16th World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 1894
Registration Code: S-B1463071376

ON THE SEISMIC DESIGN OF ONE-STORY PRECAST STRUCTURES FOR P-Δ EFFECTS

M. Ercolino [(1)], C. Petrone [(2)], G. Magliulo [(3)], A. Guerra [(4)], G. Manfredi [(5)]

[(1)] *Senior Lecturer, Department of Engineering Science, University of Greenwich, Chatham, UK, m.ercolino@ greenwich.ac.uk*
[(2)] *Research Associate, Department of Civil, Environmental and Geomatic Engineeering, University College London, London, UK, c.petrone@ucl.ac.uk*
[(3)] *Assistant Professor, Department of Structures for Engineering and Architecture, University of Naples, Italy, gmagliul@unina.it*
[(4)] *Graduate Researcher, Department of Structures for Engineering and Architecture, University of Naples, Italy, antonella.guerra89@gmail.com*
[(5)] *Full Professor, Department of Structures for Engineering and Architecture, University of Naples, Italy, gamanfre@unina.it*

Abstract

P-Δ effects can reduce the seismic safety of structures under seismic actions and they can be vital for one-story precast buildings because of the large flexibility of columns. According to European building code, P-Δ effects can be taken into account by following some design requirements. Such requirements can significantly influence the design of the structures since they may provide the amplification of the seismic demand by means of the stability factor as well as the oversizing of the elements.
This study investigates the influence of P-Δ effects on the seismic performance of precast one-story structures. An extensive parametric study is performed on one-story precast structures by varying some geometric features of the structure and the seismicity level of the site. Nonlinear dynamic analyses are performed by using Newmark's method on all the case studies and the results of the first order and second order analysis are compared and discussed. Moreover, different design approaches are adopted in order to assess the Eurocode provisions for P-Δ effects.
The results of the nonlinear dynamic analyses demonstrate that the overstrength, due to seismic detailing of columns and the materials overstrength, induces very low ductility demand for the structures. Indeed, even if P-Δ effects are totally neglected in the design phase, the overstrength due to other code prescriptions (e.g. minimum longitudinal reinforcement ratio) can still induce low ductility demand. Moreover it is demonstrated that the code prescriptions on P-Δ effects do not generally increase the structural safety and an alternative design approach is proposed, which gives both safer and cheaper structures than the ones currently designed according to Eurocodes.

Keywords: P-Delta effects, precast structures, nonlinear analysis, design requirements

1 Introduction

P-Δ effects can significantly influence the seismic response of flexible structures: under earthquakes actions, the gravity loads acting on the deformed configuration lead to a displacement amplification. Therefore, these effects can be significant for flexible structures, such as one-story precast industrial buildings.

In the last decades, several authors carried out research studies in order to investigate P-Δ effects on one-story and multi-story buildings. Firstly, research studies focused on defining when P-Δ effects are negligible [1] [2]. In these works, the authors defined whether P-Δ effects had to be considered by means of a stabi1ity coefficient approach; i.e. some limit values of these coefficients were proposed. The stability coefficient mainly depends on the lateral stiffness of the structure, the ductility demand, and the axial loads. According to the results of other studies, the magnitude of P-Δ effects can be taken into account by amplifying the seismic effects. In Bernal [3] the amplification factor was defined as the ratio of the strength required to a SDOF system in order to have a given peak displacement ductility demand, with and without P-Δ effects. This amplification factor was strictly correlated to the stability coefficient that is defined as the ratio between the axial load and the product of the lateral stiffness and the structure height.

The current European building code [4] considers the magnitude of P-Δ effects according to the above-described research studies, i.e. through the stability coefficient, and it provides additional design prescriptions if P-Δ effects are not negligible for the considered structure. The influence of P-Δ effects on the seismic response is evaluated by means of the stability coefficient, θ, according to the EC8:

$$\theta = \frac{P}{H} \cdot \frac{d_r}{V} \tag{1}$$

where P is the gravity load, V is the total seismic shear, H is the story height and d_r is the design inter-story drift. EC8 provides different prescriptions depending on the value of this factor, described in the following.

- P-Δ effects are taken into account if θ is larger than 0.1: if θ is smaller than 0.2 the seismic effects should be amplified by a factor:

$$\alpha = \frac{1}{1-\theta} \tag{2}$$

- The stability factor cannot exceed 0.3: in this case, the structural elements have to be re-designed.
- If θ is larger than 0.1, the cross-section dimensions of primary seismic columns should not be smaller than one tenth of the larger distance between the point of contraflexure and the ends of the column, i.e. the shear span. In the case of one-story precast structures, the column shear span corresponds to the column height.
- Eurocode does not provide any prescription in case θ is in the range 0.2÷0.3.

Both the influence of P-Δ effects on the design of precast buildings and the lack of studies on real structure (i.e. designed according to the code provisions) motivated the presented work. This research study aims at defining the influence of P-Δ effects on the behavior of one-story precast buildings. An extensive parametric study is performed on one-story precast structures by varying both some geometric features of the structures and the seismicity level of the site. Moreover, different design approaches are assumed in order to evaluate the reliability of the EC8 code approach by investigating the efficiency of each provision.

