SEISMIC PERFOMANCE OF R/C FRAMES WITH REGULAR AND IRREGULAR STRENGTH VERTICAL DISTRIBUTIONS

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ABSTRACT :

The paper presents the results of a research study concerning the seismic response and design of RC frames with strength discontinuities in elevation. The irregularities are obtained assigning overstrengths either to the beams or to the columns of a "regular frame" (assumed as reference). The "regular frame" is designed according to the Eurocode 8 (EC8) High Ductility Class (DCH) rules. The overstrengths of presumed irregular frames are assigned modifying the reinforcement either of the beams or of the columns at different floors.
For all frames the criteria of vertical strength irregularity of many international seismic codes are applied. To this purpose, the storey strengths are computed by two different methods: the first one only takes into account the flexural resistance of columns, while the second one also considers the beam flexural resistance. Non linear static and dynamic analyses are performed: mechanical non linearity is concentrated at the element ends. These analyses are carried out according to EC8 provisions: for non linear static analysis the N2 method is applied; in the case of non linear time-history analyses, seven real earthquakes, selected in order to fit on average the elastic design spectrum are used as input.
The seismic response of frames characterised by the assigned overstrength is not very different with respect to the "regular frame" one; this demonstrates that the sensitivity of frames, designed according to High Ductility Class, to overstrength vertical variations is low. Indeed all the frames satisfy the Ultimate Limit State, verified by the application of both the non linear static and the non linear dynamic analysis.

KEYWORDS: Vertical strength irregularity, story strength, non linear analysis, seismic design, ductility class.

1. INTRODUCTION

The irregular structures show an unfavourable seismic behaviour, characterized by the concentration of plastic demand in a limited number of sections, that can conduce them to an early collapse under strong seismic motion. The regularity in elevation is, in particular, conditioned by distribution along the height of mass, stiffness and strength. All the current international codes give rules to verify regularity of structures; if these are not satisfied, opportune provisions are provided.
The first studies on the topic concerned structures with set back and/or with soft storey. Recently, irregularity in elevation is studied discriminating the irregular distribution of mass, stiffness and strength (Valmundsson e Nau, 1997; Al-Ali e Krawinkler 1998; Magliulo et al., 2001; Magliulo et al., 2002a; Magliulo et al., 2002b; Magliulo et al., 2002c; Iorio et al., 2003; Magliulo et al., 2004; Iorio et al., 2004; Magliulo et al., 2006), in order to obtain a better evaluation of their effects. These studies demonstrate that strength irregularity induces the largest increment of plastic demand. Furthermore, the reliability of code criteria for the verification of regularity is investigated.
The present paper takes some results and considerations presented in Magliulo et al., 2004 as a starting point. As in Magliulo et al., 2004 it investigates the response of frames with not uniform distribution in elevation of overstrengths (ratio between available strength and demand due to design actions) and the reliability of code criteria for the verification of strength regularity in elevation. The original parts of the paper are: 1) the verification of regularity, based on non linear analyses performed according to Eurocode 8 (EC8) (CEN, 2003) provisions, both static and step by step dynamic ones; 2) the evaluation of the storey strength, also based on the strength of the beams.
In the paper some frames with discontinuities of strength distribution along the height, which can be often found in practical applications, are considered; they are generated by a frame characterised by a regular geometry and designed according to the Eurocodes provisions (CEN, 2002a; CEN, 2002b; CEN, 2003; CEN, 2004). A frame is assumed irregular when its verification at Ultimate or Near Collapse Limit State is not satisfied or, at least, gives much worse results than the one performed on the corresponding regular frame; obviously, the verifications are performed in non linear field of behaviour because the frames are ductile and strength discontinuities are considered.

2. DESCRIPTION OF THE ANALYSED FRAMES

The reference structure (Figure 1) is a 5 floor plane frame with 2 spans. In the design it is assumed that the frame belongs to a 3D building characterized by an infinite number of equal frames.

