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CRITICAL STRUCTURAL WEAKNESSES EXPOSED IN OLDER RC WALL-FRAME STRUCTURES DURING THE CHRISTCHURCH EARTHQUAKES

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Abstract

This study seeks to identify so-called "critical structural weaknesses" (CSWs) in older reinforced concrete (RC) wall-frame structures. These CSWs are defined as features of the building that greatly increase collapse potential, as evidenced by seismically-induced structural damage and building collapse around the world. In particular, in Christchurch, New Zealand, a significant series of earthquakes in 2010-2011 caused widespread damage and devastating collapses to this particular class of building. The motivation for identifying CSWs is to develop a tool that can be used to quickly single out the most dangerous of these buildings, from a large pool of similar buildings. The development of the CSW concept is described here in three parts. First, we identify a set of case study buildings damaged in the Christchurch earthquake and make the case that they are representative of a larger group of buildings with similar characteristics New Zealand and other countries. Next, a list of CSWs is presented. This list is generated from the case study buildings, but also from a larger body of research that evaluates the performance of buildings in the Christchurch earthquakes. These CSWs are all observable and do not require significant engineering calculations. This list is transformed into a CSW scoring tool, which provides a qualitative way of quickly and robustly identifying dangerous buildings, eliminating the need for detailed structural analysis. One particular case study building, the Securities House, is used as an example to lead the engineer through the scoring process and to show how damage and CSWs are related.

Keywords: Non-ductile reinforced concrete buildings; shear walls; frame buildings; seismic damage

1. Introduction

In the February, 2011 Christchurch earthquake in New Zealand, the two most devastating building collapses, in terms of loss of life, were the Canterbury Television Building (CTV) and the Pyne Gould Corporation (PGC). Both of these structures were reinforced concrete (RC) wall-frame (dual system) structures, which are a specific subset of RC structures that represent one of the largest seismic safety concerns worldwide [1]. After the Christchurch earthquake sequence, there was significant examination of the structural performance of these two structures because of the loss of 133 lives [2], including multiple Royal Commissions' Reports. However, there has been substantially less inquiry into structures that experienced less egregious performance. In fact, the cluster of seismic events led to an estimated 1,300 of the 3,000 structures in the Christchurch Central Business District (CBD) being demolished [3], including approximately 60% of the multi-story RC building inventory. Some buildings were demolished due to severe damage. In other cases the demolition occurred primarily because of insurance company policies and concerns about repair to meet and new stringent building codes [4].

An unknown percentage of pre-1980 RC structures in New Zealand, the U.S. and other countries with advanced seismic codes, may be at risk for collapse in an earthquake event due to a variety of failure modes not adequately addressed in design standards in older building codes [1]. Older buildings are of particular interest because their designs were based on linear static analysis and lack of capacity design thinking, both of which may lead to brittle failure modes [5]. These design deficiencies can result in poor seismic performance or collapse, as demonstrated by major earthquakes in dense urban areas around the world such as the Northridge, Christchurch, Chile, and Kobe earthquakes [1]. This paper examines the performance of RC wall-frame buildings that experienced excessive damage in the Christchurch earthquake sequence with a focus on a larger set of buildings than previously explored to identify critical structural weaknesses (CSWs) that are indicators of collapse risk. Case studies were selected after investigation into reported building damages that were available in the forms of Level 1 and Level 2 rapid assessment damage reports and Detailed Engineering Evaluations (DEEs), all of which were compiled by New Zealand's Institute of Geological and Nuclear Sciences (GNS Science). Each case study structure is a pre-1986 RC building that exhibits a variety of CSWs. This paper operates on the premise that it is not a single CSW that makes a building especially dangerous, instead the risk of collapse increases robustly with the accumulation of CSWs. Due to similarities between construction in New Zealand, the U.S. and many other countries, the CSWs identified are applicable to identification of potentially dangerous older RC structures in New Zealand and beyond.

2. Research Methods

This study began by investigating the information available in DEE reports. DEE reports are post-disaster evaluations used in New Zealand, which are conducted by professional engineers. They are comprised of images and documentation concerning building damages from the earthquake and generally provide a structural analysis, which is often in the form of a NBS rating [7]. A New Building Standard (NBS) rating is a measure of the inherent strength of older buildings in relation to an equivalent building built to modern standards. However, we learned upon further investigation that only 30% of Christchurch buildings in the CBD had been subjected to DEEs, and most of these buildings did not fall into the category of buildings of interest in this research. This discrepancy occurs because DEE reporting did not begin until months after the initial earthquake, and was discontinued after about a year as they were deemed not helpful towards the recovery program [4]. Moreover, many of the buildings that were condemned to be demolished did not undergo evaluation. As a result, Level 1 and Level 2 evaluations conducted by the Christchurch City Council (CCC), along with Royal Commissions Reports and various other research papers, became the most important sources for documented building damage. Level 1 and Level 2 evaluations are rapid assessment tools that assess the apparent danger of structures based solely on a visual observation of damages by an engineer [8].

After investigating building damage, we also collected and reviewed building plans to identify CSWs apparent in the design documentation. After several case studies were selected based on structure type and the descriptions of damage, building plans were purchased from CCC. Ultimately, more information was collected and researched than was possible to include in this paper, so only one of the case study buildings, the Securities House, is discussed in detail. The rest of the case studies are evaluated using the same methods as Securities House and building damage and CSWs are summarized here. A complete documentation of all of the buildings will be available in a report [9].

