SOME CONSIDERATIONS ABOUT THE EUROCODE 8 R.C. FRAME BUILDING ASSESSMENT PROCEDURE

V. Mpampatsikos[1] R. Nascimbene[2] and L. Petrini[3]

[1] *Ph.D. Student, ROSE School, Pavia, Italy*
[2] *Dr., EUCENTRE, Pavia, Italy*
[3] *Assistant Professor, Dept. of Structural Engineering, Politecnico di Milano, Milano, Italy*
Email: vmpampatsikos@roseschool.it, roberto.nascimbene@eucentre.it, lorenza.petrini@polimi.it

ABSTRACT :

In the European seismic countries, the seismic assessment of existing structures is a priority, since the majority of the building heritage was designed according to out-of-date or even no seismic codes. The uncertainties about the nonlinear behaviour may be relevant, since the potential development and location of inelastic zones, as well as their ductility capacity, are, in general, unknown. The direct consequence is that the nonlinear response should be faced directly, with corresponding strong increase of the complexity of the assessment process. This issue was taken into account in this work; in particular, four R.C. frame buildings, all irregular and characterized by different geometric and material properties, were selected and assessed according to all the possible methods proposed in Eurocode 8 (EN 1998-3), with the following aims: i) suggesting simplified approaches and improvements for the assessment procedure, concerning the evaluation of both seismic demand and capacity of the structural members; ii) suggesting the most appropriate definition of the effective stiffness in linear analyses.

KEYWORDS: Existing R.C. frame buildings, chord rotation, shear assessment, effective stiffness

1. INTRODUCTION

In the European seismic countries, most of the existing structures was designed according to out-of-date seismic or even to non-seismic codes. The uncertainties about the nonlinear behavior are, therefore, relevant, since, generally, the presence and location of potential inelastic zones, as well as their ductility capacity, are unknown. For this reason, it is difficult to get satisfactory results, by applying the force-based assessment, obtained through an elastic analysis and reducing the internal forces by the behavior factor q. Hence, the nonlinear response should be faced directly, with considerable increase in complexity of the assessment procedure.
In this work, the assessment of RC frame buildings has been performed according to the Eurocode 8 (EN 1998-3), which proposes a force-based procedure for brittle mechanisms (shear) and a displacement-based approach for ductile mechanisms (flexure). The evaluation of both shear and deformation of the structural members requires lengthy and complex calculations (Mpampatsikos et al., 2008). On the base of these considerations, the aims of this work may be summarized as follows. i) Suggesting simplified approaches for the assessment procedure, concerning the evaluation of both seismic demand and capacity of the structural members. ii) Suggesting the most appropriate way for evaluating the effective stiffness in linear analyses, selecting between the actual secant stiffness at yielding of the structural members and a fixed ratio of their gross stiffness. In order to achieve the above goals, four RC frame buildings, located in seismic zones and built before 1980 were examined. All the results shown in this work refer to the Limit State of Significant Damage (SD LS).

2. ANALYZED BUILDINGS

The Sede Comunale, located in Vagli Sotto (Tuscany, Italy), designed in 1965, is a two-storey RC frame building with masonry infill. The shape of the building (Figure 1) is roughly rectangular (27.25m x 13.60m). The frames are mono-directional and oriented parallel to the short sides of the building. The foundation system

consists in a mat slab and footings. From extended in-situ inspections, f_{cm}=8.3MPa was obtained. From the original drawings, f_{ym}=440MPa was assumed for both longitudinal and transversal steel (Feb44k).
The "Scuola Elementare Pascoli", located in Barga (Tuscany, Italy), designed in 1978, is a two-storey RC frame structure with masonry infills. The shape of the building (Figure 2) is roughly square (40.90m x 35.60m). The frames are bidirectional. The foundation system consists in foundation beams in both principal directions. From extended in-situ inspections, f_{cm}=30MPa was obtained. From the original drawings, f_{ym}=440MPa was assumed for both longitudinal and transversal steel (Feb44k).
The "Scuola Puccetti", located in Gallicano (Tuscany, Italy), designed in 1963, is a two-storey RC frame building with masonry infill, a small basement floor and sloping roofs of varying height. The building is strongly asymmetric and not compact with respect to both principal directions (C-shape, Figure 3). The frames are mono-directional. The foundation system consists in mono-directional foundation beams. From extended in-situ inspections, f_{cm}=18MPa was obtained. From the original drawings, f_{ym}=440MPa was assumed for both longitudinal and transversal steel (Feb44k).
The "Scuola Media Don Bosco", located in Rapagnano (Marche, Italy), designed in 1962, is a three-storey RC frame structure with masonry infills. The shape of the building (Figure 4) is roughly rectangular (23.84m x 14.44m). The frames are mono-directional and oriented parallel to the short sides of the building. The foundation system consists in piles, connected by foundation beams. From extended in-situ inspections, f_{cm}=16.6MPa was obtained. From the original drawings, f_{ym}=215MPa was assumed for both longitudinal and transversal steel (Feb22k).
For all considered buildings, *PGA*=0.25g was considered for the assessment at the SD LS.

