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# Analytical model verification for improved performance-based design of MRFs

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## Abstract

As the development of the codes of practice converges towards limiting displacements rather than forces, it is of utmost importance to be able to design the structures for predefined performance criteria. Following the objective-based design, one can accurately predict the structural displacements during realistic earthquake loading and evaluate the response of the structural elements. The main idea of this study is to demonstrate an improved methodology for design and assessment of steel moment resisting frames subjected to loading from recorded ground acceleration. Namely, material degradation model is used for modelling of the plastic hinges in the beams of the frame in order to realistically capture the behavior of the steel during earthquake. Additionally, the same frame is developed with a typical distributed plasticity model in order to validate and to prove the necessity of the improved method. Firstly, the design of the frame according to the current codes of practice is performed. Then, the performance objectives of the analysis are defined and the various limits in the existing literature are considered. In order to perform the nonlinear time history (NLTH) analysis, modelling of the two types of frames in a suitable software is conducted. Prior to that, case study for model calibration and verification is performed and the results obtained are used for the setup of parameters for NLTH analysis. The NLTH analysis is done after the selection of a reliable set of recorded ground motions. Hence, the performance-based assessment of the steel moment resisting frames is conducted and several demand parameters are recorded. Since the moment resisting frames are the most ductile type of steel structural configurations, they exhibit very large deflections before the structural damage occurs. Therefore, the inter-story drift ratios and residual drifts are measured and their compliance with the existing codes is compared. In the end, conclusions regarding the behavior of the moment resisting frames subjected to realistic earthquake loading are drawn and suggestions for improvements in the design are presented.

***Key words:*** performance-based; moment resisting frames; verification; pushover; non-linear analysis

# 1 Introduction

The MRFs are characterized as the most ductile structural type possessing many possible dissipative zones, following the fact that plastic hinges can develop in the beams and the columns. Possessing the feature of being the most ductile type, these structures exhibit very large deflections before the structural damage occurs. On one hand, that is a good characteristic as their ductility level is high and less material is used without a failure of the structure but on the other hand the large inter-story drifts can lead to great damage to the non-structural components which is a quality to be avoided. Even though the overall stability of the structure can be easily satisfied following the 'weak beam – strong column' rule from EC8 [1] which allows for maximum energy dissipation capacity, due to the MRF's low lateral stiffness, the drift effects need careful considerations.

The drift parameter and the general behaviour of the moment resisting frames have been researched in many previous studies in the field of the earthquake engineering. There are plenty of research findings on the evaluation of the realistic drifts of the steel MRFs and their comparison with the existing codes of practice. One study shows that EC8 requirements regarding the drift are a lot more rigorous than the provisions in other earthquake design codes [2], specifically characterized by the stability factor ($\theta$). This factor, as a result of the strict drift and stability constraints and the sensitivity of the moment resisting frames to the lateral deformation effects, can often govern the design, resulting in over-strength which reduces the ductility demand of the structure and affects the loads acting on it, especially if a high ductility factor is adopted. In the same research, another inconsistency in the European provisions is highlighted. That is the disregarding of the important influence of the gravity loads on the over-strength of beams for which the author introduces a new $\Omega_{mod}$ parameter as a replacement for the already existing $\Omega_{EC8}$ parameter.

Another research investigating the structural behaviour of steel moment resisting frames shows the influence of the ground motion characteristics on the drift demands [3]. Regarding the drift provisions issue in EC8, in this research it was observed that EC8 provisions are highly conservative while the US provisions seem to under-predict the global and maximum drift modification factors which agrees with the previously mentioned research. The authors of this study specifically point out the oversimplified nature of drift demand criteria adopted in the design codes, particularly in the European codes of practice. They suggest that: 'significant enhancement in the reliability of these approaches can be achieved by adopting improved models that can capture the influence of the key structural and loading characteristics.'

In this study, the implementation of an improved modelling approach for assessment of the moment resisting frames is presented. Firstly, the two separate techniques for modelling of the steel elements' plasticity are presented and distinguished. Then, a calibration methodology is shown in order to verify the improved material model and to equate the structural characteristics in order to perform a comparative analysis. Finally, the modelling approaches are investigated on a frame model and the results are compared through a case study.

