16TH EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING
THESSALONIKI 18 - 21 JUNE 2018

INTENSITY MEASURES FOR THE COLLAPSE ASSESSMENT OF INFILLED RC FRAMES

Gerard J. O'REILLY[1], Mohsen KOHRANGI[2], Paolo BAZZURRO[3], Ricardo MONTEIRO[4]

ABSTRACT

A ground motion intensity measure (*IM*) represents the interface between the ground shaking intensity and structural response. An ideal *IM* should be efficient, sufficient and practical. The first aspect, which is the focus of this paper, encompasses the choice of an *IM* that exhibits an adequately low dispersion in structural response prediction. The presence of masonry infills in RC frame buildings is well-recognised as a critical aspect that significantly modifies the RC frames' behaviour, especially in old buildings with no seismic design provisions. This renders common *IM*s such as spectral acceleration at the first mode period of structures, $Sa(T_1)$, somewhat inefficient because of the abrupt changes in stiffness of these structures, and thus of T_1, due to a sudden brittle break of the infills during shaking. This paper addresses the collapse assessment of gravity load designed (GLD) infilled RC frames by exploring different *IM*s based on single spectral ordinate, $Sa(T)$, or multiple spectral accelerations averaged in a period range, *AvgSa*. As such, a pool of candidate *IM*s are selected and evaluated to address efficient collapse assessment of these buildings. Furthermore, a discussion about the choice of building specific and generic *IM*s for portfolio seismic assessments is provided.

Keywords: collapse; assessment; intensity measure; dispersion

1. INTRODUCTION

The seismic assessment of existing reinforced concrete (RC) frames with masonry infills has been the focus of much research in recent years. In particular, buildings designed before the introduction of adequate design codes in the Mediterranean area, often referred to as gravity load designed (GLD) RC frames, have received considerable attention (e.g., Kohrangi et al. 2016; O'Reilly and Sullivan 2017b) given their common presence in the existing building stock and high vulnerability. This is owing to the fact that these GLD RC frames were designed prior to the introduction of seismic design force provisions and whose members were sized for vertical loading only. This was often done with no consideration of what are now well-established concepts in seismic design, such as capacity design and adequate core confinement in RC members to ensure ductile response. While the structural performance of GLD RC frames is well known, some issues relating to the numerical modelling are still under development. Recent work (O'Reilly and Sullivan 2017a) has proposed a numerical modelling approach for GLD RC frame members with low levels of reinforcement and smooth reinforcing bars in addition to beam-column joints with no transverse shear reinforcement and end-hook anchorage. These details were quite common practice in Italy prior to the 1970s. The modelling parameters for the beam-column members and joints were calibrated using experimental test data available in the literature. O'Reilly and Sullivan (2017a) have also shown how the proposed approach captured the hysteretic behaviour, deformed shape and damage mechanism, whereas more traditional modelling techniques were shown not to be representative for any of these.

The development of improved intensity measures (*IM*s) for seismic design and assessment of structures has been the recently focus of a large body of research. Some of the typical *IM*s used in collapse assessment of existing structures include peak ground acceleration (*PGA*) and spectral

---

[1] Research Assistant, Scuola Universitaria Superiore IUSS Pavia, Italy. gerard.oreilly@iusspavia.it
[2] Research Assistant, Scuola Universitaria Superiore IUSS Pavia, Italy. mohsen.kohrangi@gmail.com
[3] Professor, Scuola Universitaria Superiore IUSS Pavia, Italy. paolo.bazzurro@iusspavia.it
[4] Assistant Professor, Scuola Universitaria Superiore IUSS Pavia, Italy. ricardo.monteiro@iusspavia.it

acceleration at the first mode period of structural vibration, $Sa(T_1)$. The use of *PGA* and $Sa(T_1)$ is certainly not without its drawbacks and limitations. For example, issues such as spectral shape (Baker and Cornell 2006; Bojórquez and Iervolino 2011; Eads et al. 2015, 2016), ground motion duration (Kazantzi and Vamvatsikos 2015a; Chandramohan et al. 2016), period elongation and influence of higher modes (Kazantzi and Vamvatsikos 2015b; Kohrangi et al. 2016; Kohrangi et al. 2017; Orumiyehei et al. 2017) have all been noted to be pertinent parameters in response prediction, therefore calling the need for application of more advanced and representative *IM*s. Period elongation, among others, is of particular importance for GLD RC frames with masonry infills. This is due to the local collapse of infill walls that may result in significant strength degradation and subsequent period elongation before the eventual global collapse of the building. This shift in modal properties due to the presence of the infill means that an *IM* anchored to the initial, undamaged period of the frame can be rather inappropriate when assessing collapse performance. Among various recently developed advanced *IM*s, average spectral acceleration, *AvgSa*, (Kohrangi et al. 2017) appears as a prominent candidate for the next generation of *IM*s to use in building seismic assessment. When compared to other *IM*s, *AvgSa* is a good response predictor (with moderately low dispersion) along the height of building structures for both drift and floor acceleration responses, amongst other positive aspects. The use of *AvgSa* for GLD RC frames with masonry infill was initially trialled for a single GLD RC frame building by Orumiyehei et al. (2017), who further corroborated some of the qualities of *AvgSa* initially noted by Kohrangi et al. (2017).

This paper extends and examines application of *AvgSa* for collapse prediction of GLD RC frames with masonry infills. This will be done by trialling a number of *IM* definitions and evaluating their relative performance in collapse prediction. The selected *IM*s include building-specific *IM*s that are defined using the modal properties of each individual building as well as generic *IM*s to assess groups of buildings as part of a larger portfolio. These *IM*s are evaluated and compared in the following sections for a number of case study frames in order to highlight their pertinent aspects in the assessment of GLD RC frames with masonry infill.

