Steel Innovations Conference 2013
Christchurch, New Zealand
21-22 February 2013

APPLICATION OF SEISMIC LOSS ASSESSMENTS – REINFORCED CONCRETE OR STEEL?

D. Thomson[1], S. Woodhead[1], R.P. Dhakal[2], S.L. Lin[3], T.Z. Yeow[4] and G.A. MacRae[2]

ABSTRACT

The structural performance of buildings and the resulting socioeconomic impacts are important factors to consider for long term sustainability. This paper compares the seismic performance of a reinforced concrete frame and a steel frame building to identify any advantages one material has over the other. The two buildings are located in Christchurch, have identical layouts and are designed to the same standards. Structural analyses were conducted in OpenSees using range of ground motion records representing 1 in 500 year and 1 in 2500 year seismic events. Probabilistic seismic loss assessment is then conducted. As the reinforced concrete frame is stiffer and heavier than the steel frame, it has larger floor accelerations, which results in higher damage to acceleration sensitive non-structural elements and contents. Conversely, the flexible steel frame experiences larger displacements, resulting in more severe damage to the inter-story drift sensitive elements. The reinforced concrete building was found to sustain more damage in seismic events for the content layout considered and had a higher likelihood that the building is beyond repair.

Introduction

Following the Canterbury earthquakes many multi-storey buildings, the majority of which were reinforced concrete (RC), suffered severe and irreparable damage. The smaller steel building stock generally performed well with inelastic deformations lower than expected. The proportion of steel buildings in Christchurch is relatively low compared to other high seismic regions, such as Japan and California. When a city is subjected to significant seismic event the performance of the building stock will have a major effect of the morale and economy of the whole region for many years after the event. When reviewing the performance of a structure it is important to consider the damage and direct costs associated with the repair and the indirect costs including: the downtime until the building is fully operational and the chance of death or injury to occupants. Building owners and insurance companies are particularly interested in the cost of repairing the building in terms of both direct and indirect costs and the probability of the building being beyond repair. This will have consequences for the wider community as the closure of multiple premises throughout the city will have a negative effect on businesses and the work force.

With the impending rebuild of the seismically active city of Christchurch it is important to consider and investigate which types of structures perform better in seismic events. Different types of materials and technologies can be utilized in future buildings to improve the seismic performance and reduce damage, downtime and deaths. This project aims to compare the seismic performance of a typical reinforced concrete moment frame building with a steel moment frame building and give insight into which type of material would be more suitable for seismic resistant buildings in Christchurch.

To assess the seismic performance of the two buildings, pushover and time-history analyses will be performed, the latter of which uses records corresponding to 1 in 500 year (design level earthquake) and 1

[1] Undergraduate student (in 2012), Dept. of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, 8041
[2] Associate Professor, Dept. of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, 8041
[3] Post-Doctoral Fellow, Dept. of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, 8041
[4] PhD Candidate, Dept. of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, 8041

in 2500 year (maximum credible earthquake) seismic event. Probabilistic seismic loss estimation is then conducted to compare expected losses for the two buildings.

Case Study Building Details

The building considered is a typical five storey moment-frame building with the layout shown in Figure 1. The function of the building is a commercial office and the quantity and costs of components considered in the loss estimation are displayed in Table 1. For a fair comparison, the two buildings (with identical layouts) are designed for identical Christchurch conditions using capacity design principles with the same design assumptions. All design load combinations are considered in the design of the frames. The internal frames of the buildings which would be subjected to higher seismic force than the perimeter frames were designed and modelled in 2-D. Typical section sizes are initially assumed and the lateral loads at each storey are calculated using the modal superposition method. The frames are modelled in Mastan2 (Projectdesigns.org) using second order linear elastic analyses to obtain: moment demands in elements, the natural period of the building and deflections at each storey level. After moment redistribution and capacity design, the elements were designed in accordance with NZS3101 and NZS3606 to meet the force and deflection demands. This process was reiterated until suitable element sizes were obtained. The moment redistribution method used is shown in designing the Red Book building (Bull *et al* 2008) is used here as well. For simplicity, columns are assumed to be fixed at the base, and the soil-structure interaction is ignored.

