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RUTHERFORD & CHEKENE

Seismic Performance Investigation
Draft Report

Clarendon Tower

78 Worcester Street
Christchurch, New Zealand

February 2012

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Draft Report

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EXECUTIVE SUMMARY

Rutherford & Chekene has been engaged by the Canterbury Earthquakes Royal Commission to independently investigate the structural performance of the Clarendon Tower located in Christchurch, New Zealand. The Clarendon Tower suffered structural and nonstructural damage in the Darfield earthquake of September 4th 2010, the Lyttelton earthquake of February 22nd 2011, and respective aftershocks.

The Clarendon Tower suffered fairly significant damage to its thin concrete topping slab, east and west frame beams, and north and south frame shear link connections. This resulted in reduced seating of precast double-T floor elements on several floors. Nonstructural elements such as the precast cladding, historic facade and precast stairs also incurred damage with one flight of stairs completely collapsing over several stories. Much of the observed damage has been attributed to the generally strong shaking experienced at the site, frame elongation and torsional excitation.

This report captures both the torsional excitation and strong shaking effects through accurate modeling of the building geometry, parametric analysis of frame beam stiffness and by utilizing elastic and inelastic response spectra from the four recording stations in the Central Business District of Christchurch. The issue of frame elongation, other than to highlight its obvious effect on precast floor seating and cracking of the concrete topping slab, has not been investigated thoroughly. The major conclusions from the analysis results are that:

- Torsion is predicted in the elastic range of structure response owing to irregularity in the south frame.
- The north, and to a much lesser extent, south frame softening results from the elastic torsion combined with the non-ductile nature of the shear link connection detail.
- Increased north frame flexibility generally reduces south frame interstory drift ratios, except at a few concentrated stories, essentially protecting much of the south frame and keeping it from softening to match the north. This result is explainable by understanding that although the roof displacement demand increases slightly, the shift in the center of rigidity permits the same center of mass displacement for a lower south frame drift.

TABLE OF CONTENTS

1. INTRODUCTION 1

1.1 Background on Study 1

1.2 Building Description 1

Overview 1
Structural System 2
Foundation System 4

1.3 Relevant Codes & Compliance 4

Original Design Codes 4
Current Design Codes 4
Compliance & Alterations 4

2. DAMAGE DESCRIPTION & REMEDIATION 5

2.1 Overview 5

2.2 Structural Damage 5

Frame Beams & Shear Link Connection on North and South Frames 5
Frame Beams on East and West Frames 7
Diaphragm 8
Seismic Joints 12

2.3 Nonstructural Damage 12

Precast Cladding 12
Historic Facade 13
Precast Stairways 13

2.4 Other Damage 14

3. STRUCTURAL INVESTIGATION 15

3.1 Outline of Study 15

3.2 Results of Study 16

Modal Analysis 16
Response Spectra 17
Interstory Drift Ratios 19

3.3 Discussion & Findings 22

Elastic Torsion 22
Inelastic Torsion 22
Recording Station Differences 23

4. CONCLUSION 24

4.1 Summary 24

4.2 Recommendations for Future Study 24

5. REFERENCES 25

LIST OF FIGURES

Figure 1: Clarendon Tower Site and Strong Motion Recording Stations 2
Figure 2: Typical Floor Plan 3
Figure 3: Participating Walls at Lower Stories 3
Figure 4: North and South Frame Shear Link Connection Detail 5
Figure 5: Shear Link Connection Test 6
Figure 6: Shear Link Connection Damage in Clarendon Tower's North Frame 6
Figure 7: Frame Elongation Modes 7
Figure 8: Outward Column Movement Mode 7
Figure 9: Frame Elongation (left) and Bulging at levels 5 to 11 (right) 8
Figure 10: Diaphragm Cracking 9
Figure 11: Cold-Drawn Wire Mesh Fracture (left) and Partial Double-T Loss of Seating (right) 9
Figure 12: Crack Width Survey 10
Figure 13: Crack Width Survey 11
Figure 14: "Turfering" Scheme Sketch (left) and Picture (right) 12
Figure 15: Precast Panel Connection Damage 12
Figure 16: Historic Facade Damage 13
Figure 17: Stairway Damage 13
Figure 18: ETABS Screenshots 15
Figure 19: Coupling of Translation and Torsional Response 16
Figure 20: Christchurch Botanic Gardens Station 17
Figure 21: Christchurch Cathedral College Station 18
Figure 22: Christchurch Hospital Station 18
Figure 23: Christchurch Resthaven Station 19
Figure 24: Ductility of 1 and 100% Stiffness 20
Figure 25: Ductility of 1 and 80% Stiffness 20
Figure 26: Ductility of 1 and 60% Stiffness 20
Figure 27: Ductility of 3 and 100% Stiffness 21
Figure 28: Ductility of 3 and 80% Stiffness 21
Figure 29: Ductility of 3 and 60% Stiffness 21

LIST OF TABLES

Table 1: Fundamental Periods 16
Table 2: Christchurch CBD Recording Stations 17

1. INTRODUCTION

1.1 BACKGROUND ON STUDY

Rutherford & Chekene has been engaged by the Canterbury Earthquakes Royal Commission to independently investigate the structural performance of the Clarendon Tower located in Christchurch, New Zealand. The Clarendon Tower suffered structural and nonstructural damage in the Darfield earthquake of September 4th 2010, the Lyttelton earthquake of February 22nd 2011, and respective aftershocks. Of greatest interest and forming the bulk of this report is the torsional response of an apparently symmetric building and the apparent ineffectiveness of the couple formed by the perpendicular frames to control it. In order to focus the scope of this investigation we have not attempted to quantitatively predict other effects, such as frame elongation, opting instead to really understand the observed torsion.

Descriptions of damage to the Clarendon Tower, primarily documented by Holmes Consulting Group, were provided to Rutherford & Chekene by the Royal Commission. These documents were relied upon for information concerning the state of the structure at various points in time. However, detailed damage descriptions as far as extent and location were not available, particularly for the perimeter frames. Cracking of the topping slab and frame elongation were reasonably well described but the variation by floor was not. A complete list of all documents available at the time of this report can be found in the references section of this report. No representative from Rutherford & Chekene has yet visited the Clarendon Tower since it incurred damage.

1.2 BUILDING DESCRIPTION

Overview

The Clarendon Tower is a nineteen story building located at the intersection of Worcester Street and Oxford Terrace in the Central Business District of Christchurch, New Zealand. Figure 1 shows its geographic position. A parking structure that wraps around the east and south sides outside of the typical tower footprint exists at levels zero through three. Levels eighteen and nineteen have significantly reduced occupied area to the typical floor plan and generally house plant and machinery. Additionally the hotel's historic facade was preserved and tied back to the Clarendon Tower on the north and west faces. At the time of the earthquakes, the Clarendon Tower was primarily occupied by law firms and utilized as general office space between levels four to seventeen. Floors one to three were shared between parking, retail, restaurant and office space.

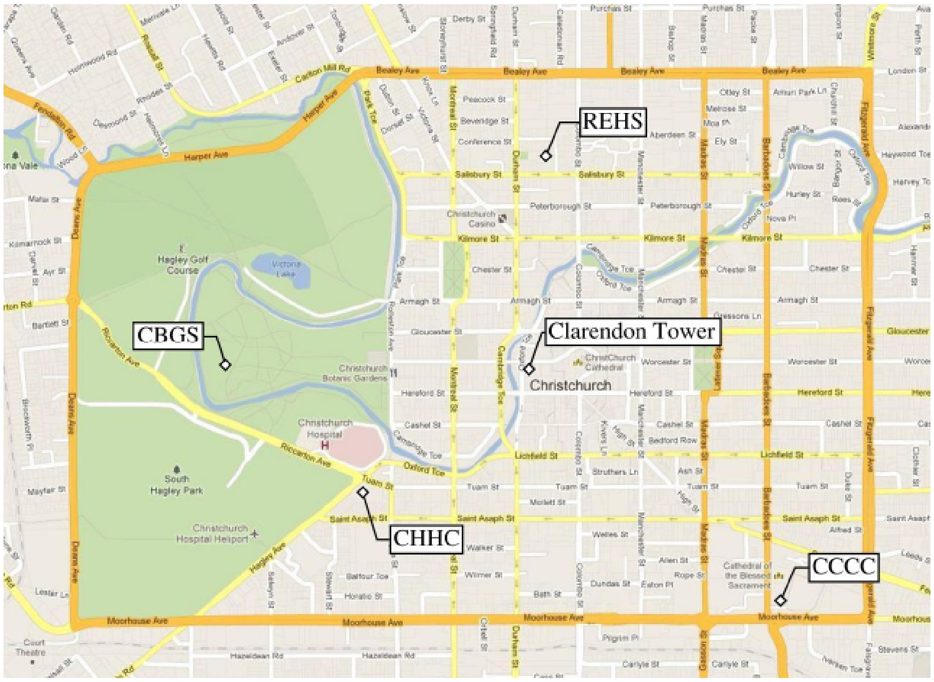

Figure 1: Clarendon Tower Site and Strong Motion Recording Stations

Structural System

A structural system composed of a combination of precast and cast-in-place concrete provides resistance to both gravity and lateral forces. Flange hung 25cm deep precast double-T units with 65mm minimum seating and cast-in-place concrete topping with cold-drawn wire mesh typically carry gravity load by spanning east-west to lines B, E, I and L. They also serve as the diaphragm for distributing lateral loads to north and south frames through 10mm U-bars at 600mm on center lapped with the wire mesh. Diaphragm forces make their way to the east and west frames through the 60mm topping concrete and wire mesh. Precast beams on lines B, E, I, and L then transfer gravity forces to the cast-in-place columns. A typical floor plan with north arrow is shown in Figure 2.