2. INTENSITY MEASURES

An *IM* is the single interface variable that desirably connects seismological and engineering aspects of the problem in structural seismic assessment procedures. Seismologists use ground motion prediction equations (GMPEs) and probabilistic seismic hazard analysis (PSHA) to evaluate the rate of exceedance of an *IM* at a specific site in a given period of time. In other words, PSHA quantifies the intensity of ground motion at different return periods using the *IM* of interest. Engineers, on the other hand, use the *IM* to examine the subsequent response of structures and to evaluate their seismic performances. This characterisation of the interface between seismology and engineering, among other reasons, intends to avoid loosely relating the building response to seismological parameters such as magnitude and distance. A desirable *IM* ought to be:

1. *Practical - IMs for which robust and modern GMPEs are available.*
2. *Efficient - structural response should exhibit relatively low variability for the parameters of interest.*
3. *Sufficient: important record-specific seismological parameters (e.g., magnitude, distance, epsilon) are represented without introducing any bias in results. This implies that unless it is shown that demands are not influenced by seismological parameters, the results are to be considered as site-specific.*

The focus of this study is on the second point above relating to the *IM* efficiency with respect to the collapse assessment of infilled GLD RC frames. For collapse assessment, the efficiency is typically represented by the lognormal standard deviation, or dispersion, of the intensity that causes structural collapse. The magnitude of this dispersion is related to the efficiency of the adopted *IM*. Ideally, this dispersion should be as low as possible but, as noted by Kazantzi and Vamvatsikos (2015), the reduction in dispersion due to a more efficient *IM* does not necessarily mean more accurate risk assessment. This is due to the potential decrease in the predictability of the *IM* due to an increased uncertainty in the GMPE. As such, it is critical to adopt *IM*s that are reasonably efficient in their prediction power of structural behaviour (i.e., low dispersion) but also possess a fair degree of practicality. Regarding sufficiency, ground motion scaling and its potential to introduce bias plays a prominent role. In theory, a perfectly sufficient *IM* would permit the unrestricted scaling of ground motion records without introducing any bias into the results, but no such *IM* exists and, therefore, it is desirable to limit scaling. For example, the most widely adopted *IM* currently, $Sa(T_1)$, has been criticised for introducing bias into severely non-linear response estimates (Luco and Bazzurro 2007) of interest when assessing the collapse of modern structures because it requires large scaling factors.

Other issues regarding the selection of appropriate *IM*s relate to the choice of the conditioning period, T^*, when using spectral acceleration, $Sa(T^*)$. $Sa(T_1)$ is typically adopted due to its physical meaning with respect to modal properties of a structure. However, for collapse assessment, the structure is expected to undergo significant period elongation when it approaches collapse. As such, the physical meaning of this *IM* loses relevance and other aspects such as spectral shape become more important. Kohrangi et al. (2017) noted that $Sa(1.5T_1)$ may be a better choice of *IM* for collapse assessment because this spectral ordinate better correlates with the structural response near collapse. However, $Sa(1.5T_1)$, is not as efficient as $Sa(T_1)$ for response prediction at the linear state of the structure. This complexity makes the choice for a good conditioning period (i.e., T^*) more difficult. Lin et al. (2013b) highlighted the relative insensitivity of risk-based decisions, such as mean annual rate of collapse, on the choice of T^*. They also note that a relatively poor choice of T^* can result in quite inefficient response prediction. To address the need to provide reasonably efficient predictions across the entire range of structural response, Kohrangi et al. (2017) proposed the use of *AvgSa* as a good comprise. *AvgSa* possesses many positive aspects for structural assessment with respect to other advanced *IM*s. *AvgSa* is defined as the geometric mean of the spectral accelerations within a user-specified range, as follows:

$$AvgSa = \left[\prod_{\mathrm{i}=1}^{\mathrm{n}} Sa(T_{\mathrm{i}})\right]^{1/\mathrm{n}} \quad \text{for } T_{\mathrm{i}} \in [T_{\mathrm{lower}}, T_{\mathrm{upper}}] \tag{1}$$

Kohrangi et al. (2017) subsequently note that, among other aspects, *AvgSa* is a more favourable *IM* since a) it is more predictable (i.e., has a lower GMPE dispersion) than any of the *Sa*'s of which it is composed; b) it is a relatively efficient *IM* across the whole range of structural response; and c) requires lower ground motion scaling at collapse levels when compared to other *IM*s. As such it is a good candidate to consider here for the collapse assessment of infilled GLD RC frames since the period elongation upon infill collapse can be accounted for, in addition to the reduction in potential bias due to lower ground motion scaling factors. The number of periods and range [T_{lower}, T_{upper}] to consider is based on the user's choice and can be tailored depending on the requirements of the analysis. It was observed how the choice of a period range equally weighted between $0.2T_1$ and $1.5T_1$ is a reasonable choice since it considers the spectral values closer to the higher modes of vibration, as well as those in the "inelastic first mode" when the structure approaches collapse.

3. CASE STUDY APPLICATION

3.1 Building typologies and modelling assumptions

In this study, five case study RC frame typologies with 2, 3, 4, 6 and 9 storeys were considered. The structures, adopted from a previous study by O'Reilly and Sullivan (2017b), have been designed for gravity load only, as specified in Regio Decreto 2229/39 (1939), along with other construction conventions common during that time in Italy. The frames were numerically modelled using OpenSees (McKenna et al. 2010) adopting the modelling recommendations of O'Reilly and Sullivan (2015, 2017a). Lumped plasticity elements were used to model the beam and column elements, using available experimental test data on both sub-assemblies and building specimens to quantify the various modelling parameters for older GLD RC frames. These were modelled using force-based distributed plasticity elements with a modified integration scheme to result in the plasticity being concentrated at the member ends (Scott and Fenves 2006). Equivalent diagonal strut models (Crisafulli et al. 2000) were adopted to incorporate the effects of masonry infill along with the proposals of Sassun et al. (2015) for the hysteretic backbone of these struts. Beam-column joints were modelled using a zero-length hinge at the joint centres to characterise the vulnerability to brittle shear failure as a result of no transverse reinforcement in this region. Different combinations of masonry infill were also considered in order to investigate relative differences in their performance. Two infill types were considered, termed *medium* and *strong* infill, as defined in Hak et al. (2012), and were modelled uniformly distributed over the height of the structure. The impacts of infill openings or out-of-plane behaviour were not considered here but are also not expected to strongly influence the findings of this study. Strength and stiffness degradation of all structural members were modelled via their individual element definition alongside the consideration of geometric non-linearity (P-Delta effects) to model with sufficient accuracy the collapse of the structures. The first and second modal periods of vibration for the case study structures are listed in Table 1. Furthermore, a modal damping model (Chopra and McKenna 2015) was adopted by applying a constant 5% of critical damping to all modes of vibration. This damping model, when combined with the adopted beam-column formulation outlined above, has the benefit of mitigating the introduction of potential errors highlighted by Chopra and McKenna (2015).

Table 1. Modal properties of case study structures, where the first and second values correspond to the first and second modal periods of vibration, T_1 and T_2, respectively.

