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A Simplified Model for Seismic Safety Assessment of Reinforced Concrete Buildings: Framework and Application to a 3-Storey Plan-Irregular Moment Resisting Frame

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Abstract
Seismic safety assessments at the large-scale are of paramount importance for densely urbanised areas with non-negligible seismic hazard. The topic is gaining increasing interest, particularly in nations with a longstanding-built environment. Methods based on non-linear history analyses of simplified building models are increasingly relevant and popular. This paper proposes a simplified model for moment resisting frame reinforced concrete buildings, and shows its efficiency in performing non-linear history analyses on a case study building. The proposed model has three degrees of freedom per storey, two translational and one rotational around the vertical axis. The building stiffness matrix and forces are derived from the column elements. Each column is modelled with two force-inter storey drift springs, one in each horizontal direction. These springs reflect the behaviour of the columns and account also for the flexibility and strength of the connected beams through calibration of their backbone curves. A genetic algorithm is used for the calibration of the parameters for each spring. Despite the approach simplicity, the proposed modelling scheme accounts for the effects associated with irregularities both in plan and in elevation. For this reason, it represents a major improvement with respect to similar approaches for large-scale vulnerability assessments, particularly for torsionally-flexible buildings. The proposed modelling approach is applied to a three-storey plan-irregular moment resisting frame building. A refined model is used as benchmark to show how the proposed approach can reproduce the main non-linear characteristics of the case study building subjected to a suite of ground motions.

Keywords: Reinforced Concrete Moment Resisting Frame; Seismic Assessment; Regional Scale; Simplified Spring Model; Torsional Behaviour; Irregular Buildings
1 INTRODUCTION

Approximately 55% of the world population currently lives in urban areas, a percentage that will increase up to 75% by 2050 (UN 2018). Densely populated areas may experience extensive damage and high number of casualties in the occurrence of disasters such as major earthquakes. To provide guidance to authorities on the best interventions for risk mitigation, the European Union promotes a regular update of risk scenarios for all member states (European Commission 2015, ICDP 2018). In this framework, large-scale seismic assessments are of paramount importance to estimate expected damage and proactively plan mitigation strategies.

Given the high number of buildings to be assessed, indirect methods (De Risi et al. 2018, Polese et al. 2019) have been preferred to direct methods based on refined structural analyses using Finite Element (FE) modelling, characterized by high computational demand. Indirect methods are based on the definition of deterministic or probabilistic earthquake scenarios where hazard, vulnerability and exposure models are predefined and eventually convoluted together. Buildings are classified into typological classes (e.g., Grüntal 1998, So and Spence 2013, Jaiswal and Wald 2008) and fragility or vulnerability models are derived using either empirical (e.g., Jaiswal et al. 2011, Martins and Silva 2020, Basaglia et al. 2020) or analytical, numerical, and mechanical approaches (e.g., Gentile et al. 2019, Cardone and Flora 2017, Rossetto et al. 2016, D’Ayala 2005). Indirect methods provide acceptable results for regional-scale assessments, though they involve rough approximations. The reliability of indirect approaches is strictly dependent upon the amount and quality of available data.

Direct methods require instead the modelling of the building. Different approaches may be adopted to reduce the FE computational demand, and to make them feasible to deal with a high number of buildings or analyses on the same building. A possible method consists in the description of large building portfolios with a suite of “archetype” models (Cross et al. 2020),
representing the main geometrical and structural characteristics, and adopting less refined approaches to analyse the seismic response (Gentile and Galasso 2021). Results obtained on selected archetypes may also be extended to include building-to-building variability using surrogate models or metamodels (i.e., models of models) (Gentile and Galasso 2020).

Alternatively, simplified mechanical models of buildings can be considered and the Engineering Demand Parameters (EDPs) can be directly derived by performing response-history analyses on the simplified model. In existing simplified mechanical models, buildings are modelled as either Single-Degree-Of-Freedom (SDOF) systems (e.g. De Luca et al. 2014, Borzi et al. 2020, Nagato and Kawase 2004) or Multiple-Degree-Of-Freedom (MDOF) systems. SDOF approaches provide a reasonable estimation of the global building behaviour (in terms of strength and ductility) with a significantly reduced computational effort. However, they have clear limitations, such as the impossibility of representing the concentration of damage at different storeys, the absence of higher modes contributions and the unfeasibility for buildings with torsional behaviour (e.g., Vamvatsikos and Cornell 2005, Dolšek and Fajfar 2007, Tothong and Cornell 2008, Lu et al. 2014, Suzuki and Iervolino 2019, among others). MDOF-based approaches overcome most of these issues (Khaloo and Khosravi 2013, Soleimani et al. 2019, Xiong et al. 2016) and recent studies have shown that MDOF "fish-bone" or "stick-like" models may still provide good computational efficiency, while achieving higher levels of accuracy. Fish-bone models (Luco et al. 2003, Khaloo and Khosravi 2013) represent each storey with a single column plus beams at both sides. Columns are modelled through plastic hinges with lumped masses at the floor level, and beams are condensed into a single rotational spring (at the beam-column joint) and a vertical displacement restraint. Fish-bone models have been extensively investigated (Jamšek and Dolšek 2020) to represent non-conforming Reinforced Concrete (RC) frames that may not follow the strong column-weak beam design approach (EC8 2003) and/or may have major irregularities in elevation. For RC
buildings, Lu et al. (2018), Xiong et al. (2016) and d'Aragona et al. (2020) MDOF stick-like models provide reliable assessments of the structural demand (Figure 1). MDOF stick-like models proposed to date (Figure 1b) represent each storey with a concentrated mass and a non-linear translational spring, which simulates the storey shear-inter storey drift response in the direction of the analysis (Figure 1b), two orthogonal directions. Each floor has two uncoupled translational degrees of freedom (DOFs). The springs backbone curves are calibrated using simulated design procedures (Xiong et al. 2017) or through a multi-objective Genetic Algorithm (GA) that employs the results of non-linear cyclic pushover analyses (PO) on refined non-linear models (d'Aragona et al. 2020). These models present several merits: the computational cost for a single building is highly reduced, thus allowing the simultaneous analysis of extensive building portfolios; they can include the effects of infills and beam-column joints with satisfactory approximation with respect to refined models (e.g. Furtado et al. 2018; Giannakouras and Ziris 2019) following the procedure proposed by d’Aragona et al. (2020); they can be coupled with soil models to include soil-structure interaction effects (Lu et al. 2018). The main drawbacks of these models derive from the assumption of a shear-type behaviour for the buildings and the uncoupled response in the global X and Y (i.e., horizontal) directions. These drawbacks make these models not reliable for simulating the response of irregular and torsionally-flexible buildings, since their behaviour is significantly different from that of regular buildings (e.g., DeBock et al. 2014). As for the soil-structure interaction implemented by Lu et al. (2018), the soil and the buildings are modelled in separate software, and a coupling scheme is needed for simulating the interaction between the soil volume and the stick-like models (Figure 1b).