The beams and columns on lines B and L, with bay widths ranging from 5.8 to 6.5m, form the east and west concrete moment frames, respectively. Lines 2 and 19, with bay widths of 2.9m, form the north and south concrete moment frames, respectively. Additionally, the southern frame is interrupted below the fourth floor, by eliminating concrete columns for example, in order to permit vehicular access to the parking areas. Frame beams of dimension 50 cm wide by 85 cm deep are consistent over the height of the building. Frame columns are generally 80x80cm up to the eighth floor and 70x65cm above with larger sections at the four corners of the tower. A special detail was used at beam midspan of the north and south frames to connect precast beam stubs together.

Although a parking deck with ramps exists on the east and south sides of the building, it has been seismically separated from all levels but the first. This is specifically mentioned in a letter by Holmes Wood Poole & Johnstone dated February 2nd, 1987 [39]. Therefore the majority, but not all, of the cast-in-place concrete and block walls do not resist lateral loads generated in the upper floors of the tower. Several notable walls that do participate are located on lines 19 and 21 of the first several stories, however, as shown in Figure 3.

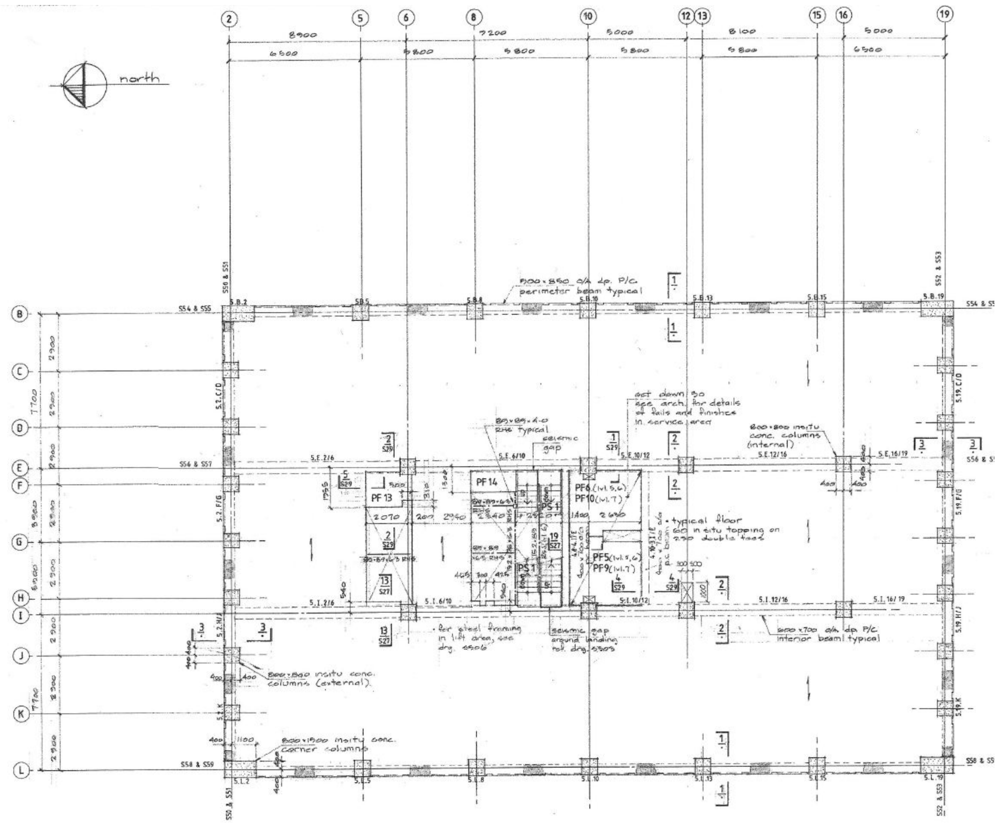

Figure 2: Typical Floor Plan

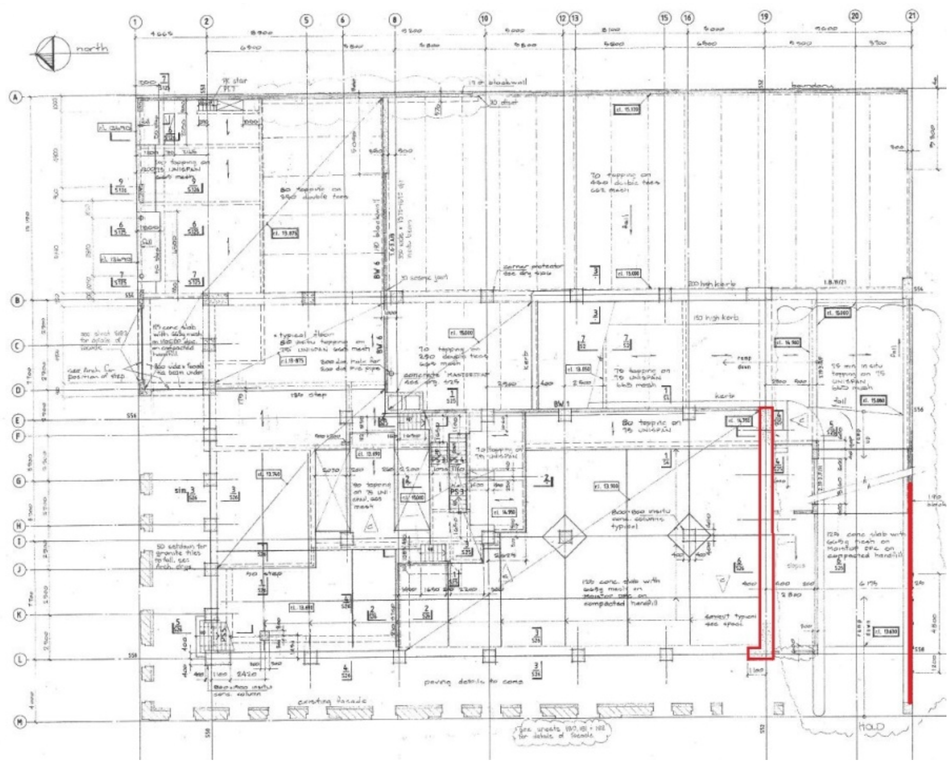

Figure 3: Participating Walls at Lower Stories

Foundation System

A linear concrete strip footing 4m wide by 1.2m deep is typically provided on lines B, L, 2 and 19 for moment frame columns or concrete walls at level zero. Similarly sized footings also support interior columns which are generally tied to at least one other interior column footing. Level zero and the eastern portion of the first floor not above level zero have cast-in-place concrete slabs-on-grade. Miscellaneous concrete and block walls have no explicit footings but instead meet the slab-on-grade.

1.3 Relevant Codes & Compliance

Original Design Codes

Constructed in 1987, the Clarendon Tower was designed by Holmes Wood Poole & Johnstone Ltd, Consulting Civil & Structural Engineers, and Warren & Mahoney Architects Ltd. Its design was carried out in accordance with the current New Zealand codes of the time, notably NZS 4203:1984 for loads and NZS 3101:1982 for concrete [39]. The original analysis, conducted in ETABS, gave a total base shear of 7730 kN based on a static base shear coefficient of 0.048.

Current Design Codes

In this report, the code level response spectra have been developed based on NZS 1170.5:2004 with amendments that came into effect May 18th 2011. These amendments principally increased the zone factor, Z, from 0.22 to 0.30 for the Christchurch area. NZS 3101:2006 represents the current standard for concrete design.

Compliance & Alterations

Rutherford & Chekene did not receive any information pertaining to compliance and alterations.

2. Damage Description & Remediation

2.1 Overview

Damage to the Clarendon Tower under the Darfield earthquake was primarily in the form of cracking in the concrete topping slab and precast frame beams. Aside from expected flexural cracking in frame beams, diaphragm and other beam cracks were attributed to frame elongation as a result of the reversing nonlinear cyclic behavior of concrete members. Repair by epoxy injection of these cracks was under way and nearly complete before the Lyttelton event occurred. Under the greater demands imposed by the Lyttelton event, the diaphragm again cracked, more severely this time, and some of the shear links on the north and south frame apparently failed in a non-ductile mode. The north frame appeared to experience more damage as a result of torsion than the south frame. This effect is investigated extensively later in this report. Frame elongation was even more pronounced than in the Darfield event, leading to concern over double-T loss of seating. A scheme to tie the building together, referred to as "turfering" in the Holmes Consulting Group reports, was proposed and implemented in order to avoid partial floor collapse.

2.2 Structural Damage

Frame Beams & Shear Link Connection on North and South Frames

Instead of allowing hinging to occur at the ends of beams on the north and south frames, as for typical moment frames, the midspan was detailed to yield similar to a coupling beam of a coupled shear wall as shown in Figure 4. A University of Canterbury test conducted after the structure was built showed an inherent flaw in the connection design that produced non-ductile behavior outside of the designed hinge zone [45]. The load-displacement relationship and image of the connection at the end of testing is shown in Figure 5.

