Improvement of a simplified method for the assessment of 3D R.C. frames

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SUMMARY
In this paper a simplified method, developed by Dolce et al. [2005], for the seismic assessment of tri-dimensional reinforced concrete frame structures will be revised and possibly improved. Strong-beam weak-column behaviour is at the basis of the method: an acceptable hypothesis for buildings in the Mediterranean area dating back to the '60s and '70s when most of the design was for gravity loads only. This procedure has been applied to three different structures which experienced strong shakings. The comparison between the estimated collapse accelerations and those experienced by the reference structures shown that the simplified method is over-conservative. It has been noticed that the modification of definition of the dynamic amplification coefficient, which is related to the first period of vibration of the structure, and of the ductility coefficient, which is linked to the nonlinear capacity of the load-bearing elements, could lead to a closer prediction.

Keywords: fast assessment procedure, reinforced concrete structures, collapse prediction

1. INTRODUCTION

Preparedness for an efficient response in case of natural disaster strongly hitting our society is rapidly acquiring importance and this trend is probably not going to change since year after year, event after event, the fragility of our socio-economic framework is sadly shown. Since 2001, the European Union started working for its resilience establishing the Community Mechanism for Civil Protection aiming to facilitate co-operation in civil protection assistance interventions in the event of major emergencies which may require urgent response actions.

In this framework, the European Commission funded a project for the Deployment of Rapid Highly-specialized Operative Unit for Structural Evaluation (DRHOUSE). This civil protection module exploits three components: (i) the Basic Seismic Assessment (BSA) module by the Italian Civil Protection Department; (ii) the Advanced Seismic Assessment (ASA) module by the European Centre for Training and Research in Seismic Engineering (Eucentre); (iii) the Short-Term Countermeasures (STC) module by the Italian Fire Brigades.

The development of the ASA module involved a number of preparatory actions to improve the effectiveness of the assessors teams and the accuracy of the delivered results. One of these action was the preparation of tools for the fast assessment of structures. Initially, the literature review allowed the identification of some assessment methods having an adequate balance between complexity and accuracy. Then a critic review of these methods was performed aiming to their improvement on the basis of the results of in-situ non-destructive tests, e.g. material characterization and dynamic identification.

The presented research work was developed in this context: the "Vulnerabilità Calcestruzzo Armato" (VC) method, originally developed by Dolce et al. [2005], was reviewed and its critical parameters were identified. Improvements have then been proposed increasing the complexity and the accuracy of the method with only minor changes to the time required for the application of the method.

2. THE ORIGINAL VC METHOD

The original method, namely "Vulnerabilità Calcestruzzo Armato" (VC) method developed by Dolce et al. [2005], is a simplified procedure by which it is possible to carry out fast assessment of reinforced concrete (R.C.) frame structures. This kind of procedure is very useful in post-earthquake scenarios since it could give a quantitative estimate of the damages that can be used for the definition of the intervention priority between structures which further intervention.

The procedure is based on assumptions which are reducing the data required for the assessment but, on the other hand, these are also limiting the kind of structures which could be analysed. The first hypothesis is that strong beam – weak column behaviour characterises the structure, which is quite reasonable for buildings designed for gravity loads only as it was in the '60s and '70s in the Mediterranean area. The latter assumption is that the displacement shape of the first mode is evaluated assuming a shear-type idealization.

The input data required for the assessment procedure are: the dimensions, the longitudinal and transversal steel reinforcement of each column, the inter-storey height, a stiffness coefficient depending on the relative stiffness of beams and columns and the axial load acting on each column. This last is just estimated depending on the relative pertinence area.

Briefly, the assessment procedure, schematically represented in Fig. 2.1, starts evaluating the columns lateral stiffness depending on the dimensions and boundary conditions of each element, the storey lateral stiffness is then equalled to the sum of these contributions. From the definition of the vertical load at each floor, the mass distribution is evaluated and the first vibration period and mode shape are calculated using the Rayleigh formula. From the mode shape, the lateral force distribution is estimated and compared to the lateral force capacity at each floor in order to define the lower collapse multiplier. The storey capacity is estimated as sum of the contributions of the single columns, which in turn are the minimum between the lateral forces inducing flexural and shear collapse. Once the collapse multiplier is defined, the maximum base shear can be evaluated as well as the collapse acceleration. Finally, assuming the shape of the acceleration spectrum and anchoring the spectral acceleration corresponding to the first period to the one just determined, the collapse peak ground acceleration (PGA) is evaluated.

