Seismic Performance of a Six Storey Reinforced Masonry Building during the Christchurch Earthquake

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SUMMARY:
During the Christchurch earthquake of February 2011, several midrise buildings of Reinforced Concrete Masonry (RCM) construction achieved performance levels in the range of life safety to near collapse levels. These buildings were subjected to seismic demands higher than the building code requirements of the time and higher than the current New Zealand Loadings Standard (NZS-1170.5:2004). Structural damage to these buildings has been documented and is currently being studied to establish lessons to be learned from their performance and how to incorporate these lessons into future RCM design and construction practices. This paper presents a case study of a six story RCM building deemed to have reached the near collapse performance level. The RCM walls on the 2nd floor failed due to toe crushing reducing the building's lateral resistance in the east-west direction. A nonlinear dynamic analysis on a 3D model was conducted to simulate the development of the governing failure mechanism. Preliminary analysis results show that the damaged walls were initially under large compression forces from gravity loads which caused increase in their lateral strength and reduced their ductility.   After toe crushing failure developed, axial instability of the model was prevented by a redistribution of gravity loads.

Keywords: Reinforced Masonry, nonlinear analysis

1. INTRODUCTION

This paper presents the study of the seismic perfomance of a six story (Reinforced Concrete Masonry) RCM building during the Christchurch earthquake of February 2011. Presented herein is a summary of the documented structural damage found after the event and of the building's structural properties. A 3D analytical model of the building was developed to simulate the time history response and governing failure mechanism during this seismic event.   The authors present preliminary observations on the performance of the RCM walls loading in-plane based on the observed damage and on the results of the analytical model.

2. BUILDING DESCRIPTION

The Rolleston Court Flats was a six storey apartment building located approximately 400 metres north-east away from the Christchurch Hospital. The overall floor dimensions were approximately 37 m spanning north to south and 14.4 m east to west. Shown in Fig 1 are views of the building from the west side (Fig 1a) and southeast side (Fig 1b).

The building had apartments at floors second to sixth, with the ground level serving as a parking garage. Each floor had 4 apartments, each offset from one another by 2.45 m towards the west direction. This offset allowed forming a balcony on the north west corner for each apartment. Available access was by means of two sets of precast spiral stairs and one elevator, all connected to the floors hallway located on the east side of the building.

3.0. STRUCTURAL PROPERTIES

This section presents the structural properties of the lateral load resisting system which are obtained from the building's structural drawings (Holmes Consulting Group 2011) and stated by the designer (Holmes 1965). Other design values, not found in the previous sources, were assumed to be equal to the building code requirements of the time, the NZSS 1900. The requirements for masonry were found in chapter 9.2 of this building code (Smith & Devine 2011).

a) West View b) Southeast View

Figure 1. View of Rolleston Court Flats

3.1. Earthquake Loads

In NZSS 1900 chapter 8 the design base shear force V is presented through an equation V=C*W, where W was the weight of the building and C was the seismic coefficient. The seismic coefficient was obtained from graphs provided in the NZSS 1900 (Davenport 2004), and was dependent on the seismic zone (Zones A, B or C) and the building's natural period of vibration, T. The seismic coefficient required by NZSS 1900 for this building would have been approximately C=0.10.

3.2. Material Properties

Mortar cylinder tests were reported to have minimum cylinder strength of 12.4 MPa, while concrete grout cylinder strengths had a minimum of 17.6 MPa. The hollow concrete block unit compression strength for a Grade A wall was required to be 6.9 MPa.

The reinforcement bars used in masonry walls were deformed mild steel bars of 3/8 in (10 mm), 1/2 in (13 mm) and 5/8 in (19 mm) diameter. For mild steel the design tensile strength was 275 MPa.

3.3. Load Bearing Walls

All walls in the perimeter of the building were designed to be masonry cavity walls. The cavity wall, a popular form of construction in New Zealand at the time, consisted of an inner wythe (load bearing), an outer wythe (veneer) and a cavity between the two of 2" (5 cm). The veneer was supported at different points by using 2.5 cm wide steel wire straps which were anchored to the load bearing wall. The veneer was also provided with 3/8 in (10 mm) diameter vertical rebar along its height and was not anchored to the reinforcing slab. The reinforcement of the veneer was implemented to reduce the risk of earthquake damage of the outer leaf. An illustration of the reinforcement detailing of a cavity wall is shown in Fig. 2.