This paper presents a simplified approach for modelling RC Moment Resisting Frame (MRF) buildings, based on the behaviour of columns, able to account for torsion and beams flexibility (Figure 1c). The current calibration approach of the model is suitable specifically for
buildings which follow the fundamental failure mode as identified from a static nonlinear analysis. The behaviour of each column is defined by two shear-inter storey drift (V-ID) springs in the two principal directions of the cross-section (Figure 1c). The V-ID backbone curves associated to the springs are derived following an approach similar to the one proposed by Xiong et al. (2017) but with some major differences. More specifically, curves are derived for the single column rather than for the entire storey and are based on approximate mechanical response curves rather than on a simulated design procedure. The V-ID backbone curves are calibrated via a GA approach (similarly to d’Aragona et al. 2020) based on the non-linear static analysis of 2D frames, modelled in a refined manner. After assembly of all structural elements, making the assumption of rigid diaphragm, the resulting simplified model has three DOFs per floor (as indicated with “global DOFs” in Figure 1c). The increased complexity of the proposed model with respect to other approaches (e.g., Soleimani et al. 2019, Jamšek and Dolšek 2020) is considered a reasonable trade-off to account for the torsional behaviour, similarly to what is proposed for the UBC-SAWS model (Rudolf et al. 1998, Zhang et al. 2020). The proposed building model is part of an ongoing broader work whose objective is to develop a simplified approach that combines the buildings with a representative soil volume in the same non-linear model. This approach is thought to carry out non-linear dynamic analyses of large building portfolios, taking into account the possible soil-structure interaction and structure-soil-structure interaction effects. The combined building-soil model is schematically shown in Figure 1d. Additional details on the work may be found in Blasone (2020).

After presenting the proposed simplified model, the paper focuses on its efficiency in performing intense non-linear analysis computations on a case study building.

In the following, Section 2 presents the assumptions of the simplified building model, its derivation, and the proposed calibration procedure. Section 3 presents a validation example, based on the application of the simplified model to a 3D plan-irregular building and includes
non-linear static and dynamic analyses, and the comparison with the results of a benchmark model. Finally, Section 4 outlines the main conclusions and points to further developments.

![Figure 1 - Schematic representation of (a) regular RC MRF building to be modelled; (b) stick-like approach with independent models and storey springs in global X (blue) and Y (red) directions, respectively, showing the global DOFs accounted for by the models; c) proposed approach with highlighted the single column model with two independent column springs in global X (blue) and Y (red), showing the global DOFs accounted for by the model (after the application of the rigid diaphragm constraint).](image)

2 METHODOLOGY

The proposed model is implemented in OpenSees (McKenna et al. 2010) using the pre- and post-processor STKO (Petracca et al. 2017).

2.1 Overview of the simplified column-based model

The building is modelled in STKO as a column-based 3D MDOF system capable of capturing the translational and torsional behaviour of both regular and irregular RC Moment Resisting Frames (MRFs) as shown in Figure 2 for an irregular building. More specifically, each column is modelled using two uncoupled V-ID springs in the global X and Y directions (Figure 2c and 2d). The column model is discussed in detail in Section 2.2.

At the floor level, nodes are linked through a rigidDiaphragm constraint in the plane orthogonal to the global Z axis. To define the rigidDiaphragm, an additional node (or master node) is defined in the centre of mass of each storey (Figure 2b). Therefore, all nodes of the
same floor are "slaved" to the $U_x, U_y$ and $R_z$ displacements of the centre of mass, while the other DOFs ($U_z, R_x$ and $R_y$) are restrained, and rigid diaphragm behaviour is achieved.

Thanks to the rigid diaphragm assumption, the assembled model has three DOFs per floor, two horizontal displacements and the rotation around the vertical axis, as schematically shown in Figure 1c where the red dots indicate the floor master nodes with their three DOFs. The application of the diaphragm constraints and the consequent DOFs’ reduction is performed by OpenSees. The global stiffness matrix and the nodal forces are assembled from the single columns through transformation matrices derived from the rigid diaphragm assumption. Similarly, the three master node masses are assembled from the nodal masses, computed using tributary areas. The global stiffness matrix (a 6x6 matrix in the example of Figure 2b) couples the translational and rotational DOFs, making the model able to represent torsion. Moreover, the DOFs of the simplified model are those of a shear-type building, in which the nodes have no rotations around the X and Y axes. However, the simplified model can represent (in an approximate way) the behaviour of the non shear-type 3D RC MRF through calibration of the responses of the column springs (V-ID in Figure 2c and 2d). Details on the calibration procedure are provided in the next sub-sections.
2.2. Column Model

The column model is central to the proposed simplified methodology. The behaviour of each column is modelled with two independent springs in the two horizontal directions (global $X$ and $Y$ directions in Figure 2). Each column is modelled as a zeroLength element: the constitutive laws of the springs are discussed in detail in Section 2.3. No torsional stiffness is assigned to the columns. Even though in STKO the columns are implemented with a finite length, the corresponding zeroLength elements generated in OpenSees have no length and connect the DOFs of the two end-nodes of the columns.

