INITIAL LESSONS ON DESIGN, RESPONSE, AND RECOVERY FROM THE 2016 KAIKOURA EARTHQUAKE

D. Ojala

ABSTRACT

While it gained somewhat less attention than the Canterbury Earthquake sequence a few years earlier, the November 14, 2016 Kaikoura Earthquake had a significant effect on the northern regions of New Zealand’s south island, as well as New Zealand’s capital city, Wellington, located at the south end of the north island. Although lessons learned from earthquake reconnaissance are often broadly applicable, the striking geographical and socio-economic similarities between Wellington and the San Francisco Bay area (and other comparable west coast cities) presents a unique opportunity to look at Wellington’s challenges with response and recovery and forecast possible issues with disaster response in the United States. This paper will identify key technical issues identified during post-earthquake reconnaissance and inspections in Wellington and surrounding cities, including: 1) the performance of precast concrete moment frames relative to their intended performance, including the impact of frame dilation over multiple cycles, 2) the performance of precast cladding and flooring systems and their compatibility with the highly-ductile frames that support them, and 3) the impact of deep and soft soil sites on the disparate levels of damaged observed throughout the city. Then, and perhaps of greater importance, the paper will explore the impact of these technical topics as they relate to big-picture issues, including: post-earthquake damage assessments, feasibility of repair of these structures, and public perception and trust of their building stock and the structural engineering profession in the months immediately following the event. Ongoing response by the government and local engineering community in Wellington will be discussed and compared to past earthquakes, with the purpose of identifying new factors which may impact our ability to respond effectively and resiliently in future events.

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Introduction

While it gained somewhat less international attention than the Canterbury Earthquake Sequence (CES) a few years earlier, the November 14, 2016 Kaikoura Earthquake had a significant effect on the northern regions of New Zealand’s south island, as well as New Zealand’s capital city, Wellington. The event offered New Zealand an opportunity to verify the importance of seismic vulnerabilities identified in its building stock and see if new programs, procedures, changes and lessons learned after the CES had been effective. In addition, the geographic and economic differences between Wellington and Christchurch, and the striking similarities between Wellington and cities like San Francisco and Seattle in the United States, offer an opportunity to study how the makeup of different communities can color their response to natural disasters.

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Although this paper will briefly discuss the effects of the earthquake on Kaikoura and the surrounding area on the South Island, the focus will be on the Wellington area and its building stock, with the goal of identifying takeaways relevant to North American practitioners. The bulk of the paper will discuss the “harder” technical details of the earthquake, Wellington’s emergency and engineering response, and observed damage patterns. The paper will conclude with a discussion of “softer” issues, including the way the CES has influenced the recovery process, the multiple hats structural engineers are asked to fill after an earthquake, the role of insurance in resilience and post-disaster decision-making, and the effect the above issues have had in coloring public perception.

Event Overview

The Mw 7.8 Kaikoura earthquake occurred just after midnight local time on the morning of Monday, November 14, 2016. The hypocenter was estimated to be 15km deep and located about 95km NNE of Christchurch, 60km SW of Kaikoura, and on the order of 240km southwest of Wellington, in a rural area of the South Island of New Zealand. As such, the destructive power of what was the most powerful earthquake to strike New Zealand since the estimated Mw 8.2 Wairarapa earthquake in 1855, was concentrated away from the dense urban centers to the north and south along the Pacific-Australian plate boundary [1]. In the immediate vicinity of the causative faults, horizontal ground accelerations up to 1.2g and vertical accelerations as high as 2.7g were recorded. As noted, however, these areas were lightly developed. The level of ground shaking in Christchurch was minimal (reporting of Christchurch-area ground motions has been minimal, but PGAs appear to have been on the order of 0.1g) [1], but the combination of distance from the causative faults and local soil conditions proved to make for a “perfect storm” for many Wellington-area buildings. While PGAs at inland rock sites were on the order of 0.1g, are lacking in high-frequency content, and the resulting spectral accelerations represented service-level (SLS) earthquake shaking, spectra at deep soil sites near the Wellington CBD were significantly higher [2]. For sites located on reclaimed land at the waterfront (which had estimated site soil periods ranging from 1 to 2 seconds), a pronounced spectral peak of about 1.5 seconds, meant that design level (ultimate limit state or ULS) accelerations may have been met or exceeded for buildings with periods ranging from about 1 to 2 seconds [3]. Unfortunately, the periods of many buildings in the area fell into that range, with building periods often closely matching the period of the site they were built upon.

