Seismic Sustainability Assessment of Structural Systems: Frame or Wall Structures?

T.Z. Yeow, G.A. MacRae, R.P. Dhakal & B.A. Bradley
University of Canterbury,Christchurch, New Zealand

SUMMARY:
A preliminary case study assessing the seismic sustainability of two reinforced concrete structures, a frame structure and a wall structure, was conducted to determine which structural system is more seismically sustainable. The two structures were designed to the same standards and were assumed to be located in Christchurch, New Zealand. A component-based probabilistic seismic loss assessment, considering direct losses only, was conducted for two ground motion records, regarded to approximately represent a 1 in 500 year earthquake event and a 1 in 2500 year earthquake event, respectively. It is shown that the wall structure results in lower direct losses than the frame structure in the less severe ground motion scenario. However, in the more severe ground motion scenario, the frame structure results in lower direct losses. Hence, this study demonstrates that which structural system has the lower direct losses depends on the ground motion intensity level.

Keywords: Seismic sustainability, component-based probabilistic loss assessment, direct losses

1. INTRODUCTION

Current seismic design standards around the world focus on life safety and collapse prevention in severe earthquake-induced ground motions in addition to immediate occupancy under minor to moderate ground motions. However recent major seismic events, such as the Canterbury earthquakes, have illustrated that losses associated with damage repair, business downtime and injuries have major economic and social impacts on society. Hence, it is increasingly evident that buildings need to be designed to be more sustainable in terms of minimizing overall life-cycle costs (i.e. initial construction cost and losses from hazard exposure during the building's expected lifetime).

Currently, engineers are faced with a wide range of options when designing buildings. Some of these options include selection of materials (i.e. steel, concrete or timber), selection of structural systems (i.e. frame, wall) and design global ductility and strength. However it is not apparent which choice leads to a more seismically sustainable structure. Therefore, there is a need to investigate the effects of these options on the seismic sustainability of structures for future design.

This preliminary case study compares the seismic sustainability of two reinforced concrete (RC) structures representing different structural systems; one frame structure and one wall structure. The two structures will be subjected to two different levels of ground motion intensity. The aim of this case study is to investigate:

1. Which structural system will perform better in terms of direct damage losses for a certain level of ground motion intensity?
2. Do the relative losses of the structures change if acceleration sensitive components are anchored to prevent damage?

2. BACKGROUND ON LOSS ESTIMATION

Loss estimation can be broken down into four distinct steps:

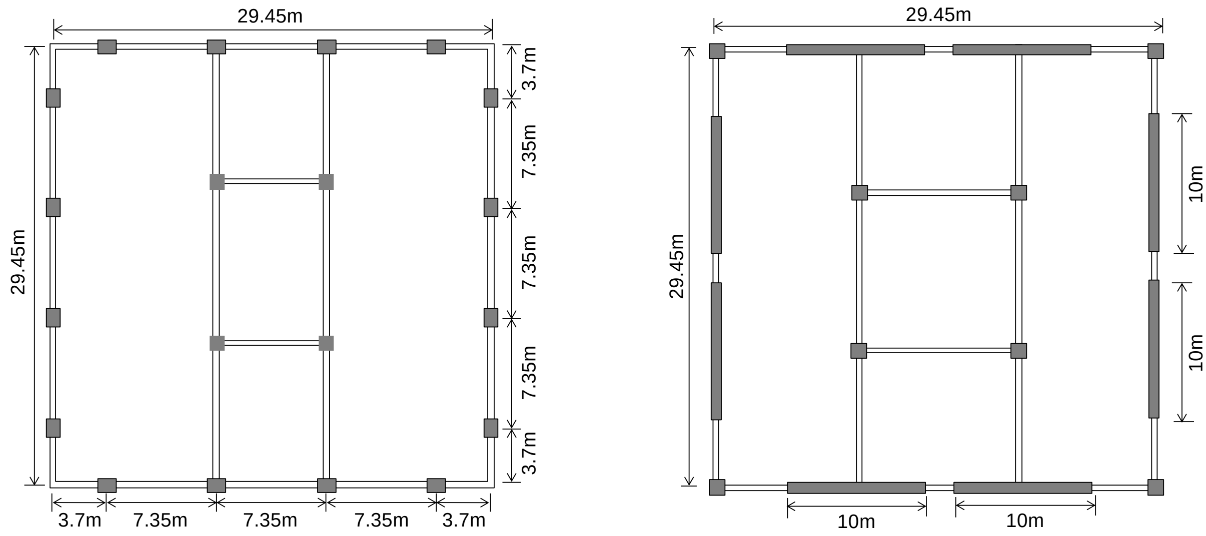
1) What is the likelihood of a certain level of ground motion intensity at the site of interest?
2) Given the ground motion intensity, what is the response of the structure?
3) Given the structural response, what is the extent of damage within the structure?
4) Given the damage, what are the losses resulting from repair cost, downtime (business interruption) or death and injury?