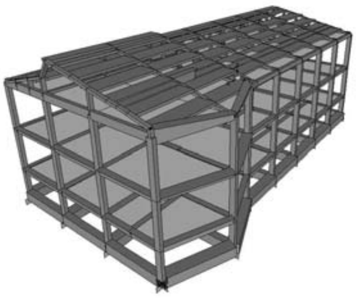

Figure 1 Sede Comunale of Vagli Sotto

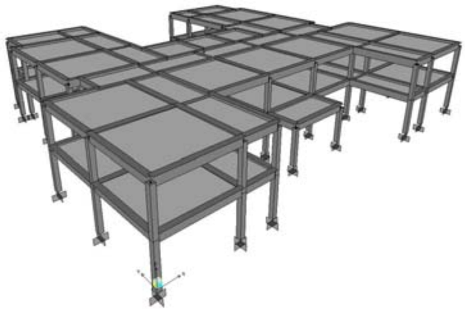

Figure 1 Scuola Elementare Pascoli

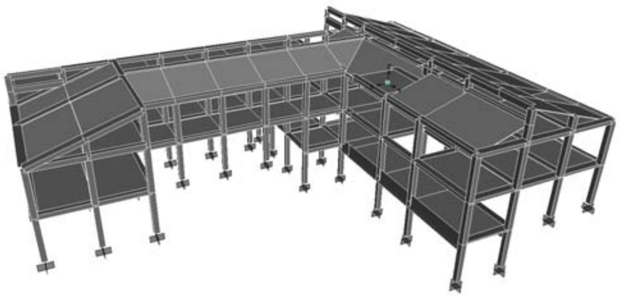

Figure 3 Scuola Elementare Puccetti

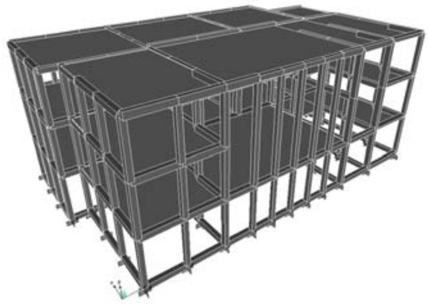

Figure 4 Scuola Media Don Bosco

3. METHODS OF ANALYSIS

Since all the structures are irregular in plan and in elevation, the conditions for the applicability of the linear static analysis are not satisfied. Hence, only linear dynamic, nonlinear static and nonlinear dynamic analyses were performed. All analyses were 3-D, because the lack of regularity did not allow to consider two planar models in the two principal directions. Close results were obtained assuming fixed foundations and modeling the soil-structure dynamic interaction (dynamic springs computed according to Gazetas, 1991). Hence, fixed foundations were assumed for all the structures.