The model does not account for the axial load-biaxial moment (PMM) interaction, similarly to other widely non-linear analysis approaches (among others, Pinto and Franchin 2016, Bousias et al. 2002, Di Ludovico et al. 2013). The PMM interaction becomes important for tall buildings (Xiong 2016) but is neglected in the proposed approach as it is meant for low- and medium-rise buildings, with one example of the latter category being investigated in the current paper.
2.3. Spring Constitutive Law

Each spring of Figure 2c and 2d is defined by an appropriate V-ID backbone curve that represents the column behaviour in the considered direction. The curve is defined following a two-steps procedure: (1) a trilinear parametric analytical V-ID curve is determined, based on the shear-type behaviour; (2) the parameters are adjusted in order to account for the actual column end restraints and more specifically the flexibility of the beams connected to the beam-column node, as previously mentioned in Section 2.1. In step two, P-Delta effects are currently not included. However, calibration of the parameters of the V-ID springs can account for P-Delta effects when they are expected to be significant for the structural response (e.g., buildings with higher number of storeys).

![Diagram](image)

**Figure 3** – (a) Column model with two end rotational springs; (b) moment-rotation (M-θ) curve by Ibarra et al. (2005); (c) resulting V-ID curve for the column under the shear-type assumption (step 1).

In step 1, each column is conceptually schematised as two zero-length moment-rotation (M-θ) hinges (Figure 3a) connected by an elastic finite-length element. The column is assumed to have the end rotations restrained (Figure 3a). Each end spring is assigned a M-θ behaviour according to the model by Ibarra et al. (2005) (Figure 3b). The resulting V-ID curve (Figure 3c) is directly derived from the M-θ curves. Additional parameters (see Equations 1 and 2) are introduced to allow modifications of the shear-type values.
The parameters are calibrated in step 2, in order to account for the actual constraints on the column end nodes. This adjustment can be made in different ways: for example, the spring response can be modified based on the comparison with a refined model or using engineering judgement. In this paper, a Genetic Algorithm (GA) is used for calibration. In the following, Sections 2.3.1 and 2.3.2 describe step 1 and Section 2.3.3 describes step 2 of the proposed procedure.

2.3.1 Step 1.a: End section moment-rotation curves

In the first part of step 1, the model proposed by Ibarra et al. (2005) is used to estimate the values of the M-θ curve of the column end rotational springs. The model is based on the column properties in terms of geometry, material and axial load due to gravity loads. The main parameters are:

- cracked, ultimate and collapse moments, $M_{40}, M_U$, and $M_C$;
- cracked stiffness, $K_{40}$;
- cracked, ultimate and collapse rotations, $\theta_{40}, \theta_U$ and $\theta_C$.

The parameter names are taken from Ibarra et al. (2005), unless for the ultimate and collapse values, which refer to the capping and residual parameters of the Ibarra formulation (Ibarra et al. (2005)), respectively.

To determine the aforementioned parameters, the yield values ($M_Y$ and $\theta_Y$) need to be computed as well. In this work, the yield and ultimate moments are derived from section equilibrium, using a parabola-rectangle stress-strain relationship for concrete and a bilinear elastic-perfectly plastic stress-strain relationship for steel (section 3.1.7 of Eurocode 2, EC2 (2005)). The concrete section is divided into a given number of layers (e.g. fifty), and a trapezoidal integration scheme is used. The yield moment, $M_Y$, is taken at the point where the first steel fibre reaches the yield strain in tension or the point where the first concrete fibre reaches a compression strain of 0.002, whichever happens first (Priestley et al. 2007). The
ultimate moment, $M_U$, is taken at the point where the first steel fibre reaches a tension strain of 0.015 or the point where the first concrete fibre reaches a compression strain of 0.004, whichever happens first (Priestley et al. 2007, ACI-318-11). This value is similar to 0.0035 used by Eurocode 2 (EC8 2005) for unconfined concrete. Since the proposed model is intended for safety assessment of existing buildings, thus non-conforming and/or with poor seismic details, confinement effects are neglected. In case of exhaustive knowledge of the building under assessment (e.g., KL 3 in Eurocode 8 part 3) and adequate seismic details, an explicit value of the ultimate strain of the confined concrete can be used. The reference value for the cracked stiffness is computed as the secant stiffness at 40% of the yield moment, $K_{40}$, as suggested by Haselton and Deierlein (2007) and Haselton et al. (2008). The values $\theta_{40}$ and $M_{40}$ are estimated according to Lignos and Krawinkler (2012), while $\theta_Y$, $\theta_U$ and $M_U$ are taken from Panagiotakos and Fardis (2001). Finally, $\theta_C$ and $M_C$ (the latter equal to 0 by definition) are estimated following Haselton and Deierlein (2007) and Haselton et al. (2008).

The Ibarra et al. (2005) model considers only two changes in the curve slope, the first at $M_{40}$ and the second at $M_U$. The M-$\theta$ curve is trilinear and only three points (plus the origin) define it. The yield point, $M_Y$, is also included in Figure 3a for completeness, as it is needed to compute $M_{40}$.

### 2.3.2 Step 1.b: V-ID parameters-dependent curves

The V-ID relationship depends not only on the section M-$\theta$ characteristics but also on the boundary conditions, that is the relative stiffness of the beams and columns at the two end joints. A set of initial equations are easily derived from converting the M-$\theta$ curves into V-ID curves (Figure 3c) for the shear-type case with double-curvature behaviour (Figure 3a). The equations are then modified through a set of five parameters ($\alpha$, $\beta$, $\gamma$, $\mu$, $\kappa$) that are used to adjust the column values to account for the actual non-shear-type configuration.
The expressions in Equation 1 are used to derive column shear forces from the end moments, while the expressions in Equation 2 are used to derive inter-storey drifts from the end springs chord-rotations. In all equations, \( L \) indicates the column height.

\[
V_{40} = 2 \frac{M_{40}}{L} \cdot \mu \\
V_Y = 2 \frac{M_Y}{L} \cdot \mu \\
V_U = 2 \frac{M_U}{L} \cdot \kappa \\
V_C = 0
\]

\[
ID_{40} = \theta_{40} \cdot \frac{L}{\alpha} \\
ID_Y = \theta_{40} \cdot \frac{L}{\alpha} + (\theta_Y - \theta_{40}) \cdot \frac{L}{\beta} \\
ID_U = \theta_{40} \cdot \frac{L}{\alpha} + (\theta_U - \theta_{40}) \cdot \frac{L}{\beta} \\
ID_C = \theta_C \cdot \frac{L}{\gamma}
\]

Two parameters \((\mu, \kappa)\) are needed for the shear values and three \((\alpha, \beta, \gamma)\) for the inter-storey drift values. For the shear-type configuration, all parameters are equal to one. This is also the only case where the conversion between the M-\( \theta \) and V-ID curves is exact. In all other cases, the five parameters need to be calibrated. In the case of asymmetric reinforcement layout, the calibration procedure is repeated for both loading directions. The accuracy of the results strictly depends on the calibration of the parameters.