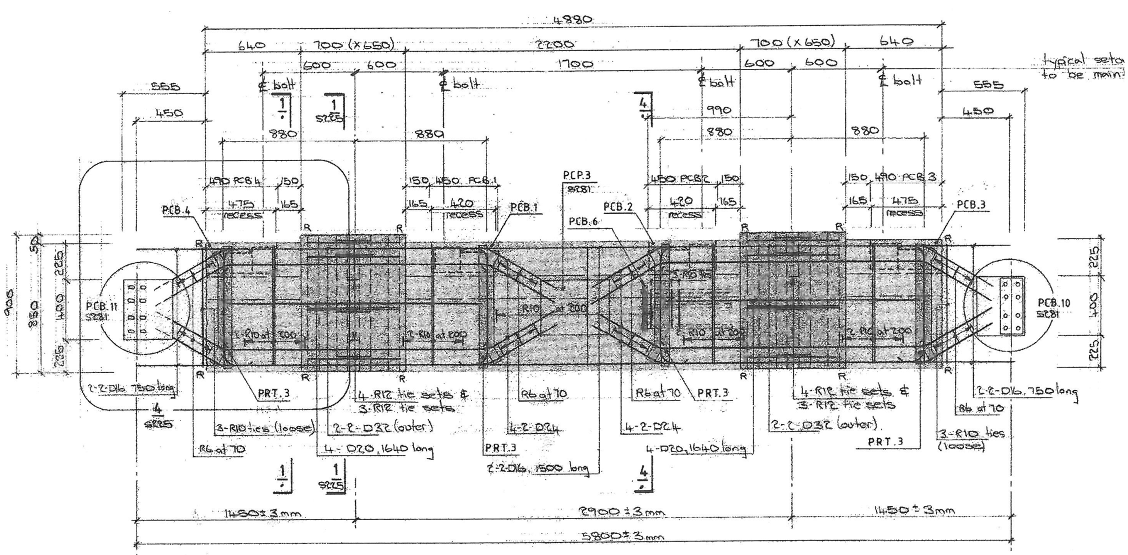

Figure 4: North and South Frame Shear Link Connection Detail

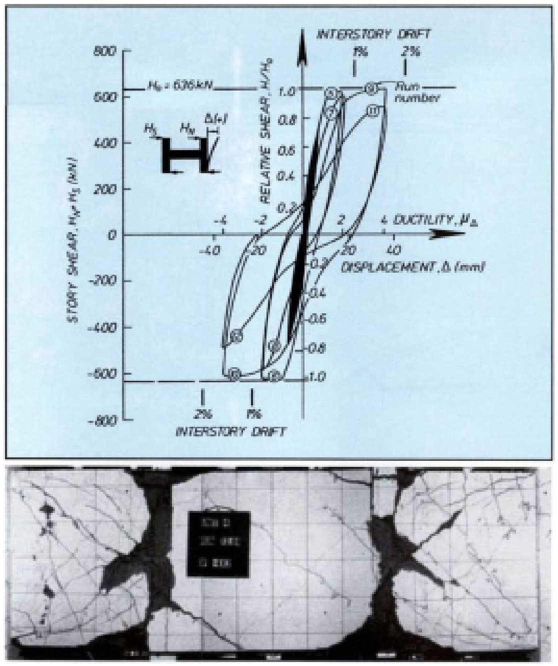

Figure 5: Shear Link Connection Test [45]

Investigation after the Darfield earthquake noted flexural cracking within the span and vertical cracking at the column faces of the south frame beams which are assumed also to occur at the north frame [2]. These cracks were in the process of being epoxy injected, which may have been completed, when the Lyttelton event occurred. Immediately after the Lyttelton event, 2mm shear cracks were observed in the north and south frame beams and it was noted that the connection did not appear to have activated properly [26]. Later 10-15mm diagonal shear cracks were discovered appearing around levels seven and eight in addition to a vertical offset, referred to as vertical stepping, at the beam midspan connection as shown in Figure 6 [28, 29]. Frame elongation was noted as more severe for the north frame, somewhere between 20 and 50mm, although it was documented for the south frame as well [28, 34]. See Figure 7.

Figure 6: Shear Link Connection Damage in Clarendon Tower's North Frame [28]

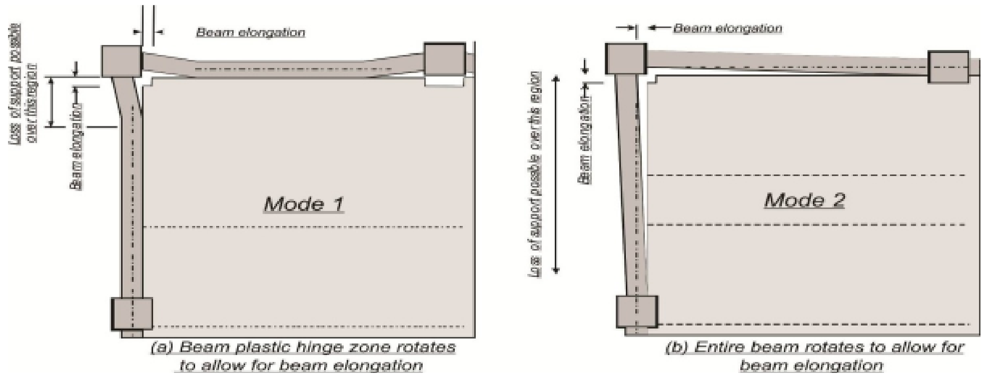

Figure 7: Frame Elongation Modes [31]

The general focus of later sections of this report is the investigation of the apparent response as indicated by the disparity in damage between the north and south frames. The pattern of damage of the north and south frame suggests a torsional response which would not be expected to be significant due the overall symmetry and the built-in torsional resistance of perimeter frames in both directions. Personnel in the building during aftershocks of the Lyttelton event however claim to have observed the building noticeably swinging about the south frame [35]. Additionally, north frame crack widths tended to remain constant while those elsewhere in the building increased as aftershocks occurred.

Frame Beams on East and West Frames

Flexural cracks in addition to a vertical crack between the precast beam and the cast-in-place column were observed on the west frame after the Darfield event [2]. These types of cracks in beams were subsequently repaired through penetration injection [13]. After the Lyttelton earthquake, new 2mm flexural cracks at the column face were observed and extensive rebar yielding was assumed to have occurred [26, 32]. Frame elongation is thought to have caused movement of the north and south frame approximately 100mm farther apart at levels five to nine in addition to resulting in bulging at levels five to eleven as shown on the left and right of Figure 9, respectively [31, 34]. The bulging that appears in Figure 9 is said to result from an outward movement of the east and west frame columns as sketched in Figure 8. The yielding of the rebar has also brought up the issue of low cycle fatigue and it is unclear, without further analysis, how much remaining capacity exists in the east and west frame beams after undergoing repeated earthquake excitation [31].

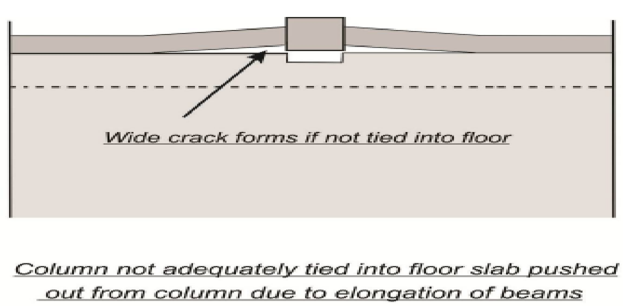

Figure 8: Outward Column Movement Mode [31]

Figure 9: Frame Elongation (left) and Bulging at levels 5 to 11 (right) [31]

Diaphragm

Large cracks between double-T units was observed after the Darfield earthquake, concentrating between levels seven and nine [2]. The cold-drawn wire mesh was fractured in at least one location at the eighth floor. Many of these diaphragm cracks were repaired before the Lyttelton event occurred. The effectiveness of the repair method is unclear though since extensive diaphragm cracking was observed at both the north and south frame lines, albeit more pronounced at the north, after the Lyttelton event. These east-west oriented cracks generally form between the frame beams and the precast double-T units and are of approximately 20-30 and 10-20mm width on the north and south frames, respectively [26]. See Figure 10. Mesh fracture also accompanied large crack widths as seen on the left of Figure 11.

Additionally, north-south oriented cracks formed where the double-T units met the precast beams on lines B, E, I and L, being most pronounced immediately adjacent to the north and south frames and tapering off as you move away. Widths of these cracks tend to be of the order of 10mm as shown in Figures 12 and 13 [28]. In at least one location these cracks were accompanied by a vertical offset of 27mm which is thought to indicate partial double-T loss of seating. See right picture of Figure 11.

Crack widths are reported to apparently increase 5-10mm with successive significant aftershocks except at the north frame indicating north frame stiffness degradation and increased torsional response [30]. Figures 12 and 13 show a crack width survey conducted by Thornton Tomasetti. Accounting for the lack of survey data at the lower floors, significant cracking tends to span from levels three to eleven and levels seven to nine on the north and south ends of the building, respectively.

Diaphragm cracking is thought to be caused by elongation of the exterior frames as sketched in Figures 7 and 8 [31]. Since these diaphragm cracks indicate reduced seating for the flange hung double-Ts, steel ties were installed across the building to restrain the floor against further expansion. Partial floor collapse was thought to be the most likely collapse scenario for this structure [27]. A sketch of the implemented scheme is provided on the left of Figure 14. There was also concern that the nearly fulllength cracking of the diaphragm at the tower perimeter would preclude transfer of diaphragm forces to the exterior frame lines [35].

