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MODELLING UNCERTAINTY IN EXISTING ITALIAN RC FRAMES

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Abstract. *This paper examines the uncertainty in the response of existing RC frames with masonry infill in Italy. This is done using a numerical modelling approach calibrated and validated against existing experimental test data, where the existing test data is used as a basis to gauge the uncertainty in the various parameters associated with the modelling of beam-column members, joints and masonry infill. The influence of the uncertainty in these parameters is then investigated through a quantification study allowing for the propagation of this modelling uncertainty to the response of the structure under seismic action to be quantified. This is performed for numerous different case study building typologies typically found in Italy. The result of this study is that typical values of dispersion associated with modelling uncertainty structures can be quantified in relation to the demand parameters typically of interest, such as peak storey drift and peak floor acceleration, in addition to the collapse capacity of the building. The impact of this study is that empirical values for these building typologies can be proposed to allow practitioners to adopt more reasonable and representative values of dispersion with respect to those available in the literature. The proposal of these empirical dispersion values fits directly into the broader frameworks typically adopted in the performance-based assessment of existing structures that aim to quantify the performance of buildings in terms of measures such as collapse safety and monetary losses.*

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1 INTRODUCTION

Following the structural analysis of a case study structure using either simplified methods or more advanced non-linear response history analyses (NRHA), the dispersion in the demand parameters of interest needs to be accounted for. In the case of NRHA using a single deterministic model, such as what is typically carried out using the PEER performance-based earthquake engineering (PBEE) methodology [1], the epistemic uncertainty associated with modelling uncertainty is incorporated into the results alongside the aleatory randomness that is already present due to record-to-record variability of the various ground motion (GM) records used in analysis. More simplified approaches, such as displacement-based assessment (DBA) (see [2], for example) and the N2 method (see [3], for example) involve using simplifying assumptions to estimate the response of the structure with a subsequent approximation of dispersion in the response due to modelling uncertainty and record-to-record variability. Both of these aforementioned methods require some form of dispersion estimate for the modelling uncertainty, which ought to come from appropriate quantification studies. As highlighted on multiple occasions during the description of using the N2 method to estimate failure probability of a structure, Fajfar & Dolšek [3] note: "*For practical applications, predetermined default values for the dispersion measures, based on statistical studies of typical structural systems, are needed.*" and "*In a practice-oriented approach, default values for the dispersion measures have to be used. Reliable data for large populations of buildings are not yet available*" before concluding the manuscript with "*Default values for dispersion measures are needed*". The aim of this paper is to estimate and provide such default values of modelling uncertainty to be used when conducting a seismic assessment of gravity load design (GLD) RC frames with masonry infill in Italy. The methodology used to quantify the modelling uncertainty is described and a study on various structural typologies is conducted such that modelling uncertainty values for the collapse fragility and demand parameters used in loss estimation are proposed. This is done using the statistical information regarding the uncertainty in the various modelling parameters established during the numerical model calibration in [4]. These values for modelling uncertainty are defined in terms of structural typology, demand parameter and limit state (LS) under consideration to provide a set of default values that account for the modelling uncertainty in structural typologies found throughout Italy.

2 METHODOLOGY

The variability in the different modelling parameters is propagated through the structural response to result in a variability in the demand parameters of interest. This is in conjunction with the inherent variability due to record-to-record variability when performing NRHA. While the record-to-record variability is typically accounted for with large sets of suitable GMs, the modelling uncertainty is somewhat more difficult to quantify. This is as the individual distributions of the various random variables (RVs) to be considered in the structure are required along with an appropriate method in which a number of different numerical model realisations can be generated. As outlined previously, a common approach is to empirically quantify the effects of this modelling uncertainty on the various demand parameters of interest and incorporate this post-analyses alongside the record-to-record variability. This section, therefore, aims to outline a method to conduct such a study to quantify the modelling uncertainty for different demand parameters such as peak storey drift (PSD) and peak floor acceleration (PFA). The approach adopted here is illustrated in Figure 1, where a number of model realisations are generated to take the variability in the different RVs considered into account. These model realisations are then analysed using incremental dynamic analysis (IDA) [5] at a number of intensity levels.

That is, for a given GM and intensity level, the dispersion in the demand parameter due to modelling uncertainty can be quantified. Likewise, for a given model realisation and intensity level, the record-to-record variability may be also computed but is not the focus of this study. In addition to examining the effects of modelling uncertainty with respect to intensity, the influence on the collapse fragility median and dispersion is also investigated.

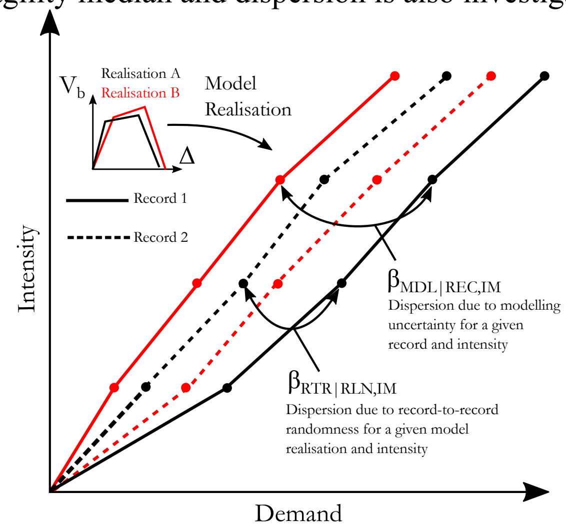

Figure 1. Illustration of the procedure used to identify the modelling uncertainty in various structural typologies with increasing intensity. The use of numerous model realisations and GMs means that the variance with respect to either of the two source of uncertainty (i.e. record-to-record variability or modelling uncertainty) can be quantified.

