



## MODELLING THE DAMAGE TO THE MANCHESTER COURTS BUILDING IN THE CANTERBURY EARTHQUAKES SEQUENCE

Guojue WANG<sup>1</sup>, Michael PENDER<sup>2</sup> and Jason INGHAM<sup>3</sup>

**Abstract:** Seismic assessment of the Manchester Courts building is presented. This building was demolished following severe damage that resulted from the magnitude 7.1 Darfield earthquake (Canterbury, NZ) on 4 September 2010, amongst widespread objection from heritage supporters who believed that this historic building could be adequately reinstated. Finite element (FE) analysis was used to undertake a performance-based assessment using time-history analyses, and the accuracy of the model was validated by comparing the simulated results with benchmark experimental data and observed damage. The finite element model utilised two-dimensional (2D) elements in three-dimensional (3D) space and the macro modelling method was used.

### Introduction

Manchester Courts was a 7-storey unreinforced masonry building that was constructed in Christchurch, New Zealand in 1905-1906 and regarded as a landmark structure due to its Chicago skyscraper architectural style (see Figure 1). Manchester Courts was a Category I historic building and until 1967 was the tallest building in Christchurch. On 4 September 2010 the Darfield earthquake struck Christchurch, causing major damage to many unreinforced masonry buildings but not causing any deaths, largely because the event occurred at 4.35 am when the central city was effectively deserted. After the Darfield earthquake the Manchester Courts building was severely damaged and there was much debate amongst the community regarding whether this historic building should be demolished or repaired, particularly as the fall-zone for the building was preventing many other surrounding businesses from re-opening for trading. Also, the building was sited at the intersection of 2 busy city streets and so there was significant potential for pedestrians and vehicle occupants in the vicinity to be at risk if the building were to collapse. After considerable media attention the building was demolished between October 2010 and February 2011.

No sooner had the Manchester Courts building been demolished than a major aftershock, referred to as the 22 February 2011 Christchurch (Lyttelton) earthquake, struck the city at 12.51 pm causing the complete collapse of several multi-storey buildings and 185 fatalities. A Royal Commission was convened to investigate events, with particular attention given to buildings in which people had been killed. Intuitively it seemed highly likely that the demolition of the Manchester Courts building saved a considerable number of lives, considering that both anyone within the building at the time of the Feb 2011 aftershock and any pedestrians and vehicle occupants outside the building would have been potential victims. This study sought to investigate the supposition that demolition on this occasion was the correct decision.

Christchurch city is well instrumented with seismic accelerometers, and since the earthquakes there have been considerable geotechnical studies undertaken to establish the soil conditions under the city. Therefore an exercise was undertaken to model the building using finite element software and time-history analyses, to establish firstly if the observed damage in the Darfield earthquake could be accurately replicated using standard modelling

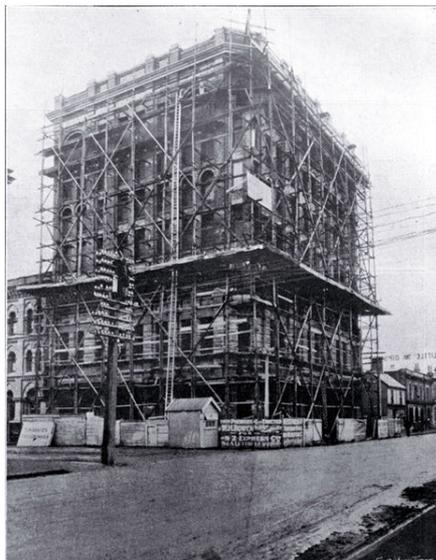
---

<sup>1</sup> Doctoral research student, College of Civil Engineering, Chongqing University, Chongqing, China, wangguojue@gmail.com

<sup>2</sup> Professor, University of Auckland, Auckland, New Zealand, m.pender@auckland.ac.nz

<sup>3</sup> Professor, University of Auckland, Auckland, New Zealand, j.ingham@auckland.ac.nz

procedures, and to then establish the likely performance of the building in the Christchurch earthquake (if the building had not been demolished). The modelling exercise involved the use of shell elements and verification of the masonry modelling procedure by comparison with laboratory data for pier tests somewhat similar to the masonry piers of Manchester Courts. Efforts were also made to incorporate the effect of soil-structure interaction. Having subjected the model to the Darfield earthquake, the forecast deformations and stress levels were compared with limits prescribed in performance-based design codes to establish if the decision to demolish the building following September 2010 was well justified. Finally, the damaged model was then subjected to incremental dynamic analysis by applying scaled versions of the 2011 Christchurch earthquake to gain some insight into the likely outcome in February 2011 if the building had not been demolished.