3. Case Study Buildings

Table 1 compiles a list of the selected case study buildings used for this research. Each building is between 4 and 8 stories tall. Each uses RC cast in-situ shear wall(s) and beam-column system together to resist lateral forces and gravity loads, constituting a so-called "dual system". None of the case studies has pre-fabricated columns, beams, or floor systems to ensure applicability of the findings to other countries where pre-fabricated systems are less common than in New Zealand. All of the structures were built before 1986, and designed according to older codes that do not explicitly prevent brittle failure modes such as column shear failure [1]. In addition, two critical sources of information were needed for the buildings to be included in the study. First, all buildings needed to have credible damage reports. In addition, we needed to be able to access original building plans in order to link recorded field damages to observable CSWs. The "CSW score" shown in the table reports the total number of CSWs that the authors observed in each structure through review of the building plans. Detailed discussion about the CSW score is provided in the following section.

Table 1 - Case study buildings.

ID	Building Name	Address	Year Built	End-of-Life	# of Stories	CSW Score
A	Securities House	221 Gloucester St.	1974	Demolished	8	14
B	AMI House	29 Latimer Sq.	1967	Demolished	7	7
C	Canterbury Television Building	249 Madras St.	1986	Collapsed	6	13
D	Bradley Nuttall House	79 Cambridge Ter.	1985	Demolished	7	8
E	TVNZ	202 Gloucester St.	1929	Demolished	4	13
F	Harcourts Grenadier	271 Madras St.	1961	Collapsed	5	13
G	Harvey Cameron	1/93 Cambridge Ter.	1960	Demolished	5	5
H	Pyne Gould Corporation	233 Cambridge Ter.	1963	Collapsed	5	13
I	Hotel SO	165 Cashel St.	1980	Rehabilitated	6	5

RC buildings in the height range of 4 to 8 stories tended to incur the greatest damage in the Christchurch earthquake, due to unique characteristics of the ground motions recorded from the significant February 2011 Christchurch event [11]. Fig. 1 illustrates these characteristics by showing the average ground motion spectra (plotted in spectral acceleration-spectral displacement space) from the four recording stations located in the CBD. These spectra are then compared to the 2002 NZS 1170 code-based demand at the maximum considered earthquake (MCE) level. The red-shaded area represents the range of periods for structures that would experience demands greater than the 2002 design requirements [11]. All buildings selected for this research are thought to have natural periods such that they fall somewhere within the red region; the impact of selecting buildings in this range is that we are able examine how buildings perform when pushed beyond design specifications. Studying the subtle differences between buildings that performed well and buildings that performed poorly offers insight into identification of the structures that have the highest collapse risk.

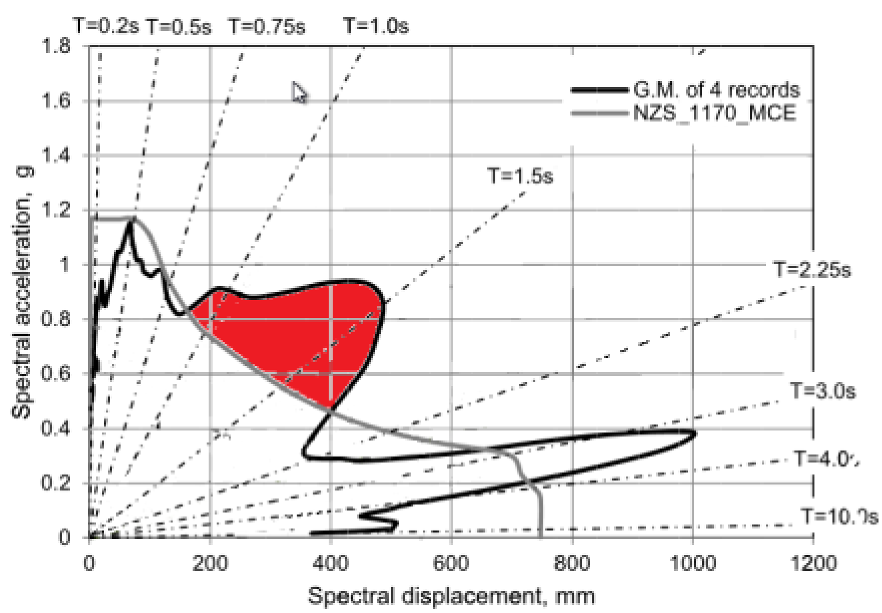

Fig. 1 - Spectral displacements and accelerations recording during February 2011 Christchurch event compared to 2002 design specifications (“NZS_1170_MCE”). The red region indicates the range in which the recordings exceeded design. Source: Adapted from [11]

All structures are located centrally in the CBD of Christchurch as shown in Fig. 2. The implication of the structures being located close to each other is that they would have experienced similar shaking intensities and were built upon similar soils [8]. However, some of these structures that are located close to the river experienced observable liquefaction near the foundation, while the others that are farther away from the river did not have observable liquefaction [2]. Other forms of liquefaction evidence, such as leaning buildings, would suggest that buildings close to the liquefied areas in the map may have experienced subsurface liquefaction. Since the bulk of case study buildings are located somewhere between the PGC (H marker) and CTV (C marker) buildings, in a relatively small region, we expect that the effects of liquefaction and soil type are relatively similar for all the structures.

