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COST-EFFECTIVE CONSIDERATION OF NON-STRUCTURAL ELEMENTS: LESSONS FROM THE CANTERBURY EARTHQUAKES

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ABSTRACT

By closely examining the performance of a 22-storey steel framed building in Christchurch subject to various earthquakes over the past seven years, it is shown that a number of lessons can be learnt regarding the cost-effective consideration of non-structural elements. The first point in this work is that non-structural elements significantly affected the costs associated with repairing steel eccentrically braced frame (EBF) links. The decommissioning or rerouting of non-structural elements in the vicinity of damaged links in the case study building attributed to approximately half the total cost of their repair. Such costs could be significantly reduced if the original positioning of non-structural elements took account of the potential need to repair the EBF links. The second point highlighted is the role that pre-cast cladding apparently played on the distribution and type of damage in the building. Loss estimates obtained following the FEMA P-58 framework vary considerably when cladding is or isn't modelled, both because of changes to drift demands up the height of the building and because certain types of subsequent damage are likely to be cheaper to repair than others. Finally, costly repairs to non-structural partition walls were required not only after the moment magnitude 7.1 earthquake in 2010 but also in multiple aftershocks in the years that followed. Repair costs associated with aftershock events exceeded those from the main event, emphasizing the need to consider aftershocks within modern performance-based earthquake engineering and also the opportunity that exists to make more cost-effective repair strategies following damaging earthquakes.

Keywords: Non-structural elements; seismic loss; repair and retrofit; Canterbury earthquakes

1. INTRODUCTION

In the design and construction of a building it could be expected that achieving superior performance will require additional expense. Thus, a good engineer will endeavour to identify the most cost-effective means of achieving a desired level of performance for a building. Traditionally, structural engineers have attempted to identify cost-effective design solutions by refining details of the structural system whilst satisfying code requirements. However, this approach does not recognize the benefits of design solutions that are likely incur less damage and losses during the life of the building. To this extent, damage to non-structural elements, such as architectural components or mechanical and electrical equipment, often far exceed losses from damage to structural elements as was seen in, for example, the February 28, 2001 Nisqually, Seattle, Earthquake (Filiatrault et al., 2001) and the 2006 Kona, Hawaii,

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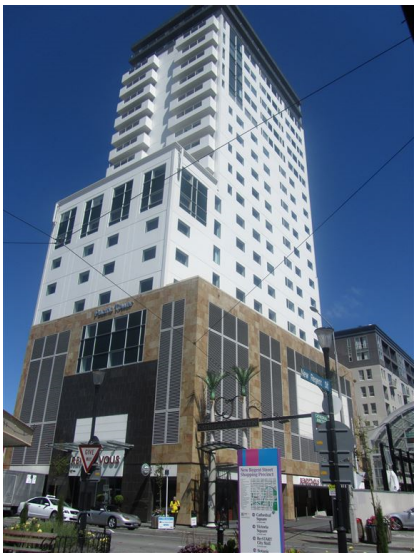
Earthquake (Chock et al., 2006; Gupta and McDonald, 2008). Thus, it would appear that adequate consideration of non-structural elements will be key to identifying cost effective design, retrofit and/or rehabilitation strategies.

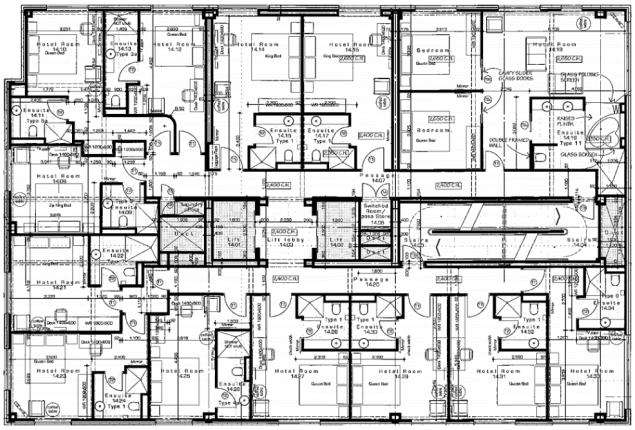
A number of publications can be found that highlight the relevance of non-structural elements to losses (e.g. Hunt, 2010) or propose improved detailing of non-structural elements (e.g. Araya-Letelier and Miranda, 2012) or even retrofit that focusses on non-structural elements (e.g. Calvi et al. 2014). However, the Canterbury earthquakes revealed a number of additional considerations that should be made in relation to non-structural elements in order to mitigate the likely repair costs to buildings in future earthquakes. This paper will explain the lessons learnt via examination of a 22-storey building that was damaged and repaired during the Canterbury earthquake sequence.

2. DESCRIPTION OF THE CASE STUDY BUILDING

A photo of the 22-storey case-study building, built in 2010 and located in the central business district of Christchurch, is shown in Figure 1(a) and an architectural layout of a typical floor (14th floor) is shown in Figure 1(b). The building is mixed-use, with carparking over lower storeys and hotel/residential use over upper storeys.



