Common Structural Deficiencies Identified in Canterbury Buildings and Differences in the Standard Practice in US and NZ

2013 NZSEE Conference

Paul Cordova, Nicholas Wetzel, Ben Weaver, Kenneth Tam

Simpson Gumpertz & Heger Inc., San Francisco, United States of America.

Will Parker

Opus International Consultants, Christchurch, New Zealand

ABSTRACT:

Canterbury Earthquake Recovery Authority (CERA) was established after the February 2011 earthquake to develop and execute a recovery and rebuild plan for Christchurch and the surrounding areas. The Canterbury Earthquake Recovery Act gave power to CERA to require a building owner, insurer, or mortgagee to carry out a full structural survey before a building is reoccupied. These surveys, called Detailed Engineering Evaluations, are generally required for all buildings in the Christchurch area. The authors have been part of these evaluations and have reviewed hundreds of buildings in Christchurch and the surrounding areas. This paper presents a list of common structural deficiencies identified during these assessments. Some of these deficiencies have been highlighted in the few prominent building collapses, but the same type of deficiencies have been found in many more buildings where damage was either limited or completely hidden until uncovered during subsequent investigation. Typically, a building with limited post-earthquake damage, especially those subjected to the Canterbury Earthquakes, should be indicative of a robust structural system and are often perceived by the public as “safe”. Unfortunately, subsequent Detailed Engineering Evaluations are identifying deficiencies in these buildings which are restricting occupancy, triggering strengthening, and, in some cases, leading to demolition. In addition to discussing these deficiencies, this paper compares the standard of practice between New Zealand and California and discusses some differences in the codes and approaches to designing and constructing buildings.

1 LOAD PATH IRREGULARITIES

New Zealand’s current “Earthquake Action Standard” (NZS1170.5:2004) is consistent with US and international codes in identifying the major building irregularities (vertical, plan, and torsional). In US practice, irregular buildings are generally penalized with amplified forces and displacements, and are required to be analyzed using a minimum of linear dynamic analysis. The NZ code approaches this in a similar manner, although some exceptions are allowed such as the equivalent static method is acceptable for irregular buildings less than 10 metres tall or with a natural period less than 0.4 seconds. In addition, torsional building demands are not amplified as in US codes, but demands from the linear dynamic method are required to be scaled to 100% of the equivalent static demands.

Stiffness and strength plan irregularities are not uncommon in New Zealand’s existing building stock. This has been highlighted in the tragic collapse of the Canterbury Television (CTV) building, the Pyne Gould Corporation (PGC) building collapse, and significant damage to the Grand Chancellor. The authors have assessed numerous other buildings with similar significant structural irregularities that experienced damage but no collapse. The floor plan below is from a four story concrete building located in Christchurch that was constructed in the late 1970s. This plan shows a large reinforced concrete core wall and a reinforced masonry wall at one end of the building. This unbalanced lateral resisting system results in a large eccentricity between the centre of mass and centre of rigidity and leads to a highly torsional building response, which was consistent with observed earthquake damage to the outlier walls, foundations, and contents. When assessed using NZS1170.5, we discovered excessive demands on the shear walls, diaphragms, and gravity columns, and identified this as an earthquake prone building (less than 33% of the current building code).