Illustrations of the building floor plans are presented in Fig 3 describing the distribution of load bearing walls. For floors 2 to 6, as shown in Fig 3a, the masonry walls in the perimeter have a total thickness of 30cm, and the interior walls and elevator walls had thicknesses of 20 cm. For floor 1, the wall distribution is shown to be different to allow for parking spaces. On the west side, instead of walls, 1.20 m x 0.20 m reinforced concrete columns were placed every 4.4m.

All masonry walls were designed to be partially grouted, grouting only cores containing reinforcement. The reinforcement in the vertical direction had 1/2 in (13 mm) diameter bars with 600 mm spacing and in the horizontal direction, pairs of 1/2 in (13 mm) diameter bars with 800 mm spacing. Vertical rebars were lap spliced with starter bars at the floor slab level, placed along with slab reinforcement before concrete pouring. For horizontal rebar, each were anchored onto a perpendicular wall.

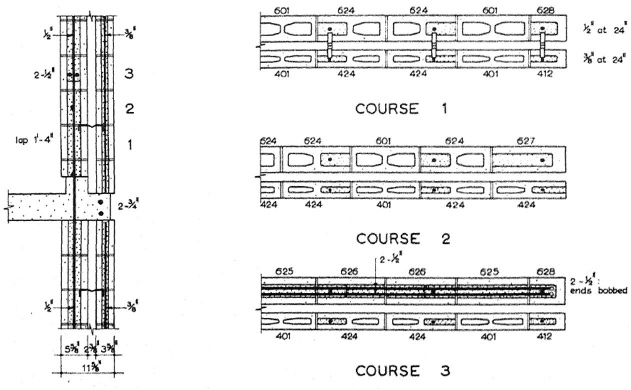
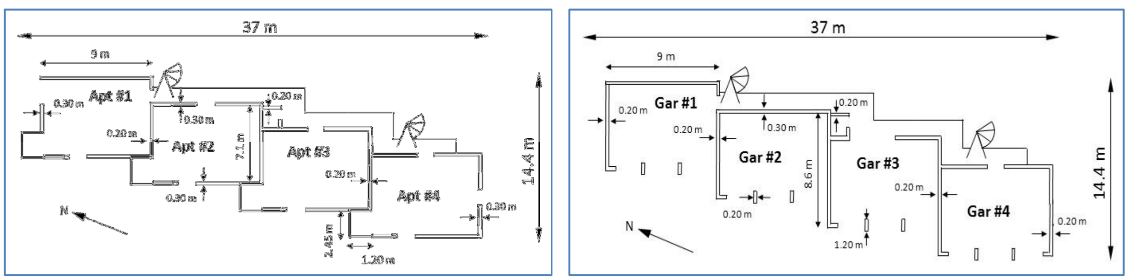


Figure 2. Illustration of a cavity wall. (Holmes, 1965)



a) Plan View of Floors 2 to 5 (Reinforced Masonry Walls)

b) Plan View of Floor 1 (Reinforced Concrete Walls)

Figure 3. Floor Plan View – dimension of load bearing walls

3. DAMAGE DUE TO THE CHRISTCHURCH EARTHQUAKE

At 12:51pm on February 21, Christchurch was shaken by a shallow Mw 6.3 earthquake centred approximately 10 km south-east of the Christchurch Central Business District (CBD) resulting in significant impact on people, buildings, and infrastructure. As part of the response to the national emergency, Civil Defence performed safety assessments of residential and commercial buildings. After each evaluation, placards were placed to indicate that the building's structural integrity had been classified as red, yellow or green.

For the assessment of the Rolleston Court Flats, the building was found to have been seriously damaged and received a red placard. Written on the placard was specified that all load bearing walls spanning on the east to west direction had failed on the second floor. The building was placed on the list of 'critical' buildings set for demolition, due to its number of stories and the risk of collapse (Environment Canterbury 2011) and was demolished on June 14, 2011. (Nikau Contractors Ltd, 2011)

After the building was red tagged, a team from the Project Masonry Recovery Project (Dizhur 2011) visited the site to document the observed damage. The team first performed an inspection of the perimeter of the building. The reinforced concrete walls in the first floor did not have any signs of structural damage. At the second floor, the veneer side of the cavity wall at the north end showed large diagonal cracks (Fig 4a); and the wall at the south end presented a toe crushing failure (Fig 4b). The east and west views showed some light mortar cracks on the face and base of the walls (Fig 4c).

a) North View b) South View c) West View

Figure 4. Damage to Veneer

The interior masonry walls on the second floor, spanning in the east to west direction, had suffered toe crushing failure and were the primary sign of failure of the building. It was estimated, based on the observed level of damage, that these walls' lateral and vertical resistance had been significantly degraded.