(a)



(b)

Figure 1. (a) Photo of 22-storey case study building from street level, (b) architectural plan from level 14.

2.1 Structural System

The building incorporates a structural steel system, with a number of eccentrically-braced frames resisting providing the main lateral resistance. The flooring is realised with composite reinforced concrete slabs on metal decks and composite steel beams. Figure 2 illustrates a 3D structural analysis model (described further in Section 3) in which it can be seen that the distribution of EBF links is not completely regular with height (in order to match architectural requirements) and some moment resisting frames (MRFs) are provided at the top levels. The building is sitting on concrete piled foundations. Cladding is also identified within Figure 2 for reasons that will be explained in Section 3.

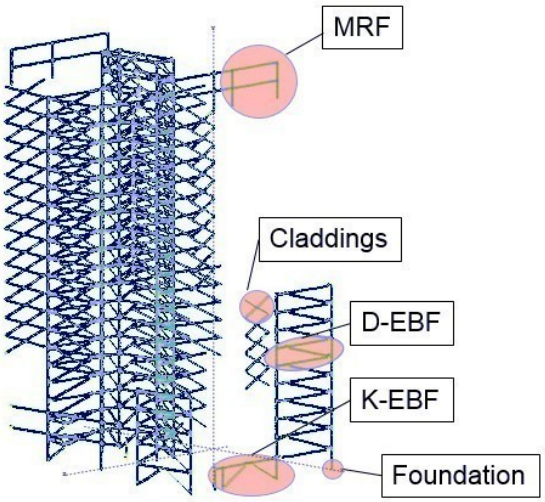


Figure 2. Perspective view of 3D structural analysis model developed in Ruaumoko (Carr, 2017).

2.2 Non-Structural Elements

The non-structural elements present are those that would be typically expected for this use and type of building. Internal partitioning is provided via lightweight metal framed plasterboard walls and suspended ceilings are present. As can be seen in Figure 1, the building is clad principally with perforated precast concrete panels.

In New Zealand, there does not appear to be a standard detailing approach for precast concrete elements and the cladding connection detail (obtained from the Christchurch City Council) for this building is shown in Figure 3. Note that this detail shows the bottom of the precast concrete panels to be integral with the composite concrete flooring system whereas the top of the panels are restrained via 75mm x 75mm steel angle sections of varying lengths. The angles are connected with single bolts to the steel framing at one end and are welded to steel plates, which are in turn fixed to the concrete panels via anchor bolts, as shown. The figure also shows that a gap is detailed between adjacent panels although talking with local engineers there is some doubt about whether this was provided during construction.

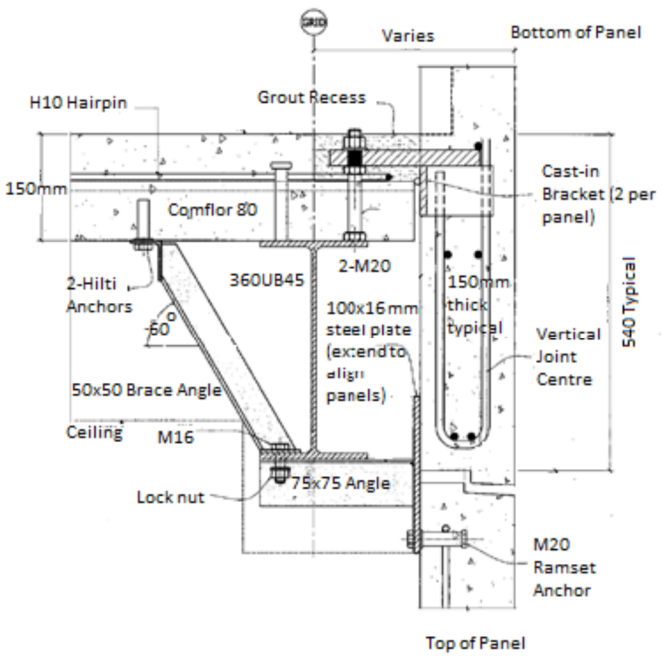
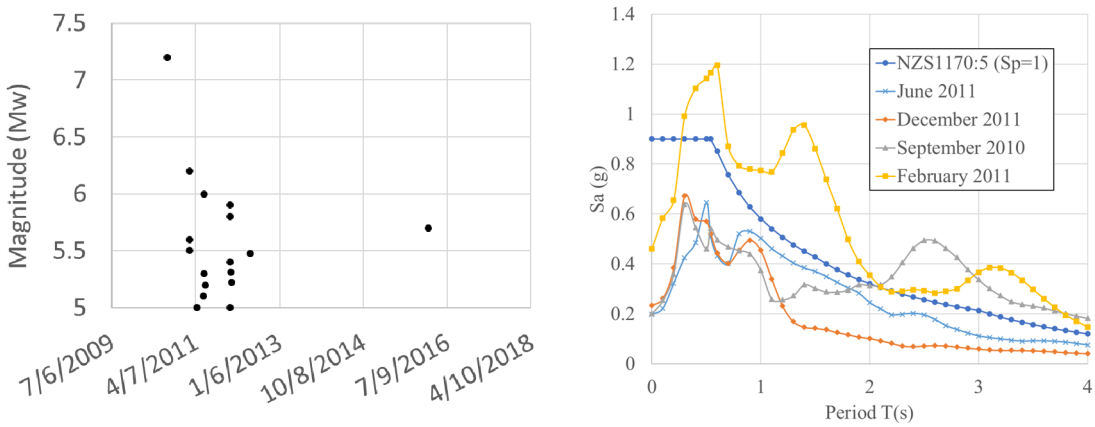


