Towards a Direct Displacement-Based Loss Assessment Methodology for RC Frame Buildings

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SUMMARY:
Various loss assessment methodologies have been proposed and developed over the past few decades to provide risk assessment on a regional scale. There is an increasing need, however, to provide engineers with practical tools for building-specific loss assessment. Recently, progress has been made towards probabilistic building-specific loss models such as the PEER PBEE framework. However, as some consider that comprehensive probabilistic methodologies could be too complex for most practicing engineers, this paper presents a simplified probabilistic loss assessment methodology that builds on a direct displacement-based assessment framework. The simplified methodology attempts to incorporate critical aspects of comprehensive loss assessment models while maintaining a computational onus suitable for the design office. The trial methodology is tested via examination of a 4-storey RC frame building and encouragingly, the direct losses estimated by the new approach are found to be similar to those estimated by the PEER methodology.

Keywords: Displacement-Based Assessment, Simplified Direct Loss Estimation, RC Frames

1. INTRODUCTION TO DIRECT DISPLACEMENT-BASED ASSESSMENT

The Direct Displacement-Based Assessment (DBA) procedure incorporates the latest advances in Direct Displacement-Based Design (DBD) principles developed over the past few decades (Priestley 1997, Priestley *et al.* 2007, Sullivan *et al.* 2012). The key concept that distinguishes the Direct DBA approach from previous force-based assessment methods is that displacement performance is the primary metric rather than a strength-based ratio. The consideration of displacement performance is quantified by the use of an equivalent linear single degree of freedom (SDOF) structure following the substitute structure concept (Gulkan and Sozen 1974, Shibata and Sozen 1976). Depending on the expected inelastic mechanism, the appropriate displaced shape and corresponding shear demands can be calculated for the particular limit state of interest. The following discussion will develop the standard procedure for the DBA assessment of frame buildings where more information on the assessment of different structural systems is provided within the work of Priestley *et al.* (2007).

Given that the expected inelastic mechanism, displaced shape, and shear distribution can be estimated (refer section 3), a substitute structure transformation is carried out through the calculation of equivalent SDOF parameters (refer Figure 1.1a). Using the distribution of storey displacements, Δ_i (refer Figure 1.1a), and the expected base shear, V_b, at the desired limit state (see F_m in Figure 1b) the calculation of the effective period, T_{eff}, is calculated following the sequence of relations shown in Eqn. 1.1. The SDOF displacement capacity, Δ_{cap}, and effective mass, m_e, are found using the Δ_i distribution and the distribution of storey masses, m_i. Dividing V_b by Δ_{cap} gives the effective stiffness, taken as the secant to the limit state displacement (refer Figure 1.1b). Finally, the effective period can be calculated using the standard equation for a SDOF oscillator.

$$\Delta_{cap} = \frac{\sum m_i \Delta_i^2}{\sum m_i \Delta_i} \rightarrow m_e = \frac{\left(\sum m_i \Delta_i\right)^2}{\sum m_i \Delta_i^2} \rightarrow K_e = \frac{V_b}{\Delta_{cap}} \rightarrow T_e = 2\pi \sqrt{\frac{m_e}{K_e}} \tag{1.1}$$

The remaining steps in the Direct DBA are necessary in order to determine the appropriate spectral reduction factor, η, to assess the seismic intensity at which a limit state will be exceeded (refer Figure 1.1c). The effects of inelastic response and energy dissipation are accounted for through an equivalent viscous damping (EVD) term, ξ_{eq}, which is calculated based on the expected ductility demand in the structure. The EVD at each storey *i* is calculated using the first term in Eqn. 1.2 where μ_i is the storey ductility and *C* is an empirical coefficient that accounts for the hysteretic behaviour of the structural system, typically ranging from 0.4 to 0.6 (refer Priestley *et al.* 2007). Storey EVD values are combined for the equivalent SDOF system in terms of storey shear and inter-storey drift profiles; represented by V_i and θ_i respectively in the second relation shown in Eqn. 1.2.

$$\xi_{eq,i} = 0.05 + C\left(\frac{\mu_i - 1}{\mu_i \pi}\right) \rightarrow \xi_{eq} = \frac{\sum V_i \theta_i \xi_{eq,i}}{\sum V_i \theta_i} \rightarrow \eta = \left(\frac{0.07}{0.05 + \xi_{eq}}\right)^{0.5} \rightarrow S_{d,el} = \frac{\Delta_{cap}}{\eta} \tag{1.2}$$

The EVD value for the equivalent SDOF system, ξ_{eq}, is used in the calculation of the spectral reduction factor, η, which assumes a relation recommended by Priestley *et al.* (2007) shown in Eqn. 1.2. Now the elastic spectral displacement demand, $S_{d,el}$, can be calculated using the displacement capacity (or any other limit state response). The final step in the assessment is the definition of the seismic intensity that will cause the development of the limit state of interest (refer Figure 1.1c).