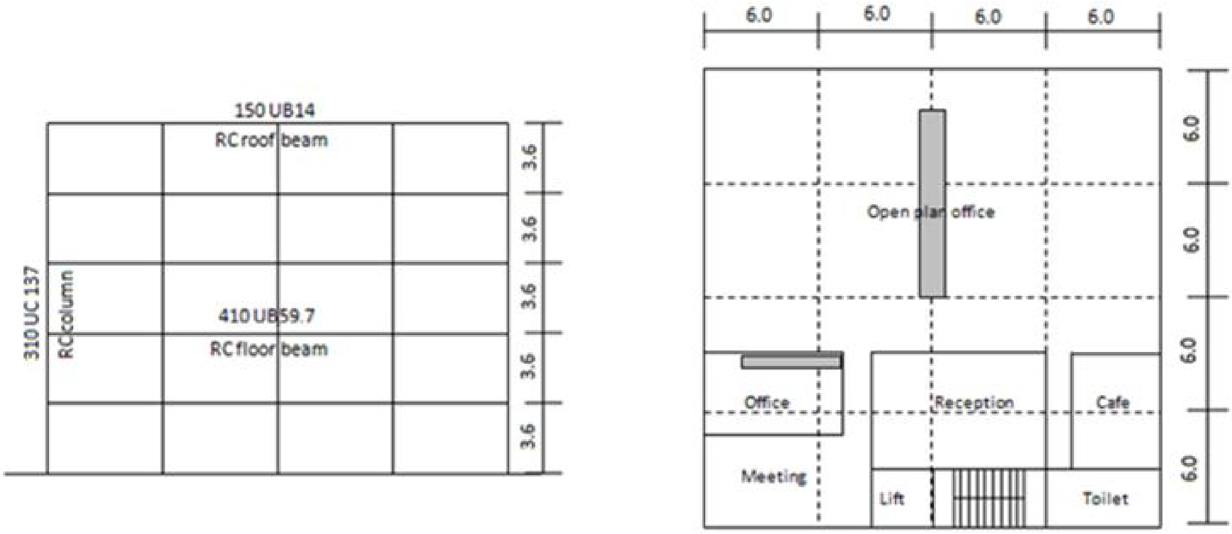

Figure 1. Elevation (left) and floor plan (right) of the case study building

Table 1. Building contents considered in case study

Component	Steel Beam Hinges	Concrete Beam Hinges	Glass Walls	Internal Partitions	Ceiling	Book Shelf	Computers
Quantity	40	40	$346m^2$	$168m^2$	$576m^2$	$50m^2$	30
Unit Cost ($)	5250	5920	514	136	288	25	1750

After considering all load combinations as per NZS1170, the design of the two buildings was found to be governed by the earthquake load combination. The RC frame was designed to the earthquake actions and concrete and standards (NZS1170.5:2004 and NZS3101:2006) for a ductility of 2.5. It has a fundamental natural period of 0.68 seconds. The response is calculated using the first five modes. The design of the RC sections is governed by moment capacity and checked for deflection. The steel frame is also designed for a ductility of 2.5 and has a fundamental period of 1.23 seconds. The section sizes are governed by the inter-storey drift limit of 2.5% and checked for a combination of moment and axial force capacity. The cross section details of the concrete members are included in the appendix and the steel sections are listed in Figure 1.

Methodology

Modelling

In this study, OpenSees (Open System for Earthquake Engineering Simulation, UC Berkeley (2006)) – a finite element analysis software program is used to evaluate the behaviour of the two frames subjected to static and dynamic loadings.

As mentioned previously, a 2D frame was developed to evaluate the seismic behaviour between the two building systems. For both building systems, the elements were modelled using fibre sections which are defined by uniaxial material stress-strain relationships. Microscopic modelling is advantageous, particularly for reinforced concrete as it is a composite material and difficult to model accurately using a section constitutive relationship. The path-dependent cyclic stress-strain relationships allow the effects of material nonlinearities and cyclic loading to be modelled accurately which results in realistic material degradation and hysteretic energy dissipation.

The 6m long beams and 3.6m high columns are modelled with nonlinear beam-column elements. Five Gauss-Lobatto integration points were placed along the length of the elements. The integration points distribute the spread of plasticity along the beam and allow plastic hinges to form at any integration point. When modelling the element geometric nonlinearity, the p-delta transformation is used for the columns. Nodes were located at the centreline of each beam column joint with the joints being modelled by rigid end blocks. The ground floor nodes are modelled as fixed. The nodes on each floor were modelled such that all the nodes on one floor had the same x direction displacement. This constraint was used as it was assumed that the axial stiffness of the beams was large relative to the flexural and shear stiffness of the beam and column elements. The masses of each floor are lumped at each node based on the tributary area.

The uniaxial material model used for fibre sections in the steel frame was Filippou (2012) and are displayed below in figure 2. The concrete sections required three different uniaxial materials for the fibre sections: longitudinal reinforcing where the Mohle et al. (2010) model is used, and core and cover concrete which are modelled by Filippou (2010). The material properties for these materials are displayed in Figure 2.

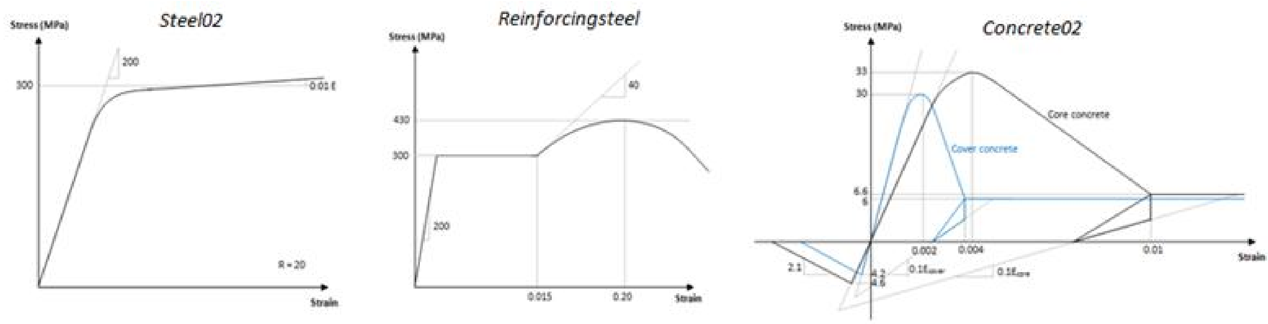

Figure 2. Material properties used for steel members (left), steel reinforcing (middle) and concrete (right)

Pushover Analysis

The static analysis performed on the building was a force based pushover test to obtain the displacement shape, and then a displacement based pushover test was performed. The lateral loads obtained from the modal superposition method are applied to the corresponding node in the same incremental fashion as the gravity loads. The model uses the default to compute the first five vibration modes on the frame. The lateral loads are increased until the building yields and the elastic displacement shape is recorded.

