DUCTILITY-BASED DESIGN OF SECONDARY SEISMIC ELEMENTS IN REINFORCED CONCRETE STRUCTURES

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Abstract: *The study of the effects of seismic actions on buildings is an open challenge for engineering and represents a constantly evolving and expanding field. The aim of this paper, which comes from the MS thesis work of the first author under the supervision of the second one, is the development of a ductility-based design approach for secondary seismic frame elements (beams and columns) in wall type reinforced concrete buildings subjected to seismic action. The main goal is to limit as much as possible the ductility checks, and satisfy them intrinsically through the control of the secondary members' reinforcement and geometry. These are therefore allowed to plasticize in a controlled manner, avoiding also the occurrence of brittle collapse such as premature shear failures. The procedure is developed as much as possible in compliance with the provisions of Eurocode 2, which allows the plastic analysis of structures. After having briefly introduced, with reference to Eurocode 8, the seismic classification of secondary elements and the implications of this choice in the current literature, the typical plastic collapse mechanisms for a frame structure subjected to imposed deformations are analyzed. Subsequently, the concepts of curvature ductility and chord rotation ductility are investigated, and, basing on this, the procedure for the flexural sectional design is presented, with emphasis on the interrelations between ductility and strength. Finally, the design approach developed is applied to a Case Study, referring to a structure classified as a primary seismic walls system with, in addition, a secondary seismic frame part, and is validated by carrying out non-linear dynamic analyses with a three-dimensional model of the structure. The research numerical code adopted implements the well proven "RCIZ" Finite Element model for RC members which combines shear force-bending moment-axial force coupling. The results related to the global and local behavior of the building allow to draw interesting conclusions on the new design proposal, highlighting its positive aspects and limitations.*

1. Introduction

The aim of this paper is the development of a new design procedure centred on the ductility developed by secondary seismic elements. In particular, reference is made to a specific typology of reinforced concrete buildingS: the structures classified as wall systems, where walls are the seismic primary elements, while beams and columns constituting the frame part are classified as secondary elements. The work is divided into two main parts. At first, it presents an introduction to the main quantities involved in the analysis and to the theoretical development of the approach, which is as much as possible compliant with Eurocode 2 (CEN (2004)) provisions. In this part, basic concepts for a clear understanding of the method are introduced and the procedure is developed from a theoretical point of view. In the second part of the work, the results obtained are applied to a Case Study.

2. Earthquake resistant design

1.1. Secondary seismic elements

It is essential to introduce the fact that, referring in particular to Eurocode 8 (CEN (2004)), a distinction between primary and secondary seismic elements exist. The secondary ones are defined as " members which are not considered as part of the seismic action resisting system and whose strength and stiffness against seismic actions is neglected". Such elements, therefore, do not have to comply with all the detailing rules given by Eurocode 8, which instead apply to the primary elements. It is also specified that they should be designed and detailed to support gravitational loads when subjected to displacements caused by the most unfavourable seismic design condition. According to Eurocode 8, this condition is automatically satisfied if internal forces (bending moments and shear actions), computed on the basis of the just mentioned displacements and a cracked stiffness (50% of the uncracked one), are lower than the resisting shear and resisting moments calculated on the basis of Eurocode 2 classical approach. It is evident that, by doing so, the secondary elements are significantly penalized. As also found in the current literature (Milićević & Ignjatović (2017), Sigmund et al. (2008)), therefore, it is not worth classifying them as seismic secondary because the benefit, in terms of reinforcement amount, that would be achieved compared to the case of a primary element assumption is not sufficient to justify the higher level of modelling complexity resulting from this classification.

It is precisely in this context that the motivation of this paper is inserted, aimed at finding a design alternative. The new proposal, indeed, is based on a design procedure that combines resistance and ductility, which are considered in parallel through the process. In this context, the procedure is developed as much as possible in compliance with the provisions of Eurocode 2, which allows the plastic analysis of structures. As to resistance, it must be provided only against gravitational loads. Under the effect of the seismic action it is, therefore, possible for the secondary seismic elements to undergo a plastic behaviour, although under a controlled condition, through the activation of plastic hinges. It is then of primary importance to prevent premature shear collapse, which, being associated to a brittle failure mode, would prevent the development of the necessary ductility which, instead, is typical of a bending collapse mechanism.

