

Critical regions of RC primary elements detailed in according to provisions rules for curvature ductility: comparisons and numerical analyses



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ABSTRACT

In moment resisting frames beams and columns are designed for flexural, axial, and shear actions due to vertical and horizontal loads. Special proportioning and detailing requirements are applied in these elements for making them capable of resisting against severe earthquakes without significant loss of strength beyond the flexural elastic limit, and avoiding brittle failure (shear mechanisms). As known, the required flexural inelastic excursions (expressed by the local ductility demand) depend on the dissipative capacity of the structure. The flexural ductility significantly increases with the transverse reinforcement amount provided to confine section core and to prevent buckling of compressed longitudinal bars.

In this paper detailing provisions adopted by some seismic codes are compared. At first, the codes provisions to be applied within critical regions of RC primary frames sections are discussed and compared as a function of the curvature ductility demand. Then, non-linear monotonic moment-curvature analyses are performed on fiber sections of columns and beams, and by taking into account the confinement effects on concrete core as well. The numerical investigations are carried out for comparing the available curvature ductility with the expected one applying the provisions mentioned by the seismic codes.

1 INTRODUCTION

In designing concrete moment resisting frames is necessary, coherently with the assumed overall structural ductility, to provide an appropriate local ductility in primary elements where inelastic excursions are expected. The structure has to be designed for dissipating energy without a substantial reduction of its resistance against horizontal and vertical loadings. A significant abatement of designing lateral action and internal forces may be applied for a structure if brittle failures are avoided and flexural inelastic excursions arise in beams rather columns (strong columns-weak beams mechanism).

Therefore, an important issue of designing a structure is to ensure that the local ductility demand can be satisfied within the elements critical regions (available ductility). This requires a particular proportioning and detailing of concrete elements taking properly into account also the concrete confinement provided by

transverse reinforcement. The latter heavily influences the compressed concrete behavior and, hence, the moment-curvature relationship of element sections.

The work presented in this paper is addressed to compare special provisions of primary elements critical regions reported in different seismic codes as function of curvature ductility demand required in critical regions of primary elements (beams and columns). Four different seismic codes are studied: Eurocode 8 (EC8, 2004), New Zealand Code (NZS-3101, 2006), Italian Code (NTC-08, 2008) and the American code (ACI-318, 2008). Afterwards, detailing rules are being applied to some reinforced concrete (RC) sections subjected to monotonic moment-curvature analyses in order to compare the available curvature ductility with the required one within the critical regions. The numerical investigations are performed on fiber sections with the open source software framework OpenSees (2004).

2 CURVATURE DUCTILITY OF RC FRAME SECTION

Curvature ductility (μ_ϕ) can be defined once one knows the moment-curvature relationship of a RC concrete section. It can be found by increasing the curvature of strain profile by satisfying the equilibrium conditions (Figure 1). On the moment-curvature curve μ_ϕ is defined as:

$$\mu_\phi = \frac{\phi_u}{\phi_y} \quad (1)$$

where ϕ_u is the post-ultimate curvature and ϕ_y the yielding curvature (Figure 2).

The moment-curvature diagram ($M-\phi$) defines the flexural capacity of a primary element and the hysteretic energy that can be absorbed at section level.

The definition of the yielding curvature ϕ_y is expressed by:

$$\phi_y = \phi_y' \frac{M_y}{M_y'} \quad (2)$$

where ϕ_y is defined, as shown in Figure 2, by the extrapolation of point (ϕ_y', M_y') with origin. ϕ_y' and M_y' are, respectively, the curvature and the corresponding moment calculated when the steel in tension is yielded or when the concrete extreme fiber in compression reaches strain of 0.002, whichever occurs first.

The ultimate curvature ϕ_u is calculated by applying the section failure criteria reported in Watson et al. (1994). Moreover, in all analyses performed the hoop spacing assigned is always less than 6 times the longitudinal rebar diameter. This provision is usually indicated in many seismic codes for avoiding the buckling of longitudinal bar and the premature section failure. The ideal bilinear moment-curvature relationship is reported in Figure 2.

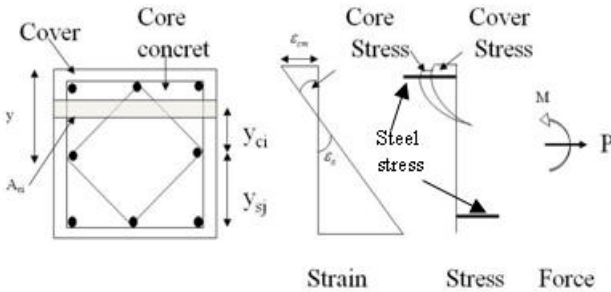


Figure 1. Profile of stresses and strains over the section for a given value of the curvature.

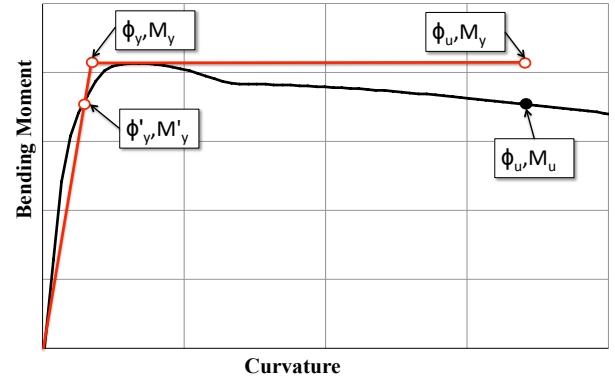


Figure 2. Definition of the ideal bilinear moment-curvature relationship.

3 COMPARISONS AMONG SECTION DUCTILITY RULES ADOPTED BY DIFFERENT SEISMIC CODES

In this paragraph are briefly summarized the ductility rules reported in some seismic codes to be applied within critical regions of columns and beams. Firstly, ductility rules are separately discussed. Then, comparisons among them are carried out. Four codes are considered: the New Zealand code (NZS-3101), the Eurocode 8 (EC8), the Italian code (NTC-08), the American code (ACI-318).

3.1 RC Columns

RC columns have the flexural response heavily conditioned by the axial load beared mainly by the compressed concrete. Therefore, design of concrete columns has to be addressed to improve the inelastic response of core section by increasing the confinement of concrete.

3.1.1 New Zealand Code

The New Zealand code (NZS-3101, 2006) adopts, for designing the quantity of transverse reinforcement of potential plastic hinge regions of columns, the formulation proposed by Watson et al. (1994), reported in Eqs. 3 and 4 for rectangular and circular sections, respectively.