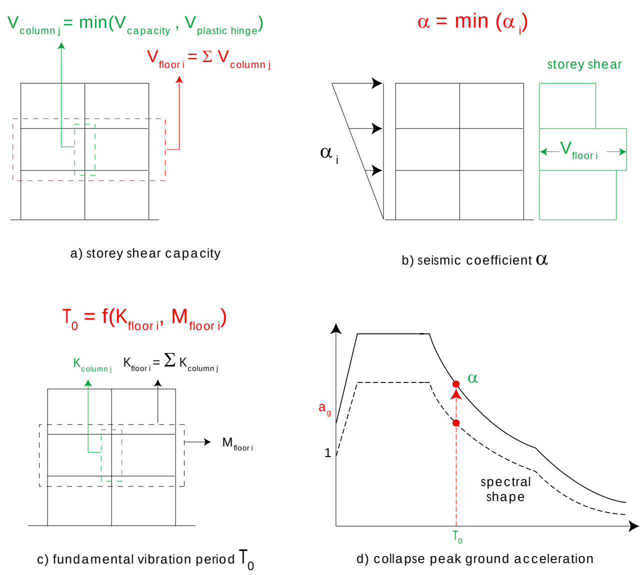

Figure 2.1. Scheme of the VC method

A number of correction factors are used within the assessment procedure to account for various effects and contributions able to produce modification of the seismic behaviour of the analysed structure (*e.g.* column ductility and infill panels contributions). One of these coefficients is the storey ductility factor, which in turn depends on the ductility factor α_D of the single columns evaluated as in Eqn. 2.1

$$\alpha_D = 3 \cdot \left(0.2 + \left(1 - \frac{\sigma_c}{f_c}\right)^{1.2} /1.11\right) \leq 3 \tag{2.1}$$

where σ_c is the average column compression and f_c is the medium compressive strength of concrete.

3. PROPOSED IMPROVEMENTS FOR THE METHOD

The first step towards the improvement of the existent method was a sensitivity analysis performed in order to understand which parameters mostly influence the results. The spectral coefficient (*i.e.* spectral amplification at the first vibration period) and the modal participation factor, the storey ductility coefficient and the dissipation factor were varied within reasonable ranges while assessing the three reference structures described in the following chapter 4. This sensitivity analysis, though very simple and limited, showed that the storey ductility factor and the spectral coefficient, and consequently the soil type assumed to characterise the construction site, were the ones mostly influencing the outcomes of the assessment procedure. The influence of the spectral amplification was particularly important for those structures having periods higher than 0.5 s, *i.e.* natural periods beyond the acceleration spectrum plateau.

Improvements on the definition of the spectral coefficient can be achieved using on-site testing techniques for the characterisation of construction site and structure. From the point of view of the application of this assessment method in the framework of the previously mentioned ASA module, the assessor teams can count on the intervention of the Eucentre Mobile Unit (MU). The technical staff of Eucentre, exploiting the instruments of such MU, can perform different tests aiding the structural assessment. The soil characteristics can be evaluated using the Multichannel Analysis of Surface Waves (MASW) method, while dynamic identification techniques based on ambient vibrations can be used for the characterisation of the structure.

On the other hand, a numerical approach is proposed to improve the estimation of the ductility coefficient. The new column ductility factor is equalled to the displacement ductility computed after the bilinearization of the actual force-displacement curve of each column. This last is evaluated by integration of the moment-curvature relation assuming a shear-type behavior. Worth to mention is the adopted bilinearization strategy (*i*) aiming to define a relation characterized by the same deformation energy, (*ii*) allowing post-yielding softening or hardening, depending on the actual behaviour of the column, and (*iii*) minimizing the function F reported in the following Eqn. 3.1 which represent the area between the effective curve and its bilinear approximation.

$$F(V_u; \delta_e) = \int_0^{\delta_u} \left|V(\delta) - V_{eq}(\delta)\right| d\delta \tag{3.1}$$

where V_u is the ultimate lateral capacity, δ_e and δ_u are respectively the equivalent yielding and ultimate displacement, $V(\delta)$ is the actual force-displacement relation and $V_{eq}(\delta)$ is the bilinear relation.