2 FLOOR PLAN WITH PLAN IRREGULARTIES

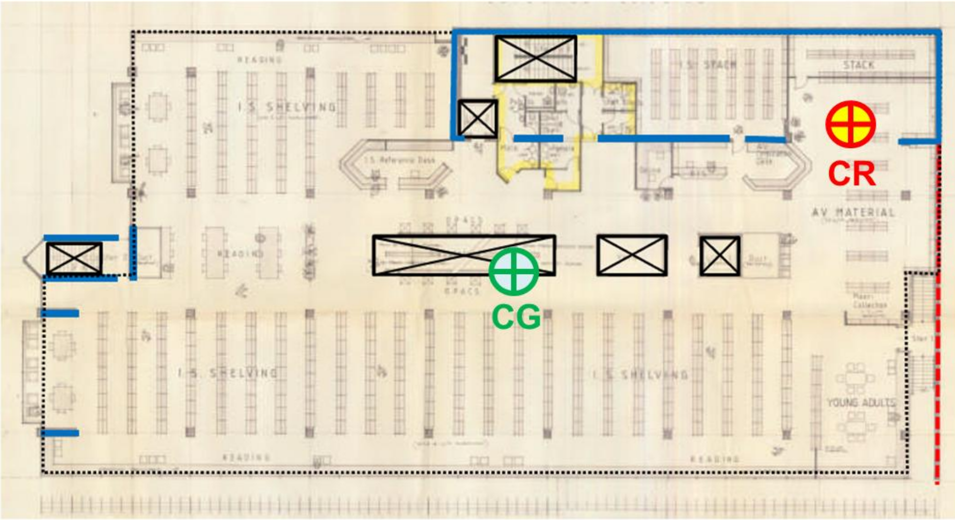

Figure 2.1: Floor plan

Other common load path problems are:

- Vertical stiffness irregularities such as shear walls that transition to moment frames and then cantilever columns at the roof.
- Discontinuous shear walls.
- Soft/weak stories.
- Irregular load paths relying on eccentric connections that impose torsion or weak-axis bending into vertical lateral resisting elements.

3 ROOF DIAPHRAGMS

A well detailed diaphragm is a critical component in a building's lateral load paths. Diaphragms and their components (chords, collectors, and ties) transmit inertial forces to the vertical lateral force resisting systems. One of the most common deficiencies observed by the authors is improperly detailed roof diaphragms, particularly in low-rise structures.

Below is a summary of observed roof diaphragm deficiencies.

- Lack of roof diaphragm - Architectural corrugated metal roofing supported on light gauge steel purlins is a widely used form of roof construction. The purlins run over the top of steel frames which in turn span between vertical lateral force resisting elements. The architectural roofing is not designed or detailed to span between the lateral force resisting elements and blocking is rarely provided between the purlins at the steel beams. Without a true diaphragm, the inertial forces are transmitted through weak axis bending in the steel frames to the vertical lateral force resisting system. This results in a very flexible roof response and caused significant damage in some buildings to non-structural ceilings and building services equipment.
- Collectors – It is common to see no design consideration of collector elements and their connections to drag load into the vertical lateral force resisting elements.
- Wall anchorage – The tops of precast walls often have inadequate anchorage into the diaphragms. Common problems were:

    - Timber ledgers acting in cross-grain bending.
    - Under designed anchorage inserts.
    - Minimal development of wall anchorage into ceiling diaphragms for framing parallel to wall.

- Tension only diaphragm bracing – Diaphragm action is often provided by tension-only elements. Common problems were:
    - Engineers designed tension-only systems to yield, and upon plastic elongation the rods went slack, introducing significant flexibility into the roof. In addition, proprietary connections were not tested for cyclic loading and, in some cases, experienced brittle fracture. After the documented failures, manufacturers and others are restating the need for elastic design of tension-only systems.
    - Bracing connections often connect eccentrically to the vertical lateral force resisting elements. A common configuration is braces in the plane of the steel framing terminating at the centroid of the beam column joint. The brace force must then be resisted by weak-axis bending in the column and delivered to a precast concrete wall via embedded bolts connected to the flange.

4 SECONDARY ELEMENTS

The performance of buildings is not dictated solely by the main lateral and vertical force resisting systems. Secondary elements may not contribute significantly to a building's lateral stiffness or strength, but, without good attention to detailing, can be subject to large load demands under imposed lateral displacements. These demands can cause brittle failure mechanisms, which may pose a threat to life.

Detailing deficiencies have been observed in the following elements:

- Architectural components: Precast panels, infill walls, partitions, ceilings.
- Gravity load resisting elements: spandrels, columns.
- Stairs.

