Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 50, No. 2, June 2017

DAMAGE TO CONCRETE BUILDINGS WITH PRECAST FLOORS DURING THE 2016 KAIKOURA EARTHQUAKE

Richard S. Henry[1], Dmytro Dizhur[2], Kenneth J. Elwood[3], John Hare[4] and Dave Brunsdon[5]

(Submitted *March 2017*; Reviewed *April 2017*; Accepted *May 2017*)

ABSTRACT

The 2016 Kaikoura earthquake resulted in shaking in excess of design level demands for buildings with periods of 1-2s at some locations in Wellington. This period range correlated to concrete moment frame buildings of 5-15 storeys, many of which had been built in Wellington since the early 1980s, and often with precast concrete floor units. The critical damage states used to assess buildings during the Wellington City Council Targeted Assessment Programme are described and examples of observed damage correlating to these damage states are presented. Varying degrees of beam hinging were observed, most of which are not expected to reduce the frame capacity significantly. Buildings exhibiting varying degrees of residual beam elongation were observed. Cases of significant beam elongation and associated support beam rotation resulted in damage to precast floor unit supports; in one case leading to loss of support for double-tee units. The deformation demands also resulted in damage to floor diaphragms, especially those with hollowcore floor units. Cracking in floor diaphragms was commonly concentrated in the corners of the building, but hollowcore damage was observed both at the corners and in other locations throughout several buildings. Transverse cracking of hollowcore floor units was identified as a particular concern. In some cases, transverse cracks occurred close to the support, as is consistent with previous research on hollowcore floor unit failure modes. However, transverse cracks were also observed further away from the support, which is more difficult to assess in terms of severity and residual capacity. Following the identification of typical damage, attention has shifted to assessment, repair, and retrofit strategies. Additional research may be required to determine the reduced capacity of cracked hollowcore floor units and verify commonly adopted repair and retrofit strategies.

INTRODUCTION

The M_w 7.8 Kaikoura earthquake on 14th November 2016 caused significant ground shaking in the Wellington region. Initial observations of damage to buildings in the Wellington CBD indicated that structural damage was not widespread and isolated to a small number of buildings. Reports by engineers of significant structural damage to some buildings highlighted potentially high deformation demands on flexible frame buildings. With a significant duration greater than 25 seconds [1], the earthquake resulted in repeated cyclic demands on buildings, increasing the possibility of beam elongation and damage to precast floor systems. In addition, examination of the recorded ground motions showed a significant amplification in the spectral acceleration demands between 1-2 seconds, in excess of the design spectra at several recording stations [1]. These findings lead to further examination of structures with 1-2 seconds period range, which typically comprise of 5-15 storey concrete moment frame buildings.

As described in Brunsdon et al. [2], the Targeted Assessment Programme was designed to address public safety issues and to provide confidence that appropriate engineering investigations of buildings most affected by the 2016 Kaikoura earthquake have been carried out. The overall objective of the Targeted Damage Evaluations (TDE) was to identify the presence of critical damage states that could affect either local or global stability, and hence occupancy of part or all of a building. The buildings selected for the TDE consisted of those typically consisting of 5-15 storey concrete moment frame buildings with precast concrete floor systems. This type of building was commonly built during the building boom in the 1980s and a large majority of these buildings contain hollowcore precast concrete floor units with potentially non-ductile detailing [3]. The observed structural damage to buildings in Wellington is described in the subsequent sections, focusing on the critical damage states that were used to categorise damage during the TDE process.

CRITICAL DAMAGE STATES

During development of the Targeted Assessment Programme [2], a working group was formed to identify the specific damage states that assessing engineers should identify during building inspections, termed herein *Critical Damage States (CDS)*. This approach helped focus the inspection process on damage that could be critical to the future seismic performance of the building.

