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DEVELOPMENT OF A SIMPLIFIED DISPLACEMENT-BASED PROCEDURE FOR THE SEISMIC DESIGN AND ASSESSMENT OF RC FRAME STRUCTURES

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ABSTRACT

Displacement-based design methods can solve many deficiencies with classical force based methods, but on the other hand they require more computational and conceptual effort. While such effort is deemed appropriate and necessary for regions of high seismicity, it is considered that a simplified alternative might be appropriate for regions of low to moderate seismicity. As such, a simplified displacement based procedure (for both assessment and new design of buildings) is proposed in this work, in which the displacement capacity is estimated assuming a soft-storey mechanism and the displacement demand is taken as the peak spectral displacement demand. In this way, no estimate is required of the building strength, stiffness or period of vibration, thereby greatly simplifying the task of seismic assessment. The testing of the simplified procedure, through the application to several case study buildings and the comparison of the results with those of non-linear dynamic analyses, indicates that the methodology performs well. However, a greater range of case study structures should be examined as part of future research to thoroughly identify the limits of applicability of the proposed approach.

INTRODUCTION

In the early ninetites (Moehle, 1992; Priestley, 1993) it was proposed that the use of deformations, rather than forces, would form a more appropriate basis for seismic design methods. As result of these observations, design methods that control the deformations, so-called displacement-based design (DBD) methods, were developed. These new DBD methods are able to remedy many deficiencies associated with force-based design, as described in detail by Priestley *et al.* (2007), but on the other hand they usually require more computational and conceptual effort than the simple equivalent lateral force method currently found in modern building codes.

While the effort required to undertake a rigorous application of DBD may certainly be appropriate and necessary for regions of high seismicity, it is considered that a simplified alternative might be appropriate for regions of low to moderate seismicity. As such, this work investigates the performance of a simplified displacement-based design and assessment procedure intended for use in regions of low to moderate seismicity. The idea at the heart of the proposed procedure, that builds on proposals made by Priestley *et al.* (2007) and Pinho *et al.* (2007), is to evaluate a building's displacement capacity using conservative approximations and simplified equations derived from DDBD, and to compare the capacity with the maximum displacement demand from an elastic response spectrum. In this way, no estimate is required of the building strength, stiffness or period of

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vibration, thereby greatly simplifying the task of seismic assessment. The approach could be used either for the assessment of an existing building or to check seismic performance of a new building during the design phase.

The objective of this work is to explore a simplified methodology for the seismic design and assessment of RC frames structures that can provide a satisfactory compromise between the accuracy of direct displacement-based design and the simplicity needed to be comparable to the equivalent lateral force method. Proposals for simplified DBD have already been made by Sullivan (2013, 2013a) but this paper aims for an even more simplified approach. In this research the proposed simplified procedure is described and then tested through the application to several case study buildings and the comparison of the results with those of non-linear time history (NLTH) analyses.

SIMPLIFIED DISPLACEMENT-BASED METHODOLOGY

The idea at the basis of the simplified method is to first compute the displacement capacity of a structure in a simplified manner as a function of the geometric proportions of the structure, the characteristics of the materials and the likely structural detailing. This displacement capacity is then compared with a demand displacement, taken as being equal to the maximum spectral displacement demand for a specific return period event. In this way, if the capacity is greater than the demand, it is assumed that the seismic risk for the building is sufficiently low and there is no need for detailed seismic design or assessment.

The procedure proposed is the same for both the cases of assessment and new design, except that in the latter a design for non-seismic loads needs to be carried out before the application of the method. To keep the procedure simple the principles of capacity design are not considered, but for new construction in Italy it is assumed that a moderately ductile behaviour is assured by complying with the seismic requirements for the ductility class B of the Italian building code (NTC08) regarding reinforcement quantities and structural detailing, which can be summarised as follows:

Beams:

- Longitudinal tension reinforcement, geometric percentage:

$$\frac{1{,}4}{f_{yk}} < \rho < \rho_{comp} + \frac{3.5}{f_{yk}} \tag{1}$$

  where ρ is geometric ratio of the tension reinforcement, ρ_{comp} the geometric ratio of the compression reinforcement and f_{yk} the characteristic yielding tension (in units of MPa).

- Transverse reinforcement, maximum distance between stirrups:

  s < min (0.25 h; 225 mm; $8\phi_l$; $24\phi_t$), 135° hooks

  where h is the section depth, ϕ_l the minimum diameter of the longitudinal bars and ϕ_t the minimum diameter of the transverse bars.

Columns:

- Longitudinal reinforcement, geometric percentage:

$$1\% < \rho < 4\% \tag{2}$$

  For the entire length of the column the spacing between bars must be less than 25 cm, and the distance between tied longitudinal bars must not be less than or equal to 20 cm.

- Transverse reinforcement, maximum distance between stirrups:

$$\frac{A_{st}}{s} \geq 0.08 \frac{f_{cd} . b_{st}}{f_{yd}} \quad (3)$$

s < min (0.5b_w; 175mm; 8ϕ_l), 135° hooks

where A_{st} is the total area of the stirrups legs in the section, f_{cd} the design strength of the concrete, f_{yd} the design yielding strength of the steel, b_{st} the distance between the external stirrups legs, b_w the width of the column section and ϕ_l the minimum diameter of the longitudinal bars.

For the longitudinal reinforcement, deformed reinforcement must be exclusively used.