The common issue amongst all these secondary elements is that little or no allowance was made for the imposed demands at peak inter-story deflection. We evaluated one building where the main lateral resisting system is a series of precast, post-tensioned exterior walls that possessed adequate strength and stiffness for the design earthquake. In addition, three story tall architectural precast panels, which form the balance of the exterior perimeter facade, are rigidly connected at each floor level via a plate welded to plates embedded in both the precast panels and the perimeter floor beam. While adequately designed to resist the panel's self-weight and out-of-plane inertial lateral loads, the detail does not allow for inter-storey displacements at the ULS event. The inter-storey drift demands generate large forces at the panel connections that far exceed the capacity, and could result in a brittle failure and a danger to life safety.

Stairs are another ancillary feature where consideration of peak inter-story displacements is crucial. The stair failure in the Forsyth Barr building during the 22 February shaking in Christchurch is a prominent example. The stairs were precast units, connected to a cast in-situ landing. Although seismic gaps were provided at the lower stair support to allow for inter-story drift, he inter-story demands of the 22 February shaking exceeded this gap. Similar issues were observed in the Grand Chancellor, Clarendon Towers, and at the stairwell in the Christchurch Bus Exchange (Royal Commission 2012, SESOC 2011)

5 PRECAST CONCRETE

Reinforced concrete buildings comprise a majority of the multi-story buildings in Christchurch. Since the early 1960's there has been a steady increase in precast concrete construction in New Zealand and it is particularly popular in Christchurch. Precast flooring systems comprise the majority of the flooring systems used in concrete buildings. These systems typically consist of precast flooring units to support vertical loads and a thin topping slab (on the order of 65mm thick) with non-ductile wire mesh reinforcement.

The authors have observed many buildings with a lack of proper consideration of diaphragm forces and a lack of robustness in precast concrete flooring systems. This includes reliance on wire mesh for shear reinforcement, lack of adequate shear transfer to vertical resisting elements, little to no consideration of collectors, and minimal seating area for precast floor units and consideration of lateral displacement compatibility. These problems are widespread in buildings assessed by the authors and have been discussed by others (Royal Commission 2012). These problems are well documented in the investigation and subsequent deconstruction of the Clarendon Tower. With typical precast floor details, the thin topping slab tore away from the perimeter moment resisting frame and the precast floor units shifted on their supports and in some locations were on the verge of collapse. The photo below shows a large crack in the topping slab at the interface of the perimeter moment frames.

Figure 5.1: Precast floor damage at Clarendon Tower

Given the popularity of precast concrete, there is a wide spread use of cast-in inserts and embedded steel plates for connections, and the authors have found many instances where they are used in critical load paths. They often do not have the capacity to meet the required demands nor develop the strength of the element attaching to them, which can lead to a brittle breakout failure of the threaded insert or embedded plate. This deficiency has been widely recognized and the SESOC Interim Design Guidelines have recommended cast-in inserts not be used for primary structural load paths. The detail shown below is an example where cast-in inserts were used to connect reinforcing steel to a cast-in place concrete wall. There are openings on either side of the wall and therefore the load path to one of the building's main lateral resisting elements is delivered entirely through these cast-in inserts. The capacity of the inserts was considerably undersized and we observed the beginning of a pullout failure.