2.1 Structural Typologies and Numerical Modelling

The case study frames examined consist of two and three storey RC frames that are adopted from a previous study by [6]. Both frames have been designed for gravity load only using allowable stress and other such design provisions specified in Regio Decreto 2229/39 [7], along with other common construction conventions prior to the introduction of seismic design provisions in Italy in the 1970's. Some typical details of these gravity-load only designs are the complete lack of capacity design considerations in the beam and column members as the columns were sized principally for axial loading. This approach was quite common during the construction boom that followed the second world war across southern Europe and resulted in many RC structures vulnerable to undesirable seismic response. In addition to the out-of-date seismic provisions (or in many cases, no seismic provisions at all) adopted during past construction, another detail regarding the construction of these buildings that leaves them quite vulnerable to seismic loading is the use of smooth reinforcing bars in the frame members that were terminated with end-hooks in the beam-column joints. This affects the bonding of the reinforcement to the concrete paste resulting in a modified ductility compared to newer ductile detailing of the frame members in addition to a potential shear mechanism in beam-column joint, as discussed in [4,8].

The general layout of the case study structures is shown in Figure 2, where each of the frames consist of a three-bay frame with exterior bays of 4.5m and an internal bay of 2m, with a constant storey height of 3m and out-of-plane tributary width of 4.5m. The strength of the reinforcing steel and concrete were 3800kg/cm^2 (372MPa) and 200kg/cm^2 (19.6MPa), respectively, whereas the floor loadings were taken as 500kg/m^2 for the roof levels and 600kg/m^2 for each floor, as per typical design manuals in use at the time of construction. As these frames were sized for gravity loading only, the beam section sizes and detailing are constant at each level in addition to the column sections. The column sections consist of 25x25cm section with four

14mm longitudinal bars and 6mm transverse stirrups placed every 100mm. The beam section consists of a 25x50cm deep rectangular section with four and two 14mm longitudinal bars placed at the top and bottom, respectively, also with 6mm transverse stirrups placed every 100mm. In terms of numerical modelling the case study frames, the developments of [4] are adopted herein.

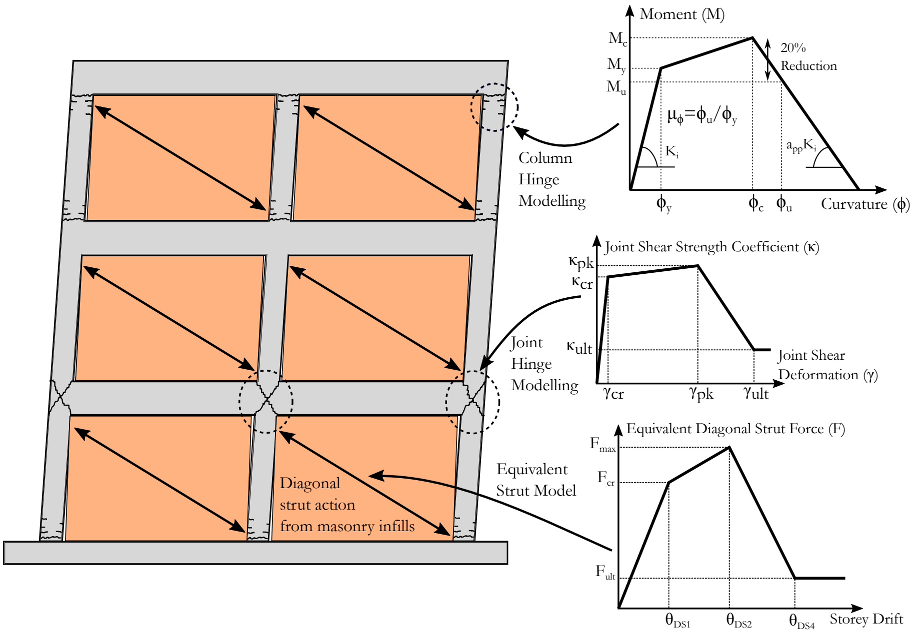

Figure 2. Illustration of the numerical modelling of the various damageable components of GLD RC frames with infills. The hysteretic behaviour of the beams, columns, beam-column joints and masonry infill struts are also illustrated.

While the case study frames have been designed as bare frames, common practice throughout southern Italy was to insert masonry infills in the structures without considering their effects on the surrounding frame during the design process. As such, a number of different infill case study frames are also investigated here to illustrate their effects on the structural behaviour and overall performance of GLD RC frames. Two different infill layouts were considered; a uniform infill throughout the height of the building and a uniform infill layout with an open ground floor, commonly referred to as a "pilotis" frame. In addition for the uniform infill frames, two types of masonry infill have been used and are termed "weak" and "strong" infill, as per [9], where weak infill corresponds to the 8cm thick single leaf infill and 30cm thick block, respectively. The effects of modelling openings such as windows or doors were not considered as part of this study.