Crucial for the definition of the bilinear approximation is the ultimate column displacement: within the proposed procedure, this value corresponds to the failure of the element, identified by the achievement of the steel or concrete ultimate strain, or, when a softening branch is present, to a 20% reduction of the maximum lateral resistance. When the recognition of the element failure is based on the ultimate concrete strain, the value assumed by this parameter is very important as small variation can induce important deviation in the definition of the ultimate column displacement.

Two alternative procedure can be used for the evaluation of the concrete ultimate strain depending on the available characteristics of concrete and steel. The first procedure estimates the ultimate concrete strain adopting the Mander model [1988] for the concrete and an elasto-plastic model for the steel. The concrete maximum strain could be then calculated according to Priestley et al. [2007] using Eqn. 3.2:

$$\varepsilon_{cu} = 1.4\left(0.004 + 1.4 \cdot \rho_s \cdot \varepsilon_{su} \cdot f_{yh}/f'_{cc}\right) \tag{3.2}$$

where f'_{cc} is the confined compressive strength of the concrete (according to the adopted Mander model), f_{yh} is the steel yielding stress, ρ_s is the percentage of area of transverse steel and ε_{su} is the ultimate deformation of the transverse steel.

The second procedure for the evaluation of the maximum concrete strain is based on an energy approach originally proposed by Mander et al. [1988]. In this case, the ultimate strain is estimated considering the effective concrete stress-strain behavior and the strain energy capacity of the transverse confining reinforcement U_{sh} estimated using Eqn. 3.3

$$U_{sh} = U_{cc} + U_{sc} - U_{co} \tag{3.3}$$

where U_{cc} and U_{co} are the strain energy of the confined and unconfined concrete and U_{sc} is the additional energy to maintain yield in the longitudinal compressed steel. Although this method leads to more accurate results, its applicability strongly depend on the available data and it can rarely be used during post-earthquake emergency intervention as the effective concrete stress-strain behavior must be obtained from laboratory compression tests.

Finally, for sake of completeness, it must be mentioned that, since different columns at the same floor could have different displacement capacity, their ductility should be evaluated with respect to the ultimate storey displacement. This last is assumed to be equal to the minimum of the displacement capacity of the single columns, which means to consider that the storey ultimate limit state is reach when one column collapses. Clearly, if any of the columns is still in the linear range when the storey failure is reached, its ductility must be set equal to 1, as well as in the case of shear failure.

It must be underlined that in case dynamic identification is not performed, the estimation of the eigen-quatities can be refined using the column equivalent linear stiffness determined through bilinearizaton of the force-displacement relations. This generally improves the representation of the cracked structural behaviour adopted by the original VC method.

4. CASE STUDIES

The original and the improved procedures have been applied to three different buildings which experienced strong shaking in order to evaluate the effectiveness of the proposed modification. The first analysed building is a 1:2 scaled structure built and tested at Eucentre, the second and the third are two different structures of the L'Aquila Hospital (Italy). For all the considered case studies the characteristics of the concrete were determined through laboratory testing allowing the adoption of the energy approach previously discussed. On the other hand, the steel characteristics were determined through laboratory testing for the first case study, while they were derived considering the age of the buildings and the corresponding design codes for the last two cases.

In the following paragraphs, a brief description of the case studies is reported and the results obtained using both the original and the improved assessment method are compared and discussed. Furthermore, the tables 4.1, 4.2 and 4.3 summarise the results of the procedures showing separately the results enhancement corresponding to the adoption of the proposed improvement: (i) new ductility factor; (ii) vibration period estimated using the equivalent linear stiffness of the columns obtained from the bilinearization of the actual force-displacement curves; (iii) adoption of the vibration period estimated using dynamic identification techniques.

4.1. Building #1

The first considered case study is a three storey R.C. frame structure, 1:2 scaled and irregular in plan. It was built as part of an experimental campaign the details of which can be found in Pavese et al. [2009]. The structure has the typical deficiencies of the Italian R.C. building stock of the ‘60s such as strong beam – weak column behavior and columns and joints with inadequate confinement.
Fig. 4.1 shows the building after its positioning on the Eucentre TREES Lab shake table (left side) and the plan view of the specimen (right side). The plane view highlights the role of column 2: a very stiff structural element which is the main source of the torsional unbalanced behavior characterizing this structure.

