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IMPLICATIONS OF A MORE REFINED DAMAGE ESTIMATION APPROACH IN THE ASSESSMENT OF RC FRAMES

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Abstract

This paper examines the influence of different definitions of demand parameters for the seismic assessment of ductile RC frames. Fragility functions used in the performance assessment of structures typically relate damage states in the structural elements, such as beams and columns, to the drifts determined during the structural analysis phase of the assessment. However, the use of such global demand parameters implies that damage is being estimated based on global response at each floor and not the local demand on the various damage zones, despite the local demand on the different elements being typically available from structural analysis. In this paper, a number of structures are designed using direct displacement-based design and a seismic damage assessment is conducted using two approaches at various intensity levels. The first is to calculate the probability of damage in the structural elements based on global demand, such as drift at the centre of mass. The second is a more refined damage analysis approach, where demand is defined in terms of local demand parameters, such as the drift at the particular grid line of the frame. By comparing these two approaches, the impact of different procedures when defining structural demand on both the damage and loss incurred on the different elements is investigated. The results show that the use of a single demand parameter at the centre of mass for regular and symmetric structures is sufficient for estimating the damage to the frame members at any location. However, the use of drift at the centre of mass is no longer adequate in cases where the structure is irregular and possesses some torsional behaviour. It is shown that this not only underestimates the damage to some frame members but also provides unconservative estimates of direct loss in the range of 15-30% for the case study structures examined. This paper's findings, therefore, present the argument for more advanced tools in the assessment of RC frame buildings. In addition to the issue of torsional response, some further aspects relating to the assessment of RC frame buildings are discussed to illustrate other areas in which more sophisticated tools and rational approaches to defining damage may lead to better performance estimates.

Keywords: RC frames; torsion; irregular structures; loss assessment; damage assessment

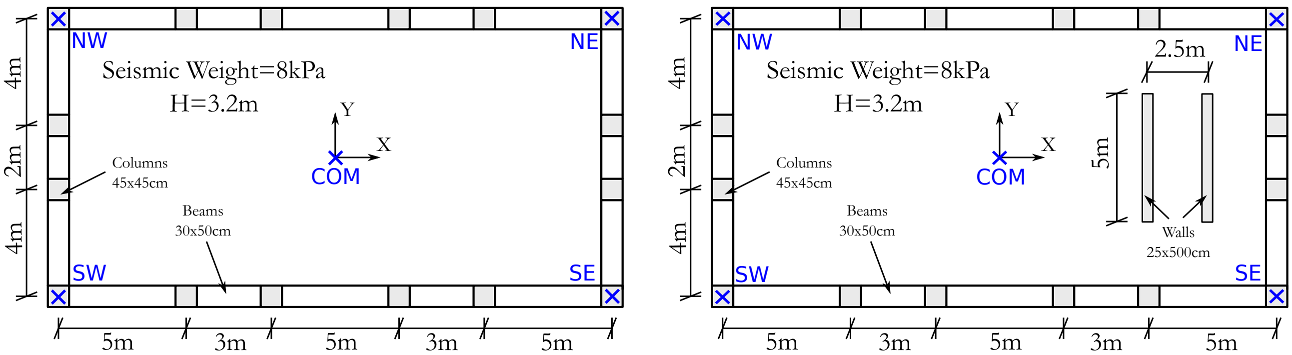
1 Introduction

Since the inception of what is now termed the PEER performance-based earthquake engineering (PBEE) methodology in the early 2000's [1], many advancements have been made in developing this framework for various types of structures. One of the main aims of such a methodology is to quantify the performance of buildings in a way that is more useful to stakeholders when making decisions regarding a new building's design or an existing building's retrofitting strategy. This framework has been extensively developed and as a result, FEMA has published the P-58 guidelines [2] to aid practitioners in implementing the method for everyday use. This document represents a major step forward in the assessment of structures as it provides a clear, yet relatively simple description on how to implement the procedure along with the Performance Assessment Calculation Tool (PACT) software [3]. One of the limitations of the procedure outlined in FEMA P-58 is that it has been primarily developed for simple, regular structures. That is, torsionally responding structures or structures with high irregularity in plan and elevation are not covered to any great extent in the guidelines. This represents a limitation to the current state-of-the-art, as many existing structures possess some degree of irregularity. As such, this paper investigates the impact of using more generalised methods, where the damage in deformation sensitive elements in a structure can be estimated using a single demand parameter at each level of the structure. This is compared with a more refined approach to estimate the damage where multiple definitions of deformation demand are defined at each level of the structure to better represent the actual demand the various damageable elements are experiencing at different parts of the building. By investigating the response of a case study RC frame building, both with and without torsional response, the impact of using a general approach to assessing damage at various intensity levels is compared to that of a more refined method. The effects of these two approaches are also quantified in terms of direct economic loss in order illustrate how such simplifications in the structural analysis and damage assessment of irregular structures can impact the salient parameters the PBEE framework aims to provide. Some additional comments on other pertaining aspects in the assessment of RC frame structures is also provided with the aim of highlighting additional areas in which a more rational approach to assessing structures may lead to improved estimates of performance.