2.2 Random Variables and Associated Distributions

Based on the numerical modelling of the different structural typologies in Section 2.1, the RVs to be selected as part of this modelling uncertainty quantification study for GLD RC frames typically found in Italy are established along with a brief description of the source of such information and the relevant justification for their consideration. Table 1 lists the initial list of RVs selected for each of the beam and column members, interior and exterior beam-column joints, masonry infill and other global modelling parameters, where the various parameters outlined are illustrated for the different components in Figure 2.

Table 1. List of RVs to be considered for quantification of modelling uncertainty in GLD RC frames, where the median value and corresponding dispersion value are provided for the lognormal distribution of each (Notation as per model definitions as per [4] and illustrated in Figure 2).

	#	RV	Description	Source	Median	Dispersion	Reference
Beams	1	M_y	Yield moment	Computed	-	0.122	[4]
	2	ϕ_y	Yield curvature			0.287	
	3	μ_ϕ	Ultimate curvature ductility			0.326	
	4	a_{pp}	Post-peak stiffness ratio			0.413	
	5	ρ_L	Longitudinal reinforcement ratio			0.250	
Columns	6	M_y	Yield moment	Computed	-	0.122	[4]
	7	ϕ_y	Yield curvature			0.287	
	8	μ_ϕ	Ultimate curvature ductility			0.326	
	9	a_{pp}	Post-peak stiffness ratio			0.413	
	10	ρ_L	Longitudinal reinforcement ratio			0.250	
Exterior Joints	11	γ_{cr}	Joint shear deformation at cracking	Test Data	0.0002	0.300	Estimate
	12	γ_{pk}	Joint shear deformation at peak capacity		0.0127	0.286	[4]
	13	γ_{ult}	Joint shear deformation at ultimate capacity		0.0261	0.229	
	14	κ_{cr}	Joint shear strength coefficient at cracking		0.135	0.166	
	15	κ_{ult}	Joint shear strength coefficient at ultimate capacity		0.05	0.091	
Interior Joints	16	γ_{cr}	Joint shear deformation at cracking	Test Data	0.0002	0.300	Estimate
	17	γ_{pk}	Joint shear deformation at peak capacity		0.0085	0.133	[4]
	18	κ_{cr}	Joint shear strength coefficient at cracking		0.29	0.237	
	19	κ_{pk}	Joint shear strength coefficient at peak capacity		0.42	0.163	
Masonry Infills	20	F_{max}	Infill diagonal strut capacity	[10]	-	0.300	Estimate
	21	θ_{DS1}	Storey drift at DS1 defined in [10]	Test Data	0.18%	0.520	[10]
	22	θ_{DS2}	Storey drift at DS2 defined in [10]		0.46%	0.540	
	23	θ_{DS4}	Storey drift at DS4 defined in [10]		1.88%	0.380	
Global	24	ξ	Elastic damping ratio	Assumed Value	0.05	0.600	[11]
	25	M	Floor mass	Given	-	0.100	

Regarding the adopted distributions for each of the structural elements, these are justified as follows. The beam and column member distributions are adopted directly from the information presented in [4], where the calibration information regarding the member capacity, stiffness and ductility capacity are adopted as they come from the calibrations to actual test data available in the literature. The noted source of "computed" for these members in Table 1 refers to how the median value is not a fixed value, but a computed value from the expressions described in [4] (e.g. the yield moment (M_y) is not a fixed value for every member, but depends on section dimensions, reinforcing content and axial load ratio), where the associated dispersion was computed from the comparison of the relevant expression to the actual test data. Other information regarding masonry infill median drifts and dispersions are adapted from the study by [10], whereas other information regarding appropriate dispersions for elastic damping and structural mass are adopted from a similar study concerning modelling uncertainty in RC frames conducted by [11].

2.3 Analysis of Model Realisations

Using the Correlation-Reduced Latin Hypercube Sampling (CLHS) approach [12], a number of realisations of each structural model discussed in Section 2.1 are developed. Each of the 25 RVs listed in Table 1 are sampled a number of times to generate N number of model realisations. As described in [12], the number of model realisations needs to be greater than the number of RVs (i.e. N>K) in order for the CLHS method to function. As such, the number of realisations was chosen here to be 40. This number was selected based on the parametric study by [13] who noted that 20 appeared to be a reasonable value. Also of note are the correlations assumed between different RVs, which were assumed to be independent and uncorrelated for simplicity of analysis. Physical relations between different RVs were maintained through the model definition, such as the yield moment and the percentage of reinforcement since the yield moment is computed using the sampled value of reinforcement through sectional analysis. Similarly, with the floor mass and the design axial loading on columns, which influences the member ductility and post-peak ductility capacity of the members as outlined in [4].

While each of the RVs are sampled according to the aforementioned distributions, care was taken to ensure that no instances of unrealistic model realisations arise. For instance, the sampled value of θ_{DS2} for the masonry infill illustrated in Figure 2 was checked to be always greater than that of θ_{DS1}. For each of the model realisations sampled here, the structural typologies discussed previously were analysed using 10 GMs taken from the FEMA P695 [14] set that were selected in order to maintain a good match in terms of median and dispersion with the original set of 44. While 10 records may be considered a small GM set, these records are not used to compute record-to-record variability but only the modelling uncertainty, where the full set of 44 records was used to quantify the record-to-record variability using the deterministic model and is discussed further in [15]. Each GM was scaled to a number of intensities for the seismic IM employed using IDA, which in this case was the spectral acceleration at the first mode period of vibration of the structure, $S_a(T_1)$.

