-structures. Further details are available in Aldeka et al. (2014).



(a) (b)

Figure 1 Irregular RC buildings (a) plan (Negro et al., 2004) and (b) elevation (all dimensions are in metres).

Table 1 Description of ground types; namely A, B, C, D, and E (EC8, 2004).

Ground type	Description of ground
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.
C	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.
E	A soil profile consisting of a surface alluvium layer and thickness varying between about 5 m and 20 m, underlain by stiffer material.

Table 2 Maximum seismic capacities of the irregular RC buildings.

Building	EC8 M3					EC8 M5					EC8 M10					EC8 M15				
Ground type	A	B	C	D	E	A	B	C	D	E	A	B	C	D	E	A	B	C	D	E
Max. seismic capacity [g]	0.69	0.72	0.76	0.83	0.79	0.64	0.68	0.74	0.78	0.74	0.57	0.59	0.63	0.70	0.63	0.50	0.54	0.58	0.64	0.58

3. CHARACTERISTICS AND MODELLING OF NSCs

The NSCs considered in this study are lightweight acceleration-sensitive mechanical, electrical or medical equipment. Normally only the fundamental mode of such NSCs is of importance therefore they can be modelled as cantilevers fixed at their bases. Single-degree-of-freedom (SDOF) mechanical oscillators are commonly used to model such NSCs (Agrawal and Datta, 1998; Opropeza et al., 2010). A modelling approach similar to that used by Sackman and Kelly (1979) and Aldeka et al. (2014) was adopted in this study where the NSCs were modelled as vertical cantilevers fixed at their bases with masses on the free ends. Full dynamic interaction is considered between the NSCs and P-structures. Value of damping ratio (ξ_c) equal to 3% (Graves and Morante, 2006) is adopted for the NSCs. Near-resonance response is most critical for the P-S systems, especially in the elastic range of the P-structures (Aldeka et al., 2014). Therefore, fundamental periods of the NSCs matching the first fundamental periods of the RC buildings were adopted in the analyses. Table 3 shows the characteristics (i.e. the lengths and lateral stiffness of the cantilever arms) of the NSCs integrated on the selected RC buildings. The values of T_1 in Table 3 are the natural periods of the considered NSCs which are equal to the first vibration periods of the studied RC P-structures. The value of the vibration period of each P-structure as presented in Table 3 was calculated using a modal analysis with the FE computer code MIDAS Gen (2012). It can be seen that, as a result of different cross-section dimensions of the columns and beams of the P-structures designed on different ground types (see Tables A1 to A5 in Appendix A), the values of the vibration periods of the P-structures were varied on different types of ground.

Table 3 Characteristics of the NSCs attached to the selected RC buildings.

Building	EC8 M3					EC8 M5					EC8 M10					EC8 M15				
Ground type	A	B	C	D	E	A	B	C	D	E	A	B	C	D	E	A	B	C	D	E
T_1 [s]	0.62	0.59	0.55	0.47	0.52	0.75	0.71	0.66	0.61	0.66	1.25	1.22	1.17	1.08	1.17	1.50	1.45	1.39	1.28	1.39
L_a [m]	1.19	1.15	1.10	0.99	1.06	1.35	1.31	1.24	1.18	1.24	1.90	1.87	1.82	1.73	1.82	2.15	2.10	2.04	1.93	2.04
K_a [N/m]	951	1053	1204	1651	1345	651	713	840	975	840	234	245	266	309	266	161	173	189	223	189