The displacement based pushover test uses the displaced shape by applying a linear load pattern proportional to displacement; and the top node, (i.e. the control node) and the lateral lower stories are proportioned accordingly. The shape updates at each step as the frame becomes nonlinear. The analysis is carried out in small displacement increments to accurately capture the nonlinear behaviour of the buildings, and the ultimate global drift was limited to 2.5% which is similar to the design displacement of the steel building, but is greater that the concrete building design displacement.

Time History Analysis

Two earthquake return periods were used in the time history analysis; 1 in 500 year event (design earthquake) and 1 in 2500 year event (maximum credible earthquake). To give the same scaled hazard level for the two buildings the twenty ground motion records (Table 3, Appendices) were picked using the following criteria:

- Spectral acceleration based on NZS1170.5, considering a fundamental period of 0.4 of the concrete frame and 1.3 of the steel frame
- Moment magnitude between 5 and 8
- Strike slip or reverse fault type

- Rupture distance between 20km and 150km
- Shear wave velocity between 180m/s and 400m/s

These criteria represent the possible size and type of earthquakes that a building in Christchurch would experience, the distance to the Greendale and Alpine faults, class D soil and the difference in periods of the frames. The ground motions are then scaled to get the required hazard levels for the two earthquake return periods.

Figure 3 shows the scaled 500 year ground motions, between 0.4 of the concrete frame and 1.3 of the steel frame, have an almost identical acceleration spectrum to the design code. This shows that the scaled ground motions will have the same hazard level for both frames and the 500 year events will test the frames against a design level earthquake.

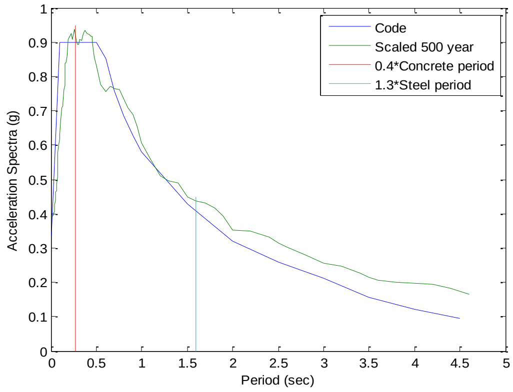

Figure 3. Comparison of code spectrum and selected geometric mean of ground motion records

The scaled ground motions are then applied at the base of the model with 5% Rayleigh damping and it is free to deform under the acceleration. The time history analysis (THA) uses the same analysis methodology and tools as the static analysis. For a representative selected ground motion record the frame's responses were recorded.

To obtain a representative building response for the 500 and 2500 year events, the analysis of 20 different ground motion records was performed. The absolute peak accelerations and inter-storey drifts were recorded at each floor. It is established that structural responses to a suite of scaled ground motions closely fit a lognormal distribution (Mander *et al,* 2007) so the lognormal mean and dispersion of the two parameters was calculated, allowing the range to be plotted and used for probabilistic seismic loss estimation. For more generic applications the absolute acceleration was normalised with ground acceleration and the inter-storey drift was normalised with roof displacement.

Probabilistic Seismic Loss Estimation

The probabilistic loss due to damage of structural and non-structural components and contents was calculated using fragility functions from the SLAT user manual (Bradley, 2009) for a selection of key components. The components include: ductile RC beams, welded-steel moment beams, exterior skin-glass curtainwall, interior GIB walls on wood studs, suspended acoustical tile ceiling systems and typical contents. For a fair comparison the two buildings have identical inventories apart from the structural elements. From the THA results, the accelerations and IDRs at each floor are recorded and lognormal curves are produced and correlated with the lognormal curves from the fragility functions. The lognormal curve of the THA results are split into ten equal segments and the probability of the component falling into a particular damage state were calculated for each segment. Using the median cost to repair each of the damage state, the total damage for each component is estimated and summed to give the total loss for the building allowing comparisons between the different sized events and building materials to be made.

Comparison of Structural Performance

Pushover Analysis

Plot of the base shear vs. roof displacement for the two frames are shown below (Figure 4). The concrete frame starts to yield at approximately 2100kN and attains a maximum strength of 3100kN, which is well above the design strength of 1555kN. There is a combination of reasons for this, including: overstrength factors, strain hardening, extra strength and ductility from the core concrete and axial compression increasing element strength. The steel frame starts to yield at approximately 800kN and the maximum force is slightly higher than 1200kN, which is expectedly greater than the design strength of 735kN. The main reason for this reserve strength is because the design was displacement-based and therefore the section sizes were larger than they would have been if just designed for force.