As shown below in Table 1, the CDS classified building damage based on risk. CDS A, B, and C, related to the damage in the primary structure, while CDS D related to damage to secondary structural and non-structural elements. CDS A identified damage where the gravity load path may have been compromised in precast floor systems, thus posing a possible risk of local collapse under gravity loading (i.e. without aftershock). CDS B identified damage posing risk of collapse, but only in the case of future aftershocks. CDS B included both local collapse of precast floor units (CDS B1 and B2) and global collapse risk due to support for, or damage

[1] *Corresponding Author, Senior Lecturer, Department of Civil and Environmental Engineering, University of Auckland, Auckland rs.henry@auckland.ac.nz, (Member)*
[2] *Lecturer, Department of Civil and Environmental Engineering, University of Auckland, Auckland (Member)*
[3] *Professor, Department of Civil and Environmental Engineering, University of Auckland, Auckland (Member)*
[4] *CEO, Holmes Group Ltd, Christchurch (Fellow)*
[5] *Director, Kestrel Group Ltd, Wellington (Life Member)*

to, concrete columns (CDS B3 and B4). CDS C identified damage, anticipated to be found in many of the buildings, but not posing a direct collapse risk, but still important to be identified in terms of assessing future seismic performance and repair decisions for the building.

While CDS A implies loss of gravity load support, it is noted that several load paths still exist after initial failure of the precast unit. However, alternative load paths are generally unreliable and were assumed to be inadequate to provide support for precast floor units with CDS A damage. For example, the bond between the topping slab and the unit will result in gravity load share between adjacent units once a single unit has experienced damage congruent with CDS A. It is noted that this load sharing depends on the tension capacity of the bond between the topping and the unit and ability for load transfer would be less for buildings with timber infill between units. Another alternative load path is arching action which may develop as the unit deflects downward and the ends of the unit jam up against the support beams. Finally, for units in the corner of floor diaphragms, typically three sides of hollowcore floor units will be supported by beams, making it difficult for the unit to collapse. The presence of these alternative load paths may explain the collapse of double-tee units in only one building and no collapse of hollowcore floor units in the Kaikoura earthquake.

Table 1: Critical damage states.

Critical Damage State	ID	Description	Notes
CDS A: Damage posing local collapse risk (possibly without aftershock)	A1	Transverse cracking at ends of hollow core floor units or diagonal cracking at the ends of ribs.	- Within 400mm of the supporting beam. - With vertical dislocation or diagonal crack in web.*
	A2	Significant damage to support for flange-hung double tee floor units.	With vertical dislocation at the support.
CDS B: Damage posing local or global collapse risk in the case of aftershock	B1	Transverse cracking at ends of hollow core floor units or diagonal cracking at the ends of ribs.	- Within 400mm of the supporting beam. - Not meeting A1 criteria.*
	B2	Reduced precast floor unit support.	- Evidence of seating loss due to elongation and/or spalling. - Not meeting A2 criteria.
	B3	Loss of lateral support for columns over multiple stories.	- Significant cracking adjacent to columns with no reinforcement ties into the floor diaphragm.
	B4	Shear damage to corner columns.	- Due to beam elongation and shear demands. - Inclined cracks greater than 0.5 mm.
CDS C: Damage to primary structure posing lower risk	C1	Plastic hinge damage.	- See criteria in Beam Plastic Hinges section below.
	C2	Web cracking in hollow core floor units.	- Splitting webs along the length of the unit. - Observed with a borescope camera.
	C3	Longitudinal cracking of hollow core floor units.	- Either bottom or top soffit.
	C4	Mesh fracture in floor toppings.	- Location of mesh fracture will affect the diaphragm load paths and column lateral restraint.
CDS D: Damage to secondary structural and non-structural elements that may cause increased life safety risk	D1	Stairs.	- Damage to stair supports. - Damage to stair unit itself.
	D2	Heavy cladding elements effecting external spaces, especially public spaces.	- Damage to panels and/or fixings with inadequate moment allowances or brittle connections.
	D3	Heavy overhead non-structural elements.	- Focus on elements posing life safety risk.