The linear dynamic analysis was carried out using the software SAP2000. The modal superposition was performed considering enough modes to obtain a cumulative mass ≥ 90% of the total mass. The pseudo-acceleration elastic spectrum was determined according to the Italian seismic classification. E_c was evaluated according to Fib Bulletin 24: E_c=0.85·2.15·$10^4(f_c/10)^{1/3}$. This formula was used, as the expression suggested in EC8 for new constructions may be improper for the assessment of existing buildings.
The nonlinear analyses were developed through the software SeismoStruct, v.4.0.3. It is a fiber-model software, which accounts for both material and geometrical nonlinearity. The following constitutive models of the materials were assumed. i) Nonlinear constant confinement concrete model (Mander et al., 1988) with no tensile strength and unconfined ultimate strain ε_{cu}=0.004. ii) Bilinear steel model, characterized by E_s=200000MPa, strain hardening parameter μ=0.005, ε_{su}=0.04 (according to Italian Code recommendations).
The nonlinear static analysis was based on 8 conventional pushover analyses, obtained applying the "uniform" and "modal" distributions of lateral loads. With reference to the N2 method (Fajfar, 1995), the demand D_i, the capacity C_i and the ratio $R_i=|D_i/C_i|$ of each *i-th* structural member were computed for each pushover. For each *i-th* member, the largest value of R_i was used to assess the response.
The nonlinear dynamic analysis was based on 7 time history analyses. For each analysis, for each *i-th* structural member, in order to capture $R_{i,max}$, R_i was evaluated in correspondence to: i) maximum and minimum D_i; ii) maximum and minimum axial load N_i (in order to minimize C_i). Since 7 accelerograms were used, the assessment was based on the average of $R_{i,max}$. All the accelerograms were artificially generated, according to the attenuation law proposed by Sabetta and Pugliese, 1996, then modified to match the elastic spectrum.

4. DUCTILE MECHANISMS: SIMPLIFIED ASSESSMENT OF CHORD ROTATION DEMAND

According to EC8, the ductile mechanisms are assessed in terms of chord rotation, at both ends of each structural member (beams and columns). The chord rotation is the angle between the chord connecting the member end to the point of contraflexure and the tangent at the member end (Figure 5(a)). Hence, each structural member is formed by two cantilevers, fixed at the member ends and characterized by a length equal to the shear span $L_s = M/V$, where M and V are the bending moment and corresponding shear demand.

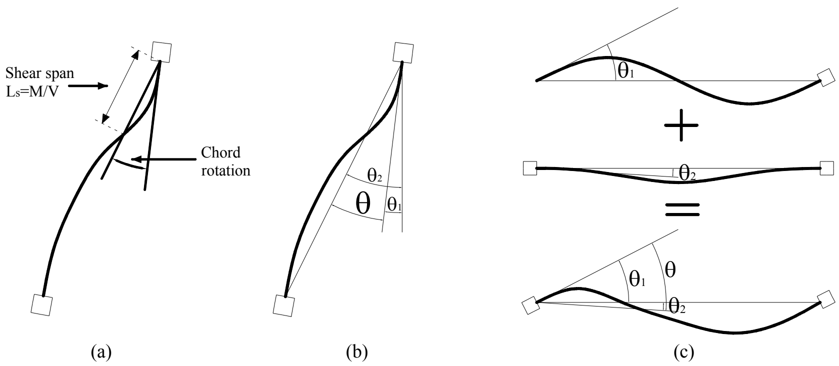

Figure 5 (a) Definition of chord rotation; (b) Chord rotation of columns; (c) Chord rotation of beams

Denoting as θ_1 the nodal rotation of the member end and as θ_2 the drift at the end of the shear span, the chord rotation demand of columns θ_c is obtained as θ_2-θ_1 (Figure 5(b)). Under seismic input, since the building is pushed laterally by the ground motion, $\theta_2>>\theta_1$. Hence, for the columns: $\theta_c = \theta_2-\theta_1 \approx \theta_2$.
The assessment of beams' chord rotation demand θ_b is more complicated, due to the presence of gravity loads. Considering the beam response as the superposition of 2 systems (beam unloaded and end sections undergoing the nodal rotations due to the seismic input; beam fixed at both ends, loaded by gravity loads), θ_b may be seen as $\theta_1+\theta_2$ (Figure 5(c)). The nodal rotations due to seismic loads, in general, are much larger than the drifts due to gravity loads. The importance of gravity loads decreases with increasing the ground motion intensity. Hence, in particular at SD and NC LS, there will be no appreciable lack of accuracy if θ_2 is neglected: $\theta_b = \theta_1+\theta_2 \approx \theta_1$.

