The Behaviour of Replaceable Buckling Restrained Fuses (RBRFs) in Composite Structures under Earthquake Events

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Abstract

Unpredictable destructive events, both natural disasters and man-made disasters, generate the awareness that buildings need to be protected. Ideally, the protection of buildings can be achieved by making the structures more robust by aiming for a performance objective of operational under a high-level event. This paper considers the use of replaceable buckling restrained fuses (RBRFs) as an energy dissipation device to protect basic importance buildings constructed using composite moment-resistant frames as the lateral force-resisting system. The structural elements are designed to remain elastic, with the exception of the RBRFs. These RBRFs could be replaced after a major event and hence would cause little disruption. A 2D building frame has been modelled for a case study and its behaviours under 100-, 500- and 2500-year return period earthquake events have been summarised. The use of RBRFs could offer an economic solution for protecting the building from major earthquake events and would lead to a speedy recovery after the event.

Keywords: replaceable buckling restrained fuse (RBRF), basic importance building, low-damage connections, earthquake event

1. INTRODUCTION

Unpredictable destructive events, both natural disasters such as those caused by an earthquake, tsunami or wind storm, and man-made disasters such as those caused by terrorism and fire, generate the awareness that buildings need to be protected. In the case of earthquake events, SEAOC (1995) defines the performance objectives for three types of buildings, ordinary, essential, and critical buildings, under various earthquake intensities as shown in Figure 1. Based on the probability of occurrence, earthquakes can be categorised as frequent, occasional, rare, and very rare with recurrence intervals of 43 (50% in 30 years), 72 (50% in 50 years), 475 (10% in 50 years), and 975 years (10% in 100 years), respectively. The return period of a very rare earthquake varies from one publication to another. In this paper, following the recommendations in (Buchanan et al. 2011, FEMA-274 1997) a 2500 year return period (2% in 50 years) has been adopted to represent a very rare earthquake.

If using the SEAOC design philosophy represented in Figure 1, the level of damage should be less for more important facilities and more for less important facilities during the same level of earthquake. SEAOC (1995) defines the performance levels using both a drift limit and the degree of repairability. The former does not necessarily reflect the damage to the building itself since some types of buildings could have a high flexibility and will be able to withstand relatively high drifts with little or no damage. Nevertheless, the damage to non-structural elements is usually considered to be related strongly to the drift limit. The degree of repairability, however, is difficult to quantify since it depends on the structure's ability to deform under an earthquake excitation. To account for this, FEMA-274 (1997) defines three discrete performance levels, which are Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP); in this case, these limits are based on the ductility of the element or building. The displacement capacity at the CP level is the ultimate displacement that the building/element can sustain without loss of axial load capacity. The displacement capacity at the LS level is estimated to be 75% of the ultimate displacement capacity in FEMA-274 (1997). Further to this, in this paper, 25% of the ultimate displacement capacity is adopted as the estimated displacement capacity at the IO level.

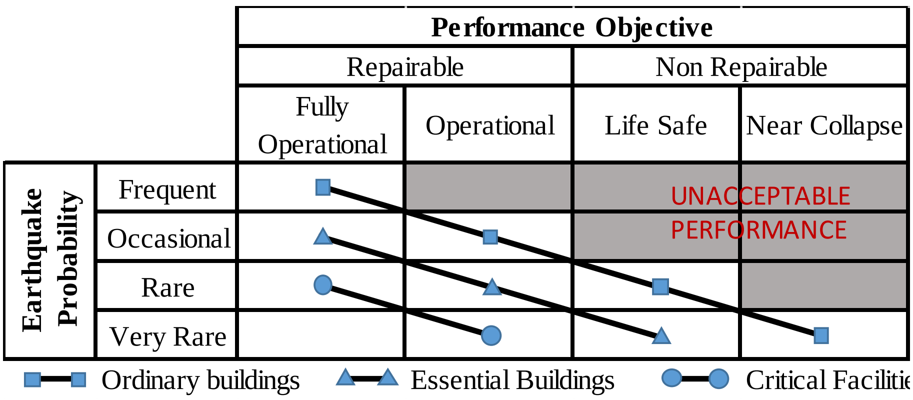


Figure 1 Performance matrix adapted from SEAOC (1995)

