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INFLUENCE OF MODELLING PARAMETERS ON THE FRAGILITY ASSESSMENT OF PRE-1970 ITALIAN RC STRUCTURES

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Abstract. *This paper investigates the importance of numerical modelling decisions on the seismic assessment of reinforced concrete (RC) structures typical of construction practice in Italy prior to the introduction of seismic design codes in the 1970's. In particular, means of accounting for the formation of a shear hinge in the beam-columns, longitudinal reinforcement slippage due to the use of smooth bars with end-hooks, and the strength and stiffness degradation associated with the lapping of longitudinal reinforcement in beam and column member end zones are discussed. A numerical model is validated by developing existing models in the literature capable of representing such behaviour and comparing with results obtained from quasi-static cyclic testing of a three storey RC frame. A case study structure designed using design provisions employed in Italy prior to the 1970's is then analysed via a series of dynamic analyses using ground motions representative of seismic hazard in Italy. An assessment of the structures is conducted at different return periods to determine the median storey drift and peak floor accelerations, as these demand parameters are typically used for refined performance assessment studies. For the considered structure, a number of cases are considered, where various modelling parameters are included or excluded to highlight the influence of each of these on the overall response of the structure. From this study, the importance of different numerical modelling decisions on the seismic assessment of pre-1970s' RC construction in Italy is highlighted and recommendations are provided for the modelling of phenomena critical to the seismic performance assessment process.*

1 INTRODUCTION

Much focus has been placed on developing methods of assessing a structure's performance to a given seismic hazard. These range from simplified pushover analyses to more rigorous numerical analysis methods. The most comprehensive existing approach for the seismic assessment of structures is what is termed the PEER PBEE methodology [1], illustrated in Figure 1, where a convolution integral of site hazard, structural response, damage and loss analysis give a rigorous approach to the seismic assessment of structures. The drawback of such a methodology is the level of numerical analysis required to determine the desired decision variables. As such, more simplified methods of analysis have been developed for RC structures [2] in order to provide the same information, but at a fraction of the computational effort and within an acceptable tolerance, which can then assist designers in quickly establishing effective retrofit solutions for structures, as demonstrated by [3].

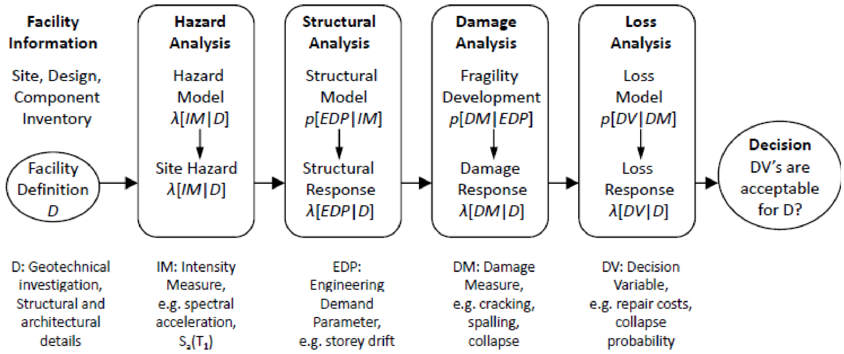


Figure 1: Overview of the PEER PBEE methodology (after [1]**).**

While more simplified approaches to seismic assessment represent an obvious advantage from a practice point of view, there appears to be limited guidance for the use of approaches for the assessment of structures designed prior the introduction of seismic design codes in the 1970's. A number of inherent assumptions regarding the structural response and strength hierarchy in modern structures do not apply to older gravity-only designed structures. In order to develop more simplified methods of seismic assessment for older RC structures, the identification of the relative importance of different possible mechanisms associated with older RC structures in Italy is of concern. As such, this paper discusses the principle behaviour of gravity load designed RC frame structures constructed in Italy prior to the introduction of seismic codes in the 1970's. Numerical modelling of such behaviour is discussed followed by identification of different considerations in relation to the modelling of various parameters. A 3-storey gravity load designed case study structure is then assessed for different levels of seismic hazard and assessed in terms of its roof drift and peak floor accelerations, which are typical demand parameters used in damage analysis step of Figure 1, to establish the relative importance of the various modelling considerations. As a result, this work highlights the importance of accounting for the numerical modelling of different behaviour mechanisms of older Italian RC frame structures.

2 RELEVANCE OF MODELLING IN SEISMIC ASSESSMENT OF RC STRUCTURES

The influence of joint behaviour on the seismic response of frame structures has been the subject of much discussion, and concerns have been raised about the cases where no flexibility or nonlinear behaviour of the joint is considered in numerical modelling. It has been long recognised that such an assumption of rigid joint behaviour is grossly inadequate, with Paulay & Priestley [4] noting that as much as 20% of the interstorey deflection results from the deformation in the joints alone. As such, numerical modelling has moved to consider this behaviour in analyses with various joint models being proposed, such as that by [5]. However, models from such studies tend to typically consider a joint that has been well detailed with deformed reinforcing bars according to modern design codes. Pampanin et al. [6] noted that RC structures constructed in Italy prior to the introduction of seismic design codes resulted in the absence of shear reinforcement in the joint region, which resulted in a shear hinge mechanism forming in the joint. Accounting for this kind of behaviour specific to Italian construction has been the focus of much work recently, with Sharma et al. [7] providing a review into existing approaches for accounting for joint behaviour. In addition, Shafaei et al. [8] conducted a study on the modelling pre-1970 Iranian RC structures where it was shown that the consideration of joint behaviour considerably affects the lateral response of the structure, in addition to similar work by Jeon et al. [9] for older RC structures in the US where Jeon et al. [9] reported similar findings regarding the importance for proper joint modelling. Therefore, it is imperative that such considerations be developed for older Italian RC construction so that seismic assessments may be carried out using numerical models that are representative in terms of the additional deformability and nonlinear behaviour characteristic of older RC frames.

In addition to consideration of the joint behaviour in older structures, the behaviour of the frame members is also of utmost importance. Experimental testing on members detailed with plain reinforcing bars and inadequate core confinement has shown that the hysteretic behaviour of these members differs quite a lot compared to well detailed ductile members, where the presence of plain bars tends to lead to a lot of hysteretic pinching between inelastic cycles [10–12]. In addition, inadequate confinement of the core concrete and large spacing of the stirrups limits the ductility capacity of older RC members. Studies on the influence of frame member behaviour by [13,14] showed the influence of considering ductile and non-ductile detailing of members to be quite significant on the collapse capacity of RC structures in the US. In addition to affecting the hysteretic properties of frame members, the presence of plain bars can result in a reduction of flexural capacity of the members, where Calvi et al. [15] demonstrates that this reduction can typically be of the order of 20%, depending on the axial loading and bond conditions of the reinforcement. Hence, such a reduction in flexural capacity particular to older Italian RC frame should be taken into consideration when conducting seismic assessments.