More detail on the loss estimation process exists in literature, such as the ATC-58 guidelines (ATC, 2011). Several computer programs are available to conduct loss estimation. One such program is the Seismic Loss Assessment Tool, SLAT (Bradley, 2011).

3. METHODOLOGY

3.1 Structural Design

The plan elevation of the frame structure and the wall structure considered in this study is shown in Figure 3.1.



(a) RC frame structure **(b)** RC wall structure

Figure 3.1 Plan elevations of RC structures considered in case study

The RC frame structure considered in this study is the Red Book Building (Bull and Brunsdon, 2006), which represents a design example using the New Zealand Concrete Standard, NZS3101 (Standards New Zealand, 2006). The frame structure is 10 stories high, with storey heights of 4m for the ground floor and 3.6m for other floors. The frame was designed according to NZS1170.5 (Standards New Zealand, 2004) and NZS3101 for Class C soils and a global ductility factor of 4.

The RC wall structure used for the comparison was designed to the same standards and the same conditions as the RC frame structure. *P*-delta effects were considered using NZS1170.5. The wall structure was designed with two 10m long walls along each side of the wall structure. All columns present in the wall structure are gravity columns and are assumed to not contribute to the lateral stiffness and strength of the structure.

3.2 Structural Modelling

2-dimensional structural analyses of the two structures were conducted using the computer program Ruaumoko-2D (Carr, 2004). The analysis options selected were:

- Dynamic time-history using Newmark constant average acceleration.
- In-elastic analysis.
- Lumped mass matrix.
- 5% damping coefficient for all modes using Caughy damping.
- P-delta effects included (co-rotational method).

For the frame structure, concrete beam-column members were used to model the beams and the columns. For the wall structure, a Giberson Beam (i.e. lumped plasticity) element was used to model the walls. It was assumed that the axial load on the walls do not change during earthquake excitation. The walls were assumed to only undergo plastic deformation at the base of the wall, similar to other wall models by Mesa (2002) and Calugaru and Panagiotou (2011). All lumped plasticity elements were modelled assuming Takeda hysteretic behaviour.

3.3 Ground Motion Record Selection

The two buildings considered in this case study are assumed to be located in Christchurch, New Zealand. Since many ground motion records were obtained during the recent Canterbury earthquakes in New Zealand (GNS Science, 2012), these ground motion records can be used as earthquake scenarios. The two buildings are assumed to be located near the Christchurch Botanical Gardens (CBGS) so that the ground motions recorded at that site can be readily applied at the base of these buildings. In particular, the N89W horizontal component of the ground motion records was considered.

Two scenario ground motions will be considered for this case study. These were produced by the 4th of September 2010 and 22nd of February 2011 earthquakes. Although these two events occurred within a short timeframe, it was assumed that the structures were fully repaired between the two events and that the effects from the two events are independent from each other. The level of shaking during the 4th of September earthquake was regarded to be similar to a 1 in 500 year event while the 22nd of February was regarded to be similar to a 1 in 2500 year event. However, a single ground motion will not necessarily represent the same level of intensity for two different structures, as shown in Table 3.1.

Table 3.1 Spectral accelerations of selected earthquake scenarios

Structure	**Fundamental Period**	**Earthquake Scenario**	**Spectral Acceleration from Scenario**	**Design Spectral Acceleration (NZS1170.5)**
Frame	1.97 seconds	04/09/2010	0.127g	0.183g (1 in 500 year event)
		22/02/2011	0.324g	0.329g (1 in 2500 year event)
Wall	1.55 seconds	04/09/2010	0.085g	0.220g (1 in 500 year event)
		22/02/2011	0.698g	0.400g (1 in 2500 year event)

