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A fibre model for push-over analysis of underdesigned reinforced concrete frames

Edoardo Cosenza, Gaetano Manfredi *, Gerardo M. Verderame

Department of Structural Analysis and Design, University of Naples Federico II, Via Claudio, 21, 80125 Napoli, Italy

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Abstract

Most of the existing reinforced concrete buildings were designed according to early seismic provisions or, sometimes, without applying any seismic provision. Some problems of strength and ductility, like insufficient shear strength, pull-out of rebars, local mechanisms, etc., could characterize their structural behaviour. The above mentioned topics lead to a number of problems in the evaluation of the seismic behaviour of reinforced concrete (RC) frames. Therefore the assessment of existing RC structures requires advanced tools. A refined model and numerical procedure for the non-linear analysis of reinforced concrete frames is presented. The current version of the model proposed is capable of describing the non-linear behaviour of underdesigned reinforced concrete frames including brittle modes of failure. Selected results of an experimental–theoretical comparison are presented to show the capabilities of this model. The results show the capacity of the model of describing both the global behaviour and the local deformation at service and ultimate state.

Keywords: Non-linear; RC frames; Beam; Column; Bond; Hook

1. Modelling of reinforced concrete frames

Many models for the non-linear analysis of RC frames are proposed in the literature. They can be classified depending on the level of discretization [1] in *point by point* model, *member by member* models and *global* models. The choice of the most suitable model depends on the goals of the analysis and by the structural properties. Structures characterized by brittle mechanisms require a non-linear analysis and then the use of highly discretized models; on the other hand, for structures with flexural collapse mechanisms, *member by member* or *global* models can be used to obtain reliable predictions. A good balance between computational effort and level of reliability of the results should be achieved in choosing the model by taking into account the amount of basic information that each model requires. In the last few years, fibre models have become more and more popular. They still keep the basic hypothesis of subdividing the structure in mono-dimensional elements, even though they could be defined as a hybrid between point by point and member by member models. The constitutive laws of concrete and steel are introduced [2]; in recent versions the hypothesis of perfect bond is removed [3] and shear collapse is taken into account [4]. Some authors introduced a joint element accounts for inelastic shear deformation and bar bond slip in program DRAIN-2DX [5,6].

Manfredi and Pecce [7] proposed for beams and columns a fibre element that introduces explicitly the bond law τ–s. Limkatanyu and Spacone [8] showed that the accurate representation of the bond–slip behaviour is crucial in predicting the response of RC frames subjected to both static and dynamic loadings.

In the assessment of the seismic capacity of existing underdesigned RC structures all the brittle failure modes are potentially active, and this occurrence requires the development of a reliable numerical model in terms of behaviour and material properties [9]. In this paper a

* Corresponding author. Tel./fax: +39 081 7683491.
E-mail address: gamanfre@unina.it (G. Manfredi).

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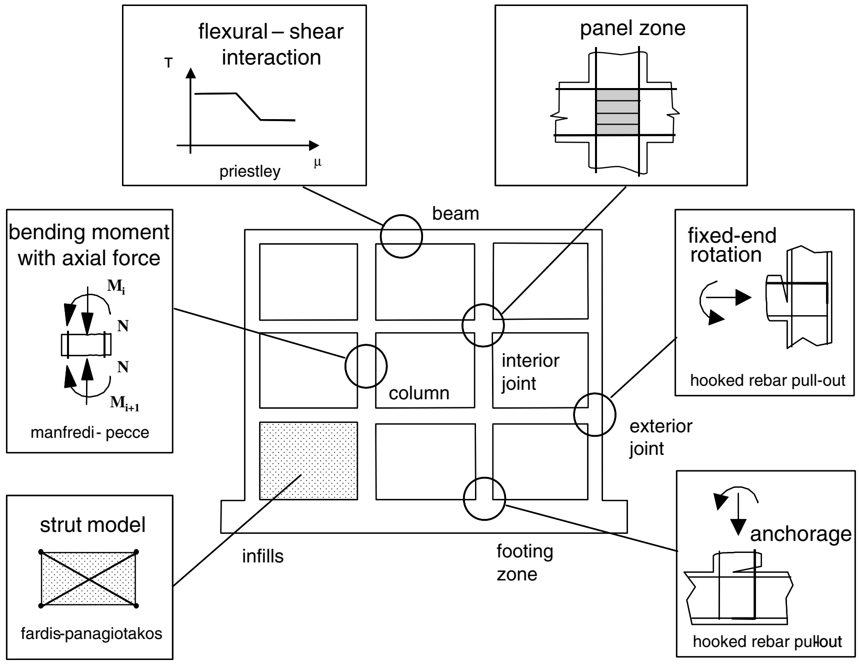

Fig. 1. The main mechanisms influencing non-linear behaviour of reinforced concrete frames.

numerical model for reinforced concrete frames is presented, which is an extension of the model proposed in Manfredi and Pecce [7]. The model is able to predict the main mechanisms influencing the non-linear behaviour of reinforced concrete frames (Fig. 1). In particular, the proposed model considers an explicit introduction of advanced constitutive bond–slip relationships that allow to describing the structural behaviour in the large post yielding field for elements under bending and axial forces and to introduce refined models for beam–column joints [13].

2. Element formulation

The beam–column element is characterized by a spread of plasticity and distributed cracking: it belongs to the fibre model family. The mechanical properties of the cross-section are evaluated by considering the constitutive laws of the materials.

The classic hypothesis of perfect bond between concrete and steel is removed and a stress–slip bond constitutive law is introduced [10]. Such an aspect allows for a more reliable assessment of the tension stiffening effect, for both elastic and plastic field, and avoids the approximations due to the assumption of the plastic hinge length.

For the column, it is possible to consider the variation of axial forces due to lateral loads, and the related effects in terms of overall strength and deformation capacity. Also, considering the axial deformation allows for a detailed simulation of the interactions between columns and infill walls.

In the beam–column joints, that plays a significant influence on the structural response, both in terms of strength (i.e., shear failure of the panel or pull-out of the rebars) and deformation (due to the cracking of the concrete and slippage of rebars), the rotation at the beam–column interface is computed taking into account either the bond between concrete and steel or the constitutive law of the hooks.

The influence of shear forces on the behaviour of the beams was modelled by Priestley et al. [11]; such a model is based on a reduction of the shear strength depending on the local ductility, as expressed in terms of linear variation of the curvature. The model introduced here represents an improvement of Priestley's since it enables the sectional ductility at any step of the analysis to be directly determined and then evaluate the shear strength of those sections located in the plastic regions. Therefore, along with predicting ductile (i.e., flexural) and brittle (i.e., shear) failures, this method allows also failures characterized by low ductility due to the bending–shear interaction to be determined.

The model for infill walls is based on the shear model by Fardis and Panagiotakos [12]. It takes into account the strength reduction due to the cracking of the panels and the post-strength degradation. It is based on four different steps: initial shear behaviour of the uncracked panel, behaviour of the cracked panel as equivalent strut, its instability after the maximum strength and final stage after complete failure characterized by constant residual strength.

2.1. The flexural model

The column is considered as a mono-dimensional element, by introducing a simplified deformation model for the cross-section, as shown in Fig. 2. As mentioned before the hypothesis of perfect bond between steel and concrete is removed. Thus, calculations in the generic cross-section