Typology	2 Storey	3 Storey	4 Storey	6 Storey	9 Storey
Bare Frame	0.85s/0.31s	1.22s/0.43s	1.52s/0.52s	1.97s/0.70s	2.72s/0.99s
Infilled Frame (Medium Masonry)	0.19s/0.08s	0.29s/0.11s	0.35s/0.13s	0.48s/0.18s	0.74s/0.26s
Infilled Frame (Strong Masonry)	0.15s/0.08s	0.21s/0.08s	0.29s/0.11s	0.41s/0.15s	0.65s/0.23s

3.2 Adopted IMs for collapse assessment

For the present study, $Sa(T_1)$ and *AvgSa* were adopted as candidate *IM*s. $Sa(T_1)$ is a building-specific *IM* and it corresponds to the spectral acceleration at the first modal period of vibration of the building of interest. *AvgSa* is herein defined by Equation 1 where different upper and lower bounds on the period range [T_{lower}, T_{upper}] are considered in its definition in order to evaluate the effectiveness of each option in response prediction. The choice of period range for *AvgSa* can be defined differently to better suit the response prediction of one or multiple-like buildings (although with varying properties) and, therefore, to be either a building-specific or a generic *IM*. Both definitions were adopted herein and the following sections describe the chain of thoughts regarding their characterisations.

- *Building-specific AvgSa*

Kazantzi and Vamvatsikos (2015), followed by Kohrangi et al. (2017), interchangeably used ranges of [T_{lower}, T_{upper}]=[$0.2T_1$, $1.5T_1$] or [T_2, $1.5T_1$] for seismic response prediction of modern ductile RC frames with no masonry infills. Infilled GLD RC frames, however, are expected to possess a short initial period due to the strut action of the infills (see Table 1), followed by a significant period elongation due to the infill collapse at low drift levels in the critical storey. The latter is followed by a non-ductile RC frame mechanism because of the loss of additional stiffness of the infill. This means that the GLD frames' first modal period primarily (and significantly) lengthens due to the infill collapse and is subsequently further lengthened due to actual RC frame damage at higher drift demands. The optimum period range for a building-specific *AvgSa* as an efficient response predictor for GLD frames, therefore, requires additional investigation compared to bare frame buildings. For each of these case study building examples and based on their static pushover (SPO) analyses (O'Reilly and Sullivan 2017b), the critical storey with highest relative displacement was identified. To estimate the infilled frames' period elongation due to infill collapse, $T_{1,\text{CollapsedInfill}}$, each infilled frame was temporarily modified by removing the infill element of the pre-identified critical storey. $T_{1,\text{CollapsedInfill}}$ was therefore assumed to approximately represent the first modal period of the infilled frames upon the collapse of the masonry infill. Note that the first modal period of the bare frame was not considered as a candidate estimate for $T_{1,\text{CollapsedInfill}}$ because this option would have assumed that the collapse of all the infills in all storeys occurs before the collapse of the then bare frame, which is very likely not the case. Table 2 lists the values of $T_{1,\text{CollapsedInfill}}$ obtained as a ratio of T_1.

Table 2. First mode period ratios, $T_{1,\text{CollapsedInfill}}/T_1$, for infilled frames where masonry infill struts have been removed at the critical storey identified via SPO analysis.

Typology	2 Storey	3 Storey	4 Storey	6 Storey	9 Storey
Infilled Frame (Medium Masonry)	3.33	2.68	2.75	1.87	1.56
Infilled Frame (Strong Masonry)	4.09	3.64	2.53	2.10	1.68

Examining the $T_{1,\text{CollapsedInfill}}/T_1$ ratios for all five cases study buildings suggests that the initial periods of the infilled frames are approximately doubled after collapse of the masonry infills at the critical storey. Combining this primary source of period elongation with the elongation due to the RC frame damage resulted in an upper bound period of T_{upper} =1.5·($T_{1,\text{CollapsedInfill}}$)=1.5·(2.0·$T_1$)=3.0. In the case of the lower bound period, T_{lower}, the second mode period was analysed in a similar fashion leading to an approximate value of T_{lower} =1.2·T_2. Therefore, a period range of [1.2T_2, 3.0T_1] was used as building-specific *AvgSa* for the GLD infilled frames. Note that, similar to the previous studies, a period range of [T_2, 1.5T_1] was adopted for the bare frames of this study. Therefore, a total of 5 structures x 3 variants per building x 2 *IM* per variant = 30 building-specific *IM*s in terms of *Sa*(T_1) and *AvgSa* were considered herein.

- *Generic AvgSa*

To investigate the use of *AvgSa* as an *IM* for a group of like buildings in a more portfolio-oriented assessment, rather than building-specific, a number of generic *IM*s were also investigated. Three varieties were chosen including: (a) height-specific, where the same *IM* can be used for all typologies of the same storey number; (b) typology-specific, where the complete group of bare or infilled RC frames can be assessed using the same *IM*; and lastly, (c) a generic *IM* that can be used for all case study structures considered. The period ranges for these generic *IM*s were selected using the limiting period ranges in each case. For example, the period range of the generic *IM* used for all infilled frames was selected by taking the minimum and maximum value of 1.2T_2 and 3.0T_1, respectively, for all infilled frames. Therefore, a total of 8 generic *IM*s were considered: 5 storey-specific *IM*s for each building height considered, 2 typology-specific *IM*s for either bare or infilled frames (no distinction between weak and strong walls) and 1 *IM* used for all buildings.

To summarise, the complete set of 38 building-specific and generic *IM*s are listed in Table 3.

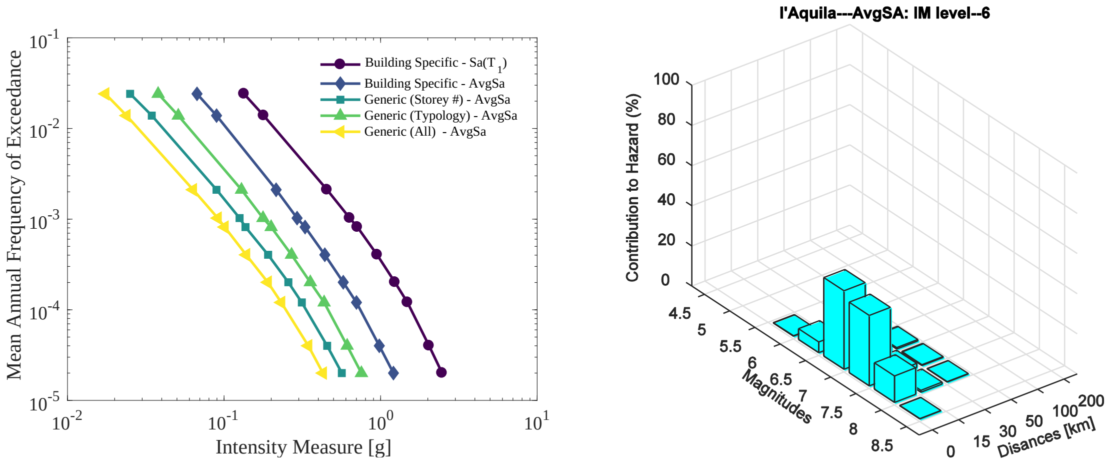
Table 3. Complete set of 38 building-specific and generic *IM*s adopted in this study