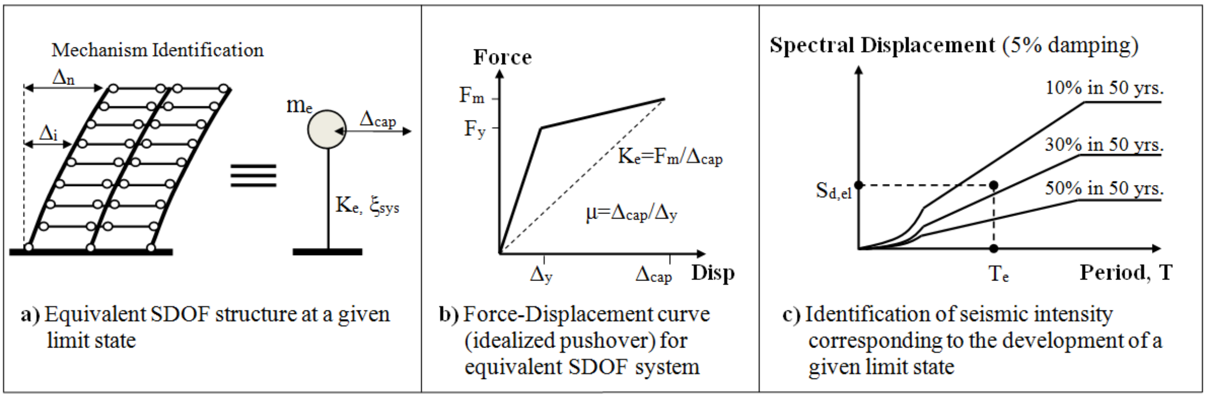
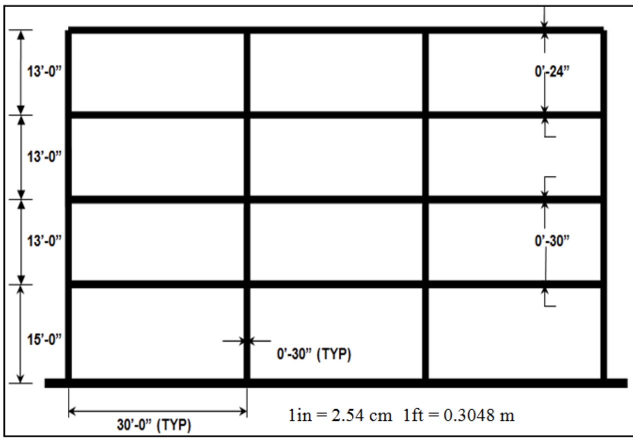


Figure 1.1. Illustration of key concepts behind the Direct DBA approach a) Illustration of substitute SDOF system b) idealized force-displacement response c) Determining the limit state intensity level

2. CASE STUDY BUILDING

In order to investigate the ability of the Direct DBA approach, a 4-storey case study building shown in Figure 2.1 is examined. The building is regular in plan with dimensions of 6 bays of 30 ft (9.14 m) in the East-West direction and 4 bays of 30 ft (9.14 m) in the North-South direction giving a total plan area of 180 ft by 120 ft (54.9 m by 36.6 m). Typical office occupancy is assumed in terms of building importance and internal component inventory. The structure is realized with a RC space frame with moment resisting frames (MRFs) on each column line. The reinforcement scheduling was taken from a 2003 IBC (ICC 2003) design completed by Hasleton and Deierlein (2007). Member dimensioning and detailing was controlled by joint shear requirements. Notably, an average strong-column weak-beam (SCWB) ratio of 1.3 was maintained in all stories but the roof level which had a SCWB of less than unity. The four storey building is represented (and analyzed) as a three-bay two-dimensional frame and beam-column elements were designed for uniaxial bending to accommodate this modelling assumption. An elevation drawing of the four storey frame is shown in Figure 2.1 along with typical reinforcement details reported by Haselton and Deierlein (2007). The building site assumed for the case study represents a site located in the Los Angeles Basin (33.996° Lat, -118.162° Long). The site is referred to as the PEER benchmark site as it has been adopted in numerous studies that implement the probabilistic PEER methodology on RC MRF buildings (Mitrani-Reiser 2007, Ramirez and Miranda 2009). The hazard information for the site adopted the results of probabilistic seismic hazard analysis (PSHA) from the study conducted by Goulet *et al.* (2007). The mean uniform hazard spectra for seven intensity levels are illustrated in Figure 2.2. The intensity levels within Figure 2.2 are defined in terms of their probability of exceedence (PE) in a given time interval (e.g. 50 years) and assumes the common Poisson recurrence law.