Wellington’s Initial Response and the Targeted Assessment Program

Wellingtonians awoke just after midnight on November 14 to roughly 90 seconds of significant ground movement. Large portions of the city were cordoned off Monday, but after it was clear early that no complete collapses or outward signs of severe structural damage had occurred, most of the area cordons were cleared Tuesday, sparking some criticism [4]. However, by Thursday, several buildings had been evacuated, including three buildings quickly flagged for demolition (61 Molesworth, Reading Carpark, and the Statistics House), and large areas of Wellington’s CentrePort development, built entirely on reclaimed land, had been cordoned off due to widespread ground movement, including liquefaction, subsidence, and lateral spreading [5]. Although structural engineers performed Rapid Building Assessments on many Wellington buildings and most were promptly cleared for reoccupancy, the Wellington City Council (WCC)
had no way of knowing which buildings had been inspected, and the performance of several more buildings came under scrutiny (including instances of initially cleared buildings being evacuated upon completion of more detailed evaluations) in the following days and weeks. Following an initial investigation into the partial collapse of precast floor units in the Statistics House [6], the Ministry of Building, Innovation, and Enterprise (MBIE) issued a letter to WCC recommending a systematic review of Wellington buildings exhibiting similar characteristics. WCC, in cooperation with the New Zealand Society for Earthquake Engineering (NZSEE) and the Structural Engineering Society of New Zealand (SESOC), established a Targeted Assessment Program (TAP). Starting on December 19, 2016 80 owners with buildings meeting the Affected Building Profile were notified that their property required a Targeted Damage Evaluation (TDE) by a structural engineer in accordance with a set of guidelines that would assure critical areas of each building were inspected. After some extensions, a final deadline of March 8, 2017 was established for completion of these inspections [7].

Per the TDE Guidelines, the Affected Building Profile included the following characteristics: Principal lateral load resistance through concrete moment frames, coupled with precast flooring systems (noting that the most vulnerable to loss of support for precast units are those where there are multiple frame bays in parallel with a single span of flooring); a natural period range of 1-2 seconds (typically 8-15 stories, but note that this occurs in some flexible frames as low as 5 stories); or sites where the shaking in the period range has been amplified. This amplification may be due to basin effects and/or soft soils. The published profile and damage types were somewhat vague and the program guidelines note that the characteristics and damage were not meant to be exclusive. In particular, it is noted that concrete wall buildings in combination with frames might still exhibit damage consistent with the profile [8].

As noted, 80 buildings were initially included on the TAP list, but after elimination of several buildings that either showed extensive damage and were already being subjected to detailed evaluations or that did not meet the profile and were clearly undamaged, 64 buildings submitted reports to WCC. Of these 64 buildings: 87% were commercial or public buildings, 64% were built between 1983 and 1995 (86% between 1975 and 1995), 70% were characterized as having “ductile concrete moment frames,” 29% appear to have had no or minimal direct ties from columns into the diaphragms, 73% had hollowcore or precast double-tee flooring, and 66% appear to have had no or minimally ductile connection between precast units and supporting beams [7]. This selection appears to be a representative sample of the commercial and high-rise building stock in Wellington’s CBD. As was apparent in the definition of the Affected Building Profile, this selection of buildings does not include the numerous 2-4 story older concrete or masonry buildings that make up significant portions of Wellington’s building stock. Detailed discussion of the observed damage mechanisms will be discussed in the next section of this report, but the results of the TAP will be summarized here. According to a report on the program prepared by The Kestrel Group [7], 14% of the 64 buildings that received a TDE had widespread structural damage (to frames and/or floors) on multiple floors, 30% had frame and/or floor damage generally on one floor, 8% had isolated instances of floor damage (some of which may not have been earthquake-related), and 48% had no structural damage identified. Nonstructural damage was generally noted in all buildings. The report notes that the eight significantly damaged buildings that ended up being excluded from the program would likely fit in the “widespread” damage category.
Moment Frame Elongation and Displacement Compatibility with Precast Floors