IM type	*IM* name	Notation	Period range
Building specific *IM*s	Building Specific- *Sa*(T_1)	*Sa*(T_1)	-
	Building Specific- *AvgSa*	*AvgSa*	Bare frames: T_2 to 1.5T_1 Infilled frames: 1.2T_2 to 3T_1
Generic *IM*s	Generic (Storey #) - *AvgSa*	*AvgSa*	Minimum to maximum of *AvgSa*'s period range for all same-height buildings
	Generic (Typology) - *AvgSa*	*AvgSa*	Minimum to maximum of *AvgSa*'s period range for all same-typology buildings
	Generic - *AvgSa*	*AvgSa*	Minimum to maximum of *AvgSa*'s period range for all buildings

Note: the periods for the computation of *AvgSa* have a 0.1s spacing in the corresponding period range

4. HAZARD ANALYSIS AND GROUND MOTION SELECTION

A site on rock (i.e. Vs_{30} = 800 m/s) at a latitude and longitude of [42.35°, 13.40°] was selected in the Italian city of L'Aquila. The OpenQuake engine (Monelli et al. 2012), which is an open-source software for seismic hazard and risk assessment developed by the Global Earthquake Model (GEM) Foundation, was used to perform the seismic hazard computations. The analysis was based on the SHARE Project (Woessner et al. 2015) area source model and the GMPE proposed by Boore and Atkinson (2008). All the seismic sources within 200km from the considered site were used for the PSHA computations. Figure 1 shows one example for the hazard curves and disaggregation analysis computed for the 6-storey infilled frame (strong masonry).



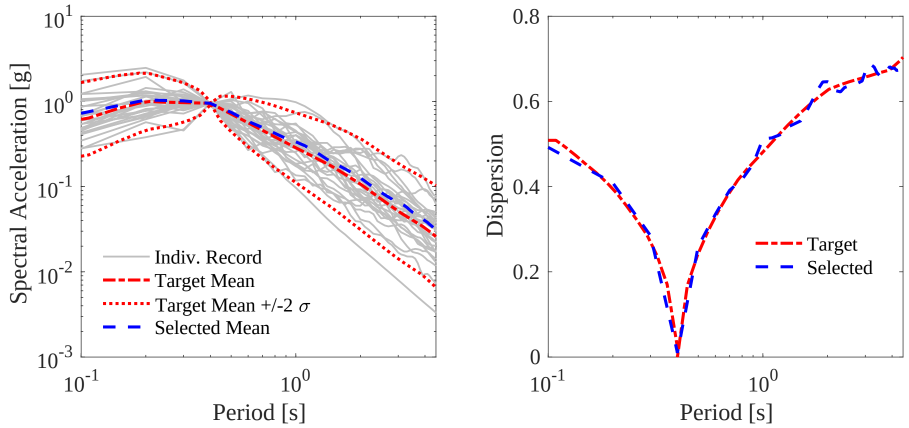
(a) (b)

Figure 1. (a) Site hazard curves for the different *IMs* adopted for the 6-storey infilled frame (strong masonry) case study structure, where the *AvgSa IM* definitions exhibit lower intensities for a fixed value of MAFE; (b) Disaggregation analysis for building specific *AvgSa* corresponding to 2% in 50 years for the same structure.

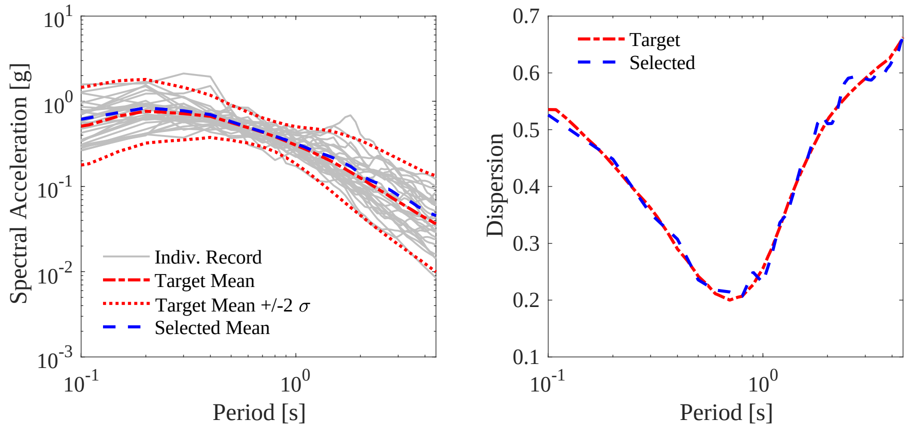
Multiple stripe analysis (MSA) was performed for 10 return periods corresponding to probabilities of exceedance of 70%, 50%, 10%, 5%, 4%, 2%, 1%, 0.6%, 0.2%, 0.1% in 50 years. For each *IM* level, disaggregation analysis was performed in terms of magnitude and distance. Consequently, 30 ground motion records were selected from the NGA-West1 database for each *IM* level and scaled using the conditional spectrum (CS) approach to collectively match the entire distribution of the CS for $Sa(T_1)$, i.e. CS($Sa(T)$) and *AvgSa*, i.e. CS(*AvgSa*). The original ground motion selection algorithm developed for CS($Sa(T)$) by Jayaram et al. (2011) and its extended version for CS(*AvgSa*), by Kohrangi et al. (2017), were used. In addition, the selection was based on the 'approximate' method of CS (Lin et al. 2013a) using the mean of the contributing scenarios obtained from disaggregation analysis. Ground motion scaling factors were limited to 4.0 during the selection and the Boore and Atkinson (2008) GMPE and Baker and Jayaram (Baker and Jayaram 2008) correlation coefficient model were used to generate the CS target spectrum. Figure 2 shows an example of the target and selected records based on building-specific CS($Sa(T_1)$) and CS(*AvgSa*) for the 6-storey infilled frame (strong masonry) at *IM* level 6, corresponding to 2% probability of exceedance in 50 years.