4. EARTHQUAKE RECORDS

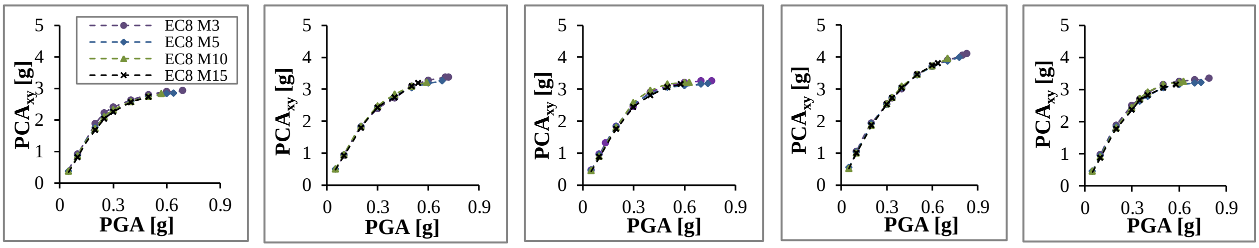
As per the EC8 (2004) provisions, average effects of at least seven artificial, recorded or simulated earthquake records should be used for nonlinear analysis purposes. So as to increase confidence in the results of this investigation, different sets of natural and artificial earthquake records consisting of 63 accelerograms were used. These records were scaled so that the average of their response spectrum in the range of periods between $0.2T_1$ and $2T_1$ matches with the EC8 (2004) Type 1 elastic RS for ground types A, B, C, D, or E. The period T_1 represents the first fundamental vibration period of the P-structure. Of these 63 records, three sets of natural earthquakes consisting of 56 records compatible with Type 1 RS for ground types A, B, C, and D of EC8 (2004) were used. For ground type E, seven artificial accelerograms were employed. The natural records were extracted from the European Strong-motion Database (ESD) using the computer code REXEL Ver. 3.2 (beta) (Iervolino et al., 2010). The artificial records were generated using SIMQKE code (Gelfi, 2007). Artificial accelerograms compatible with Type 1 RS for ground type E were selected instead of natural earthquake records due to the shortage of natural records for this type of ground in the ESD (Iervolino et al., 2010).

5. DYNAMIC RESPONSE OF NSCs

The results presented in this section are based on averages of the selected earthquake records as detailed in section 4. The values of the peak component accelerations (PCA_{xy}) were calculated as the square root of the sum of the squares (SQRSS) of PCA_x and PCA_y.

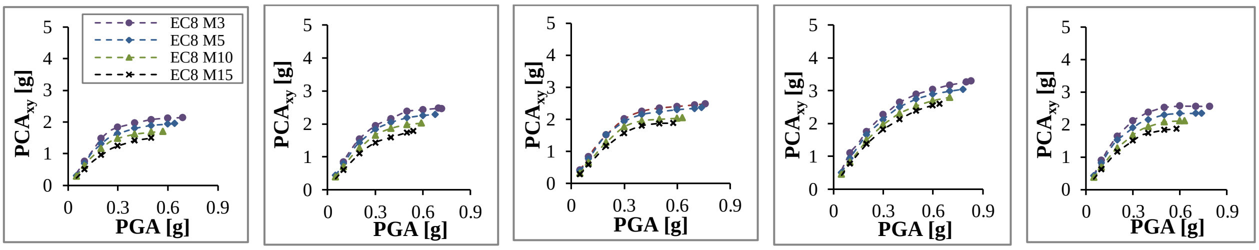
5.1 Effect of peak ground acceleration

Nonlinear dynamic FE analyses of the P-S systems were performed under the effect of peak ground accelerations (PGA) in the range between a value of 0.05 g and the maximum seismic capacity of each building (see Table 2). Shown in Figures 2 and 3 are the variations of PCA_{xy} with PGA for the NSCs with T_1 (see Table 3) and attached to the flexible sides (FS) and centres of rigidity (CR) respectively (see Figure 1(a)) of the top floors of the selected RC buildings. The legends used in Figures 2(a) and 3(a) apply to the remaining curves in Figures 2 and 3 respectively. Due to the increase in the fundamental vibration period of the P-structures with the increase in their heights (see Table 3), the response of the NSCs should be reduced. However, Figure 2 shows that the NSCs attached to the FS of buildings EC8 M3, EC8 M5, EC8 M10, and EC8 M15 had approximately the same acceleration response when these P-structures were designed for construction on a given type of ground. It seems from this result that the NSCs attached to the flexible side of the P-structures have been affected by the torsional behaviours of the buildings in addition to the lateral accelerations. The NSCs attached to the flexible side of taller buildings were more significantly affected by the torsional behaviour than those attached to the flexible side of shorter buildings as torsional rotation increases significantly with the increase of the building height. This trend is explained in Section 5.2 where the values of the torsional amplification factors (F_T) were found higher for NSCs attached to the taller buildings than for those mounted on shorter buildings. Moreover, due to the increase in the fundamental vibration periods of the P-structures with the increase in their heights as presented in Table 3, the acceleration response of the NSCs attached to the centres of rigidity (see Figure 1(a)) of the top floors of the RC buildings reduced with the increase in height of the P-structures as shown in Figure 3. These results suggest that the NSCs attached to the centres of rigidity were not affected by the torsional behaviour of the P-structures.