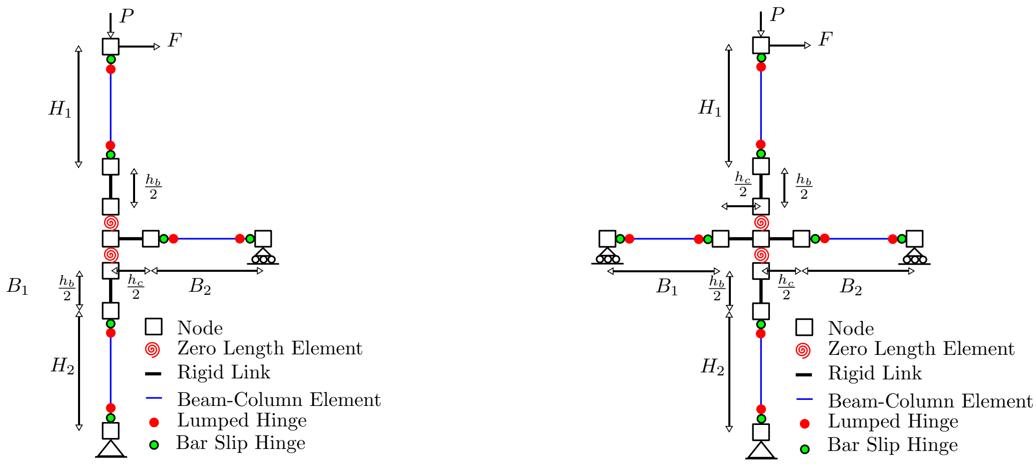
Welch et al. [16] investigated the impacts of some modelling decisions on the loss assessment of ductile RC frames. Between different considerations of initial stiffness calculation, post yield hardening ratio and elastic damping, the parameter that was shown to most heavily influence the loss assessment results was the consideration of elastic damping. Hence, the quantification of such an affect in relation to other modelling parameters considered in this study and those done by others is warranted. In addition to the response of RC frame members, the consideration of masonry infill contribution to the behaviour of RC frames is an important consideration, with [17] (amongst others) noting that the presence of masonry infills considerably modifies the strength and stiffness of the frames and ought to be considered in the numerical modelling of such structures for seismic assessment.

3 NUMERICAL MODELLING OF PRE-1970'S RC STRUCTURES

The numerical modelling of structures examined here is carried out using the OpenSees framework [18]. The model consists of a number of parameters that ought to be considered for the modelling of pre-1970's Italian RC frames such as the aforementioned joint behaviour, frame members and the masonry infills. The numerical model considers both material and geometrical nonlinearity and models the cyclic degradation of both the joint and frame elements. The following sections discuss the particular details regarding each of the modelling parameters, followed by the comparison of the numerical model with the experimental test results of a three storey gravity load designed RC frame.

3.1 Beam-Column Joint Modelling

The joint modelling of RC frames is based on previous efforts by [19,20], where the joint is modelled via a number of rigid links in the joint along with rotational springs to represent the shear response of the joint. Figure 2 shows the layout of both the interior and exterior joints, where the layout of rigid links is illustrated along with the location of the rotational springs. The properties of these springs are determined based on the consideration of the principle tensile stresses developed in the joint during shear deformation, as discussed by [19]. The upper and lower rotational springs are then slaved together in rotation and linked in the vertical direction by an axial spring with the same axial stiffness as the column section. The layout of the nodes and elements in the joint model are as per previous models, such as [20], although the hysteretic materials have been modified. The Hysteretic material rule available in OpenSees has been adopted to represent the shear behaviour of the connections, where the hysteretic parameters have been matched based on existing experimental results. For the case of exterior joints, experimental testing results from [6,21–23] were used, whereas for interior joints testing by [6,22–28] were considered. Using this database of test results, hysteretic parameters that govern the pinching behaviour and stiffness degradation of the joint have been established. In addition, the cyclic degradation parameters of the joint have also been matched to those observed in testing to give a joint model that not only represents the strength and stiffness of the joint, but also represents the hysteretic behaviour and cyclic degradation.



(a) Exterior Joint **(b) Interior Joint**

Figure 2: Joint Modelling.

3.2 Beam-Column Member Modelling

For the modelling of the frame members, a lumped plasticity beam column element is employed via the beamWithHinges element available in OpenSees. This allows the definition of the hysteretic behaviour of the member end zones with a specified plastic hinge length and an internal elastic section with cracked section properties, as shown in Figure 2. The hysteretic model used for the plastic hinges is the Pinching4 material model available in OpenSees, which allows for the specification of a number of parameters to define the pinching behaviour of the multi-linear backbone curve. These parameters are determined from comparison with experimental test data provided in [10–12,29,30] members with plain reinforcing bars and poor core confinement. The backbone of the hysteresis rule is determined from consideration of the section flexural capacity with a reduction to account for the bar slippage due to plain bars, as outlined by [15]. The yield rotation specified in the hinge region is determined based on flexural deformations, where the additional flexibility due to the slippage of the longitudinal bars is modelled separately and is discussed further in Section 3.3.

The plastic hinge length of the member end zones is determined as per the empirical expression given in [4]. However, [31] noted that this expression for plastic hinge length was developed based on testing of well detailed ductile members that can distribute plasticity along member ends, whereas testing on members with plain bars and poor core confinement has shown that these plain bar members tend to concentrate the inelasticity in the member ends. However, of the available test information of older RC members, none of the test campaigns appear to report an observed plastic hinge length with the exception of [32], where comparing the observed plastic hinge length of an RC beam with plain bars with the Park and Priestley expression is quite good and is therefore used here. However, experimental evidence suggests that this may not always be the case and is hence recommended for future work.

In addition to modelling the behaviour of poorly detailed RC frame members, this study considers the possibility that the numerical model is developed using hysteretic parameters proposed for well-detailed ductile members. The ductile RC beam column member used in this study is from the calibration study by [33]. This considers a wide range of tests to propose a set of recommended parameters for the hysteretic behaviour of the beam column element using the modified Ibarra Medina Krawinkler material model available in OpenSees.

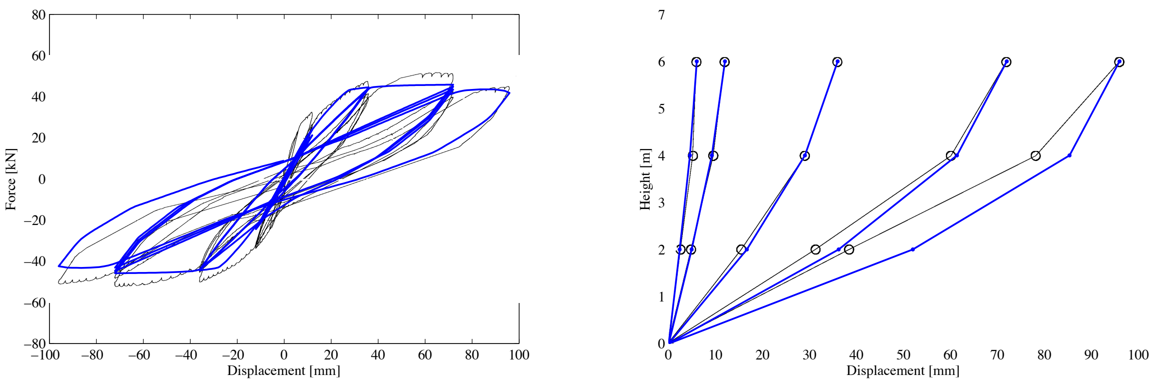
3.3 Bar Slippage

The slippage of the longitudinal bars affects the beam and column member hysteretic response in two ways. First, the slippage of the bars at the member ends results in an additional end-flexibility, therefore increasing the member's yield rotation. Metelli et al. [34] proposed a set of guidelines to determine the additional flexibility of the member ends in old RC frames considering the poor bond conditions of the plain reinforcing bars and this was incorporated into the model considered here via an additional spring at the member ends. This approach is adopted here, where the behaviour due to the slippage of the bars is added to the member ends, as illustrated in Figure 2.