**2.3.3 Step 2: Genetic algorithm for parameters calibration**

The parameters calibration is crucial for the proposed approach, as parameters are intended to represent the additional column flexibility due to the end-nodes rotational flexibility. The factors which mostly affect the calibration of the parameters are:

- the ratio between the stiffness of the column and the stiffness of the connected beams;
- the axial load acting on the column;
- the storey at which the column is located;
● the column position (external or internal);
● the characteristics (e.g., cross-section, axial load) of the columns above and/or below the one considered, if any.

For calibration purposes it is assumed that, in the 3D structure, the behaviour of the columns in a given direction is not influenced by the characteristics of the building in the orthogonal direction. This allows to perform the calibration on 2D sub-frames that are easier to handle and faster to model. Moreover, exploiting building characteristics and symmetries, the number of 2D frames necessary for the calibration may be smaller than the total number of 2D frames in the building (see Figure 5a in the case study example). For instance, if two internal frames are identical, the calibration is performed on one only, and the springs of both the frames are derived from the latter.

The calibration is performed through a GA procedure that employs the results of a non-linear pushover analysis (PO) on the selected 2D frames. The flowchart in Figure 4 presents the main steps of the calibration procedure. More specifically, once a 2D frame is selected, all the curves representing the columns of the 2D frame in its direction are calibrated. The main steps of the calibration procedure for each 2D frame are:

i. Build a refined model of the 2D frame and perform non-linear PO in the plane of the frame.

ii. From the PO results, derive the reference V-ID curves. The ID value is taken as the difference between the displacement of the storeys above and below each column and $V$ is determined from the software output (in the Gauss point of the $hingedBeam$ element in STKO).

iii. Calibrate the parameters $\alpha, \beta, \gamma, \mu$, and $\kappa$ in Equations 1 and 2 through the GA.

Steps ii. and iii. are repeated for all the springs representing the behaviour of the frame columns in the given PO direction. Step i. is repeated for all the necessary 2D frames in the building in
both principal directions of the building, in order to calibrate all the springs (two per direction for each column).

Non-linear static analyses are used in the calibration procedure as in most cases the results (in terms of observed plastic mechanisms) can be assumed comparable with those obtained from non-linear dynamic analyses, while significantly less time-consuming. The force distribution used in the non-linear static analyses affects the column responses and their calibration. A uniform distribution is suggested for brevity, but more sophisticated force distributions can be used (e.g. Brozovič and Dolšek 2014, 2016) and this choice should be made by the analyst, based on the building characteristics. Since the soft storey forms at one storey (typically, but not necessarily, the ground storey), to obtain the V-ID curve for each storey the pushover procedure is repeated for each storey imposing a linear elastic behaviour in the columns of all other stories.
GA are non-deterministic search methods based on principles of genetics and natural selection (Holland 1975, Goldberg 1989). The optimisation problem is represented by the GA as an evolution process, where a population of individuals is randomly generated, and then it is left evolving to reach an optimal solution. The maximum number of generations used in the parameter calibration is 100 times the number of variables (i.e., 5×100). The input assumptions are similar to those by Di Sarno et al. (2021).

The GA is implemented in MATLAB with the objective of minimising - for each column spring - the error function between the numerical V-ID curve, obtained from the 2D PO, and the analytical V-ID curve, where initial parameters values are those obtained from the shear-type assumption (i.e., all parameters equal to 1). The error function is expressed as:
where \( i \) indicates the step of the PO, \( V_{\text{numerical},i} \) is the target shear and \( V_{\text{analytical},i} \) is the shear evaluated on the analytical curve (see Figure 4b). The result of the procedure is the set of optimal parameters \((\alpha, \beta, \gamma, \mu, \kappa)\), that implemented in Equation 1 and Equation 2 provide the values for the points defining the optimised analytical (i.e., calibrated) V-ID curve. In order to determine the parameters of the springs in the \( X \) and \( Y \) directions (Figure 2 d, e and f), the procedure must be repeated for all 2D sub-frames (Figure 4a).

### 2.3.4 OpenSees implementation of the constitutive model of the spring

The constitutive model representing the calibrated V-ID curve of each spring is implemented in OpenSees through the uni-axial material \( \text{ModIMKPeakOriented} \) (Lignos and Krawinkler 2012). It is a cyclic relationship with degradation. The updated expressions in terms of \( V \) and \( ID \) are shown in Table A1 in the Appendix. An additional parameter that does not depend on the monotonic backbone is required to describe the cyclic relationship (Lignos and Krawinkler 2012):

\[
\lambda_{\text{cyclic}} = (170.7) \cdot (0.27) \cdot \nu \cdot (0.10) \cdot s_d
\]

where \( \nu \) is the axial load ratio, that is the ratio between the axial load acting on the section and the gross concrete section strength; \( s_d \) is the ratio between stirrup spacing and effective depth.

### 3 PROPOSED MODEL VALIDATION

In order to test the proposed model accuracy, a case study is considered. An irregular moment resisting frame building is chosen, to show that the proposed simplified model is capable of accurately capturing the torsional behaviour. Results of the proposed simplified model are compared with those of a refined model.
The case study building is adapted from a regular structure designed in the 70s and presented in Iervolino and Dolce (2018) and its plan is shown in Figure 5a. It has three storeys, with heights of 3.40, 3.05 and 3.05 m, respectively, and the length of all spans is 3.75 m. Columns may be divided into three groups, namely CA, CB and CC (Figure 5a). Within each group, columns present different geometry and reinforcement characteristics for different floors. Details of the columns and beams cross-sections are reported in Figure A1 and Figure A2 in the Appendix, respectively.