For rectangular sections:

$$\rho_s = \left\{ \frac{A_g \left\{ (K_c \phi_u / \phi_y) - 33 \rho_l m + 22 \right\} f_c' N^*}{A_c \cdot 111 \cdot f_{yt} \phi f_c' A_g} \right\} - 0.006 \quad (3)$$

For circular sections:

$$\rho_s = 1.4 \left\{ \frac{A_g \left\{ \left(K_c \phi_u / \phi_y \right) - 33 \rho_t m + 22 \right\}}{A_c \cdot 111} B \right\} - 0.0084$$

$$B = \frac{f_c'}{f_{yt}} \frac{N^*}{\phi f_c' A_g} \quad (4)$$

where ρ_s is the volumetric ratio of transverse reinforcement expressed as:

$$\rho_s = \frac{A_{sh}}{s_h h} \quad (5)$$

A_{sh} is total effective area of transverse bars in direction under consideration within centre-to-centre spacing hoop sets;

s_h, h'' are the dimensions of core rectangular or square column at right angles to direction of transverse bars under consideration measured to centre to the centre line of the perimeter hoop;

A_g is gross area of column;

A_c is core area of column;

ϕ_u / ϕ_y is the curvature ductility factor;

$\rho_t = A_{st} / A_g$, A_{st} is total area of the longitudinal reinforcement;

$m = f_y / 0.85 f_c'$, f_y is yielding strength of the longitudinal reinforcement;

f_{yt} is the yielding strength of transverse steel;

f_c' is concrete compressive cylinder strength;

N^* is axial compressive load on column;

ϕ is the strength reduction factor;

ρ_s is the ratio of volume of transverse circular hoops or spiral steel to concrete core of the column.

The NZS-3101 (2006) code requires, according to the ductility class of a structure, a certain value of curvature ductility within critical regions of primary elements. More in details, for ductile structures the behavior factor is $q=6$ and curvature ductility demand is $\mu_\phi=20$. Whereas, for structures of limited ductility $q=3$ and $\mu_\phi=10$.

3.1.2 Eurocode 8

The Eurocode 8 (EC8) provides the curvature ductility demand to be satisfied within critical regions at the base of primary seismic columns. Coherently with the overall dissipating capacity of a structure the relationship proposed is:

$$\mu_\phi = \begin{cases} 2q_0 - 1 & [T_1 \geq T_c] \\ 1 + 2(q_0 - 1)T_c / T_1 & [T_1 < T_c] \end{cases} \quad (6)$$

where T_1 is fundamental period of the building in the horizontal direction of interest;

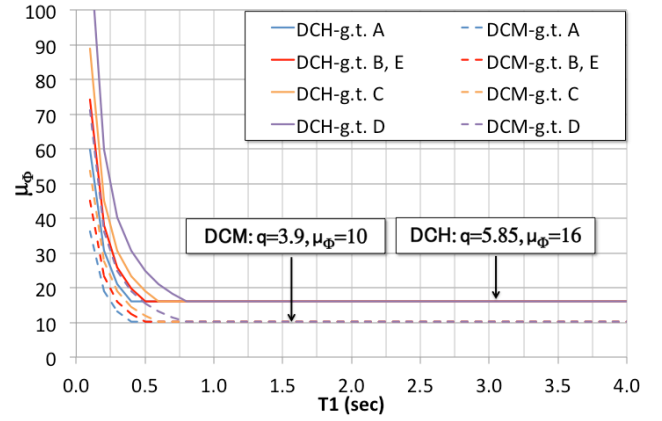


Figure 3. Curvature ductility demand for two different structural ductility classes.

T_c is the limit of the constant acceleration region of the elastic response spectrum;

q_0 is basic value of the behavior factor defined according to the type of structural type adopted;

μ_ϕ is curvature ductility of the critical section.

In Figure 3 is reported the curvature ductility demand calculated by referring to two behavior factors: $q_0=5.85$ and $q_0=3.9$ related to high (DCH) and medium (DCM) ductility class, respectively. The curves are plotted by varying the ground type from A to E. The obtained values with the Eq. 6 are multiplied by factor 1.5 as reported in EC8. As it is possible to note in the Figure 3, when the fundamental period T_1 overtakes 0.5 sec, the required ductility attains a constant value, which depends only on the structure ductility class. It is obtained $\mu_\phi=16$ and $\mu_\phi=10$ for $q=5.85$ and $q=3.9$, respectively.

In the critical regions at the base of primary seismic columns the transverse reinforcement is given by with the following equation:

$$\alpha \omega_{wd} \geq 30 \mu_\phi v_d \cdot \epsilon_{sy,d} \cdot \frac{b_c}{b_o} - 0.035 \quad (7)$$

where ω_{wd} is the mechanical ratio of transverse reinforcement given by:

$$\omega_{wd} = \frac{\text{Volume of confining hoop } f_{yd}}{\text{Volume concrete core } f_{cd}} \quad (8)$$

where

v_d is the normalized design axial force given by $N_{ed} / A_c f_{cd}$;

$\epsilon_{sy,d}$ is the design value of tension steel strain at yield;

h_c is the gross cross-sectional depth parallel to the horizontal direction in which the value of μ_ϕ ;

h_o is the depth of confined core (to the centre-line of the hoops);

b_c is the gross cross-sectional width;

b_o is the width of confined core (to the centre-line of the hoops);

α is the confinement effectiveness factor, equal to $\alpha = \alpha_n \alpha_s$.

In the case of high ductility class (DCH) a minimum value of ω_{wd} equal to 0.12 is required within the critical regions at the base of the columns, or equal to 0.08 in all column critical regions above the base. Whereas, in the case of DCM only at the base of primary columns a minimum value of ω_{wd} equal to 0.08 has to be provided.

The Eq. 7 shows that curvature ductility is proportional to mechanical volumetric ratio of confining hoops (ω_{wd}) within the critical regions. It also considers the effectiveness of the confinement (α) related to the arrangement of the hoops and longitudinal bars.

3.1.3 Italian code (NTC-08)

The NTC-08 code requires for the sections at the base of primary columns only the minimum amount of transverse reinforcement as indicated in EC8. The same minimum amount is required for all critical regions of primary columns.

$$\omega_{wd} = \frac{A_{sh}}{bs} \frac{f_{yd}}{f_{cd}} \geq 0.12 \quad CD''A''(DCH) \quad (9)$$

$$\omega_{wd} = \frac{A_{sh}}{bs} \frac{f_{yd}}{f_{cd}} \geq 0.08 \quad CD''B''(DCM)$$

where CD''A'' and CD''B'' refer to high and medium ductility class, respectively.