The damage was found to be more severe towards the west end of walls in comparison to the east end. Shown in Fig 5 is the damage found on the west end which consisted of spalling of all face shells and crushing of the grout columns. This side also presented one vertical splitting crack at the contact surface with its neighbouring wall, (Fig 5a and Fig 5c). Although damage was also considerable in the east end, there were less cases of face shell spalling, as shown in Fig 6.

Many of the failed walls showed poor quality concrete grout. On some occasions, grout presented a honeycomb shape indicating formation of air pockets due to poor compaction (Fig 7a). In other cases, in portions of a wall with toe crushing failure, the vertical reinforcement was found debonded from the surrounding grout and often buckled (Fig 7b).

The walls spanning in the north to south direction in the second floor, showed no indication of structural damage. Some mortar cracks were visible in the veneer wythe. Mortar cracks were also found at the base of these walls.

4. ANALYTICAL MODEL

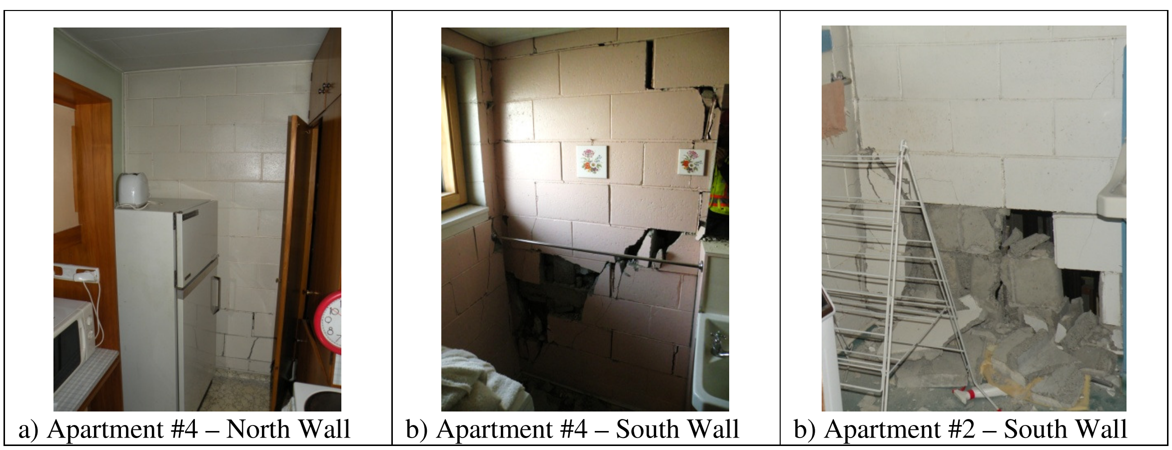
A three dimensional structural model has been developed using the OpenSees software. This model was developed to simulate the nonlinear force deformation behavior of the structure. A nonlinear dynamic analysis was performed by applying the two lateral components of a recorded ground motion time history of the Christchurch earthquake of February 21.

4.1. Nodes

First, a set of nodes was defined to allow modeling each walls' vertical axis. These nodes were defined at the location of each wall's geometric center for each floor level. Two nodes were then added defining the limits of the length of the wall, l_w, locating them at $+0.5l_w$ and $-0.5l_w$ with respect to the previous node. These nodes were then connected using rigid beam links.

a) Apartment #4 – North Wall
b) Apartment #3 – South Wall
c) Apartment #2 – South Wall
d) Apartment #1 – North Wall

Figure 5. Damage to Interior Walls – West End



a) Apartment #4 – North Wall
b) Apartment #4 – South Wall
b) Apartment #2 – South Wall

Figure 6. Damage to Interior Walls – East End

Nodes were also defined at the geometric center of each apartment floor slab and geometric center of each apartment balcony. The geometric center for the nodes of apartments number 2,3 and 4 were estimated including the hallway floor slab.