** Note that for hollowcore units where CDS B1 (transverse cracking at ends) was found, but no investigation of the webs was able to be undertaken to identify the presence of diagonal cracks, this was to be reported as CDS A1.*

BUILDING INSPECTION PROCESS

The TDE Guidelines [4] recommended a process of progressive inquiry for building damage evaluations that considered both the configuration of the building and the levels of damage observed from a review of the drawings and an initial investigation. Where only limited or no structural damage was encountered, no further intrusive investigation was required. However where damage of a certain nature and extent was encountered, a more comprehensive level of further investigation was required to be undertaken.

The process recommended for identifying damage to precast concrete floor systems is summarised as follows:

1. Review available drawings for the building, identifying the load paths, structural system and any configuration issues. From this understanding of the building, the areas where damage is to be expected can be identified - i.e. potential damage 'hotspots'.
2. Undertake an initial building investigation ensuring that the identified hotspots are inspected. If damage (structural or non-structural) is seen that was not predicted from the drawing review, then the load path identification from step 1 above should be revisited.
3. If evidence of CDS A or B is observed in hotspots or other areas, progressively extend the investigation to other regions on levels with high drift demands.
4. If evidence of CDS A or B is not identified in hotspots or other inspected locations and the damage (or lack thereof) confirms the load path/system identification, no further intrusive investigation is required.

The approach outlined above was considered to provide a reasonable likelihood of identifying CDS A or B within the subject building. Due to present of carpets and ceiling finishing in occupied buildings, full inspection of every precast floor unit was deemed impractical and hence there was a possibility that some CDS were missed.

Based on half-dozen inspections of building with a range of damage states, the following 'indicator issues' were identified that might contribute to the likelihood of damage in precast floors:

- Moment-resisting frames with multiple frame bays in parallel with a single span of flooring;
- Irregular floor layout (including L-shaped or curved floor plans and irregular layout of structural systems);
- Large openings in diaphragms impacting load path to lateral force resisting systems
- Transfer beams; or,
- Nominal (or lack of) structural ties across the floor diaphragm holding the columns of a frame or braced bay into the building.

Early inspections also identified locations of likely damage to precast floor units. Generally, damage 'hotspots' are associated with the building configuration issues noted above, and are located where localised deformation of precast floor units is necessary to accommodate the movement of the supporting seismic and gravity systems. Examples of 'hotspots' included:

- External corners of a building.
- Locations of torsional demand or concentrated deformations on precast units (e.g. between two adjacent walls or adjacent to eccentrically braced steel frames)
- At corners of large diaphragm openings.
- Precast units with continuity restraint at gravity beams near gravity columns.

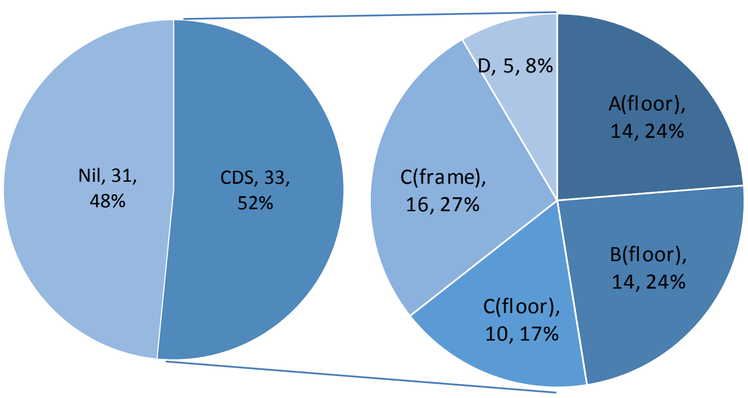
OBSERVED CRITICAL DAMAGE STATES

The following sections describe damage that was observed following the 2016 Kaikoura earthquake to concrete buildings with precast floors in Wellington, using the CDS described previously. Confidentiality restricts the identification of specific buildings, hence the typical range of damage observed is discussed rather than the performance of individual buildings. Photos of representative damage states included here were taken by the authors and do not identify the building in question. Despite the focus on damage cases below, it is emphasised that the majority of buildings in Wellington were undamaged by the Kaikoura earthquake.