Member	Columns	$Beams_{1,2}$	$Beams_{3,4}$
h, b in. (mm)	30, 30 (760)	30, 30 (760)	24 (610), 30 (760)
ρ	0.011-0.016	0.005-0.007	0.005-0.007
ρ'	-	0.011-0.013	0.011-0.013
ρ_{sh}	0.0065	0.0025-0.0023	0.0034-0.0032
s, in. o.c. (mm)	4.0 (100)	5.0 (127), 5.5 (140)	5.5 (140)

ρ – total reinforcement ratio (columns), positive reinforcement ratio (beams)
ρ' – negative reinforcement ratio (beams)
ρ_{sh} – shear reinforcement ratio
s – center to center stirrup spacing within plastic hinge region

Figure 2.1. Elevation view of three bay representation of the four-storey space frame building with typical design details reported from Haselton and Deierlein (2007)

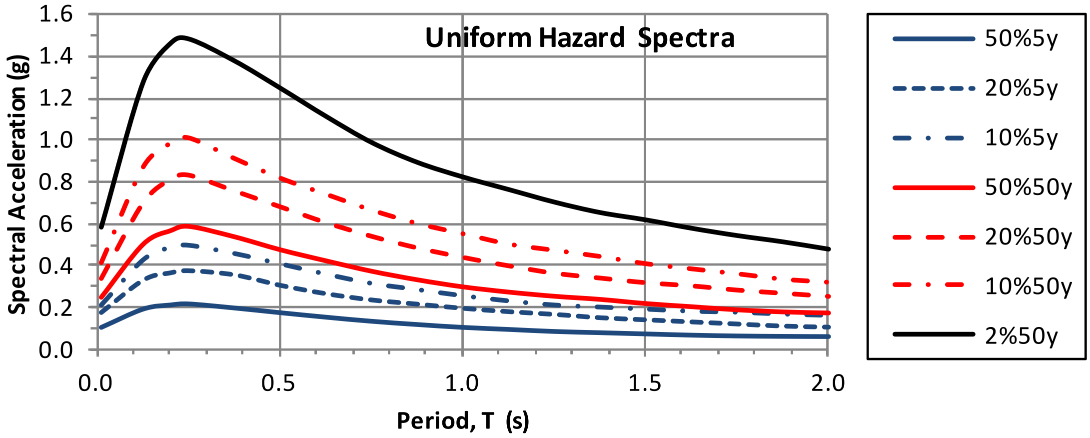


Figure 2.2. Uniform hazard spectra for PEER benchmark site (after Goulet et al. 2007)

3. DIRECT DBA OF THE FOUR-STOREY CASE STUDY BUILDING

A sample application of the Direct DBA procedure for the 4-storey case study building is developed in this section and is compared with non-linear pushover analyses in order to illustrate the existing procedure before extending the method to include direct loss estimation in section 4.

3.1 Inelastic mechanism, displaced shape, and shear profile

As mentioned in section 1, the initial step of the approach involves the identification of the likely inelastic mechanism that is likely to develop. Priestley *et al.* (2007) recommend the use of a sway potential index, S_i, that relates the relative moment capacities of beams and columns at each joint *j* across storey *i* over the height of the building. The relation for calculating S_i is shown in Eqn. 3.1:

$$S_i = \frac{\sum_j \left(M_{bl} + M_{br}\right)}{\sum_j \left(M_{ca} + M_{cb}\right)} \tag{3.1}$$

where the terms M_{bl} and M_{br} represent the expected beam moment strengths to the left and right of joint *j* respectively while M_{ca} and M_{cb} are the expected column moment strengths above and below joint *j* respectively. Moment capacities at the face of the joint are then extrapolated to the joint centroid. For a given storey level *i*, values of S_i greater than 1.0 signify that a column-sway mechanism is likely to develop while S_i less than 1.0 suggests that a beam-sway mechanism is likely. However, Priestley *et al.* (2007) suggest that the threshold limit may be conservatively reduced to 0.85 due to the implications of improperly identifying the expected inelastic mechanism. The resulting sway potential indices for the case study building suggest that a beam-sway mechanism is likely to develop and the lack of SCWB provisions at the roof level produced a S_i value close to unity as shown in Figure 3.1a. As the case study building is a 2003 IBC conforming design it is expected that a beam-sway mechanism is likely to develop and the remaining discussion will address this type of expected behaviour. However, a slightly different approach is taken when a column-sway mechanism is predicted and further information on this topic can be found within Sullivan and Calvi (2011). The displaced shape is estimated by using a first mode shape approximation. For an expected beam-sway mechanism, the displaced shape is given by Eqn. 3.2 as per recommendations in Sullivan *et al.* (2012).

$$\delta_i = \frac{4}{3}\left(\frac{H_i}{H_n}\right)\left(1 - \frac{H_i}{4H_n}\right) \rightarrow \Delta_i = \theta_c h_c \left(\frac{4H_n - H_i}{4H_n - H_c}\right) \tag{3.2}$$