In addition to the increase in member end-flexibility, the presence of plain bars with poor bond conditions can also affect the flexural capacity of the member sections. Calvi et al. [15] discuss how the flexural capacity can be reduced if the lack of a perfect bond between the bars and the joint concrete introduces a state of tension in bars that would ideally act as the compression reinforcement, therefore reducing the flexural capacity as a result of the rebalancing of the section forces. This reduction in flexural capacity was noted [15] to be a function of both axial load ratio and bond conditions, where typical reductions can be between 20 and 30% for column members. In order to consider the affect of the bar slip on the beam and column flexural capacities, the charts provided by [15] are used as a guide here considering the expected axial load ratio for the case study structure presented in Figure 4, where a reduction of 20% for columns and 5% for beams is used here, the difference between the relative reductions stems from the presence of axial load in the member. This reduction in strength is approximated from the charts provided by [15], but future work should consider a more detailed approach to quantifying this reduction. Since the purpose of this study is to determine the relative influence of modelling parameters, this approximation is deemed sufficient for the case study presented herein.

3.4 Validation of Numerical Model for pre-1970 Italian RC Structures

The above modelling parameters are combined to compare the response of the numerical model to that of an experimental test on a 2/3 scale RC frame designed for gravity loads only and detailed as per pre-1970's Italian concrete construction practice. Calvi et al. [35] outline the testing of a 3-storey RC frame and report the response to quasi-static pushover cycles of increasing amplitude. Figure 3(a) shows the quasi-static cyclic response of the frame along with that obtained from cyclic pushover analyses using the numerical model developed in this study. It is apparent that the numerical model and experimental results match very well. This is especially seen through the initial stiffness and lateral capacity of the model. In addition, the hysteretic behaviour of the numerical model matches that of the test results very well also. In addition, the displaced shape shown in Figure 3(b) is matched reasonably well with that reported in the tests, where the shear mechanism in the joint at the first floor is captured by the model. This represents an improvement of the simplified modelling of pre-1970's Italian RC frames when compared to previous efforts such as [20], where lateral capacity and hysteretic comparisons were reasonable.



(a) Cyclic Pushover Curve **(b) Displaced Shape**

Figure 3: Comparison of numerical model with 3-storey RC frame tested by [35].

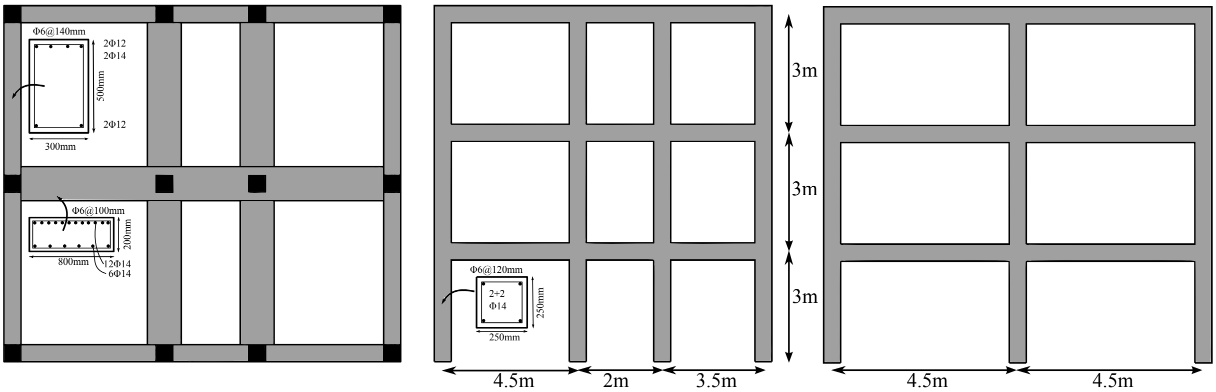
3.5 Masonry Infill Modelling

Masonry infills in the case study structure are modelled using the equivalent strut model initially proposed by [36], which consists of modelling the infill through a series of rigid offsets and a single symmetrical hysteretic behaviour. This model was then further developed and incorporated into OpenSees by [37] where calibrations were performed using experimental test data provided in [38,39]. In order to calculate the equivalent strut's hysteretic parameters, expression for first cracking, yield and ultimate lateral force of the infill wall given in [17] are used. The parameters governing the reloading and pinching behaviour of the equivalent strut model are calibrated based on experimental test observations, but agree well with previous parameters identified in [40]. To account for openings such as door and windows in the masonry infill, [37] proposes the force capacity reduction parameters outlined in [17] and comparisons with testing [39] on a number of specimens with openings showed good agreement. The advantage of using such a simple macro element model such as the one employed here is its simplicity and computational efficiency. In addition, the degradation of the lateral resistance of the infill is no longer uncoupled in either direction, where other diagonal strut models possess a single compression-only strut for each direction. This approach means that one strut may have exhausted its capacity resulting in infill collapse, whereas the other strut will not recognise this loss of strength and continue to provide resistance in the other direction. Crisafulli et al. [41] provides a review into different approaches to modelling masonry infills; where modelling is varied in complexity from a detailed finite element modelling approach down to a single strut model, such as that just outlined. It was concluded that while simple strut models were not so accurate in the distribution of loads at the corners to the surrounding frames, the overall model captures the principle behaviour of the masonry quite well. This conclusion is also mentioned by [17], where the global response of the structure is captured by such models and thus is considered reasonable for studies considered here.

4 CASE STUDY STRUCTURE

The building typology used in this study is taken from a previous study by [42]. The structure consists of a 3-storey RC frame designed for gravity only loading as per Regio Decreto 2229/39 [43], which was typical of Italian construction prior to the introduction of seismic design code provisions in the 1970's. The basic layout of the structure is shown in Figure 4 where the structure possesses three bays in the longitudinal direction and two in the transverse. The structure contains two different types of beam section sizes; exterior beams with 500mm deep sections and shallow interior beams adopted at the time to avoid impeding on ceiling space. Seismic assessment and retrofit of a similar 6-storey version of the building has also recently been reported in [44].

Construction materials at the time of construction were somewhat different to those used in modern ductile seismic design where the cylindrical compressive strength of concrete was prescribed to be at least 12MPa at 28 days and allowable stress equal to one third of the characteristic value was used in design. Reinforcement consisted of plain smooth reinforcement with hook-ended anchorage terminated in exterior joints. The shear reinforcement consisted of stirrups at a minimum spacing and closed at 90°. Since a stirrup closing of 135° is typically required for effective confining of the concrete core, the cores of the beams and columns are considered effectively unconfined. Since the structure is designed for gravity loading only, the beam members are the same section size at each level, where the reinforcement for both beam types is shown in Figure 4(a). Despite the column design forces increasing down the height of the structure, the same section size and detailing are used throughout the entire building, which was typical of construction practice and these column section details are shown in Figure 4(b). Although not shown in Figure 4, the masonry infill considered in this structure consists of a 100mm weak clay masonry infill, which has a diagonal cracking strength of 0.55MPa.