Inspection reports of 64 buildings, as required by WCC using the TDE process described above, provide an overview of the damage to multi-storey concrete buildings during the Kaikoura earthquake. A further eight buildings with significant damage were exempt from the TDE process as early inspections made it clear repairs and/or demolition would be required. Three of these buildings have been, or are in the process of being, demolished at the time of publication. Further details on the TDE buildings are provided in the WCC Targeted Assessment Programme report by Kestrel Group and QuakeCoRE [5].

As shown in Figure 1, 52% (33) of inspected buildings showed evidence of at least one CDS, with 28 reported with CDS A or B (all classified as floor CDS), 26 reported with CDS C (10 classified as floor CDS and 16 as frame CDS), and 5 buildings with reported CDS D (secondary elements). The primary focus of the TDE inspection process was on identifying CDS A or B which may impact building occupancy. Of the CDS A or B cases, 8 buildings showed distributed damage across the floor diaphragm and/or over three or more stories, with the remaining 25 buildings exhibiting more localised or isolated damage.

For the TDE buildings where data was available on rapid building inspection conducted immediately following the Kaikoura earthquake, 91% were reported to have no or minor evidence of structural damage and no further inspection was recommended in over 50% of these reports. Due to the more intrusive investigation promoted during the TDE, a significant amount of previously unobserved damage was discovered in these buildings, including damage classified as CDS A or B. CDS were reported in 11 buildings where no further inspection was recommended following the rapid building inspection. This finding highlights the importance of the WCC Targeted Assessment Programme as well as issuing guidance on the type of building and damage likely for a given earthquake scenario.



Clusters of CDS	CDS ID
A(floor)	A1, A2
B(floor)	B1, B2
B(frame)	B4, B3
C(floor)	C2, C3, C4
C(frame)	C1
D	D1, D2, D3

Figure 1: CDS identified in TDE buildings.

Beam Plastic Hinges

Over 73% (47) of TDE buildings were designed with concrete moment resisting frames. Most of these buildings were designed post-1974 and are expected to exhibit strong-column-weak-beam inelastic behaviour. The extent of beam plastic hinging observed in the TDE buildings ranged from minor insignificant cracking to extensive distributed cracking and spalling of cover concrete, as shown in Figure 2, with the majority of buildings exhibiting only minor evidence of beam hinging.

(a) Minor cracking

(b) Moderate cracking

(c) Exceeding CDS C1 criteria

Figure 2: Examples of beam plastic hinging.

Based on limited data from ongoing research at University of Auckland [6], the following criteria were established to identify cases where the residual capacity of the plastic hinge may need further investigation:

1. Total crack width in plastic hinge > 0.005d (where d is the beam depth).
2. Sliding has occurred on a crack.
3. Wide (>0.5mm) diagonal cracks.
4. Concrete degradation, indicated by significant spalling (concrete cover can be removed by hand).

It was assessed that when none of the above apply, the damage is not expected to result in significant degradation in strength, deformation capacity, or energy dissipation; however, degradation in stiffness, leading to larger displacement demands in aftershock, could still be expected. Only 25% (16) buildings exceeded the criteria set for CDS C1 that is expected to result in a reduction in beam residual capacity.