Figure 3. Precast cladding connection detailed for the 22-storey building.

3. IMPACT OF THE CANTERBURY EARTHQUAKE SEQUENCE

3.1 The Canterbury Earthquake Sequence

The Canterbury region has undergone a period of intense seismic activity over the past seven years. Figure 4(a) shows the earthquakes of magnitude greater than or equal to 5.0 that occurred within a 60km radius of the Christchurch central business district (CBD) since 2010. The main earthquake, referred to as the Darfield earthquake, was magnitude 7.1 and occurred around 40km from the Christchurch CBD. Damage to Christchurch from the main shock was significant but relatively minor compared to the damage caused by the magnitude 6.2 aftershock that occurred on February 22nd 2011. This is because the February aftershock, being located closer to the CBD, was more intense, as can be seen from Figure 4(b) which plots N-S response spectra recorded in the CBD near the case study building for the main earthquake and a number of significant aftershocks. Note that the aftershock sequence has been particularly long, with a magnitude 5.7 aftershock occurring as recently as February 2016. The stated design level shaking in NZS1170.5 for the ultimate limit state (500 year intensity level) is given in Figure 4(b). Note that the soil class for Christchurch was identified as Class D and that design of ductile EBF structures in New Zealand is commonly undertaken using a seismic performance factor, S_p, of 0.7 together with a ductility factor of 3.0, to arrive at an equivalent force reduction factor (also known as behaviour factor in Europe) of 4.3.



(a) (b)

Figure 4. (a) Record of earthquakes of magnitude Mw>5.0 within a 60km radius of the Christchurch CBD since 2010, (b) N-S response spectra, averaged across 3 sites (CHHC, CBGS and CCCC) in the CBD close to the case study building, from the most intense events.

3.2 Performance of the steel EBF structure

Considering the intensity of the earthquake shaking, the steel EBF structure performed reasonably well, as reported by Clifton et al. 2011, and the building was able to be repaired economically. A maximum residual drift in a storey was reported to be 0.3%. Figure 5 illustrates damage that occurred to EBF links, and the worst damage observed to a link, a fracture, can be seen in Figure 5(a). Figure 5(b) illustrates a link that has also undergone significant strains, as is evident from the flaked off paint, but has not fractured. Repair of such links (that did not fracture) may require heat treating and straightening. Also note the non-structural elements located close to the link itself.

(a) (b)

Figure 5. (a) Photo of fracture of an EBF link at level 6 after the February 2011 event, and (b) photo showing non-structural elements in the vicinity of a less damaged EBF link, from Clifton et al. (2011).

3.3 Performance of the precast cladding system

The cladding was damaged a moderate amount in the Canterbury earthquakes, with damage occurring to either the steel angle connections or via cracking of concrete around the bolts that anchor into the precast units (refer Figure 3). The reported damage to the panels is expected to have arisen because of the unusual connection details shown in Figure 3, which meant that the panels provided some restraint against the lateral movement of the main structure.

3.4 Performance of the non-structural partition walls

Experimental testing of plasterboard partition walls (e.g. Davies et al. 2011) has shown that they are currently one of the most fragile non-structural elements in modern buildings, with damage initiating at storey drifts as low as 0.2%. Thus, it is perhaps not a surprise that the plasterboard partitions suffered reasonably extensive damage in a number of the earthquakes in the Canterbury sequence. In some parts of the case-study building it is understood that the partitions were repaired five times. Note that such repairs to plasterboard walls around stairwells and access cores were particularly important to re-establish the fire ratings of the linings in these areas.

4. CONSIDERATION OF NON-STRUCTURAL ELEMENTS FOR MORE COST-EFFECTIVE DESIGN, RETROFIT OR REHABILITATION STRATEGIES

In order to identify cost-effective seismic retrofit and rehabilitation strategies for buildings such as the one examined here, this section describes findings from application of the FEMA P-58 (2012) (also referred to as the PEER PBEE framework), loss assessment procedure to the case study building. The hypothesis is that by estimating the expected annual losses for different retrofit strategies, and considering these against the costs associated with implementation of retrofit, more cost-effective interventions can be identified.