Where δ_i is the first mode shape at storey *i* (out of *n* stories) normalized by the total roof height, H_n. The terms θ_c and h_c represent the assumed inter-storey drift and storey height of the critical storey (typically assumed as the first storey for beam-sway). The terms H_i and H_c are the heights above the base for storey *i* and the critical storey respectively. The recommendations of Priestley et al. (2007) suggest that a linear displaced shape is appropriate for frames with 4 stories or less while a more non-linear shape is attributed to frames with more than 4 stories. However, as the case study frame shows a lack of SCWB provision at roof level and has constant column cross sections along the height, the non-linear displaced shape (Eqn. 3.2) is assumed to be a better approximation for any appreciable displacement demand. Further research efforts could undertake parametric studies of how the displaced shape is affected by strength distribution and intensity. Using the displaced shape and member moment capacities, the expected base shear demand can be calculated from Eqn. 3.3:

$$OTM = \sum_{j=1}^{m} M_{c1,j} + \sum_{i=1}^{n} V_{b,i} \cdot L \rightarrow H_e = \frac{\sum_{i=1}^{n}\left(m_i \Delta_i H_i\right)}{\sum_{i=1}^{n}\left(m_i \Delta_i\right)} \rightarrow V_{base} = \frac{OTM}{H_e} \tag{3.3}$$

where the base shear, V_{base}, is taken as the total overturning moment, *OTM*, divided by the effective height, H_e, which is a function of storey mass, m_i, taken as 1.34 kip-s^2/in (24 tonne) per floor. The *OTM* is determined by the summing the base column moment capacities and the contribution of beam shears corresponding to nominal beam flexural capacity extrapolated to the joint centroids then multiplied by the entire length, *L*, of the frame (refer to Figure 3.1b). The equivalent storey forces, F_i, to calculate the storey shear profile is found using Eqn. 3.4.

$$F_i = \left(\delta_i m_i / \sum_{i=1}^{n} \delta_i m_i \right) \cdot V_{base} \tag{3.4}$$

The expected ductility demand at each storey is found based on the assumed displacement at the critical storey (θ_c or Δ_c) and an approximated yield drift. For a RC frame with an expected beam-sway mechanism, Priestley *et al.* (2007) recommend the yield drift be approximated using Eqn. 3.5:

$$\theta_y = 0.5 \frac{\varepsilon_y L_b}{h_b} \tag{3.5}$$

where θ_y, ε_y, L_b, and h_b are the storey yield drift, yield strain of longitudinal reinforcement, beam length and beam depth respectively.

A sample presentation of the Direct DBA approach is presented for the case study building assuming a peak inter-storey drift ratio of 1.4% at the bottom storey. This value corresponds to the yield drift using Eqn. (3.5) as well as the bottom storey design drift reported for the design from Haselton and Deierlein (2007). The SDOF parameters are displayed in Table 3.1 while the displaced shape (using Eqn. 3.2) and shear profiles are shown in Figures 3.1c and 3.1d respectively. Based on site hazard information, this limit state is expected to develop at the 20% in 50 year intensity. For comparison, the response profiles are compared to a first mode non-linear pushover analysis conducted by Welch (2012) using the structural analysis program Ruaumoko (Carr 2007). The figures show that the Direct DBA response prediction is in good agreement with non-linear pushover analysis.