(a) Plan Layout **(b) Front Elevation** **(c) Side View**

Figure 4: Case study structure (after [42]).

5 VARIATIONS IN MODELLING APPROACH

Using the various modelling parameters outlined in Section 3, a number of variations in the structural model are considered for the sensitivity study conducted in Section 7. These can be divided into two main groups of structures - those containing masonry infills and those not. The rationale behind modelling structures without infills is two-fold. Firstly, the structures are modelled as bare RC frames as this is was the anticipated structural configuration at the time of design, where masonry infills were considered a non-structural element that did not contribute to the strength of the system. Although this is now widely known in the field to be wholly untrue, its implications on RC frame response relative to other permutations are highlighted here considering that there may cases where infills are not present. Secondly, the purpose of this study is to demonstrate the influence of the modelling decisions on the response of the RC frame. The combination of stiffening infills and relatively low seismicity results in a limited amount of ductility in the infill structure and therefore, the affects of these modelling decisions are not observed so easily at lower intensity levels.

Considering the two principle variations in structural modelling, a number of parameters are then varied in order to establish their influence on the response of the case study structure. These are chosen in reference to the discussion presented in Section 2 and are listed as follows:

- Reference Model - This consists of the modelling of both the bare and infilled frame with all of the expected response characteristics outlined in Section 3 fully modelled.
- Ductile Members - This changes the beam-column member modelling approach, which has been calibrated for non-seismically detailed RC frame members and replaces it with an equivalent seismically detailed element as per the parameters outlined in [33]. This comparison is intended to highlight the effect of having beam-column members with ductile detailing as opposed to older member with plain reinforcement and unclosed stirrups. In addition the effects of bond slippage on the flexural strength are not modelled in the ductile member.
- No Bar Slip - In this variation, the additional beam and column end flexibility due to slippage of the plain bars is omitted in addition to the negligence of the apparent reduction of flexural strength of the members. This is intended to highlight the importance of properly addressing the influence of plain reinforcing bars with poor bond condition compared to those with a perfect bond and no slippage.

- No Joint Detail - This modelling deficiency consists of removing the rigid offsets and shear hinges at the joint centreline to the beam and columns faces, meaning that any joint flexibility is only provided via the apparent lengthening of the beam and column elements. Omitting the modelling of the shear joint behaviour is of particular interest here, since it is considered an important characteristic behaviour of RC joints susceptible to forming a shear mechanism, such as those constructed in Italy and elsewhere prior to the 1970's.
- Elastic Damping - The variation of the elastic damping is performed as this has been previously shown to significantly affect the loss assessment in ductile RC frames by [16]. Hence, this is an important parameter that ought to be properly justified and its significance to the response of older RC frames is investigated here.

Considering the above comments on the parameters to be modelled and omitted in the various case study permutations, a total of eight structures are therefore analysed. For each of these case study structures, the modelling parameters outlined in Section 3 are utilised to create the numerical models used for the analysis. From each of these models, the first three periods of vibration in the longitudinal direction are reported in Table 1. From these first mode periods, it can seen already that the small changes in modelling have had a noticeable affect on the initial period of the structure for the vase of the bare frame, whereas the infilled frame remains relatively unchanged between variations. These initial periods are then used to determine a suitable set of conditional mean spectrum ground motions in order to carry out the sensitivity study.

Table 1: Periods of case study structures.

	Bare Frame				Infill Frame			
Mode	Full	Ductile Members	No Bar Slip	No Joint	Full	Ductile Members	No Bar Slip	No Joint
1	1.49	1.36	1.34	1.66	0.23	0.23	0.23	0.23
2	0.52	0.47	0.46	0.58	0.08	0.08	0.08	0.08
3	0.32	0.29	0.28	0.38	0.05	0.05	0.05	0.06

6 GROUND MOTIONS

The seismic hazard considered in this study was that of a site in Napoli, using a catalogue of ground motions chosen for the site by [45]. The soil type considered in the hazard information used in [45] was hard rock. Using the disaggregation information available for the site shown in Figure 5, [45] proposed a set of conditional mean spectra for a range of return periods (T_R) and conditioning periods (T*). For each return period considered, a set of 30 earthquake ground motion components were selected to match the conditional mean spectrum in terms of their mean component and corresponding epsilon value [46].

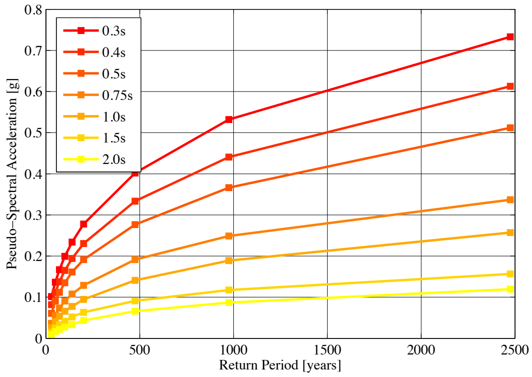
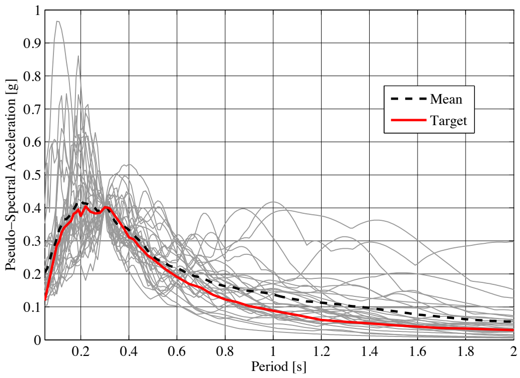
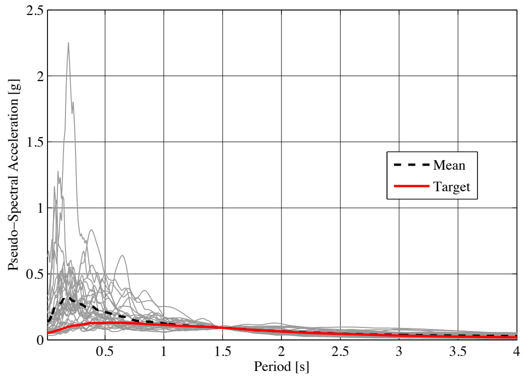


Figure 5: Seismic hazard information for Napoli site (after [45]**).**

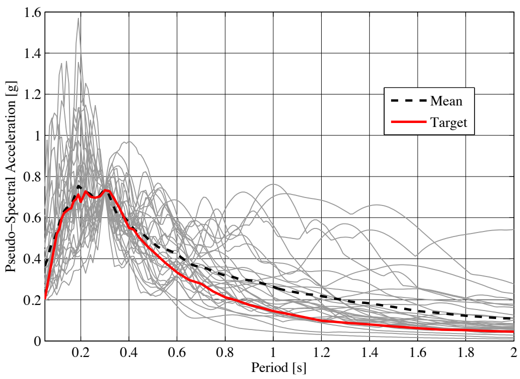
Considering the results of the modal analyses of the case study structure variations shown in Table 1, two sets of conditional mean spectra are selected for the current study; those conditioned to T*=1.5s for the bare frame cases and those conditioned to T*=0.3s for the infilled frame cases. The resulting ground motion sets for the 475 and 2475 year return periods are shown in Figure 6 along with the mean value of both the selected ground motions and the available site hazard information. Since none of the structures in Table 1 have a fundamental period that matches the conditioning period of the available ground motion sets, a degree of dispersion is to be expected in all cases as a result, regardless of nonlinear behaviour of the actual structure.