Figure 10: Diaphragm Cracking [31]

Figure 11: Cold-Drawn Wire Mesh Fracture (left) and Partial Double-T Loss of Seating (right) [31]

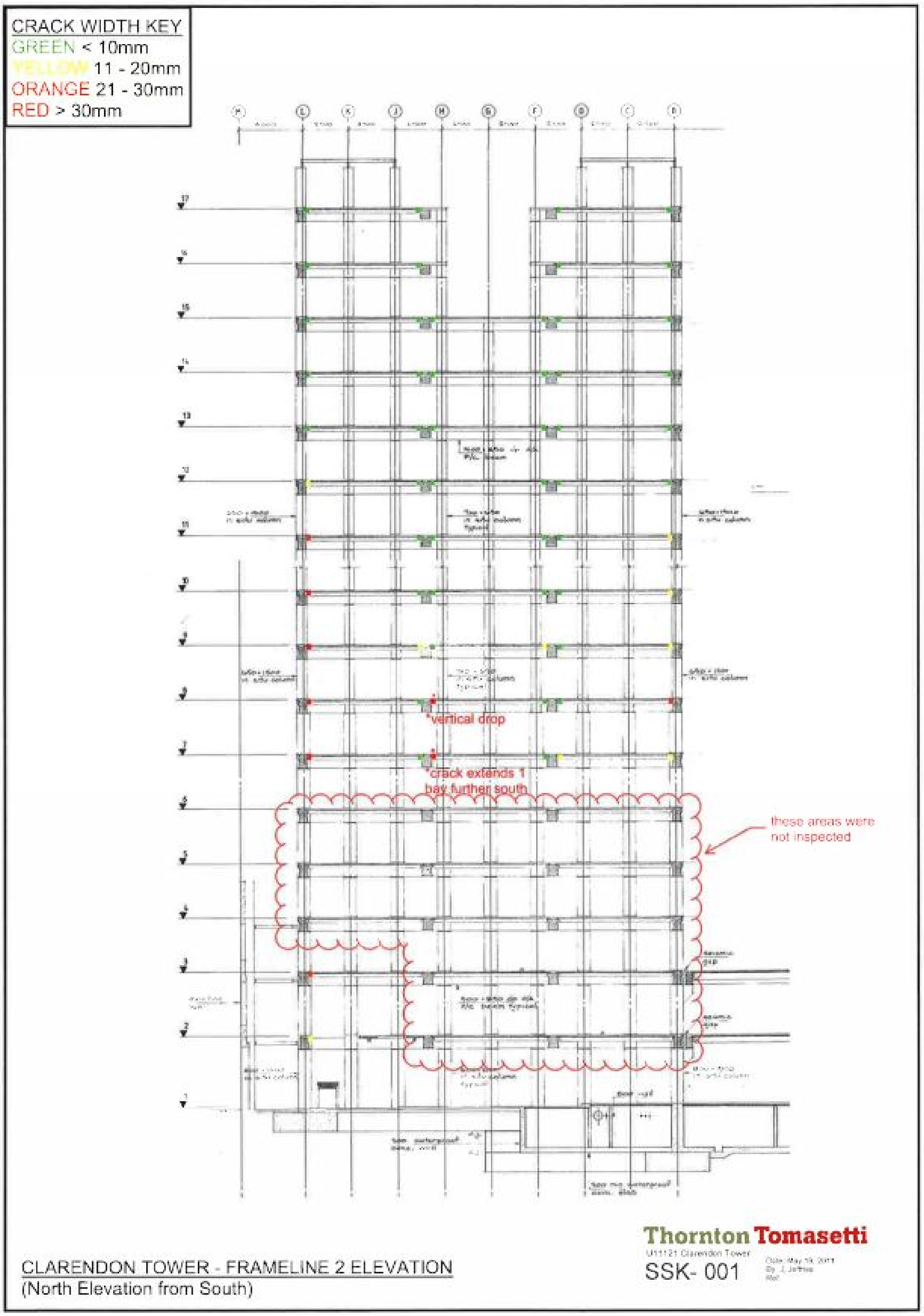

Figure 12: Crack Width Survey [30]

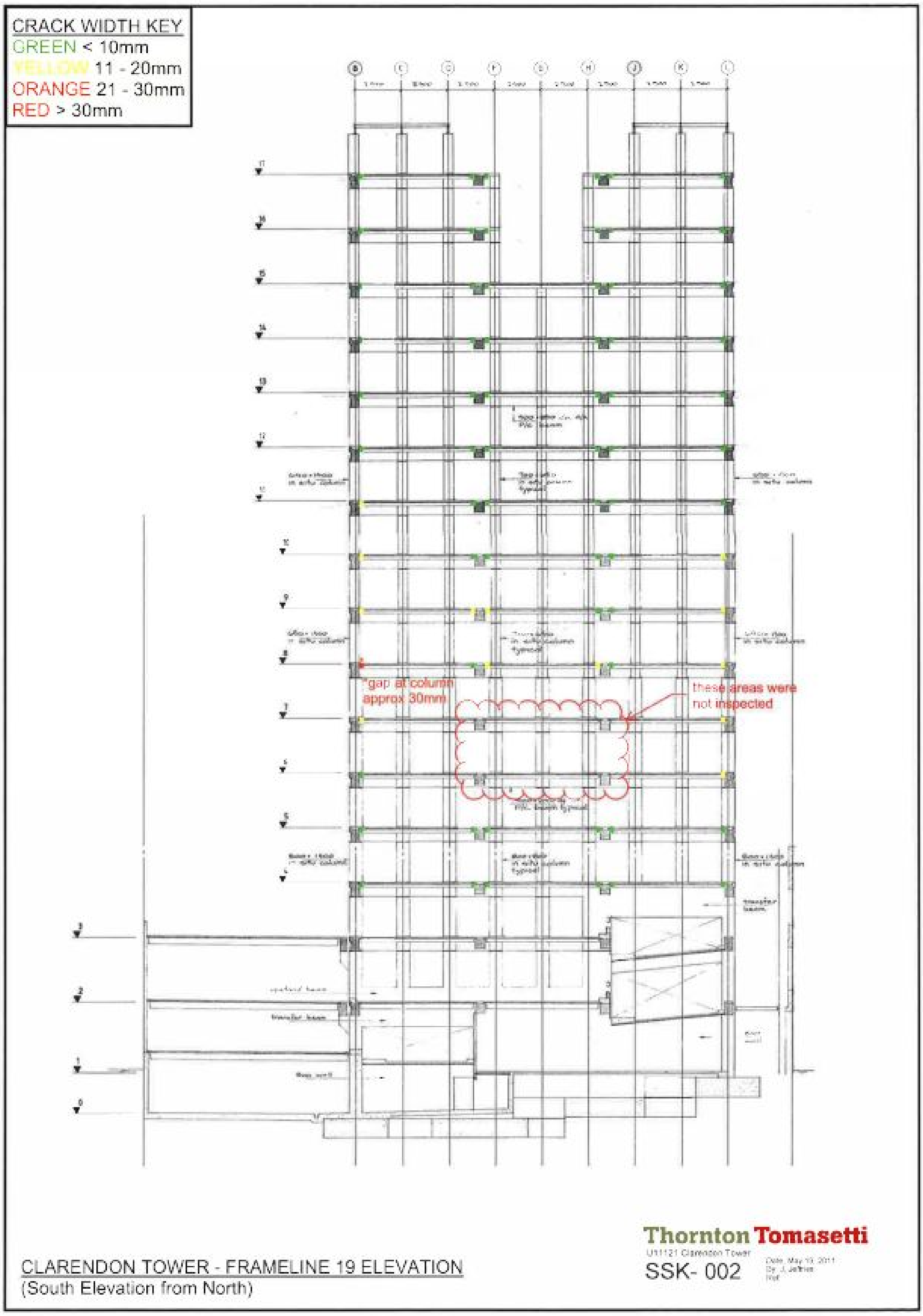

Figure 13: Crack Width Survey [30]

Figure 14: "Turfering" Scheme Sketch (left) and Picture (right) [30, 31]

Seismic Joints

Extensive damage was not observed for any of the seismic joints inspected after the Darfield earthquake. The joint between the south-west corner ramp and tower appeared to move as designed as did the seismic gap adjacent to the historic facade on the northern side of the fourth floor [1, 21]. No report submitted to Rutherford & Chekene mentioned inspection of seismic joints after the Lyttelton or subsequent aftershocks.

2.3 Nonstructural Damage

Precast Cladding

No damage to the precast cladding was observed following the Darfield event. The connection between the precast cladding and frame is shown in Figure 15. One site report noted that opening occurred between approximately eight panels of the tower cladding units on the north frame [34]. Investigations conducted after subsequent aftershocks noted failed panel cladding connections on the north face and potential danger of panels being shaken off the building [35]. Damage to the precast cladding has generally been attributed to frame elongation effects [32].

Figure 15: Precast Panel Connection Damage [31]

Historic Facade

In the Darfield event the historic facade was observed to crack and bulge slightly [3]. A falling hazard was remediated by clearing out the loose plaster [15]. The northwest corner of the historic facade was further damaged in the Lyttelton event with the north face apparently moving away from the tower [26]. Spalling of the historic facade is thought to result from frame elongation effects [32].

Figure 16: Historic Facade Damage [31]

Precast Stairways

Movement of the stairways was recorded following the Darfield event with a 2mm crack at the construction joint between precast stairs and in-situ topping concrete measured at the north stair on the third floor [1, 8]. As part of the repair work, the seismic gap between the bottom of the stairs and the concrete floor was reinstated [11]. The north stair collapsed over several stories under the Lyttelton earthquake which occurred as a result of compression hinging in the center of the stair run due to inadequate seismic gaps widths or improper sliding [26]. The south stair also partially collapsed [32].

Figure 17: Stairway Damage [31]

2.4 Other Damage

The structural and nonstructural damage previously described comprises the vast majority of all information recorded for the Clarendon Tower. Note that damage to ceilings, partitions and other nonstructural components normally affected by earthquake shaking was not addressed in any report submitted to Rutherford & Chekene although, given the level of shaking, it is expected to have occurred. No significant concrete or block wall damage was found, other than minor spalling of one block wall, and most concrete columns and beam-column joints had limited to minor cracking. Several double-T units exhibited cracking of their webs and flanges in addition to the diaphragm cracking between units during the Lyttelton event. Some liquefaction was also observed after the Lyttelton event in the car parking but no structure damage was attributed to it by investigation engineers [26].