5. DUCTILE MECHANISMS: SIMPLIFIED ASSESSMENT OF CHORD ROTATION CAPACITY

The chord rotation capacity, θ_u, depends on geometrical and mechanical properties of the member but, also, on the seismic input ($L_s=M/V$; N influences the assessment of the ultimate curvature capacity). Hence, θ_u cannot be defined as an intrinsic property of the member. The correct approach, thus, is to compute θ_u as a function of the seismic demand. Actually, the assessment would be much simpler and faster if it were possible to eliminate the demand dependence and to replace complex theoretical calculations with simpler empirical formulas. In this work, simplified approaches were examined and suggested, whenever they yield reliable results.
EC8 proposes two formulas, one based on theoretical assumptions and the other on experimental results. The empirical expression of θ_u is:

$$\theta_u = \frac{1}{1.5} 0.016 \cdot \left(0.3^{\nu}\right) \cdot \left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} \cdot \frac{f_c}{CF}\right]^{0.225} \left(\frac{L_s}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx} f_{yw}/f_c\right)} \tag{5.1}$$

where: $\nu=(N\cdot CF)/(A_c\cdot f_c)$; $\rho_{sx}=A_{sx}/b_w\cdot s_h$ is the ratio of the transverse steel parallel to the loading direction; $\alpha=(1-s_h/2b_0)\cdot(1-s_h/2h_0)\cdot(1-\sum b_i^2/(6h_0\cdot b_0))$, is the confinement factor; CF is the so-called "Confidence Factor", introduced to penalize the assessment in function of the knowledge level of the structural properties (see EN 1998-3 Section A.3.2 for the meaning of the symbols).
Eq.(5.1) is demand-dependent. N should be obtained from the analysis under the seismic load combination. Hence, if a dynamic analysis is performed, a double assessment procedure will be required (corresponding to N_{max} and N_{min}). If it were possible to consider N due only to the gravity loads (which is roughly the mean N that the columns undergo during a seismic input), the required operations would be halved. The comparison of the results obtained using the seismic and gravity N is shown in Figure 6, which refers to the percentage of mean values of $|D_i/C_i|$ (D_i and C_i are the chord rotation demand and capacity of *i-th* member), obtained through the nonlinear dynamic analysis, for the columns of all buildings. Figure 6 shows that the seismic and gravity N yield very close values. Analogous results were found for all methods of analysis. Hence, the procedure may be simplified. For slender and taller buildings, the seismic variation of N in perimetral columns could be substantial, leading to a large reduction of their ductility capacity. Nevertheless, most R.C. frame structures built in European seismic zones are low buildings and, hence, the obtained results may be regarded as general.

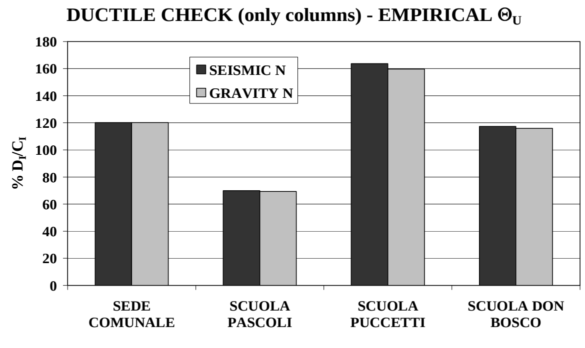

Figure 6 Nonlinear static analysis, empirical θ_u, seismic N vs gravity N

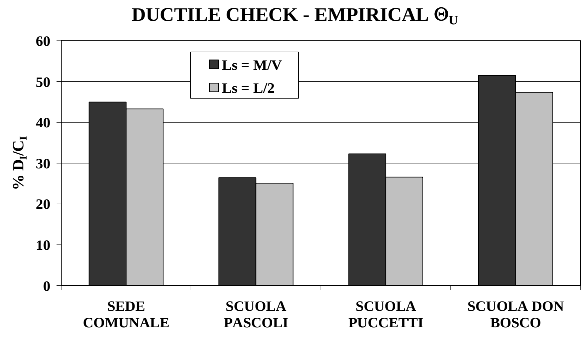

Figure 7 Nonlinear dynamic analysis, empirical θ_u, $L_s=M/V$ vs $L_s=L/2$

Computing L_s as M/V will be complex, if a linear dynamic analysis is carried out, for two different reasons: i) M and V are given as envelope values; ii). M and V grow linearly, while, when yielding take place, L_s is likely to change with respect to its elastic value. In this work, the possibility of assuming L_s equal to half the member length ($L_s=L/2$) was tested. The results are shown in Figure 7, which refers to the percentages of the mean value of $|D_i/C_i|$, obtained through the nonlinear dynamic analysis. $L_s=L/2$ and $L_s=M/V$ yield close values (differences < 15%). Analogous results were found for all methods of analysis. Hence, the procedure may be simplified.
The theoretical expression for the ultimate chord rotation capacity of structural members is:

$$\theta_u = 1/1.5\left[\theta_y + (\phi_u - \phi_y)L_{pl}\left(1 - L_{pl}/2L_s\right)\right] \tag{5.2}$$

where $\theta_y = \phi_y \cdot L_s/3 + 0.0013(1+1.5 \cdot h/L_s) + 0.13\phi_y \cdot d_b \cdot f_y/(f_c)^{0.5}$ and $L_{pl} = 0.1L_s + 0.17h + 0.24 \cdot d_{bl} \cdot f_y \cdot (CF/f_c)^{0.5}$.
In order to compute the ultimate curvature ϕ_u, section failure was conventionally considered to take place when the moment capacity M_c drops to 80% of its peak value, $M_{c,peak}$. If the spalling of concrete cover causes a drop > 20% $M_{c,peak}$, the curvature at spalling will be considered as ϕ_u. If, instead, M_c computed considering only the confined core of the section is > 80% $M_{c,peak}$, ϕ_u will be obtained at the failure of the confined concrete core. The confinement model for the compressive concrete advised in EC2 (EN 1992-1-1) was used. Since the correct procedure to evaluate θ_u according to Eq.(5.2) is complex and long, in this work attempts to propose simplified approaches were developed. Available experimental results (Priestley, 2003) showed that ϕ_y is sensitive only to h of the section and ε_{sy} of the longitudinal steel. In this work, ϕ_y was computed in three different ways: i) according to the theoretical approach; ii) according to the empirical formula (5.3), proposed by Priestley (2003); iii) according to the empirical expression (5.4), suggested by Biskinis (2006).
The comparison of the results obtained applying the different ways to evaluate ϕ_y is shown in Figure 8, which refers to the percentage of mean values of $|D_i/C_i|$, obtained through the nonlinear dynamic analysis. The theoretical and the two empirical definitions of ϕ_y yield close values. Analogous results were found for all methods of analysis. Hence, the procedure may be simplified.

$$\phi_y = 1.87\varepsilon_{sy}/h \ (rect.\ beams);\ \phi_y = 1.7\varepsilon_{sy}/h \ (T-sec.\ beams);\ \phi_y = 2.1\varepsilon_{sy}/h \ (columns) \tag{5.3}$$

$$\phi_y = 1.75\varepsilon_{sy}/h \tag{5.4}$$

The results obtained through Eq.(5.2) are very sensitive to L_s. In fact, if M is peculiarly small, it is likely that L_s will be shorter than L_{pl}. In particular, if L_{pl}>$2L_s$, θ_u will be smaller than θ_y or even negative. The results obtained considering L_s=M/V and L_s=$L/2$ were compared to check the sensitivity of Eq.(5.2) to the way of computing L_s. Figure 9 refers to the percentage of mean values of $|D_i/C_i|$, obtained through the nonlinear dynamic analysis. Applying L_s=M/V the results are much larger than assuming L_s=$L/2$.

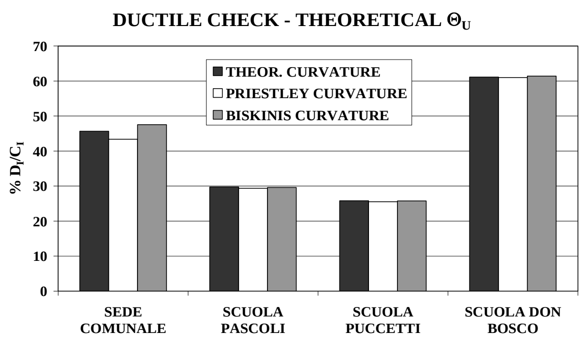

Figure 8 Nonlinear dynamic analysis, theoretical θ_u, theoretical vs empirical ϕ_y

