

ON THE SEISMIC ASSESSMENT AND RETROFIT OF INFILLED RC FRAME STRUCTURES

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ABSTRACT

The seismic assessment of existing buildings has developed from traditional measures of performance that focused on ensuring life safety to incorporating the potential economic losses incurred over time. This paper considers the assessment of existing reinforced concrete (RC) frame buildings, typically load designed before the introduction of seismic design provisions around the 1970's in Italy, and across the Mediterranean area. Issues on their seismic response quantification are first discussed, which include the development of collapse fragility functions and dispersion in structural response needed for loss estimation. The collapse fragility functions are compared to existing damage scales describing complete structural collapse and it is seen how by following a number of key guidelines proposed here, collapse fragility functions may be numerically developed in a relatively efficient and unequivocal manner. The level of dispersion in the drift response of infilled frames is investigated and is shown to be very sensitive to the location of infill failure and subsequent soft storey mechanism. Further discussion highlights how the choice of the intensity measure may also be a contributing factor that could be investigated in the future to provide more efficient estimates of structural response. Aspects relating to the loss estimation and retrofitting are discussed with the purpose of giving readers further insight on some of the potential impacts of different retrofitting decisions for infilled RC frames within the context of mitigating economic losses. Lastly, issues regarding the implementation of a seismic performance classification framework are highlighted and potential future refinements are outlined.

Keywords: Non-ductile; Collapse; Dispersion; Loss Estimation; Retrofitting.

1. INTRODUCTION

In recent decades, the seismic assessment of existing buildings has undergone significant developments, where traditional objectives focusing on life safety have been expanded to incorporate other performance measures such as economic loss. This became apparent following the 1994 Northridge earthquake in the US, where despite relatively few lives being lost, the economic impact and disruption caused suggested that a more comprehensive approach was needed. The "Vision 2000" document (SEAOC 1995) instigated such a change through the initiation of performance-based earthquake engineering (PBEE). This relates expected levels of building performance to different levels of earthquake intensity, considering not only life safety performance but the building functionality as a whole. This was developed in a more probabilistic framework in the early 2000's and the PEER PBEE methodology (Cornell and Krawinkler 2000) emerged, representing a powerful methodology to quantify building performance, as it not only considers the building life safety, but also the structural and non-structural damage in a probabilistic consistent manner. Through this methodology, more meaningful and risk-consistent metrics can be employed to quantify the overall building performance and subsequently communicate risk to stakeholders in a more effective way.

The seismic behaviour of existing reinforced concrete (RC) structures across the Mediterranean area has been the focus of much research in recent years. Prior to the introduction of seismic design provisions in the 1970's in Italy, for example, structural design was primarily based on allowable stresses for vertical gravity loads only and no consideration was typically made of seismic design forces or strength hierarchies. These typologies are referred to herein as gravity load designed (GLD) frames. In this respect, GLD RC frames are extremely vulnerable, as witnessed during numerous past earthquakes in southern Europe (e.g. (Psycharis et al. 1999;

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Augenti and Parisi 2010). To reduce their seismic vulnerability, the assessment and retrofiting of these systems represent an important task to be tackled in coming years, considering they constitute a large portion of the building stock in earthquake-prone countries such as Italy, Greece and Turkey.

Seismic assessment involves, among many other aspects, methods with which the likely structural response can be quantified in an efficient and unequivocal manner in tandem with guidelines to quantify the relative seismic performance of buildings to aid intervention and retrofit decisions. These aspects are also discussed in this paper, where Section 2 examines ways in which the collapse fragility can be established analytically and provides comparisons with empirical data from past earthquakes and experimental testing in order to highlight the state-of-the-art and provide some guidance for future research needs. Furthermore, the distribution of these infilled RC frames' response with respect to record-to-record variability is also discussed, whereby a key structural behaviour characteristic is shown to have an adverse effect on the median drift estimate's efficiency; where the term efficiency refers to the variability in structural response. Some typical dispersion values are presented and compared to existing values available in the literature to highlight their limitations and also ponder ways in which the impact of this drawback may be lessened in the future. Section 3 discusses aspects relating to the overall assessment and retrofit of these typologies within the context of economic losses. Issues relating to impacts of different retrofiting decisions on the overall performance of structures are outlined. Finally, Section 4 discusses the implementation of a seismic performance classification framework, where lessons learned from the recent earthquakes in New Zealand are discussed in tandem with the recent guidelines introduced in Italy. The benefits of these guidelines in relation to past difficulties encountered when implementing such guidelines are also discussed. Areas for future development are then highlighted with respect to some of the previous issues outlined in earlier sections of the paper.

2. QUANTIFICATION OF STRUCTURAL RESPONSE

As mentioned previously, seismic assessment using PEER's PBEE methodology involves quantifying the response of existing structures for increasing levels of intensity. This is typically conducted using incremental dynamic analysis (IDA) (Vamvatsikos and Cornell 2002). Four case study RC frame typologies were considered here, adopted from a previous study by O'Reilly and Sullivan (2017c), and have been designed for gravity load only, along with other common conventions for that period of construction in Italy. The numerical modelling was carried out using OpenSees (McKenna et al. 2010) by adopting the recommendations of O'Reilly and Sullivan (2017a; 2015) to capture the various structural elements' vulnerability as well as their degradation and eventual collapse. All analyses conducted here were done using IDA and the intensity measure (IM) was the spectral acceleration, $S_a(T_1)$, at the structure's first mode period of vibration, T_1 . The ground motion record set used to track the evolution of damage and response in the structure was the far-field set given in FEMA P695 (2009). For each of the structures, the collapse fragility function and uncertainty in response with increasing intensity levels is examined and some of the aspects pertaining to GLD RC frames with masonry infills are highlighted.