Eq. 9 expresses the ratio of confinement pressure within the section core to the unconfined concrete strength and may be rewritten as:

$$\frac{A_{sh}}{s} \geq 0.12 \frac{bf_{cd}}{f_{yd}} \quad CD''A''(DCH) \quad (10)$$

$$\frac{A_{sh}}{s} \geq 0.08 \frac{bf_{cd}}{f_{yd}} \quad CD''B''(DCM)$$

3.1.4 American code

The American code (ACI 318-08, 2008) provides a design approach for the volumetric ratio of the transverse reinforcement of columns as follows.

The volumetric ratio shall not be less than the larger value given by following equations:

For rectangular sections:

$$\rho_s = 0.3 \frac{f_c'}{f_{yt}} \left[\frac{A_g}{A_c} - 1 \right] \quad (11)$$

$$\rho_s = 0.09 \frac{f_c'}{f_{yt}} \quad (12)$$

For circular sections:

$$\rho_s = 0.45 \frac{f_c'}{f_{yt}} \left[\frac{A_g}{A_c} - 1 \right] \quad (13)$$

$$\rho_s = 0.12 \frac{f_c'}{f_{yt}} \quad (14)$$

3.1.5 Comparisons of local ductility demand in columns

For sake of clarity, transverse reinforcement amount required by the different considered codes is compared in this paragraph.

The comparison has been carried out for a square section (300mm x 300mm, $f_c'=30$ MPa, $f_y=400$ MPa) in terms of mechanical ratio of transverse reinforcement ω_{wd} by varying the axial load ratio v_d . Four amounts of longitudinal reinforcement have been considered: $\rho_{lm}=0.1, 0.2, 0.3$ and 0.4 .

The obtained results are reported in Figure 4 by considering four levels of curvature ductility μ_ϕ : 5, 10, 15 and 20. It is easy to recognize that exists a considerable scatter among the EC8 and NZS-3101 values. Whereas, the minimum values required by EC8 (and by NTC-08 code) are closer to the ones obtained with the NZS-3101 formulation. Finally, the minimum amount required by ACI-318 is higher than the minimum one of EC8.

3.2 RC Beams

When the acting axial load is so low that does not govern flexural response then RC elements can be classified as beams. For example, EC8 classifies as beams RC elements that are subjected to a normalized design axial force $v_d = N_{Ed}/A_c f_{cd}$ of lower than 0.1.

3.2.1 New Zealand code

NZS-3101 (2006) code reports that the tension reinforcement ratio ρ within the critical regions has to not exceed the following ratio:

$$\rho_{max} = \frac{f_c' + 10}{6f_y} \leq 0.025 \quad (15)$$

3.2.2 Eurocode 8 and Italian code NTC-08

EC8 provides the maximum amount of longitudinal reinforcement in tension for a beam as function of the required local curvature ductility:

$$\rho_{max} = \rho' + \frac{0.0018 f_{cd}}{\mu_\phi \varepsilon_{sy} f_{yd}} \quad (16)$$

where ρ_{max} is the maximum reinforcement ratio in tension normalized to bd , ρ' is the reinforcement ratio in compression normalized to bd , ε_{sy} is tension steel strain at yielding.

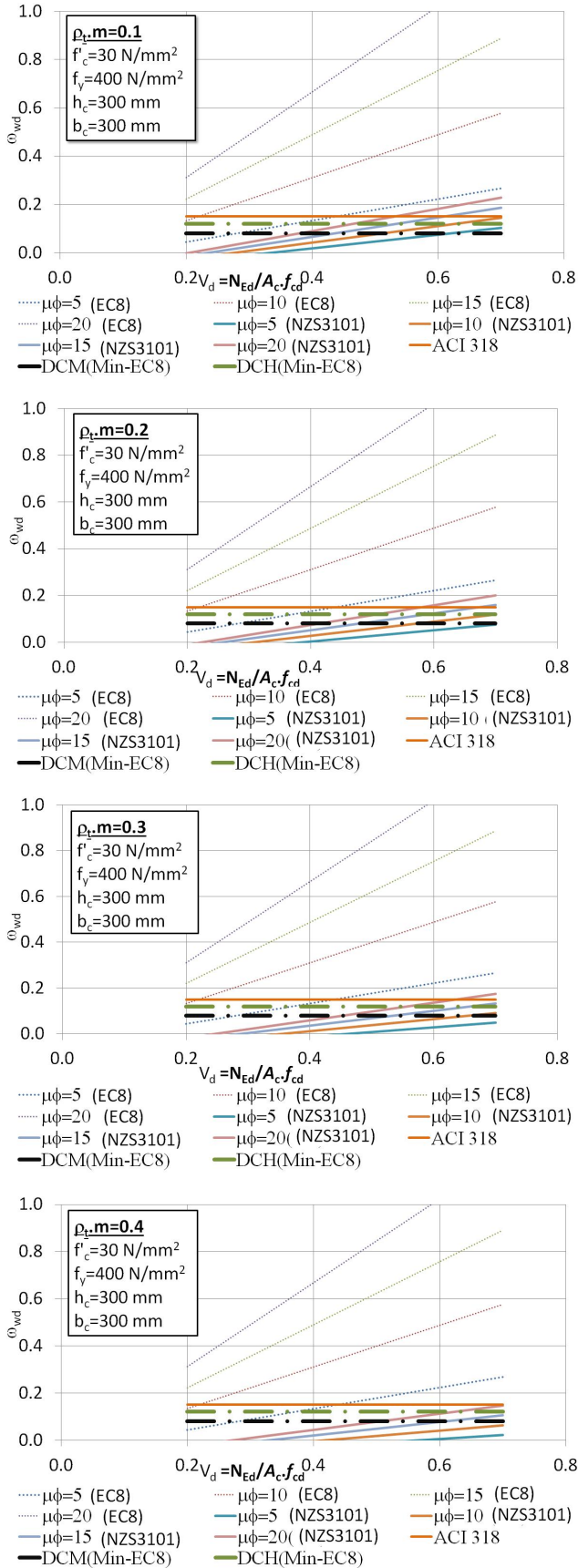


Figure 4. Transverse reinforcement requirement for columns by EC8, NZS-3101, and ACI 318.