As specific capacity design rules are not imposed, the displacement capacity is obtained with the hypothesis of the occurrence of a column-sway mechanism, concentrating all the deformations in a single floor and neglecting the elastic deformations of the other floors. To determine the capacity of the structure it is then necessary to calculate the inter-storey drift limit. Two values are considered, one corresponding to the deformation capacity of the structural elements, and one corresponding to the capacity of non-structural elements. The limit value will be the lower between the two. Since the occurrence of a column sway mechanism is considered, the drift limit for the structural elements is calculated solely for columns, which are assumed to absorb all the plastic deformation. The steel and concrete deformation limits considered are the ones proposed by Crowley *et al.* (2006), reported in Table 1 (for inadequately confined members, typical of buildings designed following old buildings codes) and in Table 2 (for adequately confined members). Also reported below is the expression used to calculate the structural drift limit, derived modifying the formulas proposed by Crowley *et al.* (2006) (the elastic part of the deformation is not calculated, assigning, with an approximation, all the rotation to the plastic hinge).

$$\theta_{\lim,s} = \frac{\varepsilon_{C(LSi)} + \varepsilon_{S(LSi)}}{h_c} L_p \approx \frac{\varepsilon_{C(LSi)} + \varepsilon_{S(LSi)}}{h_c} 0.5 h_c = \frac{\varepsilon_{C(LSi)} + \varepsilon_{S(LSi)}}{2} \quad (4)$$

where L_p is the plastic hinge length, ε_c is the concrete strain, ε_s is the steel strain and h_c is the depth of the column section. This expression is not expected to be non-conservative at times since it is not likely that both the tension and compression strain limits are attained simultaneously. In addition, the expression should only be applied for slender elements (e.g. storey height to section depth ratio of 6 or more for columns) where the plastic hinge length might reasonably be approximated as half the section depth. However, despite its approximate nature, the expression is useful since it is independent of section dimensions.

Table 1. Steel and concrete deformation limits for different limit states from Crowley *et al.* (2006) (for inadequately confined members).

Limit state	Structural damage	Materials deformations
LS1	Absent or light	Yield limit
LS2	Moderate	ε_c = 0.004 - 0.005, ε_s= 0.01 - 0.015
LS3	Extended	ε_c = 0.005 - 0.01, ε_s= 0.015 - 0.03

Table 2. Steel and concrete deformation limits for different limit states from Crowley *et al.* (2006) (for adequately confined members)

Limit state	Structural damage	Materials deformations
LS1	Absent or light	Yield limit
LS2	Moderate	ε_c = 0.004 - 0.005, ε_s= 0.01 - 0.015
LS3	Extended	ε_c = 0.01 - 0.02, ε_s= 0.04 - 0.06

For the drift limit $\theta_{lim,ns}$ related to the non-structural elements, any reasonable value could be considered and in the following applications to the cases study buildings a limit of 2% is considered for the damage-control limit state. Once θ_{lim} (minimum between $\theta_{lim,s}$ and $\theta_{lim,ns}$) is calculated, the displacement profile is obtained by considering only the deformation in the first floor, where the formation of a soft storey is assumed, and the elastic deformation in the upper floors is neglected. The displacement capacity is thus equal to:

$$\Delta_d = \theta_{\lim} . h_s \tag{5}$$

where h_s is the inter-storey height.

The value of the maximum displacement demand is taken as the maximum elastic spectral displacement S_d, corresponding to the plateau of the spectrum. The calculation of the effective period and the equivalent viscous damping of the structure typically required for a rigorous DBD solution is therefore avoided, comparing the displacement capacity with the maximum likely non-dissipative response of the structure. Figure 1 schematically illustrates the proposed procedure.