1.2. New proposal

At a conceptual level, the starting point is provided by the rotational capacity graph, present in Eurocode 2 and reported here below in Figure 1, which shows the allowable plastic rotation in reinforced concrete sections, for steel classes C and B, as a function of the ultimate normalized neutral axis depth x_u/d. Referring to the case of a concrete class < C50/60 it is possible to associate the peak of the allowable plastic rotation $\theta_{pl,d}$, and therefore the peak of the sectional ductility, to a well-defined value of the neutral axis depth. It can be noted that class C steel allows for larger plastic rotations and has therefore to be preferred in order to achieve higher ductility levels. From the graph in Figure 1 it can be seen how, for each normalized value of the neutral axis depth, a well-defined level of sectional ductility is available.

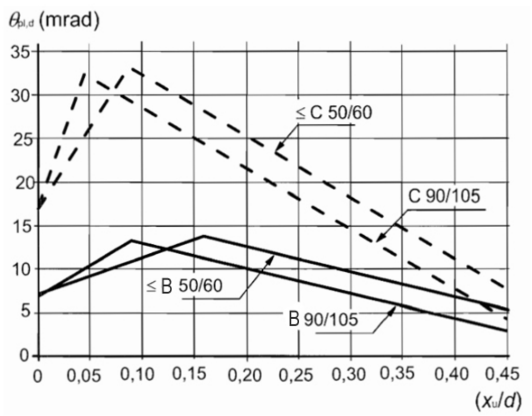

Figure 1. Basic value of allowable plastic rotation $\theta_{pl,d}$ of reinforced concrete sections for Class B and Class C reinforcement (CEN (2004)).

3. Plastic analysis

After clarifying the possibility to correlate the sectional ductility (or plastic rotation) to the cross-section properties, it is necessary, first of all, to define the ductility level required by the structure. This is achieved, remaining within the scope of the plastic analysis of structures, by studying the possible collapse mechanisms Under the effects of an earthquake motion, the deformed configuration is governed by the primary seismic elements, i.e., the walls, which act against the possible activation of a "soft-storey mechanism". Due to the set of displacements imposed by the primary elements (Figure 2), a couple of different collapse mechanisms is possible: collapse of beams or collapse of columns. Both mechanisms will be analyzed to find out the one corresponding to the maximum ductility requirement within the group of secondary elements. To this purpose, a strong hypothesis is introduced, necessary to provide the procedure with a starting point. It arises from the impossibility of easily decomposing the total rotations due to the displacements imposed by primary elements, which include, in each element, both a plastic and an elastic part. Total rotations, therefore, are assumed to be purely plastic within the considered elements. Actually, this is an overestimation of the plastic rotation demand in the element since a part of it is for sure elastic. The required ductility level, therefore, is overestimated.

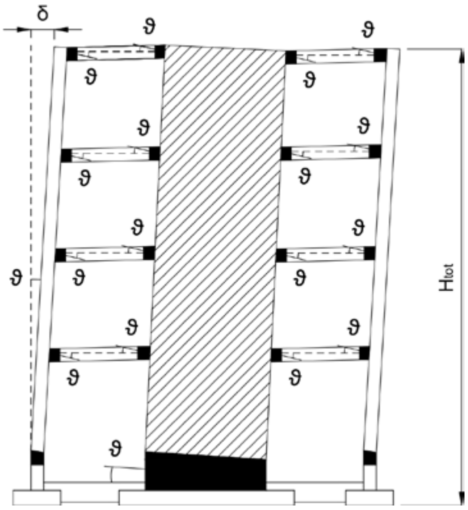

Figure 2. Typical collapse mechanism for a dual wall-frame system (adapted and redrawn from Fardis, M. N. (2009)).

1.3. Collapse of columns

The first mechanism to be investigated involves the collapse of the columns, and therefore the formation of two plastic hinges at the ends of each column (Figure 3), while beams remain in the elastic field. Extrapolating an example column, as shown in Figure 4, the plastic rotation undergone by the column is given by the total displacement (the inter-story drift) considered as totally plastic, divided by the column height:

$$\theta_{pl}^{c} = \delta_{pl}^{c}/l_c \tag{1}$$

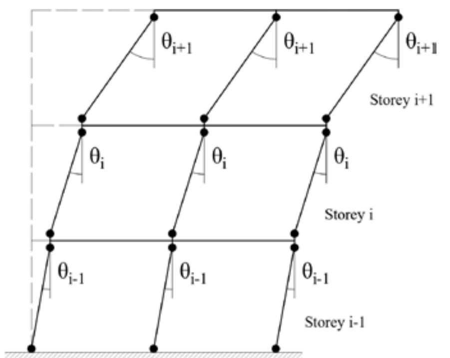

Figure 3. Plastic hinges at both the column ends.