2 Parametric study

RC precast industrial structures are usually one-story buildings, consisting of precast columns, socket foundations at the base and simply supported beams at the top. These structures usually have precast roof elements, which are supported by beams, and external precast cladding panels along the perimeter. According to the structural features, the one-story buildings are modelled as SDOF systems, characterized by the lateral stiffness of the columns and by a mass, evaluated from a tributary area approach.

In order to evaluate the influence of P-Δ effects on the seismic response of these buildings, an extensive parametric study is performed. The case studies are one-story precast structures and they are defined by varying the following parameters: 1) the height of the columns (6 m -8 m -10 m -12 m); 2) the total seismic mass (10 t-30 t-50 t-70 t-90 t-110 t-130 t-150 t) and the design peak ground accelerations (0.15 g -0.25 g -0.35 g). All the case studies refer to a soil type B, as defined in EC8. The assumed values of the seismic masses represent a set of realistic value for one-story precast structures and they are evaluated according to the tributary area of each column.

2.1 Design and discussion

Each column of the case-study is designed by means of a modal response spectrum analysis. The assumed behavior factor is equal to 3.5, according to the prescription by Italian Building Code [5] for ductility class high. Four different design approaches are conducted to evaluate the influence of each design provision about second-order effects.

- Design approach no. 1 → the structures are designed according to all the design provisions of Eurocode 8 for P-Δ effects. Since Eurocode does not provide any prescription in case θ is in the range 0.2÷0.3; in this study, the structures are designed following the prescriptions of precast structures characterized by θ larger than 0.1.
- Design approach no. 2 → the structures are designed by neglecting the limit about the minimum cross-section dimension of columns if θ is larger than 0.1 (H/10 rule).
- Design approach no. 3 → the structures are designed by neglecting both the H/10 rule and the limit on the maximum value of the stability factor (θ = 0.3).
- Design approach no. 4 → the cross-sections of the columns designed in the third approach are used and the reinforcement is designed by neglecting P-Δ effects.

The steel reinforcement has a yield characteristic strength of 450 MPa, whereas the concrete characteristic compressive cylinder strength is equal to 45 MPa. The code design values are used in the design phase. The column elastic stiffness is assumed equal to half of the corresponding gross stiffness to take into account the effect of cracking.
The results of the design are presented in the following (Fig. 1, Fig. 2 and Fig. 3) in terms of column dimensions (h_{col}). In these figures, m is the seismic mass and H is the total height of the structure. For sake of brevity other features of the structures (reinforcement details and stability coefficients) are not shown. Further details are reported in Ercolino et al. [6].

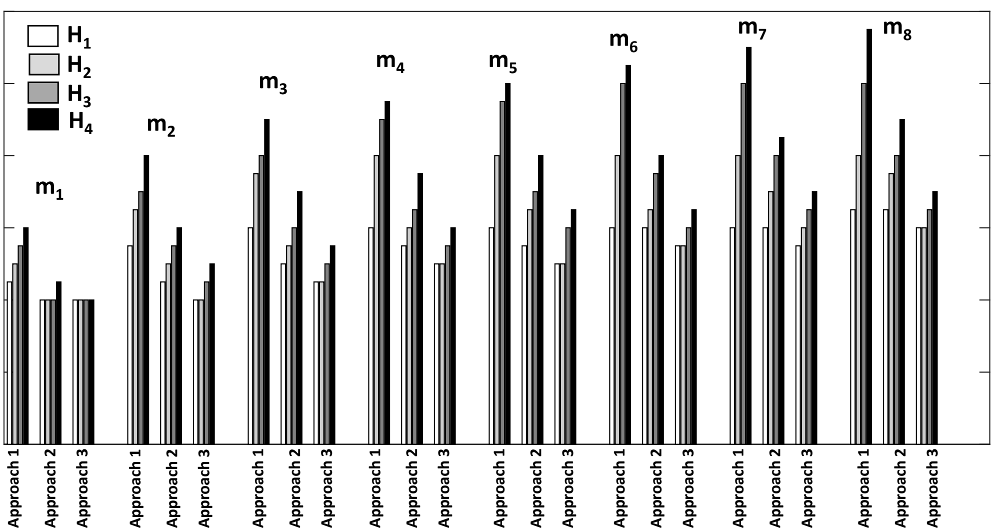

Fig. 1 - Column dimensions (h_{col}) for the different design approaches and case-studies (a_g=0.15g)