(a) Under construction in 1906  
([http://en.wikipedia.org/wiki/File:Manchester\\_Courts\\_under\\_construction.jpg](http://en.wikipedia.org/wiki/File:Manchester_Courts_under_construction.jpg))



(b) Under demolition in December 2010  
(<http://www.stuff.co.nz/the-press/news/christchurch-earthquake-2011/canterbury-earthquake-2010/4448192/Demolition-too-slow-say-critics>)

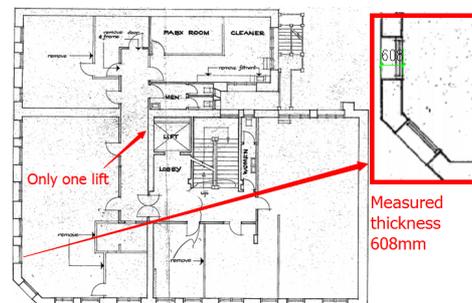
Figure 1. Early and final views of Manchester Courts

### Geometry of Manchester Courts

Manchester Courts had plan dimensions of approximately 20 m×23.6 m, with an approximate height of 37.1 m, but details of other structural elements were uncertain. As shown in Figure 2, many details of the building had changed over time due to building modifications. Consequently building geometric details were estimated from available photos (particularly those taken during demolition) and supplemented by aged design drawings.



(a) A design drawing of the second floor, July, 1951



(b) A design drawing of the second floor, February, 1969

Figure 2. Representative changes over time to some structural elements

Once the building dimensions had been adequately estimated the FE model was generated (Figures 4a,b). Wall thicknesses are shown in Figures 4c,d and Table 1.

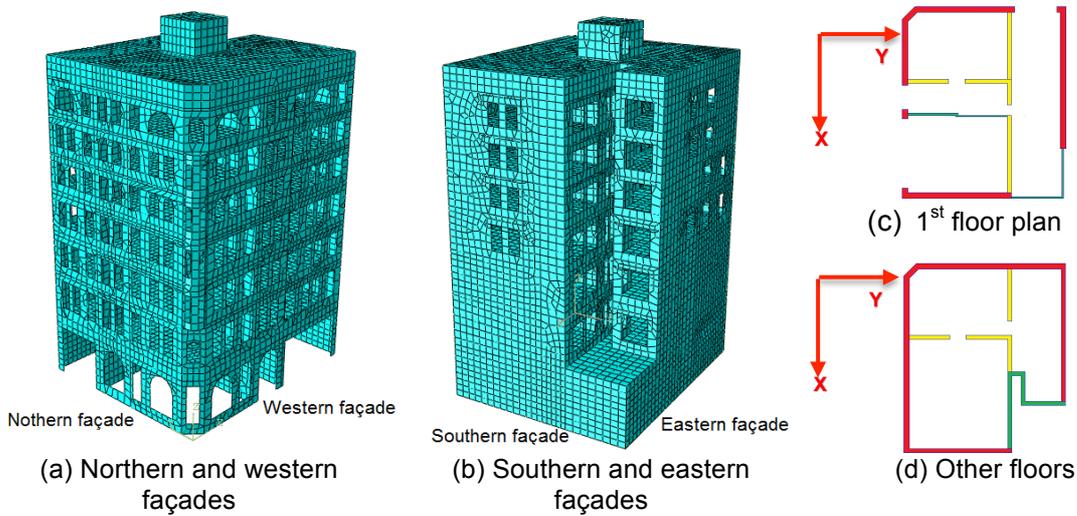


Figure 4: The meshed model and cross-section wall definitions of Manchester Courts

Table 1: Geometrical information of Manchester Courts

Floor	Height (mm)	Pier height (mm)	Wall thickness (mm)		
			Red part*	Yellow part*	Green part*
1st	6500	4050	900	650	300
2nd	5000	3000	900	650	500
3rd	4400	2600	700	500	500
4th	4400	2600	550	500	500
5th	3800	2200	550	500	500
6th	3800	2200	550	500	500
7th	3800	2200	550	500	500

\* Different colours were defined for the walls to distinguish their thickness, see Figure 4.

### Material properties

Manchester Courts was constructed with clay brick walls and timber floor diaphragms (Ingham & Griffith, 2010). Photos of in-situ construction indicated that the northern and western external walls of the bottom two storeys were constructed of reinforced concrete, with reinforcement composed of steel columns embedded in the walls. Because no specific material testing had been undertaken, the material properties required in the analyses were estimated following an extensive literature review based on measured material properties for buildings constructed at approximately the same time as Manchester Courts (see Table 2). The timber was assumed as elastic because no damage of the timber floor diaphragms was observed following the Darfield earthquake.

Table 2: Material properties of Manchester Courts

Material	Masonry	Concrete	Timber	Steel
Compressive Strength	9.6 MPa	25 MPa	elastic	250 MPa
Tensile Strength	0.5 MPa	2.5 MPa	elastic	250 MPa
Elastic modulus	2.8 MPa	23,500 MPa	10,000 MPa	200,000 MPa
Poisson's ratio	0.2	0.2	0.4	0.3

### Building damage in the Darfield earthquake

Building damage occurring in the Darfield earthquake is shown in Figure 5. The dominant cracking was diagonal stepped cracks observed in the piers of the northern and western façades at the 3<sup>rd</sup> and 4<sup>th</sup> floors. Less extensive cracking was also observed on the eastern and southern façades at the top of the building. No structural elements collapsed prior to demolition.