## 2 Improved modelling approach

The distributed plasticity models constituted of nonlinear elements are the most frequently used models in the nonlinear analysis of steel moment resisting frames and the core element used in the previous studies relevant to this. The elements possessing distributed plasticity allow for the spread of plasticity along the element length. This approach is more computationally demanding than the lumped plasticity approach with plastic hinges, as it incorporates integration of the nonlinear cross-sectional response along the element length and over the cross-section depth.

One of the first steps towards implementation of improved models for designing and assessing of steel moment frames has been undertaken in the research of Ibarra, Medina & Krawinkler [4]. Their research consists of description, calibration and application of relatively simple hysteretic models that include stiffness and strength deterioration materials which are critical for demand predictions as a structural system approaches collapse. They performed a great deal of experimental tests which allowed them to follow up with some very consistent and reliable stiffness deterioration models. According to their study, at the early stages of inelastic behaviour, both deteriorating and non-deteriorating systems exhibit similar responses, but the differences become significantly important when the post-capping stiffness is attained in the response. In another words, the response of the structural systems that undergo large inelastic excursions is controlled by deterioration in assemblies of components. The authors suggest that the implementation of a stern assessment of a structure subjected to earthquake loading is not complete without the application of models capable of recording the history of damage at different levels of seismic action until collapse.

The deterioration model developed by Ibarra, Medina & Krawinkler, referred to as Ibarra - Krawinkler (IK) model, presents the breakthrough of the use of the deteriorating stiffness models in the field of earthquake engineering. This model was then improved [5] to refer to the asymmetric element hysteretic performance, incorporating varying rates of cyclic deterioration in the two separate loading directions, implementing residual strength parameter, and inclusion of an ultimate deformation $\theta_u$ at which the strength of a component deteriorates to zero as a result of unstable crack growth and fracture, Fig. 1.

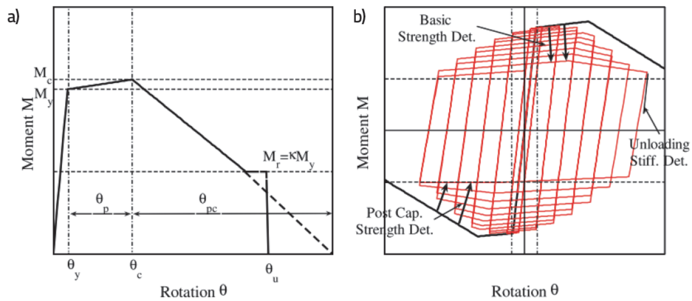


**Figure 1. Modified IK model: (a) monotonic curve; (b) basic modes of cyclic deterioration. [5]**

The main parameters incorporated in the modified IK deterioration model are: the pre-capping rotation $\theta_p$, which represents the difference between the yield rotation and the rotation at the maximum moment, Eq. 1; the post-capping rotation $\theta_{pc}$, which is a term explaining the plastic deformation increase after capping of the material until zero-strength condition, Eq. 2; and the 'reference cumulative plastic rotation' – $\Lambda$, which is a parameter defining the rate of cyclic deterioration derived on the basis of the hysteretic energy dissipated when the element is undergoing cyclic loading, Eq. 3.

$$\theta_p = 0.0865 \cdot \left(\frac{h}{t_w}\right)^{-0.365} \cdot \left(\frac{b_f}{2 \cdot t_f}\right)^{-0.140} \cdot \left(\frac{L}{d}\right)^{0.340} \cdot \left(\frac{c_{unit}^1 \cdot d}{533}\right)^{-0.721} \cdot \left(\frac{c_{unit}^2 \cdot F_y}{355}\right)^{-0.230} \quad (1)$$

$$\theta_{pc} = 5.63 \cdot \left(\frac{h}{t_w}\right)^{-0.565} \cdot \left(\frac{b_f}{2 \cdot t_f}\right)^{-0.800} \cdot \left(\frac{c_{unit}^1 \cdot d}{533}\right)^{-0.280} \cdot \left(\frac{c_{unit}^2 \cdot F_y}{355}\right)^{-0.430} \quad (2)$$

$$\Lambda = 495 \cdot \left(\frac{h}{t_w}\right)^{-1.26} \cdot \left(\frac{b}{2 \cdot t_f}\right)^{-0.525} \cdot \left(\frac{L_b}{r_y}\right)^{-0.130} \cdot \left(\frac{c_{unit}^2 \cdot F_y}{355}\right)^{-0.291} \quad (3)$$

In these equations, the parameters $h$, $t_w$, $b_f$, $t_f$, $L$, $d$ and $F_y$ are obviously parameters obtained from the cross-sectional, element and material properties, while the multipliers $c^1_{unit}$ and $c^2_{unit}$ are used for unit conversion dependent on the unit system the user prefers. If the metric system is used the value of the $c$ parameters is 1.0.