(a) (b) (c) (d) (e)

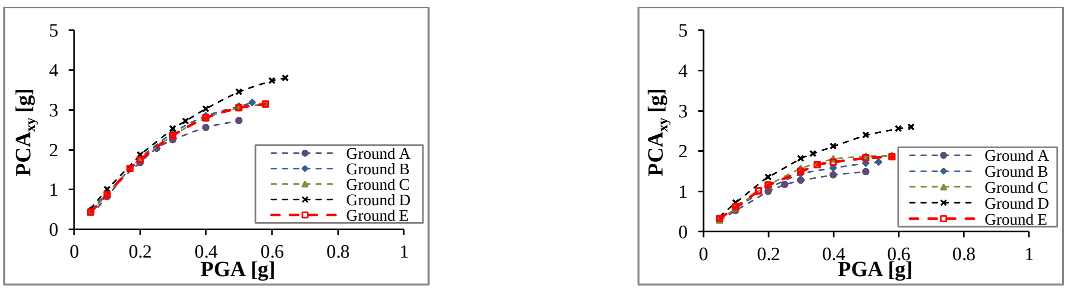
Figure 2 Variation of PCA_{xy} vs. PGA for NSCs having a period equal to T_1 and attached to the FS of the top floors of the RC buildings designed to be constructed on ground types (a) A, (b) B, (c) C, (d) D, and (e) E.



(a) (b) (c) (d) (e)

Figure 3 Variation of PCA_{xy} vs. PGA for NSCs having a period equal to T_1 and attached to the CR of the top floors of the RC buildings designed to be constructed on ground types (a) A, (b) B, (c) C, (d) D and (e) E.

To explain the effect of ground type on NSCs accelerations, Figure 4 shows the variations of PCA_{xy} vs. PGA for those NSCs attached to the FS and CR of the top floor of EC8 M15 building designed on different ground types. It can be seen from Figure 4 that the minimum and maximum values of PCA_{xy} were found for the NSCs attached to the buildings designed on ground types A and D respectively. Under the effect of the PGA values corresponding to the maximum seismic capacities of the P-structures, NSCs acceleration values equal to 2.73 g and 3.8 g were observed for the NSCs attached to the FS of the buildings designed on ground types A and D respectively. However, these two values were found respectively equal to 1.5 g and 2.6 g for the NSCs attached to CRs of the top floor of EC8 M15 building as shown in Figure 4(b). Comparable results were obtained for the NSCs attached to EC8 M3, EC8 M5, and EC8 M10.



(a) (b)

Figure 4 Variations of PCA_{xy} vs. PGA for NSCs having a period equal to T_1 and attached to the EC8 M15 building designed on different types of ground: (a) FS and (b) CRs.

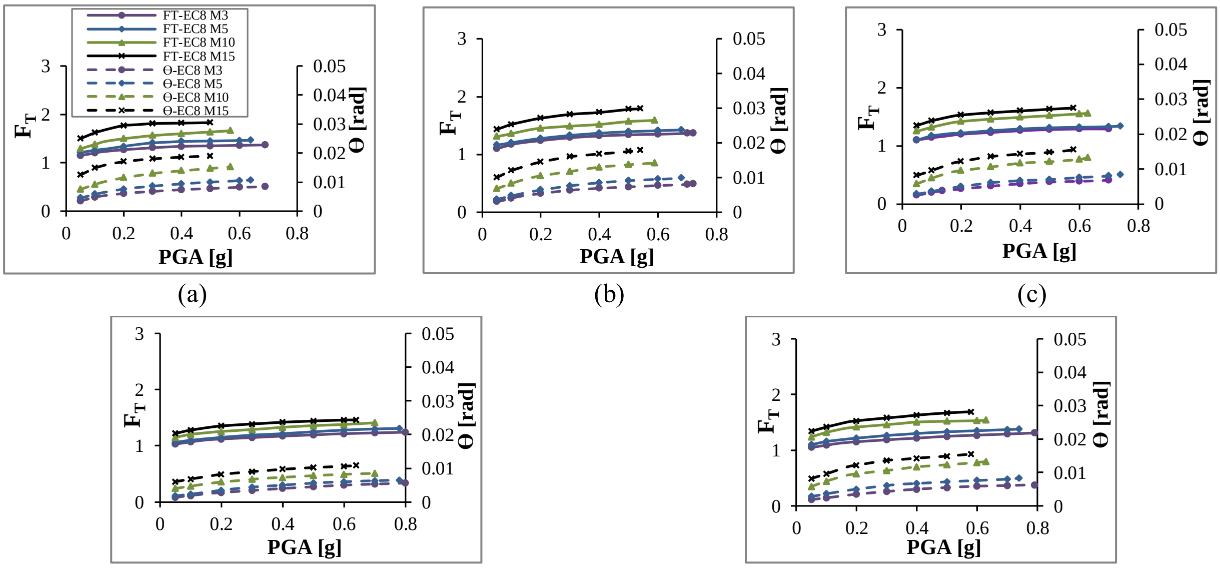
In general, the values of PCA_{xy} at the flexible sides of the P-structures designed on ground type D were higher than the corresponding values on ground types A, B, C and E by about 40.0%, 21.8%, 22.0%, and 22.2% respectively. It is worth to note that the NSCs accelerations were found to be equal for the P-structures designed on ground types C and E (see Figures 4(a) and 4(b)). The main reason of this trend is that the P-structures designed on ground types C and E had approximately equal maximum seismic capacities (see Table 2).