(a) Northern façade

(b) Western façade

Figure 5: Pier crack patterns following the Sept 2010 Darfield earthquake  
(all cracks are shown enhanced for emphasis)

### Finite element modelling

The macro modelling method was used, with clay bricks and mortar assumed to be smeared in a homogeneous continuum. The masonry walls, concrete walls and timber floors were discretized with 4 nodes shell elements (S4R) with reduced integration and hourglass control, and the mesh was automatically generated by ABAQUS (Dassault Systèmes Simulia, 2010). The concrete damaged plasticity (CDP) model in ABAQUS was used to simulate the inelastic behaviour of masonry and concrete. Basic parameters include dilation angle (DA), eccentricity, the ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress ( $f_{b0}/f_{c0}$ ), the ratio of the second stress invariant on the tensile meridian (K), and viscosity parameter (VP). According to the default values suggested in the ABAQUS manual (Dassault Systèmes Simulia, 2010) and the parameters used in previous studies (Pereira et.al 2014; Agnihotri et.al 2013), the adopted values of DA, eccentricity,  $f_{b0}/f_{c0}$ , K and VP were 36, 0.1, 1.16, 0.667, 0.0005 respectively, where eccentricity,  $f_{b0}/f_{c0}$ , K and VP are non-dimensional values, and the unit of dilation angle is degrees. For the compressive behaviour of clay brick masonry the constitutive model proposed by Kaushik et al. (2007) and validated by Agnihotri et.al (2013) was adopted. The damage parameters for compression ( $d_c$ ) and tension ( $d_t$ ) as defined by Agnihotri et.al (2013) were also adopted.

As reported by Ceroni et.al (2012), when the shear wave velocity ( $V_s$ ) of the soil around the foundation is less than 800 m/s the building should not be simulated with a fix-based condition. Wood et.al (2011) found that the shear wave velocities obtained from strong motion stations (SMS) in Christchurch city were much less than 800 m/s (the maximum value was 198 m/s), indicating that soil-structure interaction (SSI) should be modelled. As per the recommendations of NEHRP (2012), vertical springs were implemented to simulate the vertical soil-structure interaction (SSI), but no horizontal springs were included in the model. The strip foundation had dimensions of 120 m length x 1.5 m width and the stiffness of the 171 soil springs distributed beneath the model was calculated according to recommendations by Gazetas et.al (1985) and Wotherspoon (2009), with the value of spring stiffness being 31.8 MN/m. The damping ratio of the vertical spring was assumed as 0.05 followed the recommendation of Wood et.al (2011).

### Ground motions

As shown in Figure 6, four strong motion stations (SMS) in Christchurch city were located around Manchester Courts, referred to as CBGS, CCCC, CHHC and REHS, with the distances between each SMS and Manchester Courts being in the range of 0.88 km to 1.64 km. Cubrinovski et.al (2011) and Cubrinovski et.al (2013) compared the characteristics of the seismic data for the Darfield and Christchurch earthquakes which were obtained from the four SMS, and found that the pseudo-spectral acceleration and spectral displacement amplitudes were similar. Moreover, Cubrinovski et.al (2013) demonstrated that the SMS in Christchurch city were located on D class soil as defined in the NZS 1170.5 (Standards New Zealand, 2004). Consequently it was deduced that the seismic data from all 4 SMS were relevant to the current study.

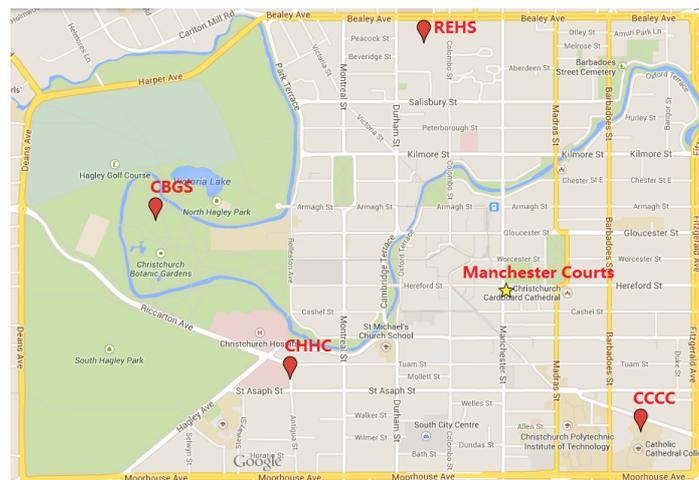


Figure 6: The locations of the ground motion stations and Manchester Courts

### Performance levels and verification of the modelling method

Three structural performance levels as defined in Eurocode 8 (European Committee for Standardization, 2004) were considered in order to conduct seismic assessment using a performance-based approach, being damage limitation (DL), significant damage (SD) and near collapse (NC). For shear failure of primary masonry components the drift limits for DL, SD and NC are 0.1%, 0.4% and 0.53% respectively. The aspect ratio of the most heavily damaged piers was approximately 2.2:1 (height to width), and so test results of clay brick masonry piers with a similar aspect ratio (2:1) (Magenes et.al 1992) were used to validate the modelling method. Figure 7 shows good agreement between the experimental cyclic behaviour and the finite element models.