2.1. Collapse Fragility Function Development

One of the PEER PBEE methodology's fundamental aspects is characterising the performance of a structure in terms of what are referred to as the 3Ds: dollars, deaths and downtime. The dollars refer to the economic losses associated with the cost of repairing a damaged building and can be computed using tools such as PACT, provided within the FEMA P58 guidelines (FEMA, 2012). These losses are typically referred to as direct economic losses since they are a direct result of building damage. Indirect losses, on the other hand, stem from aspects like loss in revenue due to downtime and closure during the earthquake's aftermath. Only direct losses are considered in this discussion as they are most closely related to structural performance. In the direct losses computation, simulations resulting in both collapse and non-collapse of the structure are considered. By making such a distinction, the total expected losses, $E[L_T|IM]$, arising from both losses conditioned on collapse, or building replacement cost, $RepC$, and no collapse, $E[L_T|NC, IM]$, cases are computed as follows:

$$E[L_T|IM] = E[L_T|NC, IM](1 - P[C|IM]) + P[C|IM] \cdot RepC \quad (1)$$

where $P[C|IM]$ represents the probability of collapse for a given IM level and C and NC denote the collapse and no collapse cases, respectively. As such, the collapse performance is required for building loss estimation. Furthermore, when estimating the number of deaths in a building, the collapse performance is another required

parameter. FEMA P58 outlines how the collapse performance may be determined using IDA, or other more simplified means of analysis when applicable, and is typically expressed via a lognormal distribution characterised by a median collapse intensity and lognormal standard deviation, or dispersion. To develop such a fragility function, however, an actual definition of collapse is required. Conceptually, it is relatively easy to understand what the term collapse means but when performing IDA, a quantitative definition is often needed. FEMA P58 qualitatively defines collapse as any of the following: (a) side-sway failure; (b) vertical collapse; or (c) exceedance of a non-simulated failure mode. The term *collapse* herein refers to the complete collapse of one or more storeys and is distinguished from what may be referred to as the *complete damage* limit state. The latter refers to when the building has remained upright following an earthquake, but the excessive damage induced means that it will more than likely be demolished at a later stage. This distinction is used to avoid introducing potential bias in the collapse fragility function development through premature consideration of eventual decisions made to demolish the building. Many factors contribute to this decision-making process and a recent study by Kim et al. (2017) following the 2011-2012 New Zealand earthquakes noted how considering further parameters associated with the likelihood of demolition following the earthquake may be incorporated in future developments. Furthermore, this distinction is also needed as it plays an important role in estimating of the number of casualties due to structural collapse on a more regional level. Villar-Vega and Silva (2017) noted that the lack of distinction between complete collapse and complete damage on some damage survey forms used following past earthquakes can contribute to biased and overly conservative estimates of total number of collapses. This was seen through the discrepancies in the estimated number of collapses and recorded data from past events in different regions of South America. Since the numerical modelling utilised here accounts for strength and stiffness degradation of each structural member, it is assumed that all potential modes of collapse can be adequately accounted for, meaning that point (c) above will not be discussed further. As such, collapse is identified through the excessive deformations during numerical analysis of the structure. This is typically done using the side-sway movement of the building as this lateral deformation is demand parameter of interest in structural analysis. Therefore, the collapse performance will be discussed in terms of the side-sway building movement, although this could be extended to vertical collapse also.

Past analytical studies have typically quantified the collapse performance using robust numerical models that are capable of achieving large values of lateral displacement without much numerical difficulty through the use of convergence algorithms. This typically results in cases where structural models that have essentially exhausted their strength and stiffness capacities - a feature noted through the *flat-lining* of the individual ground motion traces in IDA - are numerically capable of achieving large deformations but have, in reality, collapsed. It is therefore desirable to have a quantitative definition of collapse in structural models so that individual simulations may be stopped when exceeded and the analysis conducted in a more efficient manner. This way, the software would not need to waste time trying to find a converging solution; for what will in any case be identified as a collapse case, when using a computationally expensive analysis procedure, relatively speaking. Such an algorithm to perform non-linear response history analysis in a more efficient manner has been implemented in OpenSees and is available at: <https://github.com/gerardjoreilly/OpenSees-Procedures>. For example, past studies (Kazantzi and Vamvatsikos 2015; O'Reilly and Sullivan 2017c) typically defined a sufficiently large lateral peak storey drift (PSD) so as to ensure that the structure had in fact collapsed without the need to individually check the various structural members' chord rotations etc. during post-processing. Values of up to 10% PSD were typically adopted and were justified using an IDA plot (Figure 1) to show that beyond a certain value of PSD, the structure could not sustain any further increase in intensity and resulted in the IDA trace approaching its limiting intensity for that individual record. While this approach is reasonable and some algorithms are available (Vamvatsikos and Cornell 2002) to reach these limiting intensities for each ground motion as economically as possible, some quantitative estimate of a collapse drift limit, θ_{CDL} , is still required *a priori*.