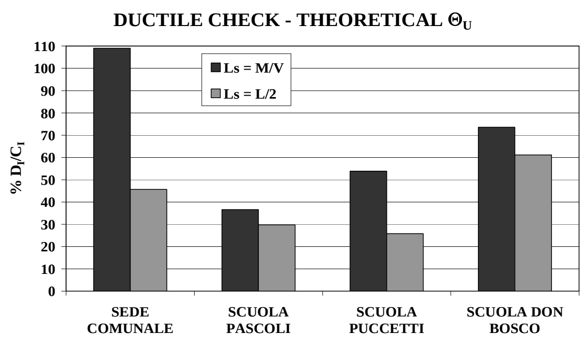

Figure 9 Nonlinear dynamic analysis, theoretical θ_u, L_s=M/V vs L_s=$L/2$

The values obtained applying the correct (seismic N, L_s=M/V, theoretical ϕ_y) and the simplified (gravity N, L_s=$L/2$, ϕ_y from Eqs.(5.3)-(5.4)) procedure of assessing θ_u to both empirical (Eq.(5.1)) and theoretical (Eq.(5.2)) formulations were compared, in order to establish which approach yields more reliable results. Figure 10 shows that the mean values of $|D_i/C_i|$, obtained through the nonlinear dynamic analysis, applying the correct approach of Eq.(5.2), overestimate sensibly all the other results. Since it has been shown that the way to assess both N and ϕ_y does not influence the results, it is clear that L_s = M/V should not be applied to Eq.(5.2), as it yields inaccurate results, due to the large sensitivity to L_s. Analogous results were found for all methods of analysis.

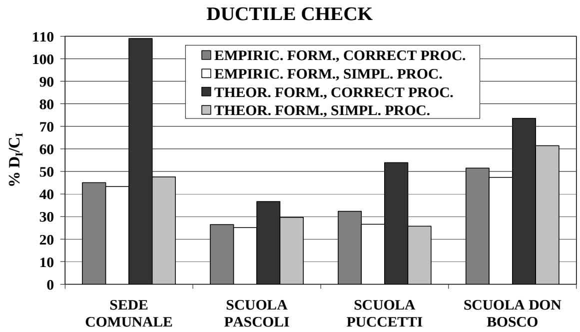

Figure 10 Nonlinear dynamic analysis, empirical vs theoretical θ_u, correct vs simplified approach

6. BRITTLE MECHANISMS: SIMPLIFIED ASSESSMENT OF THE SHEAR CAPACITY

EC8 accounts for the effects of both cycling loading and inelastic response in the assessment of V_R (see EN 1998-3 Section A.3.3 for the meaning of the symbols):

$$V_R = 1/1.15\left\{(h-x)/(2L_s)\cdot \min\left(N;0.55A_c\cdot f_{cm}/(\gamma_c CF)\right)+\left[1-0.05\min\left(5;\mu^{pl}_{\theta,dem}\right)\right]\cdot \left[0.16\max(0.5;100\rho_{tot})\left(1-0.16\min(5;L_s/h)\right)\sqrt{f_{cm}/(\gamma_c\cdot CF)}A_c+V_w\right]\right\} \quad (6.1)$$

According to Eq.(6.1), V_R decreases when the plastic part of the chord rotation ductility demand $\mu_{\theta,dem}{}^{pl}$ increases. Beyond $\mu_{\theta,dem}{}^{pl}$ = 5, assuming x constant (x does not change significantly once plastic moment developed), V_R will remain constant at its lowest value.
Eq.(6.1) is function of N, ϕ_y and L_s. In this work, the possibility of adopting simplified approaches (gravity N, empirical ϕ_y and L_s=L/2) was tested. The comparison between the results obtained considering seismic N and gravity N is shown in Figure 11, which refers to the percentage of mean values of $|D_i/C_i|$, obtained through the nonlinear static analysis. Seismic and gravity N yield very close values. Analogous results were found for all methods of analysis. Hence, it is possible to simplify the procedure.
The comparison between the results obtained considering L_s=M/V and L_s=L/2 is shown in Figure 12, which refers to the percentage of mean values of $|D_i/C_i|$, obtained through the nonlinear dynamic analysis. L_s=L/2 yields a slightly (practically negligible) overestimation of the values obtained assuming L_s=M/V. Analogous results were found for all methods of analysis. Hence, the procedure may be simplified.