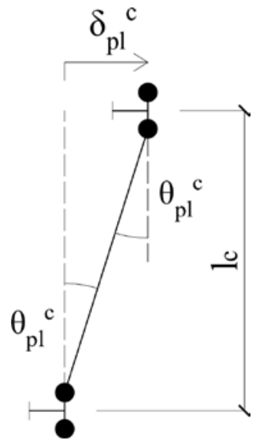

Figure 4. Detail of the formation of plastic hinges at the column ends.

1.4. Collapse of beams

On the other hand, the mechanism corresponding the collapse of beams involves the formation of two plastic hinges inside each beam, but also the activation of a plastic hinge in the column at the relevant story. In this case, the plastic hinge can occur either at the top or at the bottom of the story, as can be seen in the two mechanisms shown in Figure 5, which represent mirror situations from the point of view of the mechanism geometry and, therefore, do not differ from the point of view of the geometrical relationships.

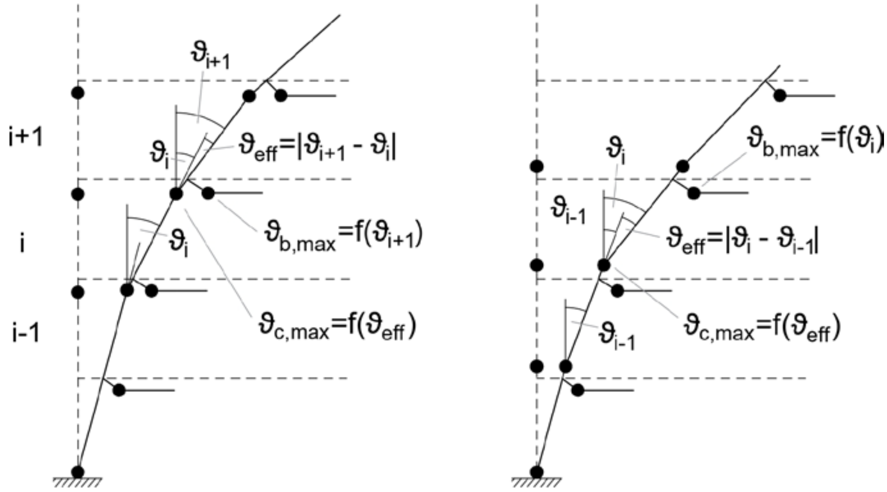

Figure 5. Plastic hinges at top/bottom ends of columns (adapted and redrawn from Jankovic & Stojadinovic (2008)).

If the mechanism is analysed on a simple sub-frame separated from the global structure, as shown in Figure 6, the relationship between the plastic rotation undergone by the beam and that of the column can be easily identified via a factor that depends on the distance between the two plastic hinges in the beam. If these are at the beam-ends, the above rotations are equal, otherwise the closer the hinges are, the larger the rotation in the beam. It can therefore be seen that the location of the plastic hinges in the beam plays an important role and should be adequately determined to avoid the underestimation of the ductility required by the element.

$$\theta_{pl}^{b} = \theta_{pl}^{c} \cdot \frac{l_b}{l^*} \tag{2}$$

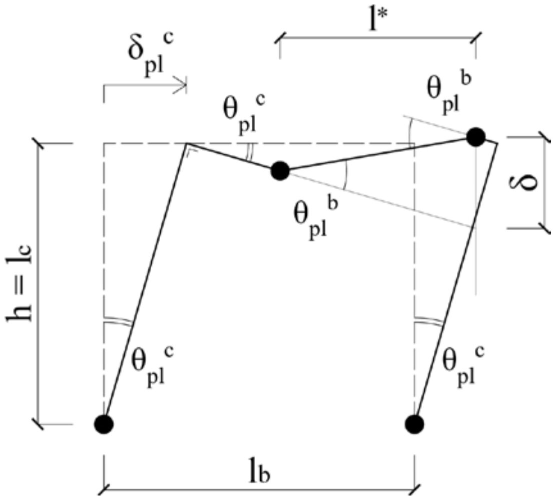

Figure 6. Beam-sway mechanism.