2 Design and Numerical Modelling of Case-Study Buildings

2.1 Design of Case-Study Buildings

To illustrate the differences between assessment approaches, a series of case-study structures have been designed. These consist of a three and six storey regular RC frame with five unequal bays in the principal X direction and three unequal bays in the Y direction, as shown in Figure 1. Some of the typical details of the frame are outlined in Figure 1, where the total seismic weight of each floor is taken as 8kPa, the expected yield strength of the reinforcing steel bars is taken as 500MPa and an expected concrete compressive strength of 30MPa. In addition, Figure 1(b) shows the same structure but with two RC walls each with a nominal moment capacity of 12,000 kNm added to one end of the structure after the seismic design of the structure has been completed. This will induce a stiffness eccentricity in the structure and result in a torsional response, whose effects will be investigated and used as an example of how torsional response affects the estimation of damage in the various frame components. The dimensions of the columns were kept constant along the height of the building for simplicity during design, whereas in practice these would typically be expected to reduce with height, but it is expected that this assumption is not likely to impact the findings of this study. The structures with a plan layout shown in Figure 1(a) will be referred to herein as Frame A, while the corresponding ones shown in Figure 1(b) will be referred to as Frame B.



(a) Frame A – Symmetric in plan. (b) Frame B – Asymmetric in plan.

Figure 1: Layout of case-study RC frames.

For the purpose of assigning reasonable levels of strength and stiffness, the structures are designed using the direct displacement-based design (DDBD) method outlined in Priestley *et al.* [4], where the most recent developments described in the model design code (DBD12) [5] are adopted here. The buildings are presumed to be located at a site characterised by seismic hazard at the damage control limit state corresponding to Eurocode 8 [6] Type 1 spectrum with PGA = 0.4g, soil type C and T_D >4s. The target design drift θ_c is taken as 2.5%, while section sizes are as illustrated in Figure 1. For brevity, a summary of the salient design parameters for each frame in both X and Y directions are listed in Table 1 to arrive at the final design base shear which is then distributed along the height of the building to arrive at the final design solution. In order to ensure a strong column-weak beam strength hierarchy in the frame, the column design moments are amplified such that the strength ratio at each joint satisfies a strong column-weak beam ratio of 1.3. This capacity design requirement is also in line with the requirements of Eurocode 8 to ensure a ductile beam-sway mechanism in the frames. The presence of the RC walls in Frame B are not accounted for during the design process and are introduced in the numerical modelling to induce a torsional behaviour on Frame B, which will be investigated further in later sections. Hence, the two frames are designed with the same sections sizes and member capacities. The frames will be labelled using the system of number of storeys followed by design variation, such that the three storey building with no RC walls is referred to as 3A, for example.

Table 1: Summary of the salient DDBD design parameters.

		X direction		Y direction			
		3 Storey	6 Storey	3 Storey	6 Storey		Reference
Design Displacement	Δ_d	0.157	0.282	0.157	0.282	m	Ref [5] Eqn 5.2
Effective Height	H_e	7.3	13.5	7.3	13.5	m	Ref [5] Eqn 5.10
Effective Mass	m_e	461	872	461	872	T	Ref [5] Eqn 5.7
Yield Displacement	Δ_y	0.076	0.141	0.060	0.110	m	Ref [5] Eqn C7.1
Ductility	μ	2.1	2.0	2.6	2.6		Ref [5] Eqn 7.1
Equivalent Viscous Damping	ξ	14.3%	14.0%	16.1%	15.9%		Ref [5] Eqn 7.4
Spectral Reduction Factor	R_ξ	0.66	0.66	0.62	0.62		Ref [5] Eqn 1.2
Effective Period	T_e	1.40	2.49	1.47	2.64	s	Ref [5] Eqn 5.6
Effective Stiffness	K_e	9349	5535	8385	4941.3	kN/m	Ref [5] Eqn 5.4
Design Base Shear	$V_{b,total}$	1515	1654	1364	1486	kN	