Following the Christchurch earthquake, Buchanan et al. (2011) proposed changes to the earthquake performance objectives and suggested that all buildings should remain operational after an earthquake, even after a very rare one, as shown in Figure 2. Buchanan et al. (2011) suggested two options to achieve this, which is by increasing the level of seismic design loading and/or by designing for a higher performance building using base isolation or damage-resistant technologies. This paper follows the latter option and considers the use of replaceable buckling restrained fuses (RBRFs) as energy dissipation devices to protect buildings constructed using composite moment-resistant frames as the lateral force-resisting system. The structural elements are designed to remain elastic, with the exception of the RBRFs. These RBRFs could be replaced after a major event and hence would cause little disruption. The illustration of the RBRF is shown in Figure 3. A 2D building frame has been modelled for a case study and its behaviours under 100-, 500- and 2500-year return period earthquake events have been summarised. The use of RBRFs could offer an economic solution for protecting the building from major earthquake events and would lead to a speedy recovery after the event. Furthermore, conclusions and further recommendations have also been summarised.

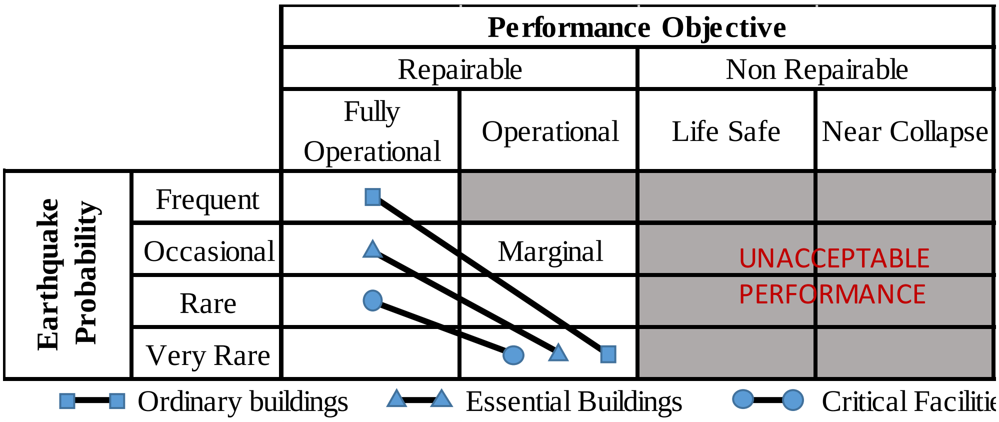


Figure 2 Performance matrix proposed by Buchanan et al. (2011)

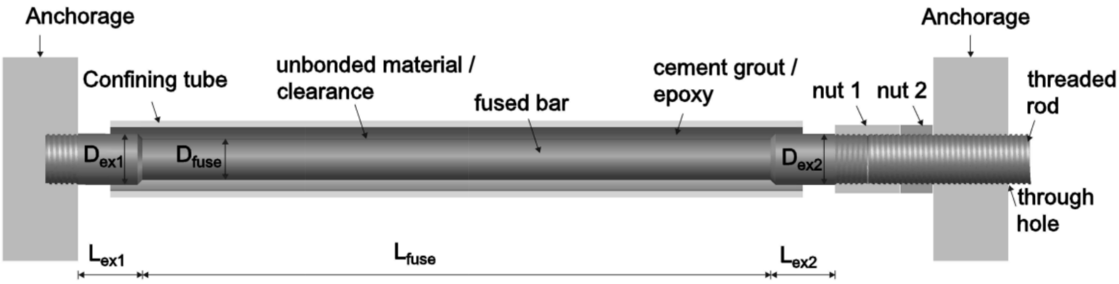
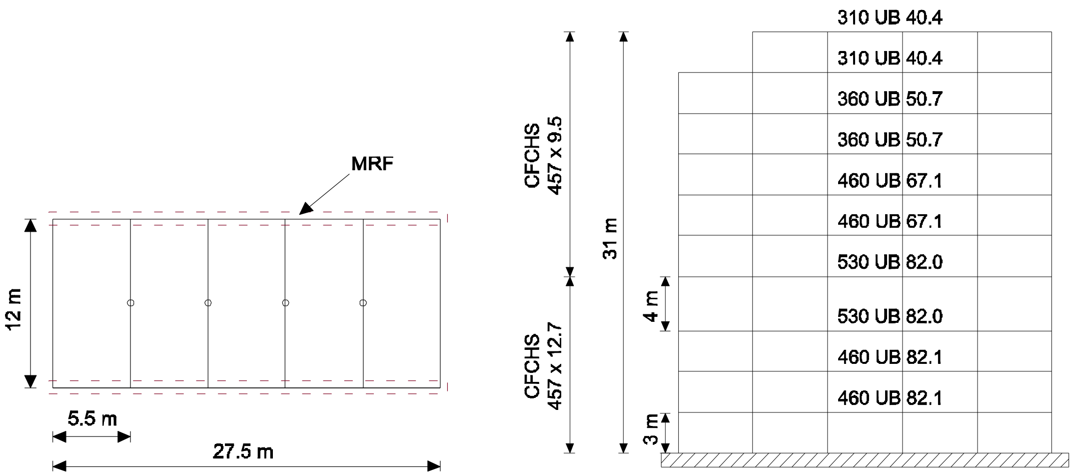