The damage states identified in the TDE Guidelines are focused on looking for failures in the load path connecting concrete moment resisting frames and the gravity load resisting components (specifically precast elements that are not cast monolithically with the frames and are often only connected by a cast-in-place topping slab). Such failures can lead to loss of stability for or load transfer to the frame and/or damage to or loss of support for the precast elements. As moment frames, especially those detailed for high ductility (and therefore generally designed for lower strength) rack back and forth, plastic hinges are expected to form at the ends of each beam as concrete cracks and the flexural reinforcement yields. In the absence of axial restraining or restoring forces provided by post-tensioning or an adjacent monolithic slab, several cycles of tensile yielding can lead to elongation of the beam, often exacerbated by prying action as diaphragm shear cracks open or debris falls into cracks, preventing closure. In multi-bay braced frames, the combined dilation of multiple joints can be significant. In order to accommodate the expansion, one or both of the following generally occurs: 1) gradual lean in columns along the line, with the largest occurring at the end columns, or 2) out-of-plane movement of the column and adjoining beams where the column is not adequately tied into the diaphragm. Where precast slabs span parallel to these moment frames, this dilation can reduce the bearing length for the precast units, leading to loss of support for the units.

The possibility of frame dilation had been studied and reported in New Zealand as early as 1993 [9], but appears not to have been widely appreciated until the early 2000s [6]. The need for consideration of frame elongation on precast unit seating is stated in the concrete design standard NZS 3101.1-2006, although it is not clear if the current recommendations for precast unit bearing lengths take into account the possibility of multi-bay frames or if predicted levels of elongation are adequate for the modern, ductile ($\mu = 6$) frames that exhibited the greatest dilation in the Kaikoura earthquake [6]. Even in the absence of frame dilation, the large interstory drifts and joint rotations that occurred in ductile frame systems can be inherently incompatible with relatively rigid precast units. For units that span across multiple bays of frames, the fact that the units are typically reinforced for single-curvature gravity loads means that overstress can occur if the unit is constrained to move along with the intermediate frame beams and columns in multiple curvature. Damage can also occur at bearings as units rock back and forth or slide (or fail to slide) on their supports, causing both spalling to supporting corbels and shear cracking or spalling of the ends of the precast units and loss of gravity support. Where precast spans perpendicular to frames and rigid units are supported across beam column joints, nonuniform bearing conditions and contact with adjacent units can induce torsion and transverse flexure and shear action. Since precast units are typically prestressed, contain minimal shear reinforcement, and are optimized for gravity loads, they are sensitive to damage. Combined with the fact that there is rarely positive connection between the topping slabs and the units, and detailing of reliable positive connection between precast units and supporting beams is a relatively recent practice, minimal precast damage can sometimes lead to local collapse. These displacement compatibility issues can also occur in slender shear wall buildings where rocking and flexural deformation can impose large rotation demands on slabs at the wall ends [10].

New Zealand has spearheaded research into the use of topped precast systems in
diaphragms in the last couple decades [10], prompted in large part by precast failures in the 1994 Northridge quake and reinvigorated by reconnaissance after the CES. As such, many of these vulnerabilities have been understood, current detailing practices address most of them, and many existing vulnerable buildings have been retrofitted to improve their ductility or provide fail-safes in the event of loss of hollowcore support. However, some vulnerabilities may remain, due in part to limitations in testing capabilities and construction practices:

- The body of testing on precast units, especially hollowcore, has been gradually expanding over the last decade. However, while techniques are improving, testing is often limited to checking isolated aspects of performance, such as the rotation capacity of hollowcore-to-beam connections [11] or torsional strength [12]. The strength of the unit and its connections in the combined longitudinal flexure, transverse flexure, torsion, in-plane shear, and gravity loading is exceptionally difficult to predict. In other cases, controlled laboratory testing has been limited, and the performance of units has been justified by extrapolation of related testing data, estimates from theoretical calculations, or limited manufacturer load testing of completed units.