Moreover, the reinforcement ratio of the primary seismic beam ρ shall not less a minimum value of ρ_{min} as expressed in Eq. 17.

$$\rho_{min} = 0.5 \left(\frac{f_{ctm}}{f_{yk}} \right) \quad (17)$$

where:

f_{ctm} is mean value of tensile strength of concrete; f_{yk} is the characteristic value of the yielding strength of reinforcing steel.

The NTC-08 provides a similar rule to EC8 for the amount of longitudinal reinforcement in tension:

$$\frac{1.4}{f_{yk}} < \rho < \rho_{Comp} + \frac{3.5}{f_{yk}} \quad (18)$$

Unlike the EC8 in the Eq. 18 there is not an explicit dependence from the curvature ductility.

Moreover, the NTC-08 and the EC8 specifies that at the compression zone has to be placed not less than half of reinforcement provided in tension. Thus:

$$\rho' \geq 0.5\rho \quad (19)$$

3.2.3 Comparisons

In Figure 5 have been reported comparisons among the considered codes. It is plotted the value of ρ_{max} by varying ρ' and by assuming four values of curvature ductility μ_ϕ : 5, 10, 15 and 20. The comparisons have been made by accounting for three different values of concrete strength: $f'_c = 25, 30$ and 35 MPa .

The filled area of Figure 5 highlights the acceptable region for the EC8 and the NTC-08 by respecting the prescription reported in Eq. 19, and by accounting for reversals of bending moments during the seismic action. For this reason the Eq. 19 becomes:

$$0.5\rho \leq \rho' \leq 2\rho \quad (20)$$

It is worth noting that, as shown in Figure 5, the Eq. 16 (reported in the EC8 and the NTC-08) refers only to the cases when $\rho'/\rho_{max} < 1$ (reinforcement in compression lower than maximum reinforcement in tension). The higher the ρ'/ρ_{max} ratio the higher the curvature ductility. In particular, when $\rho_{max}/\rho' = 1$ (the dotted red line) curvature ductility that can be reached is theoretically infinity.

Moreover, the Figure 5 shows that, relatively to the analyzed cases, the NTC-08 provides values of ρ_{max} (blue line) in better agreement with the ones of the EC8 (orange line) when curvature ductility is equal to 5 (the lowest scatter is obtained when f'_c is equal to 30 MPa).

The NZS-3101 code gives a value of ρ_{max} independent on the ratio of reinforcement in compression.

Finally, all provisions are independent on the transverse reinforcement amount and, therefore, on the confinement of section.

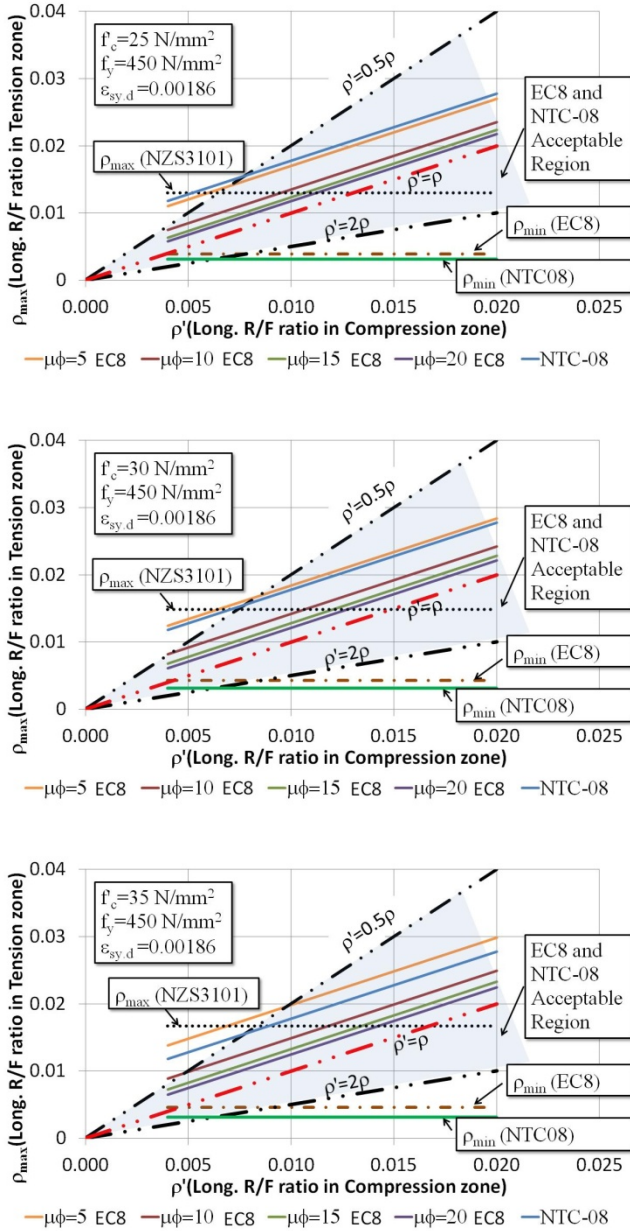


Figure 5. Comparisons of ρ_{max} by varying ρ' for different values of curvature ductility: a) $f'_c=25$ MPa, b) $f'_c=30$ MPa, and c) $f'_c=35$ MPa.

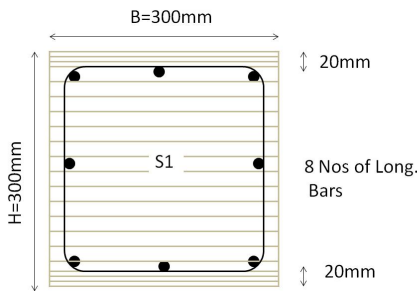


Figure 6. Column section considered for numerical investigations.

4 NUMERICAL INVESTIGATIONS

Numerical investigations have been performed with the aim of comparing the available curvature ductility of RC sections with the expected one within critical regions of primary elements by applying the design equations proposed by the considered codes.

4.1 Numerical analyses on RC columns sections

Monotonic non-linear moment-curvature analyses have been performed with OpenSees (2004) on a square fiber section whose layout is reported in Figure 6.

The analyses have been carried out by assigning an axial load to the section and by increasing the curvature until the section failure.

In these investigations the axial load ratio, the longitudinal reinforcement ratio and the unconfined concrete strength have been varied. The range of each parameter considered is reported in the Table 1. Totally, about 200 analyses have been conducted for this study.