3 ANALYSIS RESULTS

Using the methodology and model realisations described in the previous section, the results of the modelling uncertainty study are presented here. As previously outlined, each frame typology is subjected to IDA such that the response and dispersion can be quantified with respect to intensity and limit state in the buildings. These limit states are defined in accordance with the definitions outlined in the Italian National Code NTC 2008 [16], which list four limit states corresponding to Operational (SLO), Damage Control (SLD), Life Safety (SLV) and Collapse Prevention (SLC), respectively, and are illustrated in Figure 3. The exceedance of each LS is determined based on the criteria outlined NTC 2008 and the corresponding PSD established for each LS, which can be then used with IDA to characterise the exceedance of this PSD with respect to intensity. As such, the dispersion with respect to intensity and subsequently LS in each structural typology can be quantified.

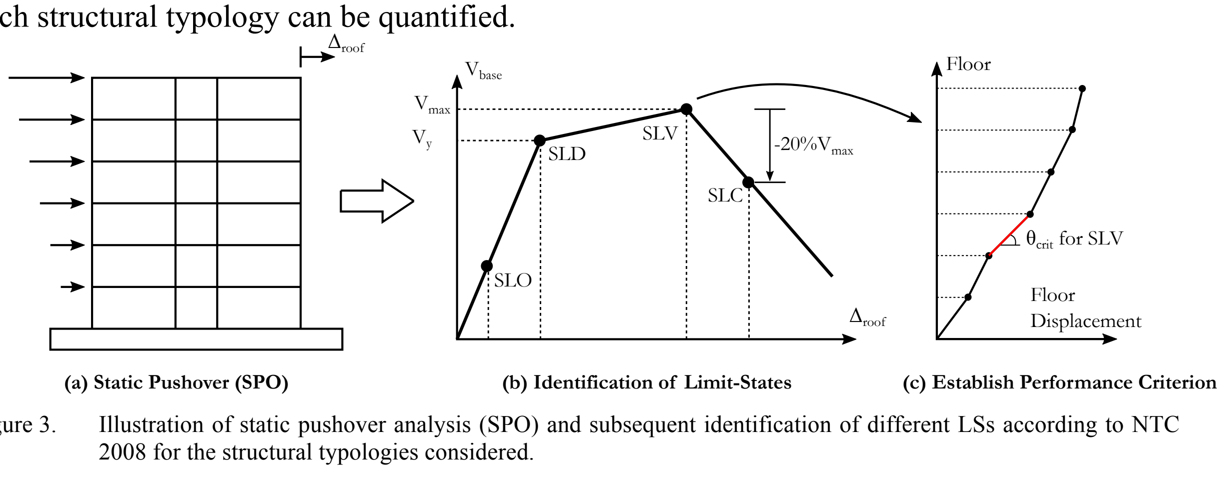

Figure 3. Illustration of static pushover analysis (SPO) and subsequent identification of different LSs according to NTC 2008 for the structural typologies considered.

3.1 Collapse Fragility

By analysing the different structural typologies at different intensity levels via IDA, the number of collapses with respect to intensity, GM and model realisation can be computed. Using the maximum likelihood method outlined by [17], the collapse fragility function of the different structural typologies can be computed with respect to each GM record or model realisation as function of intensity. Collapse is typically defined as when the IDA trace "flatlines" or becomes sufficiently large to cause dynamic instability. This point is defined quantitatively as when the maximum PSD exceeds 10%, which is deemed sufficiently large to have caused collapse and is consistent with other studies such as [18], for example. These collapse fragilities are plotted in Figure 4 for each of the structures considered, where the plots labelled RTR, MDL and TOT represent the mean collapse fragilities considering record-to-record variability only, modelling uncertainty only and both record-to-record variability and modelling uncertainty, respectively. The TOT fragility functions are considered the more representative of the three as these consider the both of the aforementioned sources of uncertainty. By performing an IDA using a given set of GMs on a deterministic model of a case study structure, the resulting collapse fragility function would correspond to that of the RTR lines plotted in Figure 4. Comparing this to the more representative TOT fragility functions, the ratios of median collapse intensity and dispersion differ somewhat. The mean ratio of TOT to RTR for the median collapse intensity is 0.95 and the 1.27 for the associated dispersions. That is, the RTR fragility functions tend to overestimate the median collapse intensity and underestimate the dispersion when compared to the collapse fragilities that account for both aforementioned sources of uncertainty; a finding consistent with existing research [13,18–20]. The difference in the dispersion is an expected result and is typically accounted for by inflating the collapse dispersion by a prescribed value to account for the effects of modelling uncertainty. This is the approach of the FEMA P695 guidelines [14], among others, that prescribe values to which the RTR collapse fragility is to be increased using a square root sum-of-the-squares (SRSS) combination. However, guidelines such as FEMA P695 only propose a modification to the dispersion when accounting for modelling uncertainty despite Figure 4 illustrating that this tends to overestimate the median collapse intensity.