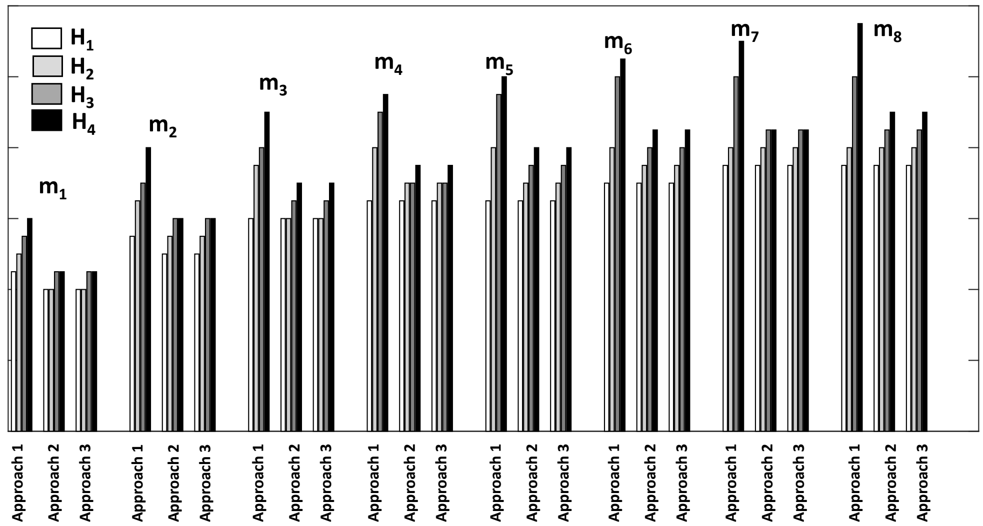

Fig. 2 – Column dimensions (h_{col}) for the different design approaches and case-studies (a_g=0.25g)

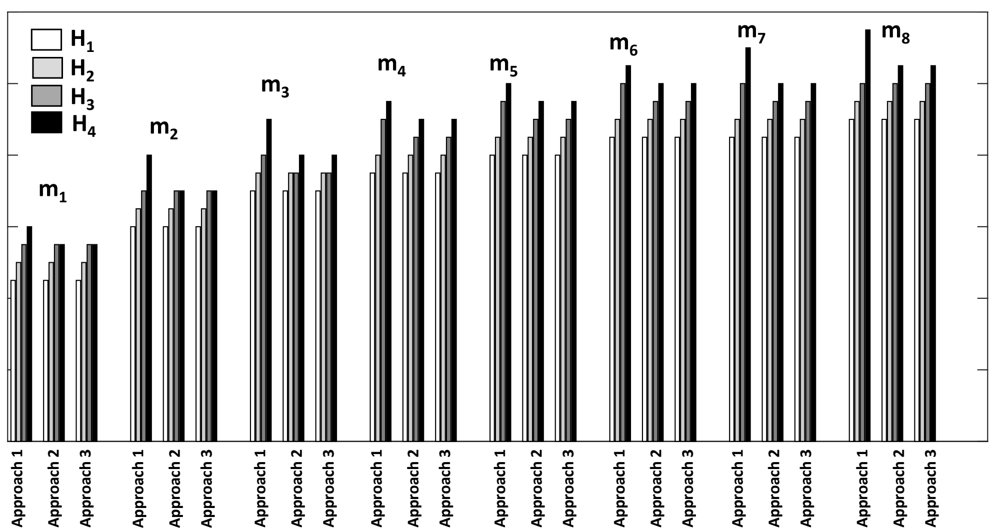

Fig. 3 - Column dimensions (h_{col}) for the different design approaches and case-studies (a_g=0.35g)

The column sections of design approach no. 2 are equal or smaller than the section of design approach no. 1. For low seismicity (a_g = 0.15 g), the absence of the section limitation (H/10 rule) caused a lower stiffness of the structures and, as a consequence, a too large value of θ (>0.3). For tall structures (10 m and 12 m) at higher seismicity (a_g = 0.25 g), θ is smaller than 0.3 and the Damage Limitation (DL) limit state influences the section dimensions (Fig. 2). For the highest a_g value (0.35 g), the difference between the two approaches is small: in approach no. 1, even if the section was defined by the H/10 rule, the final column dimension is only slightly larger than the dimension required by DL limit state (Fig. 3).

Structures designed according to approach no. 3 have the same features (section and reinforcement) of the structures designed with approach no. 2 for a_g = 0.25 g and a_g = 0.35 g. In these cases, the DL limit state influenced the design. For a_g=0.15 g, cross-section designed according to approach no. 3 are about 30% smaller than approach no. 2. These differences are caused by the different factors influencing the design: for approach no. 2, the governing rule is typically the need to re-design in case θ is larger than 0.3; for approach no. 3 the governing rule is typically the limitation on maximum longitudinal reinforcement ratio, equal to 4%.

The results of design approach no. 4 differ from the results of design approach no. 3 only in terms of longitudinal reinforcement ratio (ρ):

- for all the structures designed with the maximum value of the a_g (0.35 g), there are no differences between the two design approaches;
- for the structures designed for 0.25 g, only the ones with 6m height and 130 t and 150 t mass show some differences: the longitudinal reinforcement percentage of the design no. 3 is 1.35 times larger than in design approach no. 4;
- for the structures designed for 0.15 g, the reinforcement ratio in design approach no. 4 is smaller than the one of the design approach no. 3 in case of large masses: the ratio between the two reinforcement amounts is in the range 1.25÷3.06.