2.2 Numerical Modelling of Case Study Buildings

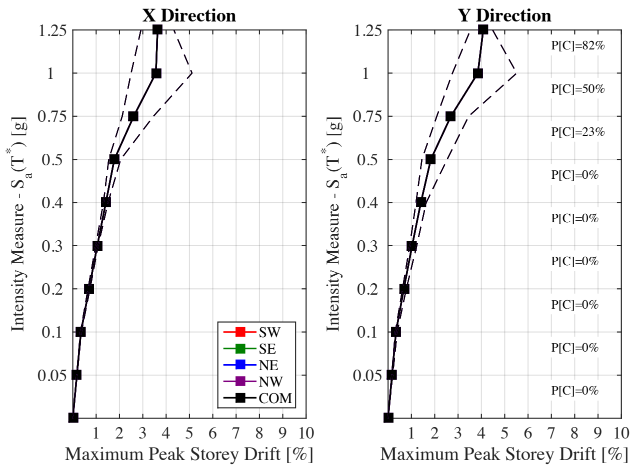
The numerical modelling is carried out using OpenSees [7], where lumped plasticity models are adopted for the frame members. The moment-curvature relation used to describe the plastic hinge behaviour is determined based on the provided member capacities determined from design and the assumed section size. The yield curvature is determined using the equations provided in Priestley *et al.* [4], which allows the computation of the cracked stiffness of the member, where the internal elastic portion of the element is also modelled using this cracked section stiffness. The post-yield hysteretic backbone parameters are determined as per the expressions given in Haselton *et al.* [8], which describe the behaviour of ductile RC frame members. The parameters are then implemented into the model using the modified Ibarra-Medina-Krawinkler [9] hysteretic model to adequately capture the post-yield behaviour of the RC members to account for concrete spalling, core crushing and rebar buckling through the strength and stiffness degradation of the members. The plastic hinge length is determined using the expression given in [10]. The floor diaphragms are assumed to be rigid in-plane and the gravity loads are modelled using a single P-Delta column placed at the centre of mass of the structure. The foundations of the structure are considered to be rigid. Table 2 shows the first mode periods of the frames in both directions, where the period of the frames are seen to be the same in the X direction for Frames A and B, but the presence of the RC walls is seen to reduce the initial period of the Frame B structures in the Y direction. For simplicity, no out-of-plane modelling of RC walls was considered. Had this been considered, one would see a slight shortening in the T_X values of 3B and 6B.

Table 2: First mode periods of vibrations in the two principal directions of the structures.

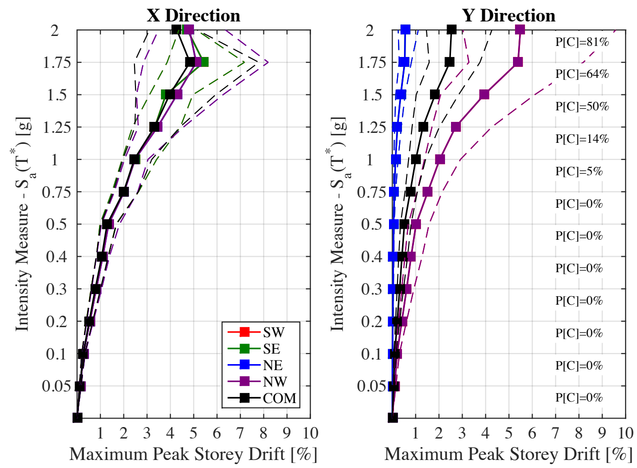
	3 Storey		6 Storey	
Layout	T_X	T_Y	T_X	T_Y
Frame A	0.94s	0.93s	1.64s	1.64s
Frame B	0.94s	0.49s	1.64s	1.08s

3 Multiple Stripe Analysis of the Case-Study Frames

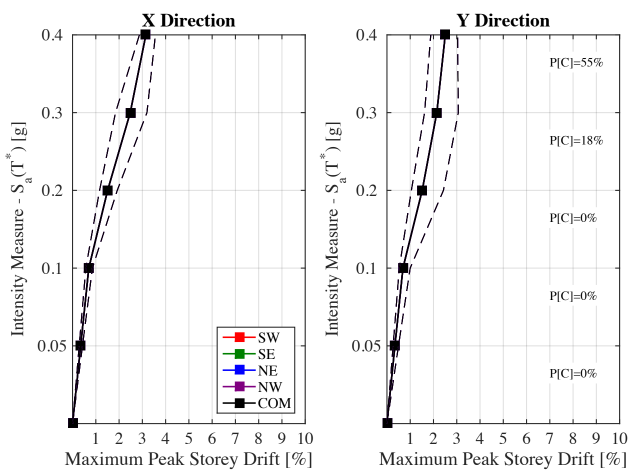
Numerical models of the frames described in the previous section have been subjected to a set of accelerograms at a number of intensities to investigate the progression of damage with increasing intensity. The set of 22 ground motions pairs available from FEMA P695 [11] are used and scaled to a number of intensities to perform what is commonly referred to as multiple stripe analysis (MSA), where the intensity measure is taken here as the spectral acceleration at conditioning period T*, where T* is defined as the average between the first periods of the structure in the X and Y direction, as suggested by FEMA P-58 [2]. Figure 2 shows some of the structural analysis results for the frames, where it is clear that for both 3A and 6A, the centre of mass (COM) response is the same as the response at different locations throughout the floor plan since the median and percentiles plot of the maximum of all the peak storey drifts throughout the height of the building at the different locations are the same, whereas the response of 3B is seen to be markedly different at different locations due to the influence of the torsional behaviour in the Y direction, which is seen through the increased demand on the west side of the building and a reduced demand on the east side, where the RC walls are located. This is, of course, an expected observation, but important to investigate further in terms of implications for damage assessment since if the COM storey drift is used to estimate drift at all locations in the floor plan, one would expect that each of the drift sensitive components are equally damaged, whereas in actual fact the components on the west side of the building floor plan are much more damaged than those on the east, as illustrated in Figure 2(b) and 2(d).