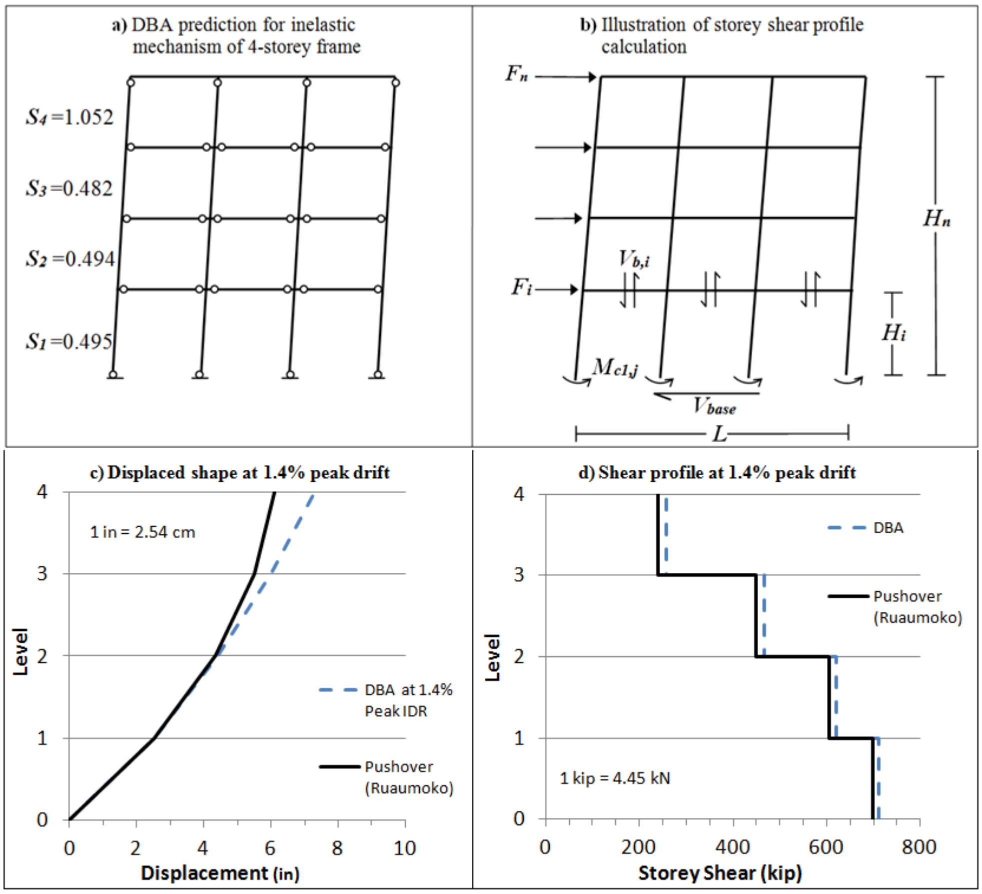


Figure 3.1. a) Sway potential indices calculated for the case study building b) Illustration of parameters used in shear profile calculation c) Displaced shape at development of 1.4% drift compared with non-linear pushover analysis d) Shear profile at 1.4% drift compared with non-linear pushover analysis

Table 3.1. SDOF parameters calculated at the development of 1.4% drift

Δ_{cap} (in)	m_e (kip-s^2/in)	V_{base} (kip)	K_e (kip/in)	T_e (s)	ξ_{eq}	η	$S_{d,el}$ (in)
5.69	4.76	709	125	1.22	5.07%	0.995	5.72

4. EXTENSION OF DIRECT DBA TO ESTIMATE DIRECT LOSSES

The estimation of building-specific losses, more specifically direct losses considering only repair and replacement costs, has been given much attention in recent research efforts, most notably through the implementation of the PEER Performance-Based Earthquake Engineering (PBEE) methodology as described in Mitrani-Reiser (2007). The methodology features four main stages once the structural type and site location are defined: hazard analysis (e.g. PSHA, record selection), structural response, damage calculation, and finally performance evaluation (e.g. monetary repair costs). The methodology can be termed as probabilistic as each successive stage relies on the probabilistic distribution of parameters from the previous in a triple integral format. The preliminary investigation into the extension of the Direct DBA procedure to incorporate direct loss estimation builds on the proposals of Sullivan and Calvi (2011), to provide simplified probabilistic loss estimation that is broadly aligned with the PEER PBEE approach.

4.1 Definition of key limit states

Performance evaluation in terms of expected annual loss (EAL) is a powerful metric for decision makers as it quantifies losses on the basis of a time period rather than an assumed intensity scenario. The calculation of expected annual loss (EAL) is typically carried out by performing single intensity-based loss estimates over an interval of intensities that range from the initial accumulation of loss to an intensity that captures the upper bound of probabilistically significant events (Mitrani-Reiser 2007, ATC 2011). Plotting mean annual frequency (MAF) versus expected loss forms a total loss curve that can be integrated in order to calculate EAL. The basis of the Direct DBA loss model is the approximation of a total loss curve by defining four key limit states termed: zero loss, operational, damage control, and near collapse (refer Figure 4.1a). The zero loss limit state represents the initial threshold of loss accumulation at a substantial annual probability. The operational and damage control limit states define the transition region between significant MAF and low loss to low MAF and increased loss as shown in Figure 4.1a. Finally, the near collapse limit state signifies a level of demand where the combination of expected loss and annual probability allow for the attribution of a MDF value of 1.0, representing the replacement cost and the upper bound for EAL integration.

The preliminary limit state displacement thresholds are assumed in the form of peak inter-storey drift ratio (IDR) for the case-study building. The zero loss limit state is assumed to be exceeded at a peak IDR of 0.2% to 0.3% based on the initiation of non-structural damage (e.g. drywall partitions). The operational limit state threshold is set to 0.5% IDR representing significant non-structural damage and slight yielding in some members. The damage control limit state threshold was assumed to be between the expected yield drift and the modern drift limit set by ASCE 7-10 (ASCE 2010) for RC frames giving a range of 1.4% to 2.0% IDR. The near collapse limit state was set to 5.0% IDR to represent the onset of 2nd order instability as well as the possibility of significant residual deformation. Further information and discussion on the limit state thresholds can be found within Welch (2012).