5. ANALYSIS RESULTS

The case study structures were analysed using the ground motion sets selected for each *IM* variation outlined above. Since the aforementioned ground motion selection algorithms produce two orthogonal pairs of acceleration time-histories and the case study structures are represented by two-dimensional planar models, a simplification was required with regards to which component to apply. To account for this, the suggestions of Baker and Cornell (2006b) were followed whereby for each ground motion pair, the acceleration history applied to the structure was randomly selected from the two orthogonal components. Lastly, since this study focuses on the collapse assessment of GLD RC frame structures, a quantitative definition of collapse was needed so as to systematically separate the collapse from the non-collapse cases at each *IM* level. A value of 10% maximum peak storey drift (MPSD) was adopted based on the findings of O'Reilly et al. (2018) as the transient drift limit beyond which it can be confidently assumed that the structure has indeed collapsed. Note that non-converging runs (with transient displacement less than 10%) were also considered as collapse cases. In the following sub-sections the results obtained from this large response history analysis are described.



(a) Conditional Spectrum for Sa(T_1)



(b) Conditional Spectrum for AvgSa

Figure 2. Selected records and the target CS (i.e., mean +/- 2σ) for 6-storey infilled frame (strong masonry) at the 2% in 50 years hazard level for: (a) CS($Sa(T_1)$) and (b) building-specific CS(*AvgSa*). Note: T_1=0.4s and *AvgSa* is defined in the [0.2s, 1.2s] period range, which are shaded in red in the respective plots.

5.1. Drift demand evaluation

Figure 3 illustrates the MPSD's median and fractiles at increasing *IM* return periods of one selected structures. For an infilled RC frame, Figure 3 shows how $Sa(T_1)$ possesses, as expected, a notably lower dispersion compared to the building-specific *AvgSa* at lower return period ground motions, but with increasing return periods, the dispersion – seen through the relative width of the 16$^{\%}$ and 84% fractiles - tends to increase for $Sa(T_1)$ whereas *AvgSa* is remains more stable. As expected, this confirms that $Sa(T_1)$ is more efficient at lower return period ground motion *IM* when the structures remain relatively elastic and, therefore, their responses are well correlated to the first-mode properties. For increasing return periods, however, *AvgSa* becomes a more favourable *IM*.

5.2. Intensity-based collapse assessment

Counting the number of collapses and dividing by the total number of analyses, the probability of collapse was computed at each return period stripe. From this discrete distribution of collapse probability versus intensity, the median collapse intensity and dispersion due to record-to-record variability, assuming a lognormal distribution, were quantified using a log-likelihood method of fitting to the truncated data obtained from MSA (Baker 2015). The Kolmogorov-Smirnov (KS) goodness-of-fit test did not reject the lognormality of the data. Figure 4 shows and compares the fragility curve parameters of the case study models based on different *IM*s of this study. It can be seen how the collapse dispersion of the building-specific *AvgSa IM* is much lower than $Sa(T_1)$ for each of the infilled RC frames, whereas they are of similar magnitude for the bare frames. The impact of masonry infill presence on the median collapse intensity is also apparent, in addition to the influence of increasing building height.

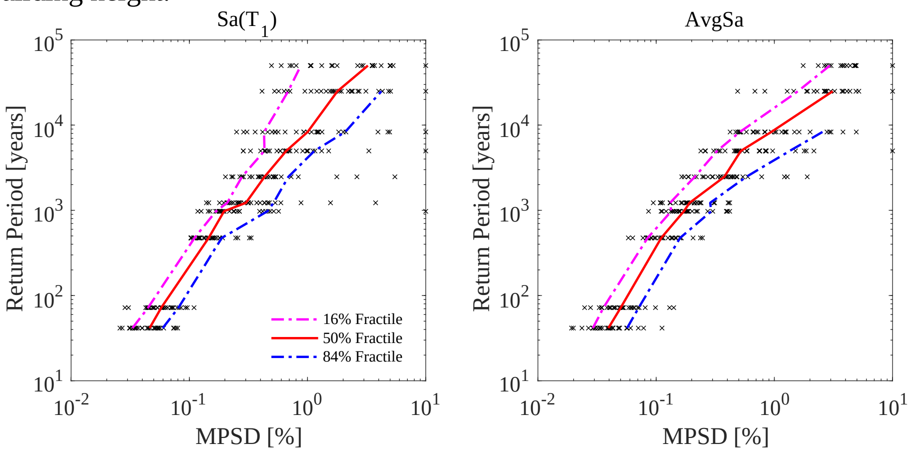


Figure 3. Analysis results for the 6 storey infilled frame (strong masonry) for the building specific *IM*s. Note: each point corresponds to a single analysis and the intensities with collapses have been marked at the limiting value of 10%.

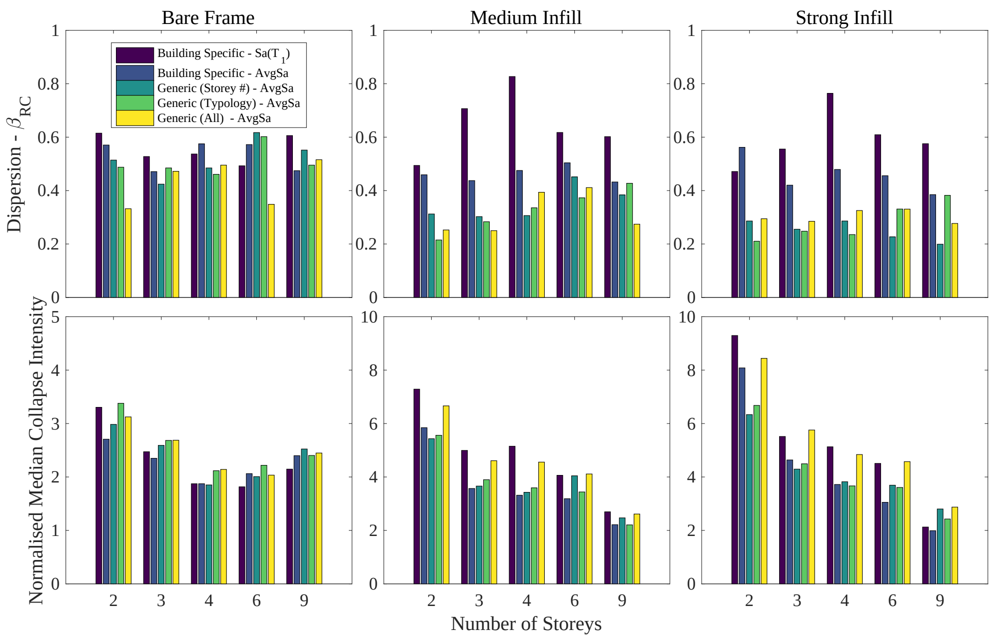


Figure 4. Collapse fragility function parameters for each of the case study structures and *IM*. Dispersion values incorporate record-to-record variability, β_{RC}, only and median collapse intensities have been normalised with respect to the 2475-year return period *IM* value.