## 3 Model calibration

One of the most important parts of the implementation of the stiffness and strength deterioration model is the calibration of the parameters which are used as input for modelling the investigated structure. The proposed modified IK model can be calibrated following the recommendations of the model developers who have performed large set of experiments with load-deformation data.

In order to understand the need of the model calibration, firstly the modelling of the steel moment resisting frame with the lumped plasticity elements should be described. Namely, this model differs from the distributed plasticity model in the way of the design of the beams. Considering the fact that the EC8 suggests the 'weak beam – strong column' approach for seismic design of frames, it can be understood that the designers always tend to assign the formation of the plastic hinges in the beam elements. That idea leads to the most effective layout of the lumped plasticity approach.

The stiffness deteriorating springs in the concentrated plasticity model are placed only at the ends of the beams as the intention is to limit the occurrence of plastic hinges during realistic earthquake loading in the beams of the structure. That leads to beam frame elements formed out of two springs at the assumed location of plastic hinges connected with an elastic finite beam - column element in the elastic central beam region.

However, the use of the elastic beam - column elements ended with rotational springs has its deficiency in the need of calibration of the model in order to obtain the real stiffness and strength characteristics, easily achieved with the distributed plasticity model. The implementation of two totally different element types provokes the problem in the second modelling approach as the structural properties of the investigated members (the beams) are a combination of the individual properties of the integral components.

The two possible solutions of this issue are stated in the following section. The rotational stiffness of the frame element (the beam in the particular case) can be represented through the Eq. 4:

$$K_{el} = \frac{1}{\frac{1}{K_s} + \frac{1}{K_{bc}}} = \frac{K_s K_{bc}}{K_s + K_{bc}} \tag{4}$$

The first solution for the constituent element stiffnesses would be the increase of the beam - column element stiffness to infinity in which case $K_s = K_{el}$. In this case, all of the element deformations would be forced into the nonlinear spring hinges which would lead to non-varying elastic spring stiffness in the nonlinear analysis and spurious damping moments at the frame joints because of the transfer of all the damping in the rotational springs.

The other option considers assigning infinite stiffness of the springs resulting in $K_{bc} = K_{el}$. This option causes numerical instability problems and makes it impossible to express the strain hardening and post-capping stiffnesses as fractions of the elastic spring stiffness [6].

For the purpose of avoiding the problems of both options, the stiffness and strength deterioration model authors implement a random multiplier *'n'*, which can be assigned to the stiffness parameters of the sub-elements. Namely, the rotational spring stiffness is assumed to be '*n*' times larger than the rotational stiffness of the beam - column element, expressed as $K_s = nK_{bc}$, where $n >> 1$. It is then possible to express the sub-elements' stiffnesses in terms of the member overall stiffness and the newly incorporated multiplier *n*:

$$K_{bc} = \frac{n+1}{n} \cdot K_{el} \tag{5}$$

$$K_s = (n+1) \cdot K_{el} \tag{6}$$

There are a few ways of application of the model calibration procedure to the actual structural model investigated in the study. In this particular research, the parameter *n* is defined in the OpenSees [7] code for modelling the lumped plasticity frame. The value of $n = 10$ is used for the modelling as the authors of the stiffness deterioration model themselves used this value for the study and the development of the calibration procedure. Then, in order to affect the specific sub-element's stiffness, the beam - columns' moments of inertia are multiplied by the proposed factor of $(n+1)/n$. Additionally, the nonlinear rotational spring's stiffness is multiplied by the $(n+1)$ factor resulting in the complete implementation of the suggestions of the developers of the deterioration model and the calibration procedure.

In order to clarify the differences between the models, a pushover analysis is performed. The main difference between the models is evaluated to occur at greater levels of inelasticity represented with more intense seismic loads and roof drift value of around 3%, observed in Fig. 2. That value presents the global frame's deformation zone when the strength and stiffness deterioration property of the lumped plasticity hinge regions occurs and starts deviating the structure's response from the one modelled following the distributed plasticity approach [8].