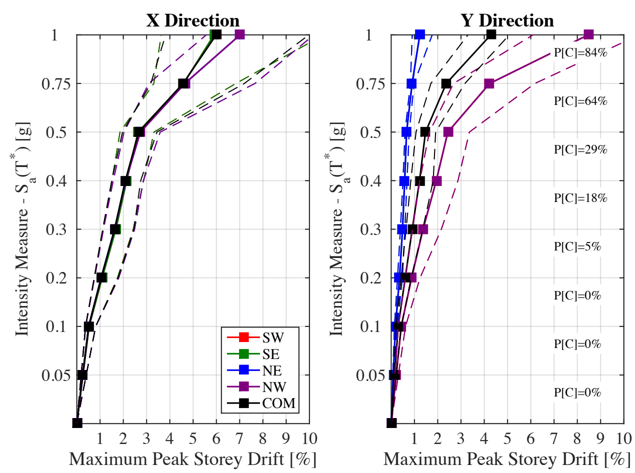
(a) 3 Storey Frame A - 3A



(b) 3 Storey Frame B - 3B



(c) 3 Storey Frame A - 6A



(d) 6 Storey Frame B - 6B

Figure 2: Median, 16th and 84th percentiles of the maximum peak storey drift of the case study frames.

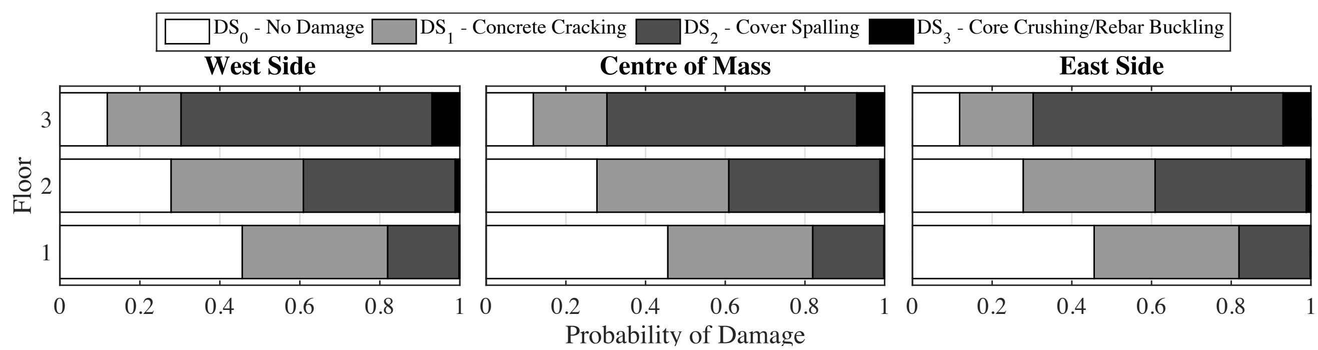
4 Damage Assessment of Case-Study Frames

Using the results of the analyses conducted in the previous section for each of the frames, a damage assessment of the structural members is conducted using two different methods:

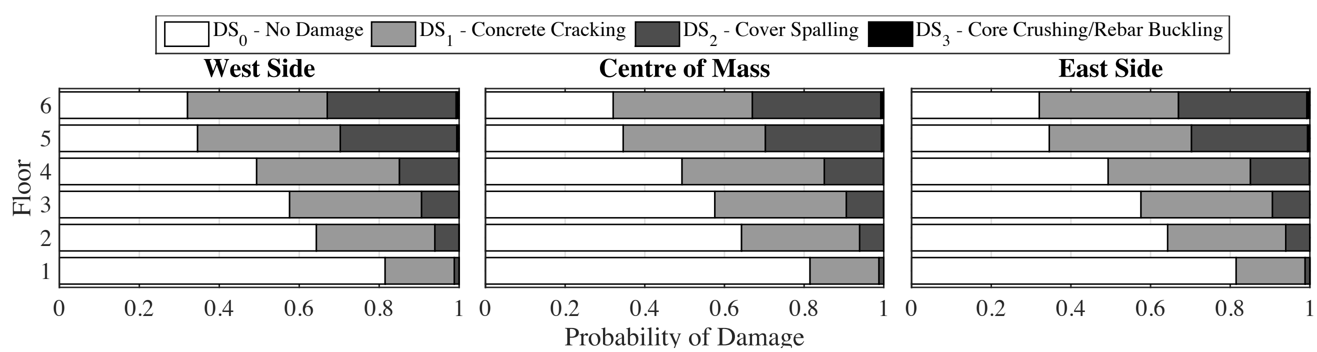
- Method 1: For each of the frames, the damage to RC frame members is computed by using the structural analysis results recorded at the centre of mass (COM) at each level of the structure. This means that despite the frame elements being located on the north/south and east/west faces of the structure, the drift at the COM is used as a representative demand parameter for the damage induced in the drift sensitive components at all locations in the floor plan. This is the default approach used in the software PACT [3] which requests a single drift in both principal directions of the structure, to which the various drift sensitive components are assigned. This is a common approach that works well, albeit with some other limitations discussed further in Section 6, in situations where the structure is regular in plan and elevation with a rigid diaphragm. However, this assumption doesn't hold well in situations where the structure is no longer regular in plan, as will be demonstrated later.