Considering the above remarks regarding the comparisons between RTR type collapse fragility functions and bearing in mind the need to provide simple ways to account for modelling uncertainty in collapse assessment of GLD RC frames in Italy, some simplified adjustments are thus proposed. These consist of providing an adjustment to both the median collapse intensity and dispersion. This is done through a prescribed reduction factor for the median collapse intensity (R_c) and a dispersion due to modelling uncertainty ($\beta_{UC,IM}$) to be combined with the existing dispersion due to record-to-record variability ($\beta_{RC,IM}$) using an SRSS combination as follows:

$$\beta_{TOT,IM} = \sqrt{\beta_{RC,IM}^2 + \beta_{UC,IM}^2} \qquad \text{Equation 1}$$

which assumes the two sources of uncertainty to be independent of each another. The above notation follows that of [21] in order to maintain consistency with the notation used in subsequent section and to distinguish different types and sources of dispersion. These prescribed modifications are proposed in terms of structural typology and are listed in Table 2, where the coefficients of variation for each modification term is included in parenthesis. This proposal is labelled as ADJ in Figure 4, where examining the ratios between this and the TOT collapse fragility functions, it is seen to work well with overall mean ratios of 1.00 and 0.98 between the median collapse intensity and dispersion of the TOT and ADJ collapse fragilities, respectively. Comparing these to some existing values in the literature from both [14] and [13], the values appear reasonable and of the same order of magnitude, with [14] proposing values of $\beta_{UC,IM}$ between 0.10 and 0.50 depending on how well the model represents the actual structural behaviour and how robust that model is. Dolšek [13] on the other hand proposed a $\beta_{UC,IM}$ of 0.52, which at first appear a little higher than those listed in Table 2. However, two differences that ought to be considered in the case of the values proposed by [13] are that this value applies to frames modelled without infill only and also that peak ground acceleration (PGA) is used as the intensity measure (IM), as opposed to $S_a(T_1)$ used here. This difference could explain the increase in modelling uncertainty with respect to the values proposed here. In any case, the quantification of the effects of modelling uncertainty in infilled RC frames has not been exclusively addressed in either of the two aforementioned studies and as such, the values in Table 2 represent a novel contribution in this regard. In addition, the modification to the median and increase in overall dispersion proposed by [18] for bare non-ductile bare frames with no masonry infill in the US are somewhat similar to the corresponding case here. An average reduction of 0.95 in the median collapse intensity and an additional dispersion to account for modelling uncertainty of 0.33 noted, which are quite similar to corresponding values in Table 2, where [18] employed the same IM as that used in this study.

Table 2. Proposed collapse fragility modification factors to account for the effects of modelling uncertainty on the median collapse intensity and dispersion, where the coefficients of variation for each term are provided in parenthesis.

Structural Typology	R_c	$\beta_{UC,IM}$
w/o Infill	0.89 (0.04)	0.30 (0.06)
Pilotis Frame	0.95 (0.04)	0.30 (0.02)
Infill Frame	0.99 (0.01)	0.15 (0.07)

Table 2 shows how the influence of the modelling uncertainty on the collapse fragility is somewhat higher for the frames without infill modelled and pilotis frame typologies than for the infill frames since the median collapse intensity remains unchanged and the increase in dispersion is lower. One possible reason for such a difference may be that the infill frame response tends to be dominated by the presence of the infill, and less so by the actual surrounding RC frame. Therefore, there are fewer RVs influencing the response of the frames as the variability mainly comes from the infill and not the frame elements such as beam, columns and joints that possess many more RVs. This above comment, however, would require further confirmation through more detailed sensitivity studies.