The curvature ductility in each analysis has been calculated by referring to the ideal bilinear moment-curvature relationship (Watson et al. 1994) as briefly summarized in the paragraph n. 2.

The BGL model (Braga et al. 2006) has been used for evaluating the confinement effects on the section core model. Whereas, longitudinal steel rebars have been modeled with an elastic-perfectly plastic relationship. Properties of steel are summarized in Table 2.

Figure 7 through Figure 10 are depicted comparisons among the mechanical ratios of transverse reinforcement ω_{wd} as a function of the axial load ratio ν_d . The Comparisons are shown by considering, for a given value of $\rho_{t,m}$, four values of curvature ductility.

Table 1. Range of Parameters Investigated.

Parameters	Range
ϕ_u/ϕ_y	5-20
$P/f'_c A_g$	0.2-0.7
f'_c (MPa)	30-40
$\rho_{t,m}$	0.1-0.4
c/h	0.02
ω_{wd}	0-0.25

Table 2. Material properties of steel.

Type of steel	B450C
Maximum energy absorption	79.5(MJ/m ³)
Yielding strength	450 Pa
Rupture Strain	20%

It is clear to recognize that the results obtained with moment-curvature analyses, accounting as well for confinement effects, are in agreement with the values obtained by applying the design equation adopted by the NZS-3101 code and proposed by Watson et al. (1994). The higher the axial load the better the agreement. In all comparison EC8 always requires, at the base of primary columns, a significantly higher amount of transverse reinforcement with respect to the one considered in the analyses performed.

The minimum values of ω_{wd} required by EC8, (the same indicated in the NTC-08 code) are in many cases insufficient to ensure always the curvature ductility demand linked to the global ductility class. By referring to the analyzed cases, the higher the axial load ratio the greater the required transverse amount compared to the EC8 minimum one, especially when the curvature ductility is high.

Starting from the obtained results with analytical investigations, it is possible to propose a first formulation of a design equation linking the transverse reinforcement amount with the axial load ratio and the curvature ductility. The design equation, referred to the parameters investigated, has been derived with the same approach applied in a previous work (Braga et al. 2011).

The obtained design equation may be proposed in the form:

$$\rho_s m = A \left(\frac{P}{f_c' A_g} \right) + B \quad (21)$$

where the coefficients A and B are obtained with linear regressions from the obtained results:

$$A = \alpha \rho_t m + \beta \quad (22)$$

$$B = 0.0029 \frac{\varphi_u}{\varphi_y} - 0.0028 \quad (23)$$

$$\alpha = 0.001 \left(\frac{\varphi_u}{\varphi_y} \right)^2 - 0.0454 \frac{\varphi_u}{\varphi_y} - 0.1112 \quad (24)$$

$$\beta = -0.0005 \left(\frac{\varphi_u}{\varphi_y} \right)^2 + 0.0235 \frac{\varphi_u}{\varphi_y} + 0.1069 \quad (25)$$

4.2 Numerical analyses on RC beams sections

About 160 monotonic moment-curvature analyses with OpenSees have been performed for determining the curvature ductility of RC beams sections.

Two different kinds of fiber sections have been investigated: a deep beam and a hidden

(concealed) beam. Dimensions of the beams sections are reported in Figure 11.

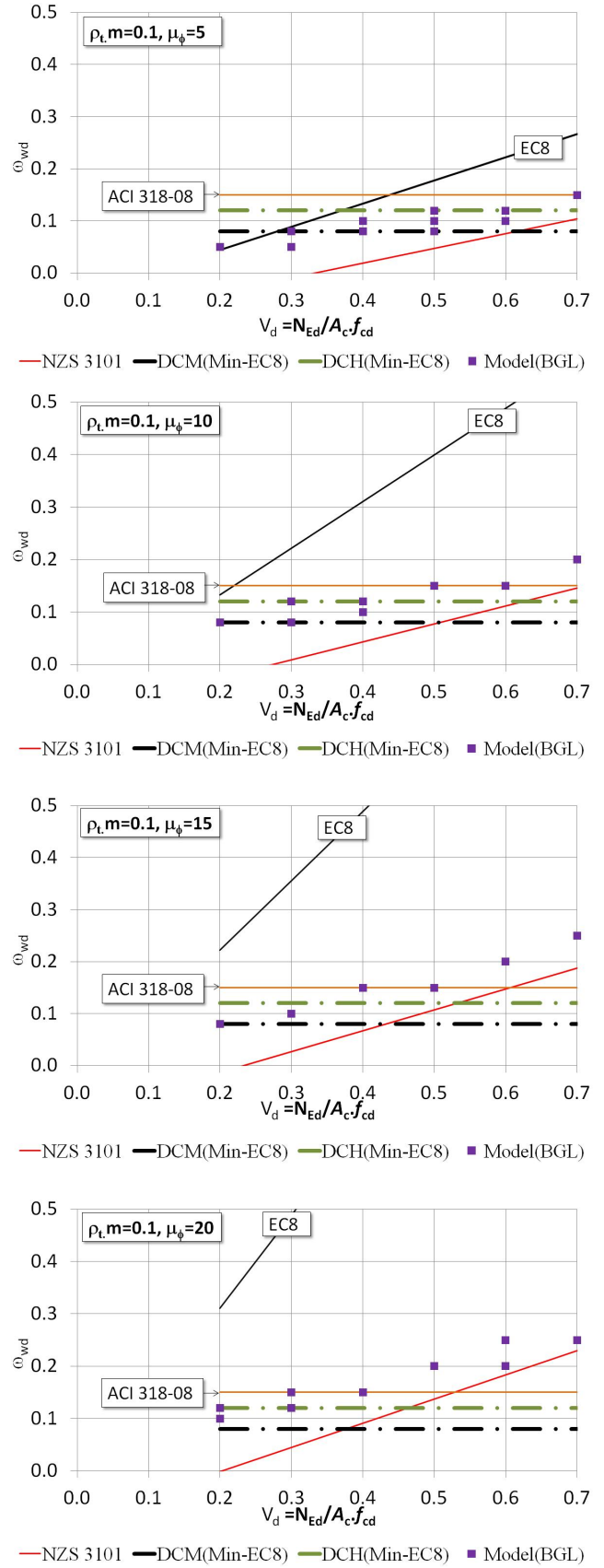


Figure 7. Mechanical ratio ω_{wd} of transverse reinforcement required for $\rho_t m = 0.1$.