In order to estimate monetary losses in line with the FEMA P-58 framework, four analysis phases need to be followed: (i) probabilistic seismic hazard assessment in order to identify the likelihood of different levels of ground shaking intensity, (ii) structural analysis to identify the likely response of the building, in terms of engineering demand parameters (EDP, such as drift and floor acceleration), for a given level of ground shaking intensity, (iii) damage analysis to establish the type of damage that could be expected at a certain value of EDP, and (iv) decision analysis in which the consequences of damage are quantified in terms, such as repair cost, that are useful for decision makers. The results of these four analyses phases can then be integrated in order to identify performance measures such as expected annual monetary loss (EAL). The freely available software, PACT (FEMA P58.3), is a tool that assists with the sampling and integration process and it was used in this work.

In order to apply the loss assessment process to the case-study building, hazard data and compatible ground motions were made available from a parallel study by Yeow et al. (2018). Non-linear dynamic analyses were conducted by constructing a non-linear model in Ruaumoko (Carr, 2017). The model, shown earlier in Figure 2, included the main steel EBF system and MRF elements and equivalent diagonal struts and connection springs to model, respectively, the pre-cast concrete panels and their connections. The steel EBF and MRF members were modelled as lumped plasticity Giberson beam elements with bi-linear hysteretic properties assigned to potential plastic hinge regions. The connection springs for the precast panels were also assigned bi-linear hysteretic properties to enable yielding of the steel angle connections. Information provided in a geotechnical report for the foundations of the building was used to set approximate vertical stiffness values for springs placed at the base of the EBFs. Whilst the geotechnical report and foundation typology is such that significant SSI effects were not expected, note that the foundation flexibility modelled in this way was seen to increase the fundamental translational periods of the building by around 3-5%. Fundamental periods of 3.56s and 4.64s in the North-South and East-West directions respectively were obtained from the Ruaumoko model.

Large displacement analyses were conducted with dummy P-delta columns used to emulate the presence of gravity loads not directly applied to the EBF system. A Newmark integration scheme was adopted with an integration time step of 0.001s. A Caughey damping model was adopted with 3% constant damping specified on all modes of vibration. The value of 3% damping is considered a high estimate for the damping, with the understanding that 1%-2% damping is usually assumed for similar buildings in Japan. Sensitivity studies were conducted by running analyses with other values of damping and it was found that the main conclusions of this study were not affected. The results of such analyses and other details of the numerical modelling and analysis approach can be found in Arifin (2017).

For fragility functions, data was taken from the literature where possible and this included the fragility functions for partition walls from Davies et al. (2011) but updated by Yeow et al. (2018b) considering NZ practice, and fragility functions for EBF systems from O'Reilly and Sullivan (2016).

For the precast cladding connections, a simplified finite element model of the connection was made (refer Arifin, 2017) so that the strength, stiffness and deformation characteristics of the anchored steel angle connection could be evaluated. The deformation capacity estimated via this model was then used to identify a median drift capacity for the precast panels' connection failure (i.e. replacement of the connection) damage state, with values for dispersion estimated as 0.40.

Consequence (loss) functions were required for the components too. A number of repair cost functions have been developed in recent years for components commonly used in the New Zealand construction industry and a summary of those found and used in this work is provided in Arifin (2017). For what requires the costs to repair the steel EBF links, local engineers were consulted and the consequence functions shown in Table 1 were established. Interestingly, it was revealed that approximately half of the cost required to repair the steel EBF links was associated with temporarily decommissioning and/or rerouting of non-structural elements in the vicinity of damaged links. The implications of this will be discussed further in the next section.

Table 1. Damage states and repair costs for steel EBF links.

Damage State	**Description**	**Repair Works Required**	**Cost (Each EBF Link)**	
			Item of Repair	**Average Cost (2011 US$)**
DS1	Damage to concrete slab above the link beam.	Replace concrete slab	Achieving access/work on services	16,600
			Floor slabs replacement	3,952
DS2	Web and flange local buckling.	Heat straightening of buckled elements	Damage state 1	20,552
			Heat straightening	6,268
DS3	Initiation of fracture in the link web and link flange.	Replace EBF Link	Damage state 1	20,552
			Remove existing link	5,377
			Install new EBF link	19,762

4.1 Gaining confidence in the loss assessment results

Prior to undertaking cost-benefit analyses, efforts were made to ensure that the assumptions and approximations made in the modelling, analysis and damage assessment process were reasonable. To do this, the response of the building to ground motions recorded near the building site in the four most intense earthquakes (see spectra in Figure 4b) was used together with the fragility and consequence functions to estimate damage and losses over the height of the building.

Figure 6 shows the peak storey drifts and floor accelerations predicted from non-linear dynamic analyses in the February 2011 event. Note that in recognition of the fact that the building had already been subject to the September 2010 event, analyses were run with the February 2011 event tagged onto the end of the September 2010 (and with a period of free oscillation in between). It can be seen that the drifts were greatest over the lower six storeys, around the 12th floor and at the top of the building in the E-W direction. Encouragingly, damage to partition walls and cladding in the actual building was located around the same areas.