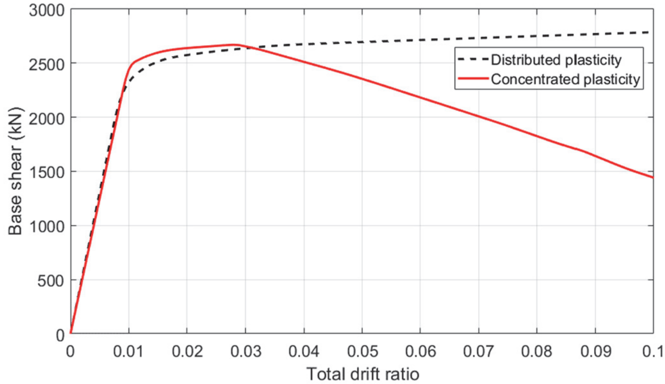


**Figure 2. Pushover curve for the case study model following both modelling techniques**

# 4 Case study

## 4.1 Structural characteristics

The studied frame consists of three bays of 6.0 m span, a first story height of 4.5 m and upper stories height of 3.5 m each. For the initial modelling and design of the frame, presented in this report, joints are assumed to be full-strength and rigid. In this specific research, the joints are not of a big concern but these MRF' constituent elements are a prospective research topic for future improving of assessment of the MRF structures. The story plan is square with uniform span lengths in both principal directions. The lateral force resisting system is placed at the perimeter of the plan of the buildings. The interior frames are assumed to be gravity frames and their lateral load resisting capacity is neglected. Two-dimensional frame models are used for the design, with appropriate selection of the separate areas for gravity and seismic loads. A conceptual schematic with the typical frame elevation developed in commercial software for preliminary design is presented in Fig. 3.


**Figure 3. The case study frame with its constituent elements**

The material for all frame elements is S355 steel with an over-strength factor $\gamma_{ov}$ =1.25 which gives a yield strength of the frame elements of 443.75 kN/m2. The Young's modulus $E$ = 210 GPa, Poisson's ratio $\nu$ = 0.3 and 0.25% linear strain hardening (as a percentage of the initial elastic modulus). The choice of the behavior factor '*q*' is mostly based on the maximum allowed by EC8 for the frame type, which is assumed to be a common assumption in standard design practice despite possible limitations. For this particular case of MRF, a value of $q$ = 6.5 is used. The lateral force method of analysis is employed for calculating the seismic action effects. No torsional effects are considered. The permanent loads, the imposed loads and the relevant combination factors are selected for an intended residential or office use of the building.

## 4.2 Ground motion records

A set of seven ground motion records is incorporated in the study. The first 7-record subset represents a medium seismic hazard scenario (MH) and is extracted from the database using the following criteria: magnitude $M$ from 5.0 to 6.5, distance from fault 10 km to 100 km, shear wave velocity $V_s$ from 180 m/s to 800 m/s, EC8 (Standard, 2005) Type 1 Soil C target spectrum with peak ground acceleration $PGA$ = 0.25 g and minimization of $D_{RMS}$ over a period range from 0.2 s to 2.0 s. Basic descriptive and seismological data, the computed scaling factors and the corresponding values of $D_{RMS}$ are presented in Table 1.

**Table 1. Seismological and scaling data for the 7-earthquake set of ground motions**

| NGA record no. | Name | Date | Magnitude | Distance from fault [km] | PGA [g] | Scale factor |
|---|---|---|---|---|---|---|
| 564L | Kalamata, Greece | 13.09.86 | 6.2 | 11.2 | 0.26 | 1.23 |
| 127T | Friuli, Italy | 11.09.76 | 5.5 | 15.1 | 0.05 | 7.55 |
| 299L | Irpinia, Italy | 23.11.80 | 6.2 | 41.7 | 0.04 | 9.83 |
| 302T | Irpinia, Italy | 23.11.80 | 6.2 | 22.7 | 0.11 | 3.49 |
| 1137T | Dinar, Turkey | 01.10.95 | 6.4 | 35.6 | 0.04 | 7.16 |
| 130L | Friuli, Italy | 15.09.76 | 5.9 | 14.3 | 0.11 | 3.03 |
| 481L | Lazio-Abruzzo, Italy | 07.05.84 | 5.8 | 45.5 | 0.04 | 9.25 |

## 4.3 NLTH results

For the need of performance assessment of the structures subjected to seismic loading, whether expressed as a realistic ground motion history, a pushover or a pseudo - static lateral force analysis, the codes of practice propose three types of performance assessment, two of which are adopted in this research.