2.2 Nonlinear model

The nonlinear behavior of the designed structures (columns) is modeled by means of a lumped plasticity approach. The column is modeled with a stiff element. A bi-linear moment–rotation envelope is assigned to the plastic hinge at the base of the columns. The values of the yield and ultimate bending moment, M_y and M_u, are evaluated with a fiber analysis on the section. Empirical formulas are adopted for yield and ultimate chord rotation, as proposed by Fardis and Biskins [7]. Two modeling approaches are adopted in this study (Fig. 4).

- *Modeling approach a*: the elastic stiffness, k_{el}, of the moment-rotation envelope is evaluated from the properties of column section, as M_y/θ_y (black solid line in Fig. 4).
- *Modeling approach b*: the initial stiffness of the moment-rotation envelope is set equal to the stiffness assumed during the design phase and the yielding moment is evaluated from the design acceleration spectrum (dashed black line in Fig. 4). The hardening is assumed equal to the values in the other model.

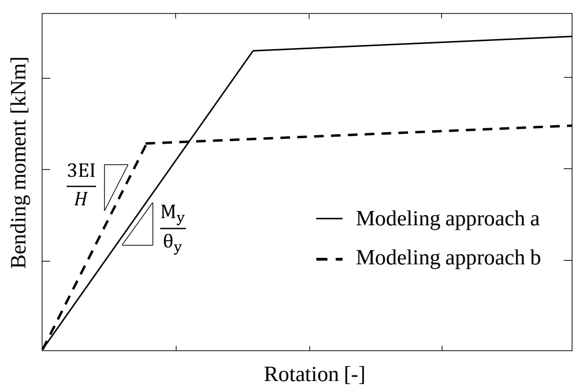

Fig. 4 - Moment-rotation curves: comparison between modeling approaches

Nonlinear dynamic analyses are performed by using Newmark's method on all the case studies without and with considering the P-Δ effects (first order and second order analysis, respectively). At this aim, fifty artificial acceleration time histories for the dynamic analyses are selected and scaled in order to match the elastic code spectrum at the three a_g values for soil type B. The generation was performed by means of the SIMQKE program [8]. The spectrum compatibility is requested in a wide period range (0.12 ÷ 6.0 sec) in order to cover all the possible periods of the case studies [9]. Further details of the adopted records are reported in Ercolino et al. [6].
The first order analysis are performed for a SDOF system by adopting the following equation of motion:

$$m\ddot{u} + b\dot{u} + k_0 u = m\ddot{a}_s(t) \tag{3}$$

where m is the mass of the oscillator, b is the damping coefficient (the damping ratio $b/2\sqrt{k_0 m}$ is assumed equal to 0.05), k_0 is the first-order elastic stiffness, u, $\dot{u}$ and $\ddot{u}$ are the relative displacement, the relative velocity and the relative acceleration of the system, respectively.

Second-order effects can be taken into account by decreasing the elastic first-order stiffness through a parameter, known as geometric stiffness, k_G, which is evaluated as the ratio of the axial force and the height of the system [10]. The equation of motion changes when P-Δ effects are considered as follows:

$$m\ddot{u} + b\dot{u} + (k_0 - k_G)u = m\ddot{a}_s(t) \tag{4}$$

In this study, geometrical nonlinearities for local effects in the columns are not considered. This phenomenon is associated with local deformation relative to the element chord between end nodes. At the typical axial load levels, these effects can be neglected, since their inclusion in the analysis will increase computational time without a significant change in the final structural response (e.g. internal forces and displacements).

3 Results and discussion

In this section the influence of P-Δ effects is investigated in terms of ductility demand for all the design approaches as well as for all the modeling methods [10].

3.1 Approach of current code

Fig. 5 shows the comparison between the two modeling approaches for the case studies designed according to design approach no. 1: the solid line refers to modeling approach a and the dotted line refers to modeling approach b. In these plots, the displacement ductility capacity is also showed with a black dashed line and it is evaluated according to Italian building code [5] as:

$$\mu_d = \begin{cases} q & \text{if } T_1 \geq T_C \\ 1 + (q-1) \cdot \frac{T_C}{T_1} & \text{if } T_1 < T_C \end{cases}, \quad \mu_d \leq 5q - 4 \tag{5}$$

In these equations, q is the behavior factor, T_1 is the fundamental period of vibration and T_C is the corner period. For all the structures, T_1 is larger than T_C and the ductility capacity is always equal to 3.5, i.e. the value of the behavior factor used in this study, according to the equal-displacement rule.

The curves show that if (a) the period of the SDOF is set equal to the one assumed during design and (b) the structural overstrength is removed, the seismic response of the structures significantly changes: the displacement ductility demand significantly increases. Moreover, in this case, the ductility demand is very close to the capacity; on the contrary, the demand for the approach a, where the actual structural overstrength is considered, is significantly lower than the capacity. This result is in line with Ercolino et al. [6], who demonstrated that the overstrength has a more important role than the difference between the design period and the period in the analyses. The largest values of overstrength refer to structures designed for a_g = 0.15 g; in this case, the ductility demand in modeling approach a is about 81% smaller than in modeling approach b. For a_g = 0.25 g and 0.35 g, the discrepancy is about 70% and 62%, respectively. This is caused by the design prescriptions on seismic detailing, which give a larger influence for low a_g.