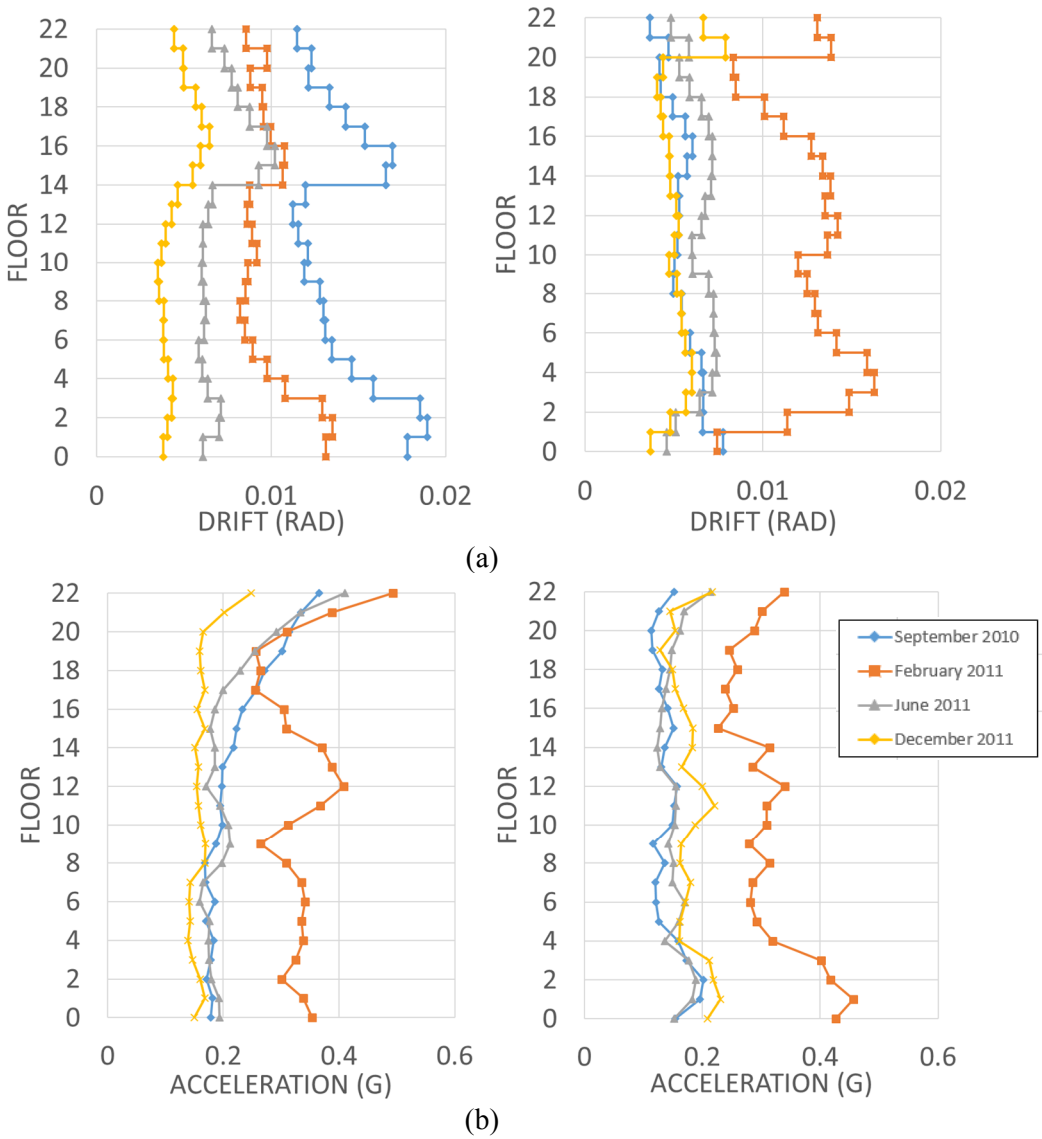


Figure 6. (a) Peak storey drifts (Left: N-S direction; Right: E-W direction) and (b) peak transient floor accelerations (Left: N-S direction; Right: E-W direction) predicted in the Canterbury Earthquake events from non-linear dynamic analyses of the case study building model.

Another important measure was that the maximum residual storey drift identified from post-earthquake inspection of the building was reported (Gardiner, 2012) as 0.3% whereas the residual drift predicted by non-linear dynamic analyses was predicted as 0.28%. This again provided some confidence that the numerical model developed in this work could provide a realistic estimate of the likely behaviour of the case study building.

The predicted losses due to repair for the four most intense earthquake events are shown in Figure 7, in which the contribution of different components to losses is identified. No structural losses appear for the September 2010 event in Figure 7 not because they were not predicted (as large drift demands can be seen in Figure 6) but because inspection of the structure was not carried out until after the February 2011 event and thus the loss values are conditional on inspection. The results align with observations as damage to the EBF links was most intense in the February 2011 event, as reported in (Gardiner, 2012), whereas precast panels and lightweight partitions were expected to suffer damage in all four events.