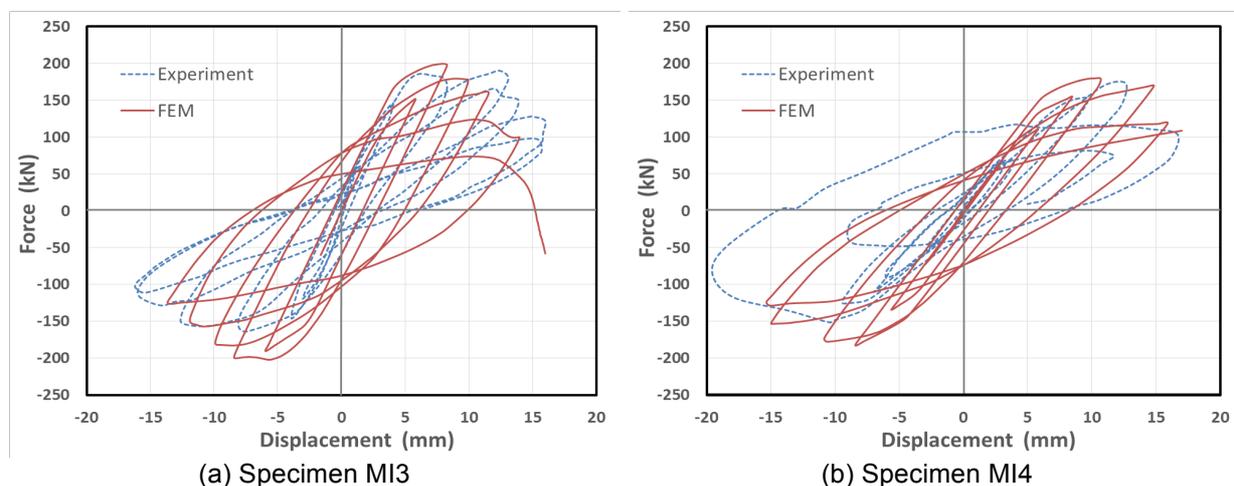


Figure 7: Comparisons between FEM results and for testing by Magenes et al. (1992)

A further verification was the comparison between the observed crack pattern (Figure 5) and that forecast by the model. In Figure 8 it can be seen that the model plastic strains mainly occurred in the piers of the 3<sup>rd</sup> and 4<sup>th</sup> floors, indicating that these piers suffered the most severe damage, corresponding with observations after the Darfield earthquake.