Nodes at each floor level were constrained assuming a rigid diaphragm behavior. Floor levels were defined for all 7 floor slabs and for the slabs of the elevator's machine room. Inter-storey height for the parking garage was set equal to 3.15 m, , for the apartments 2.54m and for the elevator machine room 2.44 m.

a) Air pockets in concrete grout | b) Debonded vertical reinforcement

Figure 7. Damage to masonry grouted cells

4.2. Weight and Mass

The slab weight for each apartment floor and balcony was estimated using a 24 kN/m^3 density and slab thickness of 20 cm, with the hallway floor slab thickness being 12.5cm. An additional weight load was included to account for partition walls using a distributed load of 0.7 kPa. The weight and mass of the floor slab and balcony were then calculated for each apartment and assigned to the respective geometric center nodes. The wall self-weight was estimated assuming a density of 17 kN/m^3. The weight and mass of each walls was applied in its own geometric center node. The weight of the elevator equipment was assumed to be 14.5 kN and was applied at the node corresponding to the top slab of the elevator machine room.

4.3. Modelling Structural Walls

Structural walls were modeled as beam-column elements with its longitudinal axis defined at its geometric center. For walls located in floors 3 to 6, the walls were modeled using the OpenSees elastic beam column element. For walls located in floors 1 to 2, the walls were modeled using the OpenSees nonlinear beam column element.

Walls in the second floor were modeled dividing the element into two segments each with three integration points. As a result each wall was modeled with one fiber section at each end of the wall height and four elastic sections in the middle. Walls in the first floor were modeled using one segment having one fiber section at each end and three elastic sections in the middle.

Walls elastic section properties were modeled following ASCE 41 recommendations for cracked sections. The elastic modulus E_m for masonry was set equal to 3.85 GPa, and for reinforced concrete was set equal to 21 GPa. For shear deformations, the walls were assumed to remain in the elastic range.

The masonry shear walls were modeled as rectangular walls and not as flanged walls. Although interior walls seemed connected to perpendicular walls at the intersection, the available information on structural drawings and construction practice was not sufficient to follow ASCE 41 and establish the vertical shear transfer capacity at the intersections. In addition, the vertical splitting cracks found between walls in the damaged structure, shown in Fig. 5, suggest that perpendicular walls moved independently in the lateral direction. However, intersecting wall elements are modeled as connected at each floor slab level by the reinforced concrete slab at the intersection point by modeling this intersection with only one node.

4.4. Material Stress-Strain Properties

The masonry and concrete stress-strain behaviors to be used in the fiber sections were modeled using the OpenSees Concrete 02 material model. The input parameters for this model are maximum compressive strength, strain at maximum strength, crushing strength and strain at crushing. The input values for these two materials are presented in Table 4.1.

Table 4.1. Values for input parameters for stress strain material models for masonry and reinforced concrete

Input Parameter	Masonry	Concrete
Maximum compressive strength, (MPa)	5.0	21.0
train at maximum strength, (mm/mm)	0.003	0.003
Crushing strength, (MPa)	1.0	4.2
Strain at crushing, epsU (mm/mm)	0.010	0.010

The maximum compressive strength for masonry was calculated for a partially grouted wall, using equation 4.1 (Paulay and Priestley 1992, Chapter 3, equation 17):

$$f'_m = [0.59\chi f'_{cb} + 0.90(1-\chi)f'_g] \tag{4.1}$$

For this calculation, the masonry block strength, f'_{cb}, was assumed to be 6.9 MPa as per NZZS 1900 requirements. The masonry grout strength, f'_g, was set at 8.6 MPa, one half of the reported masonry grout strength of 17.2 MPa and reduced to account for the poor quality conditions of the material. The ratio of net block area to gross area, χ, was estimated at 0.75. The resulting masonry compressive strength f'_m was of a value of 5.0 MPa.

For the reinforced concrete stress strain properties, concrete was assumed to be unconfined with a compression strength, f'_c, of 21 MPa. The steel reinforcement's stress strain behavior was modeled using Steel 02 material in OpenSees; with a yield strength, f_y, of 275 MPa, with an initial elastic modulus of 210 GPa and a strain hardening ratio of 0.001.