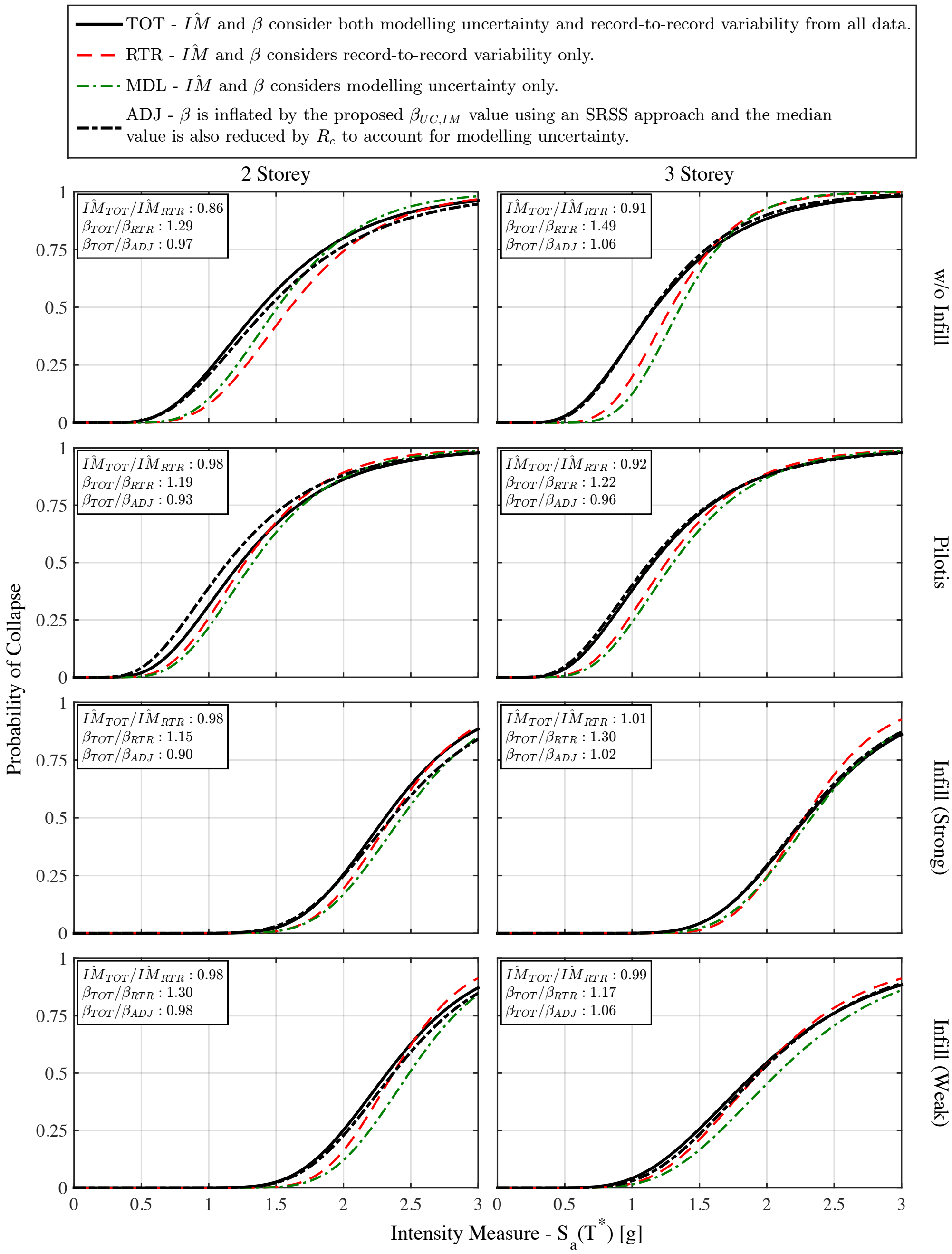

Figure 4. Comparison of the collapse fragility functions considering different sources of uncertainty compared with the two methods of adjusting the collapse fragility function to account for modelling uncertainty in GLD RC frames. Left and right-hand plots correspond to the 2 and 3 storey frames, respectively.

3.2 Demand Parameters with Respect to Intensity

While the previous section looked at the influence of modelling uncertainty on collapse, this section discusses the effects of the modelling uncertainty on the demand parameters that are typically used for the seismic assessment of structures, namely PSD and PFA. Similar to the approach adopted for the collapse fragility functions, the effects of modelling uncertainty are incorporated by using an SRSS combination with the record-to-record variability to represent the overall dispersion in the demand parameters, described by the following expressions:

$$\beta_{D,\theta} = \sqrt{\beta_{DR,\theta}^2 + \beta_{DU,\theta}^2} \qquad \text{Equation 2}$$

$$\beta_{D,a} = \sqrt{\beta_{DR,a}^2 + \beta_{DU,a}^2} \qquad \text{Equation 3}$$

which again assumed that the two sources of uncertainty are uncorrelated. The subscript D denotes demand whereas θ and a denote PSD and PFA, respectively. Further investigation described in [15] has shown that the use of an SRSS combination gives a good representation, albeit slightly conservative, of the overall dispersion in the two demand parameters.

In order to compute the dispersion in the two demand parameters due to modelling uncertainty, the results are analysed with respect to both a given GM record and model realisation, as illustrated in Figure 1. Therefore, the modelling uncertainty versus intensity will be investigated for both demand parameters at each floor for the case study structures investigated. Existing research suggests that the modelling uncertainty is not only a function of the demand parameter of interest but also to the LS being considered, where the LSs described here are as per the definitions illustrated in Figure 3. As such, the median intensities associated with each of the four LSs determined via IDA are used here. Figure 5 and Figure 6 present the modelling uncertainty versus IM for both demand parameters, where the median intensity for the four LSs considered are shown also. These proposed values are also listed in Table 3 in terms of the structural typology and demand parameter of interest. They are intended to provide an upper bound on the expected modelling uncertainty. It should be noted that these values have been developed using low-rise RC frames with two and three floors and as such, trends with respect to the number of floors etc. have not yet been identified. Future work may be carried out to identify sets of coefficients that are a function of this, or indeed the first mode period of a structure as is done in FEMA P58-1 [1], but for now a single set has been proposed.

Some initial comments that can be made regarding the values are that the PFA values tend to be much lower than those of PSD; a trend that is also present in the proposed values of [1]. In addition, the modelling uncertainty tends to increase with more severe LSs for the PSD but tends to plateau for the PFA. One reason for this may be due to the structure becoming more non-linear with increasing intensity and thus being influenced by the dispersion in more RVs for the backbone behaviour, whereas the PFAs tend to be capped by the first mode lateral yield strength of the structure. For instance, consider the modelling uncertainty values associated with the PFA in the pilotis frames. The behaviour of these frames is typically governed by the soft-storey forming in the bottom storey, and dispersion due to modelling uncertainty for PFA will come from the lateral capacity of that floor only meaning that the influential RVs are greatly reduced compared to other frames. This could explain the slightly lower dispersion of the PFA for the pilotis frame with respect to the other frame typologies. In addition, the dispersion in PSD in the upper floors for pilotis frames does not reduce significantly with respect to the ground floor in the same way the median demands do. In fact, the median demands do decrease but the relative dispersion in these values still remain significant. The PSD dispersion tends to reduce for intensities beyond the SLC LS in the cases of the frames without infill modelling and the pilotis frames. This can be explained conceptually as being due to the structure being highly non-linear and as a result, the mechanism formed acts a fuse to which all of the median PSD demand is concentrated. This increase in PSD demand and formation of a mechanism reduces the amount of RVs that are directly influencing the overall response of the frame and as a result, the dispersion would be expected to reduce with respect to the lower LSs, as is illustrated in Figure 5. This reduction is not evident for the infilled frames since the intensities up to which this study was conducted do not surpass the SLC LS intensity of the infilled frames very much to observe such a trend, although is it somewhat apparent in the case of the three-storey weak infill frame.