Figure 3 RBRF's component

2. CASE STUDY

The case study used an adaptation of steel moment resisting frames (SMRFs) designed for the Te Puni Village Tower Building at Victoria University in New Zealand, which has 5-bays and 10-storeys and is built on rock (Soil Class B according to NZS 1170.5 (2004)). This study used concrete-filled circular hollow sections (CFCHSs) as the columns, therefore, all open section columns in the original design were replaced with CFCHS sections, each with a comparable flexural stiffness to the original column. However, the plan and elevation view of the building are still the same as in the original building. Figure 4 shows the plan and elevation views of the building with the modified sections used in this study (adapted from Khoo et al. (2012)). It is assumed that this building has a basic importance level in which it should be operational or damage control under 500- and 2500-year return period event following the performance matrix as shown in Figure 2. A seismic weight of dead load plus 0.3 live load was applied to the building. Only one of the MRFs was analysed since the lateral loading would be distributed equally between the frames. Moreover, a 2D model was used since there are bracings in the transverse direction of the building which will minimise the deformation of this building in that direction.

Under a 500-year return period earthquake (DBE), a structural ductility factor (μ) and structural performance factor (S_p) equal to 2 and 0.7, respectively, were used (NZS 1170.5 2004) in the Response Spectrum Analysis (RSA). Furthermore, considering Near-Fault regions, the moment demand at each end of the beam was obtained using RSA and the results were used for designing the dimension of the Replaceable Buckling Restrained Fuse (RBRF). The summary of the maximum moment at each end of the beam at each level and the corresponding RBRF dimensions is shown in Table 1. The dimensions of the RBRF were calculated based on following assumptions: yield and ultimate strength of 570 and 710 MPa (Oktavianus et al. 2016), ultimate strain of 6%, the post-yield stiffness of 1% of elastic stiffness, ratio of maximum compression force to maximum tensile force of 1.2, slenderness ratio of 60, and elastic stiffness reduction of approximately 20% due to the semi-rigid T-stub connections (Oktavianus et al. 2015). Table 1 also shows the yield moment (M_y), yield rotation (Θ_y), and ultimate rotation of the RBRF (Θ_u).



(a) (b)

Figure 4 Building layout: (a) plan view; (b) elevation view

Table 1 Summary of the maximum moment at the end of the beams and the corresponding RBRF dimensions

Storey	Maximum moment (kNm) Exterior connection	Maximum moment (kNm) Interior connection	Properties of RBRF* L_{ex} (mm)	D_{ex} (mm)	D_{fuse} (mm)	L_{fuse} (mm)	D_{int_tube} (mm)	D_{ext_tube} (mm)	L_{tube} (mm)	M_y (kNm)	Θ_y (mrad)	Θ_u (mrad)
10	61.14	57.02	50	33	25	375	42.3	60	445	106.46	8.86	65.94
9	72.55	73.96	50	33	25	375	42.3	60	445	106.46	8.86	65.94
8	129.50	128.48	50	36	27	405	47.2	71.4	475	144.87	7.88	60.67
7	138.52	138.76	50	36	27	405	47.2	71.4	475	144.87	7.88	60.67
6	236.64	225.89	50	42	32	480	52.3	71.4	550	248.42	7.48	58.70
5	234.05	227.83	50	42	32	480	52.3	71.4	550	248.42	7.48	58.70
4	350.06	318.74	50	48	36	540	56.9	85	610	365.23	7.58	57.27
3	364.35	332.51	50	48	36	540	56.9	85	610	365.23	7.58	57.27
2	300.14	282.99	50	48	35	525	56.9	85	595	307.93	7.81	61.88
1	261.17	239.88	50	42	33	495	52.3	71.4	565	267.11	7.13	59.30