4.2 Estimation of direct loss at a given limit state

In order to implement the simplified tri-linear loss model (Figure 4.1a), the two intermediate limit states (operational and damage control) require that the MDF value be quantified for EAL calculation. Following the PEER methodology, the direct loss at a given intensity is quantified with two terms: expected loss due to near collapse and loss due to collapse.

The use of storey-based engineering demand parameter to decision variable (EDP-DV) functions developed by Ramirez and Miranda (2009) are adopted here in order to estimate the losses given no collapse. Storey-based EDP-DV functions allow for the direct calculation of repair costs given a simple set of EDP's by integrating damage fragility and cost distribution before conducting analysis based on an assumed structural system, occupancy type, and expected storey value. EDP-DV functions were developed considering only peak inter-storey drift ratio (IDR) and peak floor acceleration (PFA) as EDP's in order to simplify the necessary output from structural analyses. The study conducted by Ramirez and Miranda (2009) implemented the functions in loss estimation for the current case study building which will allows for a more sound comparison with the proposed Direct DBA loss model (see section 5). A conceptual illustration of EDP-DV functions is shown in Figure 4.1b and further information can be found within Ramirez and Miranda (2009).

To accommodate the consideration of storey accelerations the simplified recommendations of ATC-58 (ATC 2011) are adopted for the trial methodology. According to ATC-58, the storey accelerations for frame buildings can be approximated as a function of the first mode spectral acceleration, $S_a(T_1)$, the peak ground acceleration, *PGA*, the base shear at yield from a first mode pushover analysis, and the effective lateral weight. By using the aforementioned parameters, combined with empirical coefficients, the expected storey accelerations can be approximated. Although the use of nonlinear pushover results is prescribed, the base shear calculated using the Direct DBA procedure at the yield displacement would also be a reasonable estimate. The details of the simplified storey acceleration relation can be found within the ATC-58 guidelines (ATC 2011).

The consideration of losses due to collapse can be simply obtained by multiplying the probability of collapse at a given intensity by the replacement cost of the building or including additional costs to represent tear down and clearing of the site. In the case of extending the Direct DBA procedure, a lognormal collapse fragility function is assumed based on recommendations of ATC-58. For a modern code conforming building, ATC-58 recommends that the median collapse intensity be approximated as three times the intensity corresponding to the drift limit intensity. Assuming a standard ASCE 7-10 (ASCE 2010) drift limit of 2.0% for RC frames, the equivalent spectral acceleration intensity (at T_1=0.8s) is calculated to be 0.72g using the Direct DBA approach. Multiplying by 3 gives an estimated collapse intensity of approximately 2.2g which is comparable to the median collapse intensity of 2.22g at a period of 0.86s reported by Haselton and Deierlein (2007). The lognormal standard deviation of the collapse fragility assumes the reported value of 0.65 from Haselton and Deierlein (2007) which is within the range of 0.45 to 0.7 recommended by ATC-58.

4.3 Incorporation of structural response uncertainty

One crucial difference between simplified analysis methods versus those implementing time history analyses considering suites of ground motions is that the inherent randomness or aleatory uncertainty between ground motions is not considered when estimating the response. With respect to the PEER PBEE methodology, the record to record variability is accounted for in the response estimation in the form of EDP probability distributions. Conversely, the proposed Direct DBA approach returns a single realization of EDP's necessitating that the uncertainty (dispersion) be accounted for deterministically. Similarly, the effect of modelling error, by definition, cannot be readily quantified through practical analytical means and the simplistic nature of the Direct DBA approach certainly merits the consideration of this type of uncertainty. As suggested by Sullivan and Calvi (2011), the incorporation of dispersion into the Direct DBA procedure can use a simplified form of the SAC/FEMA approach as described by Fajfar and Dolsek (2010). The basic closed form relation for incorporating uncertainty for a given limit state from the SAC/FEMA approach is shown in Eqn. 4.1:

$$P_{LS,x} = \tilde{H}\left(S_{a,\tilde{c}}\right) C_H C_f C_x \tag{4.1}$$

where $P_{LS,x}$ is the probability of exceeding a limit state with *x* confidence. The mean hazard (mean annual frequency), $\tilde{H}\left(S_{a,\tilde{C}}\right)$ is scaled by a series of factors: C_H representing the difference between mean and median hazard, C_x adjusting for the desired confidence level, and C_f which incorporates the dispersion in demand and capacity. By neglecting the difference between mean and median hazard and assuming a 50% confidence level is sufficient, the factors C_H and C_x become unity and only the effect of C_f is considered. The expanded form of the simplified relation is shown in Eqn. 4.2.