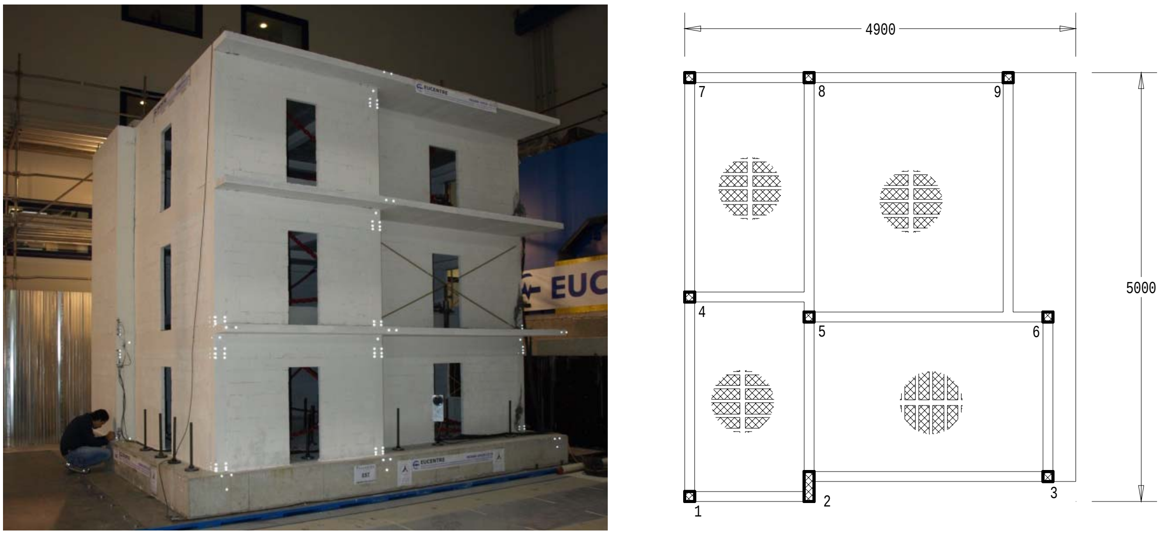

Figure 4.1. Global view (left side) and plan view (right side) of the specimen

Despite its deficiencies, the specimen was able to sustain multiple shaking up to a peak acceleration equal to 0.54 g. During the last dynamic test, most of the joints were seriously damaged and the infill panels at the 1st and 2nd floor collapsed.

The following Table 4.1 summarises the results of the original and improved methods and shows the values of some of the parameters calculated during the assessment procedure. The experimental collapse acceleration was strongly underestimated by the original VC method, while the proposed modifications led to improved results. It is interesting to note that the modification to the spectral and the ductility coefficients gave almost the same contribution towards the enhancement of the result. Furthermore, it is interesting to note that a significant improvement was reached without any additional information about the structure: the error on the estimation of the collapse PGA was reduced computing the columns force-displacement bilinearization, the corresponding ductility coefficient and updating the vibration period. The best result was achieved using the natural period estimated by the dynamic identification, that still is a reasonable test to be performed on strategic/critical structures even in an emergency context. Looking at the results in terms of estimated vibration period, it is interesting to note that the proposed procedure based on the equivalent secant stiffness of the single columns improves the estimation, although it is still underestimating the correct value. This was somehow expected in the sense that a pure numerical estimation of the vibration period of a damaged structure is surely not an easy task to achieve.

Finally, it should be noted that, using the original VC method, the infill panel contribution lowered the estimated collapse acceleration, although their effect should in this case be opposite. This decrease is due to the reduction of the ductility factor, adopted by the original VC method, the effect of which is only partially counteracted by the increase of the maximum storey-shear given by the strength of the panels.

Table 4.1. Case study 1: summary of the results of the original and improved procedures

	1st Natural Period [s]	Spectral Coefficient	Ductility Coefficient	Collapse PGA [g]
Original VC method	0.26	2.50	2.41	0.220
VC method + new ductility coefficient	0.26	2.5	3.61	0.320
VC method + new ductility coefficient + period from the equivalent curve	0.34	1.72	3.61	0.467
VC method + new ductility coefficient + period from identification	0.50	1.62	3.61	0.495

4.2. Building #2

The considered case study is a portion, separated by a seismic joint, of one of the oldest buildings of the L'Aquila Hospital (Italy) that suffered severe damages during the 2009 L'Aquila earthquake. The plan view of the structure is shown in Fig. 4.2, further details can be found in Casarotti et al. [2009].