The scenario-based assessment evaluates the response of a structure and its components to a user-specified earthquake events, usually defined and categorized according to the event magnitude and the distance between the structure and the earthquake source. Then, the intensity-based assessment, which is the most common type of performance assessment of structures subjected to seismic action, is used for computing the structural response for a predefined intensity level of ground movement. The response of the structure is represented through several parameters such as element forces and deformations, story drifts or global estimates of loss. Finally, the last type of assessment is the risk – based which is used to provide information about the structural response over a user-specified period of time. It is believed to be the most thorough assessment type as it involves a set of intensity-based assessments over the range of ground motion levels of interest. The predicted responses for each intensity-based assessment are then combined (integrated) with the ground motion hazard curve to predict the annual rates of exceedance. In this structural evaluation study, the scenario and intensity-based assessments are incorporated and the steel MRFs are subjected to several analyses.

Following the scenario based structural performance assessment, the MRFs are subjected to medium hazard scenario while regarding the intensity levels, two levels of 100% and 175% are considered. Namely, in EC8 the levels of the seismic action on structures are defined as the design earthquake scenario considering 100% intensity of the ground motion and near collapse scenario represented with 175% of the ground movement intensity. The results from the analysis regarding the maximum and residual inter-story drifts are shown in Fig. 4.

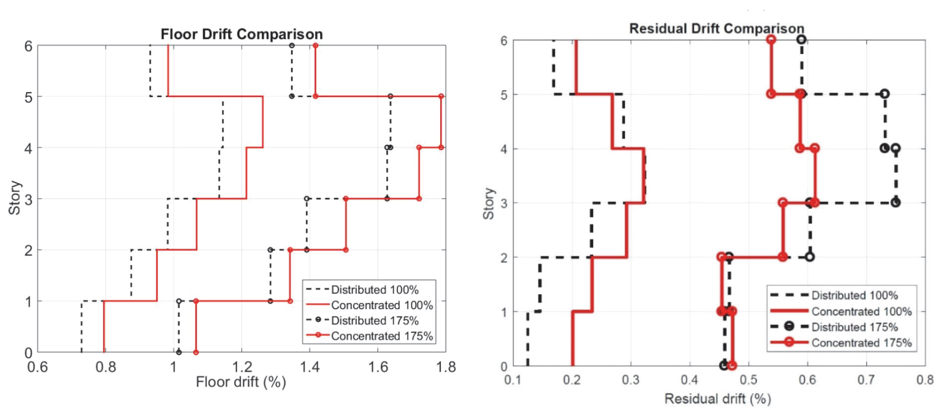


**Figure 4. The maximum (top) and residual (bottom) inter-story drifts following NLTH analysis**

## 5 Discussion

To summarize, the concentrated plasticity model differs from the distributed plasticity model in the technique of modelling of the beams. Only the nonlinear beam-column element from the previous model is replaced in the lumped plasticity method with three sub-elements: the elastic beam-column ending with two rotational springs with stiffness and strength deteriorating properties.

The general contrast between the approaches compared in the study can be observed from the graph in Fig. 2. Until the roof displacement measures about 3% of the total building height, both frames behave similarly, but after that displacement limit the strength and stiffness deterioration of the material in the lumped plasticity model starts influencing the pushover curve and the base shear capacity of the investigated frame starts decreasing.

Additionally, it is observed that the improved material model (depicted as 'concentrated' in Fig. 4) predicts more flexible structural response considering the floor drift parameter. Within this scenario, the differences between the two models are almost negligible and the greatest variation is 10 %. Regarding the residual drifts, it can be observed that both modelling approaches experience very small average residual drifts from the set of seven medium hazard earthquake records with the value of 0.75 % being far from the codified drift allowance. It is observed yet again that the drift demand of the lumped plasticity model is greater than the demand of the distributed plasticity model in the first floors of the frame. The difference from the maximum floor drifts is at the floor with the greatest residual drift demand (fourth floor) where the responses of both modelling methods match. Generally though, it can be concluded that the lumped model predicts greater residual drift demand than the distributed plasticity model.

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