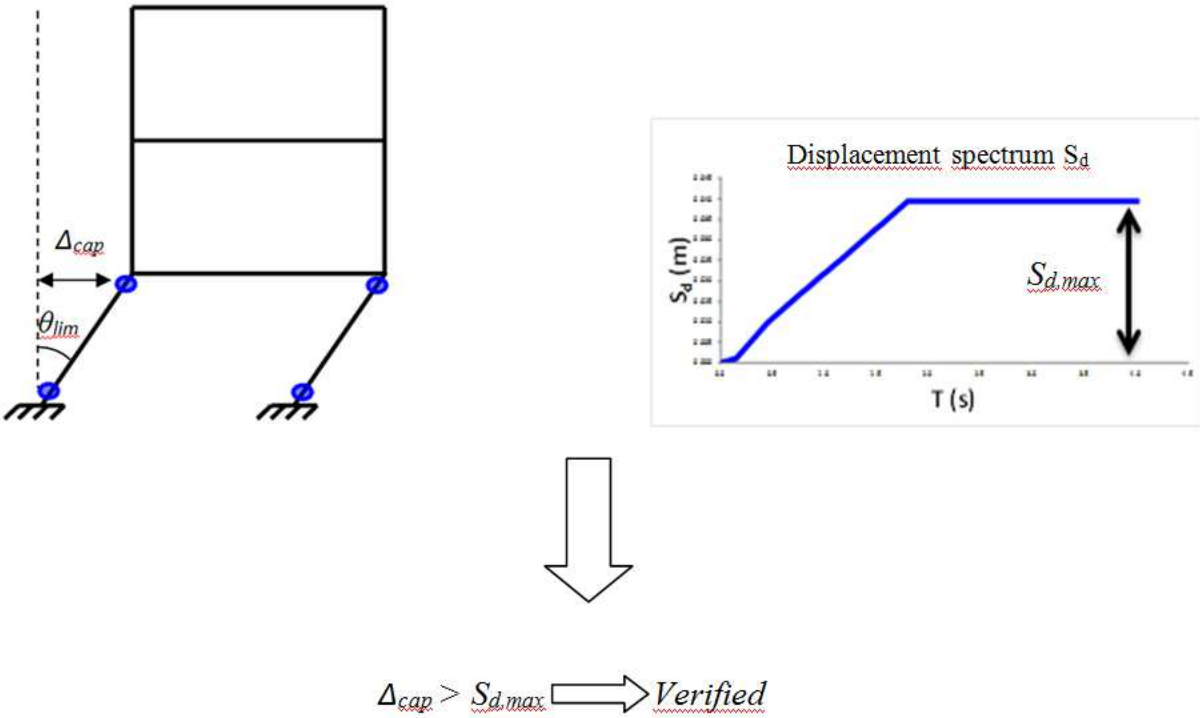

Figure 1. Scheme of the proposed procedure

TRIAL APPLICATIONS

Three buildings are designed with reference to two different zones, one with low seismicity (Milan) and one characterised by medium seismicity (Bologna). In the application of the simplified method (and in the non-linear analysis) only the damage-control limit state is considered, although the procedure proposed could be adapted to any limit state. The study cases examined are reinforced concrete buildings, designed for residential use. The buildings considered in the study are approximately regular in height and in plan and are made up of three and four parallel frames in the x and y directions respectively. The dimensions in plan are the same for all the cases, equal to 10x9 m (Figure 2), while the heights and the number of storeys (3, 6 and 9) are different. The sections of columns and beams change depending on the height of the building and the type of building code used. In Figure 2 the plan and the elevation of the frame in x-direction are shown for the three storey case.

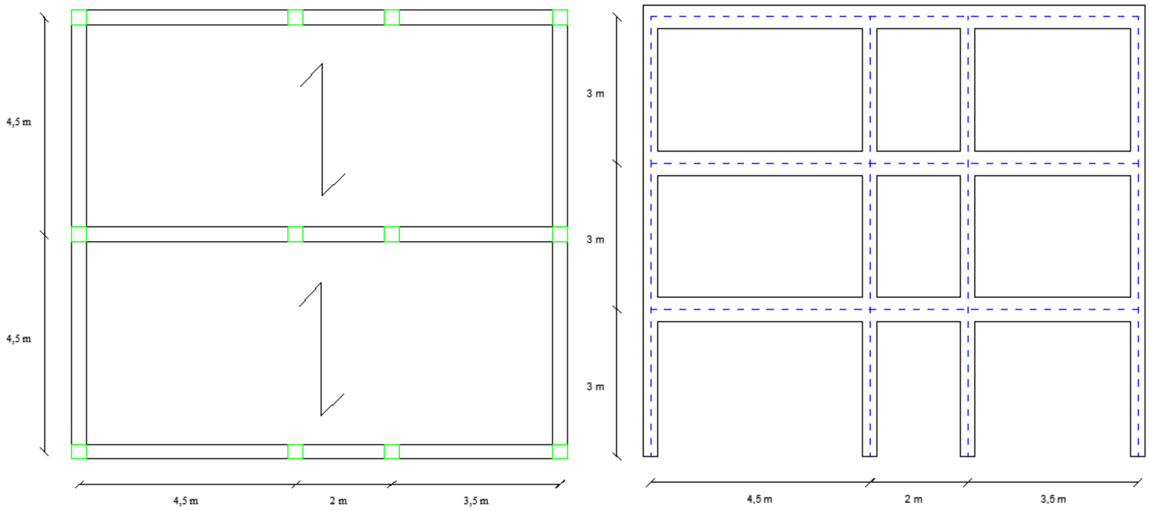

Figure 2. Building plan and x-direction frame for the 3 storey building

In the case of assessment the cases study buildings were considered designed following the prescriptions of the old building codes in use in the 1950s – 1970s. Some typical weaknesses of the buildings designed with old codes are therefore considered, such as the lack of confinement due to the use of 90 degree hooks in the stirrups, or the presence of weak beam-column nodes due to use of the smooth reinforcement bars and the lack of transversal reinforcement in the joints. From the examination of the old Italian codes it is deduced that the seismic norms until the 1970s were based mainly on the Regio Decreto 2229/39 published in 1939. A simulated design for only static loads, based on the requirements for materials, geometry and reinforcement provided by Regio decreto of 1939 (Piazza, 2014), was carried out to determine the characteristics of the study cases buildings to be used in the assessment procedure. Moreover, the elements have been designed according to the principle of allowable stresses, as was the norm in that period. The materials properties indicated in Table 3 have been considered in the design. The values adopted, typical for the old buildings, are the same used in the work of Galli (2006).

Table 3. Materials characteristics (assessment case)

Concrete		Steel	
σ_c (MPa)	**$\sigma_{c,allowable}$ (MPa)**	**σ_y (MPa)**	**$\sigma_{y,allowable}$ (MPa)**
20	6.7	380	160

In the new design case, the design of the buildings has been carried out following the prescriptions of the current Italian code for ductility class B. The material properties indicated in Table 4 have been considered in the design for static loads.

Table 4. Materials characteristics (new design case)

Concrete		Steel	
f_{ck} (kN/m^2)	**f_{cd} (kN/m^2)**	**f_{yk} (kN/m^2)**	**f_{yd} (kN/m^2)**
25	14.2	450	391

The sections dimensions and the quantity of reinforcement obtained for the study cases are shown in the Table 5, while the results of the application of the proposed methodology are shown in

Table 6. It is noted that the simplified procedure led to the same results independently of the height of the buildings.