In the context of European buildings, Rossetto and Elnashai (2003) provided a quantitative definition of different damage states including collapse. By compiling empirical data from numerous past earthquakes, they outlined a homogenised scale of damage ranging from 0 to 100 for different structural typologies, noting the degree of ambiguity in existing guidelines in this regard. They subsequently outlined a number of expressions to determine the damage state as a function of the PSD, citing numerous experimental tests available at the time. In the case of non-ductile RC frames, recent experimental testing on non-ductile RC columns (Melo et al. 2015b) and beam-column joints (Melo et al. 2015a) typical of older European practice further demonstrated the adequacy of this damage scale initially proposed by Rossetto and Elnashai (2003). Using this qualitative damage scale and expressions with which to quantitatively estimate a drift associated with each level of

damage, the level of drift beyond which collapse is to be expected is noted as 4.36% for infilled RC frames. This collapse drift limit proposed by Rossetto and Elnashai (2003) is based on interpretation of experimental test results and its use in deciphering collapse cases was numerically evaluated here. To do this, IDA results were examined by gradually increasing this θ_{CDL} definition from a value of 0.75% up to 10%. By varying the quantitative definition of collapse, at which point the analysis was stopped, the impact of θ_{CDL} on the collapse fragility function's parameters was investigated. By doing so, the point where the median collapse intensity and dispersion begin to stabilise with respect to the chosen θ_{CDL} indicates a reasonable value for this drift limit to be implemented in future studies, as illustrated in Figure 1.

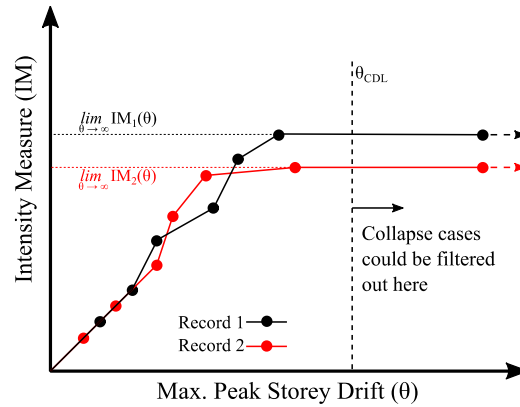


Figure 1. Illustration of two IDA traces, relating intensity measure with drift, $IM(\theta)$, to show that beyond a certain value of drift, the individual records all approach their limiting intensities and result in complete collapse.

Figure 2 plots the median collapse intensity and dispersion values for a number of case study typologies versus the chosen value of θ_{CDL} . Also plotted is the limiting value according to Rossetto and Elnashai (2003) beyond which complete collapse of infilled RC frames is to be expected. Figure 2 shows that both the median collapse intensity and dispersion of all typologies tend to stabilise beyond this proposed value of collapse by Rossetto and Elnashai (2003). As such, it can be stated that these values are indeed representative from both an experimental point of view due to the way in which they were derived, and also a numerical point of view in light of these results presented here. Therefore, it can be stated with a degree of confidence that, for infilled GLD RC frames typically found across southern Europe, $\theta_{CDL} > 5\%$ PSD represents a reasonable *a priori* value with which structural collapse can be identified from numerical models when developing collapse fragility functions. This applies to models that have sufficiently captured the individual members' strength and stiffness degradation, sufficiently modelled all possible failure modes in the building and account for P-Delta effects. This may be implemented using the aforementioned modelling approach by O'Reilly and Sullivan (2017a) and conducted in a relatively efficient manner using algorithms linked above.

2.2. Quantification of Uncertainty in Response

Further to quantifying the collapse performance of existing structures, the response prior to collapse is also of interest, as indicated in Equation 1. Response ordinates like PSD and peak floor acceleration (PFA) are often required when assembling a building performance model to estimate losses. This way, expected losses can be estimated by performing a number of random simulations to take inherent uncertainties into account, which can be divided into two sources: aleatory and epistemic. Epistemic uncertainty relates to the uncertainty in the methods of analysis employed, such as the fact that tools used in numerical analysis are not capable of predicting the exact structural response and have a degree of inherent uncertainty, for example. Aleatory uncertainty, on the other hand, relates to the natural randomness associated with the seismic assessment paradigm. The most well-known source of aleatory uncertainty is record-to-record variability. It is important that these uncertainties be reduced as much as possible so as to result in a more efficient prediction of the building response and in turn, performance. That as, the dispersion in a building response parameter such as PSD for a given intensity level should ideally be as low as possible, notwithstanding maintenance of an acceptable level of uncertainty elsewhere (e.g. in the chosen IM).

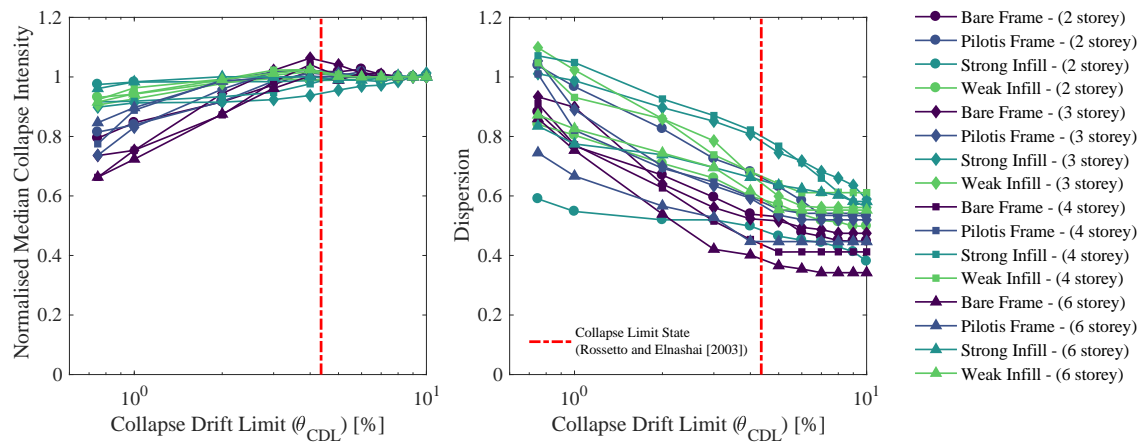


Figure 2. Median collapse intensity and dispersion values for the case study structures for different collapse drift limit values. The median collapse intensities have been normalised to the 10% collapse drift limit and the horizontal axis plotted in log-scale for better illustration.