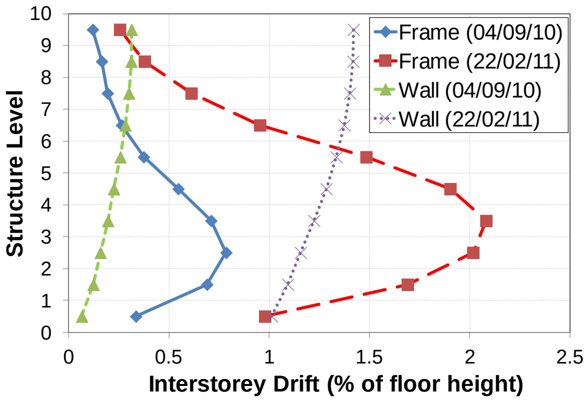
4. STRUCTURAL RESPONSE

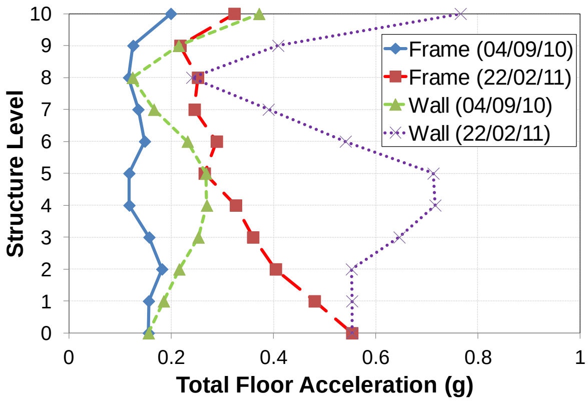
Table 4.1 shows the structure's yield displacement (obtained from pushover analysis), the maximum displacement and the response ductility of the two structures when subjected to the ground motions from the two considered scenarios. Yielding for the wall structure was deemed to occur when the base of the walls yield. However, yielding of the frame structure occurs prior to yielding of the ground columns, as the beams yield first. Therefore, even though the frame has a global response ductility of 1.2 in the 4th September event, yielding of the columns did not occur. Similarly, despite the fact that the frame structure had large global response ductility in the 22nd of February scenario, the plastic rotation at the base of the columns were not severe enough to cause collapse.

Table 4.1 Global Response ductility of frame and wall structures subjected to scenario ground motions

Structure	Yield displacement	Earthquake Scenario	Maximum Displacement	Global Response Ductility
Frame	0.093m	04/09/2010	0.112m	1.2
		22/02/2011	0.425m	4.6
Wall	0.096m	04/09/2010	0.076m	<1
		22/02/2011	0.364m	3.8

Figures 4.1a and 4.1b show respectively the maximum inter-storey drift profiles and the maximum total floor acceleration profiles of the frame and the wall structures subjected to the two ground motion records.





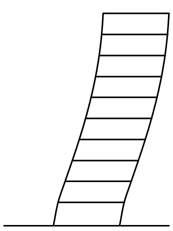
(a) Maximum Inter-storey Drift Profiles **(b)** Maximum Total Floor Acceleration Profiles.

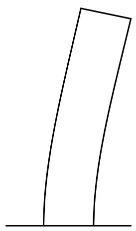
Figure 4.1 Maximum response of frame and wall structures subjected to selected ground motion records for scenario loss assessment

From Figure 4.1a, the following trends can be observed for both the 4th September and the 22nd February scenarios:

- The frame structure's peak maximum inter-storey drift occurred between the second and the third level, and the third and the fourth level of the structure and decreased towards the top of the structure.
- The wall structure's maximum inter-storey drift increased with the height of the structure.
- The peak maximum interstorey drifts were smaller in the wall structure than in the frame structure (1.4% for the wall structure and 2.1% for the frame structure).

The trends can be explained by observing the expected deformation shape of the two structures. Figure 4.2 shows the general deflection shapes of the frame and wall structures. For the frame structure, it can be seen that the interstorey drifts are larger at the bottom half of the structure compared to the top half. For the wall structure however, the interstorey drift is largely governed by the rotation of the wall and therefore increases with height.





(a) Frame Structure **(b)** Wall Structure

Figure 4.2 General deflection shapes for frame and wall structures

From Figure 4.1b, the total floor acceleration for the wall structure is larger than that of the frame structure in both scenarios. This is expected as the wall structure is stiffer than the frame structure, which results to a greater amplification of high-frequency ground motion.