3. Structural Investigation

3.1 Outline of Study

In order to assess the torsional response of the Clarendon Tower an elastic ETABS model was constructed that captured the overall mass and stiffness distribution, and could distinguish the effect of concrete and block walls at the lower floors, irregularity of the southern frame to permit car parking and connection of the historic facade. Expected material properties, rather than nominal, and effective cracked section second moments of areas are used. Rigid diaphragms are assumed for all levels except that the historic facade was not under this constraint but rather its diaphragm connection is modeled explicitly. Screenshots are shown in Figure 18. Response spectra from the four recording sites in Christchurch's Central Business District at two levels of building ductility for both the Darfield and Lyttelton events are used in modal response spectrum analysis to compute interstory drift ratios. A code spectrum is also included for reference. These results are then used to predict which north and south beam shear links would see the greatest demand. A further parametric study is conducted where the previously selected beams are softened and the response spectrum analyses rerun. This procedure allows nonlinear behavior to be approximated with some level of accuracy in the absence of nonlinear time history analyses. P-delta is included for all analyses.

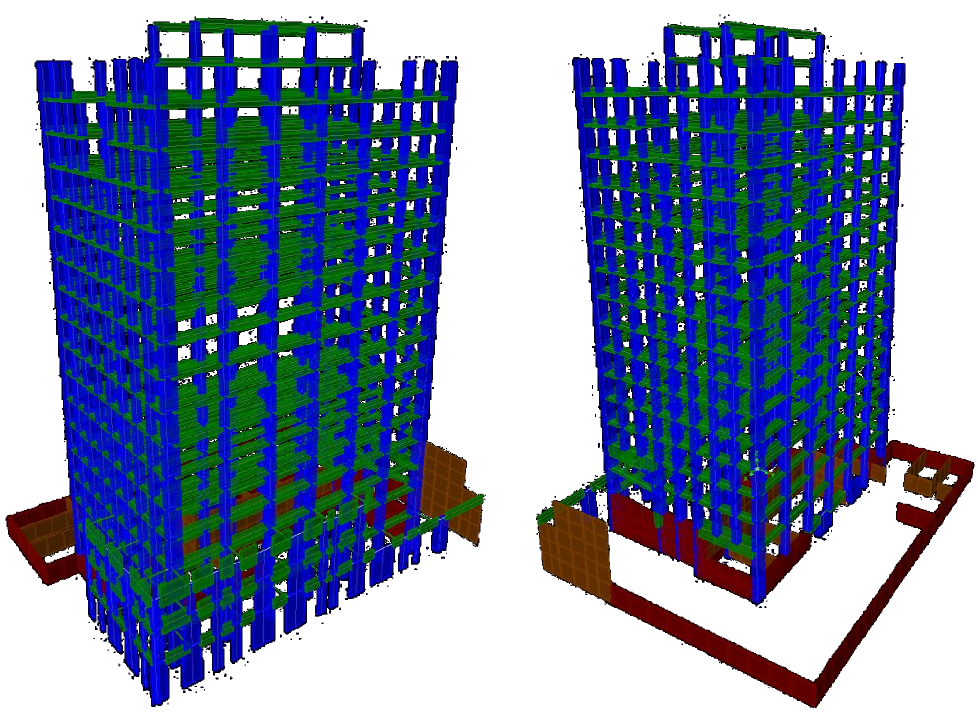

Figure 18: ETABS Screenshots

3.2 Results of Study

Modal Analysis

The first several modes for the longitudinal, transverse and torsional directions are shown in Table 1. Longitudinal signifies movement in the North-South direction. The transverse mode shape is not purely translation but instead includes some amount of torsion that eventually explains the north frame experiencing greater interstory drift ratios. See Figure 19. Approximately 65% and 10% of the mass participates in the first and second modes, respectively, for the longitudinal and transverse directions with very little mass participation in the torsional mode. The three columns of periods correspond to the beam stiffness parametric study discussed later in this report.

Table 1: Fundamental Periods (sec)

Mode	Beam Stiffness Multiplier		
	100%	80%	60%
1st Longitudinal	2.47	2.47	2.47
1st Transverse	1.85	1.91	2.00
1st Torsional	1.25	1.27	1.30
2nd Longitudinal	0.82	0.82	0.82
2nd Transverse	0.67	0.67	0.68

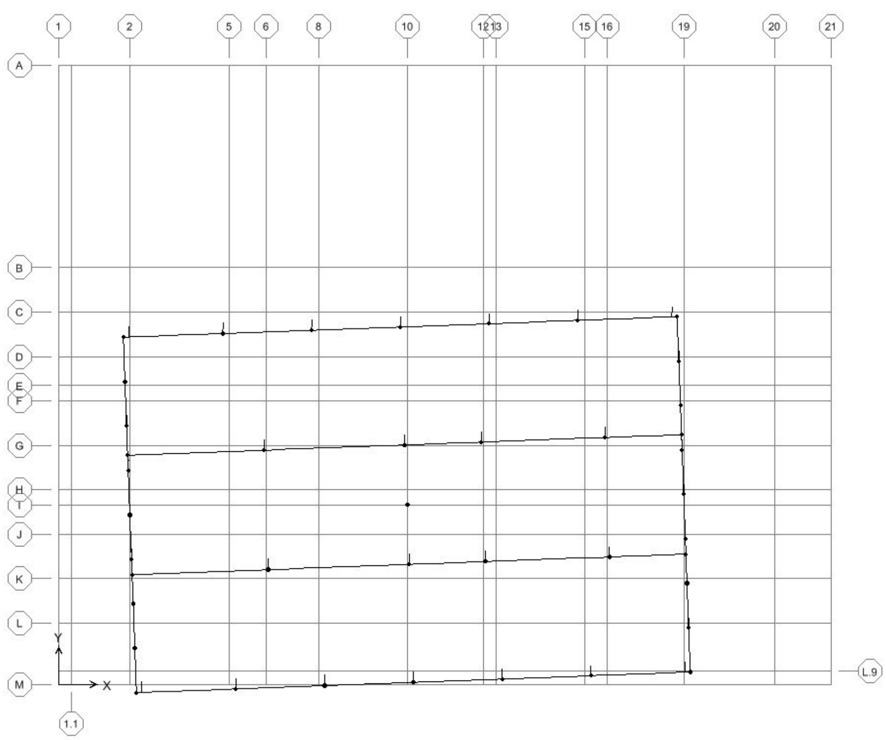

Figure 19: Coupling of Translation and Torsional Response

Response Spectra

Acceleration time histories from each of the four recording stations, see Table 2 and Figure 1, for both the Darfield and Lyttelton events are used to construct displacement response spectra for an elastic structure and one with an assumed ductility of three. The program BiSpec Professional is used to construct the spectra. BiSpec takes the acceleration time histories and computes constant ductility spectra through iteration on the yield force of a single degree-of-freedom system. A bilinear hysteretic model is assumed with a hardening slope equal to two percent of the elastic slope. Figures 20 through 23 plot the response spectra used in the Clarendon Tower analyses including the code spectrum discussed next.

The code spectrum is constructed per NZS 1170.5:2004 assuming a site subsoil class D, a hazard factor Z of 0.3, a return period factor R_u of 1.0, and no near-fault factor. Note that the hazard factor reflects the update to the 2004 code after the Lyttelton earthquake. Taking the structural performance factor S_p to be 0.7, calculating a k_{mu} for a ductility of both one and three, and considering the code lower limit then converted the elastic site hazard spectrum to a design level pseudo-acceleration response spectrum. Since this study is principally concerned with displacements, this spectrum was amplified by the ductility to produce the pseudo-acceleration response spectrum consistent with an inelastic displacement response spectrum. The code spectrum is included in this report only as a benchmark and should not be relied upon for assessing code compliance. For example, no consideration of minimum base shear based on the equivalent static procedure nor inclusion of a drift modification factor k_{dm} is made.

Table 2: Christchurch CBD Recording Stations

Station ID	Description
CBGS	Christchurch Botanic Gardens Station
CCCC	Christchurch Cathedral College Station
CHHC	Christchurch Hospital Station
REHS	Christchurch Resthaven Station