One of the features of the generic *AvgSa* is that a common *IM* allows an easy comparison between different buildings since it is not tied to any single structure's properties. For instance, $Sa(T_1)$ for the bare and infilled frames of the same building example are quite different quantities (see Table 1), which renders impossible to compare the fragility curves of the two models. On the other hand, a generic *AvgSa* is relatively sufficient for both models and can be freely used to compare their fragility curves. For example, Figure 5(a) compares the collapse fragility of all the buildings in this study using the generic *AvgSa* and some trends are more apparent. For example, the bare frames are the most vulnerable to collapse among the different typologies and collapse vulnerability generally increases with height for a given typology. This latter aspect is also further illustrated in Figure 5(b)-(d) for the typology-based *IM*s. Furthermore, the results in Figure 5(e)-(i) suggest that while the performance of 2 to 6 storey infilled frames is highly improved (i.e., larger collapse capacity) when compared to their corresponding bare frame, this improvement is limited for the 9 storey building. This limited impact may be attributed to the reduced influence of the infill on the global response since when examining Table 2, the period elongation due to the infill collapse at the critical storey tended to decrease with increasing storey number, which demonstrates a limited impact on the overall dynamic response.

These observations highlight how more meaningful comparisons can be made between the different structures that are found in a portfolio of buildings via their fragility functions on just one plot by using these generic *AvgSa*-based *IM*s. Historically, the most commonly used generic *IM* in portfolio loss estimation is *PGA* since it is not tied to one specific building's properties and it is defined relatively simply. However, the use of *PGA* is now widely known to be poorly correlated with building response when compared with *Sa*(*T*), for example (Shome et al. 1998). Therefore, the development of the generic *AvgSa* outlined here is rather beneficial as it takes into account the properties of the entire group of structures and maintains a comparable level of efficiency in response prediction compared to other building-specific *IM*s, also illustrated in Figure 3 to Figure 4.

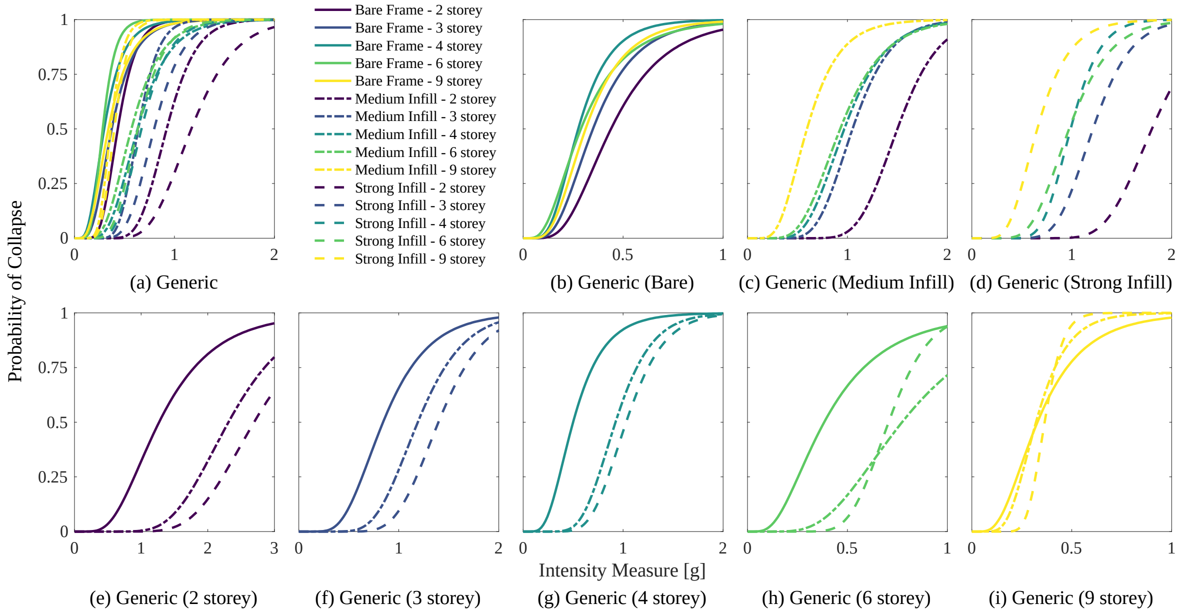


Figure 5. Collapse fragility functions for each generic *IM* definition, where (a) shows the generic *AvgSa* definition for all structures; (b) to (d) show the typology-based definition and (e) to (i) illustrate the height-based definition.

5.3. Risk-based collapse assessment

Using the fragility analysis results quantified in Section 5.2 convoluted with the hazard curves shown in Figure 1(a), risk-based quantities of seismic performance can also be computed. The simplest of these is the mean annual frequency (MAF) of collapse, $\lambda_{collapse}$, obtained by integrating the fitted collapse fragility with the hazard curve, λ, based on Equation 2:

$$\lambda_{collapse} = \int P[C|IM] \left|\frac{d\lambda}{dIM}\right| dIM \qquad (2)$$

where $P[C|IM]$ represents the probability of collapse for a given *IM* level. The results obtained for $\lambda_{collapse}$ are plotted in bar charts of Figure 6 for each case study structure. Note that while the effects of modelling uncertainty have not been accounted for in this study, these effects are expected to increase the current $\lambda_{collapse}$ values reported herein but they do not affect the overall conclusions of the work. The interested reader is referred to the work of O'Reilly and Sullivan (2017) for further details. An immediate observation regarding the performance of these structures is that the bare frames tend to have much higher rates of collapse than the infilled frames. This result could also be inferred by inspecting the results of the generic *IM*s in Figure 5. Furthermore, the overall magnitude of the $\lambda_{collapse}$ tends to increase with height for the infilled frames, reflecting their increased collapse vulnerability that was also observed in Figure 5 for the generic *AvgSa* used for all structures.

With regards to the reduced median collapse intensity for infilled RC frames when using *AvgSa* seen in Figure 4, this may be initially interpreted as an increased collapse vulnerability. However, as illustrated in Figure 1, this typically arises due to the lower value of intensity for a fixed MAFE among the *IM*s. Upon integrating these collapse fragilities with their respective hazard curves using Equation 2, some risk-consistent conclusions can be drawn. In fact, it is seen how the $\lambda_{collapse}$ tends to be lower and more consistent for *AvgSa* than $Sa(T_1)$ for the infilled RC frames. This increased $\lambda_{collapse}$ is a reflection of the additional amplification introduced via the reduced efficiency of $Sa(T_1)$ and can be seen more clearly when using the closed-form definition outlined in Cornell et al. (2002):

$$\lambda = \lambda\left(\widehat{IM}\right)\exp\left(0.5k^2\beta_{RC}^2\right) \quad (3)$$

where λ represents the MAFE of a limit state such as collapse, $\widehat{IM}$ represents the median intensity, k represents the slope of the hazard curve in the neighbourhood of $\widehat{IM}$, and β_{RC} represents the collapse fragility dispersion due to the record-to-record variability. From Equation 3, it can be seen that for a fixed rate of median intensity, an increased dispersion due to a reduced efficiency of the *IM* results in an overall increase and conservative estimation of $\lambda_{collapse}$. The work discussed here illustrates how this conservatism may be reduced using more efficient IMs to result in more accurate estimations of collapse risk.