Table 5. Sections dimensions and quantities of reinforcement

Buildings	**Columns section dimensions**	**Columns longitudinal reinforcement**	**Columns transverse reinforcement**	**Beams sections dimensions**	**Beams longitudinal reinforcement**	**Beams transverse reinforcement**
3 storey (assessment case)	250x250 mm	2+2 Φ 14	Φ 6, s = 120 mm	300x500 mm (external beams), 800x200 mm (internal beams)	2 + 2 Φ 12 , 2 Φ 14 (external beams), 6 + 6 Φ 14, 6 Φ 14 (internal beams)	Φ 6, s = 140 mm (external beams), Φ 6, s = 100 mm (internal beams)
6 storey (assessment case)	350x350 mm (for floors 1 to 3), 250x250 mm (for floors 4 to 6)	2+2 Φ 14	Φ 6, s = 140 mm (for floors 1 to 3), Φ 6, s = 120 mm (for floors 4 to 6)	300x500 mm (external beams), 800x200 mm (internal beams)	2 + 2 Φ 12 , 2 Φ 14 (external beams), 6 + 6 Φ 14, 6 Φ 14 (internal beams)	Φ 6, s = 140 mm (external beams), Φ 6, s = 100 mm (internal beams)
9 storey (assessment case)	450x450 mm (for floors 1 to 3), 350x350 mm (for floors 4 to 6), 250x250 (for floors 7 to 9)	2+2 Φ 14	Φ 6, s = 140 mm (for floors 1 to 3), Φ 6, s = 140 mm (for floors 4 to 6), Φ 6, s = 120 mm (for floors 7 to 9)	300x500 mm (external beams), 800x200 mm (internal beams)	2 + 2 Φ 12 , 2 Φ 14 (external beams), 6 + 6 Φ 14, 6 Φ 14 (internal beams)	Φ 6, s = 140 mm (external beams), Φ 6, s = 100 mm (internal beams)
3 storey (new design case)	300x300 mm	4+4 Φ 14	Φ 8, s = 110 mm	300x500 mm	2 + 2 Φ 12 , 2 + 2 Φ 14	Φ 8, s = 90 mm
6 storey (new design case)	400x400 mm (for floors 1 to 4), 300x300 mm (for floors 5 to 6)	4+4 Φ 16 (for floors 1 to 4), 4+4 Φ 14 (for floors 5 to 6)	Φ 8, s = 100 mm (for floors 1 to 4), Φ 8, s = 110 mm (for floors 5 to 6)	300x500 mm	2 + 2 Φ 12 , 2 + 2 Φ 14	Φ 8, s = 90 mm
9 storey (new design case)	500x500 mm (for floors 1 to 4), 400x400 mm (for floors 5 to 7), 300x300 (for floors 8 to 9)	4+4 Φ 20 (for floors 1 to 4), 4+4 Φ 16 (for floors 5 to 7), 4+4 Φ 14 (for floors 8 to 9)	Φ 8, s = 80 mm (for floors 1 to 4), Φ 8, s = 100 mm (for floors 5 to 7), Φ 8, s = 110 mm (for floors 8 to 9)	300x500 mm	2 + 2 Φ 12 , 2 + 2 Φ 14	Φ 8, s = 90 mm

Table 6. Results of the simplified procedure

	$\theta_{lim,s}$	$\theta_{lim,ns}$	θ_{lim}	Δ_d [m]	$S_{d,max}$ [m] (Milano)	$S_{d,max}$ [m] (Bologna)	$\Delta_{cap} > \Delta_{dem}$ (Milano)	$\Delta_{cap} > \Delta_{dem}$ (Bologna)
Assessment case	0.02	0.02	0.02	0.06	0.039	0.157	0.06 > 0.039 (verified)	0.06 < 0.157 (not verified)
New design case	0.03	0.02	0.02	0.06	0.039	0.157	0.06 > 0.039 (verified)	0.06 < 0.157 (not verified)

MODELLING AND ANALYSIS

In order to verify the proposed procedure, several time history analyses have been carried out using the non-linear software Ruaumoko (Carr, 2009). The models of the buildings are shown in Figure 3 (only for the bare frame case).

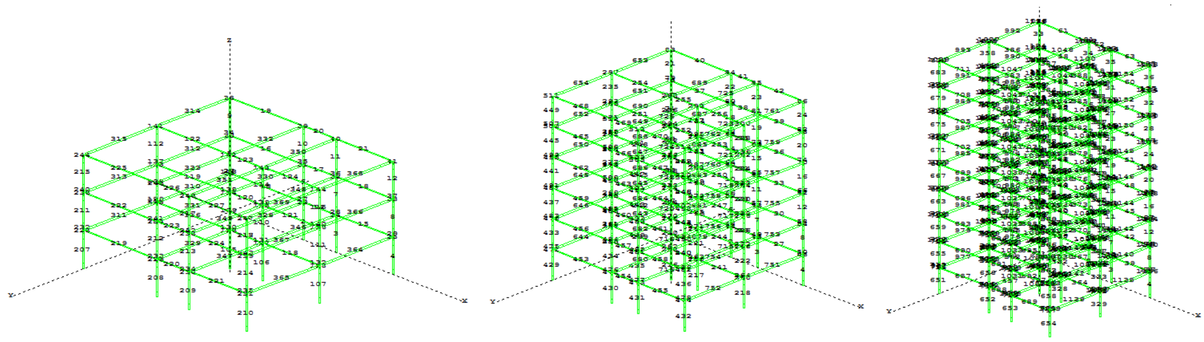

Figure 3. Ruaumoko buildings models (3, 6 and 9 storey, bare frame case)

As explained earlier, the simplified proposed method is based on initial (probably conservative) assumptions regarding the mechanism of collapse and the drift limit. While approximate flexural deformation limits and non-structural drift limits are considered in the simplified method, it is clear that many other mechanisms could affect the response and so in the non-linear analysis a wide evaluation of the many possible mechanisms is needed to verify the methodology. The mechanisms of failure considered in this work are listed below:

- Curvature failure - to find the curvature limits a moment-curvature analysis was carried out with the software Cumbia (Montejo and Kowalsky, 2007)
- Shear failure (the shear resistance is calculated with the NTC08 prescriptions)
- Masonry failure - a drift limit for the masonry infills is considered, as suggested by Calvi (1999)
- Beam-column joint failure - only in the assessment cases, a limit value for the rotation γ of the joint is considered, as described by Pampanin *et al.* (2002)

Several configurations of the buildings (Figure 4) were also investigated in this work, as listed below:

- Bare frame
- Infilled frame
- Pilotis frame (only the first floor is without infills)
- Asymmetric infilled frame (infills are disposed only along one side of the building)
- Short column frame of 0.5m length (infills do not extend up the full inter-storey height, but just for 2.5m)
- Bare frame with weak beam-column joints (only in the assessment case)