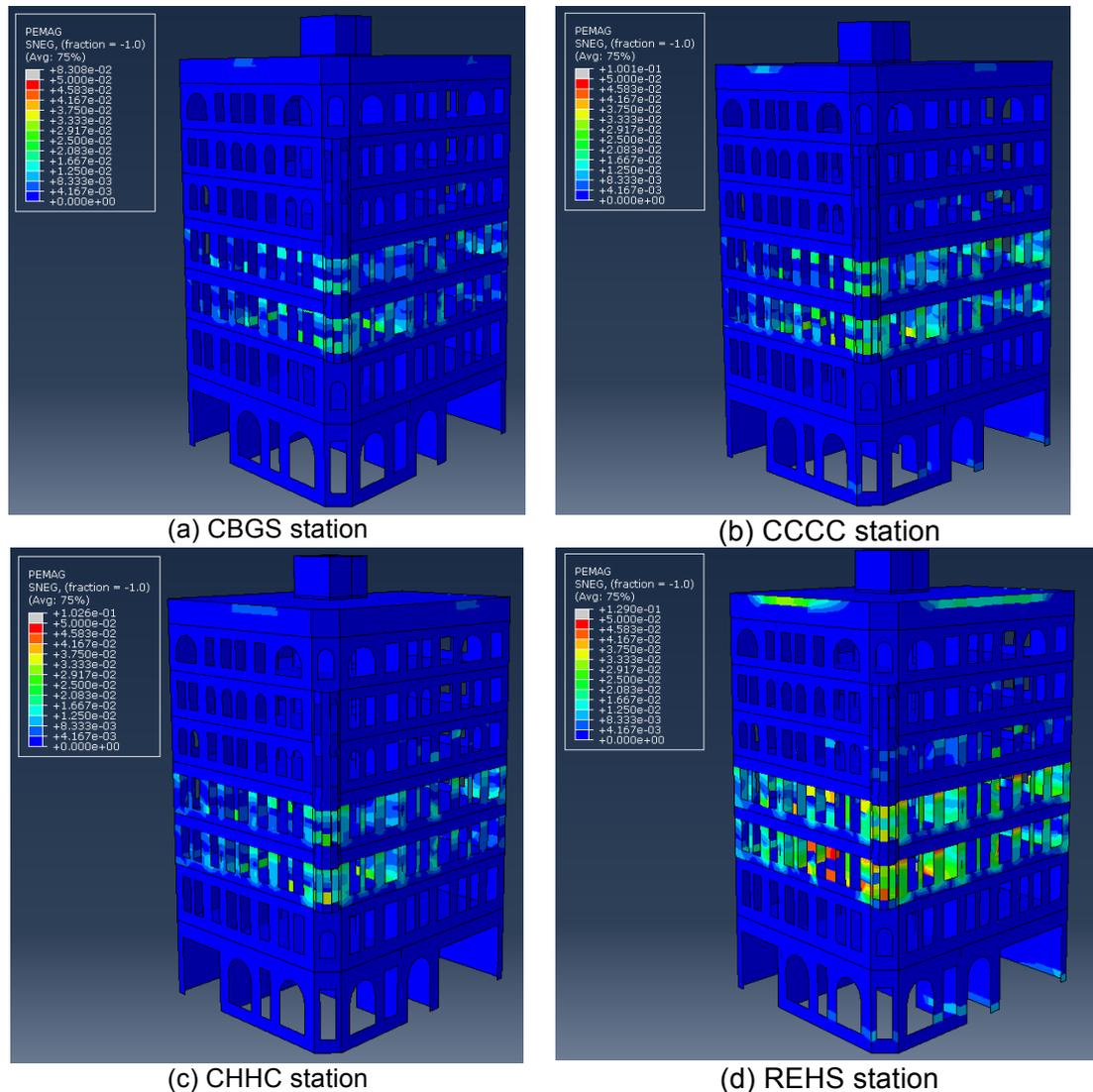


Figure 8: Model plastic strains for the Darfield earthquake, separately calculated using the 4 SMS shown in Figure 6

### Performance-based assessment

To assess performance levels a ‘mean storey drift of piers’ (MSDP) parameter was introduced as shown in Figure 9, based on reference points located at the top and bottom of each pier, such that the governing height was the pier height rather than the storey height.

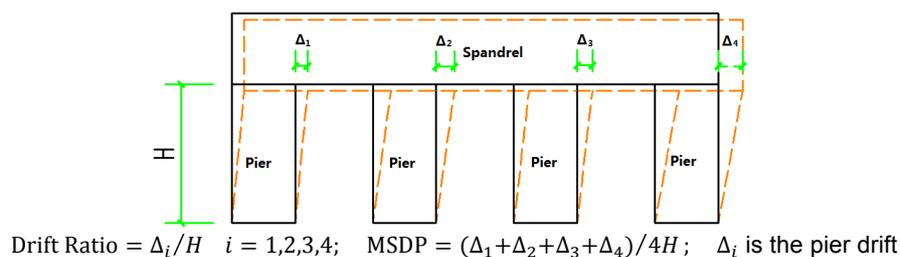


Figure 9: An example of masonry piers and spandrel

Figure 10a shows the seismic data measured at CBGS (X direction), with the corresponding MSDP of the western façade at the 3<sup>rd</sup> and 4<sup>th</sup> floors shown in Figure 10b, c. The peak value of MSDP was chosen as the governing parameter for the performance-based analyses because no significant spandrel damage was observed and global response appeared to be exclusively dictated by pier behaviour, as shown in Figure 5 and Figure 8. For a typical pier height of 2.6 m and a typical storey height of 4.4 m (see Table 1) the effective storey drift is approximately 60% of the MSDP, dependent on the extent of spandrel deformation. Although only the X-direction motion is shown in Figure 10, all three components of recorded ground motion were used in the finite element time-history analyses. In Figure 10b the peak value of MSDP for Part 2 of the motion input is 0.59%, exceeding the adopted NC state of 0.53%.