$$P_{LS,x} = \tilde{H}\left(S_{a,\tilde{C}}\right) \cdot C_f = \tilde{H}\left(S_{a,\tilde{C}}\right) \cdot \exp\left[\frac{k^2}{2b^2}\left(\beta_{DR}^2 + \beta_{CR}^2\right)\right] \tag{4.2}$$

The parameters β_{DR} and β_{CR} represent the demand (record to record) and capacity (modelling) dispersion values respectively while the parameter b is the exponential increase of demand with respect to intensity. Assuming a linear demand-intensity relationship, b becomes 1.0. The factor k represents the exponential of the hazard curve fit assuming the common power fit assumption. Due to lack of existing data, Fajfar and Dolsek (2010) assume that the values of β_{DR} and β_{CR} are 0.32 which are a reasonable approximation compared to collapse analysis by Haselton and Deierlein (2007) which found dispersion parameters in the range of 0.35 to 0.45.

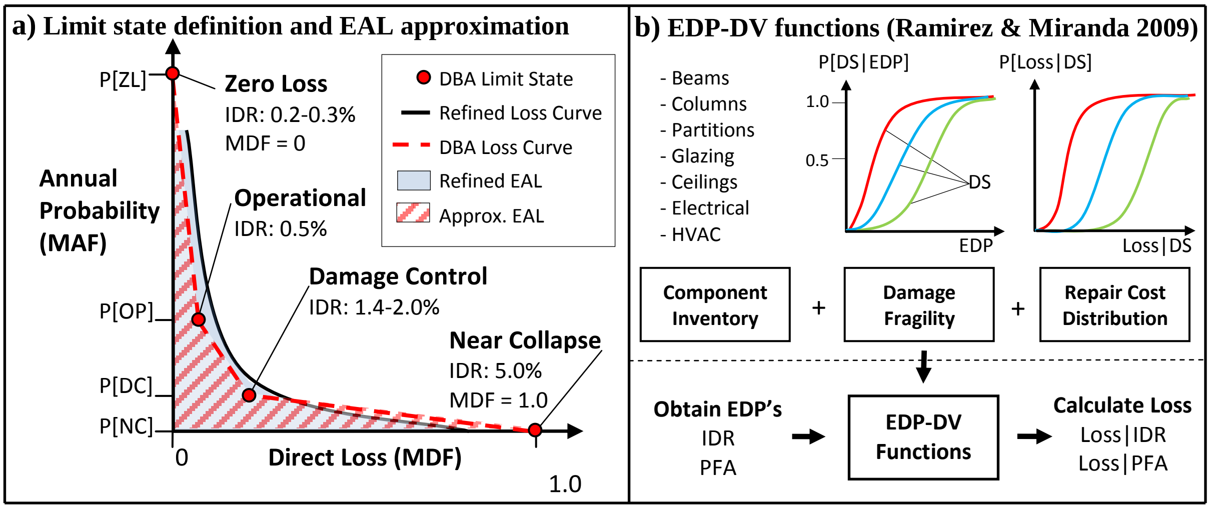


Figure 4.1. a) Proposed Direct DBA loss model with key limit states for case study building defined b) Conceptual diagram for implementing EDP-DV functions developed by Ramirez and Miranda (2009) adopted for estimating direct loss within the Direct DBA model

4.4 Definition of hazard curve fits

In order to account for dispersion using the simplified SAC/FEMA approach described in section 4.3, the total site hazard curve for a period of 1.0s (not shown) is approximated using local power fits in the form of $MAF=k_0(S_a(1.0s))^{-k}$, which are implemented order to determine k. Two hazard fits are assumed, a low intensity for the zero loss and operational limit states, and a design intensity fit for the damage control and near collapse limit states. The fits are determined by assuming a target spectral acceleration, $S_{a,target}$, and then fitting the hazard curve between 0.25 and 1.5 times this value as per recommendations of Dolsek and Fajfar (2008). The low intensity fit assumed a target value of 0.3g which was controlled by the lower bound of the fit and the design fit assumed 0.55g corresponding to the Design Basis Earthquake (DBE) intensity level of 10% in 50 years. The hazard fit parameters are displayed in Table 4.1 below with R^2 values comparing the fit to the full hazard curve.

