ABSTRACT: Presence of masonry infills and detailing of reinforcement can have a significant influence on the response of the older R/C frames under seismic excitation. After several major earthquakes failure of infilled frames were observed due to the presence of shear deficient columns and inadequate detailing of smooth reinforcing bars near the joints. These aspects are quite often neglected in linear or nonlinear models because of the modelling complexity and increase in computational effort. The objective of this work is to identify the consequences of bond-slip in critical zones such as plastic hinge locations and the presence of shear deficient columns. To investigate these effects on the response of a sample 2D frame, three different infilled-frame configurations are analyzed using an accurate yet viable approach based on the diagonal strut scheme (ASCE41/FEMA356): a) bare frame, b) partially infilled frame (pilotis frame) and c) uniformly infilled frame. Bond-slip effects are directly incorporated into the nonlinear fiber section models (Spacone et al. 1996) through the simplified approach proposed by Braga et al. (2012), and nonlinear shear behavior of columns is aggregated at the element level. Static pushover analyses and Incremental Dynamic Nonlinear Response History Analyses (IDA) are performed spanning a wide range of hazard levels. Results show that incorporation of infill-induced shear damage and bond slip into the frame models the results in 1) significant loss of strength for the uniformly infilled configuration; and 2) increase of flexibility for the bare-frame and partially infilled configurations.

KEY WORDS: Infilled frame, Shear failure, Bond-slip, Pushover, Incremental dynamic analysis

1 INTRODUCTION

Prior to the development of seismic building codes of practice most of the structures were designed to resist only gravity loads. This resulted in a large proportion of the building stock with inadequate reinforcement detailing, which currently represent a high risk in regions of medium and severe seismicity.

There are several complex aspects associated with the stock of older infilled frame reinforced concrete structures such as interaction of R/C frame and infill panels, presence of unreinforced joints, bond-slip or anchorage slip near the joints, flexure shear interactions in frame components, axial-flexural interaction in frame components etc. These effects can significantly change the local as well as the global response and are captured by accurate nonlinear models of structures and components.

This research work highlights the influence of the aforementioned phenomena on the seismic behavior and response of infilled frame structure focusing mainly on shear failure of column and bond-slip effects in the longitudinal reinforcements.

1.1 Shear Failure of Columns

A common cause of poor performance in infilled frame reinforced concrete structures are presence of a soft story (Figure 01a) and captive columns (Figure 01b). Several major earthquakes have also shown that severe damage and collapse may also occur in the case of uniformly infilled frames having shear deficient surrounding columns (shown in Figure 01c and Figure 01d).

Figure 01. Damage in Infilled Frame Structures due to (a) Presence of Soft story (2009 L’Aquila) (b) Presence of Captive Columns (2001 Atico earthquake, reprinted from Ayhan Irfanoglu et al 2009, [1]) (c)/(d) Interior/Exterior shear deficient column (Reprinted from Haldar et al 2013, [2])

Several researchers have addressed the issue of incorporating inelastic shear response in the assessment of

In this study, in order to simulate the localized shear failure of a column, a nonlinear shear force-deformation constitutive model is used at the section level, together with a classical fiber section (Spacke et al 1996) [12] for the axial and bending effects to make it analogous to the Timoshenko beam (Timoshenko 1970) [13]. Petrangeli et al. 1999 [14] extend the fiber section model originally developed for the section of an Euler-Bernoulli beam to the uniaxial bending section model of a Timoshenko beam, which was quite rational but computationally intensive. An alternate but more simplified approach, to capture shear deformation in RC members, was introduced by Martino & Spacone et al. 2000 [15]. In which they employed phenomenological $V-\gamma$ law which was further investigated by Marini and Spacone 2006. In this model bending forces become coupled at the element level because the equilibrium is imposed along the beam element, however shear deformations are uncoupled from flexural and axial effects in the section stiffness.

The present work generalizes the approach and extends the capacity assessment of existing frame structures in the performance-based response analysis context using fiber models [16], [17] with particular attention to shear deficient columns, infill interaction and bond slip.

1.2 Bond-slip phenomenon

Bond-slip effects are pronounced in structures with poor bond conditions, such as older concrete buildings generally reinforced with longitudinal smooth bars or inadequate detailing [18]. This effect is typically amplified by an insufficient lap splice of bars, poor bond strength of concrete, and a low confinement level [19]. Slippage becomes particularly significant under lateral loads at the interface of beam-column joint panels or at the footing of columns. Therefore, if one neglects the additional deformation resulting from bar-slip, the calculated lateral response of an RC structure may differ considerably from those obtained in experimental tests [20].