5.2 Effect of the torsional behaviour of the RC P-structures

During earthquakes, the accelerations recorded at the CR of a P-structure are due to transitional modes only of the P-structures. Recorded values of the accelerations at the FS of a building give accelerations due to lateral modes and any torsional modes (Hart et al., 1975). Therefore, the torsional amplification factor (F_T) for the NSCs is defined as the ratio of the peak component acceleration at the flexible side ($PCA_{xy,FS}$) to the corresponding value at the centre of rigidity ($PCA_{xy,CR}$), i.e. ($F_T = PCA_{xy,FS}/PCA_{xy,CR}$) (Aldeka et al., 2014).

As the P-structures considered in this paper have the same plan layout, but different heights and cross-sectional dimensions in the beams and columns, values of the top floors rotations (θ) rather than the dimensions of the buildings were adopted to evaluate the torsional amplification factor F_T. The values of θ were estimated during the implementation of nonlinear dynamic analyses of the P-S systems under the effect of PGA values in the range between 0.05 g and the maximum seismic capacities of the P-structures.

For the NSCs with vibration periods, T_C equal to T_1 and attached to the top floors of the RC buildings, Figure 5 shows the variations of F_T and top floor rotation (θ) of the P-structures with PGA. The legend used in Figure 5(a) applies to the remaining curves in Figure 5.



(d) (e)

Figure 5 Variations of F_T and θ with PGA for the NSCs with T_C=T_1 attached to the buildings designed on ground types: (a) A, (b) B, (c) C, (d) D, and (e) E.

It can be seen from Figures 5(a) to 5(e) that, for buildings designed on different types of ground, both the values of the torsional amplification factor of the NSCs and top floor rotation of the P-structures increased with the increase in the values of PGA. Furthermore, it can be observed from Figure 5 that at a given value of PGA, the values of F_T and θ increased with the increase in total height of the P-structures. In addition, it can be seen that the NSCs with T_C = T_1 and attached to the flexible side of the top floor of EC8 M15 designed on ground type A had a maximum value of F_T equal to 1.83, which was produced due to the maximum value of the top floor rotation of 0.019 rad as shown in Figure 5(a). The minimum value of F_T was 1.25 for the NSCs attached to the flexible side of EC8 M3 designed on ground type D which had a minimum value of θ equal to 0.0057 rad as displayed in Figure 5(d).

For a given P-structure, Figures 5(a) to 5(e) suggest that there is a strong correlation between F_T and θ. Figure 6 shows that the relationship between F_T and θ may be expressed as shown in Eq. (1) when the NSCs are attached to P-structures designed on grounds types A, B, C, D, or E.

$$F_T = 43.13\,\theta + 1.0 \tag{1}$$

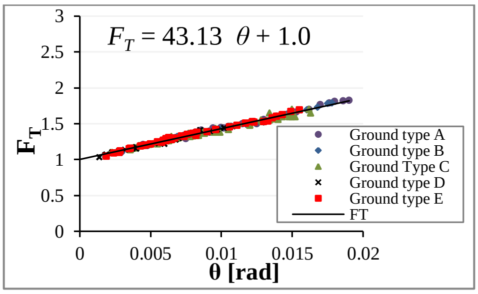


Figure 6 Relationship between the values of F_T for NSCs with T_1 and the values of θ for RC buildings designed in full compliance with the EC8 provisions on ground types A, B, C, D, and E.

Eq. (1) is valid for both symmetrical (non-torsional) and asymmetrical (torsional) P-structures. For a regular building that does not experience floor rotations during seismic loading, the proposed equation predicts a value of F_T equal to 1.0.