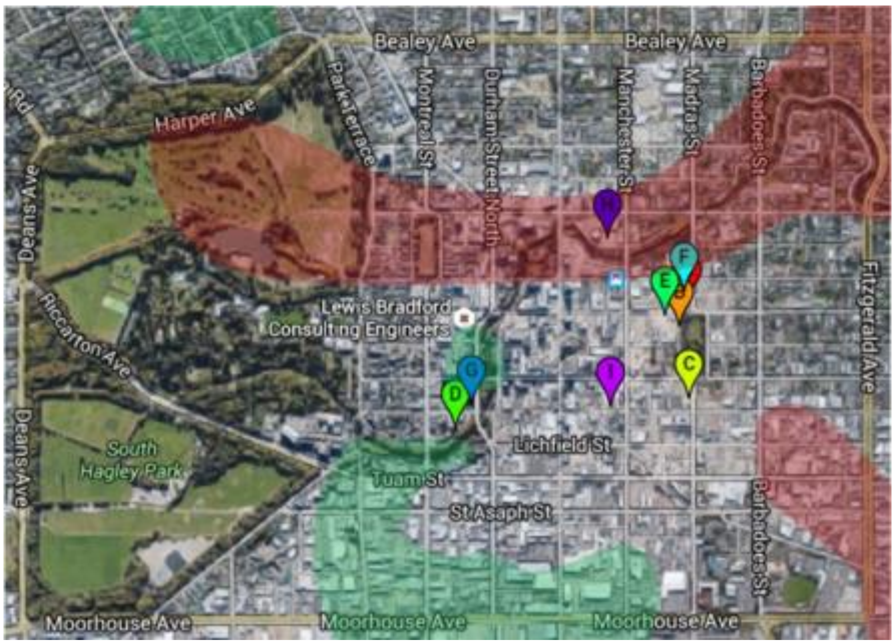

Fig. 2 - Case study building locations, with respect to areas of liquefied soil in Christchurch CBD. Red area = moderate to severe liquefaction; green area = low to moderate liquefaction. Source: Adapted from [12]

4. Observable Critical Structural Weaknesses

The CSWs discussed in this paper are structural characteristics that are known to lead to poor seismic performance or undesirable failure modes. Some CSWs represent differences between reinforcing details of structural members in older buildings, compared to modern reinforcement detailing used in newer buildings. Unlike older design codes, modern design specifications define a hierarchy of failure modes, preventing brittle undesired failure modes [1]. Other CSWs reflect architectural irregularities such as soft stories and non-symmetric building layouts. This paper focuses on observable CSWs in a structure that can be identified by close examination of the building plans, and excludes CSWs that require detailed calculations to identify.

Table 2 summarizes the various observable CSWs found to be common to older RC dual system structures in Christchurch. It is expected that older buildings in the United States and other parts of the world share many of these attributes. The information in the table was compiled from a literature review of Royal Commissions Reports, DEEs, Level 1 and Level 2 Rapid Assessment Reports, Christchurch City Council Property Files, and University of Canterbury student reports. These sources all provided details of the structural damage experienced by each case study structure, and offered insights into the design deficiencies that led to those specific damages.

Table 2 - Observed Critical Structural Weaknesses (CSWs) in Christchurch.

Global Critical Structural Weaknesses, Observable from Floor and Elevation Plans		
#	**Critical Structural Weakness**	**How to Identify**
G1	Torsional Irregularity from Location of Walls or Core Section	When walls or a core section of a building are not located symmetrically in one or both cardinal directions, torsion is generated. Infill walls and other architectural components may be less obvious factors of torsion. An eccentricity between centers of mass and rigidity is discernible in most structures without detailed calculation.
G2	Narrow Core Section	Walled core sections of buildings are frequently square or rectangular in plan. Core sections that have a large aspect ratio (e.g. exceeding a 2 to 1 length to width ratio) are considered narrow and may have worse performance because of a lack of torsional stiffness.
G3	Non-Redundant Loading Paths for Lateral Loads	Redundancy is a well-known concept that has to do with the presence of alternate load paths. In this study, we consider a structure that has a single core wall section, and columns to support gravity loads, to be non-redundant because the non-redundant wall takes the majority of the seismic load, despite the ability of the columns to absorb some of the lateral demand.
G4	Indirect or Incomplete Load Path between Wall and Ground	G4-G6 all address irregularities in elevation or vertical irregularities. G4 deals with discontinuities between the wall and the foundation. Walls ending before the 1st floor are easy to identify, and create a soft or weak story. Less apparent is the situation where subgrade foundation beams are prone to failure because the wall configuration creates a large bending moment in the center of the grade beam.
G5	Discontinuous Wall from Ground to Roof	Another vertical irregularity exists when walls are discontinuous at upper stories. This can create a change in strength and/or stiffness causing a weak story and can lead to failures in upper levels. This CSW should be considered for any structure with walls that do not extend fully from the base of the structure to the roof.
G6	Vertical Stiffness Irregularity	The vertical stiffness irregularity deals with any story that is substantially taller than another story, or a story that is inset/cantilevered over another story, which can create a soft or weak story.
G7	Infills, Cladding, or Deep Spandrel Beams between Columns	Structural or architectural features that span between columns may incidentally create short columns because of inadequate gapping between the infill and the columns that will not allow for large displacements caused by large earthquakes and changes the force demand on the columns.
Local Observable Critical Structural Weaknesses from Plan Details		
L1	Walls with Single Layer of Reinforcement	Walls with single layers of reinforcement tend to be more prone to buckling. Wall detailing, especially at the 1st and 2nd levels, provides information as to whether there are (typically) one or two rows of reinforcement specified.
L2	Lack of Boundary Confining Reinforcement Detailing in Walls	Walls that lack highly detailed confined at the ends of the walls or in plastic regions at lower levels (i.e. boundary regions) may be vulnerable to buckling or crushing failure. A wall sandwiched between two columns is assumed to have end confinement.
L3	Lack of Strong Columns Relative to Beams	This CSW addresses whether failures will begin in columns or beams. Structures that do not satisfy the “strong column weak beam” principle can be distinguished from structures that do based on engineering judgment and the size of the beams in relation to the size of the columns in lieu of more detailed calculation.
L4	Column Tie Spacing greater than ~6in	Older structures may have large spacing in column ties, which can cause columns to be shear critical.
L5	Discontinuous Tie and Stirrup Detailing throughout Joints	Column ties and stirrups should pass continuously through the joint to ensure that the joint strength will surpass the column and beam strength. Joint failure can create a story mechanism even if the columns remain intact.