Columns at the first, second and third storey are subjected to an axial load due to gravity loads of 0.3 $P_{cu}$, 0.2 $P_{cu}$, and 0.1 $P_{cu}$ respectively (where $P_{cu}$ is the ultimate axial compressive bearing capacity of the column). All beams have a 30 x 50 cm cross-section. The concrete and steel reinforcement properties (from Iervolino and Dolce 2018) are $f_c = 23.5$ MPa, $f_y = 440$ MPa. They represent mean values.

**Figure 5** – (a) Typical floor plan of the case study building, highlighted are the 2D frames necessary for the springs calibration; (b) refined model of 2-Y frame for simplified model calibration.

### 3.1 Simplified Model

The simplified model derivation follows the procedure described in Section 2. As an example, the calibration of frame 2-Y (Figure 5b) is presented in the following subsection.

#### 3.1.1 Calibration of Frame 2-Y
The external columns of frame 2-Y belong to the CB group, whereas the internal columns belong to the CC group (see Figure A1 in the Appendix). Moreover, each column is different at each storey, thus a total of six backbone curves are derived. Plots in Figure 6 show, for each column, the analytical curve for the shear-type case, the reference curve from the pushover on the refined 2D-frame model and the calibrated V-ID backbone curve.

**Figure 6** - Comparison of V-ID curves for columns CB (top) and CC (bottom): (a) first storey, (b) second storey, (c) third storey

Results for the 2-Y frame show that, even in the case of a regular frame where all beams have the same stiffness and length, the calibrated backbone curves well capture the differences with respect to the shear-type case. The main differences appear in the initial elastic phase. More specifically, the calibrated curves can approximate the moment redistribution due to column yielding, as well as the different positions of the counter-flexure point in the column, with respect to the simplified shear-type approximation of 0.5$L$, with $L$ being the column height.
3.2 Benchmark Model

A refined model (Figure 7a) is used as benchmark for the simplified model (Figure 7b).

![Figure 7](image)

**Figure 7** - (a) extruded STKO view of the benchmark model; (b) STKO view of the simplified model; Node 1: node taken as a reference to show the torsional effects.

For the refined model, columns and beams are modelled as lumped plasticity elements using the `hingedBeam` element in STKO. This command creates an element consisting of two `zeroLength` end rotational springs connected in series with an elastic beam-column element (McKenna et al. 2010). It is worth noting that the overall stiffness of the `hingedBeam` element (taken as $K_{40}$ from Ibarra and Krawinkler 2005) should remain unchanged. Therefore, the stiffness of its sub-elements (plastic hinges and elastic element in the middle) is modified accordingly. This is done by taking $K_{40}$ as the stiffness of three springs in series and deriving back the stiffness of each sub-element, as described in Ibarra and Krawinkler (2005). For each `zeroLength` defining a plastic hinge, two inelastic materials are defined and assigned respectively to the rotation against the two orthogonal bending axes, whereas a rigid material is specified in all other DOFs. The plastic hinges are modelled using the cyclic model with degradation `ModIMKPeakOriented` by Lignos and Krawinkler (2012). The yield moment is
computed using the simplified formulation proposed in Panagiotakos and Fardis (2001) for the beams and using section equilibrium for the columns. The lumped plasticity model considers both ductile and brittle behaviours through the pre-classification approach proposed by De Luca and Verderame (2013).

A floor diaphragm is implemented at each storey level. Axial loads are added as concentrated forces and masses are added as nodal masses, equal in the two translational DOFs. A Rayleigh damping of 5% is applied on the elastic elements only, taking as reference the first and second frequencies of vibration of the structure, derived from modal analysis. No damping is used in the simplified model, that is entirely nonlinear. It is worth noting that, considering a lumped-plasticity model as benchmark implies additional simplifications with respect to a distributed plasticity model. More specifically, the effect of the axial load variation on the seismic behaviour is not taken into account. However, the effect should be negligible since the considered building has only three storeys (e.g., Mazza 2014; Huang and Kwon 2015). Thus, the results of the lumped plasticity model can be considered reasonably accurate. When dynamic non-linear analyses are performed, the same damping model should be used for the refined and simplified model. In this case, Rayleigh damping is used with a 2% damping ratio for the first and third vibration mode on both models.

### 3.3 Pushover Analyses

PO analyses are performed on the simplified and the benchmark models, using both Uniform and Linear lateral force distributions. The loads are applied using the *DisplacementControl* static integrator in OpenSees, setting the top floor centre of mass as control node and a target displacement of 0.2 m, corresponding to a roof drift ratio equal to about 2%. The analysis is performed in both horizontal directions.

*Figure 8* shows the results for the two distributions in terms of total base shear-top displacement in the Y direction, where the torsional effects are more relevant.
Given the similarity of results, the following discussion is referred only to the Linear distribution, as suggested by FEMA 440 (ATC, 2005). Figure 9 compares the results in terms of total base shear and Inter-storey Drift Ratio (IDR) values along the height, in the Y direction. The maximum difference between the refined and the simplified models in terms of base shear and IDR is about 2%; the difference at the first storey, where in both models the maximum IDR takes place, is approximately 0.2%.

Figure 10 shows the deformed shapes of the structure for the benchmark (left) and the simplified (right) models, at the step corresponding to a top displacement of 0.1 m (indicated with a dot in Figure 9). The simplified deformed shape follows quite closely that of the refined model, thus accurately capturing the torsional behaviour of the building.

The analysis in the X direction leaded to even lower differences.
Figure 9 - (a) Comparison of pushover curves; and (b) IDRs for the simplified model and benchmark refined model; all results in the global Y direction.

Figure 10 - Comparison of deformed shape corresponding to a top displacement of 0.1 m (indicated with a dot in Figure 9a) for: (a) benchmark refined model; and (b) simplified model; results in global Y direction. (Units: m).