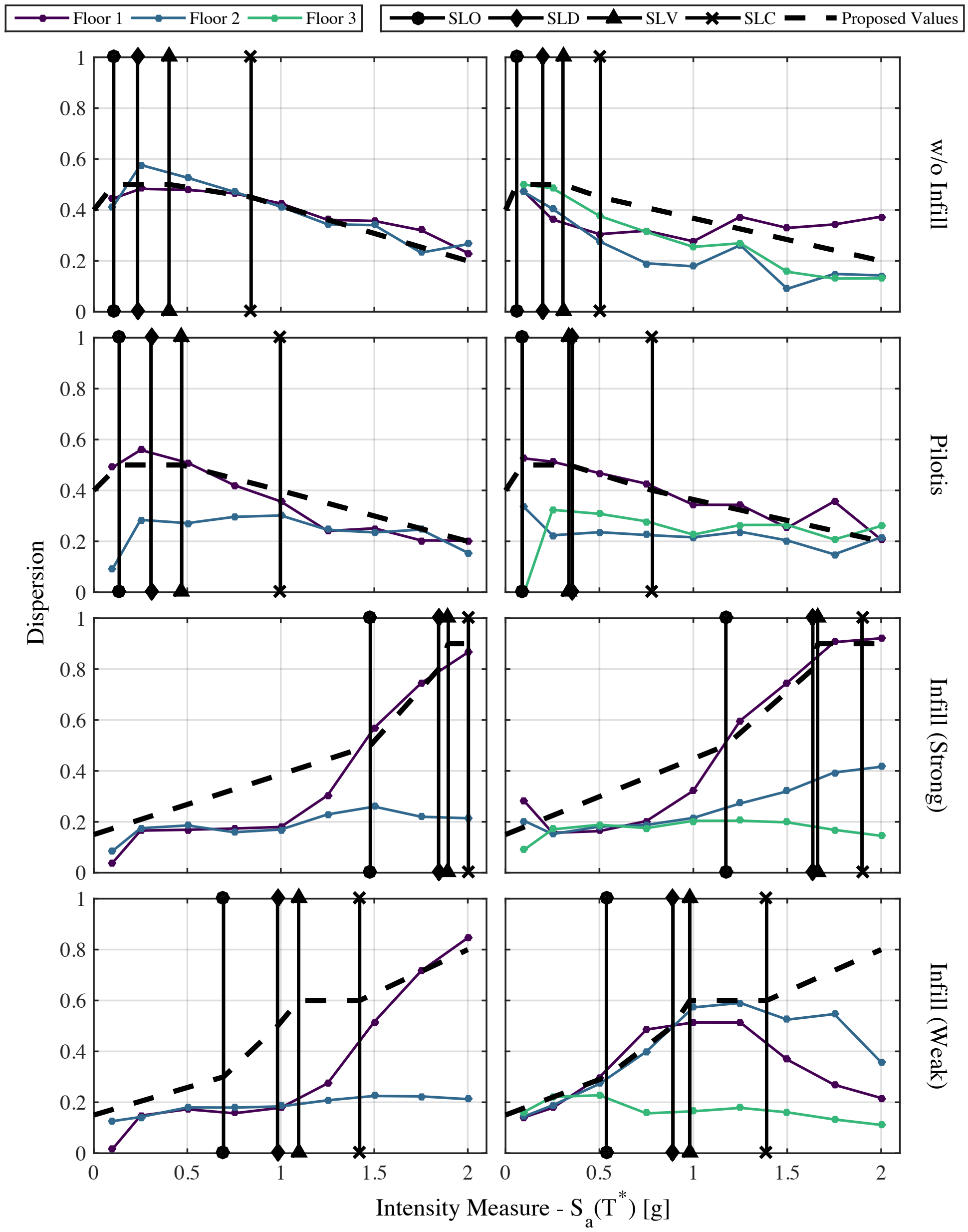

Figure 5. Modelling uncertainty associated with the PSD ($\beta_{DU,\theta}$) versus intensity for the different structural typologies, where the median intensities for the four LSs of interest are plotted alongside the proposed values.