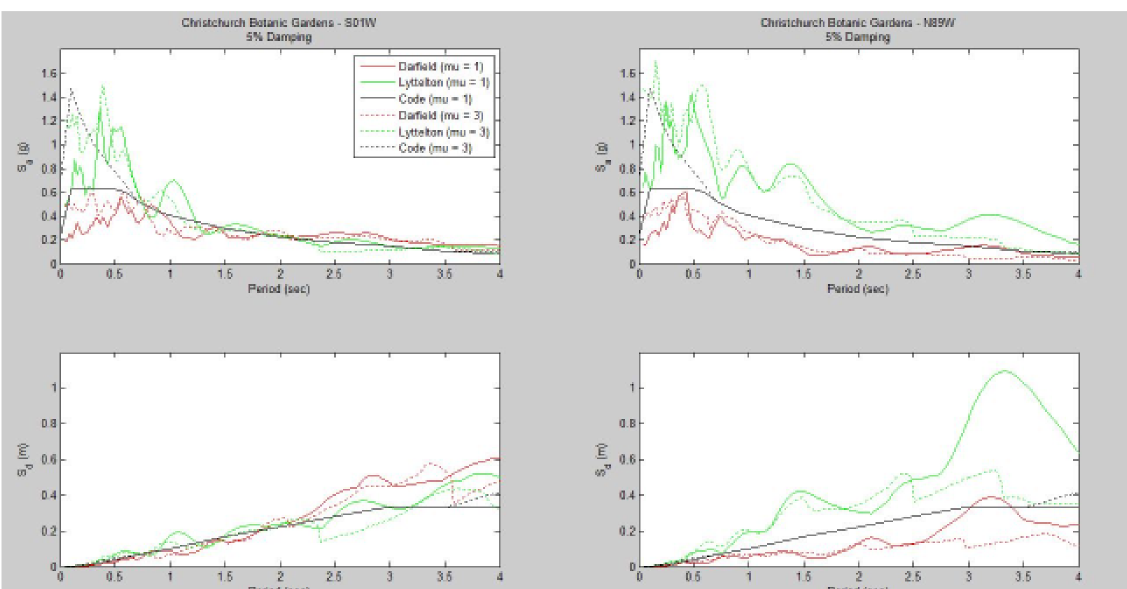

Figure 20: Christchurch Botanic Gardens Station

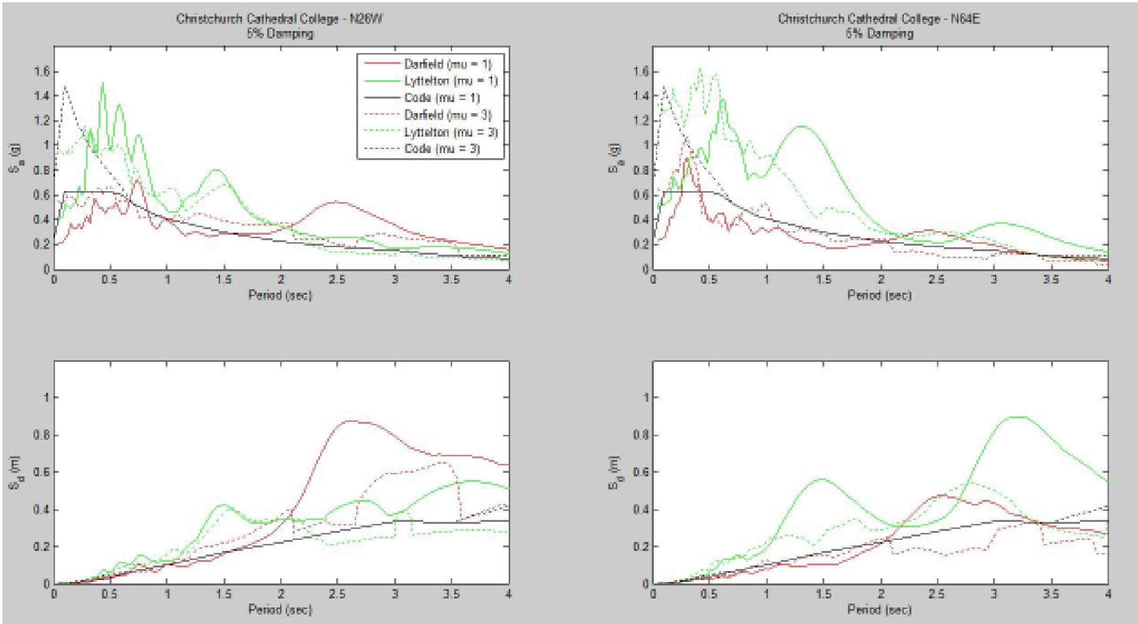

Figure 21: Christchurch Cathedral College Station

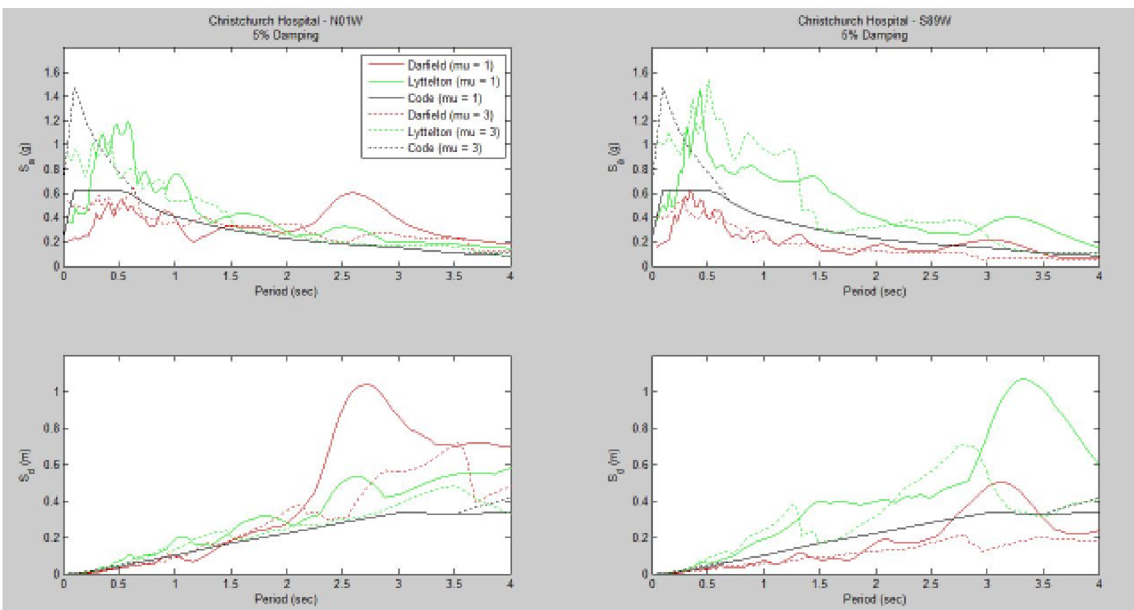

Figure 22: Christchurch Hospital Station

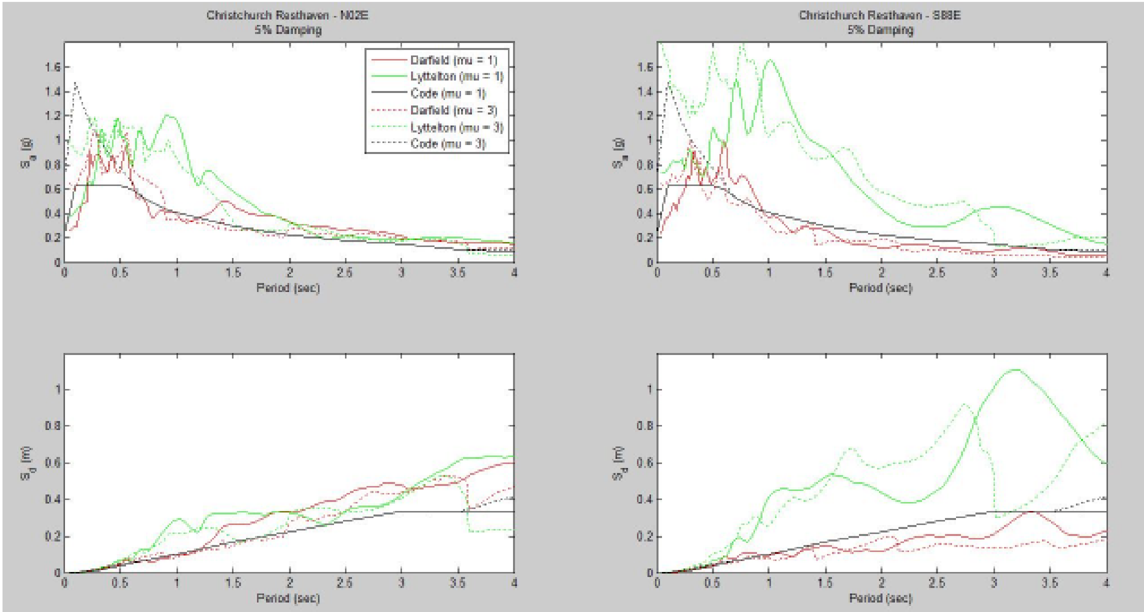

Figure 23: Christchurch Resthaven Station

Interstory Drift Ratios

Using the response spectra in linear response spectrum analysis produces predictions of maximum interstory drift ratios for each of the four frames for eighteen different cases (2 earthquakes x 4 recording stations x 2 ductilities + 2 code spectra). It is observed that the north frame drift ratios exceed those of the south by approximately 5-10%, as can be seen in the following figures, and that they are more uniform over the central stories whereas those of the south tend to concentrate at stories nine and ten. The east and west frames have nearly identical interstory drift ratios except at the lowest several floors. This information, combined with the damage assessments described in earlier sections of this report, motivated a further parametric study where the stiffness of beams at levels four through ten and nine through ten on the north and south frames, respectively, was reduced. Two levels of stiffness reduction are considered. The first reduces the frame beams' shear areas and effective cracked second moment of areas to 80% of their original values. The second reduces them to 60%. In the end, this produced fifty four different combinations (18 cases x 3 stiffness ratios) that are plotted in Figures 24 through 29.