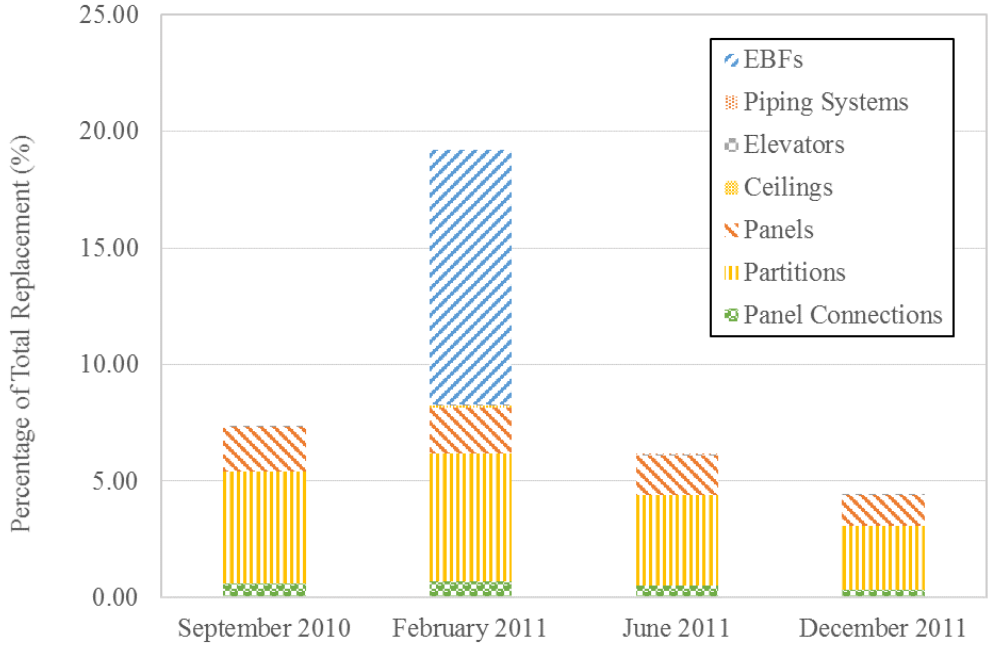


Figure 7. Predicted losses for the case study building, conditional on inspection (the EBF structure was inspected only after the February 2011 event).

4.1 Reparability of Structural Systems

The damage to the EBF links of the type illustrated in Figure 5, required costly repair work, as would be expected in order to replace sections of the steel structure. However, as can be seen from Table 1, an apparent lesson to be learnt from this study is that the decommissioning or rerouting of non-structural elements (and in particular, services) in the vicinity of damaged links reportedly attributed to approximately half the total cost of their repair. Such costs could be significantly reduced or possibly even avoided if, as part of the building's original design, the positioning of non-structural elements took account of the likely need to repair the EBF links.

In light of such observations, increased consideration is being given to the reparability of buildings in New Zealand. This includes development and repair strategies as well as the emergence of systems that have replaceable structural fuses (Baird et al., 2014) that are intended to be damaged and dissipate energy in an intense earthquake. Evidently, however, reparability can be greatly affected by considering how access to the structure will be achieved and how this could require alternative strategies for the positioning of non-structural elements. Talking with local engineers it would appear that one designer's innovative strategy to address this point has been to position access hatches at potential plastic hinge locations so that partitions, services and linings do not need to be disturbed in order to quickly view the state of the structural system post-earthquake.

4.2 Potential Advantages and Disadvantages of Sliding Precast Cladding Details

Good earthquake engineering practice would encourage the detailing of precast cladding connections so as to allow lateral movement of the main structural system without deforming or straining the panels. The detail drawn for the case-study building (Figure 3) did not appear to ensure this and hence the question was posed as to whether retrofit of the cladding connection details could have been an effective strategy for reducing damage, disruption and losses in the building. Figure 8 compares the peak storey drifts and floor accelerations obtained from non-linear dynamic analyses with and without the precast cladding stiffness present in the model. As would be expected, the presence of the precast cladding tends to reduce peak storey drifts but increase peak floor accelerations. Note that the cladding panels were not present on all floors (refer to Arifin, 2017). As such, only floors with panels are significantly affected.