6. COMPARISON BETWEEN FE RESULTS AND EC8 RECOMMENDATIONS

In this section, comparisons are made between the predictions of EC8 (i.e. Eq. 4.25 of EC8 (2004)) and the numerical results of the NSCs attached to the RC buildings designed on different types of ground. All buildings were designed for an a_g value of 0.25 g on ground type A. Considering the soil factors of 1.0, 1.2, 1.15, 1.35, and 1.4 for ground types A, B, C, D, and E respectively, the design ground accelerations on these types of ground were 0.25 g, 0.30 g, 0.29 g, 0.34 g, and 0.35 g respectively. Therefore, for comparison purposes between the FE results and the predictions of EC8 (2004), the adopted earthquake records were scaled in such a way that their PGAs were equal to the above-mentioned values of design ground accelerations.

Values of PCA at roof level equal to 1.375 g, 1.65 g, 1.6 g, 1.87 g, and 1.925 g are predicted by the EC8 provisions for the NSCs having a period equal to T_1 and attached to P-structures designed on ground types A, B, C, D, and E respectively. Finite element results of PCA_{xy} recorded at the flexible and centre of rigidity regions of the top floors of the considered buildings for those NSCs having a period equal to T_1 are shown in Figure 7. The legend used in Figure 7(a) applies to the remaining results in Figure 7.It can be observed from Figure 7 that, in general, EC8 underestimates the NSCs accelerations at the design ground acceleration values. EC8 (2004) underestimated the NSCs accelerations on average by about 34.8%, 32.1%, 35.9%, 31.5%, and 28.2% when they were attached to the flexible sides of the top floors of the buildings designed on ground types A, B, C, D, and E respectively.

Only few cases of the numerical results were comparable to the predictions of EC8, especially for those NSCs attached to the centres of rigidity of the top floors of buildings EC8 M10 and EC8 M15.

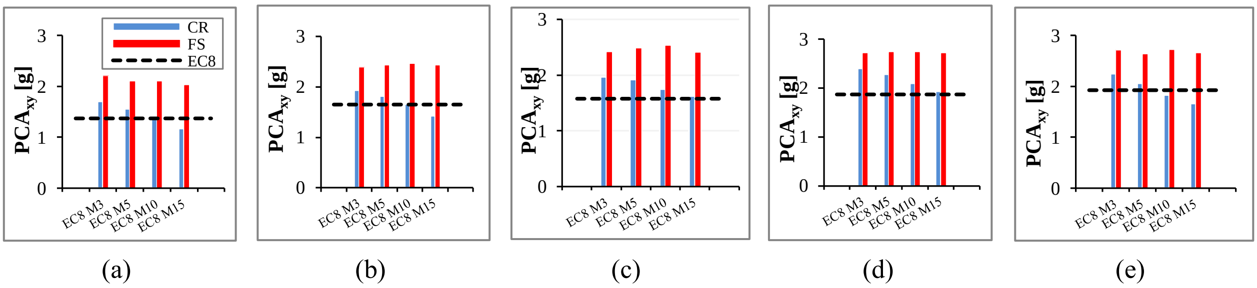


Figure 7 Comparison between the numerical results and the predictions of EC8 for the accelerations of the NSCs having a period equal to T_1 and attached to the FS and CR of the top floors of buildings designed on ground types (a) A, (b) B, (c) C, (d) D, and (e) E.

7. CONCLUDING REMARKS

In this paper, a general relationship has been developed (see Eq. (1)) between the torsional amplification factor of NSCs (F_T) and the torsional response of irregular P-structures designed on different types of ground. As the P-structures considered in this paper had an irregular plan configuration, values of the floor rotation (θ) were adopted to represent their torsional behaviour during base motions. The proposed relationship between F_T and θ allows predictions the amplifications in the NSCs accelerations due to torsional response when the NSCs are attached to the flexible sides of asymmetrical buildings. The results showed that the torsional amplification factor increased with the increase in the height of the buildings. This trend seems to be affected by the floor rotation values of the P-structures which increased with the increase in height of the P-structures.

For a given ground type, the NSCs had approximately the same acceleration response when they were attached to the flexible sides of the irregular RC P-structures with different heights. However, the NSCs had accelerations that were inversely proportional to the heights of the P-structures when they were attached to the centres of rigidity. This result suggests that the NSCs attached to the flexible sides of taller buildings were more affected by the torsional behaviour than those NSCs attached to the flexible sides of shorter buildings.

Although the NSCs accelerations had comparable trends when they were attached to P-structures designed on ground types A, B, C, D, and E, the magnitudes of the NSCs accelerations were affected by these types of ground. The minimum and maximum values of PCA_{xy} were found for the NSCs attached to the buildings designed on ground types A and D respectively. This result can be related to the maximum seismic capacities of the P-structures (see Table 2). For a given P-structure designed on different ground types, the higher the value of the maximum seismic capacity, the higher the NSCs acceleration.