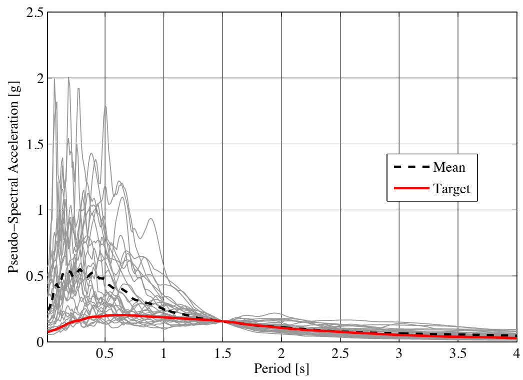
(a) 475 year return period with T*=0.3s



(b) 475 year return period with T*=1.5s



(c) 2475 year return period with T*=0.3s



(d) 2475 year return period with T*=1.5s

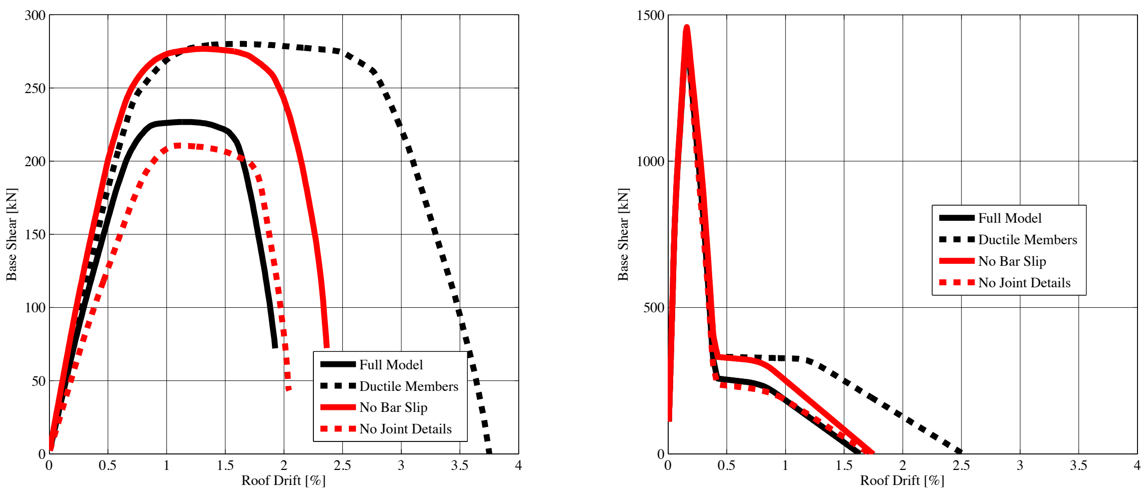
Figure 6: Selected ground motion record sets (after [45]**).**

7 ANALYSIS RESULTS

For each of the variations in the numerical model for the case study structure outlined in Section 5, each of the ground motion sets at both return periods are run using the numerical model to determine the median response parameters and hence examine the sensitivity of the modelling variations of demand parameters of principle interest, namely the storey drifts and peak floor accelerations observed. A pushover analysis is first conducted for each of the structures to illustrate the difference in backbone curve response between model variations, followed by a small discussion into the assumptions for the modelling of the structure for dynamic response and the various assumptions made. The results of the sensitivity study are then presented for each of the modelling variations for both return periods considered.

7.1 Pushover Analysis

Using the modelling outlined in Section 3, a displacement-controlled pushover is performed on each of the configurations of the case study structure using a linear lateral load distribution. Figure 7 shows the pushovers of both the bare and infilled frame with the various modelling approaches. Considering the bare frame in Figure 7(a), the affects of excluding the joint modelling are immediately obvious as the flexibility increases suggesting the joints are stiffer than the equivalent beam elements in addition to a small drop in lateral capacity due t the increase in the span of the beam and column elements. The absence of the bar slip force reduction results in an expected increase in both lateral capacity and stiffness, in addition to the consideration of ductile frame members. The use of ductile frame members also has the anticipated effect of increasing the ductility capacity of the frame by almost double that of the fully modelled frame. Regarding the infilled frame, it is immediately obvious from Figure 7(b) that the presence of masonry infills results in a large increase in lateral capacity compared to that of the corresponding bare frame configuration. Figure 7(b) also shows how the presence of the masonry infill is dominating the initial stiffness, which was already seen through the observed initial periods in Table 1, and the difference between model configurations is not noticeable until the lateral capacity of the infill has been exhausted resulting in the remaining resistance being provided by the bare frame.



(a) Bare frame **(b) Infill Frame**

Figure 7: Pushover analysis.