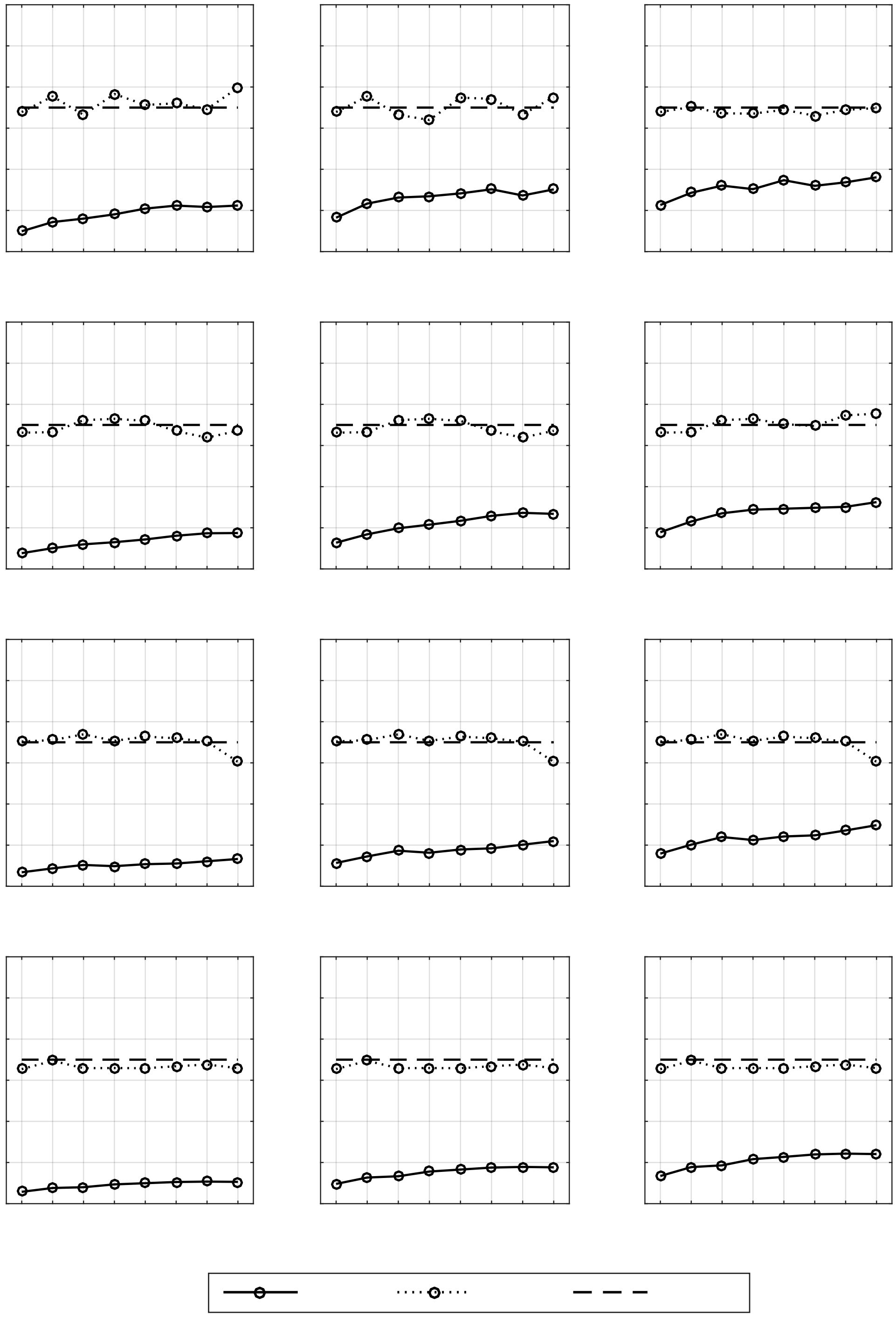

Fig. 5 - Ductility demand for the investigated case studies, designed according to design approach no. 1

In order to evaluate the influence of the overstrength, all the design approaches are used with modeling approach a. Fig. 6 shows the ductility demands along with the stability factors for all the case studies: the differences between the approaches are negligible as well as the influence of the stability factor (i.e., P-Δ effects influence). The overstrength due to seismic details and materials properties significantly increase the structure strength and this effect compensates the increase of the displacement demand on the structure due to the geometric nonlinearities.

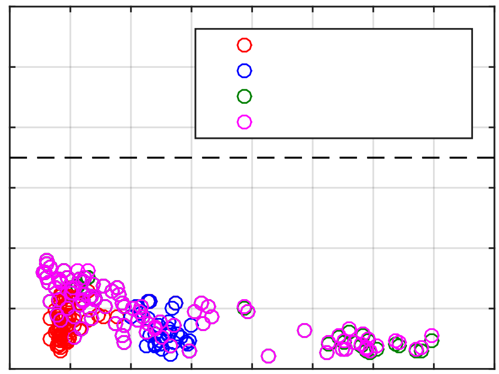

Fig. 6 - Ductility demand in terms of displacement for all the case studies modelled with approach a.