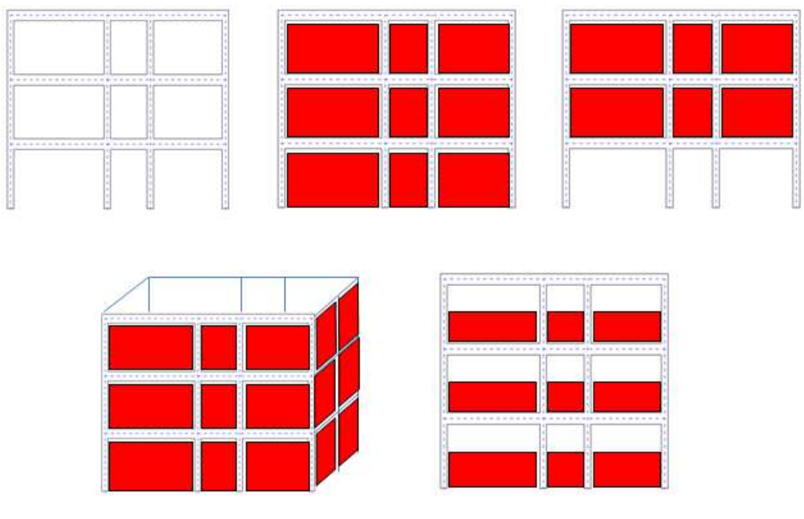

Figure 4. Overview of the various masonry configurations considered during the non-linear dynamic analyses (in the figure only the three storey case is shown).

In the NLTH analyses, frame elements were used for the columns and the beams, with hysteretic behaviour defined by the Takeda hysteresis (with r = 0.05, α = 0.5 and β = 0), while spring elements were adopted for the infills and the joints. The infills were modelled with a single strut approach, following the model proposed by Bertoldi *et al.* (1993). The typology of infills chosen is the double layered infill, from the work of Hak (2010) and Hak *et al.* (2012). To consider the behaviour of the beam-columns joints, only in the assessment case, the model proposed by Pampanin (2002) was used, with the modifications made by Trowland (2003).

The damping matrix, defined as ICTYPE 1 in Ruaumoko, is based on a Rayleigh damping model and uses secant stiffness of the structure at any time step as the tangent damping matrix. An initial damping value equal to 5% at the first and the second period of vibration has been assigned. The masses for each floor were considered as lumped masses in the nodes at the ends of the columns, which, at every level, are tied to a master node, thus creating a rigid diaphragm.

A set of 10 accelerograms was used as input for the non-linear analyses (Piazza, 2014). The records have been scaled so that their average matched the displacement spectrum of the two locations considered (Figure 5).

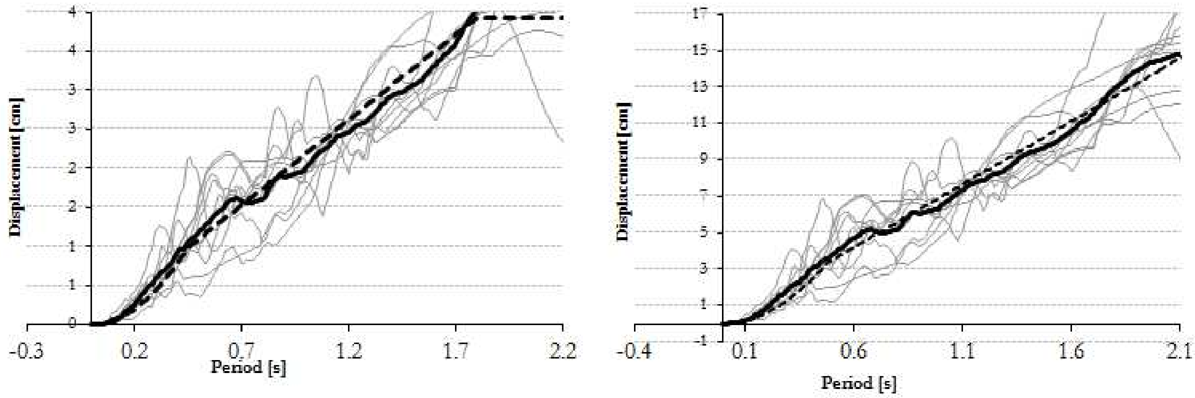

Figure 5. Elastic displacement spectra from the building code (black dashed lines, respectively for the towns of Milan, on the left, and Bologna, on the right), spectra of the individual scaled accelerograms (in gray) and their average (black continuous lines).

The numerical integration was performed using the Newmark method with constant acceleration, with a time step of 1/10 of each earthquake accelerogram time step.

RESULTS

After the non-linear analyses, for each type of structure, the maximum demand/capacity ratio among those obtained for different mechanisms was taken and compared with that determined with the simplified method. The graphs shown in Figure 6, 7, 8 and 9 illustrate the comparison between the results obtained with the non-linear analysis (histograms, representing the median of the analysis results for the ten accelerograms) and the result obtained with the simplified method (dashed line in the graphs), for both locations considered (and for assessment and new design cases).

Comparing the results obtained from the analysis with the result of the simplified procedure, in terms of maximum demand/capacity ratios, it has been possible to make some observations, the generality of which are restricted by the small number of buildings, mechanisms of failure and infill configurations considered. Regarding the assessment case, it is observed that in all the configurations of the buildings the demand/capacity ratio expected from the simplified displacement assessment is larger than the values obtained from NLTH analyses. Increasing the number of storeys the reliability of the simplified assessment remains satisfactory and the method is conservative in all the cases. These observations were made for both the low and medium seismicity cases (Milan and Bologna). Considering the results for the new design case, the methodology appears to be even more conservative. As for the assessment, in all the cases the demand/capacity ratio estimated with the simplified displacement-based approach is larger than the values obtained from the analysis. The proposed procedure was also found to be conservative for the higher buildings of 6 and 9 storeys, where the effect of higher modes and P-delta, not considered in the simplified method, are stronger. The slight increase of the level of safety of the simplified methodology in the design case is related to the availability, in the static design step, of the seismic structural details prescribed by the Italian building code for the ductility class B. These structural details ensure improved behaviour of the buildings, in terms of shear and curvature capacity, over the buildings considered for the assessment case, designed without seismic detailing as was common in the past. Moreover, with the current building code prescriptions there is an overall increase of the strength and the stiffness of the structure.