* f_y = 570 MPa; f_u = 710 MPa; M_u^+ =1.2M_y ; M_u^- =1.56M_y

Figure 5 shows the hysteresis loop of the RBRF used in this research. Since the beams and columns were kept elastic even until the failure of the RBRF, there is no need to include beam or column plastic hinges in the model. Moreover, the IO, LS and CP performance levels will be used in each RBRF as shown in Figure 5. Even if an elasto-plastic hysteresis loop is assumed for the RBRFs, the residual displacements of the structure after the DBE and MCE are expected to be small due to the shake-down effect and the elastic condition of the other structural elements. This residual deformation will be examined using non-linear response history analysis (NRHA) in the subsequent section.

Both the capacity spectrum method (CSM) and nonlinear response history analysis (NRHA) were performed in this research using ETABS (CSI 2015). The first was used to identify the strength hierarchies in the building and the sequence of hinge occurrence in the structure. The latter provided information about the overall behaviour of the building taking into account the higher mode effects. P-Δ effects were considered in all analyses. The capacity spectrum method used in this research followed Eurocode 8 (CEN 2004) and is called the "N2 method". This was chosen because it does not require any iteration process to obtain the performance point and gives a reasonably accurate prediction, especially for a first-mode dominant building (Causevic and Mitrovic 2011).

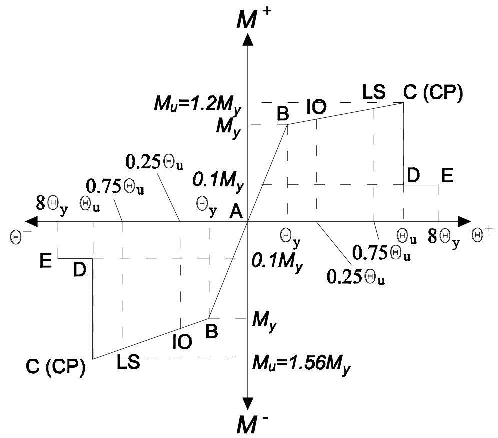


Figure 5 Hysteresis loop of RBRF

Table 2 shows the selected and scaled ground motions used in this research which were obtained from the PEER database (PEER 2013). The ground motions were selected assuming that the site was in a near-fault region and the target response spectrum of NZS1170.5 for a 500-year return period (Design Based Earthquake, DBE) for soil class B was used to scale them with a weighting factor of 1 for periods between 0.1 to 5 sec such that they have minimum mean squared error (MSE) to the target spectra. Both the target spectra from (NZS 1170.5 2004) and the mean spectra from the scaled ground motions under DBE are shown in Figure 6. For the 2500-year return period demand (Maximum Considered Earthquake, MCE), an additional magnifier factor of 1.8 was applied to the scaled ground motions listed in Table 2. Furthermore, 100-year return period earthquakes (Serviceability Level Earthquake, SLE) were also applied to the structure by applying a factor of 0.5 to the scaled ground motions listed in Table 2. The maximum displacement at each storey and also the corresponding residual displacement are summarised and discussed in the next section within this paper.

Table 2 Selected and scaled ground motion

No.	Earthquake Name	Year	Station Name	Magnitude	Mechanism	Rrup (km)	Scale Factor
1	"Irpinia_ Italy-01"	1980	"Bagnoli Irpinio"	6.90	Normal	8.18	2.35
2	"Loma Prieta"	1989	"Gilroy - Gavilan Coll."	6.93	Reverse Oblique	9.96	1.42
3	"Northridge-01"	1994	"LA Dam"	6.69	Reverse	5.92	0.85
4	"Kobe_ Japan"	1995	"Nishi-Akashi"	6.90	strike slip	7.08	0.98
5	"Chi-Chi_ Taiwan"	1999	"TCU122"	7.62	Reverse Oblique	9.34	1.32
6	"Duzce_ Turkey"	1999	"Lamont 531"	7.14	strike slip	8.03	3.53
7	"Tottori_ Japan"	2000	"SMNH01"	6.61	strike slip	5.86	1.01
8	"Christchurch_ New Zealand"	2011	"LPCC"	6.20	Reverse Oblique	6.12	0.90