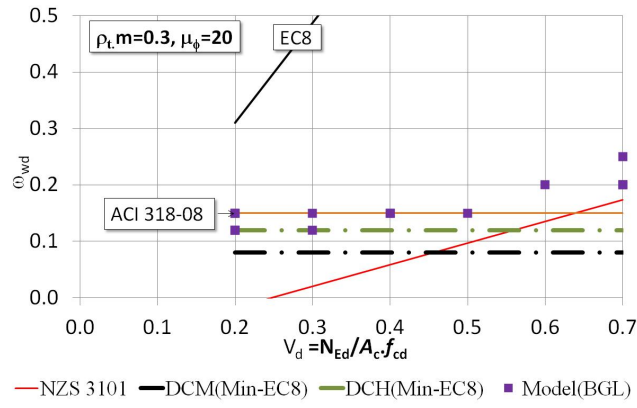
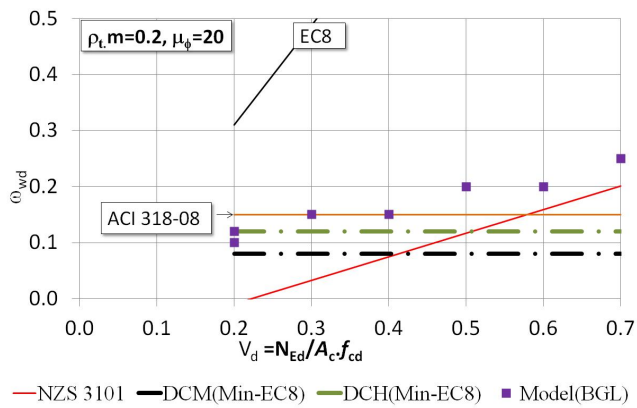
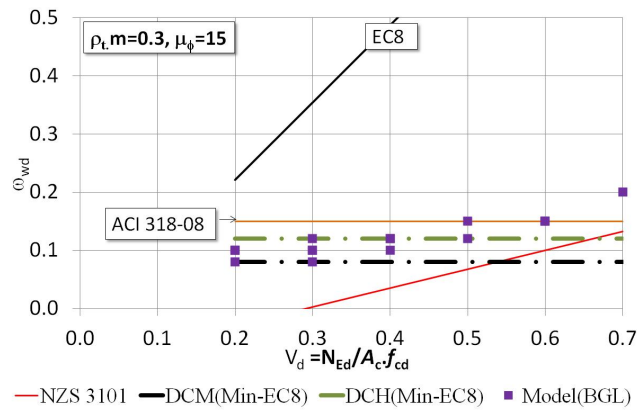
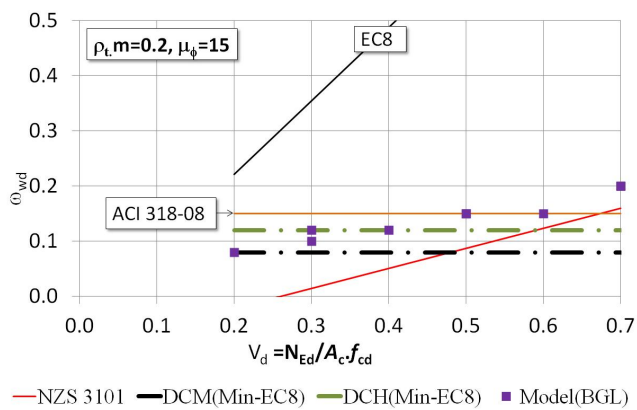
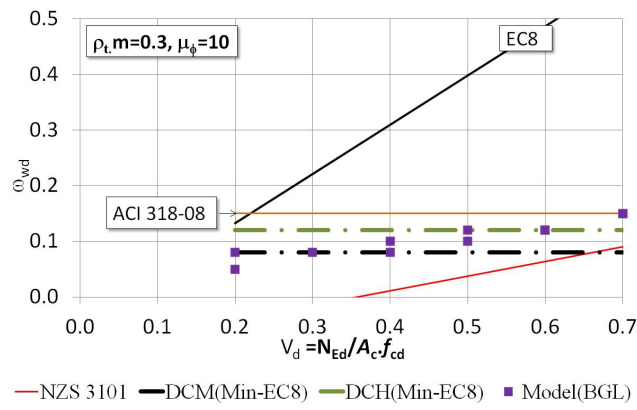
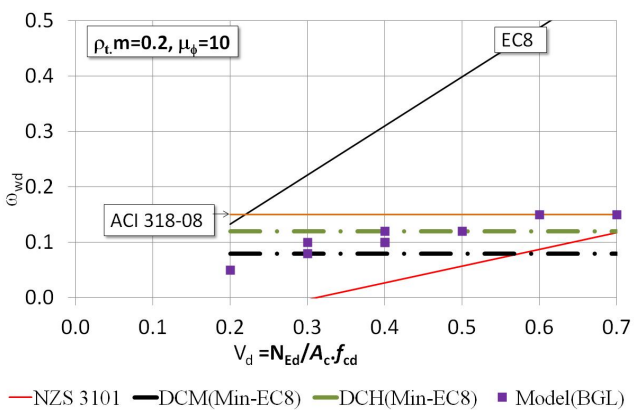
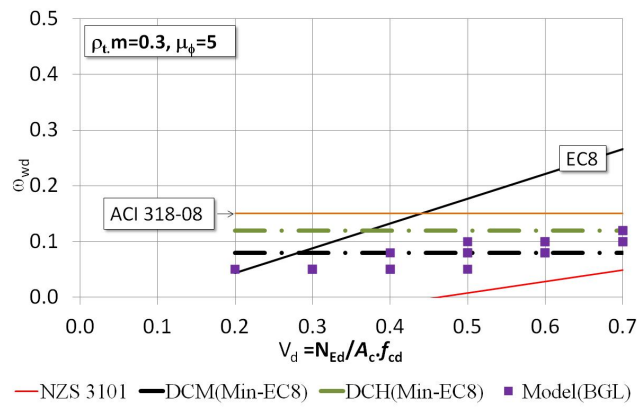
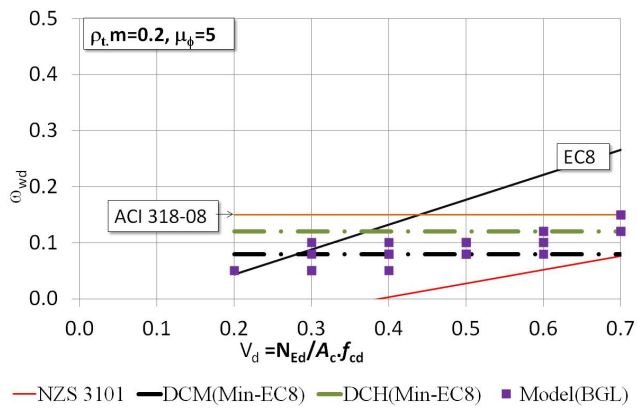


Figure 8. Mechanical ratio ω_{wd} of transverse reinforcement required for $\rho_t m = 0.2$.