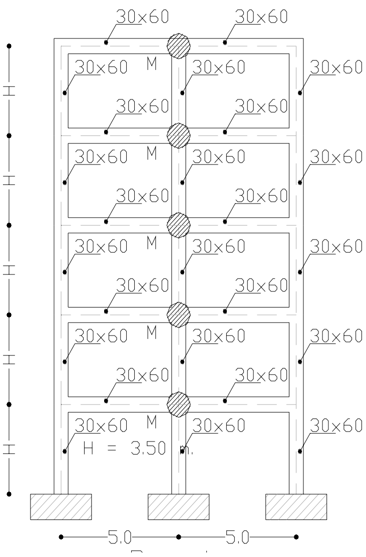

Figure 1 “Reference” frame

The members (beams and columns) have all the same section, equal to 30×60 cm. The concrete characteristic compression cylindrical strength is equal to 35 N/mm^2, the steel characteristic yielding strength is equal to 450 N/mm^2. The design is performed according to the High Ductility Class (DCH) Eurocode rules. Soil type B spectrum, with accelerations on rigid soil a_g characteristic of the zone 1 according to the (OPCM 3431, 2005), i.e. 0,35g, is considered. The behaviour factor is assumed equal to 5.85, also considering that the structure is regular in terms of mass, stiffness and strength; indeed, the assigned strength is the demanded one according to the modal response spectrum analysis, also considering the reinforcement provided by EC8 for ductility and general detailing.
The first elastic period of the "reference" frame is equal to 0,706 s, with effective modal mass equal to 83%, the second one is equal to 0,223 s, with effective modal mass equal to 11%.
Six frames, characterized by discontinuous distributions of overstrengths in elevation, are generated, according to simplifications in distributions of reinforcement, that often, for advantage in realization, are adopted in technical application; they are presented in the following.
The Pb(1-3, 4-5)H frame is obtained varying the strength of beams. The columns present the same reinforcement of the "regular" case, while the beams at the first tree floors have the same reinforcement which, for each side of each section, is equal to the maximum at that side of the sections of the first tree floors of the reference frame. The same is for the last two floors. Obviously the strength is varied increasing it, because its decrement would cause that the frames would not satisfy the code provisions in terms of minimum strength.
The case named Pb(1-3, 4-5)HH presents the same reinforcement of case Pb(1-3, 4-5)H, but, due to its variations with respect to the reference frame one, the column reinforcement is, if necessary, increased in order to satisfy the capacity design rule:

$$\sum M_{Rc} \geq 1{,}3 \sum M_{Rb} \tag{2.1}$$

Indeed, due to the variations of the strength assigned to the reference frame in order to obtain the Pb(1-3, 4-5)H one, in such case the (2.1) could be not satisfied.
The Pc case is obtained by the "reference regular" frame varying the strength of columns. The beams present the same reinforcement of "regular" frame, while for each column, the reinforcement is equal at both ends and equal to the maximum among these two sections.
The Pbc(1-3, 4-5)H case presents the same variations with respect to the reference frame which characterize both the frames Pb(1-3, 4-5)H and Pc. The case named Pbc(1-3, 4-5)HH presents the same reinforcement of case Pbc(1-3, 4-5)H, but the column reinforcement is, if necessary, increased in order to satisfy the capacity design rule (2.1).
The nomenclature of the frames is defined in order to resume their characteristics: b (c) means that the beams (columns) strength is varied, (1-3, 4-5) that the variations are assigned from the 1st to the 3rd storey and from the 4th to the 5th storey , HH that the capacity design rule (2.1) is also applied after having modified the beam strength with respect to the reference frame.
Finally the frame Alfa zero is obtained from the regular one multiplying the strength at the bottom of each column at the first level by the same coefficient assigned at the top of the same column in order to satisfy the capacity design rule (2.1). This case is introduced in order to understand if variations in seismic behaviour can be noted with respect to the reference frame, where first storey columns, according to EC8 previsions, presents at the bottom a reinforcement and, consequently, a strength, which is not lower than the top one, but which could present a lower overstrength.

3. NON LINEAR ANALYSIS

The non linear analysis are performed by CANNY99 (Li, 1996). Beams e columns are modelled as lumped plasticity elements, with a non linear rotational spring at each end. The moment-rotation relationship is trilinear, characterised by a cracking rotation equal to the corresponding curvature multiplied by the length of the member divided by 6; the yielding rotation is assumed equal to the EC8 (A10b) (CEN, 2005). In the model such skeleton curve does not depend on the axial load and it is computed assuming the vertical loads of seismic