It should be noted that this problem depends only on the vertical load acting on the beam and, obviously, on the resisting moments and the span length. This problem is only present in beams and not in columns because there are no transversal loads acting on the vertical elements.

4. Ductility

Once the mechanisms have been analysed and the request in terms of plastic rotation in the elements has been specified, the focus goes back to sectional ductility. The parameter chosen for sectional ductility, as previously introduced with reference to Eurocode 2, is curvature ductility (μ_ϕ). It is defined as the ratio between the ultimate (χ_u) and yield (χ_y) curvature; it is influenced mainly by the axial load (decreases as the axial load increases) and by the percentage of reinforcement in compression (increases as reinforcement in compression increases).

The relationship connecting curvature ductility to plastic rotation (θ_{pl}) is obtained by introducing plastic curvature, which is given by the difference between the two curvatures mentioned above, and by the plastic hinge length L_{pl}. The latter is a conventional quantity along which the curvature is assumed constant and equal to the plastic curvature (Figure 7). Through this relationship it is possible to express the plastic rotation as a function of curvature ductility for the cross section undergoing plasticity.

$$\theta_{pl} = (\chi_u - \chi_y)L_{pl} = \chi_y(\mu_\phi - 1)L_{pl} \tag{3}$$

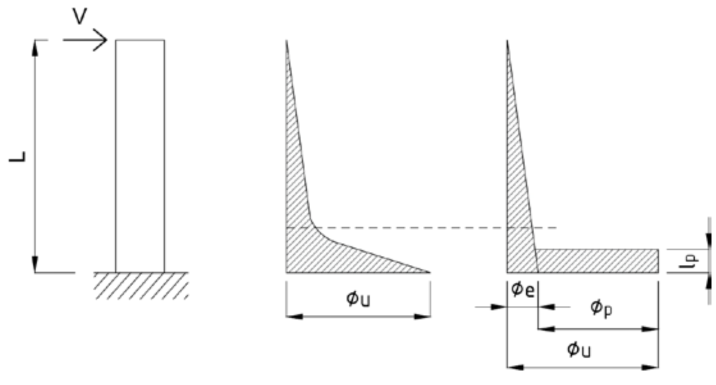

Figure 7. Plastic hinge length definition (adapted and redrawn from Bae et al. (2009)).

Curvature ductility μ_ϕ is selected as the reference parameter because, by substituting the yield and ultimate curvature values in the curvature ductility definition, it is possible to obtain the starting parameter for the Eurocode 2 procedure, i.e., the normalized ultimate value for the depth of the neutral axis x_u/d, as shown by the graph in Figure 1.

5. Sectional design

At this point, the actual sectional design is carried out following the concept of parallelism between ductility and resistance, both of which must be guaranteed at the same time. The design is based on the imposition of a certain neutral axis depth x_u dictated by the demand for ductility, obtained as previously described.

More in detail, the procedure needs to manage separately beams and columns. This is because the axial load is negligible in beams and, consequently, the imposition of the neutral axis depth implies that a sufficient sectional ductility will be available. The same cannot be done for the columns, for which the axial load, especially at the lower stories, plays a negative role. Therefore, the simple imposition of the neutral axis depth fails to guarantee acceptable sectional ductility values and makes the introduction of confinement strictly necessary. The columns will therefore be designed first for pure resistance and, at a second step, the ductility request will be satisfied through a suitable amount of confining reinforcement.

Figure 8 shows the graphical explanation of the conceptual procedure for cross section design, both for beams and for columns.

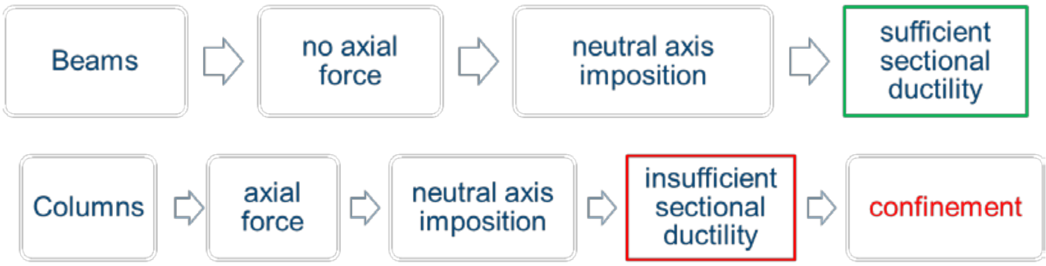

Figure 8. Flowchart for the conceptual design of cross sections.