Figure 9. Mechanical ratio ω_{wd} of transverse reinforcement required for $\rho_t m = 0.3$.

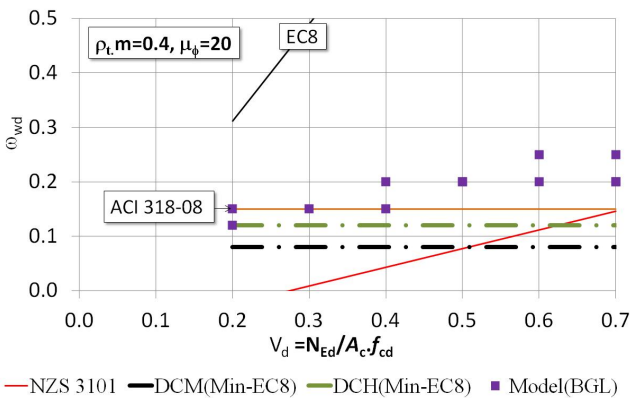
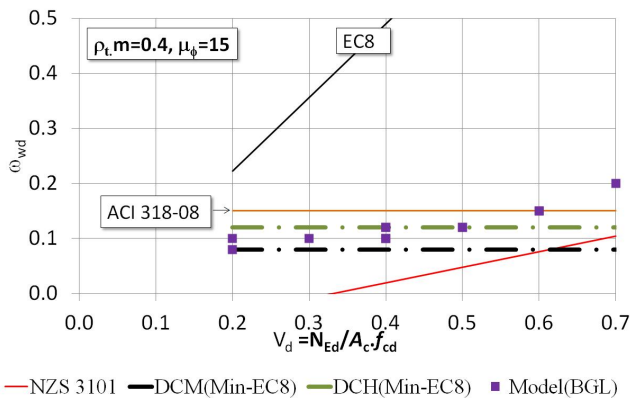
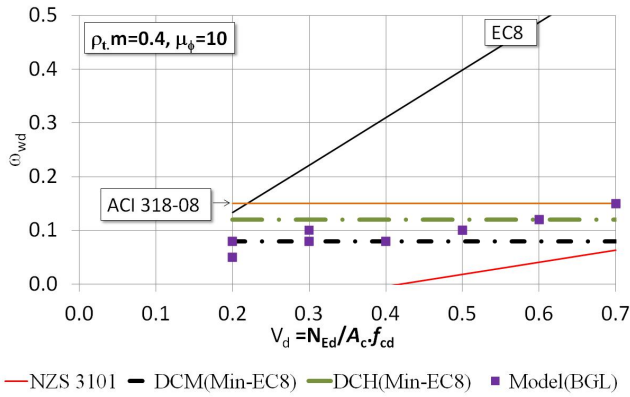
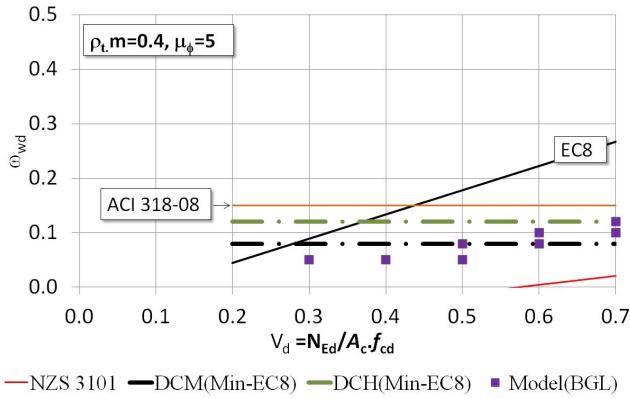


Figure 10. Mechanical ratio ω_{wd} of transverse reinforcement required for $\rho_t m = 0.4$.

Two different concrete strengths have been considered: 20 MPa and 30 MPa. Whereas, the steel strength has been set equal to 450 MPa.

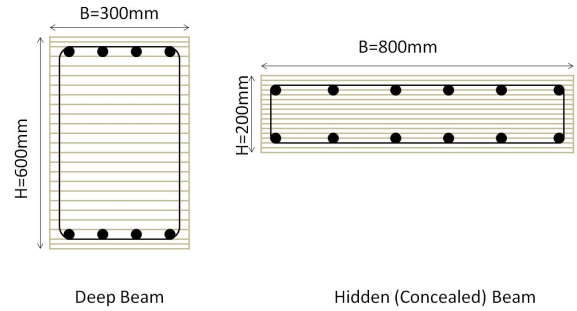


Figure 11. Layout of the fiber sections considered in the analytical investigations of beams.

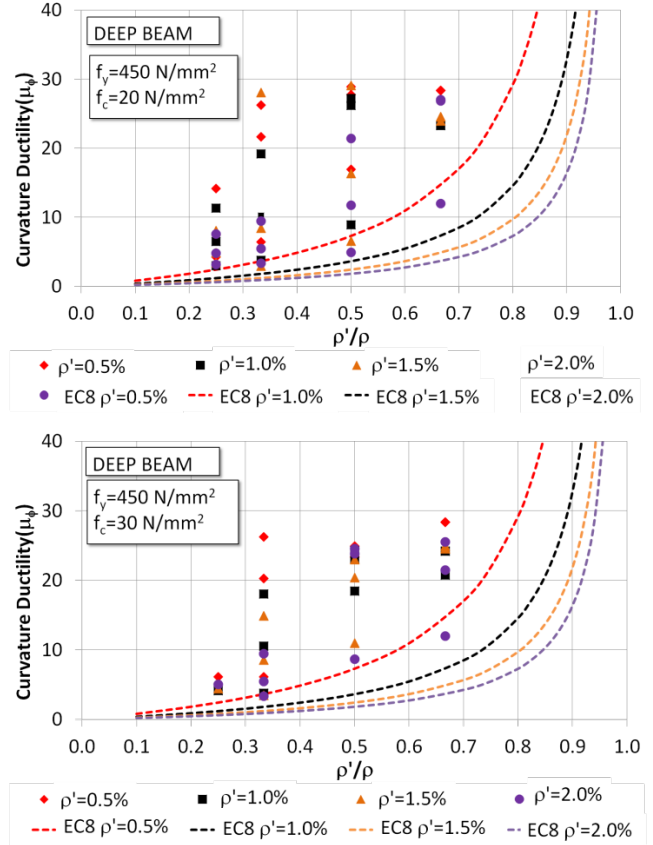


Figure 12. Curvature ductility obtained for the deep beam analyzed.