The impact of these uncertainties in seismic assessment is rather evident. For example, Pinto and Franchin (2014) have shown how incorporating uncertainty can amplify the annual probability of exceeding a given limit state by around 2.25. Furthermore, O'Reilly (2016) demonstrated how neglecting uncertainty in loss estimation can result in expected annual losses being underestimated by as much as 26% for infilled GLD RC frames. These uncertainties can be taken into account in different ways in practice. For example, more simplified methods of assessment (Welch et al. 2014) determine the intensity level at which a certain performance limit state is exceeded and subsequently accounts for uncertainty in a simplified manner. Other more extensive methods of assessment quantify the building response for a given range of intensity levels and compute the building performance based on this estimated level of structural response. Regardless of the analysis method chosen, incorporating these uncertainties is paramount for a more accurate quantification of seismic performance.

When using tools like PACT, the structural response at a number of intensity levels is required as input. Whether simplified or more extensive numerical methods are used to determine the median structural response, the user is required to input an estimated value of additional dispersion due to modelling uncertainty. For infilled GLD RC frames, O'Reilly and Sullivan (2017d; 2017b) propose a set of values. If numerical analyses with ground motions are used, the record-to-record variability is inherently taken into account by using a suitable set of ground motion records. However, if more simplified methods of analysis were to be used to estimate the median response, an estimate of record-to-record variability would be required. For structural typologies typically found in the US, FEMA P58 provides default values for these. However, O'Reilly and Sullivan (2017d) recently showed that the structural behaviour of infilled RC frames tends to fundamentally differ from other structural typologies as it is characterised by a high initial strength and stiffness followed by a reduction in capacity due to infill failure, in addition to non-ductile failure modes such as a soft storey.

To investigate the record-to-record variability of infilled frames, the case study frames' IDA results were utilised to examine the impacts of including masonry infills. Figure 3 plots the observed PSD record-to-record variability with increasing intensity alongside the values provided by FEMA P58, where the variability tends to increase with intensity. In the case of bare and pilotis frames, these default dispersion values from FEMA P58 tended to conservatively envelope the observed dispersion. For the infilled frames, however, the general trend of the dispersion tended to increase to relatively large values of dispersion compared to the other typologies. For each structure, there tended to be a specific storey, or pair of storeys, that exhibited a pronounced dispersion with respect to the others. Comparing these locations with the results from pushover analysis outlined in O'Reilly and Sullivan (2017c), it was noted that in each case, the mechanism location in the structure corresponded with the location of highest dispersion shown in Figure 3. This is reasonable when considering that this is where most of the non-linear behaviour is focused. In this respect, O'Reilly and Sullivan (2017d) made a distinction between the proposed dispersion values due to modelling uncertainty for storeys where a mechanism forms, recognising this difference in dispersion at critical locations of the structure. This is contrary to more ductile frames that would have a more uniform distribution of damage throughout the height and consequently in dispersion as a result of forming a more stable beam-sway mechanism.

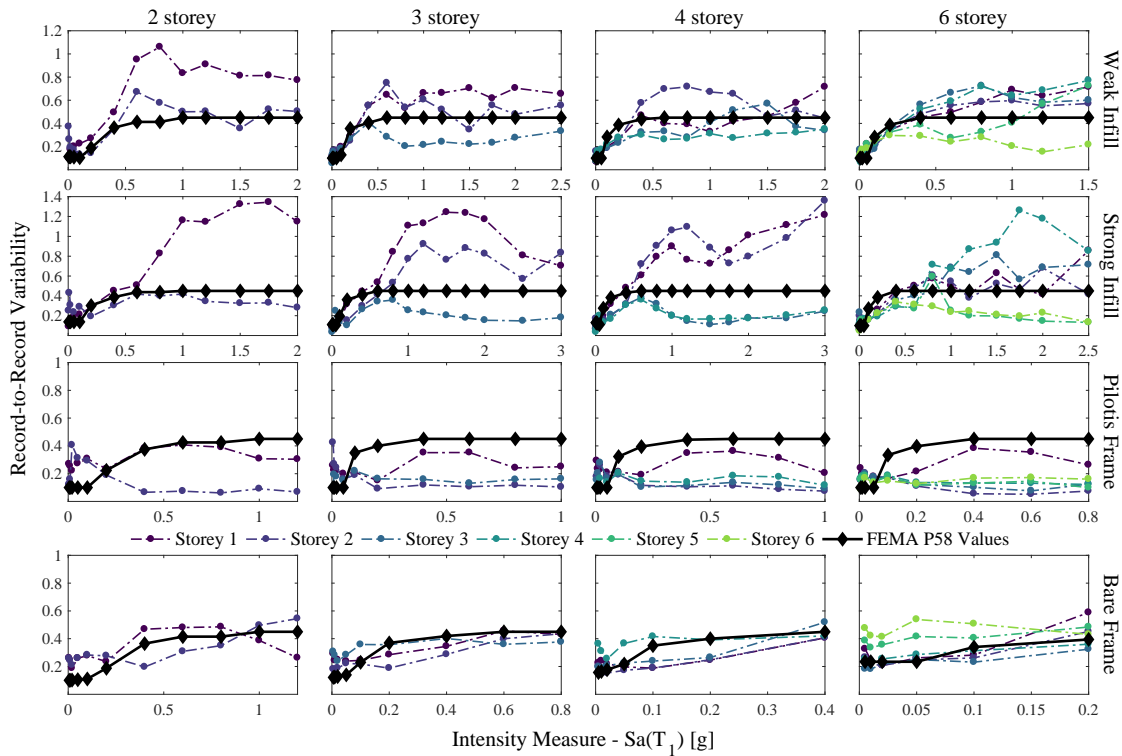


Figure 3. Record-to-record variability with respect to intensity for the case study structures.