3.2 Discussion on code requirements

The presented results refer to the second order dynamic analyses performed on structures designed according to approaches 1, 2 and 3 and modeled with the approach b.

Concerning H/10 rule, the design approach no. 1 is compared to the approach no. 2. Fig. 7 shows the average values of the displacement ductility demand of structures. In these plots the different line typologies refer to the three values of seismicity: solid line for a_g = 0.15 g, dashed line for a_g = 0.25 g and dotted line for a_g = 0.35 g.

For the lowest peak ground acceleration (a_g = 0.15 g), passing from approach no. 1 to approach no. 2, the ductility demand decreases for almost all the structures. This result is mainly justified by the increase of the amplification coefficient α: the values of the stability factor θ are close to 0.30 for design approach no. 2. The reduction of the ductility demand is larger for tall structures. This evidence may be justified considering that period elongation due to geometric nonlinearities in tall structures, characterized by a fundamental period close to 2sec, does not cause a significant displacement demand increase. For short-to-medium period structures the displacement increase is much more significant, given the shape of the code spectrum.

The limitation of the minimum column dimension (H/10 rule) influenced the design of most of the structures of approach no. 1 and this design provision leads to significantly oversizing structures [11], particularly for low a_g values. Hence, it can be concluded that the H/10 rule does not significantly increase the seismic safety of the structure; on the contrary, it leads to more expensive buildings.

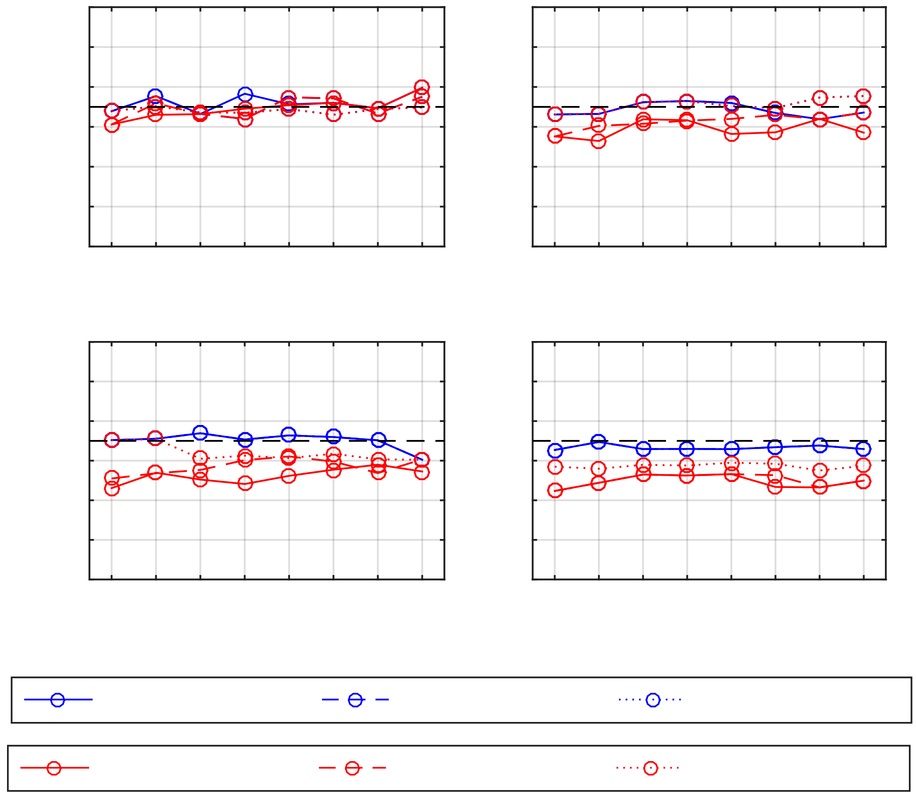

Fig. 7 - Displacement ductility demand vs mass values: comparison between design approaches no. 1 (blue line) and no. 2 (red line)

According to EC8, the columns have to be re-designed if θ is larger than 0.3. By neglecting this provision (e.g., design approach no.3), the results of the second order analyses can be compared to the design approach no. 1 (Fig. 8). In this figure some curves are overlapping (H =6 m and 8 m subjected to a_g = 0.35 g).

The ductility demand in approach no. 3 is generally smaller than the demand for approach no. 1. In design approach no. 3, the overstrength is due to the amplification coefficient α and this overstrength decreases for higher peak ground acceleration, because of the lower values of stability factor. Hence it can be concluded that structures designed without any upper bound on the stability coefficient are safer than structures designed according to current building code. The large value assumed by the factor α for θ larger than 0.30 leads to such a conclusion.

Finally, it should be underlined that for some structures an additional source of overstrength is caused by the prescription on the minimum design spectral acceleration, which cannot be smaller than 0.20 times a_g. Therefore, for long period structures, the corresponding spectral acceleration in the nonlinear analysis is lower than the value assumed during the design phase.