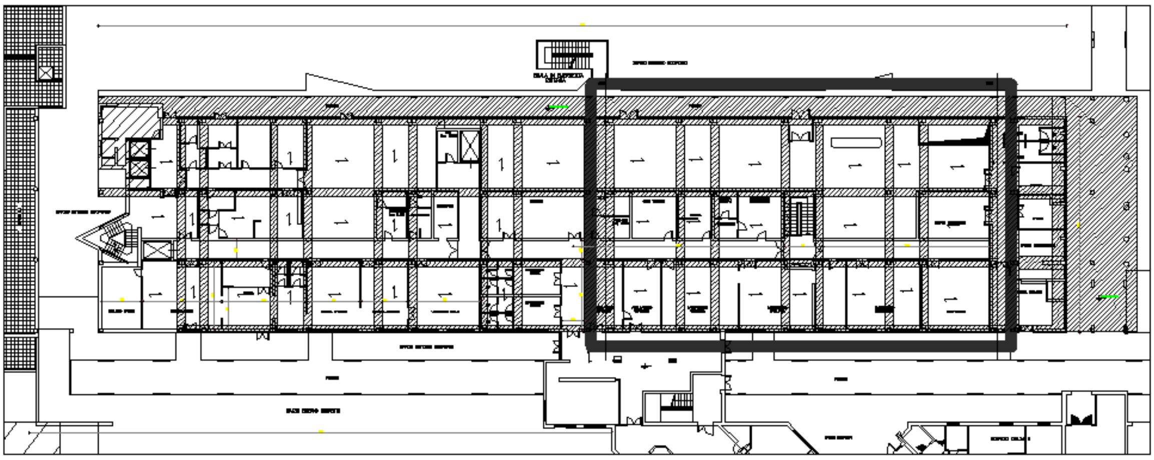

Figure 4.2. Plan view of the analysed structure

The concrete compressive strength, initially characterized with on-site SONREB tests, has been estimated to be equal to 30 MPa. Since the structure was designed and built at the end of the '70s, and considering that ribbed bars were visible through the main cracks of the columns, the steel type was assumed to be FeB38K and its characteristics were derived by the Italian design code in force during the end of the '70s [D.M. 16/06/1976].

The following Table 4.2 shows the results of the methods: the two principal directions were considered separately, X and Y are respectively the horizontal and vertical direction with respect to the previous Fig. 4.2. The collapse PGA estimated by the original VC method is significantly lower than the one experienced by the structure which was about 0.52 g (peak acceleration value recorded by a monitoring station in the Hospital vicinity). Also in this case, the proposed modification led to the improvement of the results although a possible over-estimation of the collapse PGA is present when using the newly estimated vibration period. Unfortunately, in this case the dynamic identification tests were not performed.

Table 4.2. Case study 2: summary of the results of the original and improved procedures

	Considered Direction	1st Natural Period	Spectral Coefficient	Ductility Coefficient	Collapse PGA [g]
Original VC method	X	0.372	2.37	1.90	0.253
Original VC method	Y	0.527	1.65	1.86	0.339
VC method + new ductility coefficient	X	0.372	2.37	2.43	0.325
VC method + new ductility coefficient	Y	0.527	1.65	2.34	0.430
VC method + new ductility coefficient + period from the equivalent curve	X	0.923	1.35	2.43	0.569
VC method + new ductility coefficient + period from the equivalent curve	Y	1.016	1.23	2.34	0.575

4.3. Building #3

As in the previous case study, the analysed structure is a portion one of the buildings of the L'Aquila Hospital (Italy), the plan view of which is shown in the bolt rectangle in Fig. 4.3. This building dates back to the end of the '70s but in this case plain rebars have been used, hence the steel type has been assumed to be FeB32K. The concrete compressive strength was estimated to be 30 MPa through on-site SONREB tests, value confirmed by the laboratory tests on the concrete core drills.