The results of this paper together with the findings of Aldeka et al. (2014), where NSCs were attached to the flexible sides along the heights of irregular RC P-structures with different total heights (9 m, 15 m, 21 m, 30 m, 39 m and 45 m), suggest that EC8 (2004) underestimates the seismic responses of the NSCs with $T_C = T_1$. It is envisaged that the main reason of this discrepancy is that EC8 does not explicitly consider the increase in the NSCs accelerations at the FS caused by the torsional behaviour of the P-structure. One possible approach to improve EC8 predictions is to use the torsional amplification factor (see Eq. (1)) to take into account the amplification in NSCs accelerations caused by the P-structure torsional behaviour.

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REFERENCES

Agrawal, A. and Datta, T. (1998) Seismic response of a secondary system mounted on a torsionally coupled non-linear primary system, Journal of Earthquake Engineering, Vol 2, No 3, pp 339-356.

Aldeka, A., Chan, AHC. and Dirar, S. (2014) Response of non-structural components mounted on irregular RC buildings: comparison between FE and EC8 predictions, Earthquakes & Structures, Vol 6, No 4, pp 351-373.

Chen, Y. and Soong, T. (1988) State of the art review seismic response of secondary systems, Engineering Structures, Vol 10, No 4, pp 218-228.

EC8 (2004) EN 1998-1 Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings, European Committee for Standardization, Brussels, Belgium.

Gelfi, P. (2007) SIMQKE_GR, Programma per la generazione di accelerogrammi artificiali spettro-compatibili. Italy: University of Brescia.

Graves, H. and Morante, R. (2006) Recommendations for revision of seismic damping values in regulatory guide 1.61, Report BNL-NUREG-77174, Office of Nuclear Regulatory Research, Washington, DC: U.S. Nuclear Regulatory Commission.

Hart, G. C., Lew, M. and DiJulio, R. M. (1975) Torsional response of high-rise buildings, Journal of the Structural Division, Vol 101, No 2, pp 397-416.

Iervolino, I., Galasso, C. and Cosenza, E. (2010) REXEL: Computer aided record selection for code-based seismic structural analysis, Bulletin of Earthquake Engineering, Vol 8, No 2, pp 339-362.

Mander, J., Priestley, M. N. and Park, R. (1988) Theoretical stress-strain model for confined concrete, Journal of Structural Engineering-ASCE, Vol 114, No 8, pp 1804-1826.

Menegotto, M. and Pinto, P. E. (1973) 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, In Proceedings of IABSE Symposium on the Resistance and Ultimate Deformability of Structures Acted on by Well Defined Repeated Loads: International Association for Bridge and Structural Engineering, Zurich, Switzerland, pp 15-22.

MIDAS-Gen (2012). Analysis manual. http://manual.midasuser.com.

Negro, P., Mola, E., Molina, F. J. and Magonette, G. E. (2004) Full-scale PSD testing of a torsionally unbalanced three-storey non-seismic RC frame, In Proceedings of the 13th World Conference on Earthquake Engineering (WCEE), August 1-6, Vancouver, B.C., Canada, 15pp.

Oropeza, M., Favez, P. and Lestuzzi, P. (2010) Seismic response of nonstructural components in case of nonlinear structures based on floor response spectra method. Bull Earthquake Engineering, Vol 8, pp 387-400.

Paz, M. (1994) International handbook of earthquake engineering: codes, programs, and examples, New York: Chapman & Hall.

Phan, L. T. and Taylor, A. W. (1996) State of the art report on seismic design requirements for nonstructural building components, Report NISTIR-5857, Gaithersburgh, MD: National Institute of Standards and Technology.

Sackman, J. L. and Kelly, J. M. (1979) Seismic analysis of internal equipment and components in structures. Engineering Structures, Vol 1, No 4, pp 179-190.

Villaverde, R. (1997). Seismic design of secondary structures: state of the art. Journal of Structural Engineering, Vol 123, No 8, pp 1011-1019.

Whittaker, A. and Soong, T. (2003) An overview of nonstructural components research at three US earthquake engineering research centers, In Proceedings of Seminar on Seismic Design, Performance, and Retrofit of Nonstructural Components in Critical Facilities: Applied Technology Council-Mulyidisciplinary Center for Earthquake Engineering Research, Redwood City, California, pp 271-280.

APPENDIX A

The information provided in this appendix is the cross-section details of the buildings designed on ground types A, B, C, D, and E (all dimensions are in millimetres).