- Method 2: The local demand parameters associated with each of the various damageable components located throughout the structure are used. For example, the drift demand recorded on the west side of the structure during the numerical analysis is used to compute the damage in the corresponding elements. This is essentially using a better demand parameter to estimate actual damage since the drifts that that particular side of the building has experienced are being used to compute the damage in the elements. This implies that even in the case of and irregular plan building, a demand parameter that is representative of the damage that the frame is experiencing at that location on the floor plan is still being used.

The approach advocated by Method 2 may seem like a simple and logical process, but one must consider that current software tools to conduct such an assessment, whereby multiple definitions of demand parameters exist per floor of the structure, are not widely available and as such, more simple definitions of demand parameters are typically adopted to conduct damage assessments and subsequent economic loss assessments. This does not imply that current methods are wrong, but rather that they rely on certain simplifications to the idealised structure, which if the actual structure being assessed does not fit, leads to greater uncertainty being introduced to the performance results due to a lack of alternatives. This paper argues the case for such an alternative with the eventual provision of software tools that enable this more refined assessment. The fragility functions used to evaluate the probability of various damage states are those available within PACT for ductile RC beams with interior and exterior beam-column joints (ID's B1041.002a and B1041.002b in PACT). These consider three damage states corresponding to member cracking (DS_1), spalling (DS_2) and crushing of core concrete and/or rebar buckling (DS_3). The following sections describe the damage assessment of the various frames using the aforementioned fragility functions and two methods of assessment.



(a) Frame A – 3 storey (3A) at $S_a(T^*) = 1.0g$

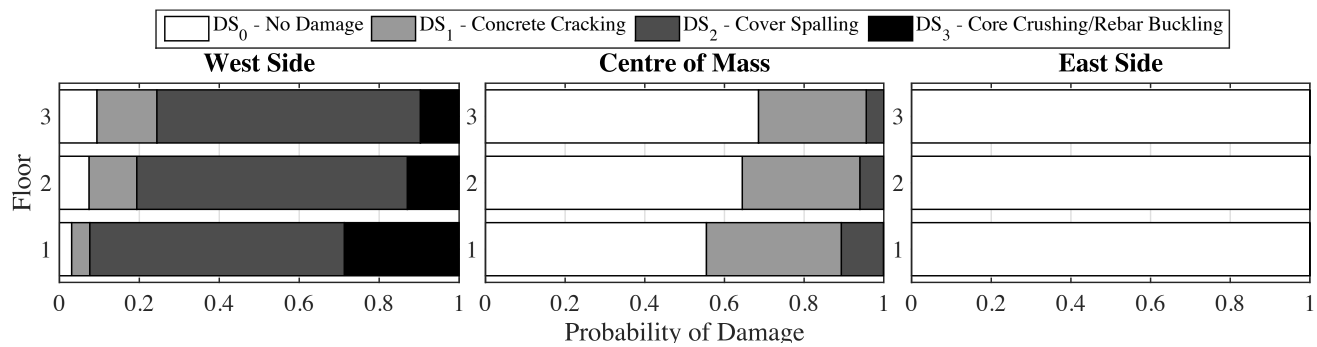


(b) Frame A – 6 storey (6A) at $S_a(T^*) = 0.4g$

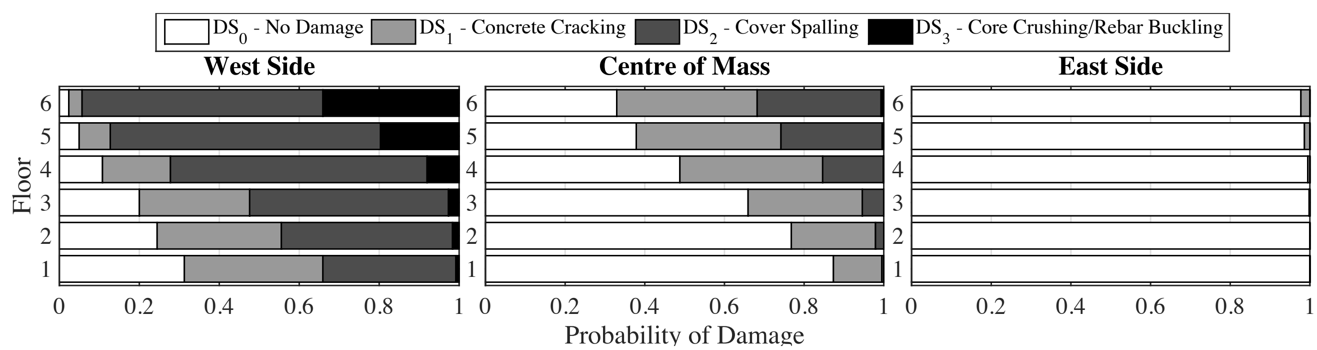
Figure 3: Damage probabilities in the Y direction at each floor of the Frame A type buildings where the median response was used to compute the probabilities of each damage state.