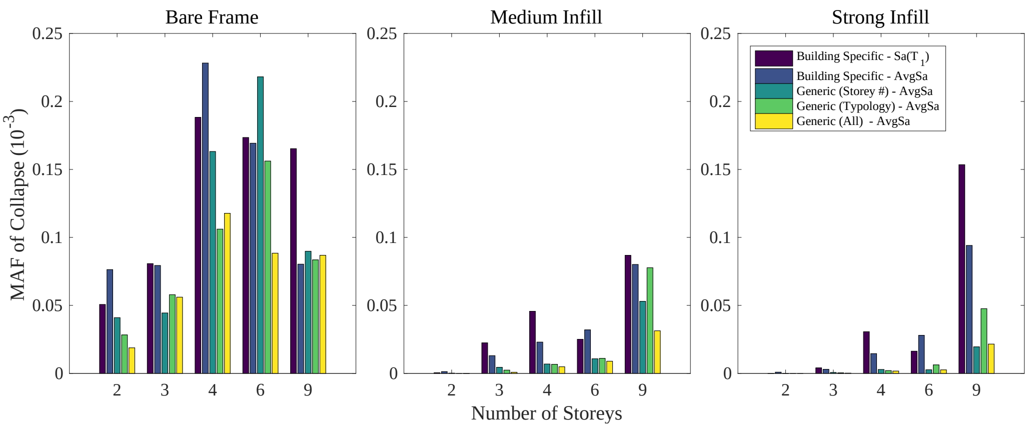


Figure 6. Mean annual frequency of collapse, $\lambda_{collapse}$, for each of the case study structures and *IM*.

6. CONCLUSIONS

This paper discussed the collapse assessment of gravity load designed (GLD) frames with masonry infill using different *IM* definitions. It was seen how by using a more flexible *IM* definition, i.e. *AvgSa*, both building-specific and generic *IM*s can be developed to assess the performance of both single and groups of buildings. This exercise was carried out with reference to a common *IM* currently used in seismic assessment, $Sa(T_1)$, as a reference. Based on the collapse assessment results presented here, some observations can be made regarding the use of different *IM*s when characterising the collapse performance of infilled GLD RC frames. These include:

- The collapse fragility dispersion is generally lower for *AvgSa* than $Sa(T_1)$. While this fact alone does not necessarily indicate that *AvgSa* is a better predictor of collapse than $Sa(T_1)$ because the variability may have simply been moved elsewhere, Kohrangi et al. (2017) noted how the *AvgSa* GMPE dispersion is in fact lower meaning that the total uncertainty is indeed reduced. Furthermore, it was noted that the $Sa(T_1)$ dispersion in drift demands was typically lower than *AvgSa* at lower return periods, where the structure would be expected to have limited non-linear behaviour, but for higher return periods where extensive damage is expected, *AvgSa* is a superior predictor of infilled GLD RC frame response.
- It was shown that by using generic definitions of *AvgSa*, more meaningful and hazard-consistent comparisons can be made between groups of structures. This was demonstrated here for all of the structures collectively, in addition to further subdivisions based on structural typology and storey number. This approach has the benefit of being applicable to large groups of structures typical of a regional assessment. The use of *AvgSa* was shown to maintain a comparable level of response

prediction efficiency when compared to the other examined building-specific *IM*s. This represents an improvement compared to other traditional *IM*s like *PGA* that are generic enough to be used for all structures, but are known to be poorly correlated to structural response of most buildings.

- When examining the risk-based quantities for each case study structure using the different *IM*s trialled, it was seen how many of the trends previously identified using the generic *AvgSa* were confirmed. It was also noted how the mean annual frequency of collapse, $\lambda_{collapse}$, computed using $Sa(T_1)$ was typically larger than that computed using the different definitions of *AvgSa*. While it is not possible to know the correct value of $\lambda_{collapse}$, the relative consistency between the different *AvgSa*s implies that it most likely lies in that range. The conservatism of $Sa(T_1)$ was attributed to the reduced efficiency of the *IM* in collapse assessment of the GLD RC frames illustrated via its larger dispersion.

7. ACKNOWLEDGEMENTS

The funding provided by the ITERATE project is gratefully acknowledged.

8. REFERENCES

Baker JW (2015) Efficient Analytical Fragility Function Fitting Using Dynamic Structural Analysis. *Earthquake Spectra*, 31(1), 579–599. doi: 10.1193/021113EQS025M.

Baker JW, Cornell CA (2006a) Spectral shape, epsilon and record selection. *Earthquake Engineering & Structural Dynamics*, 35(9), 1077–1095. doi: 10.1002/eqe.571.

Baker JW, Cornell CA (2006b) Which Spectral Acceleration Are You Using?. *Earthquake Spectra*, 22(2), 293–312. doi: 10.1193/1.2191540.

Baker JW, Jayaram N (2008) Correlation of Spectral Acceleration Values from NGA Ground Motion Models. *Earthquake Spectra*, 24(1), 299–317. doi: 10.1193/1.2857544.

Bojórquez E, Iervolino I (2011) Spectral shape proxies and nonlinear structural response. *Soil Dynamics and Earthquake Engineering*, 31(7), 996–1008. doi: 10.1016/j.soildyn.2011.03.006.

Boore DM, Atkinson GM (2008) Ground-Motion Prediction Equations for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA at Spectral Periods between 0.01 s and 10.0 s. *Earthquake Spectra*, 24(1), 99–138. doi: 10.1193/1.2830434.

Chandramohan R, Baker JW, Deierlein GG (2016) Quantifying the influence of ground motion duration on structural collapse capacity using spectrally equivalent records. *Earthquake Spectra*, 32(2), 927–950. doi: https://doi.org/10.1193/122813EQS298MR2.

Chopra AK, McKenna F (2015) Modeling viscous damping in nonlinear response history analysis of buildings for earthquake excitation. *Earthquake Engineering & Structural Dynamics*, doi: 10.1002/eqe.2622.