- Like similar, highly optimized structural elements such as pre-engineered steel and wood trusses, castellated/cellular beams, and voided slabs, design margins are often thin and performance poorly understood, and manufacturing processes limit the types and quantities of reinforcement that can be applied. For hollowcore units, prediction of shear capacity of is challenging [13], and at long spans, with high levels of prestressing, the brittle limit states of shear at bearings and longitudinal web shear can control. While fiber reinforced webs have come into use in recent years, most existing hollowcore units contain only longitudinal reinforcement.

- Design of precast connections is often a delicate balancing act that is difficult to perfect. Current recommendations for hollowcore connections to supporting beams use capacity design concepts to attempt to create a ductile connection that will yield in flexure and accommodate end rotations before causing a brittle shear failure of the unit. Over-strengthening one component can therefore lead to other undesirable failure modes. For precast tees and hollowcore units, insufficient end bearing lengths for stems makes units susceptible to loss of seating due to frame dilation and can concentrate bearing forces on stems. However, excessive lengths, especially when used in combination with inadequate bearing pads, can reduce the rotation capacity of the end connection, leading to spalling of support corbels and damage to the precast units [10].

- While precast bearing design, especially bearing lengths, is a game of millimeters, the remaining construction trades may not be operating under the same tolerances. Loads on precast units during transport and installation must be carefully limited to prevent damage, but many units will inevitably arrive with some damage, which may hinder eventual seismic performance [14]. In addition, corbel reinforcement in supporting precast beams must be placed as close to the nose of the corbel as possible to be effective. However, many instances have been noted where such reinforcement was set back into the beam closer to normal beam tolerances, rendering the corbel reinforcement nearly ineffective and allowing deep spalls that nearly led to complete loss of precast support.

- After earthquakes, damage inspection is a tall order for hollowcore units because typically only the bottom surface of the unit is visible. Critical cracks running through the bearing area, up the side webs, or town interior webs are not visible without the use of a borescope to look at the interior of cells.
While conceptual repairs have been discussed in the literature [14], and visible exterior cracks appear to have been repaired with epoxy and fiber reinforced polymers in the past, there does not appear to be any consensus method for the repair of many precast units. Given the difficulty of demolishing and replacing a unit within an existing building, development of reliable repair methods is of the utmost importance.

As with similar manufactured building components, the structural engineer of record typically delegates design of precast units to the manufacturer. However, perhaps more so than other similar components, the two parties must coordinate carefully to complete their respective tasks. The manufacturer must be aware of the gravity demands as well as any imposed force or displacement demands due to seismic. The engineer of record must understand the expected strengths, design assumptions, and other limitations of the unit to make sure that all connection detailing is adequate, and verify that a precast unit is an appropriate choice for the intended performance [10].

**Additional Performance Notes**

While the interaction of precast flooring with ductile frames proved to be the primary focus of post-earthquake investigations, the author noted some additional points based on their observations in Wellington, many of which have since been echoed by the Kestrel Group report on the Targeted Assessment Program [7]:

- It has already been noted that site soil periods at ground motion recording locations near the Wellington waterfront ranged from 1 to 2 seconds, according to GNS. Based on spectra for these sites, buildings with periods ranging from 1 to 2 seconds may have experienced shaking close to or in excess of 500-year design levels (ULS). It is worth noting that some of the most heavily damaged buildings may have had periods that closely matched the sites they sat upon. The best example of this may be at the BNZ Harbour Quays building, which was instrumented during the quake. According to GNS, the site period was 1.2 seconds, and the building period inferred from the sensors installed in the building was also on the order of 1.2 seconds. These both compare to the ground motion’s spectral peak of 1.5 seconds. Past studies and earthquake reconnaissance have noted that shaking can be amplified in buildings whose natural period matches the natural period of the soil deposit [15, 16], so this may have been a contributing factor to the scope of damage observed.