4.5. Ground Motion

The acceleration time history applied was the recorded ground motion at the station CHHC, located at Christchurch Hospital. It has a PGA of 0.33 g in the N-S component and 0.36 g in the E-W component, see Fig 8a, with a duration of severe shaking of approximately 10 seconds. The response spectra for the ground motion, shown in figure 8b, has the higher spectral accelerations between periods of T = 0.25 sec and T = 0.65 sec.

4.6. Viscous Damping

Damping was modelled as Rayleigh damping, with ξ = 3% for first and fourth mode of vibration.

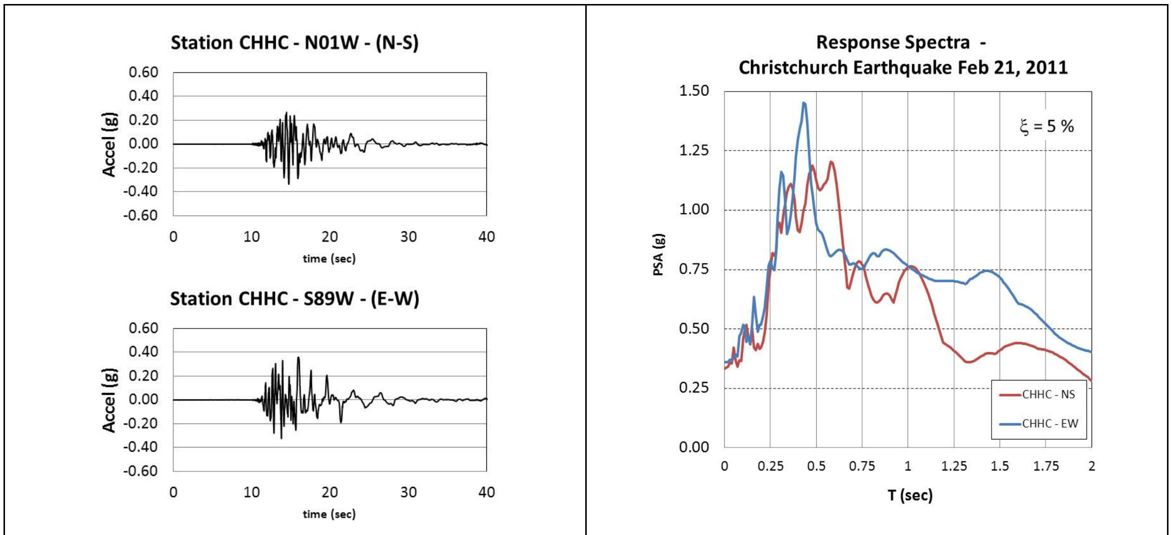
4.7. Dynamic Analysis Properties

The dynamic analysis was run using variable transient analysis using the Newmark transient integrator with β = 0.25 and γ = 0.5. P-delta effects were included in the analysis to model dynamic instability in the structural models time history response.