Cornell CA, Jalayer F, Hamburger RO, Foutch DA (2002) Probabilistic Basis for 2000 SAC Federal Emergency Management Agency Steel Moment Frame Guidelines. *Journal of Structural Engineering*, 128(4), 526–533. doi: 10.1061/(ASCE)0733-9445(2002)128:4(526).

Crisafulli FJ, Carr AJ, Park R (2000) Analytical Modelling of Infilled Frame Structures - A General Review. *Bulletin of the New Zealand Society for Earthquake Engineering*, 33(1), 30–47.

Eads L, Miranda E, Lignos DG (2015) Average spectral acceleration as an intensity measure for collapse risk assessment. *Earthquake Engineering & Structural Dynamics*, 44(12), 2057–2073. doi: 10.1002/eqe.2575.

Eads L, Miranda E, Lignos DG (2016) Spectral shape metrics and structural collapse potential. *Earthquake Engineering & Structural* Dynamics, 45(10), 1643–1659. doi: 10.1002/eqe.2739.

Hak S, Morandi P, Magenes G, Sullivan TJ (2012) Damage Control for Clay Masonry Infills in the Design of RC Frame Structures. *Journal of Earthquake Engineering*, 16(Supp1), 1–35. doi: 10.1080/13632469.2012.670575.

Jayaram N, Lin T, Baker JW (2011) A Computationally efficient ground-motion selection algorithm for matching a target response spectrum mean and variance. *Earthquake Spectra*, 27(3), 797–815. doi: 10.1193/1.3608002.

Kazantzi AK, Vamvatsikos D (2015) A Next Generation Scalar Intensity Measure for Analytical Vulnerability Studies. *COMPDYN 2015 - 5th ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering*. Crete Island, Greece.

Kazantzi AK, Vamvatsikos D (2015) Intensity measure selection for vulnerability studies of building classes. *Earthquake Engineering & Structural Dynamics*, 44(15), 2677–2694. doi: 10.1002/eqe.2603.

Kohrangi M, Bazzurro P, Vamvatsikos D (2016) Vector and Scalar IMs in Structural Response Estimation: Part II – Building Demand Assessment. *Earthquake Spectra*, 32(3), 1525–1543. doi: 10.1193/053115EQS081M.

Kohrangi M, Bazzurro P, Vamvatsikos D, Spillatura A (2017) Conditional spectrum-based ground motion record selection using average spectral acceleration. *Earthquake Engineering & Structural Dynamics*. doi: 10.1002/eqe.2876.

Kohrangi M, Vamvatsikos D, Bazzurro, P (2016) Implications of Intensity Measure Selection for Seismic Loss Assessment of 3-D Buildings. *Earthquake Spectra*, 32(4), 2167–2189. doi: 10.1193/112215EQS177M.

Lin T, Haselton CB, Baker JW (2013a) Conditional spectrum-based ground motion selection. Part I: Hazard consistency for risk-based assessments. *Earthquake Engineering & Structural Dynamics*, 42(12), 1847–1865. doi: 10.1002/eqe.2301.

Lin T, Haselton CB, Baker JW (2013b) Conditional spectrum-based ground motion selection. Part II: Intensity-based assessments and evaluation of alternative target spectra. *Earthquake Engineering & Structural Dynamics*, 42(12), 1867–1884. doi: 10.1002/eqe.2303.

Luco N, Bazzurro P (2007) Does amplitude scaling of ground motion records result in biased nonlinear structural drift responses?. *Earthquake Engineering & Structural Dynamics*, 36(13), 1813–1835. doi: 10.1002/eqe.695.

McKenna F, Scott MH and Fenves, GL (2010) Nonlinear Finite-Element Analysis Software Architecture Using Object Composition. *Journal of Computing in Civil Engineering*, 24(1), 95–107. doi: 10.1061/(ASCE)CP.1943-5487.0000002.

Monelli D, Pagani M, Weatherill G, Silva V, Crowley H (2012) The hazard component of OpenQuake: The calculation engine of the Global Earthquake Model. *15th World Conference on Earthquake Engineering*. Lisbon, Portugal.

O'Reilly GJ, Sullivan TJ (2015) Influence of modelling parameters on the fragility assessment of pre-1970 Italian RC structures. *COMPDYN 2015 - 5th ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering*, Crete Island, Greece.

O'Reilly GJ, Sullivan TJ. (2017a) Modelling Techniques for the Seismic Assessment of Existing Italian RC Frame Structures. *Journal of Earthquake Engineering*. doi: 10.1080/13632469.2017.1360224.

O'Reilly GJ, Sullivan TJ (2017b) Probabilistic seismic assessment and retrofit considerations for Italian RC frame buildings. *Bulletin of Earthquake Engineering*. 16(3), 1447-1485, doi: 10.1007/s10518-017-0257-9.

O'Reilly GJ, Sullivan TJ (2017c) Quantification of Modelling Uncertainty in Existing Italian RC Frames. *Earthquake Engineering & Structural Dynamics*. doi: 10.1002/eqe.3005.

O'Reilly GJ, Sullivan TJ, Monteiro R (2018) On the seismic assessment and retrofit of infilled RC frames structures. *16th European Conference on Earthquake Engineering*. Thessaloniki, Greece.

Orumiyehei A, Kohrangi M, Bazzurro P (2017) Seismic Performance of 3-D Infilled and Bare Frame RC Building Models using Average Spectral Acceleration. *EURODYN 2017 - X International Conference on Structural Dynamics*. Rome, Italy.

Regio Decreto (1939) Norme per l'esecuzione delle opere conglomerato cementizio semplice od armato - 2229/39. Rome, Italy (in Italian).

Sassun K, Sullivan TJ, Morandi P, Cardone D (2015) Characterising the In-Plane Seismic Performance of Infill Masonry. *Bulletin of the New Zealand Society for Earthquake Engineering*, 49(1).

Scott MH, Fenves GL (2006) Plastic Hinge Integration Methods for Force-Based Beam–Column Elements. *Journal of Structural Engineering*, 132(2), 244–252. doi: 10.1061/(ASCE)0733-9445(2006)132:2(244).

Shome N, Cornell CA, Bazzurro P, Carballo JE (1998) Earthquakes, Records, and Nonlinear Responses. *Earthquake Spectra*, 14(3), 469–500. doi: 10.1193/1.1586011.

Woessner J, Laurentiu D, Giardini D, Crowley H, Cotton F, Grünthal G, Valensise G, Arvidsson R, Basili R, Demircioglu MB, Hiemer S, Meletti C, Musson RW, Rovida AN, Sesetyan K, Stucchi M (2015) The 2013 European Seismic Hazard Model: key components and results. *Bulletin of Earthquake Engineering*, 13(12), 3553–3596. doi: 10.1007/s10518-015-9795-1.