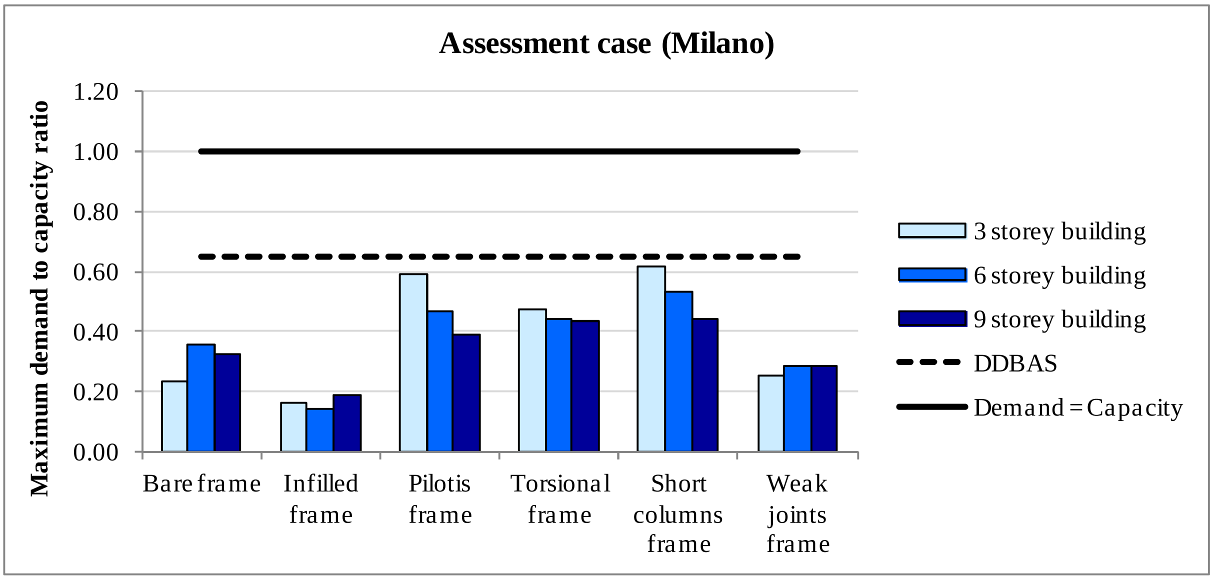

Figure 6. Demand-Capacity ratios for the 3, 6 and 9 storey buildings situated in Milano (assessment case): histograms represent the median of the ratios found with the non-linear analysis, for different structural configurations, while the two horizontal lines represent the limit ratio $\Delta_{dem}/\Delta_{cap} = 1$ (black line) and the simplified method result (dashed black line).

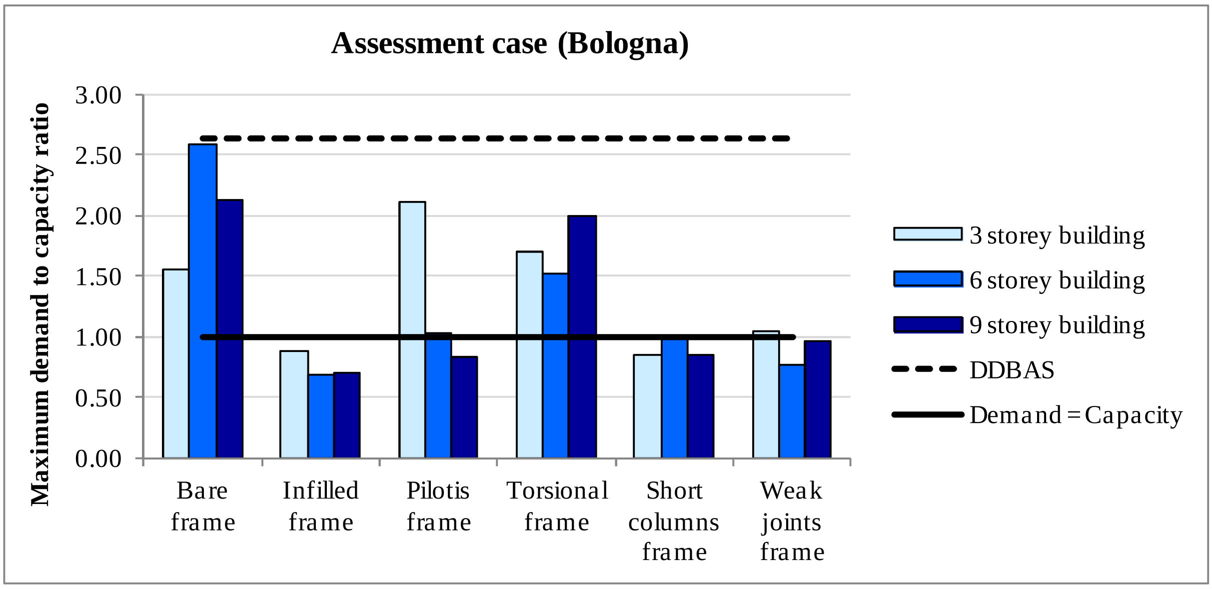

Figure 7. Demand-Capacity ratios for the 3, 6 and 9 storey buildings situated in Bologna (assessment case): histograms represent the median of the ratios found with the non-linear analysis, for different structural configurations, while the two horizontal lines represent the limit ratio $\Delta_{dem}/\Delta_{cap} = 1$ (black line) and the simplified method result (dashed black line).