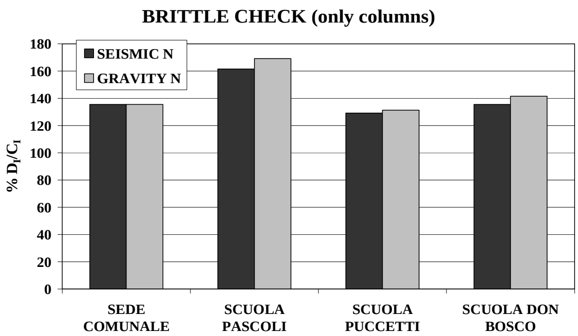

Figure 11 Nonlinear static analysis, seismic N vs gravity N

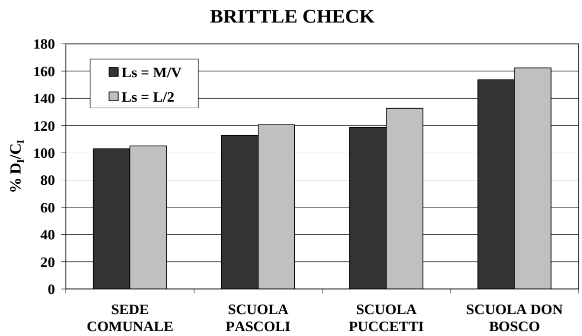

Figure 12 Nonlinear dynamic analysis, L_s = M/V vs L_s = L/2

The comparison among the results obtained considering the theoretical and the empirical formulas of ϕ_y is shown in Figure 13, which refers to the percentage of mean values of $|D_i/C_i|$, obtained through the nonlinear dynamic analysis. Theoretical and empirical formulas of ϕ_y (Eqs.(5.3)-(5.4)) yield very close values. Analogous results were found for all methods of analysis. Hence, the procedure may be simplified. Figure 14 refers to the percentage of mean values of $|D_i/C_i|$, obtained through the nonlinear dynamic analysis, considering both correct and simplified approaches. It is clear that the simplified approach may be applied without significant loss of accuracy.

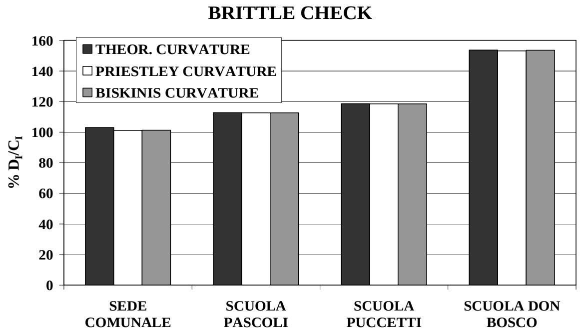

Figure 13 Nonlinear dynamic analysis, theoretical vs empirical ϕ_y

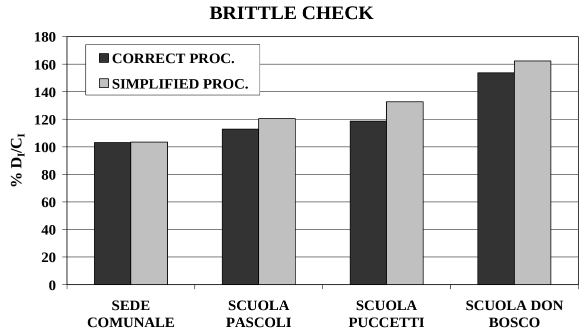

Figure 14 Nonlinear dynamic analysis, correct vs simplified approach

7. EFFECTIVE STIFFNESS IN LINEAR DYNAMIC ANALYSIS

The results of the assessment performed according to the linear dynamic analysis could be influenced by the value of the stiffness of the structural members. It is likely, in fact, that the sections undergo wide cracks under strong earthquakes, with the consequence of reducing their effective stiffness, EI_{eff}. Usually, this effect is accounted by modeling EI_{eff} of the members as a fixed ratio (i.e. 50%) of their gross stiffness. EC8 does not suggest any rule to choose properly this ratio, but recommends to compute EI_{eff} as the mean value of the secant stiffness at yielding, $EI_{sec.yield} = M_y L_s/3\theta_y$. Assuming $L_s = L/2$: $EI_{\sec,yield} = L/24\sum_{i=1}^{4} M_{y,i}/\theta_{y,i}$, where i refers to both positive and negative flexure at the two ends of the member. At a first sight, this method could seem to yield the most accurate results. Nevertheless, modeling every member as $EI_{sec.yield}$ would correspond to state that all members develop a nonlinear behavior at the "yielding" point of the structure. Instead, it is likely that the nonlinear mechanism involves only part of the structural members. In this work, EI_{eff} was assumed as EI_{gross}, 50% EI_{gross} and $EI_{sec.yield}$. The values obtained from the three ways to model EI_{eff} were compared to those got through the nonlinear dynamic analysis, taken as reference method, in order to understand which choice of EI_{eff} yields the most reliable values.