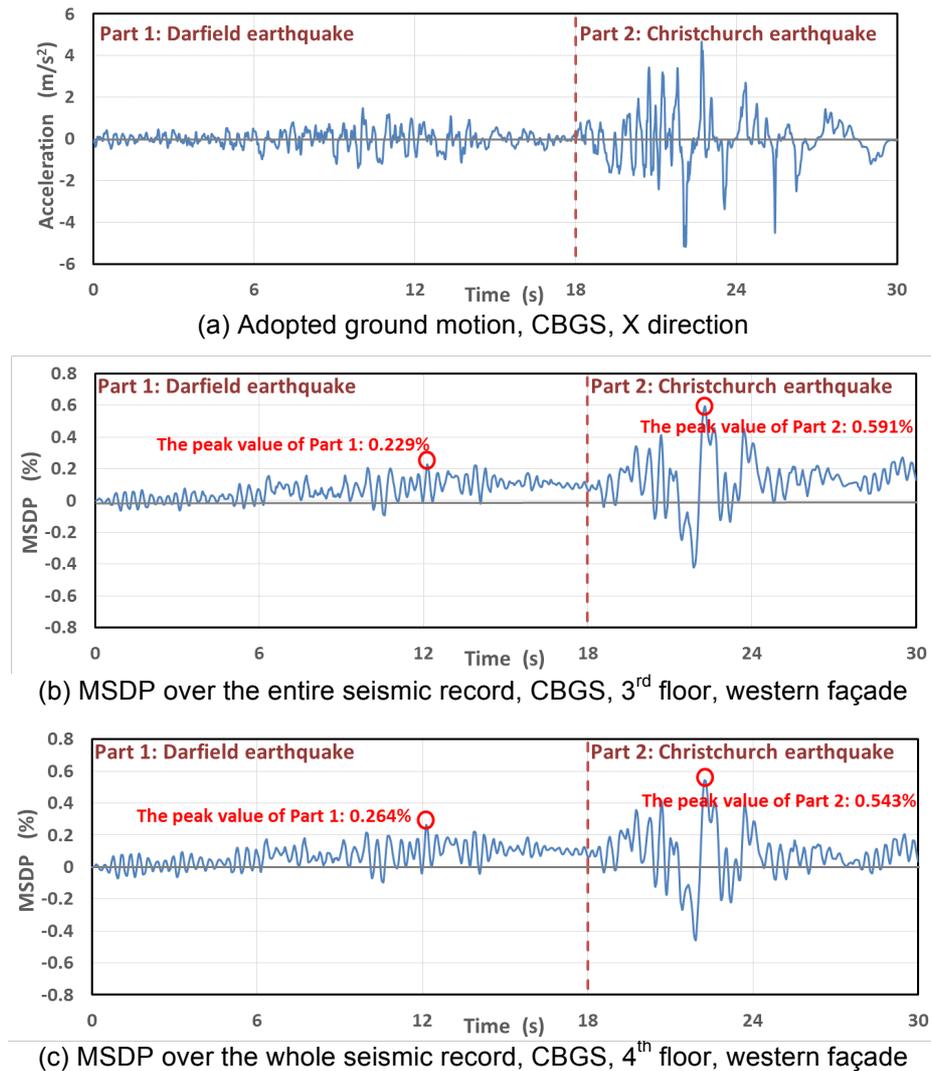


Figure 10: an example of the seismic data and response

Figure 11 demonstrates that in the Darfield earthquake no MSDP exceeded the NC state (0.53%) and that most MSDP were less than the SD state (0.4%), suggesting that the piers of the model would not collapse (corresponding with observed response). The MSDP of the 3<sup>rd</sup> and 4<sup>th</sup> floors were much greater than those of other floors and well in excess of the DL state (0.1%), indicating that these piers were forecast to be severely damaged. All simulation results were in accordance with the observed performance described in the damage report (Ingham & Griffith, 2010). The weighted mean value (WMV) was introduced to represent the overall performance of the building, with the weighting being the reciprocal of the distance between each SMS and Manchester Courts.

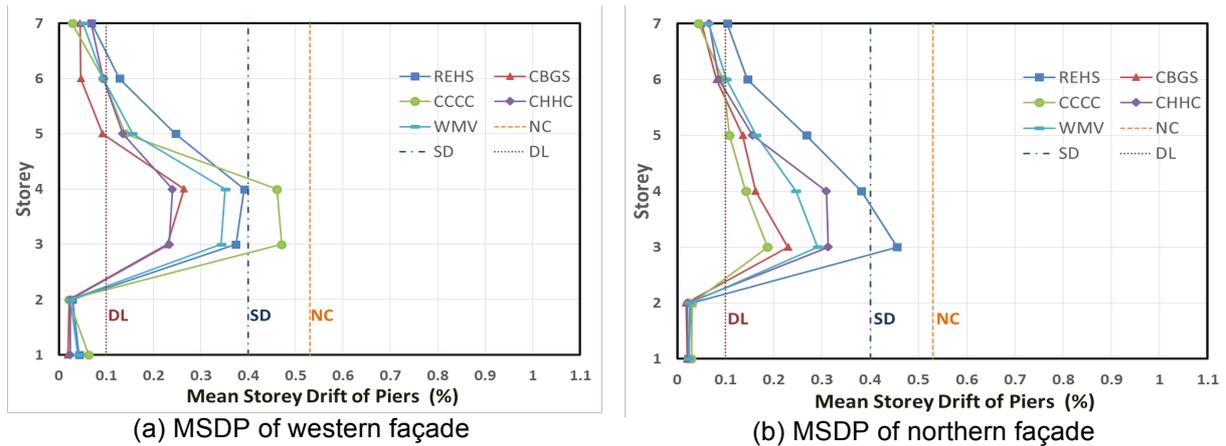


Figure 11: Peak values of the mean storey drift of piers, 2010 Darfield earthquake

Figure 12 presents the MSDP obtained from the model when subjected to 100% of the Darfield earthquake followed by 100% of the Christchurch earthquake. It is seen that the maximum MSDP occurred in the 3<sup>rd</sup> and 4<sup>th</sup> floors, and that regardless of which seismic data was used, at least two MSDP in one façade were over the NC state (0.53%), indicating that the piers would collapse. In addition, the distribution of MSDP can be summarized using their WMV, which were all in excess of the NC state. Note that the MSDP are related to the drifts of piers in the whole storey, not those of a single pier. Consequently, the collapse would occur in most piers of the 3<sup>rd</sup> and 4<sup>th</sup> storeys. In Figure 12a all MSDP of the 3<sup>rd</sup> floor are greater than those of the 4<sup>th</sup> floor, indicating that in the western façade the pier collapse would start from the 3<sup>rd</sup> floor. For the northern façade, Figure 12b shows a mix of peak MSDP between the 3<sup>rd</sup> and 4<sup>th</sup> floors.