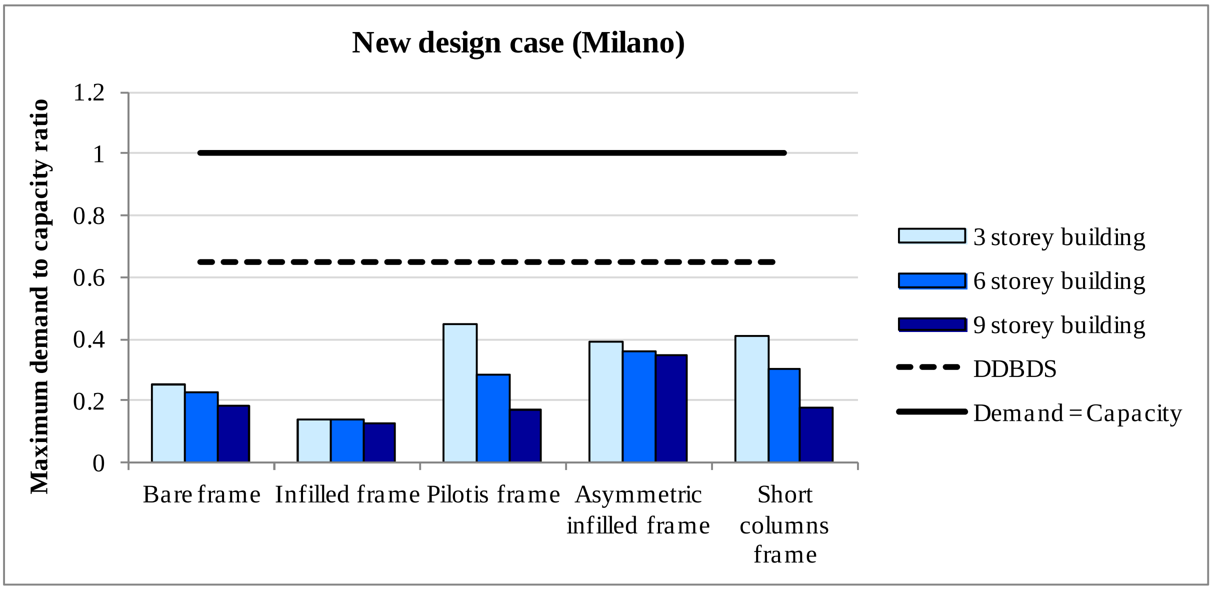

Figure 8. Demand-Capacity ratios for the 3, 6 and 9 storey buildings situated in Milano (new design case): histograms represent the median of the ratios found with the non-linear analysis, for different structural configurations, while the two horizontal lines represent the limit ratio $\Delta_{dem}/\Delta_{cap}$ = 1 (black line) and the simplified method result (dashed black line).

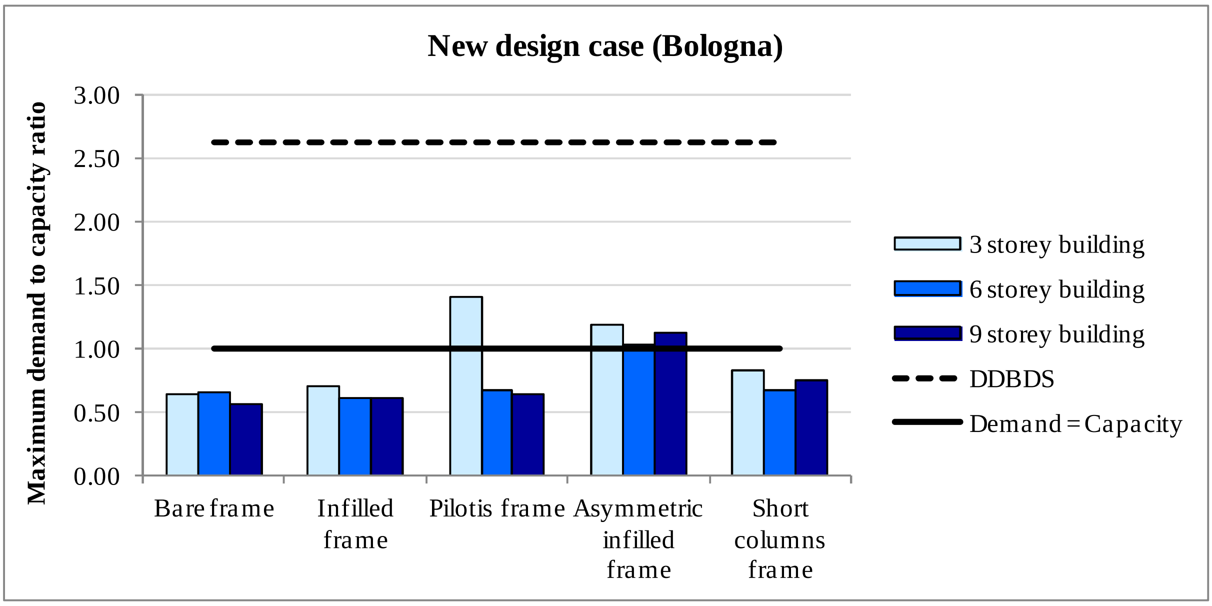

Figure 9. Demand-Capacity ratios for the 3, 6 and 9 storey buildings situated in Bologna (new design case): histograms represent the median of the ratios found with the non-linear analysis, for different structural configurations, while the two horizontal lines represent the limit ratio $\Delta_{dem}/\Delta_{cap}$ = 1 (black line) and the simplified method result (dashed black line).