Regarding the pronounced dispersion values for infilled RC frames, further investigation showed that this tended to arise when the large resistance of the masonry infill needed to be overcome to enter a relatively large range of drift, as illustrated in Figure 4. Upon reaching the shear capacity at a given storey, the infill strength, and subsequently the overall structural strength, rapidly degrade meaning that the drift demand tends to increase significantly for a given IM level. The period elongation of infilled frames due to the onset of infill damage also meant that the acceleration demand differed between ground motion records conditioned on T_1 , although aspects like frequency and duration characteristics are noted to also play a role. This leads to cases where the building period will elongate and shift into a period range with weaker demand than the mean of the ground motion set for some records (Figure 4(b)), whereas other records may have stronger demand (Figure 4(c)). The end result is that there may be instances in which the initial period elongation means sufficient acceleration demand is not inflicted on the building to push the building past its peak and remains in a tight pre-peak band of PSD (Figure 4(b)), whereas some other ground motions will and result in large values of PSD (Figure 4(c)). This results in a pronounced increase in dispersion and was termed the *cliff effect* by O'Reilly and Sullivan (2017d). It can be applied to any structural system with a similar hysteretic backbone behaviour whereby a significant loss of initial strength and stiffness capacity doesn't immediately result in global collapse and allows the structure to keep going past this reduction in strength capacity before eventually collapsing. Furthermore, it was noted that the lognormality of these demand distributions was maintained when performing a Lilliefors test at the 5% significance level. This suggests that the current approach of using $Sa(T_1)$ as IM for infilled RC frames is actually quite inefficient for demand-based assessment. This is since the IM is tied to T_1 that has little relevance on the actual structural characteristics once the masonry infill has begun to damage and the RC frame begins to yield. Approximate analyses were conducted by the authors and showed that T_1 tended to approximately double following the collapse of the infill. This significant increase in T_1 could be expected to have a significant impact on the efficiency of the adopted IM. Future research ought to investigate more efficient IMs that try to incorporate this feature of infilled RC frames in order to reduce the associated dispersion in demand parameters.

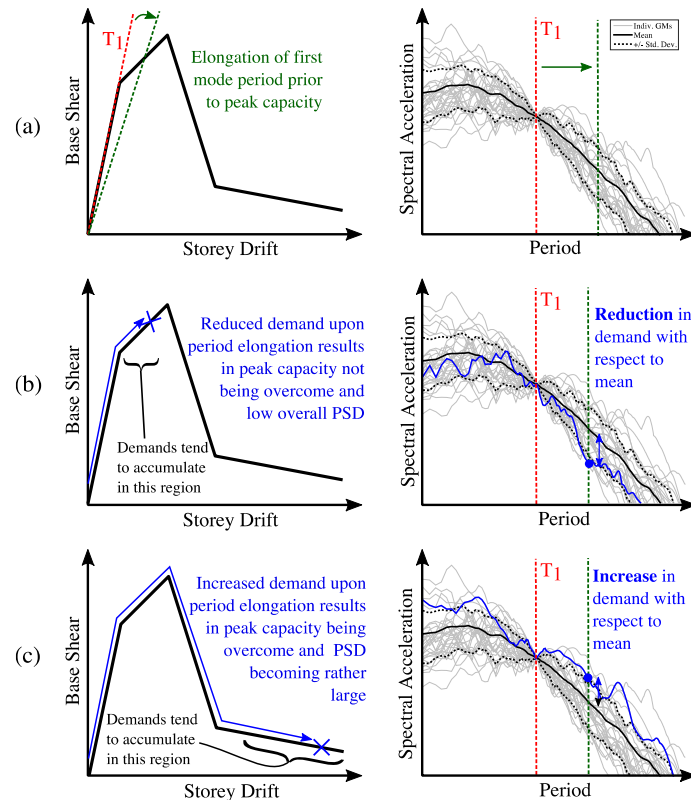


Figure 4. Illustration of the impact of the initial peak capacity of the infilled GLD RC frames on the record-to-record variability in the PSD demands, termed the *cliff effect*.

3. POTENTIAL IMPACTS OF LOSS ESTIMATION AND RETROFITTING

Section 2 discussed specific issues relating to the performance of GLD RC frames and the impact of certain aspects related to quantifying the structural response. These ranged from collapse performance and ways in which to determine this numerically to the expected levels of dispersion associated with the PSD. This section, on the other hand, discusses other aspects relating to the implications of some of the broader decisions to be made during loss estimation and retrofitting. Similar to Section 2, this section adopts some of the results presented in O'Reilly and Sullivan (2017c) to facilitate the discussion. Using the structures previously outlined, a complete loss estimation was carried out assuming the seismic hazard of L'Aquila, Italy and a representative building component inventory. From this, a number of retrofitting strategies were trialled to examine the impacts of both structural and non-structural retrofitting techniques on the seismic performance defined in terms of expected annual loss (EAL). The retrofitting solutions and corresponding labelling system adopted herein are listed here for clarity:

- Retrofit A – Improved detailing for both the drift and acceleration sensitive non-structural elements.
- Retrofit B – Improved detailing for the drift sensitive non-structural elements.
- Retrofit C – Improved detailing for the acceleration sensitive non-structural elements.
- Retrofit D – Structure retrofitted by RC jacketing the columns and beam-column joints and masonry infill isolated from surrounding frame.
- Retrofit E – Structure retrofitted by inserting an RC wall and masonry infill isolated from surrounding frame.