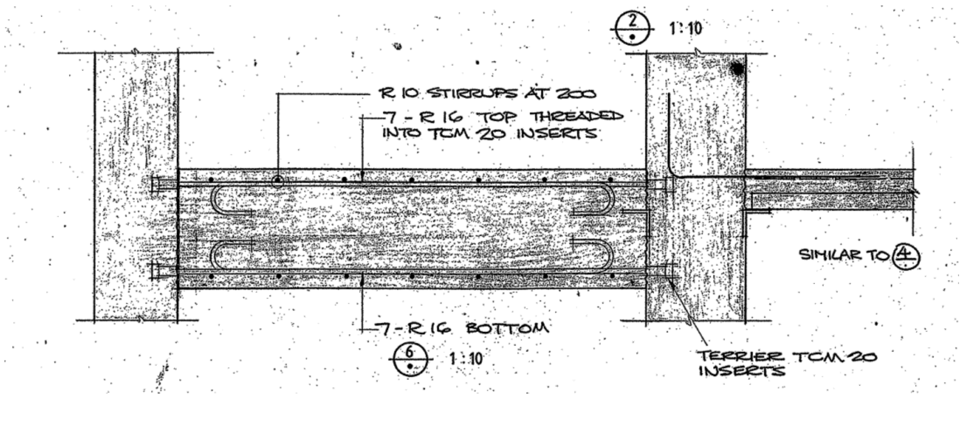

Figure 5.2: Collector detail with cast-in inserts

We have also found flexural controlled concrete walls (precast and cast-in-place) with low vertical reinforcing steel ratios. In many instances, the walls did not meet the code minimum requirement for vertical steel reinforcing ratio. There are reported cases of a single crack forming at the bottom of a concrete wall, which at first glance appears innocuous, but upon further investigation revealed fractured reinforcing bars. This has highlighted a potential short coming in both US (ACI 318) and New Zealand (NZS3101) design codes. Both concrete codes require a steel reinforcing ratio based on 28 day concrete compression strengths, lower bound tensile concrete strength, and steel yield strength. It is the authors' opinion that both code requirements are insufficient to safeguard against an under-reinforced flexural member. While we assume lower bound 28 day concrete strengths for design, in reality the average strength is at least 1.5 times larger. There are reports of tested concrete strengths in Christchurch being over 2 times their specified 28 day strength. In addition, as noted in ACI 363, the modulus of rupture of concrete can be up to 1.6 times larger than the lower bound strength. By considering the upper bound tensile strength of concrete and the expected concrete compressive strength (1.5f'c), the authors propose the minimum area of steel required for a flexure member to safeguard against being under-reinforced is:

$$A_{s,tension\ \ steel} = \frac{\sqrt{f'_c}}{3f_y}A_g \tag{1}$$

The performance of concrete shear walls has been well documented in previous articles and reports. (SESOC 2011, EAG 2011). We refer the reader to the above references for a more in-depth discussion on the observed performance of concrete shear walls during the Canterbury earthquakes. In addition to our previous discussion, other notable failures include slender wall buckling, bar buckling and concrete crushing in wall webs, grouted duct splices, and wall construction joint failure.

6 REINFORCED CONCRETE DESIGN & DUCTILITY

NZS 1170.5 and NZS 3101 define the minimum standards for seismic design and detailing of reinforced concrete buildings in New Zealand. The concrete standard defines three basic categories of seismic design based on the ductility of the structural system; Ductile (μ = 4-6), Limited Ductility (μ = 2-3) and Nominally Ductile (μ = 1.25). Design actions are derived from 1170.5 based on the system ductility. Requirements for capacity design and potential plastic hinge regions are defined for Ductile and Limited Ductility structures. Nominally Ductile structures are exempt from the additional seismic requirements of NZS3101, provided that member proportioning ensures that mechanisms could only develop in the same form as those permitted for ductile structures. This effectively means that regular structures are intended to be designed to prevent the column sidesway mechanism in order to provide some inherent ductility and ensure collapse prevention at the 'maximum considered earthquake', as referred to in the commentary (we note that the commentary uses the term 'creditable'). If the preferred mechanism cannot be achieved, it is required to assess local ductility demand and provide detailing to ensure that adequate deformation capacity is provided.

NZS 3101 is a substantial document with some complexity. The commentary is comprehensive and it provides significant guidance, although this may not be explicit. One reason for the popularity of design of nominally ductile structures is the simplicity of design and detailing with this method. We consider that this has enabled the design of low rise reinforced concrete buildings by practitioners who may be less familiar with the design and detailing requirements for ductile and limited ductility structures, especially for taller or more complex buildings where capacity design is very carefully considered. We would also suggest that this has led to the design of buildings that may not comply fully with the requirements of NZS3101, particularly relating to regularity, the actual mechanism, and the detailing required to ensure the required performance. This complexity, and industry awareness of design and ductility shortcomings presumably contributed to the need for the Red Book (NZCS and CCNZ "Examples of Structural Design to New Zealand Standard NZS 3101:2006"), which provides guidance on design and detailing for different ductilities. This document was first issued in 1982 as