4.1 Frame A

For simplicity, the damage to the perimeter frames located on the east and west ends of the structure are presented here for Frame A, which is the regular plan frame shown in Figure 1(a). Figure 3 shows the probabilities of each of the damage states for the three and six storey frames using both damage assessment methods. The central subplot shows the probabilities of damage for all frame elements if just the drifts at the COM are used (Method 1), whereas the plots on either side show the probabilities of damage for the respective perimeter frame elements using the actual drift demands recorded at that end of the building (Method 2). It is clear from Figure 3 that the damage distribution at each floor for both frames is the same regardless of whether Method 1 or Method 2 is used. This is obviously because the frame in symmetric and a rigid floor slab has been assumed. As such, the use of Method 1 would suffice to estimate the damage to the frame members at all locations of the building. This is in line with current assessment approaches for structures as it fits the typical assumption of a regular floor plans that does not possess any torsional response.



(a) Frame B – 3 storey (3B) at $S_a(T^*) = 1.5g$



(b) Frame B – 6 storey (6B) at $S_a(T^*) = 0.75g$

Figure 4: Damage probabilities in the Y direction at each floor of the Frame B type buildings where the median response was used to compute the probabilities of each damage state.

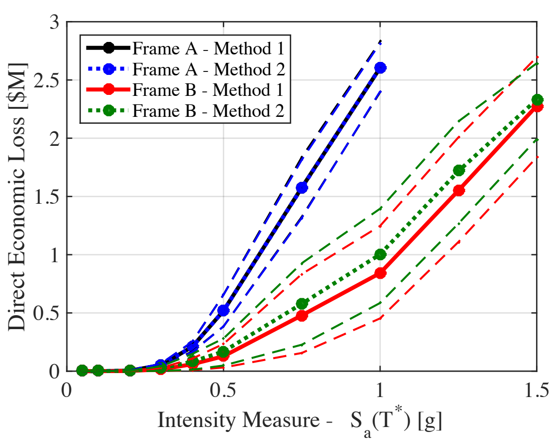
4.2 Frame B

Now let us consider the case of the Frame B type structures, which are essentially the Frame A type structures just with additional RC walls added to the east side in order to induce a torsional response of the structure. Again, only the response in the Y direction is discussed here and Figure 4 shows the estimation of damage using both Methods 1 and 2 as before. As can be seen from the central column of both figures, using the drift demand at the COM of the structure gives a moderate level of damage at each level for both frames, which by definition of Method 1 means that all of the RC frame members across the floor plan in the Y direction will possess such damage.

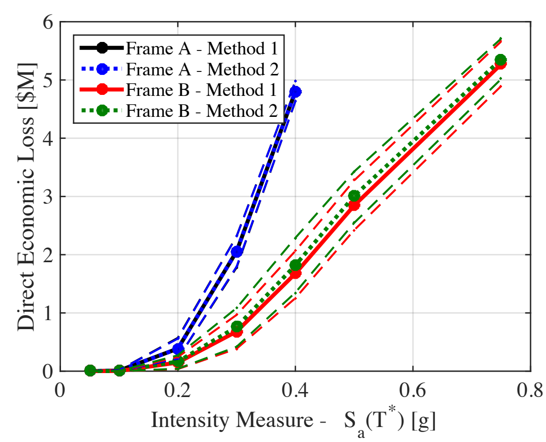
However, by examining the estimated damage in the outer columns that used the local drift demand at the respective ends of the frame (Method 2), it is clear that there is some degree of discrepancy. The torsional response of the frames due to the presence of the RC walls on the east side of the building means that the west end will move much more due to the east end being relatively stiffer. As a result, the west end will experience much more damage that the east end, which is clearly illustrated in Figure 4 as the east end shows essentially no damage, whilst the west end is experiencing heavy damage. While this is an expected outcome, it is important to note that if one had used the COM drifts to estimate damage, a much lower level of damage would have been estimated. This highlights the need for tools to conduct damage and loss assessment on irregular structures, such as Frame B, since the assumption of using a single demand parameter at each level leads to unconservative and erroneous estimates of damage across the building floor plan, which in turn will affect the final decision variables that performance-based assessment aims to provide.

5 Loss Assessment of Case-Study Frames

The previous sections have shown how the estimation of damage in RC frame members using two different demand parameter approaches leads to the same results in the case of Frame types A, which were regular and symmetric frames, and unconservative estimations of damage by using Method 1 for Frame types B, which were irregular due to the presence of additional RC walls on the east side of the structure. This section takes these damage estimates for both Method 1 and 2 to compute the resulting direct economic loss associated with repairing the RC frame elements by using what is now widely known as the PEER PBEE methodology [1,12]. Tools developed by the authors that apply this methodology are used, where additional options to enable to the application of a more refined assessment approach (Method 2) with relatively little extra effort are available. This will illustrate how such discrepancies in the damage assessment of the structure impact the direct economic losses at the various intensities investigated in the MSA of the buildings. The consequence functions used to estimate the repair costs for each damage state are taken from the same source as the fragility functions for ductile RC frames in PACT.