design combination (CEN, 2002)a. A Takeda like hysteresis model is assigned for non linear dynamic analysis (Figure 2), which takes into account the pinching effect.
As above mentioned, non linear static analyses (NLSA) are performed according to EC8 previsions. The capacity curves, base shear – top displacement, are obtained using the two provided distributions of horizontal forces: one proportional to the first vibration mode displacements times the masses (Mxϕ), the other one proportional to masses distribution (M). The target displacements at the Ultimate (ULS) and Near Collapse (NC) Limit State are obtained according to N2 method; in the first case the action is represented by the EC8 elastic spectrum used for the design of frames, in the second one the same spectrum is amplified by a coefficient equal to 1.5, as provided by (OPCM 3431, 2005). The demand in terms of rotation is compared to the capacity provided by EC8 ((A1) in CEN, 2005), which is equal to $\frac{3}{4}\theta_u$ at ULS and θ_u at NC.
The results obtained by the regular frame are compared to the ones of "supposed" irregular frames in terms of capacity curves and rotational demand/capacity ratios.

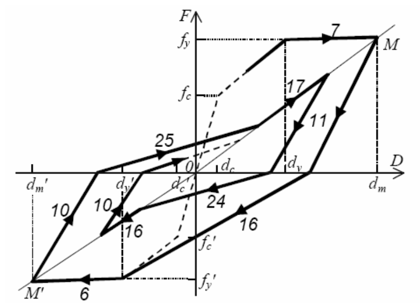

Figure 2 Hysteresis model for non linear dynamic analysis

The non linear dynamic analyses (NLDA) are performed using a set of 7 earthquakes, compatible, according to the EC8 provisions, with the elastic spectrum used for the design of frames (soil B, a_g=0,35). These accelerograms are downloaded from the site www.reluis.it, even though the only earthquake with code 000187 (Teheran) is scaled by a coefficient equal to 1.08, being sufficient the compatibility with the spectrum over a range of periods more narrow than 0.04 – 2 sec. The two components of each earthquake are used separately because the frames are plane; consequently, for each frame, 14 analyses are performed and the commented results, reported in terms of rotational demand/capacity ratios, are the mean of 14 maxima.

4. RESULTS OF NON LINEAR ANALYSIS

The capacity curves, in terms of top displacement, divided by the total frame height, – base shear, are shown in Figure 3, considering the distribution of forces Mxϕ, and in Figure 4, considering the distribution M. Each pushover is plotted until the displacement corresponding to the available capacity at Near Collapse Limit State; furthermore, the circle represents the demand at this Limit State, while the rhombus and the triangle the availability and the demand respectively at the Ultimate Limit State. The small square point indicates the displacement at the mechanism.
Capacity curves given by the MxΦ forces distribution (Fig.3) present a trilinear shape, as expected considering the trilinear skeleton curve assigned to the plastic hinges; variations between frames of post-cracking stiffness are due to differences in reinforcement.
All structures are largely verified, both at Ultimate and at Near Collapse Limit State, with very little differences in terms of response of the frames with overstrength discontinuities with respect to the reference one. The mechanism always largely precedes the availability, showing a positive ductile behaviour. Consequently, it can be stated that the seismic behaviour of all the considered frames is regular. Besides, the results showed in (Magliulo et al., 2004) appear confirmed; in particular, it can be observed that the capacity design applied at the beam – column joints by the (2.1) regularizes the response of frames with overstrength discontinuities in elevation. The observed results are also conditioned by the not large overstrength variations between adjacent storeys.