3.4 Non-Linear History Analyses

3.4.1 Simplified model validation

Non-linear history analyses (NLHA) are performed on both the simplified and the benchmark models. The comparison is performed for two ground motions (Table 1).
<table>
<thead>
<tr>
<th></th>
<th>Related Event</th>
<th>PGA</th>
<th>Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Signal 1</strong></td>
<td>Imperial Valley (1979)</td>
<td>0.24 g</td>
<td>50.00 s</td>
</tr>
<tr>
<td><strong>Signal 2</strong></td>
<td>Chi-Chi (1999)</td>
<td>0.36 g</td>
<td>52.78 s</td>
</tr>
</tbody>
</table>

Results are similar for the two ground motions, thus for the sake of brevity, only results obtained with the Chi-Chi ground motion records are discussed. The analysis is performed using the Newmark transient integrator with parameters $\beta = 0.25$ and $\gamma = 0.5$. The integration time step is 0.01s. The adaptive time step option in OpenSees is used in the refined model in order to achieve convergence. The base shear history shows a very good agreement between the simplified and the refined model (Figure 11), with differences in the maximum base shear smaller or equal than 1%.

The same behaviour is observed for the base-shear top-displacement hysteretic behaviour (see Figure 12). The plot also includes the pushover curves, to show that the dynamic behaviour is generally consistent with the non-linear static results, for both models. It is worth noting that in the Y direction the dynamic curves reach a higher base shear than in the static curves. This happens because the maximum shear recorded in the static analyses derives from a different bending moment distribution than in the dynamic case. In the dynamic case, the point of counter-flexure is closer to column mid-height with respect to the static case. This phenomenon gets more significant as the beams become more flexible. In the extreme case with no beams, it happens that during static analyses the columns of the first (bottom) floor reach yielding (and then failure) at the bottom section, while the top section is still far from yielding. On the other hand, during dynamic analyses, the top sections undergo higher moment demand, resulting in overall higher shear demands on the columns. The simplified model is able to accurately capture this behaviour.
Figure 11 – Base-shear response histories for the simplified and benchmark refined models in (a) X direction and (b) Y direction with reference to the Chi-Chi ground motion record.

Figure 12 – Hysteretic behaviour of the simplified and benchmark refined models in (a) X direction and (b) Y direction with reference to the Chi-Chi ground motion record.

In terms of maximum IDR demand, the same distribution along the height is observed (Figure 13a-b). More specifically, both models predict a larger IDR at the base storey, with a relative difference in the value of about 10%. As for the Peak Floor Accelerations (PFAs), the maximum value is predicted with differences smaller than 7% (Figure 13c-d).
Table 2 – Comparisons between the simplified and refined models, with reference to the Chi-Chi ground motion record.

<table>
<thead>
<tr>
<th></th>
<th>Simplified</th>
<th>Refined</th>
<th>Relative Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Base Shear [kN]</td>
<td>x</td>
<td>1572.3</td>
<td>1588.0</td>
</tr>
<tr>
<td></td>
<td>y</td>
<td>1836.0</td>
<td>1823.4</td>
</tr>
<tr>
<td>Max IDR [% of h_{storey}]</td>
<td>x</td>
<td>2.56 (1-2)</td>
<td>2.34 (1-2)</td>
</tr>
<tr>
<td></td>
<td>y</td>
<td>1.96 (1-2)</td>
<td>1.77 (1-2)</td>
</tr>
<tr>
<td>Max PFA [m/s²]</td>
<td>x</td>
<td>5.03 (L3)</td>
<td>4.64 (L3)</td>
</tr>
<tr>
<td></td>
<td>y</td>
<td>4.82 (L2)</td>
<td>4.75 (L3)</td>
</tr>
</tbody>
</table>

Figure 13 – Comparison in terms of IDR (a, b) and PFA (c, d) between the simplified and the benchmark refined models for the direction X (a, c) and Y (b, d) with reference to the Chi-Chi ground motion record.
In order to show that the simplified model reproduces the torsional behaviour of the building, Figure 14 compares the response of the top floor centre of mass (right) and that of Node 1 (see Figure 7) in terms of $U_x$ versus $U_y$ displacements. Two main observations are drawn. First, the simplified model response is quite close to the response of the refined model, with differences for Node 1 slightly higher than for the centre of mass, due to the influence of the building rotation around the vertical axis. Second, the displacement histories of Node 1 and of the centre of mass are quite different, particularly in the Y direction, as a result of the non-negligible torsional rotations of the building around the vertical axis.

The above results show that the simplified model is able to reproduce the refined model results with satisfactory accuracy for both non-linear static and non-linear dynamic analyses. Furthermore, the simplified model accurately captures the torsional response of the benchmark structure.

3.4.2 Statistical characterisation of modelling bias

Having shown the proposed simplified model capability in capturing the structure behaviour of the case study building, the same simplified model is further investigated using a
larger set of ground motions. The set is made of 150 unscaled ground motions extracted from the SIMBAD database, already used in other studies (Giordano et al. 2021 and Gentile and Galasso 2019). Both the X and Y horizontal components of the ground motions have been simultaneously applied during the analyses (see Figure A3 in the Appendix for the elastic spectra of the ground motions).

For each analysis, the difference between the refined and the simplified models is considered, again with reference to the IDR and PFA values. The choice is driven by the fact that these are the most commonly used EDPs for loss assessments (e.g., ATC 2018a,b, d’Aragona et al. 2021). Figure 15 and Figure 16 show the median (\(\eta\)) and the standard deviation (\(\beta\)) of the IDR and PFA values, respectively, at each storey level for the refined (labelled Ref.) and simplified (labelled Simp.) models. The results refer to Node 1 (see Figure 7) and Node 2, which is the diagonal opposite of Node 1. A plan layout of the analysed building with highlighted the considered Node is added to each plot for increased readability.

The comparison in terms of IDRs (Figure 15) shows a close match of the median results for the first and third storey, with larger differences in the median for the second storey. In the latter, the absolute values of the IDRs are lower with respect to the other stories, for both the refined and simplified models, as damage seldom concentrates at this story in the 150 runs performed. At the first and third storey a significant dispersion (i.e., high \(\beta\)) is observed, both in the refined and simplified models. Indeed, these are the storeys where failure tends to concentrate in the 150 analyses performed. More specifically, there is prominence of failure at the first storey (where the upped boundary of \(\eta + \beta\) exceeds 3%), and this is captured by both the models.