Reduced Precast Unit Support

As observed in previous research and past earthquakes, axial elongation of beams due to plastic hinging and damage to precast floor unit support connections can result in potential loss-of support of precast floor units [7-9]. The only case of reported loss-of support leading to localised collapse of the floor units occurred in the Statistics House building. Residual frame dilation in Statistics House was estimated to be in the order of 100-150 mm over the first three levels of the north and south frames [10]. The frame configuration of two bays of frame to a single floor span, coupled with a sliding support detail at the outer end, led to a likely seating reduction in the order of 60 mm in the worst case, at the outer end support. This elongation combined with rotation of the support beam and damage to the flange-supported double tee units led to the loss of support of several units, as shown in Figure 3. The loop bar detail used in the supports of the double tee units in Statistics House is still used sporadically in current construction practice, despite being identified as potentially non-compliant and dangerous by Hare et al. [11]

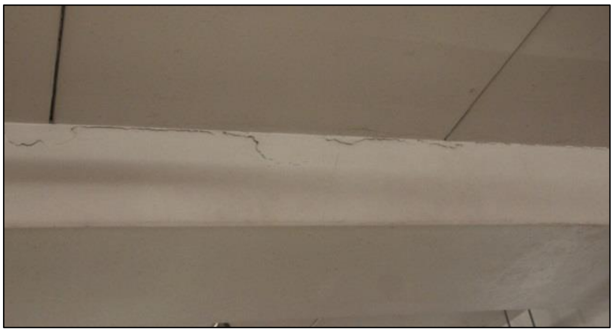
(a) Building overview

(b) Collapsed floor unit in corner of ground floor

Figure 3: Statistics House floor collapse.

Beam elongation was identified in at least 8 buildings, predominantly affecting unrestrained corner columns that were pushed out from the building with associated floor corner cracking (as shown in Figure 7 to Figure 9 below). The beam elongation generally reduced the seating width for units spanning into corners, but with the exception of Statistics House, sufficient residual seating of precast units was maintained. In restrained beams away from the corners, hinging was often less significant, with cracks that had generally closed leading to minimal residual beam elongation.

In addition to beam elongation, the building drift and associated rotation of the support beam can lead to damage at the support. Minor spalling of the support ledge was observed in many buildings with hollowcore floor units seated directly on the support beam, as shown by the example in Figure 4a. The use of a bearing strip, as required by NZS 3101:2006 [12] and used since the mid-2000s, can delay the onset of such minor spalling by allowing the unit to slide relative to the support beam. However, spalling of support ledges is still observed (e.g. Figure 3b) when the building is subjected to high drift demands with bearing stresses and prying effects increased. Examples of significant spalling of the support beam are shown in Figure 4c and d where the entire cover concrete has been lost. The seating length in this building was adequate to ensure that a residual bearing area was maintained to prevent complete loss of support and potential collapse of the floor units. Despite the extensive damage observed, Figure 4c highlights the importance of proposed changes to design standard requirements in NZS 3101:2006 [12] (draft amendment 3) were seating lengths must be sufficient to accommodate demands due to both elongation and rotation, as well as the loss of bearing area due to spalling. In addition, the ledge reinforcing is visible in the photos, indicating that that adequate ledge reinforcement is essential to maintaining support during large earthquakes.



(a) Minor spalling (without low-friction strip)

(b) Minor spalling (with low-friction strip)

(c) Spalled ledge exposing reinforcing

(d) Deep crack at support ledge with concrete still in place

Figure 4: Support ledge spalling.

In cases of spaced hollowcore units, examples were observed of spalling of the support beams at the unit corners, or cracking at the corners of the hollowcore unit, as shown in Figure 5. This damage pattern was also observed during the 2010/2011 Canterbury earthquake sequence [9] and is thought to be related to torsional rotation of the hollowcore units due to diaphragm demands or deformation incompatibility. Hollowcore units spaced by timber infills are more susceptible to concentrations of damage at the corners due to the lack of restraint from adjacent units.

Cracking of Floor Diaphragms

Beam elongation and rotation can cause extensive cracking to floor diaphragms due to displacement incompatibilities [7]. Locations of stretched or offset floor coverings are often one of the first places to look when assessing precast floor damage. Examples of the cracks observed in floors when the floor coverings were uplifted are shown in Figure 6. Many of these cracks initiate as shrinkage cracks which typically propagate at the joints between precast concrete floor units, or between the floor unit and the support beam. In some cases these shrinkage cracks are widened during the earthquake as additional deformations are imposed on the floor.