Table A1 Cross-section details of the buildings designed on ground type A.

Building	Storey	Columns						Beams		
		C1,C2,C3,C4, C5,C7,C8,C9		C6		Shear hoops (critical region)	Joint shear hoops	Cross section	Long. Steel: bottom* top$^+$	Shear hoops
		Cross section	Long. steel	Cross section	Long. steel					
EC8 M3 (9 m high)	1-3	300×300	8∅20	300×750	16∅20	2∅8@90	3∅8@80	300×450	4Ø16* 4Ø16$^+$	Ø8 @90
EC8 M5 (15 m high)	1-2	350×350	10∅20	350×780	14∅20	2Ø8 @80	3Ø8 @70	300×500	5Ø16* 3Ø16$^+$	Ø8 @90
	3-5	300×300	10∅20	300×700	14∅20					
EC8 M10 (30 m high)	1-2	500×500	18∅20	500×900	22∅20	3Ø8 @70	3Ø8 @65	300×500	7Ø16* 4Ø16$^+$	Ø8 @90
	3-4	450×450	18∅20	450×825	18∅20					
	5-7	400×400	14∅20	400×725	16∅20	2Ø8 @70				
	8-10	375×375	10∅20	375×600	12∅20					
EC8 M15 (45 m high)	1-2	675×675	20∅25	675×1000	22∅25	3Ø8 @65	3Ø8 @65	350×550	7Ø20* 4Ø20$^+$	Ø8 @80
	3-4	600×600	20∅25	600×800	22∅25					
	5-6	575×575	18∅25	575×725	18∅25					
	7-9	525×525	16∅25	525×650	16∅25					
	10-12	475×475	12∅25	475×550	12∅25					
	13-15	400×400	10∅25	400×450	10∅25	2Ø8 @65				

Table A2 Cross-section details of the buildings designed on ground type B.

Building	Storey	Columns						Beams		
		C1,C2,C3,C4, C5,C7,C8,C9		C6		Shear hoops (critical region)	Joint shear hoops	Cross section	Long. Steel: bottom* top$^+$	Shear hoops
		Cross section	Long. steel	Cross section	Long. steel					
EC8 M3 (9 m high)	1-3	335×335	10∅20	335×800	18∅20	2Ø8 @100	3Ø8 @90	335×450	4Ø16* 4Ø16$^+$	Ø8 @90
EC8 M5 (15 m high)	1-2	400×400	12∅20	400×850	18∅20	2Ø8 @100	3Ø8 @90	350×500	5Ø16* 4Ø16$^+$	Ø8 @90
	3-5	350×350	12∅20	350×750	16∅20					
EC8 M10 (30 m high)	1-2	550×550	20∅20	550×1000	24∅20	3Ø8 @90	3Ø8 @80	350×500	7Ø16* 5Ø16$^+$	Ø8 @90
	3-4	500×500	20∅20	500×900	20∅20					
	5-7	450×450	16∅20	450×800	18∅20	2Ø8 @90				
	8-10	400×400	10∅20	400×700	14∅20					
EC8 M15 (45 m high)	1-2	750×750	22∅25	750×1100	26∅25	3Ø8 @80	3Ø8 @80	350×600	7Ø20* 5Ø20$^+$	Ø8 @90
	3-4	675×675	22∅25	675×900	26∅25					
	5-6	625×625	22∅25	625×800	22∅25					
	7-9	575×575	18∅25	575×700	20∅25					
	10-12	525×525	16∅25	525×625	14∅25					
	13-15	450×450	12∅25	450×500	14∅25	2Ø8 @80				

Table A3 Cross-section details of the buildings designed on ground type C.

Building	Storey	Columns: C1,C2,C3,C4, C5,C7,C8,C9		Columns: C6		Columns: Shear hoops (critical region)	Columns: Joint shear hoops	Beams: Cross section	Beams: Long. Steel: bottom* top+	Beams: Shear hoops
		Cross section	Long. steel	Cross section	Long. steel					
EC8 M3 (9 m high)	1-3	350×350	8ø20	350×850	16ø22	1Ø8 @120	1Ø8 @120	350×450	5Ø16* 4Ø16+	Ø8 @120
EC8 M5 (15 m high)	1-2	450×450	16Ø20	450×1000	20Ø20	2Ø8 @120	3Ø8 @100	350×500	5Ø16* 4Ø16+	Ø8 @90
	3-5	400×400	16Ø20	400×850	20Ø20					
EC8 M10 (30 m high)	1-2	650×650	30Ø20	650×1200	34Ø20	3Ø8 @110	3Ø8 @90	350×500	8Ø16* 5Ø16+	Ø8 @90
	3-4	600×600	30Ø20	600×1100	30Ø20					
	5-7	550×550	24Ø20	550×950	28Ø20	2Ø8 @110				
	8-10	500×500	16Ø20	500×800	22Ø20					
EC8 M15 (45 m high)	1-2	850×850	30Ø25	850×1250	32Ø25	3Ø8 @90	3Ø8 @90	450×650	7Ø20* 6Ø20+	Ø8 @100
	3-4	750×750	30Ø25	750×1000	32Ø25					
	5-6	700×700	28Ø25	700×900	28Ø25					
	7-9	650×650	24Ø25	650×800	24Ø25					
	10-12	600×600	20Ø25	600×700	18Ø25					
	13-15	500×500	16Ø25	500×550	16Ø25	2Ø8 @90				