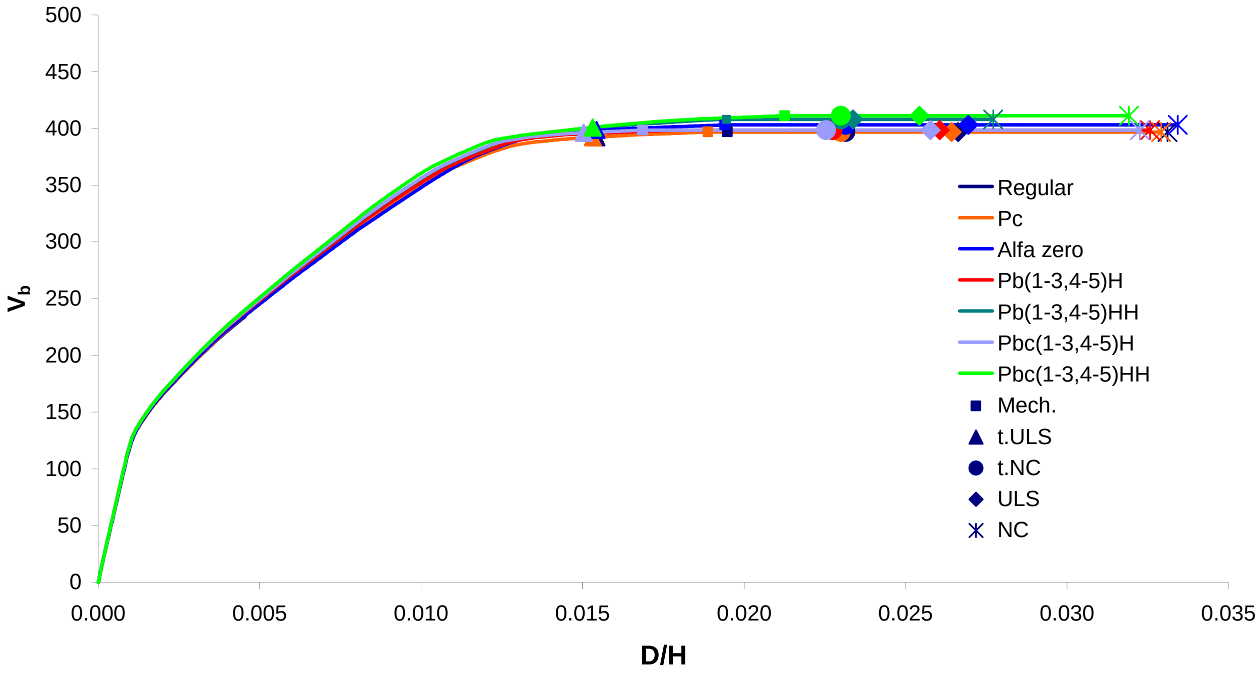

Figure 3 Capacity curves of analysed frames with available and demanded displacements (MxΦ forces distribution)

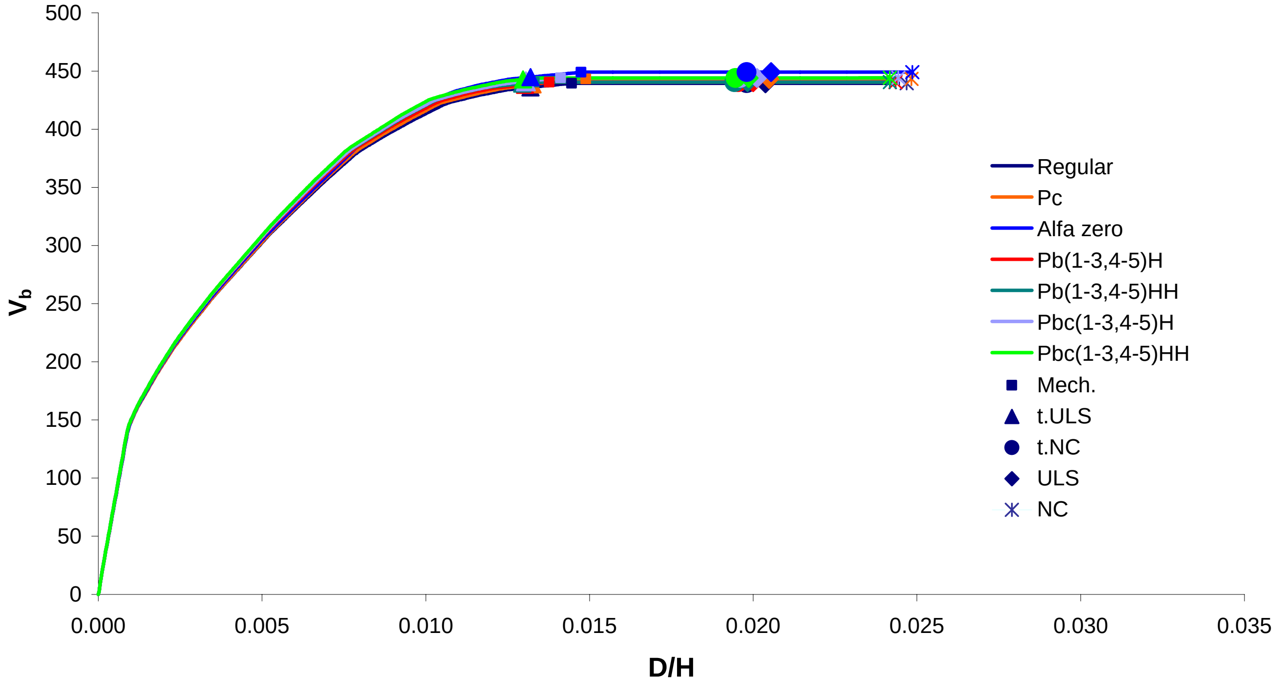

Figure 4 Capacity curves of analysed frames with available and demanded displacements (M forces distribution)