This paper describes a CSW scoring system for RC wall-frame buildings, and the procedure's scoring details are provided in Table 3. The procedure works by identifying which of the CSWs in Table 3 are present in the building. Sub-scores for a given CSW vary from 1 to 2 points depending on the authors' judged severity of a CSW toward global performance. The sub-scores are then added together to get a building CSW score. Under this system, the buildings earning the highest CSW scores, identified in Table 1, exhibited a number of different CSWs. The intent of the procedure is to allow engineers to conduct rapid visual assessments of wall-frame structures and determine whether they pose a high risk of collapse due to the presence of excessive CSWs. The idea is that a high CSW score could be used to prioritize rehabilitation and precede a more detailed quantitative assessment.

Identifying some of these CSWs can be subjective and requires sensible engineering judgment. For example, identifying beams that will fail before columns may not be possible without detailed strength calculations. Engineering judgment will determine if the beam spans seem long and the beams seem to possess a greater bending moment capacity than their adjoining columns. Some cases will be obvious as either yes or no, and many cases will reside within the grey area, providing qualitative guidance. However, there is some variability as scoring could vary according to engineer's experience and confidence. If in doubt, an engineer can conduct some representative calculations or favor the conservative answer. Some guidelines on how to identify some of these CSW are described in more detail in subsequent sections using Securities House as an example.

Table 3 - Critical structural weakness (CSW) assessment tool.

		Sub-Scores			**Example Building**		
#	**Critical Structural Weakness**	**0**	**1**	**2**	**Securities House**		
G1	Torsional Irregularity from Location of Walls or Core Section	No	1 Dir	2 Dir		1	
G2	Narrow Core Section	No	Yes	-	0		
G3	Non-Redundant Loading Paths for Lateral Loads	No	-	Yes			2
G4	Indirect or Incomplete Load Path from Wall to Ground	No	Yes	-	0		
G5	Discontinuous Wall from Ground to Roof	No	Yes	-	0		
G6	Vertical Stiffness Irregularity	No	-	Yes			2
G7	Infills, Cladding, or Deep Spandrel Beams between Columns	No	-	Yes			2
L1	Walls with Single Layer of Reinforcement	No	-	Yes	0		
L2	Lack of Confining Reinforcement Detailing in Walls	No	Yes	-		1	
L3	Lack of Strong Columns Relative to Beams	No	-	Yes			2
L4	Column Tie Spacing greater than ~150mm	No	-	Yes			2
L5	Discontinuous Tie and Stirrup Detailing Throughout Joints	No	-	Yes			2
	Total CSW Score		**?**			**14**	

Fig. 3 shows the trend between the CSW score in each of the case study structures and the damage level quantified in terms of the placard issued during the Level 2 rapid assessment [10]. Level 2 placards either green yellow or red and within each category are numbered from 1-3 [8]. Least squares linear regression results suggest that buildings scoring nine or above would have the highest collapse risk, while buildings scoring below nine would have a lower collapse risk. These results indicate that the risk of collapse is increasing with an increasing number of CSWs with R^2 value of 0.82. In addition, the findings support reported structural damage in Christchurch, which indicated that no structure experienced collapse or was damaged beyond repair due to any single design weakness. Since placards were issued based on observational judgement of the engineer and not on a quantitative measure, then there will be sensitivity between the actual damage level of the buildings and the placard score. Likewise, testing the procedure and plotting the results using a much larger pool of buildings would help to quantify, as well as potentially reduce, the uncertainty between the CSW score and the damage level a building might experience.