7.2 Dynamic Analysis Modelling

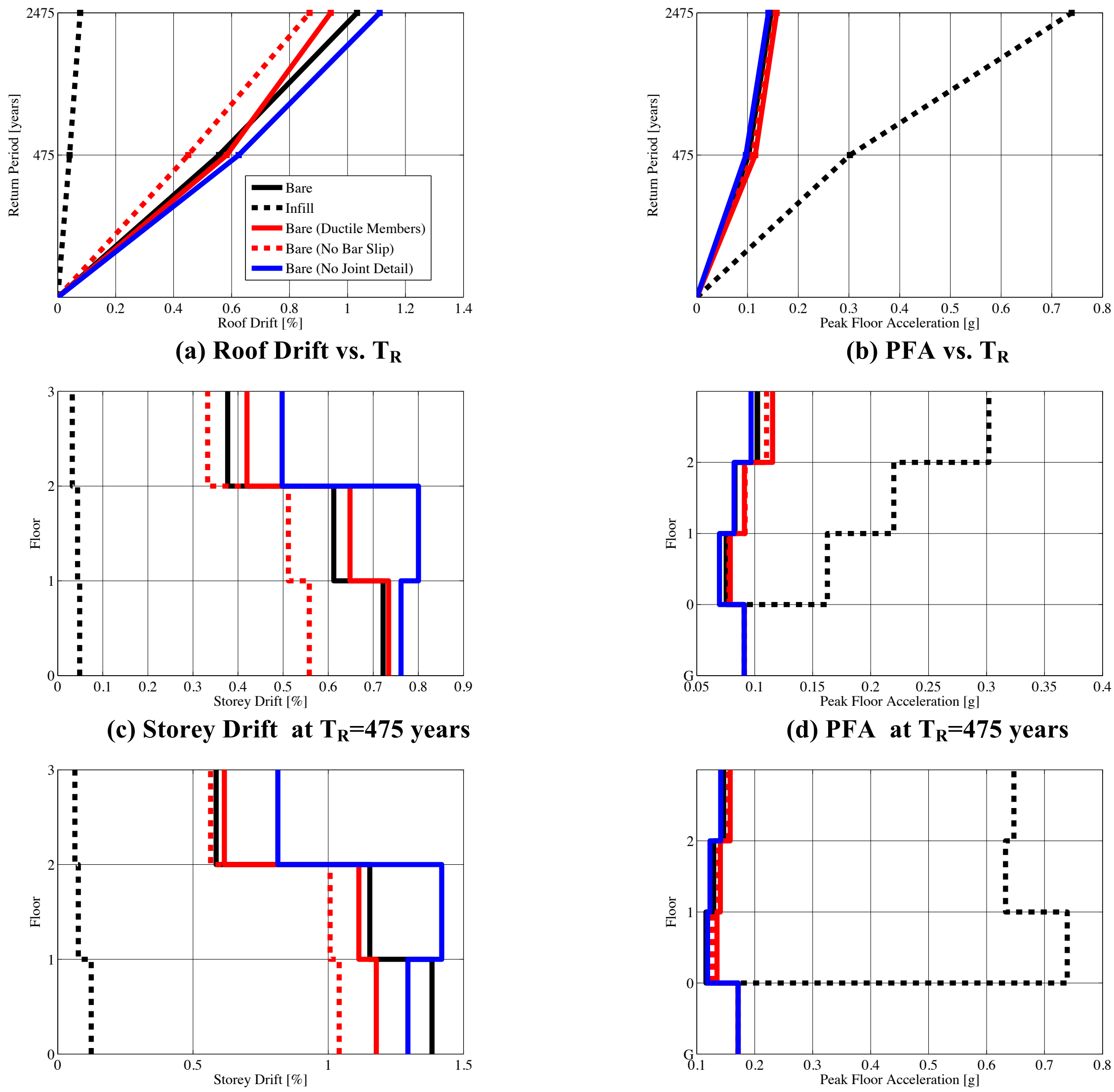
For dynamic analysis, the tributary mass was modelled as a lumped mass at each column centreline node and the corresponding gravity load applied also. A gravity analysis was carried out before each record, whereby the vertical loads were applied and maintained throughout the dynamic analysis to capture second order effects. For the dynamic analysis, an initial time-step of 0.005s was used with a Newmark average acceleration method integrator along with an Energy Increment test tolerance of 1e-8. If the analysis failed to converge after 100 iterations, a sequence of time-step reductions are used for that point of the analysis, where the tolerance is reduced to a minimum of 1e-6 with 500 iterations. Further details on these parameters can be found at [18]. The analysis was carried out in the longitudinal direction and no transverse excitation was applied. For the reference case study structure, 5% tangent stiffness proportional Rayleigh damping was applied to the first and third modes of longitudinal response of the structure, as per recommendations by [47].

7.3 Dynamic Analysis Results

Figure 8 to Figure 11 show the median response values for each of the 30 ground motions of the two considered return periods for each numerical model variation. The first comparison is between the bare frame model, which used the record set at T*=1.5s, whereas the infilled frame was carried out using the T*=0.3s set. In addition, the affects of the elastic damping are shown for both the infilled and bare frames in Section 7.3.3.

7.3.1. Bare Frame Structure

Figure 8 shows the plots of the median storey drifts and peak floor accelerations (PFA) for each of the numerical model variations at both seismic intensity levels. In order to obtain a meaningful comparison between the infilled and bare frame structure, the infilled frame model was also run using the T*=1.5s records and its response included with the bare frame results. From Figure 8 it can be seen that the lack of any joint modelling details results in larger storey and roof at both intensity levels. Although the PFA appears to have reduced slightly with the lack of joint modelling, compared to the increase in PFA caused by the presence of masonry infills it is seen to be relatively small. The addition of the masonry infills is by fare the most influential parameter, with a significant decrease in storey drift and increase in PFA with respect to the bare frame case. This is seen through the amplification of the PFA throughout the height of the structure compared to the relative little amplification of PFA in the bare frame cases. This highlights the fact that the modelling of masonry infills is essential in the seismic assessment of structures and the influence of global structural response is significant, as opposed to considering the infill damage to be solely as a function of the bare frame storey drifts.



(a) Roof Drift vs. T_R

(b) PFA vs. T_R

(c) Storey Drift at T_R=475 years

(d) PFA at T_R=475 years

(d) Storey Drift at T_R=2475 years

(e) PFA at T_R=2475 years

Figure 8: Bare frame structure response.

Comparing the observed median roof drift at the T_R=2475 year intensity with the pushover curve in Figure 7 indicates that the majority of bare frame models have just yielded and exhibit a relatively low ductility. This is more apparent for the case of the infilled frame, where a predominantly elastic response is observed at both intensity levels. Examining the response of the models which considered the ductile members and neglected bar slip with the reference model it can be seen that the use of ductile members does not represent a significant change in response compared to that if the model lacking any bar slip consideration, where the increased flexural capacity of the members results in a noticeable difference in the response. This highlights the importance of the proper consideration of the effects of bar slippage on both the increased member flexibility and the reduction of frame member capacity.

Overall, the difference between some of the results is not particularly large, especially comparing the case of ductile beam and columns with non-ductile member modelling. This is because the principle difference in the two modelling approaches was in the ductility capacity of the structures, with the ductile members model having a much larger ductility capacity.

Since the ductility demand on these structures is relatively low, this difference is not so apparent in the plots of Figure 8. However, if the seismic intensity was incrementally increased (i.e. incremental dynamic analysis [48]), this difference in behaviour would be more apparent in a study such as collapse fragility, where the median collapse drift would be expected to be much higher than that of the non-ductile member model.

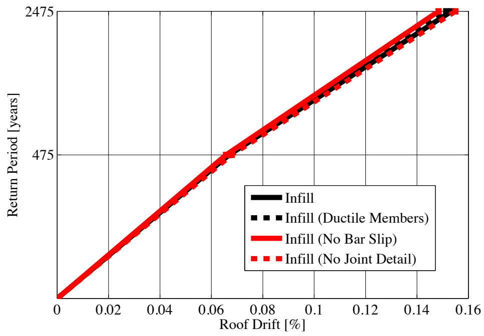
7.3.2. Masonry Infill Structure

Figure 9 shows the response of the various model variations for the case of the infilled frame structures. As it obvious from Figure 9, the presence of the infill results in quite similar median response values for each model variation and the median roof drift value not exceeding the drift a peak force observed in a pushover analysis. This is mainly due to the dominance of the infill frame on the initial stiffness response of the structure. Hence, the combination of low seismicity with limited ductility results in a linear elastic response at both intensities, which from Figure 7 can be seen that the initial stiffness's are quite similar, whereas the post peak behaviour when the infill has failed is quite different. As before with the ductile vs. non-ductile member comparison, this difference would be highlighted more in the collapse fragility of the structure through an incremental dynamic analysis study, where the ductility capacity is fully utilised. Dolsek and Fajfar [17] also noted this behaviour where it was reported that unless the masonry infill exceeds the drift at maximum capacity, the infill would tend to dominate the response of the structure, as can be seen in Figure 9. This is because unless the masonry elements lose significant resistance, the structure essentially becomes a braced frame system with the masonry infill acting as the diagonal bracing members.