The reported conclusion is also confirmed by the demanded/available rotational ductility ratios showed in Figures 5 and 6. In the first one the maximum availability/capacity ratios obtained by NLSA at Ultimate (left) and at Near Collapse (right) Limit State are reported in percentage; for each frame, they are evaluated considering the maximum among the values obtained at all the beam and column ends (obviously for beams both positive and negative rotations are considered). Demanded/available rotational ductility ratios obtained by NLDA are reported in Figure 6; for each element end, the average of 14 maximum ratios, obtained by the 14 considered accelerograms, is computed and the maximum for each frame is reported.

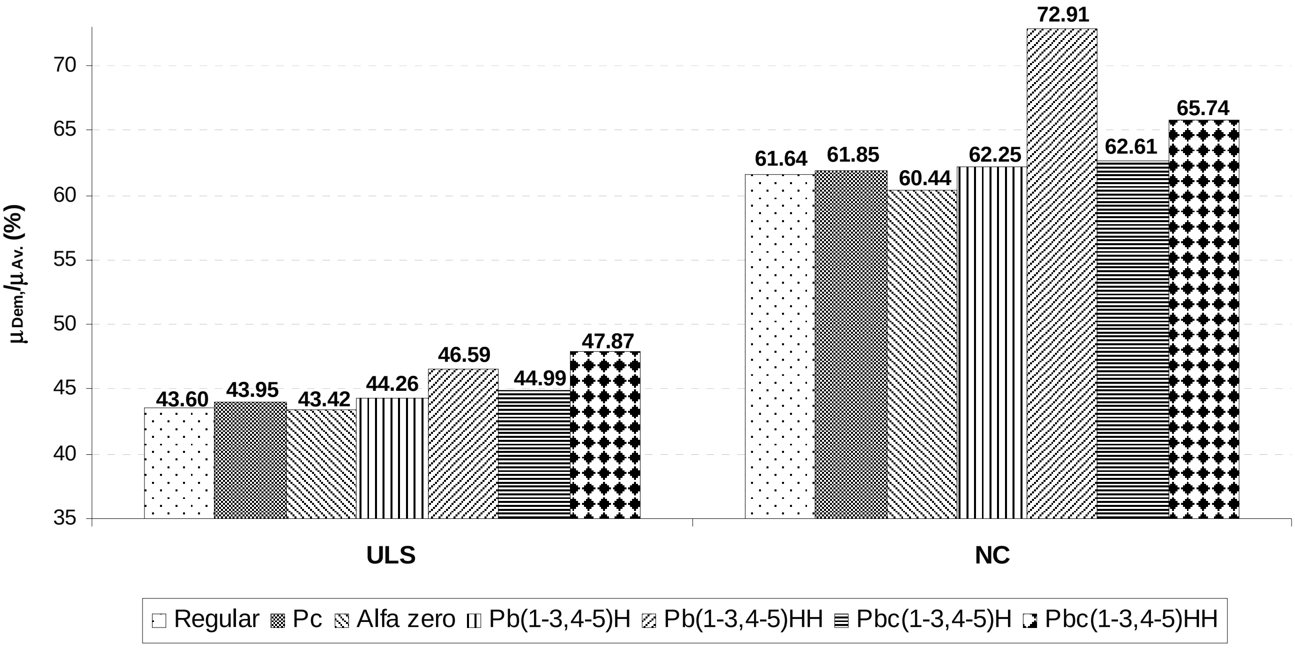

Figure 5 Maximum demand/availability ratios in terms of ductility obtained by the NLSA (MxΦ forces distribution)