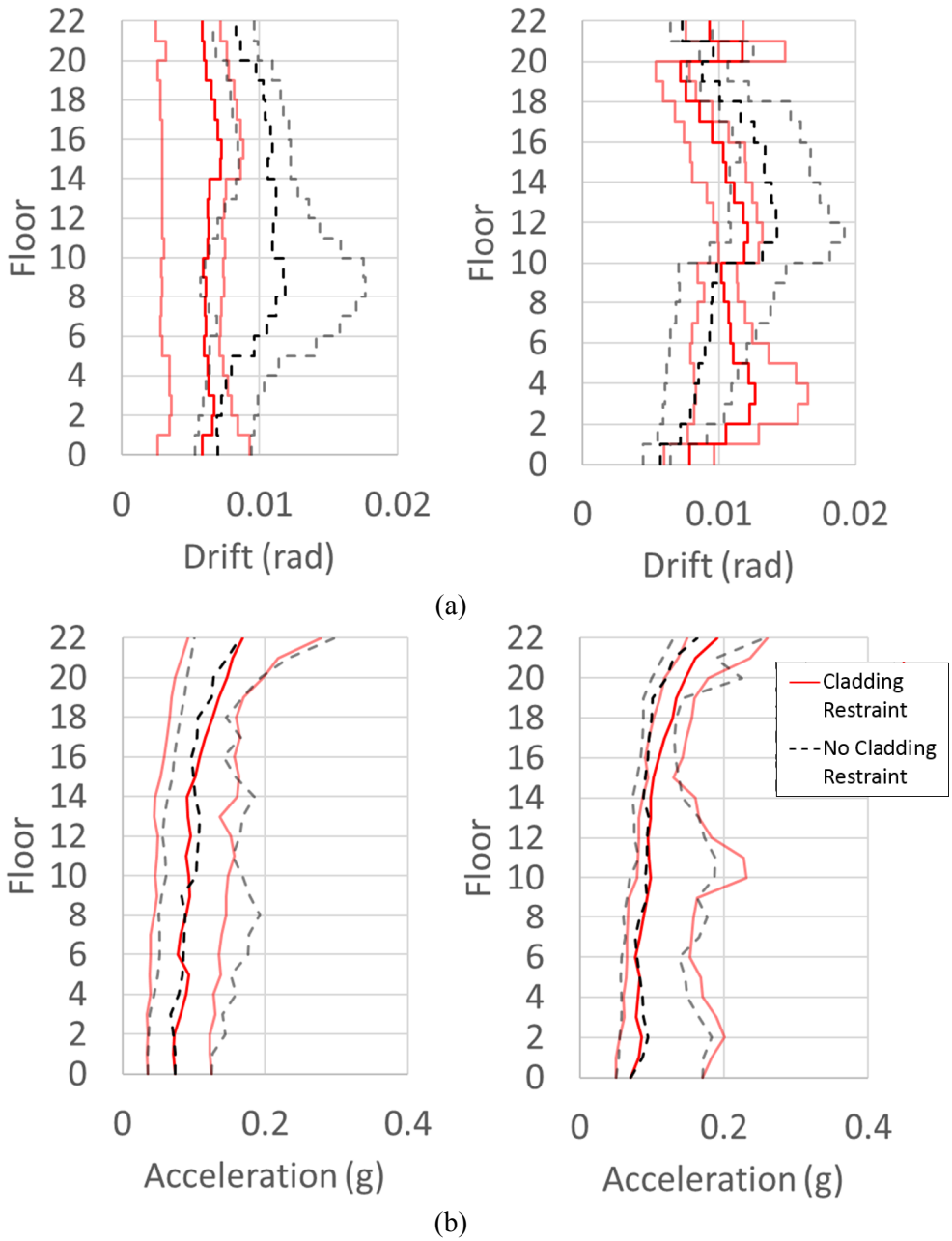


Figure 8. (a) Peak storey drifts (Left: N-S direction; Right: E-W direction) and (b) peak transient floor accelerations (Left: N-S direction; Right: E-W direction) predicted in a 2% in 50-year event from non-linear dynamic analyses of the case study building model with and without precast cladding stiffness present.

Inputting the EDPs obtained from the analyses of a system without cladding restraint into the PACT model, expected annual loss estimates were computed with and without the presence of cladding. From this exercise it emerged that the expected annual loss for the building and overall repair costs would likely have been greater if the cladding had been detailed to provide no restraint, even though the cost of repairing the cladding itself was removed with good detailing. This occurred because even though there was no damage or repair costs for the precast panels themselves, the additional drifts that would be expected over the height of the system would have implied significant increases in damage to the partition walls.

In light of the results obtained, it is concluded that loss estimates can be expected to vary considerably when cladding that provides partial restraint is or isn't modelled both because of changes to drift demands up the height of the building and because certain types of subsequent damage are likely to be cheaper to repair than others. For the case study building, even when cladding panel repair costs were eliminated because of good detailing, overall repair costs increased because the greater drifts lead to more extensive and costly damage to the partitions.

4.3 Impact of aftershocks of cost-effective repair and rehabilitation strategies

Another important observation from Figure 7 (and from discussions with local engineers about building behaviour around Christchurch) is that costly repairs to non-structural partition walls were required not only after the moment magnitude 7.1 earthquake in 2010 but also in multiple aftershocks in the years that followed. The repair costs associated with the aftershock events exceeded those from the main event, emphasizing both the need to consider aftershocks within modern performance-based earthquake engineering assessments and also the opportunity that exists to make more cost-effective repair strategies following damaging earthquakes. For example, if it had been decided following the first event that damaged partition walls should be replaced with low-damage partition systems, the difference in cost compared to simply reinstating like-for-like partitions would have been easily recovered. This observation may be particularly useful for insurers interested in identifying cost-effective repair options.