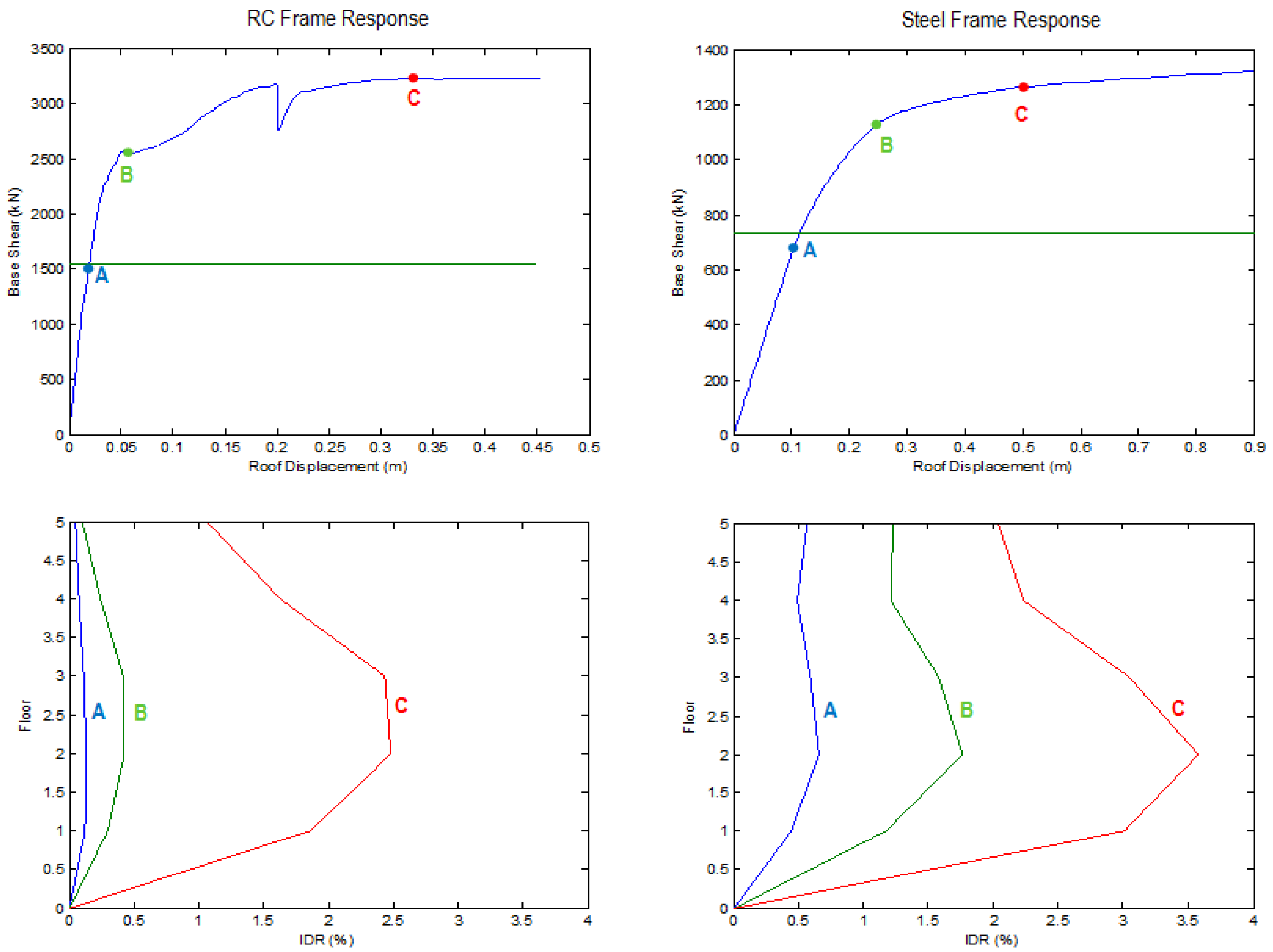

Figure 4. Plots comparing the base-shear vs roof displacement (top) and the interstorey drift (bottom) for concrete (left) and steel (right).

Both frames form the weak beam – strong column mechanism, which was expected as capacity design was used. After the pushover displacement exceeds the design displacement, hinges start to form at the top of the third and fourth floor columns in some grids as shear forces reverse to maintain static equilibrium and cause moments to reverse (shown in Figure 5). This does not from a not a soft storey mechanism as the majority of the displacement occurs over three floors. The drop in the RC frame pushover coincides with the initial formation of column hinges and causes a localised compression steel reinforcing failure. The moment reversal that causes column hinging only occurs after the design displacement has been exceeded, and is not anticipated in design.