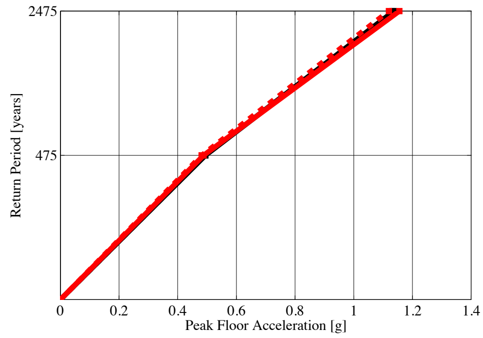
7.3.3. Elastic Damping

Figure 10 and Figure 11 show the response of the bare and infilled reference frames for different levels of elastic damping, respectively. It is seen how the changes in damping result in a noticeable change in the median response ordinates of the structure. This agrees with the finding by [16], where the elastic damping term was shown to heavily influence the results of loss assessment for ductile RC frames compared to other parameters such as initial stiffness and post yield stiffness ratio. It is therefore shown that relative to the other parameters discussed in previous sections for both bare and infilled frames, the level of elastic damping assumed is an important parameter is seismic assessment.

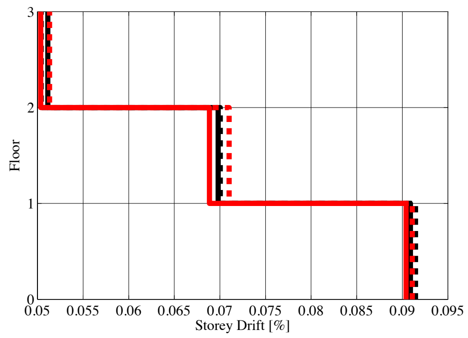
Since the elastic damping parameter is intended to represent the response of all the other elements not modelled, its inclusion is justified. However, should many of the non-structural elements such as cladding and internal partitions be modelled, their response is accounted for and the elastic damping should be reduced accordingly, which has been shown by [16] and also here to be an important parameter which should be properly justifies and not assumed using rules of thumb. Some guidance on the level of elastic damping to be expected from various structural may be obtained from [49].



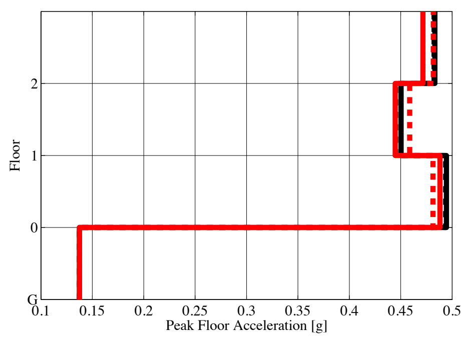
(a) Roof Drift vs. T_R



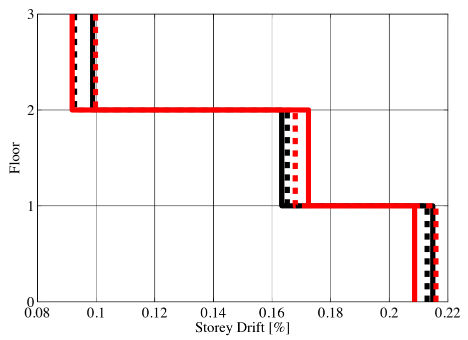
(b) PFA vs. T_R



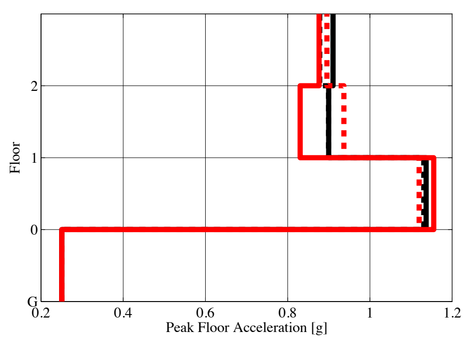
(c) Storey Drift at T_R=475 years



(d) PFA at T_R=475 years



(d) Storey Drift at T_R=2475 years

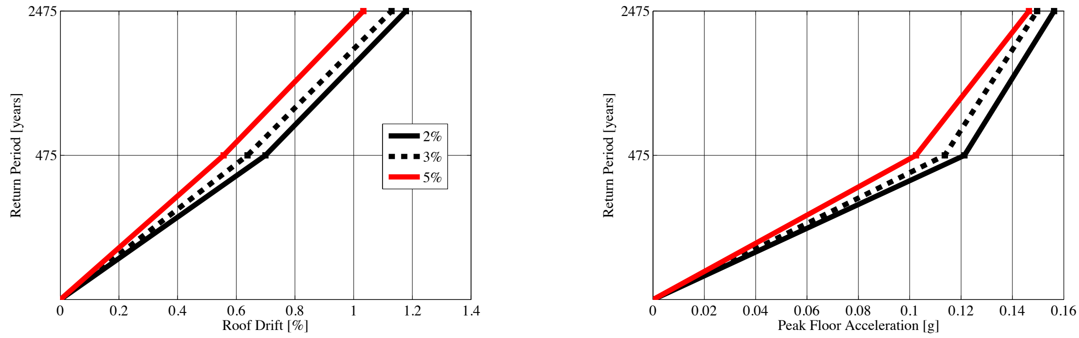


(e) PFA at T_R=2475 years

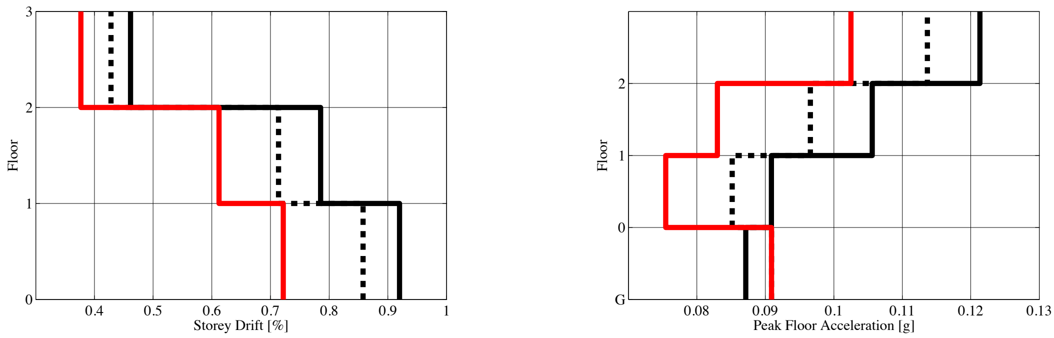
Figure 9: Infill frame structure response.