(a) 3 Storey



(b) 6 Storey

Figure 5: Mean and 16th/84th percentile of direct economic loss versus intensity for both Frame types A and B using both Methods 1 and 2 to estimate the damage to the RC frame members.

Figure 5 shows the trend of the mean direct economic losses in the buildings versus intensity measure, also commonly known as a vulnerability function, which consists of the direct losses associated with repairing the RC frame elements only. No other structural or non-structural elements are considered since the purpose of this figure is to illustrate the economic impact of the different damage assessment methods discussed in the previous section. The same concept is applicable to many other types of damageable components, both drift and acceleration sensitive, therefore the conclusions drawn here are not limited to RC frame elements. In the case of Frame type A, Figure 5 shows the two methods of damage assessment provide the same mean vulnerability function, as the two lines are exactly the same. This is to be expected when one considers that in Figure 3, the estimates of damage were the same regardless of which damage assessment method was adopted. As such, the direct losses associated with the repair of these elements should also return the same answer. However, in the case of Frame B, there is a clear discrepancy between the two approaches where Method 2 results in higher direct losses versus intensity compared to that of the Method 1 results. This discrepancy is in line with the observations of Figure 4, whereby using Method 1 to assess damage in the structure, a lower pattern of damage was observed in the overall structure. However, Method 2 showed that the west side of the building had experienced extensive damage and the east side relatively little. The overall discrepancy between the two methods in terms of direct losses is quantified as approximately 30% for the three storey frame, and about 15% for the six storey frame. The largest discrepancy can be seen in the intermediate intensities since at lower intensities, the torsional response has not become pronounced enough to result in significant difference between the two methods. In addition, as the intensity increases the difference also reduces the direct losses due to collapse of the building become more probable, whose economic consequence is the same for both building types. As such, the intermediate intensities are seen to exhibit the biggest differences, which is interesting to note since these would correspond to, depending on the site's mean hazard curve, the more frequent seismic events where most of the annualised losses are saturated, further highlighting the importance of properly establishing the losses at such intensities. The conclusions made here in terms of losses have been in reference to the repair costs associated with the RC frame elements of a bare frame, although the same conclusions may be drawn for an entire building with a full inventory of damageable structural and non-structural components. Such a study was not conducted here in order to clearly demonstrate the point being made regarding the differences in damage and loss for a single component.

6 Other Aspects to Consider

The previous sections have highlighted the influence of different approaches in defining demand parameters in the damage and loss assessment stages of PBEE assessment of structures. These sections highlighted this through the use of a relatively regular RC frame both with and without torsional irregularity. The results showed that by using simplified measures of structural demand, such as the drift at the structure's COM, unconservative estimates of damage and subsequently loss can be obtained for a torsionally responding building. This issue is not limited to torsional response of structures and this section aims to highlight other issues that have not been discussed in depth but are highlighted here in any case.

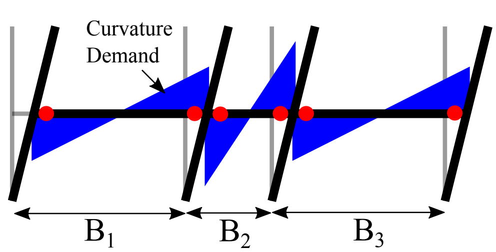


Figure 6: Beam curvature demands for frames with different bay lengths.

The first of these is the case of estimating damage in RC beams, such as the sub-assembly shown in Figure 7. The fragility functions adopted in previous sections to estimate the damage to the plastic hinges occurring in the beams was defined as a function of drift, whereby a certain level of drift at the storey would return various probabilities of each of the damage states associated with all of the plastic hinges located in the frame being examined. However, if one considers the case shown in Figure 6, where a frame of unequal bay lengths and constant beam depth is being assessed, some inconsistencies arise. The use of a fragility function defined in terms of drift implies equal damage at all plastic hinges, but from mechanics-based reasoning, it can be shown that the curvature demands on the plastic hinge regions of the interior bay of shorter length are much larger that those of the longer exterior bays. As such, the interior beam's plastic hinges will be damaged much earlier than those in the exterior bays. One can also see this when considering the expression of yield drift of an RC frame sub-assemblage, shown in Figure 7, given in Priestley *et al.* [4] as:

$$\theta_y = 0.5\left(\frac{\varepsilon_y L_b}{h_b}\right) \tag{1}$$

where ε_y is the yield strain of the rebar, h_b is the beam height and L_b is the bay length. This clearly shows how the drift at the yield limit state of the beams is clearly linked to the length of the bays. As such, the use of a single fragility function expressed in terms of drift without any consideration of bay lengths introduces additional error to the damage estimates. A solution to this inconsistency would be to define the damage in terms of the member's chord rotation since this has been widely used in scientific community to define the damage states of RC members during experimental testing and subsequent predictive equation development. This would require multiple definitions of a beam or column chord rotation demand parameters at each floor, which can, of course, become cumbersome to handle during analysis, in addition to new fragility functions required to be defined in terms of chord rotations. However, such experimental data already exists and has been used to calibrate the expressions provided by Haselton *et al.* [8], for example.