The equal DOF constraint which forces nodes on one floor to have the same x direction displacement was removed after pushover analysis. This was due to the results from the concrete frame, where elements have a non-central neutral axis, so as nonlinear behaviour develops in the composite elements, the tension steel stretches, where as the compression steel compresses only slightly. This results in axial elongation at the centre of the elements, which if constrained leads to a high axial compression force, which increases the effective bending strength of the beams (Peng *et al*, 2011). The elongation was approximately 7mm (1% of beam depth) at each hinge along the first floor and resulted in axial force in the order of 2000kN. This increased the moment capacity of the beams beyond the capacity of the columns; thereby forcing column hinges to form, leading to a soft storey mechanism. As a result of these findings, all nodes should be left free in analysis as the equal lateral DOF assumption is not valid for nonlinear composite members. The phenomena could be observed in reality when a floor diaphragm that is connected to the beams restricts the axial deformation, thus induces a large axial force on the composite beam elements.

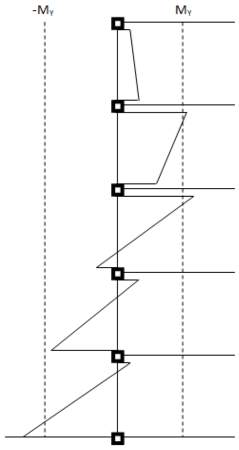

Figure 5. Moment profile showing column yielding on upper floors

Time History Analysis of One Ground Motion

The 1233 FN (Chi-Chi, Taiwan) ground motion record was used to compare the performance of the two frames for a 500 year and 2500 year return period. For the steel frame and 500 year event the columns only yielded at the bottom of the first floor and only the beams in floors one, two and three yielded. For the 2500 year event the tops of the second and third floor columns also yielded. As this is not a soft storey failure this does not cause a collapse and is expected from the larger 2500 year return period. The concrete frame yielded at the base of the first column and the beams on the first, second, third and fourth floors for both the 500 year events. For the 2500 year event hinges form at the tops of the third and fourth floor columns, but does not cause a soft storey mechanism. For both frames this behaviour is due to moment reversal which is not anticipated in the design as was only event for the maximum credible earthquake. The hinge patterns observed in THA are similar to those in static analysis where the tops of columns form hinges after the drift under design loading is exceeded.

The moment curvature plots of the beams showed that both material types exhibited typical hysteretic energy dissipation and confirmed the accuracy of the non-linear response. The design ductility was compared to the actual ductility via the base shear vs. displacement plots (shown in Figures 6 and 7) where the ratio of ultimate to yield displacement was close to 2.5 for both frames. The base shear vs. roof displacement of the concrete frame resembles the shape for the pushover curve envelope. The steel frame non-linear behaviour also replicates the push over curve, but not as closely as the RC frame. This is because the flexible steel frame experiences a whiplash effect and is affected more by higher order modes than the rigid RC frame.

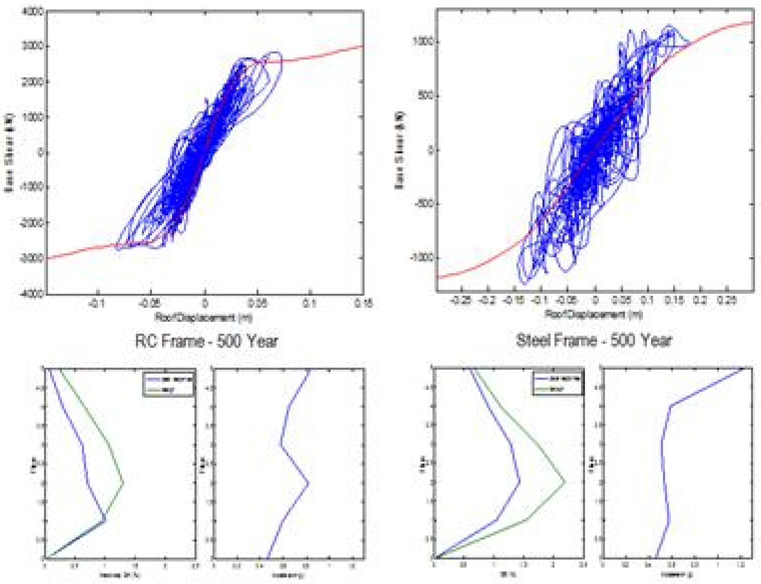

Figure 6. Performance of reinforced concrete frame structure (left) and steel frame structure (right) for 1 in 500 year event

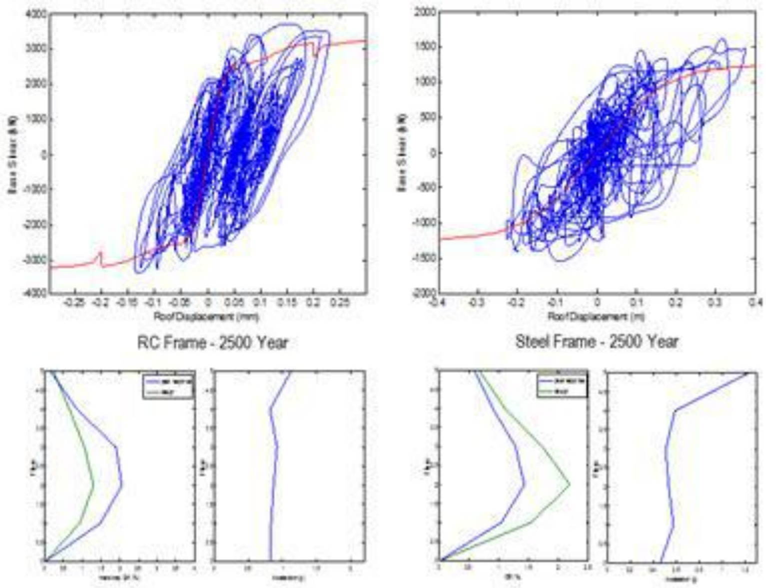

Figure 7. Performance of reinforced concrete frame structure (left) and steel frame structure (right) for 1 in 2500 year event