Cracks in the concrete topping do not necessarily pose a significant risk in terms of potential collapse, but may result in fracture of mesh reinforcing, a reduction in diaphragm stiffness, and potential loss of diaphragm load paths. As such, the topping cracks alone were not considered a critical damage state, but further investigation was required to identify if cracks propagated through hollowcore units (CDS A, B & C) or if mesh fracture had occurred (CDS C).

(a) Spalled support beam *(b) Cracked corners of hollowcore*

Figure 5: Corner cracking or spalling of spaced hollowcore.

Figure 6: Examples of cracks in floor topping.

The most commonly observed damage pattern of the floor diaphragms was cracking of floor precast units in the corners of the floor diaphragm. As shown in Figure 7, corner cracking typically takes one of three forms: 1) localised with only one or two, typically fine, cracks near corner column (examples in Figure 8); 2) a complex intersection of many, typically wider, cracks, generally limited to the first unit (examples in Figure 9); or 3) a single diagonal crack set back from the corner, crossing more than one unit, and intersecting the frames roughly halfway along the bay lengths. The first two corner damage patterns result from deformation incompatibility between the beam and the floor unit leading to twisting of the first unit running parallel to the external frame, and can be exacerbated by beam elongation. Hollow core units are week in torsion and crack at low deformation demands [7, 8]. In 2004, an amendment to NZS 3101:1995 [13] introduced a requirement to include a link slab between the exterior frame and the first precast unit to accommodate the deformation of the beam relative to the floor unit. Due to the small number of post 2004 buildings included in the TDE, the performance of link slabs was not able to be assessed in detail. The orientation of the third damage pattern noted above suggests this cracking may be accentuated by bidirectional movement of the building and the resulting bending of the floor diaphragm in the corner, in addition to beam elongation demands.

In at least one significantly damaged building, the layout of the structural system led to failure of the diaphragm with wide cracks extending through the topping and hollowcore units (10-15mm wide in some locations with topping steel mesh fracture). The principal crack ran longitudinally along the unit and stepped across cells at discrete locations, with damage to internal webs of the unit being likely. As illustrated generically in Figure 10, connection between the diaphragm and the short seismic frames at perimeter was interrupted by the presence of stairwells, leading to high shear demands on a short length of diaphragm. Floor unit demands were further exacerbated by limited beam elongation in the corner of the building and reasonably high building drifts (estimated at 1.0 - 1.5%).