CONCLUSIONS

In this work a simplified displacement-based seismic design/assessment methodology has been proposed for the purpose of rapid seismic verification of an RC frame building for both design and assessment. The basis of the proposed procedure, that builds on the work of Pinho *et al.* (2007), is to evaluate a building's displacement capacity using conservative approximations and simplified equations derived from DDBD (Priestley *et al.* 2007), and to compare the capacity with the maximum spectral displacement demand. In this way, no estimate is required of the building strength, or stiffness or period of vibration, making the procedure very quick. The approach could be used either for the assessment of an existing building or to check seismic performance of a new building during the design phase.

The aim of this research was to define the steps and the few equations of the simplified procedure, applying it to three buildings with the same plan dimensions but different height. A number of time history analyses have been run, in order to verify the method comparing the results of the analysis in terms of displacement/capacity ratio. In the non-linear analyses, different building configurations were considered, characterised by different arrangements of the masonry infills. The objective was to evaluate how the infills presence, not considered in the simplified procedure, can influence the response of the buildings. Moreover, different failure modes have been checked in the analysis.

The proposed procedure, to reach an appropriate level of simplicity that is its primary purpose, does not directly consider a number of issues, such as, irregularities in plan, P-delta effects, the presence of infills irregularities in height, shear problems caused by short columns. Some of these problems have been considered in the validation process of the procedure by non-linear analysis; however, the research carried out in this project is limited to a small number of buildings, critical mechanisms and infill configurations and does not allow an exhaustive validation of the procedure proposed, but only determines which aspects are more critical and need further investigation. In light of the results obtained, it could be stated that, despite the limited parameters considered in the simplified method, for both the assessment case and the new design case the procedure is typically conservative and could be used for all the configurations considered. The large conservative approximations especially in the demand estimate, at the basis of the procedure, allow it to balance the limited accuracy in the evaluation of all the mechanisms of failure. While efforts were made to consider particularly prone structural configurations in testing the method, it is recognised that a larger set of case study structures should be examined in the future, particularly cases in which P-delta, torsion and shear mechanisms might be more significant. On the other hand, the approach may also be too conservative in some cases and future research will therefore investigate the potential benefits of incorporating simplified strength calculations into the procedure. Overall, the main aspect that can be underlined from this research is the speed of the proposed approach which, despite its limits, can justify the development and the use of a simplified displacement-based method for regions of low to moderate seismicity.

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REFERENCES

Bertoldi SH, Decanini LD and Gavarini C (1993) "Telai tamponati soggetti ad azione sismica, un modello semplificato: confronto sperimentale e numerico" (in Italian), *atti del 6° convegno nazionale ANIDIS*, vol.2, pp.815-824, Perugia 13-15 Ottobre 1993

Carr A (2009) RUAUMOKO 3D inelastic three-dimensional dynamic analysis program: User manual, University of Canterbury – Department of Civil Engineering, Christchurch, NZ

Crowley H, Pinho R, Bommer JJ, Bird JF (2006) "Development of a Displacement-Based Method for Earthquake Loss Assessment", Research Report Rose 2006/01, ISBN: 88-7358-033-5

Galli M (2006) "Evaluation of the seismic response of existing RC frame buildings with masonry infills", Master Thesis, European School for Advanced Studies in Reduction of Seismic Risk (ROSE School), University of Pavia, Italy

Hak S (2010) "Seismic Behaviour of Clay Masonry Infills in Framed Structures" Master Thesis, European School for Advanced Studies in Reduction of Seismic Risk (ROSE School), University of Pavia, Italy

Hak S, Morandi P, Magenes G, Sullivan TJ (2012) "Damage Control for Clay Masonry Infills in the Design of RC Frame Structures" *Journal of Earthquake Engineering*, Vol.16, sup1, pp. 1-35

Moehle JP (1992) "Displacement Based Design of RC Structures Subjected to Earthquakes", *Earthquake Spectra*, August 1992, Vol. 8, No. 3, pp. 403-428

Montejo LA and Kowalsky MJ (2007) "CUMBIA – Set of codes for the analysis of reinforced concrete members" CFL Technical Rep. No. IS-07-01, Dept. of Civil, Construction, and Environmental Engineering, North Carolina State Univ., Raileigh, N. C.

NTC08 (2008) Decreto ministero delle infrastrutture 14 gennaio 2008, Norme tecniche per le costruzioni (in Italian), G.U. 4 febbraio 2008, n. 29, s.o.

Pampanin S, Magenes G and Carr AJ (2002) "Modelling of Shear Hinge Mechanism in Poorly Detailed RC Beam/Column Joints", *fib Symposium on Concrete Structures in Seismic Regions*, Athens, paper n. 171, 2002

Piazza A (2014) "Development of a simplified displacement-based procedure for the seismic design and assessment of RC frame structures", Master Thesis, European School for Advanced Studies in Reduction of Seismic Risk (ROSE School), University of Pavia, Italy

Pinho R, Crowley H, Faravelli M (2007) "Preliminary study on the identification of thresholds to displacement spectra for the design of new buildings", Project S5 for DPC, Deliverable 7, Task 2, 8 pp

Priestley MJN (1993) "Myths and Fallacies in Earthquake Engineering - Conflicts Between Design and Reality", Bull. NZNSEE, 26, 3, pp329-341

Priestley MJN, Calvi GM, Kowalsky MJ (2007) Displacement-Based Seismic Design of Structures, IUSS Press, Pavia, Italy

Sullivan TJ (2013) "Highlighting differences between force-based and displacement-based design solutions for RC frame structures" *Structural Engineering International*, 2/2013

Sullivan TJ (2013a) "Simplified Displacement-Based Seismic Design of Regular RC Frame Buildings", *Proceedings of the XV Convegno L'Ingegneria Sismica in Italia*, ANIDIS 2013, Padova, Italy

Regio Decreto (1939) Norme per la esecuzione delle opere in conglomerato cementizio semplice ed armato, (in Italian), 16/11/1939 n. 2229

Trowland M (2003) "Modelling the Shear Hinge in Beam Column Joints", Third Professional Year Project, Department of Civil Engineering, University of Canterbury, New Zealand, 2003