The inter-storey drift ratios (IDR) were calculated at different times during the time history analysis and the peak was compared against the design inter storey drift ratios. For the steel frame and 500 year return period the IDRs showed a similar shape to the design IDR. The max IDR is smaller than the design IDR which is expected. The shapes of some of the IDRs for the 2500 year return period are different than the design and 500 year IDRs. This happens when the plastic hinges form in the third and third floor columns. The IDR profile of the concrete frame for the 500 year event is smaller than the design IDR, however the maximum inter-storey drift occurs on the first floor instead of the second. The 2500 year event IDR profile expectedly exceeds the design profile and is a different shape due to reasons described above and the hinges that form in the columns. The peak floor accelerations for the two frames reveal that in general, the RC frame experiences higher accelerations than the steel frame. The very high acceleration experienced in the roof of the steel frame was not anticipated as it was expected the rigid RC frame would experience higher accelerations.

The nonlinear behaviour of the frames resulted in residual deformation as there were no restoring forces. The RC frames experienced higher residual displacements with a peak of 10.3mm for the 500 year event and 56.4mm for the 2500 year event compared with 6.5mm and 39.3mm for the steel frame. The higher residual displacement occurs in the RC frame as the sections experience more nonlinear behaviour, reducing the ability of the frame to self restore. The residual shape of the buildings is highly dependent on the extent of the nonlinear behaviour and the nature of the ground motion as the peak residual displacements occur on the second floor for the 500 year event and on the top floor in 2500 year event. The RC beams experience beam elongation which is specific to composite materials and does not occur in the steel frame. The maximum residual beam elongation was 4.3mm in the second floor beams.

Time History Analysis for a Range of Ground Motions

The steel frames IDRs are larger than the concrete frames for both return periods. This shows that the steel frame is more flexible than the concrete frame. Normalizing the floor displacements to the roof displacement shows that the concrete frame has two distinct IDR shapes, one which resembles the design shape and the other which was observed in the detailed THA for reasons previously described. The normalized IDRs show a standard shape for the steel frame.

The concrete frame experiences larger peak floor accelerations than the steel frame. This is expected because the acceleration increases for stiffer buildings with smaller fundamental periods. The roof of the steel frame has extremely large peak accelerations, which was not anticipated. The roof acceleration is over twice as big as any other floors in the steel frame and larger than any of the concrete frame peak accelerations. This behaviour could relate to the small ratio of mass in the roof compared to the other floors and the smaller elements used in the roof beams lead to a reduced stiffness. This phenomenon was also observed in the concrete frame, but to a lesser extent as the roof mass ratio is larger than ratio in the steel frame.