3. RESULTS AND DISCUSSIONS

Modal analysis was performed in the linear elastic range and the natural period of the building was found to be equal to 1.96 s which is similar to the one mentioned by (Khoo et al. 2012). The modal participating mass ratios obtained from modal analysis were equal to 75.5%, 11.5% and 5.7% for the first three dominant modes. This shows that the behaviour of the structure will be governed by the first mode. However, this also shows that the contribution of the higher modes cannot be neglected.

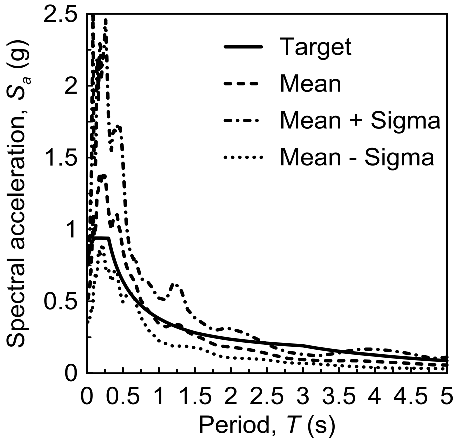


Figure 6 Target spectra and the mean value of the selected ground motions for 500-year return period.

The results obtained from the N2 method are shown in Figure 7. Inelastic response spectra (IRS) was used instead of elastic response spectra (ERS) to obtain the performance point of the structure. It shows that the performance point for the DBE occurred at spectral displacement (S_d) and spectral acceleration (S_a) of 228.6 mm and 0.124g, respectively. This is equivalent to a roof displacement of 318 mm or roof drift of 1.03%. Moreover, a ductility of 1.92 occurred at this stage; this is expected since the building was designed with ductility of 2 under the DBE as mentioned previously. Moreover, the performance point for the MCE occurred at an S_d and S_a of 351.3 mm and 0.132g, respectively. This is equivalent to a roof displacement of 495 mm or roof drift of 1.6%. From the pushover analysis, it was determined that the structure has a roof displacement capacity of approximately 1343 mm or roof drift of 4.3% at which point the fracture of the RBRFs at level 6 occurred which indicates that the collapse prevention (CP) stage at the RBRFs, shown in Figure 5, has been reached. This means that the structure could sustain an earthquake even larger than the MCE earthquake before the fracture of the RBRFs. The structure has roof displacements equal to 428 mm (roof drift of 1.4%) and 1053 mm (roof drift of 3.4%) when the RBRFs reach their IO and LS performance level, respectively. Moreover, the contribution of the higher modes may alter the displacement and drift demand obtained from the N2 method which only considers the first mode of the building.