Peak residual inter-storey drifts were obtained from the structural analyses in addition to the inter-storey drift and acceleration profiles. The peak residual inter-storey drifts of the frame structure and the wall structure were all less than 0.15% for both scenarios. This value is within the initial construction tolerances and it means that the building is not likely to require demolition as a result of high residual displacements. In this paper, it is assumed that the structures can be repaired.

5. PROBABILISTIC SEISMIC LOSS ASSESSMENTS

5.1 Structure Inventory and Component Fragility

The inventory considered for the structure is based on Bradley et al. (2009) and is shown in Table 5.1. Initial costs were estimated using Rawlinson & Co. (2011), Aslani (2005) and Bradley (2009). The only difference in the initial cost between the two structures considered is the structural components. Table 5.2 shows the total initial cost of each component type (structural, non-structural drift sensitive or non-structural acceleration sensitive) for each floor. The component fragility used in the loss assessments were based on SLAT's fragility library (Bradley, 2011) and Ji et al (2007).

Table 5.1 Building inventory and initial costs

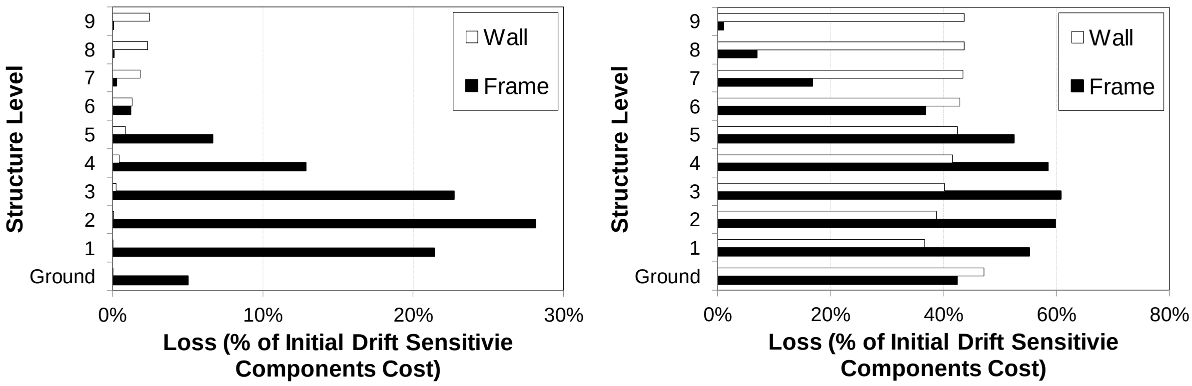
Component	Description	Cost (Frame)	Cost (Wall)
Beam		$1,393,000	$840,000
Column		$666,000	$182,000
Wall		$0	$1,460,000
Column-Slab Connection		$1,420,000	$1,420,000
Total Structural Cost		**$3,480,000**	**$3,900,000**
Partition	721 m^2/floor	$1,290,000	$1,290,000
Exterior Glazing	99 panes/floor	$130,000	$130,000
Drywall Paint	721 m^2/floor	$120,000	$120,000
Generic Drift Sensitive Non-structural component	$100,000/floor	$1,000,000	$1,000,000
Total Non-structural Drift Sensitive Component Cost		**$2,540,000**	**$2,540,000**
Ceiling Systems Suspended acoustical tile	693 tiles/floor	$2,010,000	$2,010,000
Automatic sprinklers	23 sections/floor	$207,000	$207,000
Desktop Computers	$93000/floor	$930,000	$930,000
Servers and network equipment	$200,000/floor	$2,000,000	$2,000,000
Roof mounted equipment	$600,000 on roof	$600,000	$600,000
Conveying - hydraulic elevator	$56,000 each	$112,000	$112,000
Generic acceleration sensitive non-structural component	$100,000/floor	$1,000,000	$1,000,000
Total Non-structural Acceleration Sensitive Component Cost		**$6,860,000**	**$6,860,000**
Total Building Value		**$12,900,000**	**$13,300,000**

Table 5.2 Initial costs of component groups per floor

Floor Level	Total Structure Initial Cost		Total Non-structural Drift Sensitive Cost	Total Non-structural Acceleration Sensitive Cost
	Frame	Wall		
Roof	N/A	N/A	N/A	$712,000
1 – 9	$347,000	$388,000	$254,000	$615,000
Ground Floor	$350,000	$406,000	$254,000	$615,000

5.2 Loss Deaggregated By Floor Level

Figure 5.1 shows the losses from damage to structural and drift-sensitive non-structural components for each level of the structure as a percentage of the initial cost of all drift sensitive components present on the particular level.