6. Case study

In the following, the method is applied to a case study, consisting in a multi-story reinforced concrete building with 6 stories above the ground level; the plan view is shown in Figure 9. The structure is classified as a wall system and is composed by four perimeter walls, *4 m* long each, which constitute the primary seismic part, while the remaining frame elements constitute the secondary seismic part.

For the peak ground acceleration, the value of *0.3 g* is selected and both medium (DCM) and high (DCH) ductility classes are investigated to verify how this assumption affects the ductility requirements of the frame.

1.5. Linear analysis

The structure is first modelled for linear analysis, with the help of two geometrically identical models:

- the first one is the seismic model, to which only seismic forces are applied; no gravitational load is present. The secondary elements are introduced with the real constraint conditions, but all the stiffness values (for axial, bending and shear actions) is reduced by a scale factor of *1/1000*. The primary elements are inserted with the cracked stiffness;
- the second model is the one subject to gravitational loads only (no seismic load). All the elements are characterized by the cracked stiffness and, of course, by the real constraint conditions.

Two different numerical analyses are then performed: a linear response spectrum analysis with the seismic model and a classic static analysis with the second model. The results from the analyses have to be manually superimposed, which becomes an easy task thanks to the geometric compatibility of the two models. From the second model, the action effects due to the gravitational loads are obtained and, therefore, the resistance demand is known. On the other hand, the first model provides the ductility demand coming, for the frame, from the displacements induced by the earthquake. It should be underlined that the ductility demand is practically independent from the adopted ductility class (medium or high) and, in general, it increases in the structure with the elevation, both for beams and columns. Figure 10 shows an example of displacement demand for the building frames in the Y direction (frames 1, 2 and 3, respectively).

The cross section design, which is carried out for each secondary seismic element of the structure, is based on the ductility demand coming from these displacement patterns, also considering the resistance demands provided by the analyses.

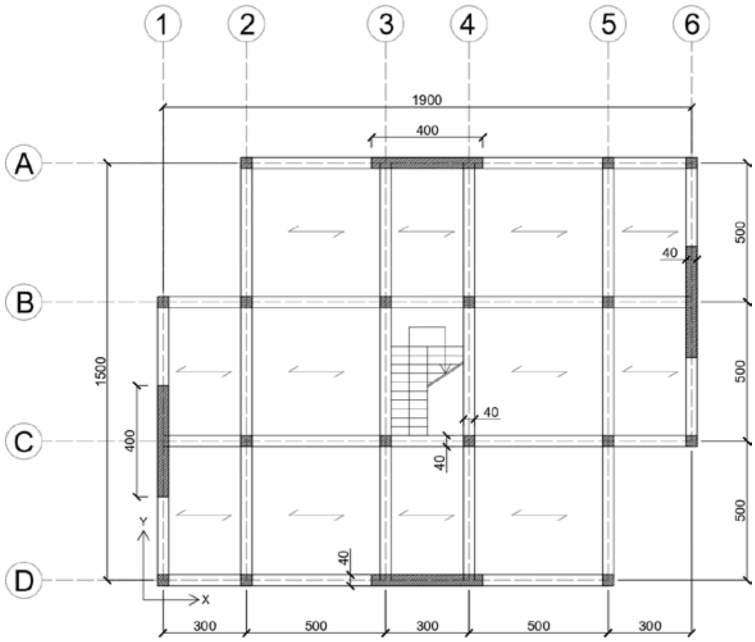

Figure 9. Plan view of the building (units are cm).