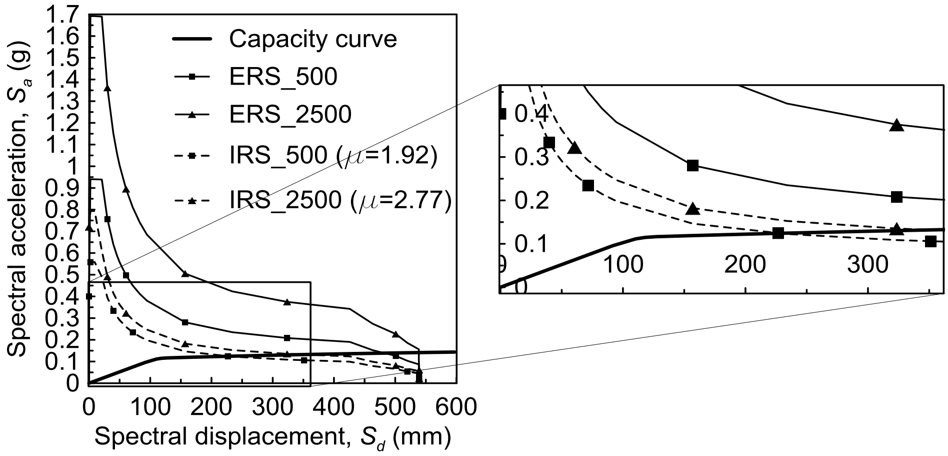


Figure 7 Performance point obtained by using N2 method

In order to take the higher mode effects into account, NRHA was performed. Figure 8 shows the maximum inter-storey drift response in each storey. The mean values of maximum inter-storey drift that resulted from all of the selected and scaled ground motions listed in Table 2 are 0.38%, 0.74% and 1.47% for the SLE, DBE and MCE, respectively. The Irpinia and Northridge earthquakes generated much larger response than the mean value. The Irpinia earthquake caused maximum inter-storey drifts of 0.62%, 1.17% and 2.48% for the SLE, DBE, and MCE, respectively. The Northridge Earthquake caused maximum inter-storey drifts of 0.85%, 1.39% and 2.33% for the SLE, DBE, and MCE, respectively. The mean values of maximum absolute roof displacement of the building due to the SLE, DBE and MCE are 105, 193 and 347 mm, respectively. However, the maximum absolute roof displacements are caused by the Northridge earthquake and these are 212, 346, and 548 mm under the SLE, DBE and MCE, respectively. Based on the performance levels obtained in the pushover analysis, the building was operational even under the DBE Northridge earthquake and had a performance level of immediate occupancy (IO) under the MCE Northridge earthquake. Northridge earthquake generated the maximum displacement under the SLE and DBE, and Irpinia earthquake produced the maximum displacement under the MCE as shown in Figure 8. In addition, according to NZS 1170.5 (2004), the allowable maximum inter-storey drift is 2.5% and the structure is generally well below this limit.

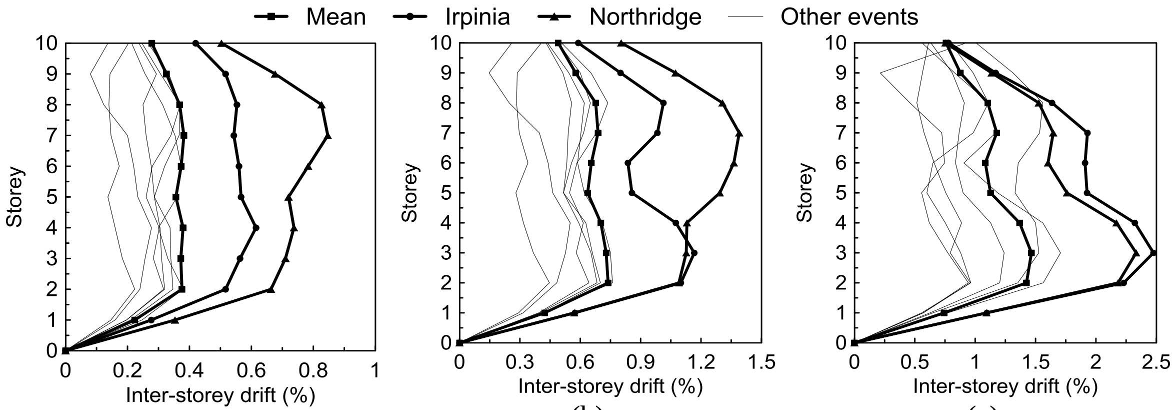


Figure 8 Maximum inter-storey drift obtained from NRHA: (a) SLE:100-year return period; (b) DBE: 500-year return period; and (c) MCE: 2500-year return period.

Figure 9 shows the residual inter-storey drift obtained from the NRHA. This residual inter-storey drift was obtained by adding 20 seconds of zero acceleration to the building after each event. The mean values of residual inter-storey drift resulting from 8 ground motions are 0.03%, 0.16% and 0.3% for the SLE, DBE and MCE, respectively. Only the Northridge earthquake caused very large residual inter-storey drifts with values of 0.23%, 0.7% and 0.8% for the SLE, DBE, and MCE, respectively. Considering the construction tolerance of 0.2% suggested in NZS 3404 (1997) for buildings up to 60 m height as the maximum allowable residual drift, the residual drift generated by all of the earthquakes was larger than this limit under the MCE. However, only the Northridge earthquake caused this limit to be exceeded under the DBE.