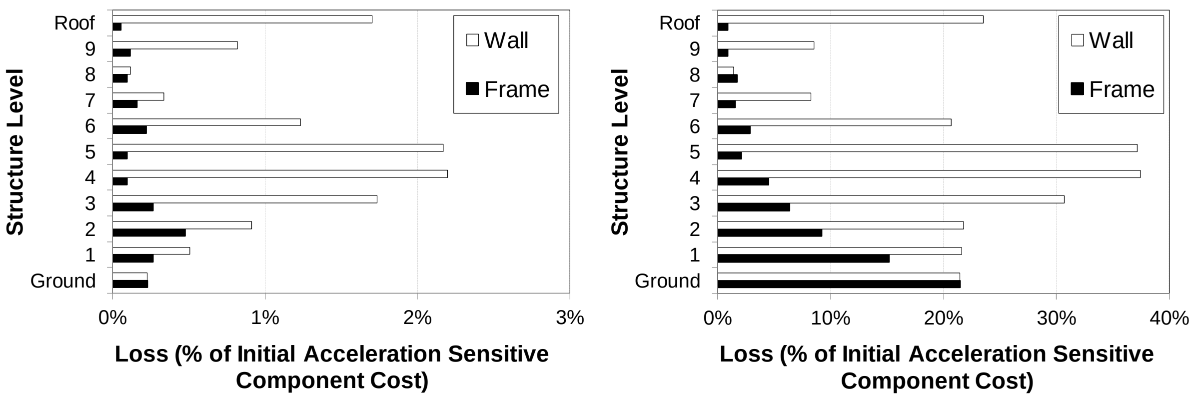
(a) 4th September 2010 scenario **(b)** 22nd February 2011 scenario

Figure 5.1 Distribution of drift induced losses (structure and non-structural) for each storey level

From Figure 5.1a, it can be observed that the frame structure exhibits significantly larger drift losses (a maximum value of 28%) compared to the wall structure (a maximum value of 2.5%) in the 4th September scenario. However, the wall structure has larger drift induced losses on the top four floors. This is expected as the inter-storey drifts of the wall structure exceed that of the frame structure on the upper floors. It is also interesting to note that the wall structure drift losses increases with height of the structure while the frame losses peak at the 2nd floor. These observations can also be correlated to the inter-storey drift profiles of the two different structures in Figure 4.1a.

Similar trends can also be observed in Figure 5.1b for the 22nd February scenario. However, the percentage of drift losses for the wall structure is more prominent (a maximum value of 47%) when compared to the frame structure (a maximum value of 61%). Even more significant is the fact that the wall structure sustains large drift losses throughout the height of the structure, while the drift losses for the frame over the top three floors is small in comparison.

Figure 5.2 shows the non-structural acceleration sensitive component losses for each level of the structure as a percentage of the initial cost of all non-structural acceleration sensitive components present on the particular level. Generally, the wall structure has larger acceleration losses than the frame structure in both scenarios. This is to be expected as the total floor accelerations of the wall structure were generally higher than that of the frame structure as shown in Figure 4.1b.