The bond-slip model adopted in this work assumes a linear slip field along the embedded bar in a concrete block and provides a simplified stress-strain relationship to assign to the longitudinal reinforcement [20]. The adopted bond-slip model is very convenient from a computational standpoint and does not require the iterative procedure of several other refined models. The displacement field and the stress-slip law were obtained through the model for both the cases shown in Figure 02. The problem of the shift from the obtained uniaxial stress-slip law ($\sigma-u$) to a uniaxial stress-strain one ($\sigma-\epsilon$), necessary for the numerical implementation of the model in common structural analysis software, can be solved in the case of lumped plastic hinges models with fiber-sections. This is done considering the axial displacements $u_a$ as the integrated axial displacement of a longitudinal bar including steel deformation and bond slip phenomena, developed along the length characterized by a constant sectional response, coinciding, in this case, with the plastic hinge length. The stress-strain relationship obtained from this approach directly incorporates the bond-slip phenomenon and can be considered as a pseudo stress-strain law of the steel reinforcement within the plastic hinge length, derived from the stress-slip law model and necessary to be implemented in the fiber-section model.

2 METHODOLOGY

This research work presents results from nonlinear static analysis and Incremental Dynamic Analysis (IDA) carried out on three different configurations by using OpenSees [21] and OpenSeesMP [22]. The ground motion records have been selected using REXEL beta (Iervolino 2010) [23]. The 14 unscaled signals are extracted from the European strong motion database by matching the Italian Code NTC08 [24], site specific target spectrum with a code-specified period range. Ground motion records are then linearly scaled over a range of sixteen hazard levels from minor to severe hazard levels by using the probabilistic hazard curve made available by INGV [25].

All non-linear elements in the model are force-based fiber-section beam-column elements. Each beam-column element has five integration points, the length of the first and last integration points are close to the plastic hinge length $L_{pl}$. Different model parameters are assigned to unconfined and confined concrete. Section shear is also modelled using the section aggregator command of OpenSees and providing a nonlinear shear constitutive law. Infill panels are modelled using truss elements. The Giuffrè-Menegotto-Pinto, [CEB1996][26], stress-strain model is used for the steel reinforcement in all cases except for the model used in the cases of bond-slip consideration, in which the modified stress-strain law is imposed on the fiber-section near the joints (Braga et al 2012). The modified Kent and Park 1971 [27],
stress–strain relationship is used for both confined and unconfined concrete fibers (with zero tensile strength). For infill panels, the backbone curve is determined according to the ASCE41 [28] provisions and assigned to bi-diagonal truss elements by using a uniaxial bilinear hysteretic material. To capture the local shear behaviour of frame members, linear and nonlinear shear laws are assigned to column elements. The mass of the structure is modelled using lumped masses at the nodes. Model masses are directly computed from the total dead load including the self-weight and the superimposed dead load. The live loads are accounted for with a 30% contribution in the model mass. The damping characteristics of the building are modelled using mass and stiffness proportional damping with 2% of the critical damping for the first two modes of vibration. The periods of these two modes are estimated from the eigensolution using the initial elastic stiffness matrix.

Newmark–β method is used as the time integrator with coefficients γ=0.50, β=0.25 and a time step of 0.01. For solving the nonlinear equilibrium equations the Newton–Raphson solution algorithm is used. Other algorithms may be used depending on the convergence of the solution. Peak floor acceleration and interstory drift are used as Engineering Demand Parameters (EDP). For the IDA probabilistic characterization of the response the variation of the input Intensity Measure (IM) is expressed in terms of the peak ground acceleration (PGA) of the code-reference UHS on which the coherence of the set of GMs is assessed.

3 CASE STUDY (2D FRAMES)

This work is focused on structures designed according to pre-1970s Italian codes, which commonly exhibit a number of deficiencies related to design for vertical loads only, inadequate confinement in the areas of potential formation of plastic hinges, insufficient transverse reinforcement in the nodal regions, inadequate detailing of both longitudinal and transverse reinforcement, low concrete strength.

The objective is to identify the effects of inelastic shear law on column and of bond-slip phenomena, therefore the case study frames (shown in Figure 03 and Figure 04) are categorized into four cases: 1) frame with linear/elastic shear law [designated as BF/UIF/PIF], 2) frame with nonlinear/elastic shear law [designated as BF(Is)/UIF(Is)/PIF(Is)], 3) frame with bond-slip [designated as BF(bs)/UIF(bs)/PIF(bs)], and; 4) frame with bond-slip and nonlinear/elastic shear law [designated as BF(bs+Is)/UIF(bs+Is)/PIF(bs+Is)].

![Figure 03. Concentric Strut Scheme with Elastic Shear Law](image)

![Figure 04. Eccentric Strut Scheme with Inelastic Shear Law](image)

4 NUMERICAL RESULTS

Results from nonlinear static analysis are shown in Figure 05, where the demand is represented based on the N2-method [29], deterministic approach for three different limit states: damage limit state (PE 63%); life safety limit state (PE 10%); and near collapse limit state (PE 5%) are imposed on all configurations. By comparing Figure 05a and Figure 05b it is observed that in the configurations of bare frame and partially infilled frame accounting for the nonlinear shear law in columns does not influence the response prior to attainment of the maximum strength.