A further detailed study by Arifin (2017) into the potential impact of aftershocks on the results of loss assessment led to the conclusion that, by neglecting the hazard posed by aftershocks, EAL estimates obtained from application of procedures such as PEER PBEE will generally be underestimated. To demonstrate this, Arifin (2017) first computed the likely losses for the 22-storey case study building in the event that it were subject only to a number of large magnitude events in Japan and New Zealand. Subsequently, the losses were re-computed considering damage from the main shock and aftershocks for the same events. This provided data for Figure 9 that presents the ratio of estimated losses with and without consideration of aftershocks. Figure 9 shows that the extent to which losses are underestimated is likely to be a function of the intensity of the main shock, since it can be seen that as the peak storey drift generated by the main shock increases, so too does the likely loss due to aftershocks. This reflects the notion that larger intensity aftershocks are more likely to follow larger intensity main shocks. Also shown in Figure 9 are the ratios obtained considering plus and minus one standard deviation of the data, illustrating that there is considerable variability in the effect of aftershocks on losses.

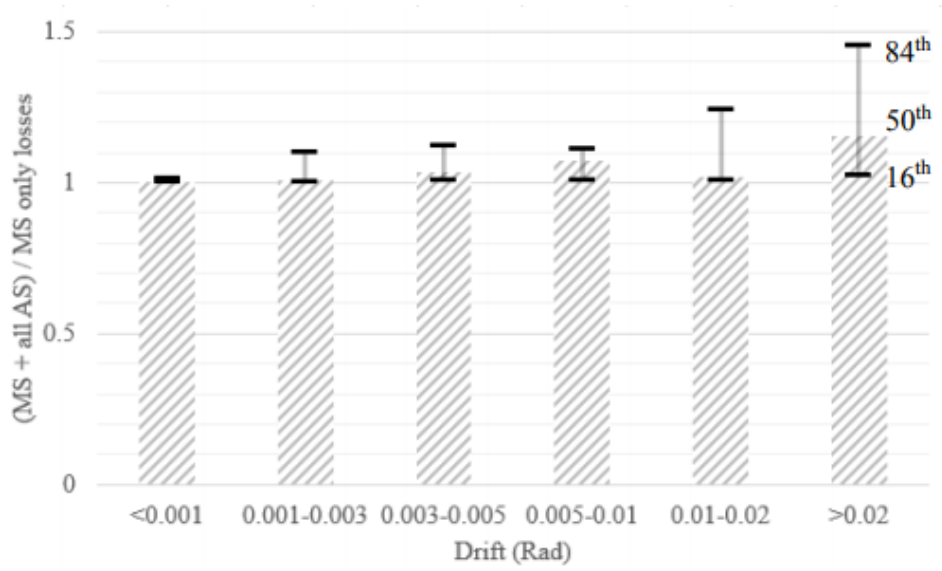


Figure 9. Relationship between the change in losses due to inclusion of aftershocks and the storey drift predicted in the main earthquake event.

5. CONCLUSIONS

Non-structural elements have been seen to perform poorly in earthquakes for decades and therefore much of the non-structural damage observed during the Canterbury earthquakes was to be expected. However, by closely examining the performance of a 22-storey steel framed building in Christchurch subject to various earthquakes over the past seven years, it has been seen that a number of lessons can be learnt regarding the cost-effective consideration of non-structural elements, as listed below.

5.1 Decommissioning and rerouting of non-structural elements can affect losses significantly

The first conclusion that has been made is that the costs associated with the decommissioning or rerouting of non-structural elements in the vicinity of damaged links (or structural elements) can represent a very significant repair cost component. For the case study building considered, providing access represented around 50% the total repair costs. Such costs could be significantly reduced or possibly even avoided if, as part of the building's original design, the positioning of non-structural elements took account of the likely need to repair the EBF links.

5.2 The effect of pre-cast cladding on losses may be hard to estimate without modelling and analysis

A second interesting observation made in this work relates to the role that non-structural pre-cast cladding play on the distribution and type of damage in a building. Loss estimates obtained following the FEMA P-58 framework were seen to vary considerably when cladding is or isn't modelled, both because of changes to drift demands up the height of the building and because certain types of subsequent damage are likely to be easier and cheaper to repair than others.

5.3 Aftershocks can increase losses greatly and should be accounted for in loss assessment studies

A third observation was that costly repairs to non-structural partition walls were required not only after the main earthquake in 2010 but also in multiple aftershocks in the years that followed. The repair costs associated with the aftershock events exceeded those from the main event, emphasizing both the need to consider aftershocks within modern performance-based earthquake engineering assessments and also the opportunity that exists to make more cost-effective repair strategies following damaging earthquakes. Given this, adequate treatment of aftershocks is considered to represent an important challenge for loss assessment in years to come.

6. ACKNOWLEDGMENTS

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