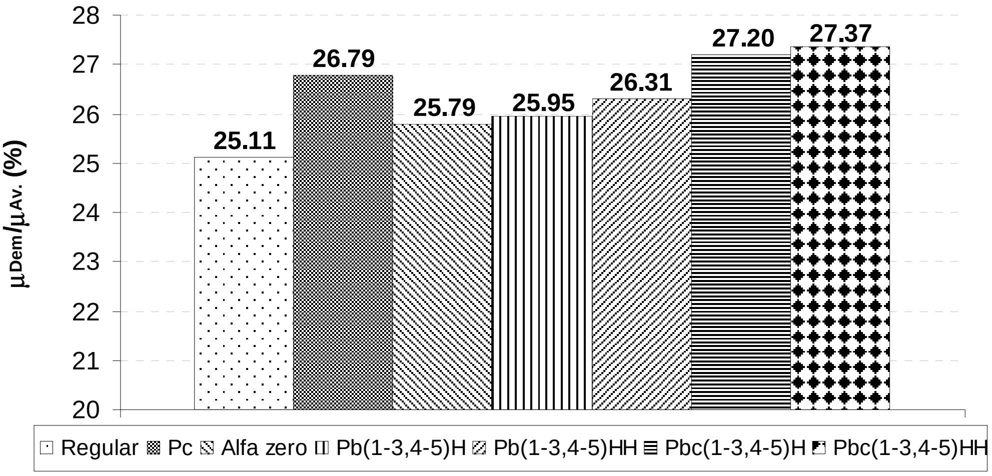

Figure 6 Maximum demand/availability ratios in terms of ductility obtained by the NLDA

5. DISCUSSION ON THE CODE CRITERIA FOR THE ASSESSMENT OF THE STRENGTH REGULARITY IN ELEVATION

In order to apply the vertical strength distribution regularity criteria of the most modern seismic international codes, it is necessary to evaluate the storey strength. If the flexural deformability of the beams is considered,

such parameter cannot be univocally defined; consequently in the paper it is evaluated according to two different methods: 1) storey strength is equal to the sum of the yielding bending moments at the ends of the columns belonging to the storey divided by the storey height; 2) such method is equal to the previous one, but the yielding bending moment at each end of the storey columns is multiplied by the ratio between the sum of the yielding bending moments of the beams and the sum of the yielding bending moments of the columns converging into the same joint, if such ratio is lower than one. The yielding bending moments at elements ends are assumed equal to the maximum bending moments of the corresponding sections.
The judgement concerning vertical strength distribution regularity (R: regular, I: irregular) of the analysed frames given by Eurocode 8 (CEN, 2003), by the New Italian Seismic Code (OPCM 3431, 2005), by NBC (http://www.nationalcodes.ca/consult/tc/nbc/part4/part4_en.pdf, 2005), by IBC (International Code Council, 2000), by SEAOC (SEAOC, 1999), ATC40 (Applied Technology Council, 1996), and by NEHERP (Building Seismic Safety Council, 2001) are reported in Table 1; since the Eurocodice 8 only provides a qualitative regularity threshold, this is assumed equal to the OPCM 3431, i.e. a variation of the overstrength between adjacent storeys equal to 20%. The two methods of storey strength evaluation provide for all the frames and for all the considered codes the same judgement, consequently in the Table they are not distinguished. The positive judgement of regularity of OPCM 3431 is conditioned by its provision which states that all the buildings designed according to High Ductility Class rules are regular in terms of vertical strength distribution: the analysis confirms the congruity of this positive judgement. Furthermore, it seems that the overstrength variation limit equal to 20% is too low and should be increased. In particular, also comparing the obtained results to others shown in (Magliulo et al., 2004), it can be stated that if the frames are designed applying the capacity design rule (2.1) at the beam-column joints, their seismic behaviour is generally not conditioned by vertical strength discontinuities. The United States Codes, whose criteria are based on the absolute storey strength, judge as regular all the considered frames, as resulted by non linear analyses.

Table 3.1 Judgments of regularity according to some international codes

CODES	Reference	Pc	Pb(1-3,4-5)H	Pb(1-3,4-5)HH	Pbc(1-3,4-5)H	Pbc(1-3,4-5)HH	Alfa zero
EC8	R	I	R	I	I	I	R
OPCM 3431	R	R	R	R	R	R	R
NBC	R	R	R	R	R	I	R
IBC SEAOC ATC40 NEHRP	R	R	R	R	R	R	R

4. CONCLUSIONS

All the examined frames are largely verified both at the Ultimate and at the Near Collapse Limit State considering both static and time-history non linear analyses; consequently they exhibit a regular behaviour. This is also due to the application of beam-column capacity design rule.
The two considered methods for the evaluation of the storey strength provide equal results.
The regularity limit of variation of the overstrength between adjacent storeys equal to 20%, provided by the Italian Code and by some Authors according to analyses made on shear type frames, seems to be too low.

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