Table A4 Cross-section details of the buildings designed on ground type D.

Building	Storey	Columns: C1,C2,C3,C4, C5,C7,C8,C9		Columns: C6		Columns: Shear hoops (critical region)	Columns: Joint shear hoops	Beams: Cross section	Beams: Long. Steel: bottom* top+	Beams: Shear hoops
		Cross section	Long. steel	Cross section	Long. steel					
EC8 M3 (9 m high)	1-3	410×410	14ø20	410×950	26ø20	2Ø8 @140	3Ø8 @130	400×500	6Ø16* 5Ø16+	Ø8 @100
EC8 M5 (15 m high)	1-2	580×580	26ø20	580×1150	20ø20	2Ø8 @140	3Ø8 @130	400×500	5Ø18* 4Ø18+	Ø8 @100
	3-5	500×500	24ø20	500×950	20ø20					
EC8 M10 (30 m high)	1-2	750×750	26ø25	750×1300	34ø20	3Ø8 @120	3Ø8 @110	400×500	7Ø18* 4Ø18+	Ø8 @100
	3-4	700×700	26ø25	700×1200	30ø20					
	5-7	630×630	20ø25	630×1100	28ø20	2Ø8 @120				
	8-10	575×575	14ø25	575×900	22ø20					
EC8 M15 (45 m high)	1-2	1000×1000	40ø25	1000×1300	40ø25	3Ø8 @120	3Ø8 @110	450×650	8Ø20* 6Ø20+	Ø8 @110
	3-4	935×935	40ø25	935×1200	40ø25					
	5-6	875×875	38ø25	875×1000	38ø25					
	7-9	800×800	36ø25	800×900	36ø25					
	10-12	750×750	32ø25	750×850	32ø25					
	13-15	600×600	22ø25	600×650	22ø25	2Ø8 @120				

Table A5 Cross-section details of the buildings designed on ground type E.

Building	Storey	Columns: C1,C2,C3,C4, C5,C7,C8,C9		Columns: C6		Columns: Shear hoops (critical region)	Columns: Joint shear hoops	Beams: Cross section	Beams: Long. Steel: bottom* top+	Beams: Shear hoops
		Cross section	Long. steel	Cross section	Long. steel					
EC8 M3 (9 m high)	1-3	390×390	10ø22	390×900	18ø22	2Ø8 @130	3Ø8 @120	375×500	6Ø16* 4Ø16+	Ø8 @90
EC8 M5 (15 m high)	1-2	450×450	16Ø20	450×1000	20Ø20	2Ø8 @120	3Ø8 @100	350×500	5Ø16* 4Ø16+	Ø8 @90
	3-5	400×400	16Ø20	400×850	20Ø20					
EC8 M10 (30 m high)	1-2	650×650	30Ø20	650×1200	34Ø20	3Ø8 @110	3Ø8 @90	350×650	8Ø16* 5Ø16+	Ø8 @90
	3-4	600×600	30Ø20	600×1100	30Ø20					
	5-7	550×550	24Ø20	550×950	28Ø20	2Ø8 @110				
	8-10	500×500	16Ø20	500×800	22Ø20					
EC8 M15 (45 m high)	1-2	850×850	30Ø25	850×1250	32Ø25	3Ø8 @90	3Ø8 @90	450×650	7Ø20* 6Ø20+	Ø8 @100
	3-4	750×750	30Ø25	750×1000	32Ø25					
	5-6	700×700	28Ø25	700×900	28Ø25					
	7-9	650×650	24Ø25	650×800	24Ø25					
	10-12	600×600	20Ø25	600×700	18Ø25					
	13-15	500×500	16Ø25	500×550	16Ø25	2Ø8 @90				