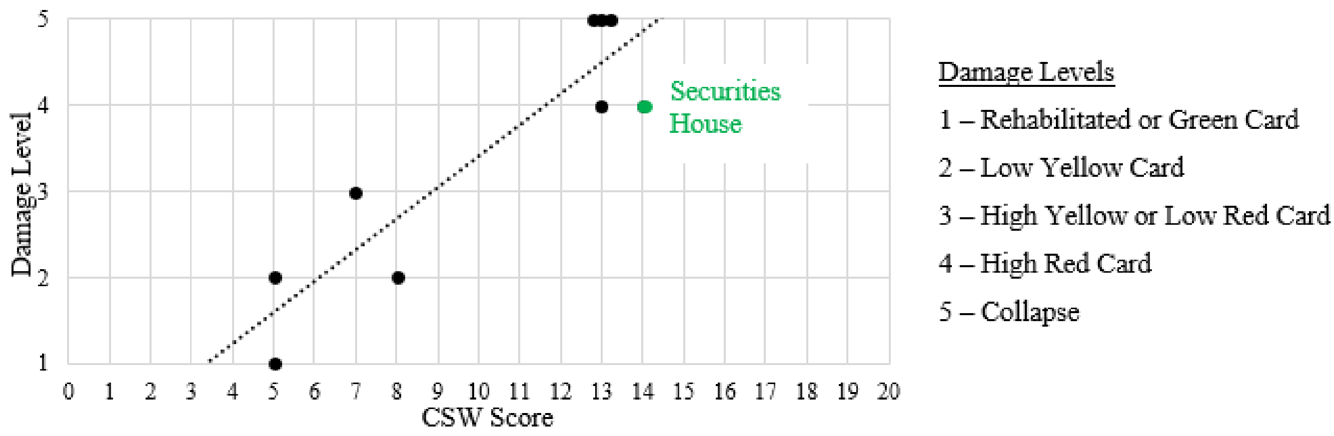

Fig. 3 - Trend between number of CSWs and building damage. Each black dot represents the CSW score of each case study building and the dotted line is the linear trend-line obtained.

5. Detailed Critical Structural Weakness Evaluation for the Securities House

The Securities House was an 8-story office building, first built in 1974 then demolished in 2012 due to multiple structural failures in the Christchurch earthquake sequence. This RC wall-frame structure had a 3500 sq. ft. footprint, and though seemingly simple and basic in design, exhibited many CSWs, making this structure an excellent archetype for this research. Though not indicated by the plans, the building was likely designed using either ACI 318-71, which was common to do in New Zealand at that time, or New Zealand's building Warrants of Fitness (WOF), published by the Ministry of Works [6]. Both codes were similar in content and the structure is representative of buildings that may be found in New Zealand or the U.S. from that era.

5.1 Overview of Damage

The engineer conducting the Level 2 damage assessment noted that it was not possible to enter the structure because the infill around the entry lobby had collapsed. Moreover, the engineer immediately deemed the structure unsafe to enter and unstable, because it appeared that the ability to resist gravity loads had been considerably reduced by earthquake damage [10]. A DEE was never completed, because the entire structure was slated for demolition based solely on the Level 2 visual assessment [4]. Still, damages that could be seen from the exterior of the structure were documented, and provide sufficient information to link damaged components to building plan details.

The perimeter frames sustained heavy damage, particularly at the beam column joints. In addition, there was significant cracking and spalling to columns on the northern perimeter (see Fig. 4) [10]. The engineer that assessed the damages suggested that the structure appeared to have "twisted vigorously" around the shear core, leading to perimeter frame damage; the engineer believed the building to be near collapse. In addition, the building had developed a lean towards the south-west, which would suggest differential settlement of the foundation, and made salvaging the structure impractical [10].

Fig. 4 shows an image of typical column shear failure on the north side of the structure. The shear failure happened as a result of the contact between the column and the cladding when extreme torsion caused excessive displacements at the perimeter farthest from the center of rigidity [2]. Shear failure in the columns suggests that there was inadequate detailing at the point where the column made contact with the cladding panels. Along with the column shear failure, the image also shows joint failure above the column. Joint failure suggests inadequate detailing through the beam-column connection.

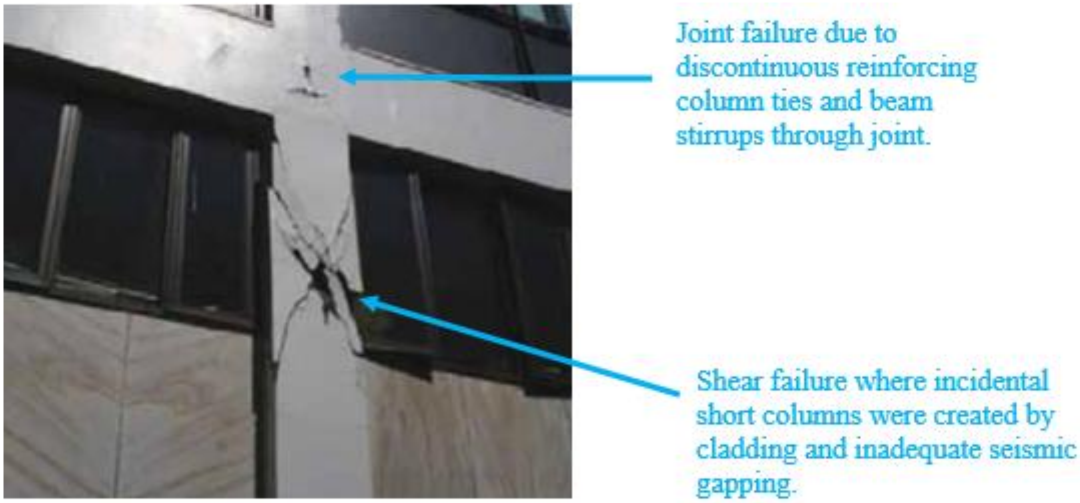

Fig. 4 - Column and joint damage on north side of Securities House. Source: [2], and annotated by authors.