For each retrofit scheme, O'Reilly and Sullivan (2017c) repeated the loss estimation, compared the results to the existing structure and examined the relative performance. In a number of instances, the structural retrofitting of the buildings (Retrofit D and E) actually downgraded the building's performance, in terms of EAL, which can be seen in Figure 5, where a plot of the EAL before and after retrofitting using the different techniques is shown. This was rather unexpected at first, as one would expect the performance of the structure to be improved following structural intervention. As previously mentioned, the EAL is the integration of the vulnerability function with the hazard curve, meaning a reduction in vulnerability ought to correspond to a reduction in EAL. However, this assumes that the hazard curve has remained unchanged in addition to a

uniform improvement of the vulnerability curve at all intensities with respect to the original.

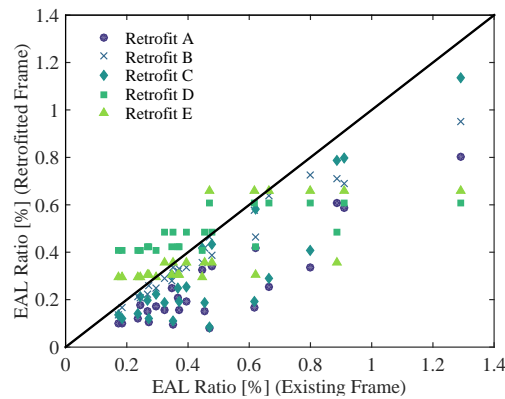


Figure 5. Illustration of the relative improvement in performance defined in terms of EAL with respect to the existing structure. Points above the 1 to 1 line correspond to cases where the EAL of the structure worsened following retrofitting.

Structural retrofitting is illustrated as a strengthening and stiffening of the structure in Figure 6(a), although other options such as supplemental damping could also be considered. With such retrofit solutions, there may be a trade-off, where the expected demand in one source decreases and possibly results in an increase in the other, as illustrated in Figure 6(b). This is also combined with the fact that the new prominent source of losses may actually be occurring at lower, more frequent intensities to further increase the weighting during hazard integration, shown in Figure 6(c) through the decrease in the intensity required to begin accumulating losses. This was observed for hospital buildings during the recent 2016 earthquake in Ecuador, where Morales et al. (2017) noted that following the 1998 earthquake in Ecuador, the Ministry of Public Health embarked on a programme to retrofit existing hospital structures via strengthening and stiffening to a level far beyond the building code requirements and achieve an almost elastic building response. Following the 2016 earthquake, post-earthquake reconnaissance of the hospital buildings by O'Connor and Morales (2016) reported that whilst none of the hospital facilities collapsed, 22 were left inoperative due to excessive non-structural damage to acceleration-sensitive elements. This resulted in medical aid being provided from temporary shelters, which resulted in overcrowding and unsanitary conditions. The observation of these counterproductive retrofitting measures carried out in Ecuador echo the above observations in Figure 5 regarding the potential worsening of performance due to excessive structural retrofitting when more efficient solutions may be available. The last aspect of the structural retrofitting highlighted here and illustrated in Figure 6(d) is the modification of T_1 as a result of stiffening. This change in T_1 which, when combined with the IM employed, $S_a(T_1)$, means that the mean hazard curve must also be updated to maintain consistency (Figure 6(e)). In cases like those illustrated in Figure 6(d), this may result in an increase in the mean annual frequency of exceedance (MAFE) for a given level of intensity, which when integrated with the modified vulnerability function (Figure 6(c)) may result in cases where the performance defined in terms of EAL actually worsens, as illustrated in Figure 6(f).

4. SEISMIC PERFORMANCE CLASSIFICATION FRAMEWORK

As previously mentioned, Section 2 discussed issues relating to response quantification of GLD RC frames, whereas Section 3 focused more on the conceptual impacts of different retrofitting schemes when aiming to reduce expected monetary losses. This section puts these two issues together and discusses them in the context of current European guidelines and requirements. To this end, a recent discussion by Pampanin (2017) is of particular relevance as it drew from lessons learned following the 2011-2012 New Zealand earthquakes and discussed ways in which the current approaches may be improved. Pampanin (2017) noted how, in many cases, the lack of prioritisation and simplified methodologies with which to make better risk-based decisions in assessment had often been the primary obstacle for implementation in practice. Pampanin (2017) highlighted the need to produce state-of-the-art, yet accessible, tools to tackle this issue. Many methods exist to perform a reasonably adequate seismic assessment, but often times lack sufficiently developed guidelines and supporting studies to provide many of the necessary inputs for all structural typologies. While perhaps not immediately apparent, the information presented on specific aspects relating to GLD RC frames in Sections 2 and 3 is

intended to bridge some of these gaps, even if discussion of the specific roles of such aspects within a complete assessment framework is beyond the scope of this paper.