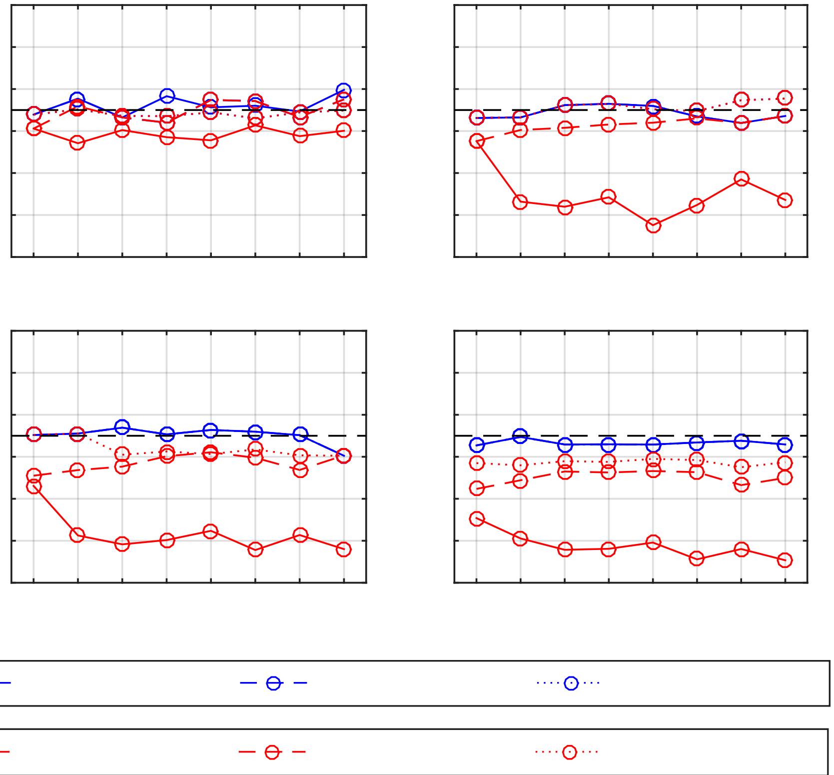

Fig. 8 - Displacement ductility demand versus mass values: comparison between design approaches no. 1 (blue line) and no. 3 (red line)

3.3 Modification of current code approach

Fig. 9 shows the comparison between the first order analysis (gray markers) and the second order analysis (black markers) in terms of the ductility demand. This comparison is presented for all the case studies, designed according to the first three design approaches and all the peak ground acceleration values. The adopted modeling approach is the method b. The ductility capacity is also reported in the same figure with a dashed black line.

Most of the structures exhibit ductility demand very close to the capacity value. The distribution of the ductility demand is very similar for the four height values and it can lead some interesting conclusions. Since the considered approaches provide the amplification of the seismic effect if θ>0.1, it can be stated that the amplification rule can take into account P-Δ effects and it generally gives a safe response of the structure. However, the results show that the amplification is necessary even if θ<0.1, since second order effects produces an increase in the ductility demand which is not negligible. The largest differences between first and second order analyses are recorded if the stability factor is in the range (0.2-0.3) and for low height: in these cases, the influence of the flexibility on the displacement demand can justify the results. For very large values of the stability factor (θ>0.4), the ductility demands of the first and second order analyses are very similar. In these cases, the very large values of the amplification factor α compensate the neglected geometric nonlinearities and the structures are still in the elastic range, i.e. ductility close to 1.0. It is worth to note that the very large value of the stability factor can lead to significant values of drift: some consideration and comments about the seismic safety of other components (e.g. connections and cladding panels) should be also taken into account in the design phase.