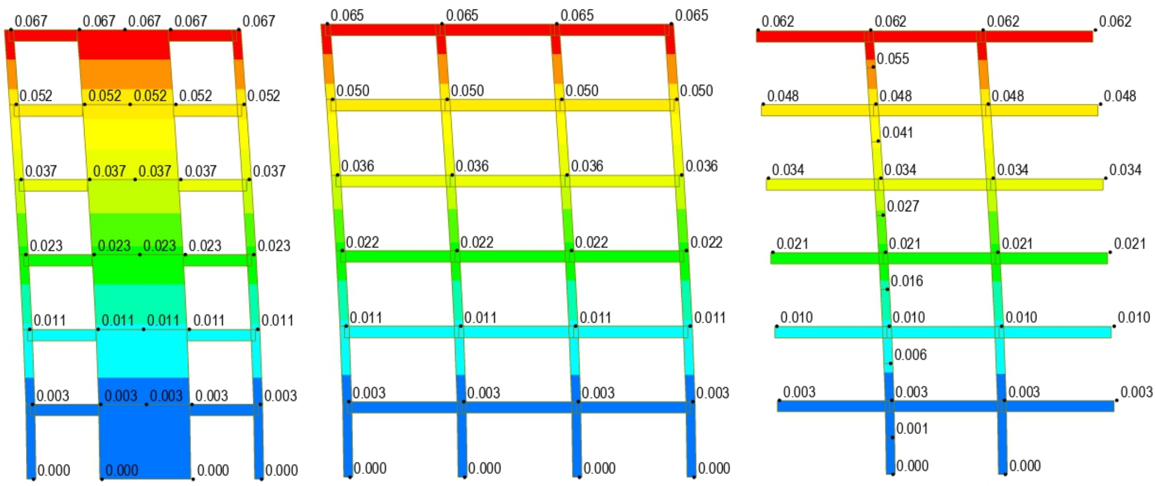

Figure 10. Deformed configuration of the seismic model, frames 1, 2 and 3 (units are m).

1.6. Non-linear dynamic analysis

To validate the results obtained from the proposed approach, non-linear dynamic analyses are carried out. In particular, the validation consists in a total of eight analyses, using for each a pair of artificially generated spectrum-compatible accelerograms. The analyses are performed using a *3D* model of the structure with *1572 dofs* and with the aid of a research numerical code capable of implementing the well proven "RCIZ" Finite Element model for RC members, which combines shear force-bending moment-axial force coupling (Martinelli (2008)).

Each element has been discretized with five cross sections, defined with equally spaced fibers. Moreover, for each type of element (beam, column or wall) the definition of the fibers in the cross section is different.

The results provided by such analyses make it possible to draw conclusions on the design methodology, first in global terms and then in local terms. Figure 11 shows the graphs of the total story displacements and of the story drift, both in X and Y direction, providing a direct comparison with the output from the linear analysis. It can be noted that the displacements from the linear analysis are slightly greater, but generally comparable, to those from the nonlinear time-histories. This difference was expected because the displacement pattern coming from a linear analysis does not take into account the stiffness contribution of secondary seismic members, which are modelled with a scaled (near to zero) stiffness. On the basis of these results, it is reasonable to expect a low level of exploited ductility within the secondary elements, because they were designed on the basis of displacements which are greater than those really occurring.

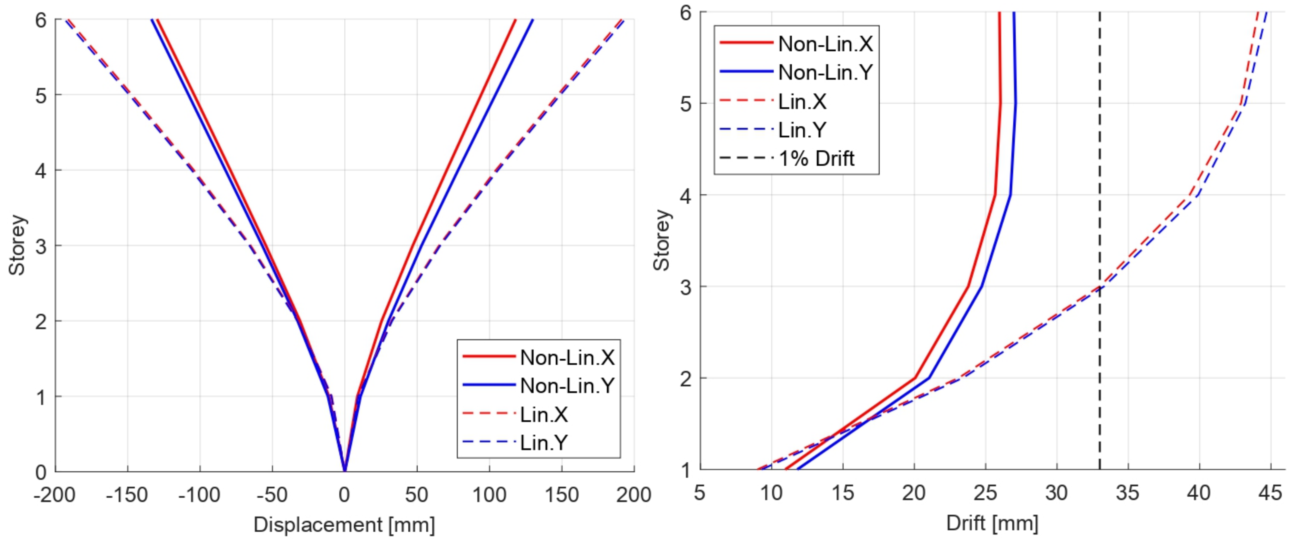

Figure 11. Story displacements and drift comparison.