5.2 Global Critical Structural Weaknesses

Securities House has an apparent soft-story vertical stiffness irregularity (G6, following the CSW numbering scheme in Table 2) at the first level, which is evident in Fig. 5. The columns at the 1st story are taller than the upper level columns and in addition the entire ground level is open, which makes this a CSW because of the vertical discontinuity in stiffness and, likely, strength. It would take detailed strength calculations based on the member detailing in the plans to confirm this, but qualitative engineering judgment can easily identify this weakness.

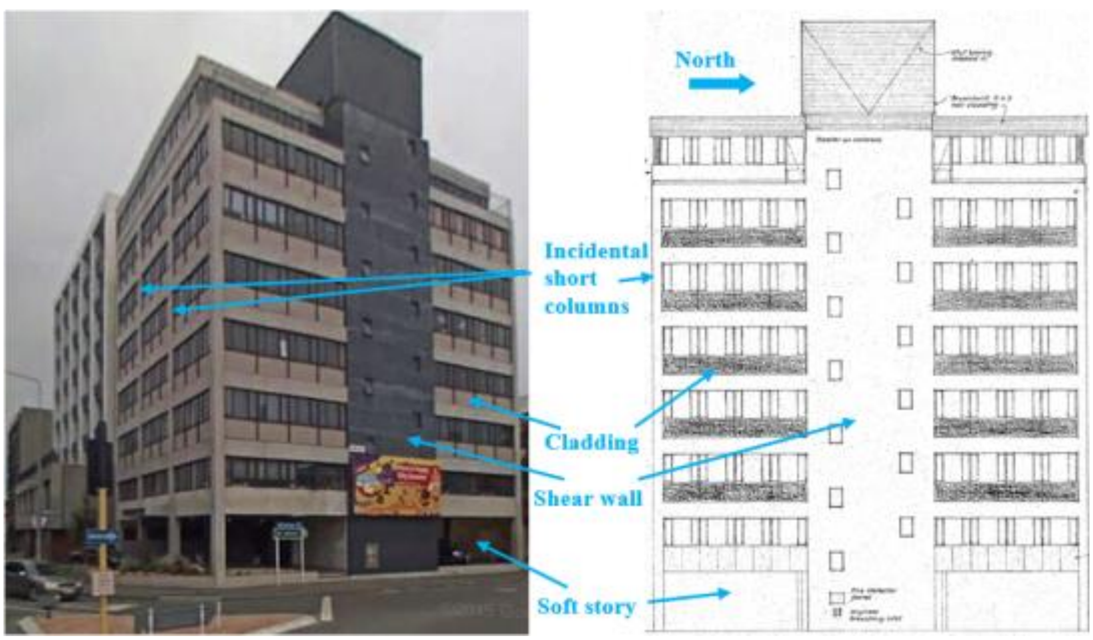

Fig. 5 - Elevation of the Securities House. Sources: left image google maps, right image building plans [15], annotated by the authors.

A common (G1) CSW exhibited by many of the most severely damaged Christchurch buildings is torsional eccentricity caused by nonsymmetrical wall layout. The shear wall core of Securities House is located to the far east of the building as indicated in Fig. 6. Reasonable engineering judgment locates the center of stiffness offset from the center of mass, because of nonsymmetrical wall layout in one direction. During a seismic event, rotation of mass around the center of rigidity will lead to the largest displacements at the north and west walls [13]. This conclusion comes as no surprise and is supported by the Level 2 damage report which states that the building appeared to have "twisted vigorously" around the shear core and that most of the column damage was on the north and west side [10].

The core section is not quite square, but does not qualify for a G2 CSW (narrow core). Since the configuration of the core section is a C then there is only a single wall in the North-South direction to resolve the lateral force. Furthermore, despite the inherent ability for the beam-columns connections to resist moment forces, the single shear wall is the primary lateral force resisting system for the entire structure in one direction. This is the definition of a non-redundant system and leads to the identification of a non-redundant G3 CSW for this structure.

It is important to examine the load paths through shear wall. The elevation view in Fig. 5 shows that there are not major discontinuities along the height. Had there been a discontinuity it would create a vertical stiffness irregularity or a weak story G5 CSW. Likewise, the loading path through the wall goes directly into the piers below. Some of the case study buildings had loads from the walls above transfer into foundation beams which then had to divert the load to piers. These foundation beams failed and, as a result, the loading path from the wall to the foundation is an important (G4) CSW to check.

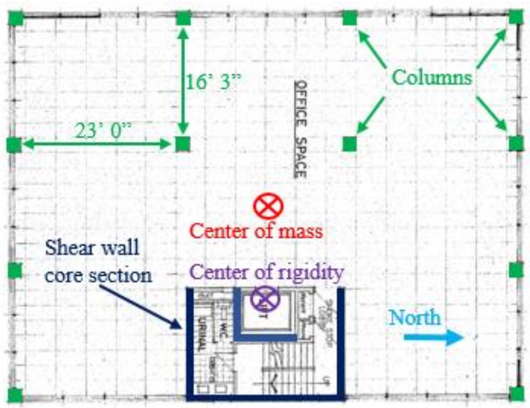

Fig. 6 - Plan view of Securities House. Source: Building plans [15], annotated by the authors.