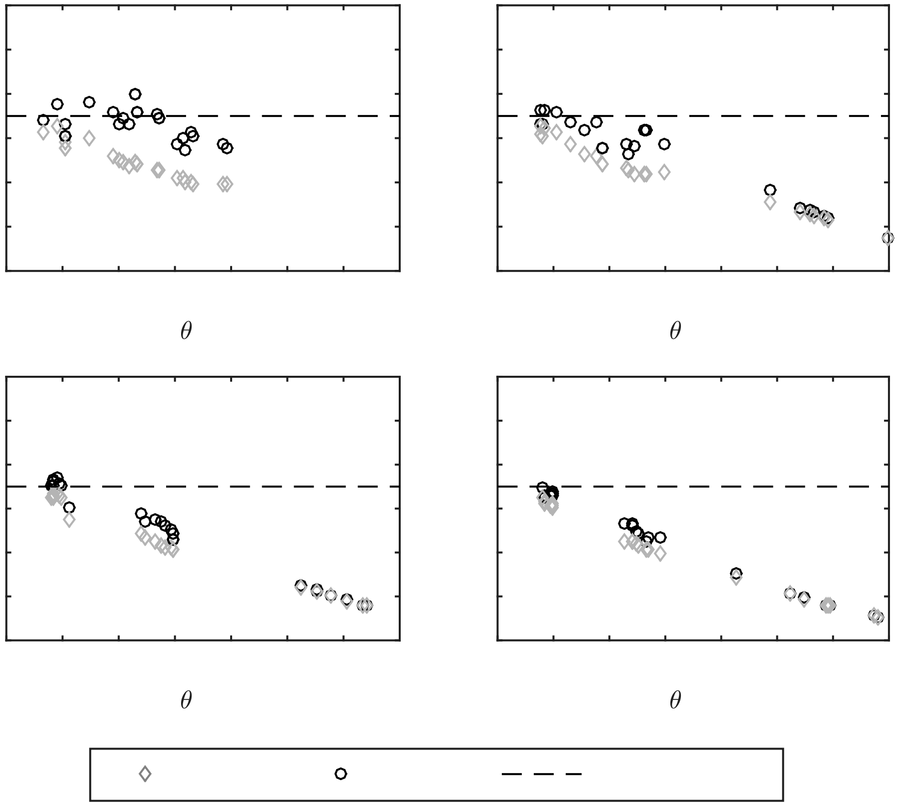

Fig. 9 - Comparison between the ductility demand in the analysis of first (gray markers) and second order (black circles)

4 Conclusions

This study aims to: 1) the evaluation of the influence of P-Δ effects on the seismic behavior of one-story precast structures and 2) the assessment of code approach used to take into account P-Δ effects in the design phase of this structural typology. An extensive parametric study is performed on one-story precast structures by varying both some geometric features of the structure and the seismicity level of the site. The one-story structures are modelled as SDOF systems and the nonlinear dynamic analyses are performed by using Newmark's method. Both first and second order analyses are performed by adopting two modelling approaches, i.e. two bilinear moment–rotation envelopes are assigned at the base of the SDOF system. Moreover, four design approaches are assumed in the study in order to evaluate the efficiency of each EC8 provision.

The results of the nonlinear dynamic analyses with the second order effects demonstrate that the actual design approach provides very low ductility demand for the structures. This result is mainly caused by two factors: the difference between the design period and the period in the analyses and the structural overstrength, due to seismic details and the materials overstrength. If P-Δ effects are totally neglected in the design phase, the overstrength in precast structures due to other code prescriptions (e.g. minimum longitudinal reinforcement ratio) still induces low ductility demand.

Concerning the code prescriptions on second order effects, the code limitations do not generally increase the structural safety with respect to the seismic actions. The application of these provisions leads to stiffer and more expensive structures without an effective improvement in the seismic performance. Finally, it is demonstrated that a modification of the current code approach would ensure both a safer behavior and more economic structures by: 1) removing the limit on the stability coefficient (0.3); 2) removing H/10 rule; 3) amplifying the seismic effect for any value of the stability coefficient (even it is smaller than 0.1).

In this study the seismic performance of both connections and cladding systems is not considered; in future studies the authors will investigate the behavior of these components if the proposed design approached is used.

5 References

[1] Moss PJ and Carr AJ. The effects of large displacements on the Earthquake Reponse of Tall Structures, *Bulletin of the New Zealand national society for earthquake engineering*, 1981, 13: 317-320.

[2] Paulay T. A consideration of P-Delta effects in ductile reinforced concrete frames, *Bulletin of the New Zealand national society for earthquake engineering*, 1978, 11: 151-160.

[3] Bernal D. Amplification Factors for Inelastic Dynamic P-Delta Effects in Earthquake Analysis, *Earthquake Engineering & Structural Dynamics*, 1987, 15(5): 635-651.

[4] CEN (2005). "Eurocode 8: design of structures for earthquake resistance - Part 1: general rules, seismic actions and rules for buildings. EN 1998-1", Brussels, Belgium.

[5] DM 14/01/2008. Norme tecniche per le costruzioni, *Roma, Italia, Gazzetta Ufficiale*.

[6] Ercolino M, Petrone C, Magliulo G and Manfredi G. Critical review of Eurocode 8 seismic design of single-story precast structures for P-Δ effects, *The Structural Design of Tall and Special Buildings*, 2016, under review.

[7] Fardis MN and Biskins DE. Deformation capacity of RC members, as controlled by flexure or shear, *In: Kabeyasawa T, Shiohara H (eds) Performance-based engineering for earthquake resistant reinforced concrete structures, a volume honoring Shunsuke Otani, University of Tokyo*, 2003.

[8] SIMQKE (2008). *Simulation of Earthquake Ground Motions*,

[9] Magliulo G, Ercolino M and Manfredi G. Influence of cladding panels on the first period of one-story precast buildings, *Bulletin of Earthquake Engineering*, 2015, 13(5): 1531-1555.

[10] Amara F, Bosco M, Marino EM and Rossi PP. An accurate strength amplification factor for the design of SDOF systems with P–Δ effects, *Earthquake Engineering and Structural Dynamics*, 2014, 43: 589–611.

[11] Brunesi E, Nascimbene R, Bolognini D and Bellotti D. Experimental investigation of the cyclic response of reinforced precast concrete framed structures, *PCI Journal*, 2015, 60(2): 57-79.