Table 4.1. Hazard curve fits used in simplified loss estimation

DESC.	k	k_0	$S_{a,target}$ (g)	$S_{a,range}$ (g)	R^2
Low-Intensity	1.8	0.000195	0.3	0.05 - 0.45	0.986
Design	2.8	0.00035	0.55	0.1 - 0.8	0.976

5. COMPARISON OF DIRECT DBA LOSS ESTIMATION WITH PEER PBEE STUDIES

The preliminary Direct DBA loss model is compared with two previous studies on 4-storey RC space frame buildings. The first study conducted by Mitrani-Reiser (2007) implemented the PEER PBEE methodology using component-based fragility functions assuming a replacement cost equal to the total repair cost of all damageable components that were assigned damage fragility. The second study conducted by Ramirez and Miranda (2009) implemented the PEER methodology using storey-based EDP-DV functions (see section 4.2) assuming a replacement cost considering all components within the building. Both studies assumed the same site location, office occupancy, and similar site hazard characteristics. The Direct DBA approach is compared to the two studies using expected annual loss (EAL). Mitrani-Reiser (2007) reported EAL of 0.55% of the building replacement cost while Ramirez and Miranda (2009) found the EAL to be approximately 0.91%. The Direct DBA model is implemented considering variations of the limit state ranges displayed in Figure 4.1a. The EAL results shown in Figure 5.1 are presented such that each variation shows the initial loss estimate (darker section) with the influence of dispersion highlighted above while the values from the previous PEER PBEE studies are shown as horizontal lines for comparison.

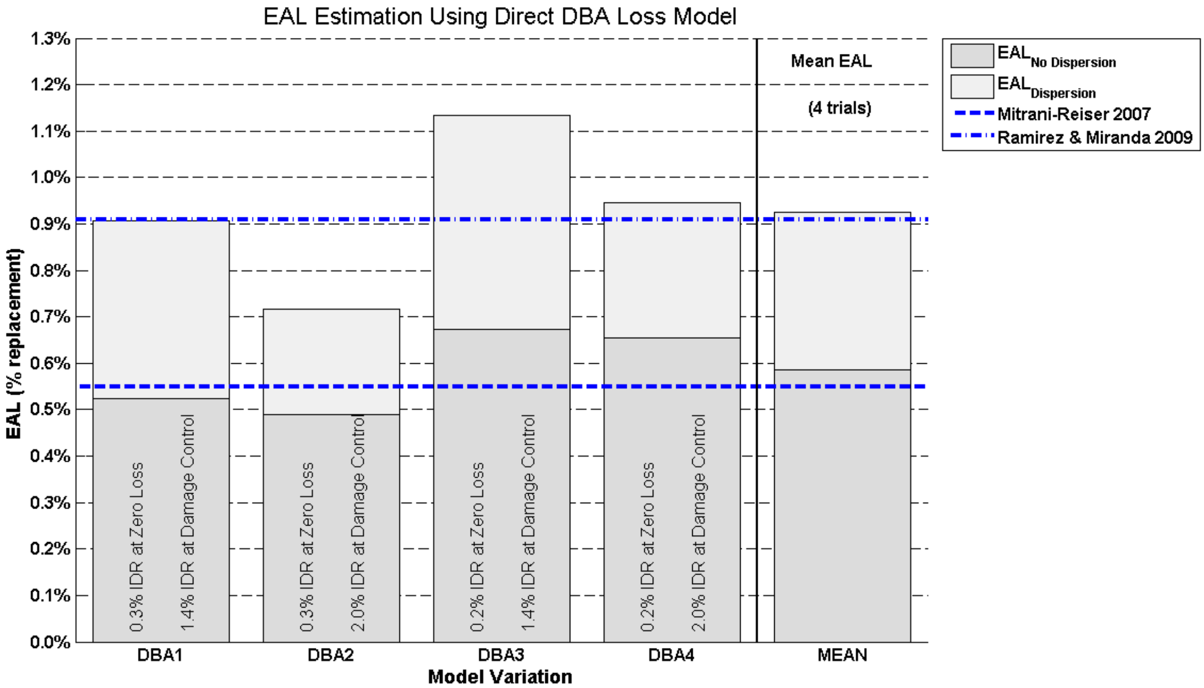


Figure 5.1. Expected annual loss estimation from the Direct DBA loss model compared with previous PEER PBEE results for the case study building

The EAL results from the Direct DBA show a reasonable agreement with previous PEER PBEE studies with EAL ranging from 0.49% to 1.13% the building replacement cost. Although a small number of limit state variations are presented, the mean values essentially define the range of EAL from the PEER studies which suggest that similar reliability may be incorporated through further development of the methodology. More detailed information about the implementation the proposed Direct DBA approach can be found within Welch (2012).

6. CONCLUSIONS

The Direct DBA approach is a relatively simple assessment methodology but has received limited probabilistic considerations. The PEER PBEE approach offers comprehensive probabilistic building-specific loss assessment but is arguably too complex for practicing engineers. As such, this paper extends the DBA approach of Priestley *et al.* (2007) to incorporate probabilistic considerations in line with recommendations of Sullivan and Calvi (2011), developed to provide a means of quantifying performance with direct loss estimation. The trial application of the procedure to a 4-storey RC building suggests good potential and that extending Direct DBA for building-specific loss assessment is feasible. Future research should consider the refinement of methodology as well as different structural typologies through the examination of additional case study buildings.

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