The limited shear capacity of the columns becomes influential for the uniformly infilled frame configuration where shear failure of the edge columns occurs prior to the failure of infill panels. This results in a considerable decrease in the maximum strength. However, by imposing pseudo stress-strain law of steel for longitudinal bars near the joints considerable softening can be clearly noticed in the bare and partially infilled frame, while the response of uniformly infilled frame remains unchanged, as it is shown in Figure 05c. No significant changes are observed assigning these two phenomena/deficiencies simultaneously, as it is shown in Figure 05d, which means that for this particular base-case frame, the response of the uniformly infilled frame is not affected by bond-slip. On the other hand, the configurations
with bare and partially infilled frame are not affected by the limited shear capacity of column.

Response of frames with bond-slip [BF(bs)/UIF(bs)/PIF(bs)] (d) Response of frames with bond-slip and nonlinear/inelastic shear law [BF(bs+Is)/UIF(bs+Is)/PIF(bs+Is)]

Results are also presented in the form of IDA capacity curves shown in Figure 06, which are obtained from the IDA curves by assuming the reference spectrum PGA as IM. IDA results are processed in terms of the mean of the max values of the EDPs peak maximum interstory drift ratio (max IDRmax), maximum peak floor acceleration (max PFA) and maximum base shear (max base shear). Good agreement is generally found between the IDA capacity curves and the pushover capacity curves in terms of the maximum capacity and the initial stiffness for all the configurations, except for the bare frame configuration probably due to the multi-modal contribution which is captured by IDA (Mohammad et al. 2013) [30]. These curves give a measure of the dynamic capacity of the structure obtained through the IDA at different hazard levels. The demand obtained for IDA at 63%, 10% and 5% in 50 years probability of exceedance, are highlighted to compare the structural capacity at the code-mandated performance limits. Similar observations can be made regarding the higher impact of shear strength in the short columns only for the uniformly infilled model at the higher hazard levels. Similar to what observed in the pushover analysis, the partially infilled and bare frame configurations are not significantly affected by the limited shear capacity of the column but become flexible due to the bond-slip effect near the joints. For the sake of comparison, the maximum roof drift demand obtained from IDA is also superimposed on conventional pushover curves for three distinct hazard levels, as it is shown in Figure 05.

Figure 05. Pushover curves with N2 and IDA based demands (Green Curves for Bare Frame, Red Curves for Partially Infilled Frame and Blue Curves for Uniformly Infilled Frame) (a) Response of frames with linear/elastic shear law [BF/UIF/PIF] (b) Response of frames with nonlinear/Inelastic shear law [BF(Is)/UIF(Is)/PIF(Is)] (c) Response of frames with bond-slip [BF(bs)/UIF(bs)/PIF(bs)] (d) Response of frames with bond-slip and nonlinear/inelastic shear law [BF(bs+Is)/UIF(bs+Is)/PIF(bs+Is)]

![Figure 05. Pushover curves with N2 and IDA based demands](image_url)

Figure 06. IDA capacity curves (a) Response of frames with linear/elastic shear law [BF/UIF/PIF] (b) Response of frames with nonlinear/Inelastic shear law [BF(Is)/UIF(Is)/PIF(Is)] (c) Ratios of Max IDRmax

![Figure 06. IDA capacity curves](image_url)
Figure 07 to Figure 09 show the impact of the NL shear behavior and of the bond-slip effect in the different models by taking the ratio of the max EDPs from case to case and over the range of hazard levels covered. It is evident how the response of the bare frame model and of the partially infilled model is not affected by the shear behavior, neither in terms of drift, acceleration nor base shear. Instead, in the case of uniformly infilled frame there is a significant influence of the shear behavior on the overall drift response. The shear behavior is activated from hazard levels higher than the 63%PE in 50 years, and results in drift amplifications in the order of 2 to 2.3 for PGA intensities in the range of 10% to 5%PE in 50 years. While bond-slip effects result in decreased maximum peak floor acceleration and maximum base shear of 10-20% and increase in the maximum interstory drift ratio of 5-15% for the bare and partially infilled frames.

5 CONCLUSIONS

This research work focused on the effects of frame-infill interaction-induced shear failure of columns and of bond slip effects in proximity of the beam-column joints for older infilled frame building structures by using simplified models. In general several approaches are available ranging from simplified empirical to refined finite elements models. A frame structure was considered, which was designed for gravity loads only and was conceived as representative of older construction without sufficient earthquake resistance.

The numerical results from pushover and IDA analyses demonstrate the importance of bond-slip and shear failure of the columns for seismic assessment of existing building structures. It is observed that shear failure of columns prior to infills failure decreases the strength of uniformly infilled frame while other configurations are not significantly affected by this nonlinear aspect. The flexural failure mode of columns still predominates in the case of bare frame and partially infilled frame and incorporation of bond-slip effects increases the flexibility of these configurations.

Current developments of this study are focusing on the incorporation of simplified models of the beam-column joint panel flexibility and strength and on the assessment of the impact of this additional mechanism on the system response.

REFERENCES