5. ANALYSIS RESULTS

5.1. Modes of Vibration

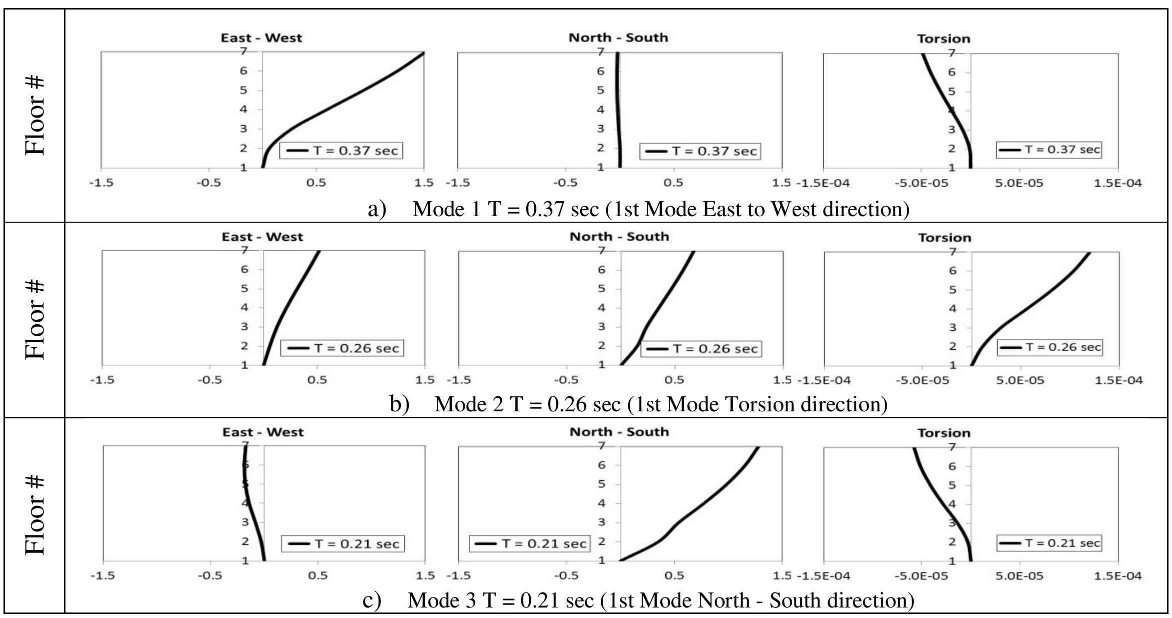
The analysis of modes showed that the first three modes of vibration were coupled lateral-torsional modes. The first mode (T_1 = 0.37 sec) was the first mode of vibration for the east to west direction, the second mode (T_2 = 0.26 sec) the first for the torsion direction and the third mode (T_3 = 0.21 sec), the first for the north-south direction. The modes are described in figure 9, drawn with respect to the buildings geometric center. The following three modes of vibration were T_4 = 0.11 sec, T_5 = 0.086 , T_6 = 0.079 sec.



a) Lateral components of motion time history

b) Response Spectra

Figure 8. Recorded ground motion time history



a) Mode 1 T = 0.37 sec (1st Mode East to West direction)

b) Mode 2 T = 0.26 sec (1st Mode Torsion direction)

c) Mode 3 T = 0.21 sec (1st Mode North - South direction)

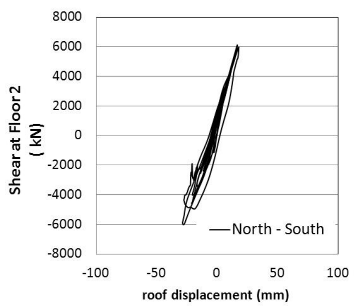
Figure 9. Modes of Vibration

5.2. Gravity Loads

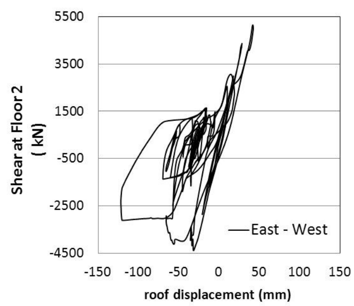
The analysis of gravity loads estimated a total weight of 17400 kN. Perimeter walls, spanning in the north to south direction, were loaded with 71% of the total weight, while interior walls were loaded with 29%. The compression stress in each interior wall due to axial loads was of 1MPa. This value is equivalent to 20% of the estimated masonry compression strength, f'_m.

5.2. Dynamic Analyisis

To characterize the building model's seismic response, plots of the base shear force at the second floor vs roof displacement were made for the north-south direction (Fig. 10a) and east to west direction (Fig. 10b). The plotted hysteretic response shows that for the north south direction of motion the model remains elastic while in the east to west direction the response is inelastic with a non-ductile force-deformation behavior,. The lateral strength of the model is shown to be unsymmetrical, having a higher strength (5150 kN) for positive displacements (east) and a lower strength (-4390 kN) for negative displacements (west). The hysteretic plot shows that strength degradation develops only for the negative displacements while remaining elastic for the positive displacements.

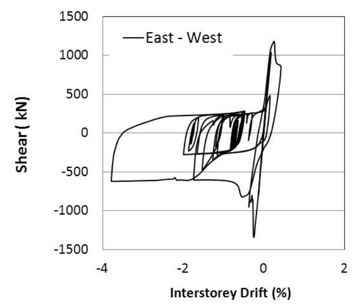


a)



b)

Base shear vs Roof Displacement at floor level 2
a) North-South direction b) East-West direction



c) Hysteretic response of wall element at floor level 2

Figure 10. Time History response at roof level

The development of damage is observed in the plot of force vs inter-storey drift for the second floor of an interior wall element (fig. 10c). The hysteretic plot shows toe crushing failure develops for an inter-storey drift of 0.26%. Shown in the plot are the development of toe crushing for both directions of loading, with deformations on the negative direction reaching a maximum of 3.67% drift.