4. LIMITATIONS

There are several limitations in this study as follows:

1. The RBRFs used in this research were designed using some assumptions which need to be confirmed through experimental work. The assumptions are as

follows: yield and ultimate strength of 570 and 710 MPa, ultimate strain of 6%, the post-yield stiffness of 1% of elastic stiffness, ratio of maximum compression force to maximum tensile force of 1.2, slenderness ratio of 60, and elastic stiffness reduction of approximately 20% due to the semi-rigid T-stub connections.
2. If an inter-storey drift limit of 0.2% is used as the maximum allowable residual drift, this design using RBRFs would need to be improved. One solution would be by adding self-centering devices.
3. The behaviour of the building under an SLE event following the DBE or MCE has not been analysed in this study. This is important to ensure that the building could maintain its rigidity.

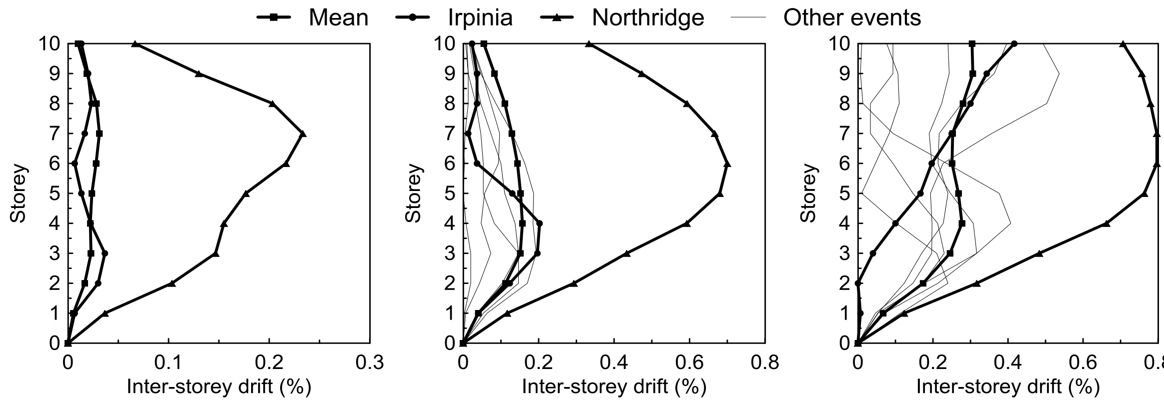


Figure 9 Residual inter-storey drift obtained from NRHA: (a) SLE:100-year return period; (b) DBE: 500-year return period; and (c) MCE: 2500-year return period.

5. CONCLUSIONS

This paper considers the use of replaceable buckling restrained fuses (RBRFs) as energy dissipation devices to protect basic importance buildings constructed using composite moment-resistant frames as the lateral force-resisting system. The structural elements are designed to remain elastic, with the exception of the RBRFs. A 2D building frame has been modelled and analysed using both the Capacity Spectrum Method (CSM) and Nonlinear Response History Analysis (NRHA). Several conclusions have been made as follows:

1. The modal participating mass ratios obtained from a modal analysis in the linear-elastic range were equal to 75.5%, 11.5% and 5.7% for the first three dominant modes. This shows that the behaviour of the structure will be governed by the first mode. However, this also shows that the contribution of the higher modes cannot be neglected.
2. The Capacity Spectrum Method was used to identify the sequence of hinge occurrence in the structure and to estimate the deformation capacity of the building. The results show that a roof displacement of 318 mm and 495 mm are obtained under the DBE and MCE, respectively. Moreover, the structure has a maximum roof displacement capacity of approximately 1300 mm prior to fracture of the RBRFs at level 6. This means that the structure could sustain an earthquake even larger than the MCE earthquake before the RBRFs would be expected to fracture.
3. Non-linear Response History Analysis (NRHA) was performed to consider higher mode effects. According to NZS 1170.5 (2004), the limit for maximum inter-storey drift is 2.5% and the structure has a maximum inter-storey drift

demand which is below this limit for all of the earthquakes considered in the NRHA.

4. Using the construction tolerance of 0.2% suggested in NZS 3404 (1997) for buildings up to 60 m height as the maximum allowable residual drift, the residual drifts generated by all of the earthquakes considered in the NRHA were larger than this limit under the MCE. However, only the Northridge earthquake caused this limit to be exceeded under the DBE.
5. The use of RBRFs could offer an economic solution for protecting the building from major earthquake events and would lead to a speedy recovery after the event since these RBRFs could be replaced and hence would cause little disruption, especially under the DBE.

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