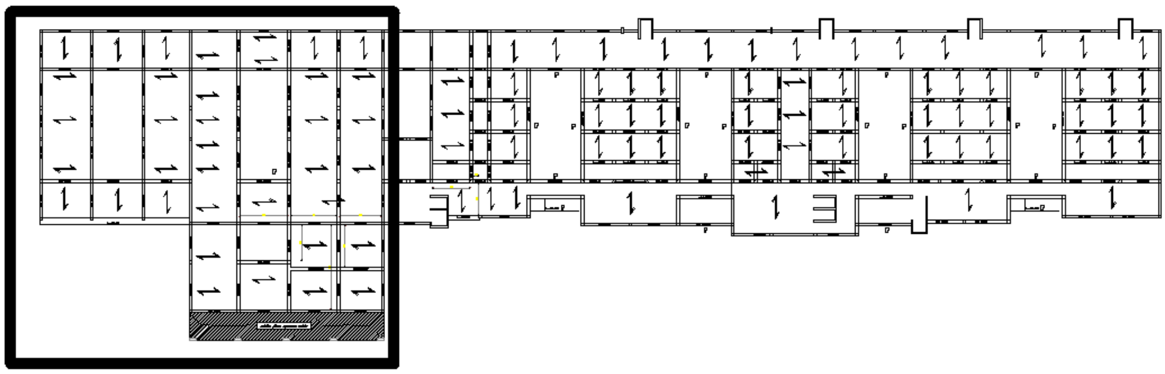

Figure 4.3. Plan view of the analysed structure

Table 4.3 summarises the results of the original and of the modified VC method from which is evident that the outcomes of the latter are closer to the maximum recorded PGA (about 0.52 g at the closest recording station). It is interesting to note that while the original V.C method estimates the same ductility coefficient for the two considered directions (X and Y direction are respectively the horizontal and the vertical direction in Fig. 4.3), the modified procedure leads to different coefficients as they do not depend only by the average compression on the elements. Also in this case study, the modified procedure leads to a possible slight over-estimate of the collapse PGA.

Also in this case, the dynamic identification test was not performed, hence it is not possible to comment about comparison between the vibration period determined by the original and the improved procedure. Once more, it has to be underlined that the application of the improved method did not implied an increase of the time required by the assessment since all the procedures for the estimation and bilinearisation of the actual force-displacement curve of the single columns were implemented in the adopted software solution and are not requiring any additional effort by the user.

Table 4.3. Case study 3: summary of the results of the original and improved procedures

	Considered Direction	1st Natural Period	Spectral Coefficient	Ductility Coefficient	Collapse PGA [g]
Original VC method	X	0.567	2.20	2.51	0.374
Original VC method	Y	0.659	1.89	2.51	0.411
VC method + new ductility coefficient	X	0.567	2.20	3.49	0.551
VC method + new ductility coefficient	Y	0.659	1.89	3.14	0.513
VC method + new ductility coefficient + period from the equivalent curve	X	0.608	2.05	3.49	0.589
VC method + new ductility coefficient + period from the equivalent curve	Y	0.673	1.86	3.14	0.524

5. CONCLUSION

The assessment of the seismic vulnerability of existing buildings is one of the main topics in the Mediterranean countries, particularly in Italy where the great majority of structures have not been design for seismic loads. Fast assessment methods are therefore necessary, in post-earthquake scenario, in order to create priorities between buildings which need retrofit interventions or to evaluated the structural safety of damaged structures.

One of these procedure is the VC Method by which is possible to analyse reinforced concrete frame structures. One of the results given by the method is the collapse peak ground acceleration of the building. Comparing it to the one prescribed in the seismic hazard code for that area, it is possible to check if the assessed structure is safe or calls for seismic upgrade or retrofit.

The procedure has been applied to three different structures and it has been noticed that the results were over-conservative. For this reason, it was decided to try to improve the performances of the method using the results of non-destructive tests and using newly developed automated, hence time-effective, capacity assessment procedure to be applied to the structural elements composing the structure. First step of the presented research was a sensitivity analyses to identify those parameters having the strongest influence on the results: (*i*) the ductility coefficient and (*ii*) the spectral amplification at the first vibration period. The first, function of the capacity of the structural elements, was improved using the results of non-destructive tests, if available, and adopting an automated procedure to evaluate the actual force-displacement bilinear curve and the corresponding displacement ductility. The latter was refined replacing the vibration period roughly estimated by the original method with a new one based on dynamic identification, when available, or corresponding to the previously assessed equivalent linear stiffness of the structural elements.

The modified procedure, applied to the three considered case studies, was able to predict a collapse acceleration closer to the ones actually experienced by the structures. The VC method with the proposed modification is currently under implementation to become a fast-assessment tool to be used during post-earthquake emergency, particularly within the EU funded DRHOUSE project.

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