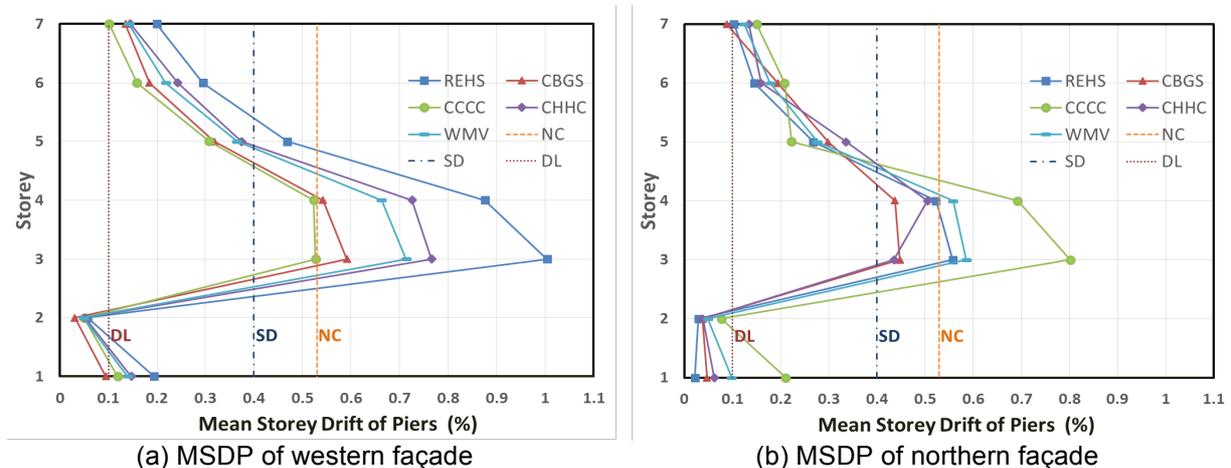


Figure 12: Peak values of the mean storey drift of piers for the 2010 Darfield earthquake and 2011 Christchurch earthquakes combined

### Incremental dynamic analyses

The study was concluded by undertaking incremental dynamic analyses (IDA) to investigate the forecast level of shaking necessary to initiate collapse on 22 February 2011, by beginning the analysis with the building already damaged following the Darfield earthquake. Figure 13 demonstrates the IDA results using scaled REHS data. The initial MSDP of the IDA curves are non-zero because of the residual drifts caused by the Darfield earthquake. The IDA curves of the western façade exceeded the NC damage state and those of northern façade did not. It is seen that the western façade of the 3<sup>rd</sup> storey was the forecast location for the onset of collapse because the corresponding IDA curve exceeded the NC state at 70% of the unscaled earthquake input. Table 3 shows the critical scale factors for each of the four

considered ground motion stations (see Figure 6), indicating that the Near Collapse (NC) damage state was reached in a range of 69.9% to 92.4% of the unscaled records. Considering the WMV weighted ratio, the critical scaling factor was 81.5%.

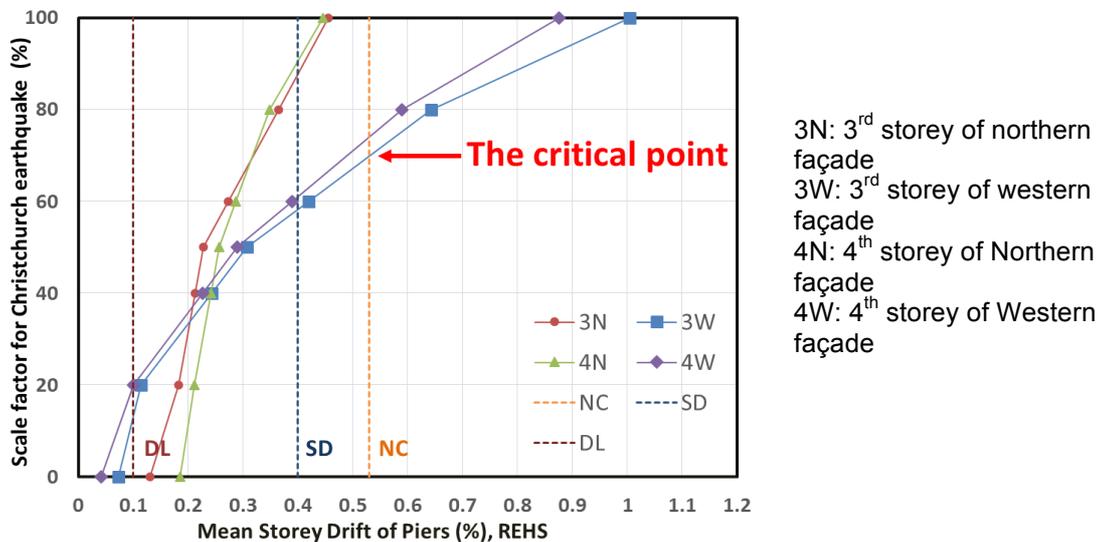


Figure 13: An example of IDA curves

Note that the length of the original seismic data is 140 seconds (Darfield earthquake) and 95 seconds (Christchurch earthquake) respectively, but that the adopted seismic data had a duration of only 30 seconds, being 18 seconds of the strongest shaking from the Darfield earthquake at 100% scaling (i.e. unscaled), and 12 seconds of the strongest shaking from the Christchurch earthquake data, scaled as part of the IDA procedure. In addition, aftershocks were not considered. Consequently, actual building damage would probably be greater than forecast based on the presented simulations.