PFA results (Figure 16) show a very good match between the refined and simplified models. This EDP is characterised by inherent high variability that is reflected in relatively large \(\pm \beta\) boundaries for both models. The median results for the first storey are very close
while there is some minor mismatch between median results at the second and third storeys, though the results are still very satisfactory.

**Figure 15** – Statistical comparison of the differences between the refined and simplified model for IDR at each storey in the X and Y direction for the (a) and (b) Node 1 and (c) and (d) Node 2.
Figure 16 – Statistical comparison of the differences between the refined and simplified model for PFA at each storey in the X and Y direction for the (a) and (b) Node 1 and (c) and (d) Node 2.

Table 3 – Comparisons between simplified and refined models – results of the 150 analyses.

<table>
<thead>
<tr>
<th>EDP</th>
<th>Storey</th>
<th>Direction</th>
<th>Node 1: $\eta (\beta)$</th>
<th>Node 2: $\eta (\beta)$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Simplified</td>
<td>Refined</td>
</tr>
<tr>
<td>IDR</td>
<td>1st</td>
<td>x</td>
<td>1.24 (1.67)</td>
<td>1.27 (1.70)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>y</td>
<td>1.51 (1.64)</td>
<td>1.42 (1.58)</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>x</td>
<td>0.65 (0.56)</td>
<td>0.58 (0.46)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>y</td>
<td>0.79 (0.52)</td>
<td>0.71 (0.45)</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>x</td>
<td>0.60 (0.46)</td>
<td>0.62 (0.49)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>y</td>
<td>1.17 (0.53)</td>
<td>1.16 (0.56)</td>
</tr>
<tr>
<td>PFA</td>
<td>1st</td>
<td>x</td>
<td>3.61 (0.44)</td>
<td>3.62 (0.44)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>y</td>
<td>3.98 (0.43)</td>
<td>3.95 (0.45)</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>x</td>
<td>3.80 (0.42)</td>
<td>3.73 (0.41)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>y</td>
<td>4.06 (0.42)</td>
<td>4.22 (0.41)</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>x</td>
<td>3.69 (0.42)</td>
<td>3.82 (0.42)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>y</td>
<td>4.02 (0.41)</td>
<td>4.07 (0.40)</td>
</tr>
</tbody>
</table>
3.4.3 Discussion on Modelling Approach and Computational Efficiency

The proposed simplified model presents significant advantages in terms of computational efficiency. It shows much faster convergence with respect to the refined model, with an overall significant reduction in runtime, as shown in Figure 17. More specifically, each point in the plot represents the analysis of one ground motion among the 150 considered. The x-axis indicates the running time (in minutes) of the analyses using the simplified model while the y-axis shows the running time of the refined model. It is evident that the simplified model requires significantly lower time to perform the analysis being on average 10-20 times faster with peaks that can reach even 100 times when convergence issues arise in the refined model. This is highlighted by the red dotted line, which represents the 45-degree reference line (bisector).

![Figure 17](image)

**Figure 17** – Comparison of computational time between simplified (x axis) and refined (y axis) models.

This significant reduction in computational times makes the proposed simplified model suitable for all those applications where a large number of ground motions are applied to the building, such as Cloud (Jalayer et al. 2015) or Multiple Stripe (Ricci et al. 2018) analyses. On the other hand, where the building-to-building variability is the main object of the study, and a only a few ground motions are considered, the proposed model may not present significant
advantages with respect to a refined approach. Indeed, the time required by the modelling may be comparable in the two cases, given the effort needed in the current calibration procedure. The most time-consuming step is currently the modelling of the 2D frames to derive the reference curves from the PO analyses. On the other hand, the time needed by the GA is negligible, as it never exceeded 10 seconds per column.

4 CONCLUSIONS

In this paper, a simplified model for RC MRF buildings is proposed. The modelling approach overcomes the main drawbacks of existing shear-type stick-like simplified models, as it accounts for both end-nodes rotational flexibility and irregularities in plan and/or in elevation, thus capturing the torsional behaviour of a building.

In the proposed simplified approach, each column is modelled as two independent springs that represent the V-ID curves in the two global horizontal directions. Backbone curves for the springs are defined by parameter-dependent analytical equations and are calibrated to reflect the actual building configuration (e.g. non-shear-type with end-nodes rotational flexibility). The calibration is performed through a genetic algorithm that minimizes the differences between the simplified and the target curves. The target curves are obtained from non-linear static analyses on 2D sub-frames modelled with a refined approach. The overall definition of the simplified model requires additional effort when compared to other existing approaches, but this is considered a reasonable trade-off to account for the torsional behaviour and moreover the modelling time becomes less and less relevant as the number of analyses to perform increases.

The proposed model is applied to a 3D plan-irregular RC MRF building, and both non-linear static and dynamic analyses are performed for validation. Results are compared with those obtained from a refined benchmark model and show very good agreement between the responses of the two models. Most importantly, the simplified model can capture the torsional
behaviour of the case study building with high accuracy. The simplified model leads to a significant reduction in computational time, proving to be an efficient alternative to more refined models when a huge number of non-linear dynamic analyses is required.

Despite the novel contributions of the proposed model, additional issues can be addressed in future work. More specifically, the model needs to be applied to diverse case study buildings in order to validate its wider applicability. Furthermore, the overall process of defining the simplified model could be significantly sped up by automatising all the necessary steps (i.e., refined modelling of the frames, pushover analyses and spring calibration). Likewise, a significant speed-up could be gained by developing an alternative data driven calibration technique, able to derive the optimized version of the parametric V-ID curves without the need for any refined model. In addition, the model could be modified to include the effects of the infills by considering the increase in lateral stiffness and the irregularity caused by the configuration of the infill walls. The objective is also to extend the model applicability to buildings with limited geometrical and structural data. Finally, the proposed simplified model is part of a broader work, aimed at investigating the soil-structure and structure-soil-structure interaction by performing analyses including a finite volume of soil.