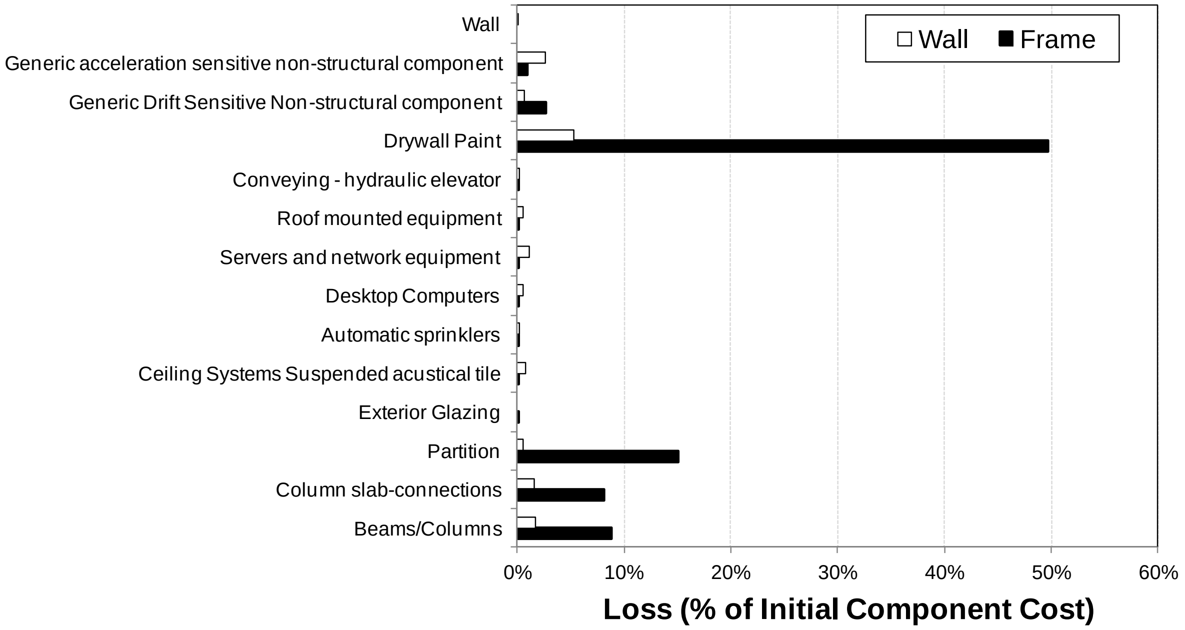


(a) 4th September 2010 scenario **(b)** 22nd February 2011 scenario

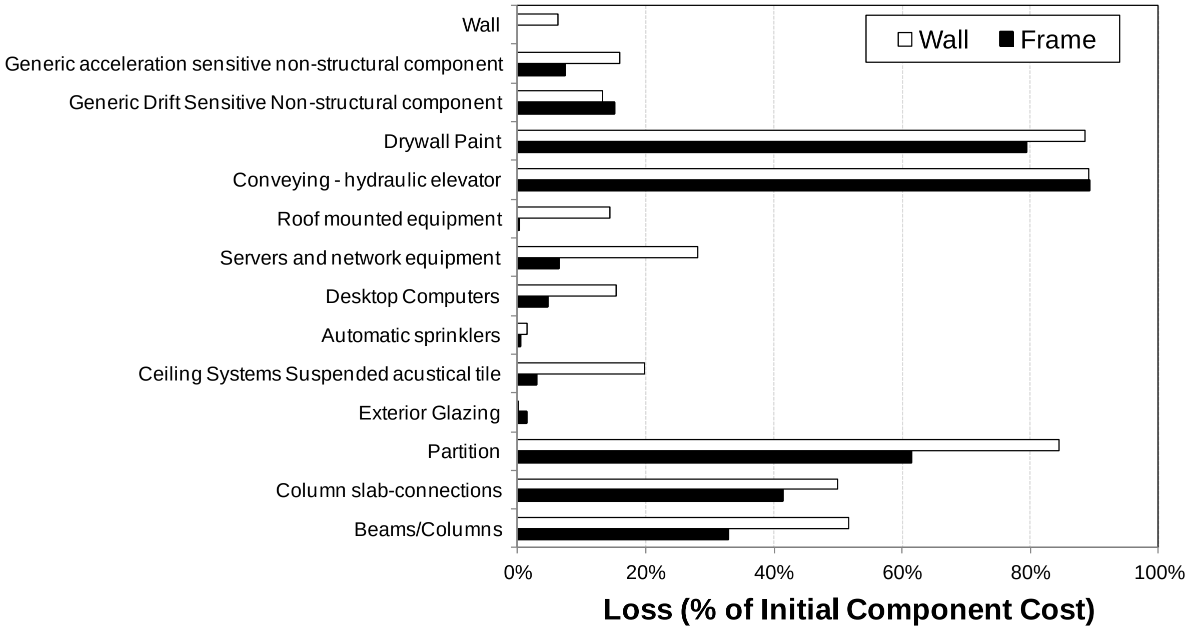
Figure 5.2 Distribution of acceleration induced losses for each storey level

5.3 Loss Deaggregated by Individual Components

Figure 5.3 shows the losses for each individual component (i.e. partitions, sprinklers) as a percentage of the component's total initial cost. As expected for the 4th of September scenario, the losses associated with drift sensitive components is higher in the frame structure than the wall structure, while the opposite is true when considering losses associated with damage to acceleration sensitive components.