Table 3: Critical IDA scale factors and location of first collapse

Seismic data	CBGS	CCCC	CHHC	REHS	WMV
Scale factor for Christchurch earthquake	92.4%	79.3%	82.1%	69.9%	81.5%
Collapse location*	3W	3N	3W	3W	3W

\* See legend of Figure 13

## Conclusions

Nonlinear time-history analyses of the historic unreinforced masonry Manchester Courts building were conducted using the finite element method. The main purpose of the study was to investigate whether the Manchester Courts building would likely have collapsed in the Christchurch earthquake, after having been previously damaged in the Darfield earthquake. Thus, only the ground motion data of the Darfield earthquake and the Christchurch earthquake were adopted, using data from four SMS located near Manchester Courts. From the analyses it was forecast that Manchester Courts would have collapsed in the Christchurch earthquake, for the reasons described below:

- 1) A comparatively high value of the Near Collapse (NC) damage state was used (0.53%), as prescribed in Eurocode 8. In comparison, FEMA 356 prescribes a Collapse Prevention (CP) limit state for primary unreinforced masonry (URM) components of 0.4%;
- 2) The MSDP is the average value of the peak drift of all piers in the façade at any floor, and thus if the MSDP exceeded the NC state then complete collapse at that storey was forecast, rather than collapse of isolated piers;
- 3) Only part of the Darfield earthquake record was used, but in reality the building was subjected to both the complete Darfield earthquake and numerous aftershocks prior



to the Christchurch earthquake. Thus the building was subjected to greater seismic input than assumed in the model;

- 4) Regardless of which seismic motion station was considered, there was always at least two MSDP of the northern or western façade at the 3<sup>rd</sup> or 4<sup>th</sup> floors that were greater than the NC state.

From data reported in Table 3 it was determined that global collapse would have started in the piers of the 3<sup>rd</sup> storey. The MSDP of the western façade are higher than those of the northern façade, except when the ground motion data from the CCCC SMS was used. This result suggests that global collapse would probably have started from the western façade of 3<sup>rd</sup> floor.

## References

- Agnihotri, P., Singhal, V., & Rai, D. C. (2013). Effect of in-plane damage on out-of-plane strength of unreinforced masonry walls. *Engineering Structures*, 57, 1-11.
- American Society of Civil Engineers. (2000). FEMA 356 Prestandard and commentary for the seismic rehabilitation of buildings. Reston, Va. Washington, D.C., American Society of Civil Engineers; Federal Emergency Management Agency.
- Ceroni, F., Sica, S., Pecce, M., & Garofano, A. (2012). Effect of Soil-Structure Interaction on the dynamic behavior of masonry and RC buildings. Paper presented at the 15th World Conference on Earthquake Engineering (WCEE), Lisbon, Portugal.
- Cubrinovski, M., Bradley, B., Wotherspoon, L., Green, R., Bray, J., Wood, C., & Wells, D. (2011). Geotechnical aspects of the 22 February 2011 Christchurch earthquake. *Bulletin of the New Zealand Society for Earthquake Engineering*, 44(4), 205-226.
- Cubrinovski, M., Pampanin, S., & Bradley, B. (2013). Chapter 1: Geotechnical and Structural Aspects of the 2010–2011 Christchurch (New Zealand) Earthquakes. In M. Garevski (Ed.), *Earthquakes and Health Monitoring of Civil Structures* (pp. 1-35): Springer Netherlands.
- Dassault Systèmes Simulia (2010). Abaqus Analysis User's Manual version 6.10.
- European Committee for Standardization. (2004). Eurocode 8: Design of Structures for Earthquake Resistance - Part 3: Assessment and retrofitting of buildings. Brussels, Belgium: European Committee for Standardization.
- Gazetas, G., Dobry, R., & Tassoulas, J. L. (1985). Vertical Response of Arbitrarily Shaped Embedded Foundations. *Journal of Geotechnical Engineering-Asce*, 111(6), 750-771.
- Ingham, J. M., & Griffith, M. (2010). Severe damage to Manchester Courts. from <http://eqclearinghouse.org/co/20100903-christchurch/wp-content/uploads/2010/09/Damage-to-manchester-Courts.pdf>
- Kaushik, H. B., Rai, D. C., & Jain, S. K. (2007). Stress-Strain Characteristics of Clay Brick Masonry under Uniaxial Compression. *Journal of Materials in Civil Engineering*, 19(9), 728-739.
- Magenes, G., & Calvi, G. M. (1992). Cyclic behaviour of brick masonry walls. Paper presented at the Tenth World Conference on Earthquake Engineering (WCEE), Madrid, Spain.
- NEHRP Consultants Joint Venture. (2012). *Soil-Structure Interaction for Building Structures*. Washington, D.C: National Institute of Standards and Technology.
- Standards New Zealand. (2004). *Structural design actions Part 5: Earthquake actions–New Zealand, NZS 1170.5:2004*. Wellington, New Zealand: Standards New Zealand.
- Pereira, J., Campos, J., & Lourenço, P. B. (2014). Experimental Study on Masonry Infill Walls under Blast Loading. Paper presented at the 9th International Masonry Conference, Guimarães, Portugal
- Wood, C. M., Cox, B. R., Wotherspoon, L. M., & Green, R. A. (2011). Dynamic site characterization of Christchurch strong motion stations. *Bulletin of the New Zealand Society for Earthquake Engineering*, 44(4), 195-204.
- Wotherspoon, L. M. (2009). *Integrated modelling of structure-foundation systems*. Doctoral dissertation, University of Auckland.