Moving on to the local checks, which are carried out for each structural element, they are performed on the exploited ductility at a local (member) level. The parameter chosen to this purpose is the ultimate chord rotation ratio, expressed as demand/capacity, which allows to estimate the percentage of ductility exploited within each element. To the purpose of effectiveness in the representation of results, Figure 12 reports the elements of the third story and, as it can be seen, the local ductility is not fully exploited in general (peaks are about 50% in columns and 40% in beams). This is due to a couple of reasons: a first one has to do with the real structural displacements, which are lower than those resulting from the linear analysis, as previously seen; the other lies in the safe-side hypothesis taken at the beginning of the approach, assuming rotations as totally plastic.

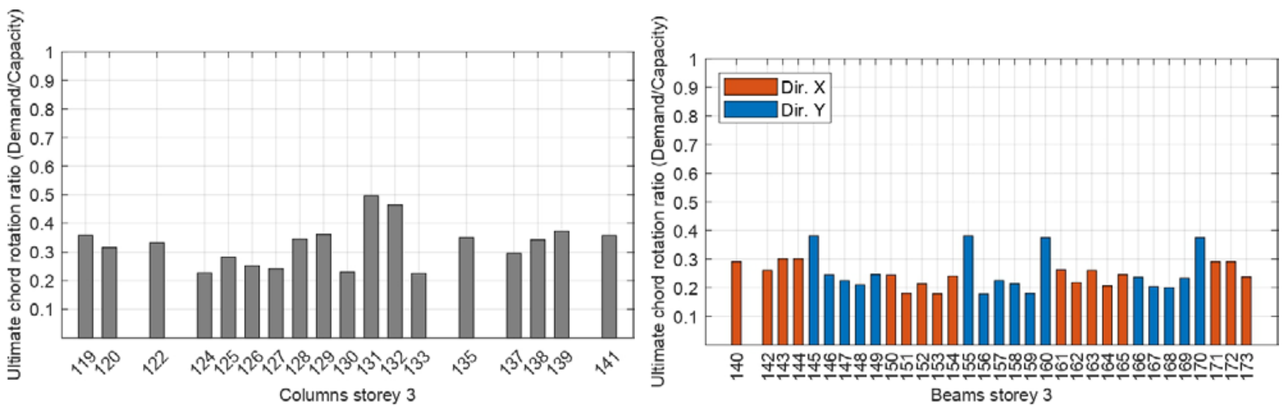

Figure 12. Local ductility checks on ultimate chord rotation.

7. Conclusions

In view of drawing a general conclusion on the new design approach, it can be stated that the method can be easily implemented for beams because through the sole imposition of a normalized neutral axis it is possible to generally satisfy the ductility requirement of the cross-section. In addition, considering that, in general, lower reinforcement quantities correspond to higher ductility levels, the approach could be refined directly designing the most unfavorable beam condition and extending its reinforcement layout to all the beams in the structure, provided that resistance to gravitational loads is fulfilled everywhere. The same cannot be done for columns, as the axial load plays a fundamental and detrimental role in the development of ductility at the cross section level. It is therefore necessary to analyze each column separately and to provide an adequate level of confinement, which cannot be achieved any more simply imposing the neutral axis depth. Furthermore, the design of the secondary seismic elements carried out in this way is independent from the ductility class adopted in the definition of the design response spectrum.

In terms of ductility, non-linear time-history results have shown that both element types (beams and columns) designed with the proposed method have sufficient ductility to follow the deformations imposed by the primary seismic elements when subjected to the seismic action. Therefore, it can be concluded that the proposed approach can be safely applied with respect to ductility. However, at the same time, it is evident that these structural elements are far from exploiting their full ductility resources, thus proving that the proposed approach lies on a sound basis.

8. References

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