8 CONCLUSIONS

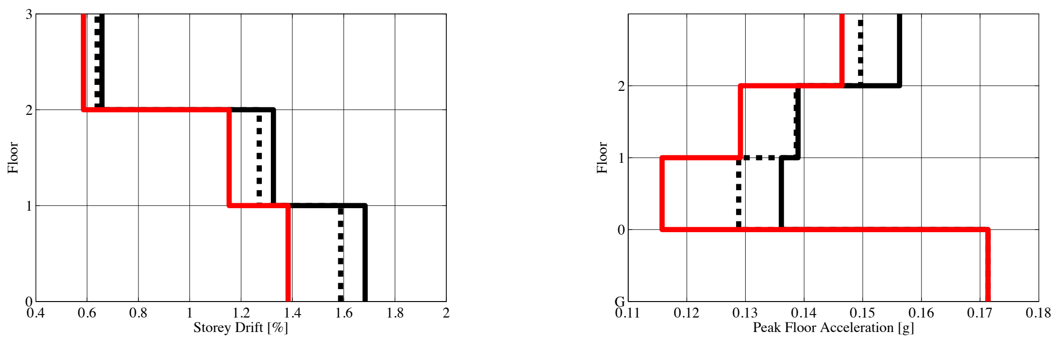
- The numerical modelling of Italian RC frames constructed prior to the introduction of seismic design codes in the 1970's has been discussed. The modelling of the behaviour associated with the joint and beam and column members constructed with plain reinforcing bars and poor seismic detailing was outlined and the behaviour attributed to such characteristics accounted for in a numerical model developed in OpenSees based on available experimental test data. The effects of the bar slippage on both the member flexibility and flexural capacity was discussed and incorporated into the determination of the modelling parameters.



(a) Roof Drift vs. T_R **(b) PFA vs. T_R**



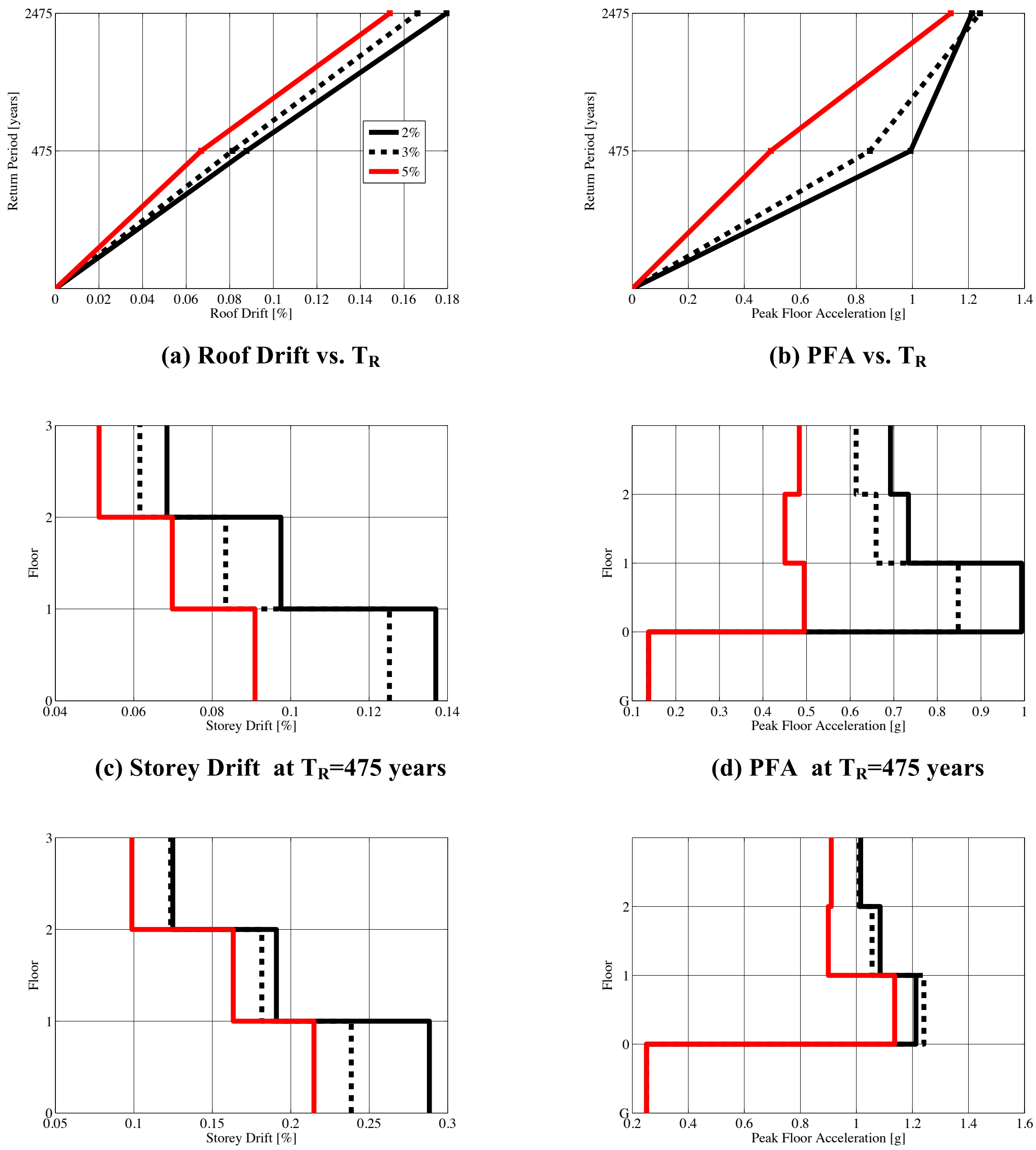
(c) Storey Drift at T_R=475 years **(d) PFA at T_R=475 years**



(d) Storey Drift at T_R=2475 years **(e) PFA at T_R=2475 years**

Figure 10: Influence of elastic damping on bare frame response.

- The aforementioned numerical model was then compared experimental test results of quasi-static cyclic testing carried out on a two-thirds scale three-storey RC frame designed for gravity loads only. The comparison showed the numerical model represents the behaviour very well in terms of strength and stiffness in addition to the hysteretic behaviour and stiffness transitions between reverse cycles.
- A number of variations in modelling of a case study RC frame designed using provisions used in Italy prior to the 1970's were discussed and their expected influence on the seismic response of gravity load designed RC frames considered. As such, a number of numerical model variations were established for an assessment at two different return periods using the proposed numerical model to establish the relative importance of each of these modelling considerations on the response of the structure, which was evaluated in terms of storey drift and peak floor accelerations.



(a) Roof Drift vs. T_R

(b) PFA vs. T_R

(c) Storey Drift at T_R=475 years

(d) PFA at T_R=475 years

(d) Storey Drift at T_R=2475 years

(e) PFA at T_R=2475 years

Figure 11: Influence of elastic damping on infill frame response.

- The sensitivity study showed that for the bare frame, the modelling of the joint behaviour particular to older RC frames is an influential parameter that should be considered. For the infilled frames, it was shown how the presence of masonry infills resulted in the diagonal strut behaviour dominated the behaviour and resulted in other parameters becoming less relevant at lower storey drifts where the drift at maximum capacity of the infills had not yet been exceeded.
- The variation in elastic damping was shown to be an influential parameter in terms of the storey drift and peak floor accelerations for both the bare frame and infilled frame cases, demonstrating that the amount of elastic damping assumed in seismic assessment needs to be considered carefully with reference to the additional damping provided by elements such as non-structural components.

- While this study has shown that joint modelling and consideration of bar slip can be important parameters in seismic assessment, other parameters such as the use of ductile detailed members did not appear to significantly affect results. This is due to the relatively low level of demand relative to the actual lateral capacity of the frames resulting in relatively low ductility demands, especially in the case of infilled frames. It is therefore recommended for future work that a study such as that presented here be extended to consider the collapse fragility of the structures such that the influence of the entire hysteretic backbone curve will be evident, instead of a small ductility range, where strengths and capacities remain relatively similar for some cases.

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