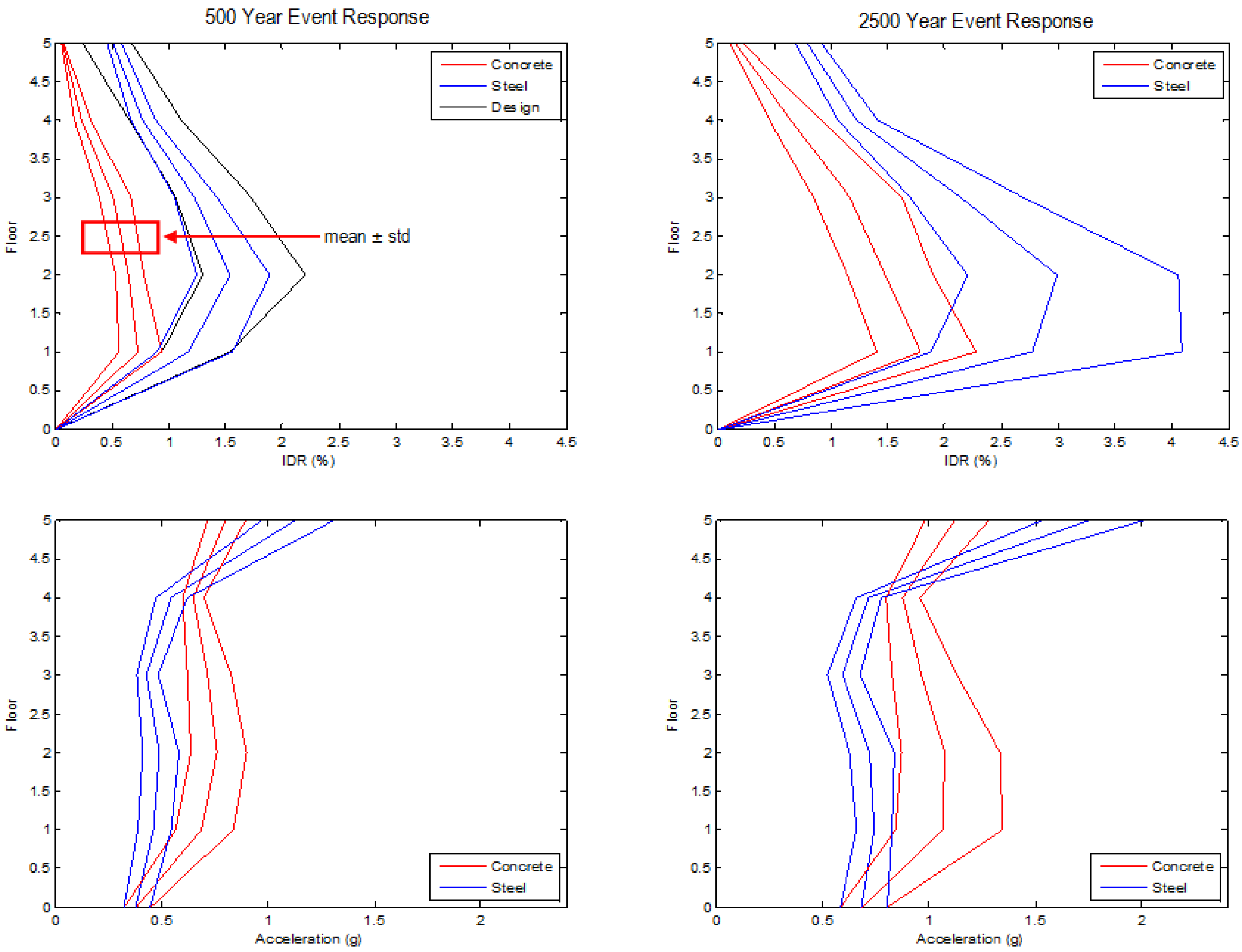

Figure 8. Comparison of interstorey drift and total acceleration response of each floor between the reinforced concrete and steel frame structure for 1 in 500 year event (left) and 1 in 2500 year event (right)

Probabilistic Seismic Loss Estimation

The IDR and accelerations resulting from the time history analysis were used to calculate and compare the expected loss incurred by the two frames for both magnitudes of ground motions. It was found the steel frame causes less damage to the computers and book shelves because it has lower floor accelerations for both return periods. The ceilings also receive less damage in the steel frame even though there is more damage for the 4th floor ceiling because of the larger roof acceleration. This is because the extra damage from the 4th floor ceiling does not make up the difference from the other floors. The internal partitions and external glass curtain walls receive less damage in the concrete frame. This is because the IDR at every floor are smaller for the concrete frame. Even though the RC frame has smaller IDRs the concrete beam hinges are damaged more than the steel beam hinges as the concrete hinges are more fragile than the steel hinges. The concrete and steel beam hinges were similar in cost so the difference in loss is only because of the amount of damage. Because the hinges and the ceilings are the most expensive items, and these items are damaged more in the concrete frame, the concrete building has larger loss for both return periods. A summary of the probabilistic seismic loss estimation results is shown below in Table 2.

For the 500 year earthquake the total loss for the concrete frame is approximately twice as large as the steel frame and for 2500 year earthquake the difference is 50%. The difference in cost is smaller for the maximum credible event, because more of the ISRs and floor accelerations are in the largest damage state, giving similar costs for some components. The RC frame had a higher chance that the building would be damaged beyond repair as the brittle superstructure is more likely to suffer irreparable damage. The larger repair costs associated with the concrete building would also take longer to repair. The longer downtime would result in further economic detriments to the tenants and building owner due to the indirect costs relating to the loss of amenity. This would negatively affect the other businesses and families in the surrounding community that are reliant on the services and income provided by the building.

Table 2 Comparison of losses for both the 1 in 500 year event and 1 in 2500 year event

Component		500 Year Event				2500 Year Event			
		Concrete		Steel		Concrete		Steel	
		Total Damage Ratio	Loss ($)	Total Damage Ratio	Loss ($)	Total Damage Ratio	Loss ($)	Total Damage Ratio	Loss ($)
Strucutural (Drift sensitive)	Concrete Beam Hinges	0.1445	171062	NA	NA	0.4024	476454	NA	NA
	Steel Beam Hinges	NA	NA	0.0029	3091	NA	NA	0.0961	100941
Drift sensitive	Glass Walls	0.0000	6	0.0064	5652	0.0208	18452	0.2035	180975
	Internal Partitions	0.2011	22969	0.3495	39930	0.3283	37507	0.5212	59537
Acceleration Sensitive	Ceiling	0.3140	260447	0.2174	180284	0.5857	485766	0.4123	341953
	Book Shelf	0.8508	5317	0.6491	4057	0.9860	6162	0.9492	5932
	Computers	0.1613	42333	0.0627	16451	0.3409	89488	0.1854	48671
Total Cost ($)			502135		249464		1113830		738009
Percentage Cost (%)			15.3		7.9		33.9		23.4
Building Beyond Repair (%)			0.5		0.1		17.3		13.1
Total Cost of Components ($)			**3285650**		**3151650**		**3285650**		**3151650**
Estimated Building Cost ($)			**4968000**		**4968000**		**4968000**		**4968000**

Further Investigation

The results of this study are very specific to one type of building (five storey moment-resisting frame) with one type of use (commercial office) and is designed for one location (Christchurch conditions with a hazard factor of 0.3 and soft soil). Further studies will be conducted investigating other parameters such as number of stories, bay width, floor height, irregularities and others.