- Related to the point above, accurate prediction of building stiffness (and by extension, period) is critical for mid-rise buildings like those affected by the quake (based on the shape of typical design spectra). Unfortunately, neither underestimation nor overestimation of building stiffness is consistently conservative, as one choice can overestimate seismic forces while underestimating drifts and p-delta amplification, and vice versa. Ideally, a bounding technique would be used to determine critical combinations of initial and elongated periods based on expected levels of ductility that might result in peak forces, peak deflections, or match the site period reported by the geotechnical engineer. A review of design documents for numerous structures, including modern designs, suggests that stiffness has frequently been overestimated by designers.

- A consequence of overestimating stiffness is that both first order and second order deflections may be underpredicted. Observations from the TAP indicate that many buildings, including modern ductile frame buildings, experienced significant interstory drifts. While only one partial structural collapse was noted (Statistics House), other
buildings are claimed to have been one strong cycle away from similar floor failures, at least one major egress stair collapse occurred, and non-structural damage that could have caused injury was widespread. Furthermore, several buildings were deemed too dangerous to repair, including the Reading Carpark and 61 Molesworth. Since the margin between collapse and life safety is generally arbitrary, it is difficult to say with certainty that the Wellington building stock met its life safety objective. Further study will be required to determine if the drifts experienced were consistent with the design-level ground motions experienced by many of these buildings.

- As mentioned, much of New Zealand’s concrete construction is precast. Floors and frames are typical, although precast walls are frequently used both structurally and non-structurally (as cladding and rated partitions). While fast and economical to construct, such buildings generally offer the seismic mass of a concrete building without the stiffness or inherent redundancy of monolithic cast-in-place concrete. A possible benefit of precast is the ability to use ductile connectors to concentrate damage (discussed further below) at replaceable fuses, but most followed traditional detailing techniques. Performance varied between the myriad systems in use, but most appear to have performed no better or even worse than equivalent cast in place systems. Construction/cold joints and architectural reveals often led to weak planes for shear loads and predefined flexural crack locations, concentrating strain into short hinges that resulted in concentrated yielding in the reinforcement. These essentially formed fuses that were difficult or impossible to replace. This mixed performance of precast systems was previously reported after the CES [17].

- Also consistent with CES reconnaissance and a possible consequence of precast construction was extensive damage to diaphragms and collectors. The thin topping slabs concentrated diaphragm shears through a narrow interface. Interior cracks typically occurred at precast panel joints and were often exacerbated by prying action as precast units rocked back and forth. Wide cracks often formed at perimeters in buildings where cold-drawn wire mesh was the only reinforcement provided. These perimeter cracks were often a mixture of shear cracking and tensile cracking due to frame dilation and rocking of precast units below [8, 10].

- The Kestrel Group report [7] notes that the adequacy of rapid safety evaluations can be questionable for many multi-story buildings. Of the TAP buildings 57 received Level 2 Rapid Assessments in accordance with the MBIE Rapid Building Usability Field Guide. Five of these reports flagged critical structural damage and 24 flagged the building for further inspection. However, 11 of the 27 buildings that requested no additional inspection were later found to have one or more Critical Damage States, two of which were widespread. The author notes that the MBIE guide was based on the American ATC-20 methodology. While this method states the importance of performing a detailed evaluation in many older steel frame buildings, precast construction is not as common in high-seismic regions in the United States and ATC-20 does not clearly encourage the user to explore the sort of damage states identified by the TAP. In either event, the need to utilize evaluators with a strong knowledge of the performance of the building types they are assessing is critical for large or complex buildings after significant earthquakes.

- It is noted that one of the common indicators of the presence of structural damage or loss of structural capacity is residual damage, both nonstructural (ceilings, partitions, glazing, enclosures) and structural (such as cracks in concrete informing the severity of damage to reinforcement). Experiences in both Christchurch and Wellington suggest that as
nonstructural components become more resilient and are detailed to accommodate larger deflections and accelerations, this may no longer be as reliable an indicator as was previously thought. Many of the instances of inaccurate rapid assessments noted above were a result of a lack of obvious nonstructural damage to suggest damage to the underlying structure. In addition, as field and laboratory testing of reinforcing bars is becoming standard in post-earthquake damage and repair assessments, it is becoming clear that in certain elements, small residual crack sizes may belie appreciable strain in the underlying reinforcement. The consequences of limited yielding of isolated bars to the global performance of the structural element or building is a matter of ongoing debate, but the potential unreliability of crack sizes as a primary indicator of reinforcement damage has been studied [18] and should be appreciated. The width of cracks accompanied by concrete spalling or crushing and buckling of reinforcement may be more meaningful indicators of damage thresholds.