The displacement time history at the roof level is shown in figure 11a. The model's response has larger deformation demands in the east-to-west direction. It is observed that model is incrementally thrust towards the west direction.

Despite toe crushing failure in interior walls, axial failure in the model is prevented due to redistribution of axial forces. Shown in figure 11b and 11c are the time histories of the total axial forces of interior walls and perimeter walls, respectively. A redistribution initiates with the first formation of toe crushing, at t=12.66 sec As toe crushing increases, the axial force at each interior wall is reduced with the difference taken by the perimeter walls. This redistribution of dead loads finishes at t=15 sec when interior walls carry 6% of the total weight and perimeter walls carry 93%.

6. CONCLUSIONS

The aim of this study was to identify the seismic performance of a six storey RCM building during the Christchurch earthquake of February 2011. In order to do this, an assessment was made of the structural damage and a computer simulation of the buildings structural response was performed. The results of this study show that each of the interior masonry walls were subjected to high compression stresses due to gravity loads, (20% f'_m), which increased their lateral strength and changed the yielding mechanism to a non-ductile mode of failure. The wall failure mechanism was toe crushing, more severe in the west end of the building. Axial failure was prevented due to redistribution of vertical loads from interior walls to undamaged perimeter walls.

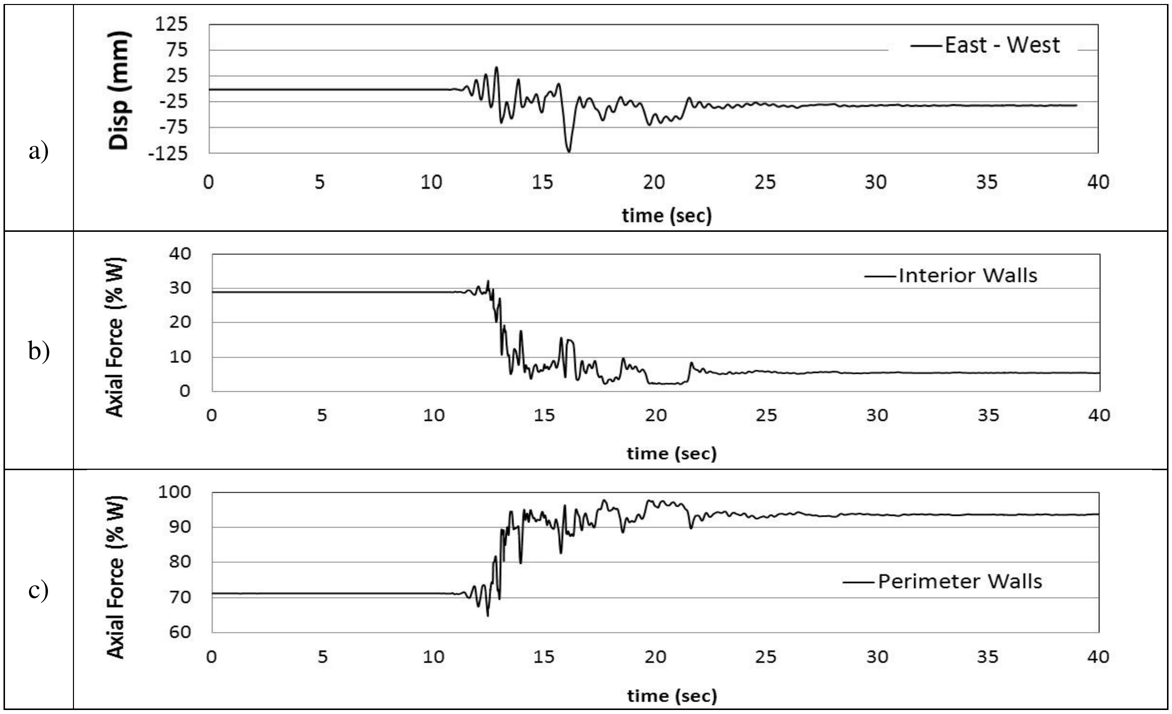


Figure 11. Time History response a) roof displacement
b) Axial Force - Interior Walls c) Axial Force – Perimeter Walls

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