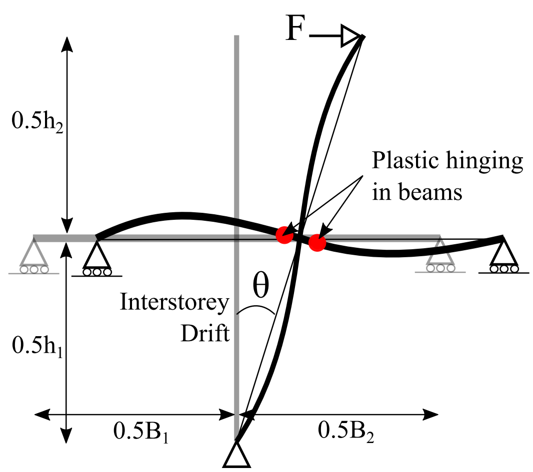


Figure 7: Definition of drift typically used in RC beam-column sub-assemblages, referred to here as interstorey drift, which is taken as the relevant displacement between column contraflexure points (illustrated here to be at the mid-height of the storey) divided by the distance between them.

Another issue with currently fragility function definition highlighted here is the definition of drift in the actual building itself. If one were to examine a series of experimental test reports and papers examining the behaviour of RC frame sub-assemblages, a typical response plot would report the actuator force versus the interstorey drift, described in Figure 7. Such a test assembly is quite common and a large amount of existing fragility function sets for RC frames have been based on such a definition of interstorey drift, which has been taken as the relative displacement between the contraflexure points divided by the distance between them. However, when conducting numerical analysis of a case study structure, such as that shown in Figure 8, the drift is often defined as the relative displacement between two slabs over the storey height, whose more appropriate term would be storey drift, since it refers to the drift of an actual storey and not between the storeys. This introduces another inconsistency in the definition of currently available fragility functions as the definition of drift from which fragility functions were developed (interstorey drift) and the definition with which they are being applied (storey drift) are not the same. They are strongly correlated, but not equal. Furthermore, one may speculate how appropriate it is to use the storey drift from the lower storey to predict damage in the beams located between two storeys. The use of interstorey drift is more appropriate but then a difficulty arises in actually obtaining this since the relative distance between contraflexure points must be recorded during analysis, which can often only be done by introducing more intermediate nodes to the member, which results in an increased model size that may run much more slowly. The same points also hold when evaluating the performance of beam-column joints, whose damage is often defined in terms of interstorey drift, but whose numerical modelling is defined in terms of joint rotation [13]. On the contrary, the use of storey drift is an excellent demand parameter when considering the damage to non-structural elements such as partitions, since the relative displacement of the floor slabs is what is damaging them and not a combined drift of two adjacent floors. As a result, it appears that two separate definitions of drift are required in assessment, but rarely is such a distinction actually made.

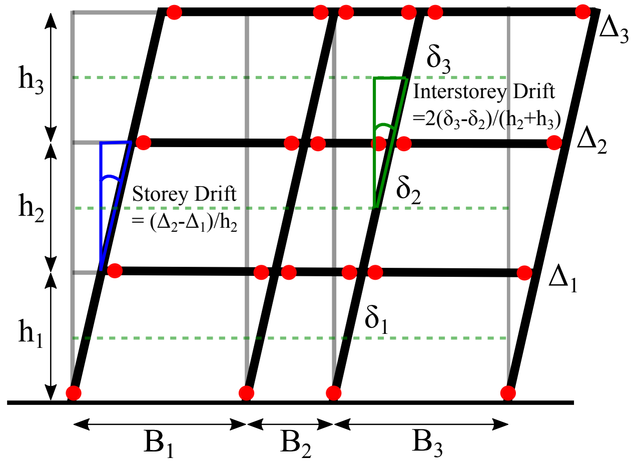


Figure 8: Typical definition of drift used in structural analysis, referred to here as storey drift, which is taken as the relavant displacement between floor slabs divided by the storey height.

7 Summary and Conclusions

This paper examined the various approaches to estimating damage and subsequent economic losses of structures within the PBEE framework. A series of case study structures were designed and assessed by using different approaches to demand parameter definition. The results of multiple stripe analysis show that for regular and symmetric structures, the use of a single drift at the centre of mass suffices in estimating the damage to the frame members at any location across the structure's floor plan. However, in cases where the structure is no longer regular and possesses some torsional irregularity, the use of drift at the centre of mass is no longer accurate and not only underestimates the damage to the frames members, but also provides unconservative estimates of direct loss in the range of 15-30% for the case study structures examined. These finding illustrates the need for more advanced tools for seismic assessment that incorporate many definitions of demand per floor to provide a more representative estimate of damage and losses for structures where the simplifying assumptions required for current methods no longer hold true. In addition to the problem of torsional response in damage and assessment, other issues relating to unequal bay lengths and inconsistent definitions of drift demand parameters were highlighted to further make the case for the development of such tools that allow for more complex and irregular structures to be assessed with more confidence.

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