(a) 4th of September 2010 scenario



(b) 22nd of February 2011 scenario

Figure 5.3 individual component losses

In the 22nd February scenario, higher losses are observed in almost all components in the wall structure than in the frame structure. This is expected for acceleration sensitive components as the wall had consistently higher total floor accelerations than the frame structure. However, it is interesting that the same trend has occurred for the drift induced losses as well, despite the frame structure exhibiting larger values of interstorey drift. However, this can be explained by closely examining the losses of particular drift sensitive components.

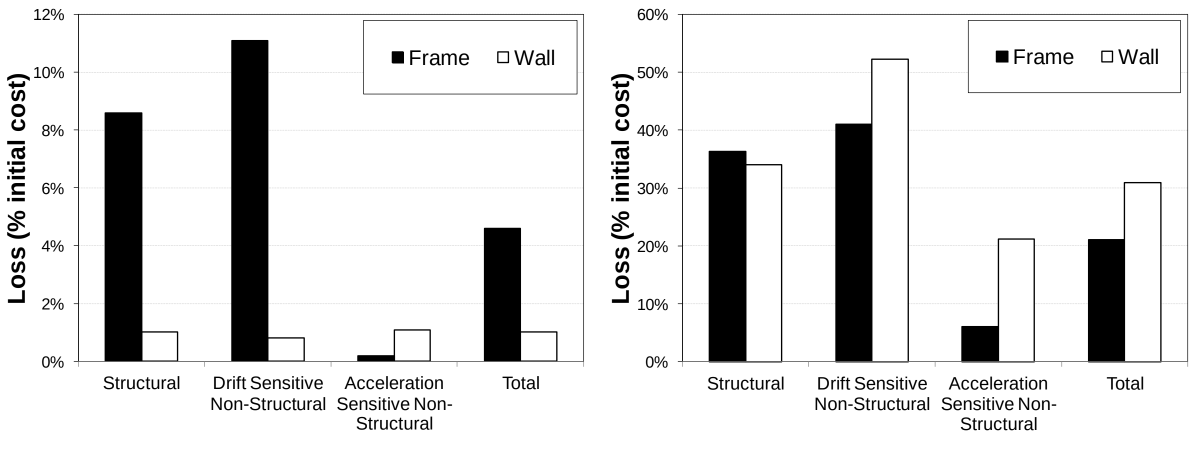
Table 5.3 shows the damage states, the median interstorey drifts which causes the respective damage states, and the median cost of repair for each damage stage for the partitions considered in this study. The median interstorey drift which causes damage state 2 is 0.85%. Observing the 22nd February scenario drift profiles in Figure 4.1a, it can be seen that the wall structure exceeds this interstorey drift on all floors; while the frame structure only exceeds this interstorey drift in the bottom 7 floors. This would mean that, on average, all of the partitions in the wall structure fall into damage state 2 while only those in the bottom 7 floors of the frame structure are likely to do so. Therefore, the losses in the wall structure due to partition damage will be much higher, which is as observed in Figure 5.3b. Note however that the partition loss is not 100% in the 22nd February scenario. This is because the inter-storey drift which causes the onset of damage state 2 is not fixed, but rather it is an uncertain value. Therefore, a small percentage of partitions would not have reached damage state 2. Similar observations can be made with other drift sensitive components. This explains why the wall structure generally had larger drift losses compared to the frame structure.

Table 5.3 Damage states of partitions considered in study

Damage State	**Description**	**Median Interstorey Drift**	**Median Cost of Repair/replacement per m^2 of partition**
1	Visible damage and small crakes in gypsum boards	0.39%	\$29.90
2	Extensive cracking or crushing in gypsum boards	0.85%	\$178.70

5.4 Loss Deaggregated By Component Type

Figure 5.4 shows the losses of each component type (structural, non-structural drift sensitive and non-structural acceleration sensitive) as a percentage of the initial cost of the respective component type. As previously observed, the frame structure obtains larger drift losses in the 4th of September scenario while the wall structure obtains higher acceleration losses. It can be seen from Figure 5.4a that the frame structure has larger total losses (when expressed as a percentage of the total cost of the structure) compared to the wall structure.