“NZCS Applications of NZ Standard Code of Practice for the design of Concrete Structures NZS3101:1982” and provides design approaches for ductile design of reinforced concrete frames and shear walls. In 2010, the Red Book Part C was issued by CCNZ. This section covers nominally ductile and limited ductile design approaches for Reinforced Concrete Frames commonly being used in low rise buildings. The intent of the guide is to ensure that the design incorporates a good building configuration with a predetermined pattern of yielding and well defined, redundant load paths. The guide also provides detailing recommendations to ensure structural members and their connections will achieve the required strength and ductility.

Findings of our assessments suggest that systems which appear to have been designed as limited ductility and nominally ductile did not meet all the requirements of the standard for ductility and detailing. A factor in these shortcomings could be the complexity of the New Zealand standard.

7 CONSTRUCTION QUALITY AND SPECIAL INSPECTION

It is noted that there were some typical details that have not been universally well constructed. Examples include:

- Poor consolidation of concrete at construction joints in walls.
- Improper installation of epoxy anchor bolts.
- Ducts in precast concrete walls were not well grouted.
- Block walls not grouted properly.
- Poor positioning of confining steel.
- Poor execution of tie-down details or other connections in timber structures.
- Introduced eccentricities in concentric bracing elements through poor alignment.
- Take-up of movement allowances as construction tolerances i.e intended movement allowances were not provided because of construction inaccuracy.

Poor construction practices have come to light and in some cases contributed to increased levels of damage and in a few instances, collapse. We note that in many cases the deficiencies have not had a significant effect, often resulting in minor damage but not failure or collapse. We consider that the potential consequences of these deficiencies may have been reduced by the low number of earthquake cycles and availability of secondary load paths.

A lack of sufficient involvement of engineers during construction has been highlighted by some common deficiencies. Typically the engineer is engaged to provide construction monitoring, generally on a fixed fee basis. It should be further noted that the level of construction monitoring followed is generally considerably less than full supervision, and relies on the construction producer statement provided by the contractor. The level of the engineer’s sign-off is therefore based on a pre-determined level of construction monitoring, on a scale of 1 to 5, based on four factors; size, complexity, contractor experience and consequence of non-compliance. Although the obligation is related to the work activities, it is noted that the fee seldom varies regardless of the contractor and their QC systems, as this is often not determined when the engineer prices the project. Hence there may be pressure on engineers to soak up cost if the contractor has particularly poor QC systems in place. This effectively results in the engineer ensuring quality is maintained while minimising time spent on inspections.

A possible solution could be to follow a similar practice to that followed in the US. In the US, when drawings and calculations are submitted to the local authority, the engineer of record (designing engineer) is required to provide a list of structural elements requiring special inspection. Details of what special inspection is required is specified by the building code. A special inspector is a completely independent reviewer engaged by the owner to ensure that all identified elements are implemented as per the engineers’ drawings.

8 NZSEE ASSESSMENT RECOMMENDATIONS AND ASCE31 AND ASCE41

Detailed Engineering Evaluation performed in New Zealand follows a two-step approach, Qualitative and Quantitative Assessments. The Qualitative Assessment is often performed using the Initial Evaluation Procedure (IEP) outlined in the NZSEE Recommendations (2006). The IEP procedure compares the design base shear of historical New Zealand building codes to NZ1170.5, adjusting for assessed ductility, location, and occupancy to come up with a base line percentage of New Building Standard (%NBS). This base line number is then adjusted by various factors, termed Critical Structural Weaknesses (CSW) to arrive at the building's assessed %NBS. The methodology provides some guidance for 5 types of CSWs (Plan irregularity, Vertical Irregularity, Short Columns, Pounding Potential, and Site Characteristics) and also includes "Other Factors", based on the reviewing engineer's judgment.