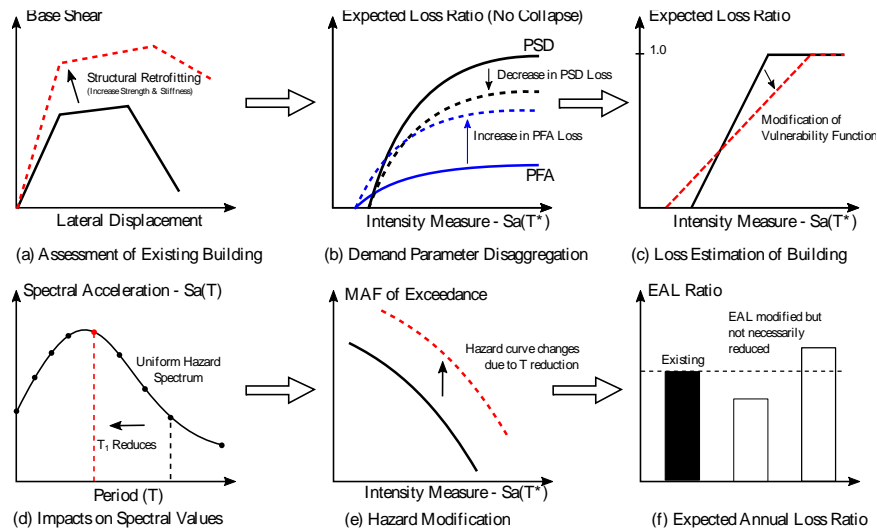


Figure 6. Illustration of the potential impact of structural retrofitting where strengthening and stiffening of the structure can actually worsen the EAL performance due to the combined effect of an increase in loss from higher PFA and a shift in the mean hazard curve due to period shortening.

Pampanin (2017) highlighted that a somewhat passive approach by decision makers was common in the past since detailed instruction was typically not available. In New Zealand, however, this changed in 2004 when a new building act defined a minimum level of seismic safety whose careful legal wording eventually resulted in the concept of *percentage of new building standard* (%NBS) that is commonly found in New Zealand-related literature. From this, it was understood that existing structures were to provide a minimum level of safety by possessing at least 33%NBS. Interestingly, the phrasing of %NBS became a proxy for collapse safety, which in a way it is, but the deterministic nature of computing %NBS raises issues when inferring a probabilistic meaning (see Vamvatsikos (2017)), in addition to Belliss et al. (2016) having shown it to not result in uniform risk solutions. Structures falling below the 33%NBS limit were to be deemed unsafe and require intervention at the owner's own expense, should structural alteration or change of building use be required. Moreover, buildings falling between 34-99%NBS (i.e. unsafe) were assigned a letter-based rating but did not legally require any intervention. This meant the requirements were rather circumventable as it resulted in a situation where a classification framework was indeed available to identify and improve existing buildings, but the financial burden and lack of incentive for owners tended to result in no action being taken. Pampanin (2017) went on to note how the findings of the Canterbury Earthquakes Royal Commission in 2012 have since resulted in a marked change in the attitude in the case of New Zealand, where a more active approach to improvement is being taken. Specific timelines within which the assessment of buildings must be carried out and subsequent decision be made are outlined, but it is still noted that no owner-specific motivations (e.g. financial subsidies) are envisaged to promote a more proactive approach aside from it being a legislative requirement.

From the above discussion, four separate issues relating to the case of New Zealand can be identified regarding the implementation of a seismic classification framework: (a) classification framework was originally in place but action not mandatory, although recent 2017 revision stipulates action; (b) lack of financial incentive on the owner's behalf; (c) primarily focused on life safety and economic losses typically not considered; and (d) quantities that were determined representative of seismic safety are not risk consistent. With these points in mind and turning the attention back to Europe, more specifically Italy, the Italian Ministry of Infrastructure and Public Works have recently introduced a set of seismic performance classification guidelines termed D.M. 58/2017 (Decreto Ministeriale 2017). Similar to the case of New Zealand noted in point (a) above, the guidelines were recently introduced but their implementation is not yet mandatory. They do, however, foresee the provision of tax benefits for owners opting to upgrade their structures within the classification framework provided, which addresses the point (b) outlined above. Regarding point (c), the recent guidelines in Italy do focus on life safety in much the same way as the current approach in New Zealand via a life safety index,

which is equivalent to the definition of %NBS. The peak ground acceleration (PGA) required to exceed the life safety limit state (SLV) is determined via non-linear static procedures (see Figure 7(a)-(d)) and is expressed as a ratio of the expected PGA of the limit state's design return period for new buildings. It follows that a ratio of 1.0 would correspond to a code complying structure, akin to 100%NBS. This analysis approach outlined in D.M. 58/2017 fits in very well with the existing approach prescribed by the Italian National Code (NTC 2008) meaning that it is very accessible to practitioners and provides an indication of a margin of safety with respect to current code requirements.

The D.M. 58/2017 takes the classification one step further (Figure 7(e)-(f)) by prescribing a relatively simple way in which the EAL can be computed. This process is illustrated in Figure 7 and represents a significant development in the classification of existing structures as it takes not only the life safety performance of the structures into account, but also the expected monetary losses. Similar to New Zealand, a letter-based scoring system is provided where the overall building score is the more critical of the life safety index and EAL score. As illustrated in Figure 7, a pushover analysis is performed and limit states identified. Using non-linear static analysis, the intensities required to exceed each limit state are identified and their corresponding MAFE determined. By defining fixed expected loss ratios for each limit state, the EAL is simply computed as the area beneath the curve in Figure 7(f). From the computed EAL, a score is assigned based on the prescribed ranges. Cosenza et al. (2017) discuss the process with which these different classification ranks were established, where the theoretical limits of 0 to 10% for the EAL of a building were established and the classification scheme defined as eight distinct ranges within these bounds. O'Reilly and Sullivan (2017c) recently discussed these range limits and suggested that the actual values of EAL computed following a more rigorous approach may be somewhat lower. Whilst the absolute values of EAL may be somewhat conservative, O'Reilly et al. (2018) have shown that the overall trend of identifying vulnerable buildings defined in terms of EAL using this simplified approach is still consistent with the more rigorous approach outlined in FEMA P58. Future revisions of the classification framework may look to incorporate extensive sensitivity analyses in addition to empirical data to refine the absolute values used to define these classification ranges. For example, the discussion regarding response uncertainty in infilled GLD RC frames discussed in Section 2.2 is a further aspect to be considered in this regard. Still, given the unequivocal nature of the current procedure and how it reduces the complexity required for the process, it represents a positive step in terms of providing practitioners with accessible tools to implement.