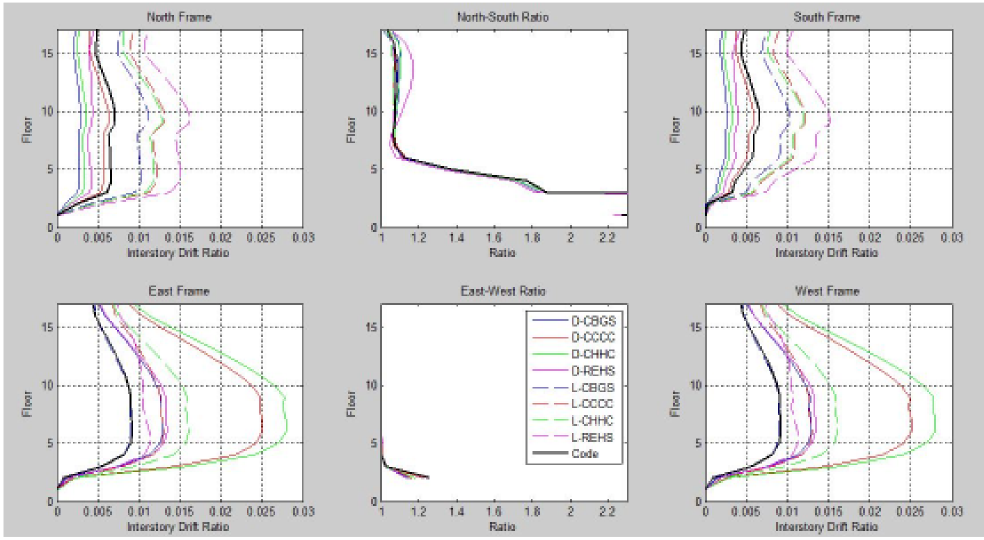

Figure 24: Ductility of 1 and 100% Stiffness

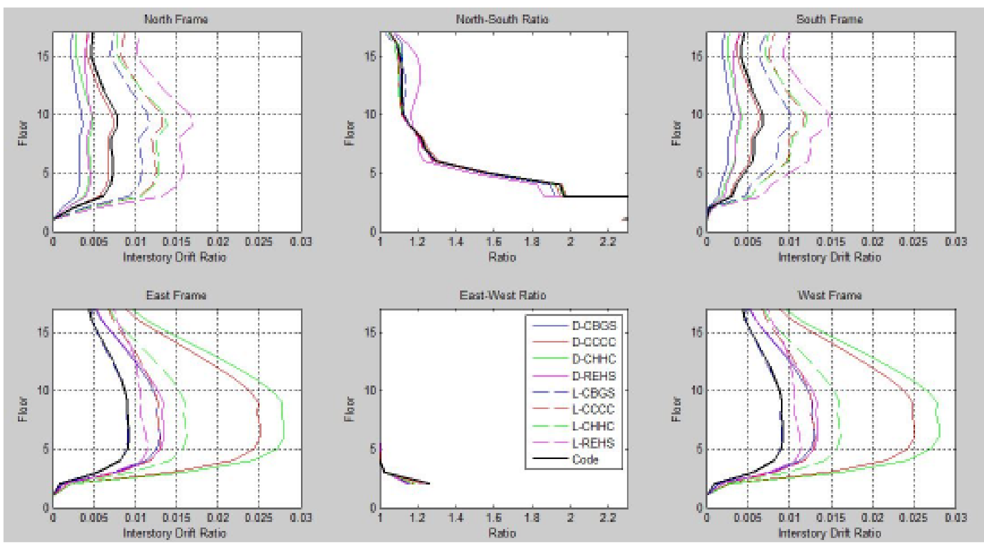

Figure 25: Ductility of 1 and 80% Stiffness

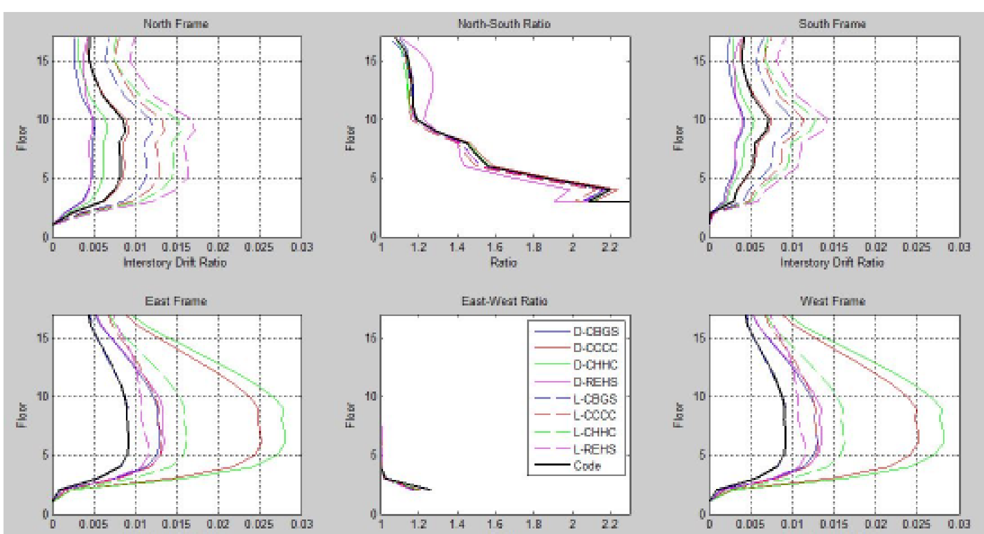

Figure 26: Ductility of 1 and 60% Stiffness

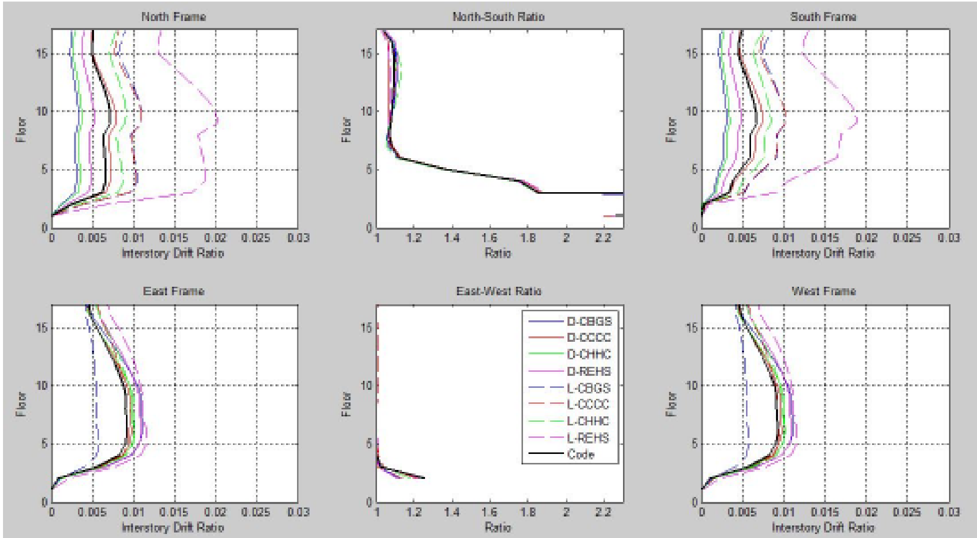

Figure 27: Ductility of 3 and 100% Stiffness

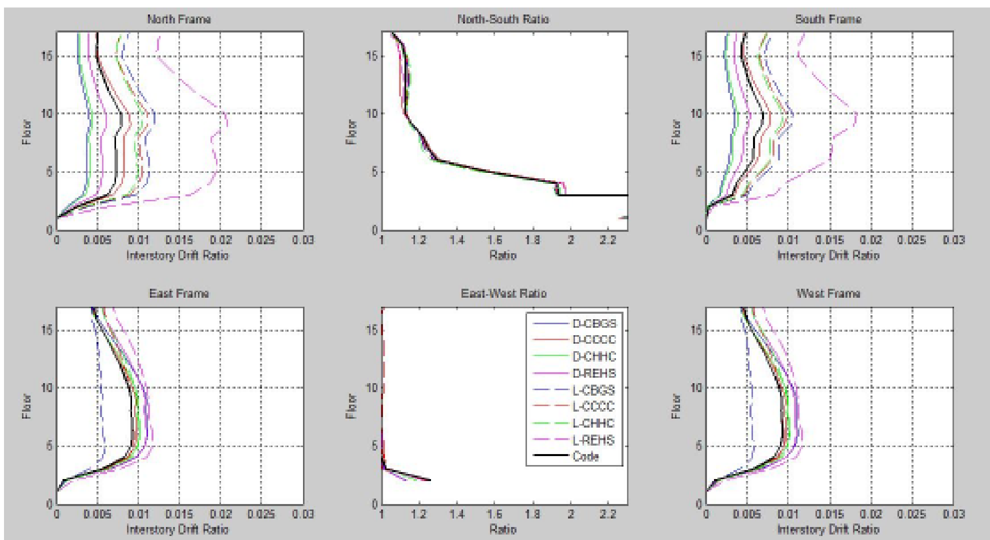

Figure 28: Ductility of 3 and 80% Stiffness

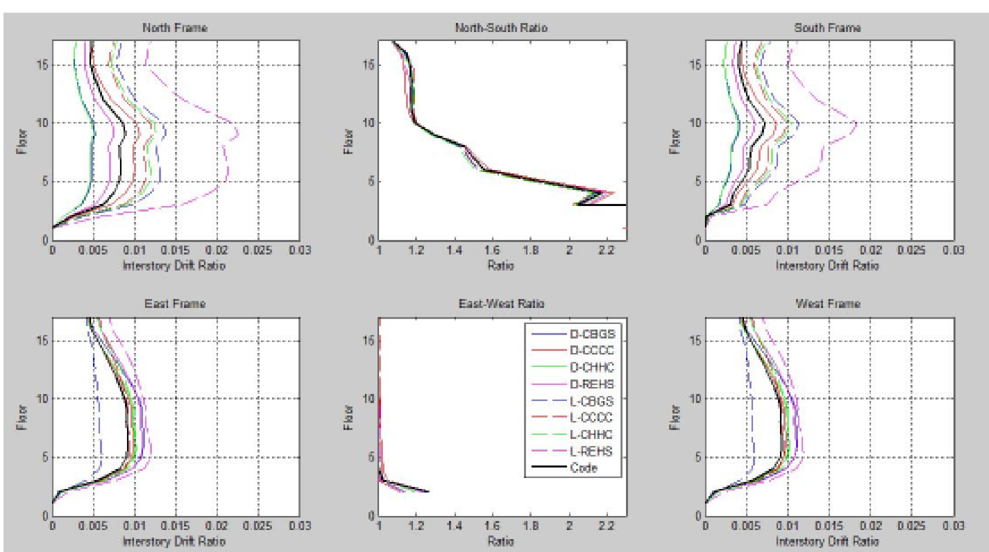

Figure 29: Ductility of 3 and 60% Stiffness

3.3 Discussion & Findings

Elastic Torsion

Although the Clarendon Tower appears to be nearly symmetric, the irregularity of the south frame introduced in the original design to accommodate vehicular movement via the south ramps produces torsion even when the structure remains elastic. This is created through the addition of concrete and block walls and enlarged cast-in-place concrete members. The additional mass from the historic facade and the eccentricity of paper storage on the office floors also contributes slightly. Even ignoring accidental mass eccentricity, as has been done in all analyses in this report, there is a clear coupling between building translation in the transverse direction and torsional excitation. This is expressed as at least a 5-10% difference between the north and south frame interstory drift ratios for all floors. The disparity increases further at the lower levels.