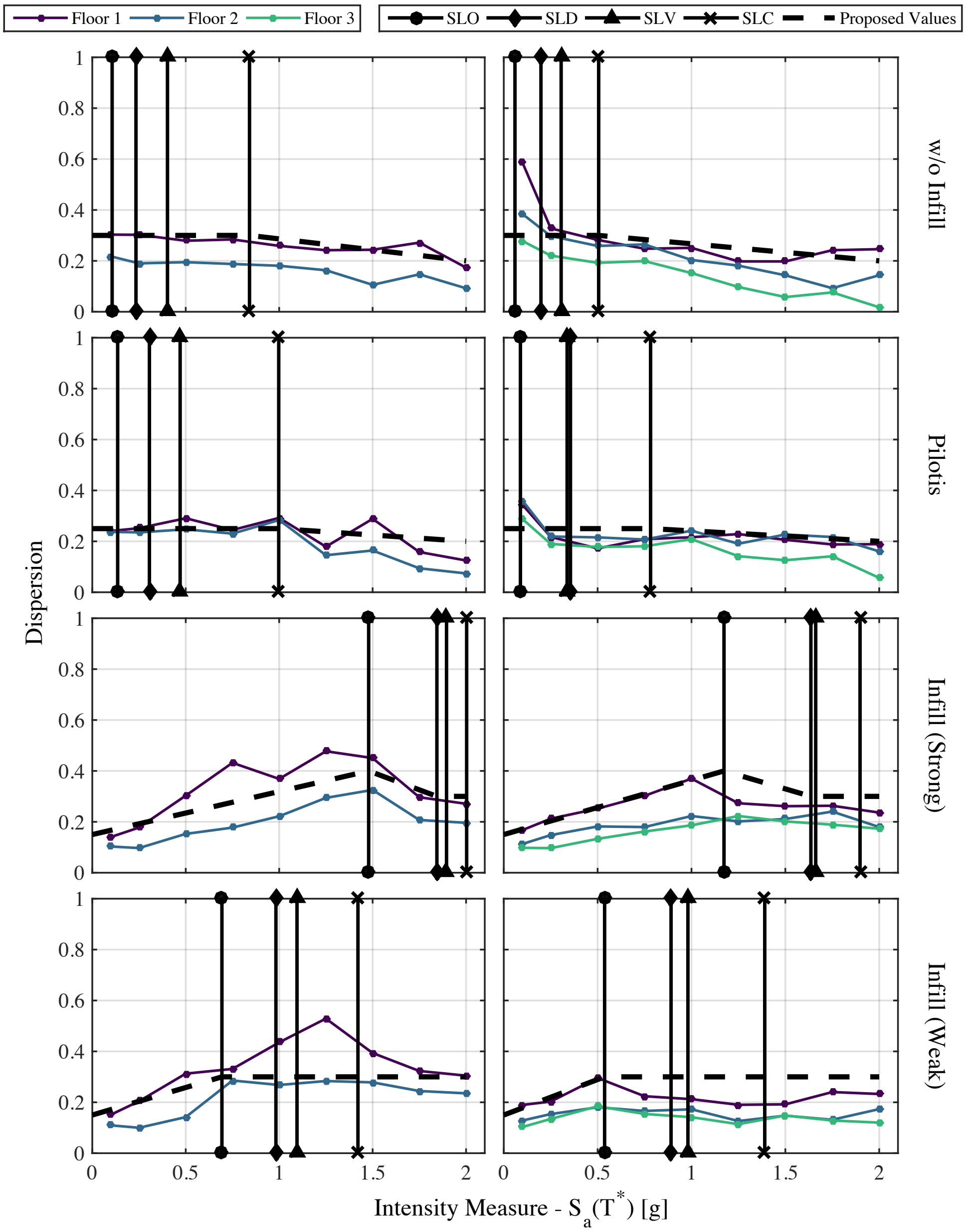

Figure 6. Modelling uncertainty associated with the PFA ($\beta_{DU,a}$) versus intensity for the different structural typologies, where the median intensities for the four LSs of interest are plotted alongside the proposed values.

Table 3. Proposed modelling uncertainty values (β_{DU}) for both PSD and PFA as a function of structural typology and anticipated LS.

Structural Typology	Modelling Uncertainty					
	<SLO	SLO	SLD	SLV	SLC	>SLC
	Peak Storey Drift ($\beta_{DU,\theta}$)					
w/o Infill	0.40	0.50	0.50	0.50	0.45	0.20
Pilotis Frame	0.40	0.50	0.50	0.50	0.40	0.20
Infill Frame (Strong)	0.15	0.50	0.80	0.90	0.90	0.90
Infill Frame (Weak)	0.15	0.30	0.50	0.60	0.60	0.80
	Peak Floor Acceleration ($\beta_{DU,a}$)					
w/o Infill	0.30	0.30	0.30	0.30	0.30	0.20
Pilotis Frame	0.25	0.25	0.25	0.25	0.25	0.20
Infill Frame (Strong)	0.15	0.40	0.30	0.30	0.30	0.30
Infill Frame (Weak)	0.15	0.30	0.30	0.30	0.30	0.30

4 SUMMARY AND CONCLUSIONS

This paper discussed the quantification of the modelling uncertainty associated with the various demand parameters typically used in the assessment of GLD RC frames Italy. This was then followed by the description of the modelling realisations using the various RVs established for the structural members such that their effects on the collapse fragility and the PSD and PFA demand could be quantified. From this quantification study, the following conclusions can be drawn:

- The effects of modelling uncertainty on the collapse capacity of the GLD RC frame structures show that the median collapse intensity tends to reduce and the dispersion tends to increase. From the analysis conducted here, empirical values for the reduction of the median collapse intensity and the increase in the dispersion for the collapse fragility are provided with respect to structural typology.
- Similarly, the effects of modelling uncertainty on the dispersion of the PSD and PFA of the different structural typologies were investigated, where the dispersion was seen to increase significantly when considering the model parameter uncertainty. From the analysis of the model realisations generated for each structural typology, a set of empirical dispersion values to account for modelling uncertainty were proposed as a function of the different LSs, structural typology and the demand parameter of interest that can be aggregated into the record-to-record dispersion via SRSS combination.
- Comparing the proposed values with existing empirical values available in the literature has shown that the increased dispersion associated with modelling uncertainty to be quantitatively different from other structures such as modern ductile RC frames without masonry infills to the point where default values provided in guidelines such as FEMA P58 cannot be reasonably adopted.

5 ACKNOWLEDGEMENTS

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