ACKNOWLEDGEMENTS

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REFERENCES


Lu, X., Han, B., Hori, M., Xiong, C., & Xu, Z. (2014). A coarse-grained parallel approach for seismic damage simulations of urban areas based on refined models and GPU/CPU
cooperative computing. Advances in Engineering Software, **70**, 90-103. DOI: https://doi.org/10.1016/j.advengsoft.2014.01.010


### Appendix

Table A1 – OpensSees parameters, formulas and values used for the column CC L1 of the case study building.

<table>
<thead>
<tr>
<th>OpenSees Parameter</th>
<th>Description (for M-θ)</th>
<th>Formula (for V-ID)</th>
<th>e.g. Value (CC L1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$matTag$</td>
<td>Integer tag identifying material.</td>
<td>integer</td>
<td>1</td>
</tr>
<tr>
<td>$K0$</td>
<td>Elastic stiffness.</td>
<td>$\frac{V_{40plus}}{ID_{40plus}}$</td>
<td>3392.07</td>
</tr>
<tr>
<td>$as_Plus$</td>
<td>Strain hardening ratio for positive loading direction.</td>
<td>$\frac{V_{uplus} - V_{yplus}}{(ID_{uplus} - ID_{yplus}) \cdot K_0}$</td>
<td>0.040538</td>
</tr>
<tr>
<td>$as_Neg$</td>
<td>Strain hardening ratio for negative loading direction.</td>
<td>$\frac{V_{uneg} - V_{yneg}}{(ID_{uneg} - ID_{yneg}) \cdot K_0}$</td>
<td>0.040538</td>
</tr>
<tr>
<td>$My_Plus$</td>
<td>Effective yield strength for positive loading direction.</td>
<td>$V_{40plus}$</td>
<td>49.98</td>
</tr>
<tr>
<td>$My_Neg$</td>
<td>Effective yield strength for negative loading direction (negative value).</td>
<td>$V_{40neg}$ (with negative sign)</td>
<td>-49.98</td>
</tr>
<tr>
<td>$Lamda_S$</td>
<td>Cyclic deterioration parameter for strength deterioration.</td>
<td>$\frac{\lambda_{yc} \cdot (ID_{yneg} + ID_{yplus})}{2}$</td>
<td>0.801275</td>
</tr>
<tr>
<td>$Lamda_C$</td>
<td>Cyclic deterioration parameter for post-capping strength deterioration.</td>
<td>$Lamda_S$</td>
<td>0.801275</td>
</tr>
<tr>
<td>$Lamda_A$</td>
<td>Cyclic deterioration parameter for accelerated reloading stiffness deterioration.</td>
<td>$Lamda_S \cdot 1000$</td>
<td>801.275</td>
</tr>
<tr>
<td>$Lamda_K$</td>
<td>Cyclic deterioration parameter for unloading stiffness deterioration.</td>
<td>$Lamda_A$</td>
<td>801.275</td>
</tr>
<tr>
<td>$c_S$</td>
<td>Rate of strength deterioration. The default value is 1.0.</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>$c_C$</td>
<td>Rate of post-capping strength deterioration. The default value is 1.0.</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>$c_A$</td>
<td>Rate of accelerated reloading deterioration. The default value is 1.0.</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>$c_K$</td>
<td>Rate of unloading stiffness deterioration. The default value is 1.0.</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>$theta_p_Plus$</td>
<td>Pre-capping rotation for positive loading direction (often noted as plastic rotation capacity).</td>
<td>$ID_{uplus} - ID_{40plus}$</td>
<td>0.068696</td>
</tr>
</tbody>
</table>

41
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\theta_{p_Neg}$</td>
<td>Pre-capping rotation for negative loading direction (must be defined as a positive value).</td>
<td>$ID_{\text{Uneg}} - ID_{\text{Aoneg}}$</td>
<td>0.068696</td>
</tr>
<tr>
<td>$\theta_{pc_Plus}$</td>
<td>Post-capping rotation for positive loading direction.</td>
<td>$ID_{\text{Cplus}} - ID_{\text{Uplus}}$</td>
<td>0.064210</td>
</tr>
<tr>
<td>$\theta_{pc_Neg}$</td>
<td>Post-capping rotation for negative loading direction (must be defined as a positive value).</td>
<td>$ID_{\text{Cneg}} - ID_{\text{Uneg}}$</td>
<td>0.064210</td>
</tr>
<tr>
<td>$Res_Pos$</td>
<td>Residual strength ratio for positive loading direction.</td>
<td>$\frac{V_{\text{Cplus}}}{V_{\text{Yplus}}}$</td>
<td>0</td>
</tr>
<tr>
<td>$Res_Neg$</td>
<td>Residual strength ratio for negative loading direction (must be defined as a positive value).</td>
<td>$\frac{V_{\text{Cneg}}}{V_{\text{Yneg}}}$</td>
<td>0</td>
</tr>
<tr>
<td>$\theta_{u_Plus}$</td>
<td>Ultimate rotation capacity for positive loading direction.</td>
<td>$ID_{\text{Cplus}}$</td>
<td>0.147641</td>
</tr>
<tr>
<td>$\theta_{u_Neg}$</td>
<td>Ultimate rotation capacity for negative loading direction (must be defined as a positive value).</td>
<td>$ID_{\text{Cneg}}$</td>
<td>0.147641</td>
</tr>
<tr>
<td>$D_Plus$</td>
<td>Rate of cyclic deterioration in the positive loading direction (this parameter is used to create asymmetric hysteretic behaviour for the case of a composite beam). For symmetric hysteretic response use 1.0.</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>$D_Neg$</td>
<td>Rate of cyclic deterioration in the negative loading direction (this parameter is used to create asymmetric hysteretic behaviour for the case of a composite beam). For symmetric hysteretic response use 1.0.</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>
Figure A1 – Sections of columns CA, CB and CC of the case study building.

Figure A2 – Section of the beams of the case study building.
Figure A3 – Elastic acceleration response spectra of the two components of the 150 ground motions employed in the analysis (applied respectively along x and y direction of the building) showing the shape of the median ($\eta$) response and $\eta$ plus and minus one standard deviation of the logarithms ($\pm \beta$).