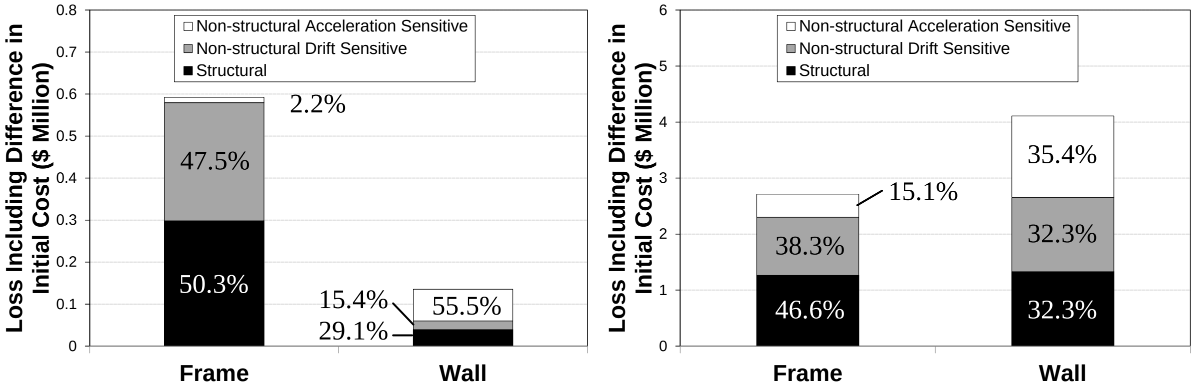
(a) 4th September 2010 scenario **(b)** 22nd February 2011 scenario

Figure 5.4 Loss for each component group (percentage of component group's initial cost)

From Figure 5.4b, the losses due to non-structural damage are far greater for the wall structure than for the frame structure. However, the structural loss of the frame structure is slightly higher. This is because most of the damage to the wall occurs at the base of the wall. This causes a large rotation at the base of the wall, resulting in sizeable inter-storey drifts throughout the height of the structure. Therefore, although there are large drift losses on non-structural elements and non-lateral load resisting elements, the losses from the wall damage are small (9% as shown in Figure 5.3b), resulting in slightly smaller structural losses than for the frame structure. Note that if the damage at the base of the wall is large, it could mean that the structure has to be replaced and hence 100% loss of the building could be incurred. Overall, the wall structure performed worse than the frame structure in the 22nd of February scenario.

5.5 Seismic Sustainability Assessment

The loss assessment findings show that the structure exhibiting the largest scenario loss can change depending on the shaking intensity Figure 5.5 shows the total losses of the frame and wall structure in the two different scenarios.



(a) 4th September 2010 scenario **(b)** 22nd February 2011 scenario

Figure 5.5 Overall cost comparison (difference in initial cost and seismic losses)

It can be observed from Figure 5.5a that the wall structure incurs significantly less losses than the frame structure ($140,000 for the wall structure and $590,000 for the frame structure). As 55.5% of the losses incurred for the wall structure are acceleration induced losses compared to 2.2% for the frame structure, if acceleration sensitive components are anchored to prevent acceleration induced damage, the wall structure would be even more economical than the frame structure (as low as $60,000 for the wall structure compared to $580,000 for the frame structure). Therefore, the wall structure is more economical than the frame structure in this specific scenario.

From Figure 5.5b, it can be seen that in the 22nd of February scenario the wall structure has significantly higher losses ($4,100,000) compared to the frame structure losses ($2,700,000). However, 35.4% of the wall losses are attributed to acceleration incurred losses, compared to only 15.1% for the frame structure. If the acceleration sensitive components were anchored, the wall structure loss and the frame structure loss could be as low as $3,100,000 and $2,300,000 respectively. While the difference in the loss between the two structures is still large, this difference is much less than when the acceleration sensitive components were included. Overall, the frame structure is more economical in this particular scenario.

6.0 CONCLUSIONS

The seismic sustainability in terms of direct damage was assessed for a RC frame structure and a RC wall structure considering 2 ground motion records obtained at the same location near the Botanical Gardens in the 4th of September 2010 and the 22nd of February 2011 Christchurch earthquakes.

It was found that:

1. The wall structure incurred less loss (and hence better seismic sustainability) during the 4th of September earthquake. However, during the more intense shaking from the 22nd of February earthquake, the frame structure incurred the lower loss. This indicates that one type of structural system may not necessarily have the minimum losses at all ground motion intensity levels.
2. Anchoring the acceleration sensitive components did not change which structure had the better seismic performance for the scenarios analyzed. However, it does have an impact on the degree of which one structural system performed better seismically than the other.

ACKNOWLEDGEMENT
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