The analyses have been performed for four ratios of ρ'/ρ (0.25, 0.33, 0.50 and 0.67) by considering different the value of ρ' (0.5%, 1.0%, 1.5% and 2.0%). In each analysis the confinement effects within the section core have been evaluated with the BGL model (Braga et al. 2006, D'Amato et al. 2013). Three different spacing of hoops have been assigned (25 mm, 50 mm and 200 mm), simulating both heavily confined and unconfined sections.

The obtained curvature ductility for the analyzed cases is reported in Figure 12 and Figure 13. The results are plotted as function of the ratio ρ'/ρ . In the figures is reported the EC8 design equation of beams (Eq. 16).

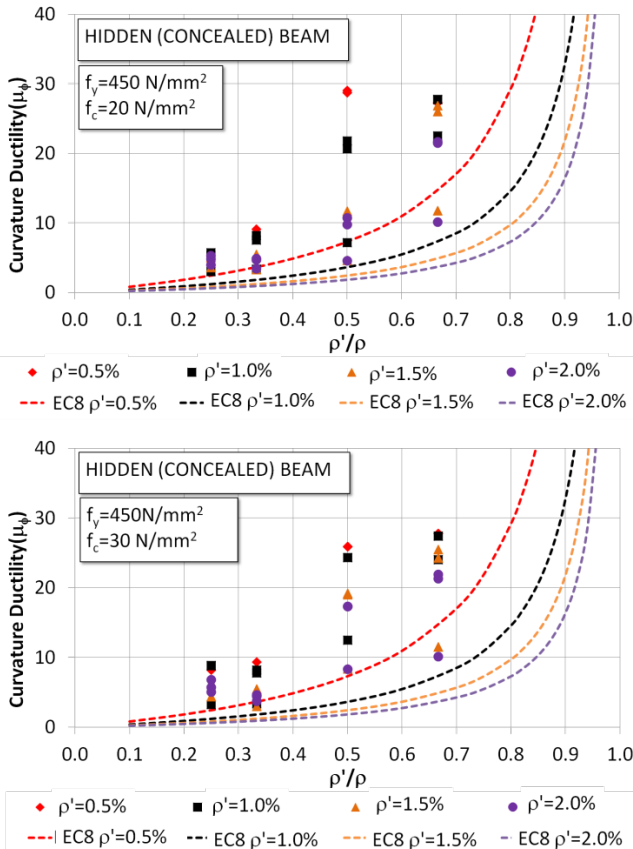


Figure 13. Curvature ductility obtained for the hidden beam analyzed.

Relatively to the analyzed cases it is possible to conclude that:

- The confinement has influenced the curvature ductility only in the cases when sections are over-reinforced (section failure due to failure of concrete).
- For a given value of ρ'/ρ the ductility increases as decreases the value of ρ' .
- For a same value of curvature ductility the EC8 always requires higher values of ρ'/ρ than the ones required in the numerical analyses.
- The considered beams have reached in the numerical analyses the curvature ductility level required in the critical regions ($\mu_\phi=10$ and $\mu_\phi=16$ for a DCM and DCH, respectively).

5 CONCLUSIONS

In this paper the special provisions of primary elements critical regions reported in some seismic codes have been investigated. The design equation for curvature ductility refers to: the transverse reinforcement amount and the axial load ratio in the case of columns; the longitudinal reinforcement ratio in the case of beams.

Monotonic moment-curvature analyses on fiber sections of columns and beams have been

performed for comparing the expected curvature ductility within the critical regions with the available one for a given amount of transverse and longitudinal reinforcement.

The investigations carried out have shown that the minimum amount of columns transverse reinforcement required by EC8 and NTC-08 code is not sufficient in satisfying the required curvature ductility, especially when the axial load is high. Instead, for a given value of ductility level the transverse amount indicated by EC8 for critical regions at the base of primary columns is largely higher than the corresponding amount required by other codes and by the one obtained with moment-curvature analyses.

As far as the design of longitudinal reinforcement ratio of beams is concerned, in the conducted comparisons the EC8, and therefore the NTC-08, assigns for a given value of ductility, always higher values of ρ'/ρ than the ones required in the numerical analyses.

REFERENCES

- ACI 318, 2008. *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318r-05)*.
- Braga, F., Gigliotti, R., Laterza, M., 2006. Analytical Stress-Strain Relationship For Concrete Confined By Steel Stirrups And/Or Frp Jackets, *Journal Of Structural Engineering Asce*, 132(9), pp. 1402-1416.
- Braga, F., Gigliotti, R., Laterza, M., D'Amato, M., 2011. Progettazione dell'armatura trasversale in funzione della duttilità di curvatura richiesta: prima formulazione di base. *XIV Convegno Anidis "L'ingegneria Sismica In Italia"*, 18-22 Settembre 2011, Bari.
- Eurocode8. *Design Of Structures for Earthquake Resistance - Part 1: General Rules, Seismic Actions And Rules For Buildings*, December 2004.
- D'Amato, M., Braga, F., Gigliotti, R., Kunnath, S., Laterza, M., 2012. A Numerical General-Purpose Confinement Model For Non-Linear Analysis Of R/C Members, *Computers And Structures Journal*, Vol. 102-103, pp.74-75.
- Eurocode 8. *Design Of Structures for Earthquake Resistance - Part 1: General Rules, Seismic Actions And Rules For Buildings*, December 2004.
- NZS 3101:2006. *Concrete Structures Standard, Part 1- The Design Of Concrete Structures And Part 2 Commentary On The Design Of Concrete Structures, Standards Association Of New Zealand*, Wellington, New Zealand.
- OpenSees, 2004. *Command Language*, University of California.
- Watson, S., Zahn, F. A., Park, R., 1994. Confining reinforcement for concrete columns, *Journal Of Structural Engineering*, Vol. 120, No. 6, pp. 1798-1824.