Figure 15 shows the results of the assessment of the ductile mechanisms obtained assuming the empirical formulation of θ_u. Assuming $EI_{eff} = EI_{sec.yield}$ for each structural member leads to underestimate grossly the elastic slope of the bilinear approximation of the F-Δ curve of the whole structure and, hence, to overestimate sensibly the results in terms of mean values of $|D_i/C_i|$. For all the considered structures, a value of EI_{eff} between 50% and 100% EI_{gross} seems suitable to obtain values close to those got through the nonlinear dynamic analysis. In particular, for the Sede Comunale (f_{cm}=8.3MPa), $EI_{eff} = EI_{gross}$ yields the most accurate results while, for the other three buildings, characterized by larger values of f_{cm}, EI_{eff} close to 50% EI_{gross} is the most reliable solution.

Figure 16 shows the results of the assessment of the brittle mechanisms. For all buildings, all the three ways to model EI_{eff} yield values rather close to those got through the nonlinear dynamic analysis and, also, close to each other (negligible differences, except for the Scuola Puccetti, i.e. the most irregular building). It may be justified considering that, according to EC8, the shear demands obtained directly from the linear dynamic analysis should be limited, accounting for the development of the flexural nonlinear mechanisms at the ends of the members. Hence, it may be concluded that, if the seismic action is large enough to produce a nonlinear response and if the building is not extremely irregular, the results will be independent of the way to model the members' stiffness, EI_{eff}, and, hence, in order to reduce the computational efforts, the use of the same values of EI_{eff} considered for the assessment of ductile mechanisms is advised.

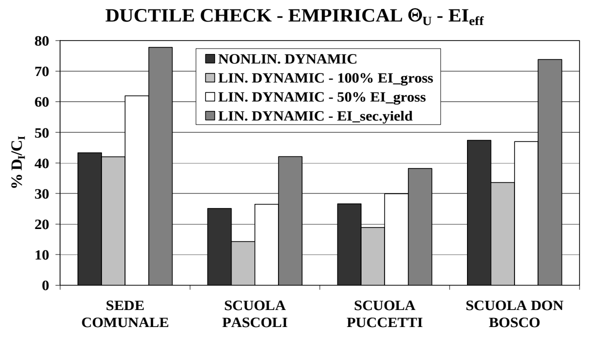

Figure 15 Ductile check, nonlinear dynamic analysis vs. linear dynamic analysis with different EI_{eff}

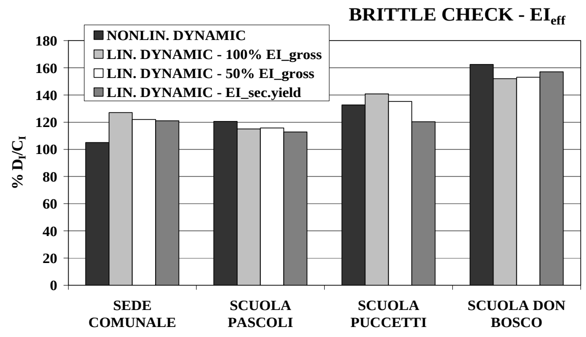

Figure 16 Brittle check, nonlinear dynamic analysis vs. linear dynamic analysis with different EI_{eff}

8. CONCLUSIONS

In this work, simplified approaches for the assessment procedure of R.C. frame buildings were proposed and verified, examining four existing structures, located in seismic zones and built before 1980. Considering the different structural configurations of the analyzed buildings and the wide number of assessed structural members, characterized by different shape, dimension, length and reinforcement content, the conclusions may be judged as satisfactory, although influenced by the considered numerical models.

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