Assessments performed in the US commonly begin with ASCE 31 "Tier 1" checklists. The checklists consist of common "deficiencies" that were found in various building types in the United States. Many of the deficiencies observed by the authors during the detailed engineering evaluations of Christchurch Earthquakes are also found in the Tier 1 check list. For example, "Load Paths", "openings adjacent to walls", etc are a few of the deficiencies are highlighted in the "Tier 1" check list. While a structural engineer experienced in earthquake engineering may be able to identify many of these deficiencies, the ASCE 31 type check list provides additional guidance and also provides some level of consistency for assessments.

The Quantitative Assessment methodology in NZSEE guidelines and ASCE41 both provide some guidance to engineers with regards to using linear procedures (forced based methods), and non-linear procedures (displacement based methods). Some of the issues brought to the forefront of the discussion following the Christchurch earthquake can be assessed using the ASCE41 framework. For example, the consideration of a Collapse Limit State (CLS) can be assessed by performing explicit check for "Collapse Prevention" under MCE. Another example is the assessment of diaphragms, chords, and collectors. The NZSEE guidelines focus on the vertical elements with few guidelines on assessing diaphragms components. ASCE41-06 contains explicit consideration of components of a horizontal lateral load resisting system, i.e. diaphragms, chords, collectors. ASCE 41-06 also provides the frame work for performing non-linear analysis with guidelines for developing backbone curves developed through extensive testing.

While the ASCE 41 approach may be seen by some as overly prescriptive, the NZSEE Qualitative/Quantitative assessment procedure relies heavily on engineering judgment and the results of assessment can vary widely depending on the reviewing engineer.

9 CONCLUSIONS

Hundreds of assessments of Christchurch buildings have been undertaken and this has enabled the authors to observe a pattern of common deficiencies in design, detailing and construction. The effectiveness and consistency of the assessment guidelines and design standards has also been reviewed by comparing New Zealand and North American (ASCE 31 & 41) approaches.

The common deficiencies we have identified are largely consistent with those found by the consultant community in NZ, and specifically SESOC who have produced an 'Interim Design Guide". Although the guidelines are not definitive or finalised at this stage, our findings support these guidelines and the interim measures capture the shortcomings we have identified. We consider that the wider use of performance based design considerations, both for design and assessment, including use of ASCE 31 & 41 documents, would be beneficial.

We recommend that the upcoming review of the NZSEE Guidelines consider use of ASCE 41, either by incorporation of the ASCE 41 performance based framework, or by adoption of ASCE 41with amendments to suit NZ practice. We further recommend that NZS 3101 be revised to include the finalised SESOC design guidelines and to simplify the provisions for ductility levels and detailing to make the standard more user friendly.

10 REFERENCES

American Society of Civil Engineers (2003), “ASCE 31-03: Seismic Evaluation of Existing Buildings”.

American Society of Civil Engineers (2006), “ASCE 41-06 including Supplement Number 1: Seismic Rehabilitation of Existing Buildings”.

Canterbury Earthquake Royal Commission, 2012. Final Report: The Performance of Christchurch CBD Buildings, Volume 2, Christchurch, New Zealand

Canterbury Earthquake Royal Commission, 2012. Final Report: Summary and Recommendations in Volumes 5-7, Volume 5, Christchurch, New Zealand

Canterbury Earthquake Royal Commission, 2012. Final Report: Canterbury Television Building (CTV), Volume 6, Christchurch, New Zealand

Engineering Advisory Group (EAG) (2011), “Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury Part 2 Evaluation Procedure”, Structural Engineering Society New Zealand (Inc.), Rev 5.

Engineering Advisory Group (EAG) (2011), “Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury Part 3 Technical Guidance”, Structural Engineering Society New Zealand (Inc.)

NZSEE, 2006, Assessment and improvement of the structural performance of buildings in earthquakes, New Zealand Society for Earthquake Engineering.

Red Book, 2008, *Examples of Concrete Structural Design to New Zealand Standard 3101*, CCANZ, Wellington, New Zealand

Structural Engineering Society New Zealand (SESOC), 2011, *Practice Note: Design of Conventional Structural Systems Following the Canterbury Earthquakes*, New Zealand