The Positives

If buildings on WCC’s Affected Building Profile were characterized by their 1-2 second periods, and locations on deep, soft soil sites, those not meeting that profile fared quite well. Shaking was generally considered to be at service level for most buildings on rock sites in the hills rising sharply away from the Wellington CBD, home to most of the region’s residential neighborhoods. There were few reports of anything more than cosmetic damage to these residences. Also, as noted previously, many stiff older concrete and unreinforced masonry buildings (UMBs) making up Wellington’s historic Te Aro and Newtown neighborhoods to the south of the CBD survived unscathed. This meant that the majority of Wellington’s housing stock was unaffected. Despite this fortunate break, government agencies were careful not to ignore the hazard these buildings would pose during a local earthquake on the Wellington fault. In January 2017, the New Zealand government asked that UMBs in areas at a heightened risk for aftershocks (including about 250 Wellington buildings, later reduced to about 100, and 50 buildings in nearby Lower Hutt) undergo façade and parapet retrofit work by March 31, 2018. It was noted, however, that as of October 2017, none of the Wellington UMBs had completed the work, and only 5 had expressed intent to do so by the deadline. A shortage of available engineering and construction resources due to the remaining ongoing recovery work and the government’s underestimate of the cost of the work when creating a $3M fund for subsidizing the work, were likely contributors to the apparent lack of compliance [19].

Although extensive damage has been noted to precast concrete construction, some precast buildings, or portions thereof, utilized good detailing practices or specifically targeted low-damage performance in accordance with Precast Seismic Structural System (PRESSS) techniques [20]. Such detailing included the use of distinct ductile connection elements like U-shaped Flexural Plates (UFPs), steel coupling beams, unbonded steel post-tensioning, and elements designed for rocking. Where thought was given to isolating nonlinearity to such distinct, readily inspected fuse elements like these, buildings appear to have performed as intended, allowing for prompt, focused inspection and limiting down time, mirroring similar observations from Christchurch. As such, interest in low-damage structural systems has increased and new guidelines are in development [21].
Widespread geotechnical damage was noted in some waterfront areas (most notably Wellington CentrePort) causing associated damage to wharves and buildings with shallow foundations, including damage to slabs on grade due to subsidence [5]. However, most modern, multi-story buildings near the waterfront were founded on deep foundations. While investigations are still ongoing, surveys and limited observations performed to date have not revealed significant damage or hinging of piles/piers or grade beams/caps.

Glazing, cladding, and curtain wall systems generally performed well. Given the observed issues with precast flooring, another delegated design item requiring accommodation of seismic movement, this is somewhat surprising. While broken panes could be seen in many buildings, multi-story glazing and curtain wall systems generally performed well. Even in highly ductile frames that experienced large transient and residual interstory drifts (including the effects of frame dilation), modern curtain walls appear to have accommodated the movement without glass breakage. Some damage to precast concrete cladding was observed, often in concrete in the vicinity of attachment points. In some cases, this may have been due to failure of bolts to slide in their slotted holes. However, residual drifts or movement at joints often damaged flashing and other weatherproofing details, resulting in water intrusion. In a rainy marine environment like Wellington, this quickly became problematic for some buildings.

Conclusions

Though it may be easy to brush aside the Kaikoura earthquake as yet another aftershock in the Christchurch Earthquake Sequence, there are plenty of fresh lessons to be learned. The targeted nature of the far field ground motions on Wellington’s waterfront structures has drawn new attention to understanding building stiffness and the interaction between building period and site period; the poor performance of precast concrete frame structures has revealed the complexity of the systems and the need for careful consideration of compatibility between the highly ductile frames and highly brittle precast floors; and the success of using initial reconnaissance findings to implement a targeted damage assessment program on critical building types was demonstrated. As the earthquake recovery is ongoing, we can look forward to more details and new stories coming to light in the coming years.

References

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