Conclusion

Two identical (five storey, four by four bays) buildings were designed for the same seismic conditions using RC and steel moment-resisting frame and analysed under static and dynamic loading to compare their performances and loss in design level and maximum credible earthquakes. The RC frame was heavy and rigid and attracts a larger acceleration. By comparison, the steel frame was light and flexible but experiences higher displacements. In the pushover test and time history analysis, both frames performed as designed, forming a weak beam–strong column mechanism. However once the design displacement was exceeded, moment reversal started to occur and plastic hinges started to form in column tops and was most evident in the concrete frame. There was some residual deformation in both frames, but it was higher for the RC frame in both ground motions. The flexible steel frame experienced much larger IDR compared to the concrete frame for both scales of earthquakes, which was expected as the steel frame is governed by displacement limits. As predicted the stiff concrete frame attracts a higher acceleration compared with the steel frame apart from the roof, where the steel frame building experienced very large accelerations due to the small ratio of mass in the roof compared with the other floors.

The large acceleration in the RC building, results in higher damage to acceleration sensitive non-structural elements and contents. Conversely, the steel building experiences larger inter-story drift compared with the RC frame. Despite this, the drift sensitive hinge damage is less when compared to the RC building because the flexible behaviour of the steel connections can accommodate a higher drift than the brittle RC connections. Non-structural drift sensitive components sustained more damage in the steel building, but they had a relatively low cost compared to the rest of the structure. The RC building was found to have a higher likelihood that the building was damaged beyond repair for both events. As RC building sustains more damage it would take longer to repair and as a consequence there would be larger indirect costs incurred by the wider community.

For a sustainable rebuild of Christchurch, structural engineers should give more consideration into using steel to build seismic resistant structures. This is would increase the proportion of steel buildings in the region, making the building stock more similar to other high seismic regions, such as Japan and California. From the higher direct and indirect cost associated with the seismic performance of the concrete frame, it is recommended that if a medium-rise moment-resisting commercial office building is to be constructed in the Christchurch, steel should be the preferred material over reinforced concrete.

Appendix

Figure A shows the cross section of the reinforced concrete members.

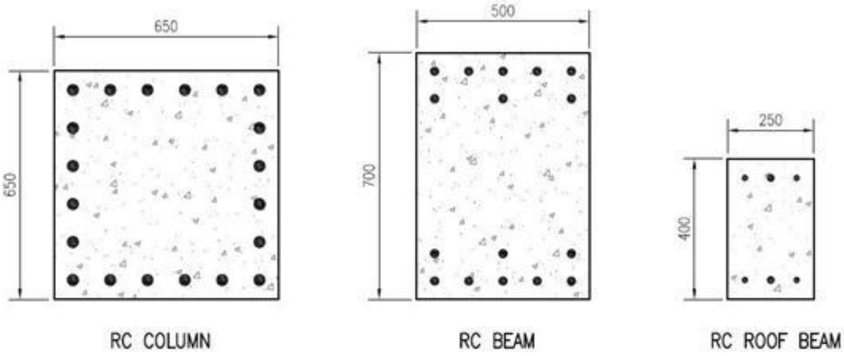

Figure A Cross section of reinforced concrete members

References

Bradley, B. A. (2009). User Manual for SLAT: Siesmic Loss Assessment Tool version 1.14. Christchurch. Universtity of Canterbury.

Bull D K, B. D. (2008). *Examples of concrete structural design to New Zealand Standard 3101.*

Filippou, F. (2010, september 16th). Retrieved from http://opensees.berkeley.edu/wiki/index.php/Concrete02_Material_--_Linear_Tension_Softening

Filippou, F. (2012, September 26th). Retrieved from http://opensees.berkeley.edu/wiki/index.php/Steel02_Material_--_Giuffr%C3%A9-Menegotto-Pinto_Model_with_Isotropic_Strain_Hardening

MacRae G, C. C. (2011). *Steel building damage from the Christchurch earthquake series of 2010 and 2011.* Christchurch.

Mander, J. B., Dhakal, R. P., Mashiko, N., & Solberg, K. M. (2007). *Incremental dynamic analysis applied to seismic financial risk assessment of bridges.* Christchurch.

Mohle, J., & Kunnath, S. (2010, June 16th). Retrieved from http://opensees.berkeley.edu/wiki/index.php/Reinforcing_Steel_Material

Peng, B. H., Dhakal, R. P., Fenwick, R. C., Carr, A. J., & Bull, D. K. (2011). Elongation of Plastic Hinges in Ductile RC Members. *Advanced Concrete Technology*.

StandardsNewZealand. (2004). *Structural design actions (NZS 3101:2004).* Wellington.

StandardsNewZealand. (2006). *Concrete structures standard (NZS 3101:2006).* Wellington.

StandardsNewZealand. (2009). *Steel structures standard (NZS 3404:2006).* Wellington.