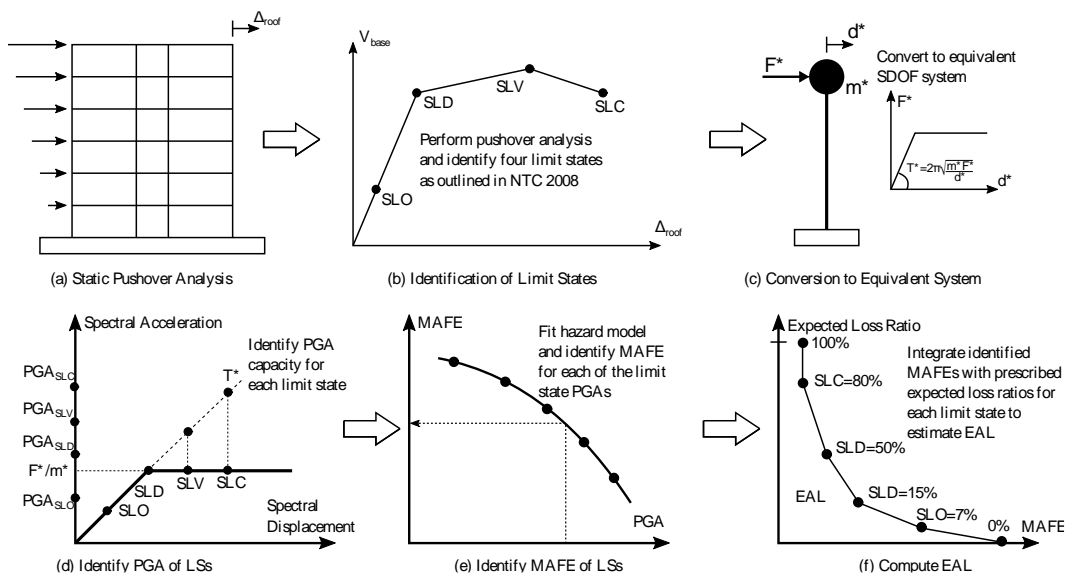


Figure 7. Fundamental steps involved in computing the EAL using the D.M. 58/2017 (Decreto Ministeriale 2017).

Lastly, regarding point (d) above, it has been previously stated that using %NBS as a proxy for collapse indicator is not risk consistent. Similarly, in Italy, the definition of a life safety index by comparing two intensities in a deterministic and non-probabilistic fashion should not be expected to provide risk consistent solutions. This is owed to the uncertainties entailed at the various levels of the computation. For example, the uncertainty in the numerical modelling is not considered when following the procedure outlined in Figure 7(a)-(d), nor is the record-to-record variability. The approach of providing a life safety index may be interpreted to

mean that a structure with a value of 1.0 possesses sufficient capacity to avoid collapse. However, as discussed in Section 2.1, this is not the case as record-to record variability alone means that for any given intensity, the collapse fragility function outlines a probability of collapse, and only through integration with the site hazard curve, as opposed to deterministically comparing demand and capacity at a single intensity, can these uncertainties be incorporated and the collapse safety be computed in a more risk consistent fashion. Similar to the argument put forward by Vamvatsikos (2017) regarding design methods, %NBS or life safety index in D.M. 58/2017 are not bad methods but are also not risk consistent, reason for which future refinements of performance guidelines should strive to incorporate this aspect.

5. CONCLUSIONS

This paper has discussed a number of issues relating to response quantification, assessment and retrofitting within the context of a seismic performance classification framework for infilled GLD RC frames. Regarding the response quantification, it was shown that by defining a sufficiently large collapse drift limit in numerical analyses, collapse fragility functions of the structural typologies can be developed in unambiguous manner and also remain consistent with existing damage scales developed from empirical and experimental data. The need to consider different sources of uncertainty in seismic assessment was also discussed. Some typical values for dispersion in infilled GLD RC frames were examined and compared with values provided by FEMA P58. Due to the large dispersion in specific storeys of infilled RC frames, it was seen how the values provided by FEMA P58 were a reasonable placeholder for bare and pilotis frames but not applicable to infilled frames. It was shown how dispersion arises due to the characteristic behaviour of infilled frames whereby a relatively high initial stiffness and strength are significantly reduced following the failure of masonry infills. The impact of the IM efficiency used in demand-based assessment, in addition to some ways in which it can be improved in future studies, was also discussed. Section 3 regarded aspects relating to the assessment and retrofitting of infilled GLD RC frames within the context of economic losses. Issues relating to impacts of different retrofitting decisions on the overall performance of structures were outlined and it was noted how, depending on the choices made during retrofitting, the overall performance defined in terms of EAL is not guaranteed to improve. It was shown that retrofitting solutions consisting of excessive strengthening and stiffening can be counterproductive and result in a worsening of the EAL. Finally, Section 4 discussed the implementation of a seismic performance classification framework. Some of the issues relating to the difficulty in implementing such a framework were outlined and it was noted how the recent guidelines introduced in Italy appear to have built on past lessons learned in New Zealand regarding aspects such as accessibility and motivation to upgrade on the owner's behalf. It was seen how the recent guidelines in Italy have moved to include not only the collapse safety of existing buildings but also their resilience to accumulating excessive monetary losses. These represent positive steps in the field as they aim to encompass more advanced measures of building performance, although it was noted that there are a number of possible areas for improvement to bring such guidelines to a more risk consistent level.

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