5.3 Local Critical Structural Weaknesses

Evidence supporting the claim that the deformation capacity of the columns is not sufficient to resist large lateral displacements is prevalent in many of the specific details of the columns, beams, and beam-column connections. By comparing typical beam column size throughout the structure, as provided in Fig. 7, the beams are clearly very deep compared to the height of the column. Also, comparing the quantity of rebar in the beams and columns suggests that columns are likely the weakest link without more detailed calculation (L3 CSW).

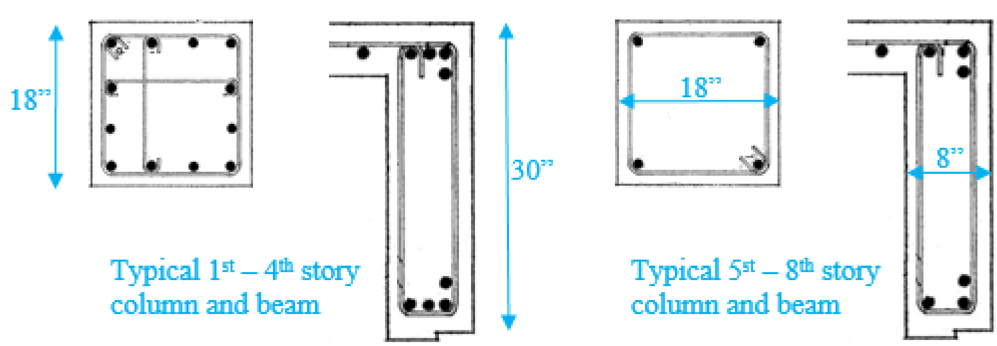

Fig. 7 - Typical beam and column configuration at lower and upper levels, showing likely failure of columns preceding beams. Source: Building plans [15], annotated by authors.

The damage evidence suggests that short columns were incidentally created at the perimeter, due to contact with cladding due to inadequate seismic gap between the cladding and the column [2]. Referring back to Fig. 5, it can be ascertained from both the physical building and the building plans that cladding is prevalent throughout the height of the structure. Furthermore, the cladding is inset flush with the exterior, essentially sandwiched between the columns. The consequence of this configuration is that when the displacements of the building are large enough, contact between cladding and column is possible, changing the moment and shear demand on the columns [2]. In addition, Fig. 8 has a typical perimeter column reinforcing plan detail which shows that there was not additional reinforcement detailing specified which would allow for this contact. In short, Securities House has a (G7) CSW because there is cladding between the columns.

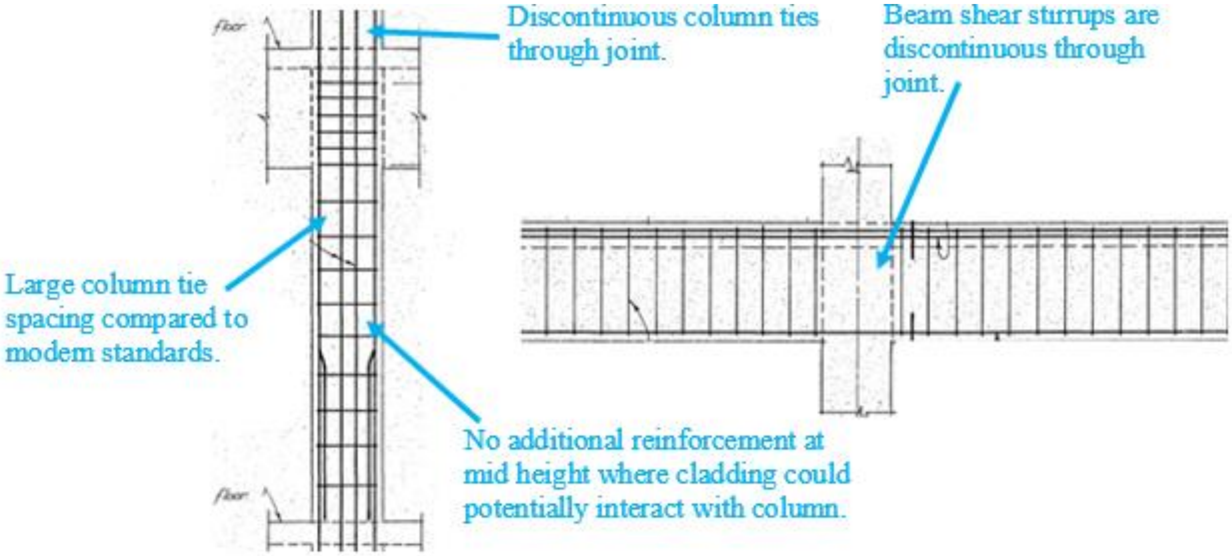

Fig. 8 - Inadequate tie and stirrup spacing in columns and at joints. Source: Building plans [15], annotated by the authors.

Fig. 8 also shows us that the column tie detailing is inadequate because of large spacing compared to modern detailing standards. This is typical of older concrete structures, but also a dangerous L4 CSW because it allows the possibility of shear failure in the columns. In addition, Fig. 8 also shows that both the beam stirrups and column ties are discontinuous through the joints. Modern detailing requires joints to be much stronger than the beams or columns, and also requires confinement of the joints [14]. Confinement would be difficult to achieve without sufficient reinforcing, and the lack of confinement/continuous reinforcing through the joints is considered a (L5) CSW.