Inelastic Torsion

Although a 5-10% difference in elastic interstory drift ratio would not normally raise such alarm, the non-ductile behavior of the shear link connection on the north and south frames complicates the problem. Instead of maintaining its resistance with plastic displacement, the shear link fails more abruptly near the drift ratios predicted and thus softens fairly rapidly. Additionally, the north frame's more consistent interstory drift ratio distribution with height, compared to that of the south which peaks around floor nine, leads to softening extending over more beams. As can be seen by examining the stiffness parametric study for a constant ductility, softening of the north frame results in generally lower drift ratios of the south frame. Thus early softening of the north frame in a way protects the south frame from a similar fate. For if the south frame were to degrade as significantly as the north, the resulting torsion would decrease substantially. Degradation in north frame stiffness shifts the center of rigidity toward the south frame, ultimately explaining the increasing disparity between north and south frame drifts as is seen by comparing the plots of 60% stiffness to those of 100%.

The hypothesis that the east and west frames would experience greater drifts if the north frame were to soften and the center of rigidity were to shift toward the south does not appear to be correct. Instead the interstory drift ratios of the east and west frames remain fairly insensitive to the north versus south frame stiffness as shown in the parametric study. This can be contextualized by understanding that as the north frame softens and the center of rigidity shifts south, the lever arm between the north frame and the center of rigidity increases. So even though the north frame drifts increase, the distance over which these displacements are divided also increase. Thus the diaphragm rotation may remain fairly constant and thus displacements imposed by torsion at the east and west frames will also.

Although reference has been made in the site reports of crack sizes increasing with aftershocks subsequent to the Lyttelton event, it is not clear that this can be explained without extensive nonlinear analysis. Most importantly, the state of the diaphragm during these aftershocks is considerably deteriorated and conventional modeling assuming a rigid constraint is unlikely to be adequate.

Recording Station Differences

Less relevant to the performance of the Clarendon Tower but nonetheless highly interesting is the widely varying response predicted depending on the ductility and station acceleration record chosen. Although it is well documented for the Canterbury earthquake sequence that the equal displacement rule is a poor predictor of inelastic spectra, the Resthaven station under the Lyttelton shaking shows a dramatic change in expected drifts for the north and south frames depending on whether you use an elastic or inelastic spectrum [46]. The Christchurch Cathedral College and Christchurch Hospital under the Darfield shaking show similar extreme changes in the east and west frames.

4. Conclusion

4.1 Summary

The Clarendon Tower suffered fairly significant damage to its thin concrete topping slab, east and west frame beams, and north and south shear link connections during the Darfield earthquake, Lyttelton earthquake and subsequent aftershocks. Nonstructural elements such as the precast cladding, historic facade and precast stairs also incurred damage with one flight of stairs completely collapsing over several stories. Much of the observed damage has been attributed to generally strong shaking experienced at the site, frame elongation and torsional excitation.

This report captures both the torsional excitation and strong shaking effects through accurate modeling of the building geometry, parametric analysis of frame beam stiffness and by utilizing elastic and inelastic response spectra from the four recording stations in the Central Business District of Christchurch. The issue of frame elongation, other than to highlight its obvious effect on precast floor seating and cracking of the concrete topping slab, has not been investigated thoroughly. The major conclusions from the analysis results are that:

- Torsion is predicted in the elastic range of structure response owing to irregularity in the south frame.
- The north, and to a much lesser extent, south frame softening results from the elastic torsion combined with the non-ductile nature of the shear link connection detail.
- Increased north frame flexibility generally reduces south frame interstory drift ratios, except at a few concentrated stories, essentially protecting much of the south frame and keeping it from softening to match the north. This result is explainable by understanding that although the roof displacement demand increases slightly, the shift in the center of rigidity permits the same center of mass displacement for a lower south frame drift.

4.2 Recommendations for Future Study

Assuming that more data can be collected on the Clarendon Tower before demolition, especially more detailed mapping of where and to what degree the north and south frame shear links have been damaged and better quantitative measurement of frame elongation effects, it would be highly interesting to attempt at capturing more of the structure's response. This would almost exclusively necessitate nonlinear time history analyses with each recorded earthquake applied in succession. An ideal structural model would also include elongating hinges on all four frames and somehow account for diaphragm nonlinearity and damage.

The issue of sensitivity to which station record is chosen and whether this can be described by differences in the subsurface profile also requires attention. Installation of inexpensive motion recorders at the site could possibly indicate, by comparison with the existing stations, how motions at this site may have differed. Furthermore installation of motion recorders within the structure could also yield technically valuable information about the structural response.

5. References

[1] Holmes Consulting Group. *Rapid Structural Assessment Site Report*. September 4, 2010.

[2] Holmes Consulting Group. *Rapid Structural Assessment Site Report*. September 7, 2010.

[3] Goleman Exterior Building Care. *Clarendon Tower Exterior Observational Building Report*. September 15, 2010.

[4] Holmes Consulting Group. *Site Report*. October 18, 2010.

[5] Holmes Consulting Group. *Site Report*. October 26, 2010.

[6] Holmes Consulting Group. *Site Report*. November 2, 2010.

[7] Holmes Consulting Group. *Site Report*. November 9, 2010.

[8] Holmes Consulting Group. *Site Report*. November 15, 2010.

[9] Holmes Consulting Group. *Site Report*. November 18, 2010.

[10] Holmes Consulting Group. *Site Report*. November 19, 2010.

[11] Holmes Consulting Group. *Site Report*. November 23, 2010.

[12] Holmes Consulting Group. *Site Report*. November 25, 2010.

[13] Holmes Consulting Group. *Site Report*. November 29, 2010.

[14] Holmes Consulting Group. *Site Report*. November 30, 2010.

[15] Holmes Consulting Group. *Site Report*. December 2, 2010.

[16] Holmes Consulting Group. *Site Report*. December 16, 2010.

[17] Holmes Consulting Group. *Site Report*. January 13, 2011.

[18] Holmes Consulting Group. *Site Report*. January 17, 2011.

[19] Holmes Consulting Group. *Site Report*. January 21, 2011.

[20] Holmes Consulting Group. *Site Report*. January 27, 2011.

[21] Holmes Consulting Group. *Site Report*. January 28, 2011.

[22] Holmes Consulting Group. *Site Report*. February 2, 2011.

[23] Nixon, Alan. *Engineers Re-inspection of Damaged Buildings*. Christchurch City Council. February 8, 2011.

[24] Holmes Consulting Group. *Site Report*. February 15, 2011.

[25] Holmes Consulting Group. *Site Report*. February 16, 2011.

[26] Holmes Consulting Group. *Rapid Structural Assessment Site Report*. February 27, 2011.

[27] Priestley, Nigel. *Response to David Hopkins Email*. March 17, 2011.

[28] Holmes Consulting Group. *Preliminary Structural Review Report*. Prepared for Hawkins Construction Ltd. April 4, 2011.

[29] Holmes Consulting Group. *Structural Update*. Prepared for Hawkins Construction Ltd. April 26, 2011.

[30] Holmes Consulting Group. *Structural Update*. Prepared for Hawkins Construction Ltd. May 20, 2011.

[31] Bull, Desmond. The *Performance of Concrete Structures in the Canterbury Earthquakes*. May 31, 2011.

[32] Holmes Consulting Group. *Presentation to the Insurers*. May 31, 2011.

[33] Holmes Consulting Group. *Structural Damage Assessment Update*. Prepared for Hawkins Construction Ltd. June 14, 2011.

[34] CAL Engineering Management. *Initial Report on Clarendon Tower, Christchurch*. July 11, 2011.

[35] Holmes Consulting Group. *Structural Damage Assessment Update*. Prepared for Hawkins Construction Ltd. August 10, 2011.

[36] Fenwick, Richard. *Calculation Set 1*. January 24, 2011.

[37] Fenwick, Richard. *Calculation Set 2*. September 7, 2011.

[38] Soils & Foundations Ltd. *Preliminary Site Investigation Report*. Prepared for Paynter Developments Ltd. December 23, 1986.

[39] Holmes Wood Poole & Johnstone. *Clarendon Hotel Project Memo*. Prepared for Mr. B. Bluck, City Engineer, Christchurch City Council. February 2, 1987.

[40] Hawkins Construction. *Communication*. Sent to Wayne Roden of Christchurch City Council. October 7, 2010.

[41] Holmes Consulting Group. *Structural Specifications*. Repair of Darfield Earthquake Damage. September 8, 2010.

[42] Hawkins Construction. *Temporary Access Plan*. May 5, 2011.

[43] Holmes Consulting Group. *Clarendon Emergency Access for Tenants*. May 12, 2011.

[44] Standards New Zealand (2004a). *NZS 1170.5:2004 - Structural design actions. Earthquake actions*. Wellington, New Zealand.

[45] Restrepo, J.I., Park, R., and Buchanan, A.H. *Tests on Connections of Earthquake Resisting Precast Reinforced Concrete Perimeter Frames of Buildings*. PCI Journal, V. 40, No. 4, July-August 1995, p44-61.

[46] Carr, Athol J. *Inelastic Response Spectra for the Christchurch Earthquake Records*. Report to the Canterbury Earthquakes Royal Commission. August 8, 2011.

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