Modern shear walls would generally have additional confinement rebar specified for end “boundary” sections. The Fig. 9 detail is the core section for Securities House. This image and the elevation detailing images which are not included in this paper all show that there is no special detailing for boundary confinement. The lack of detailing is a L2 CSW because lightly reinforced walls have been shown to buckle. Fig. 9 also emphasizes that there is a bit of subjectivity required in the decision making about CSWs. In this case, the external core section has a double layer of reinforcement, while the inner core section has a single layer of reinforcement. The single layer would be considered a L1 CSW and the double layer would not. Since we neglected the inner core in our assessment of a redundant system then it seems appropriate to neglect it here as well, assuming it is not a critical part of the lateral force resisting system.

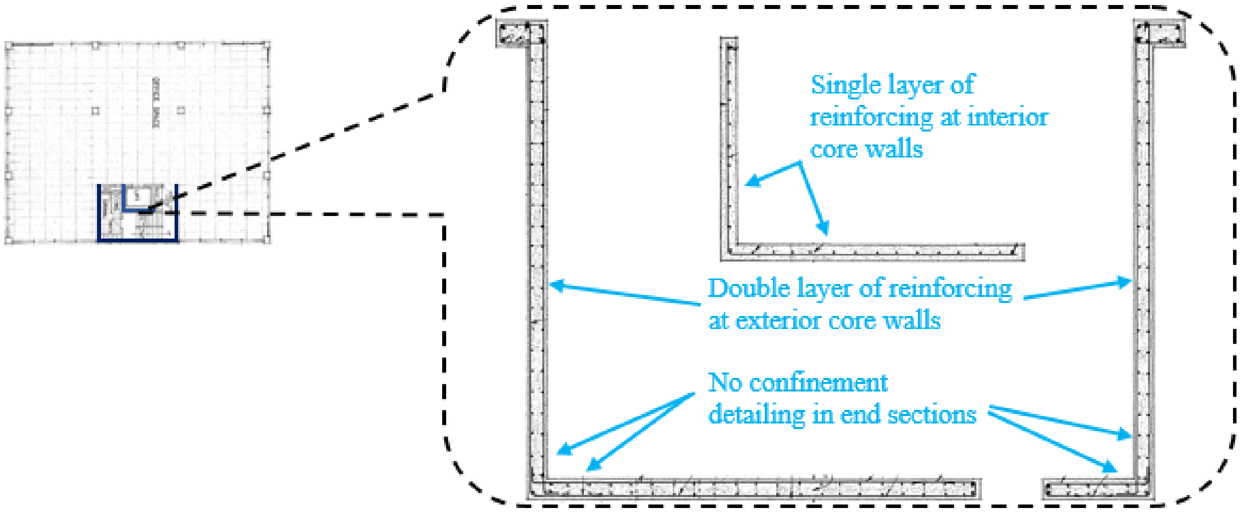

Fig. 9 - Reinforcement detailing in wall core section, shown with respect to location on the building plan view. Source: Building plans [15], annotated by the authors.

5.5 Securities House Summary

Table 3 summarizes the CSWs identified at Securities House. In total, the building earns 14 CSW points out of 20 possible. The most subjective points are associated with determining (L1) if the walls should be considered to have a single layer of reinforcement, (G3) if the loading path is redundant, and (L3) if the columns are weak compared to the beams. However, changing these assessments would only change the CSW score by 2-4 points in either direction, and regardless, Securities House would be pooled with the most dangerous buildings among the group of buildings considered here. Thus it would still be identified as a high-risk structure and would qualify for further structural analysis or seismic retrofit. In Fig. 3, the CSW score for Securities House is plotted against the damage rating. Securities house was given a damage rating of 4 because the engineer conducting the Level 2 rapid assessment issued the building a Red 2 placard due to substantial observable structural damage.

6. Conclusions

The goal of this research was to investigate building performance in the Christchurch earthquakes and to better understand the role critical structural weaknesses played in structural response. To achieve this, we studied many damage reports and selected nine buildings that had similar characteristics, such as height of 4-8 stories, cast in place reinforced concrete wall-frame structural systems, and damage that could be associated with critical structural weaknesses (rather than liquefaction or other effects). If a building was deemed to have suitable damage information, then inquiry was begun to locate the original drawings. With both a set of drawings and damage reports, we were able to analyze the buildings' CSWs and conduct a damage assessment rating. This process was illustrated through the Securities House example.

The study shows first that the accumulation of multiple CSWs in older buildings increases the likelihood of collapse. That observation supports claims that older building codes, fail to protect against brittle failure modes and that the damage amplifies in magnitude as more CSWs begin to interact with one another. The paper also shows that the CSW scoring method has promise for identifying the riskiest buildings because there is a high correlation between the CSW score and the damage. Results from these case studies indicate that buildings scoring approximately nine or larger should be prioritized for retrofit, but this threshold needs to be evaluated with further cases. The qualitative nature of the procedure is good because it constitutes a relatively quick assessment. This feature will be especially beneficial for communities where costly structural analysis of all dual systems buildings may not be possible.

Based on these findings, there seems to be value into looking closer at the interaction between multiple CSWs. Future research will investigate this using the Securities House building as a model archetype for building simulation, since this building was determined to have the most CSWs of all the case study buildings.

7. Acknowledgments

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8. References

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