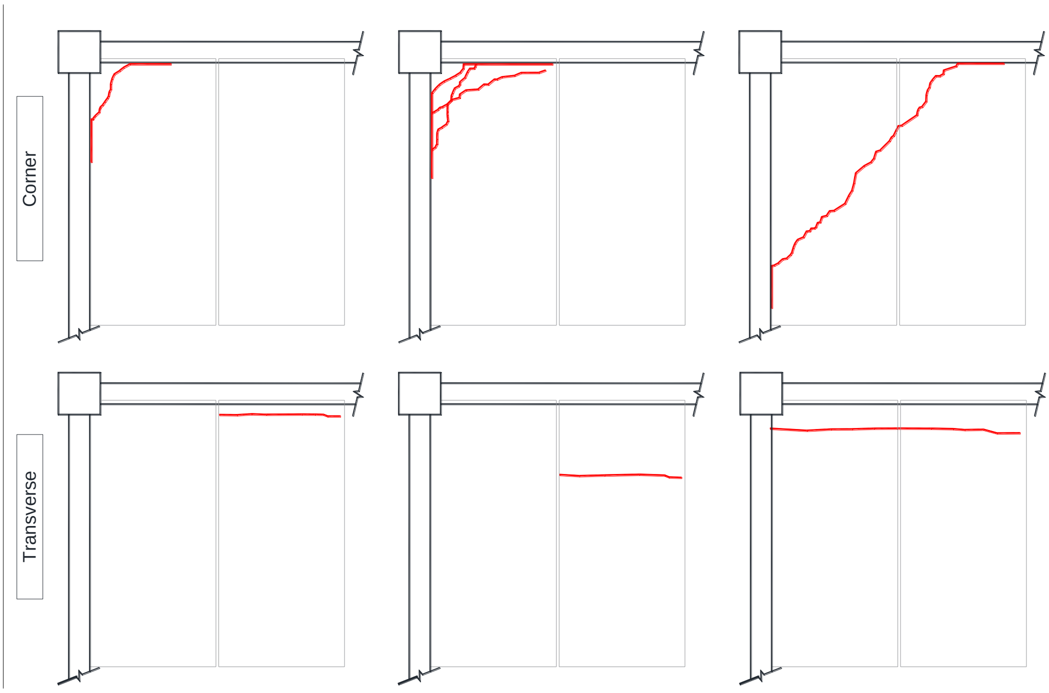
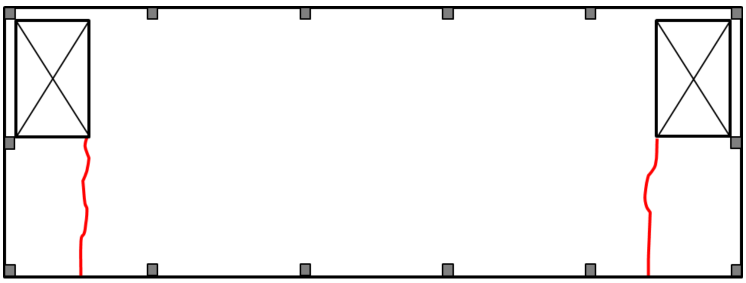


Figure 7: Plan views of typical crack patterns observed in hollowcore floor units (from below or from above).

Figure 8: Damage in corners of floor diaphragm – Single cracks.

Figure 9: Damage in corners of floor diaphragm – Significant cracking.



(a) Plan view

(b) Wide floor cracks

Figure 10: Generic illustration of rectangular moment frame building with diaphragm failure at openings for stairwells.

Transverse Cracks in Hollowcore

When assessing the critical damage states, it was important to identify whether the cracks in the floor topping concrete had propagated between the joints of precast units or through the unit itself. Different types of cracks found in hollowcore units are shown in Figure 11. Hollowcore units manufactured in New Zealand do not contain transverse reinforcement and bottom flange prestressing strand is not well developed at the ends of the unit and may also be affected by strand draw-in or slip. As a result, transverse cracks near the support can significantly impact the shear capacity near the ends of the hollowcore floor units [7]. Transverse cracking in hollow core units was observed in 14 of the 64 TDE buildings. In three of these buildings the transverse cracks were identified to have propagated diagonally through the web of the hollowcore (CDS A1), but in the remaining 11 buildings the orientation of the cracking through the hollowcore unit had not been confirmed. Transverse cracking close to the supports of hollowcore units was also observed during the 2010/2011 Canterbury earthquake sequence [9], but was more prevalent in the Kaikoura earthquake due to the prevalence of 1980s hollowcore unit construction in Wellington and the earthquake characteristics.

The most common hollowcore floor unit damage patterns observed in Wellington buildings can be described as a combination of crack sections shown in Figure 12 and plan views shown previously in Figure 7. The top row would be classified as CDS A as the gravity load support has been compromised with a diagonal crack in the web. Engineers were encouraged to use a borescope camera, such as that shown in Figure 13, to inspect cracking in web of units. If webs were not inspected, the TDE guidance document [4] instructed the engineer to assume the presence of a diagonal crack. In the case of spaced hollowcore, the sides of hollowcore units were visible and diagonal cracking could be seen in the webs, as shown in Figure 14. Past research has demonstrated that units with transverse cracking top or bottom, combined with diagonal cracking in webs, can be close to sudden brittle failure [14, 15].

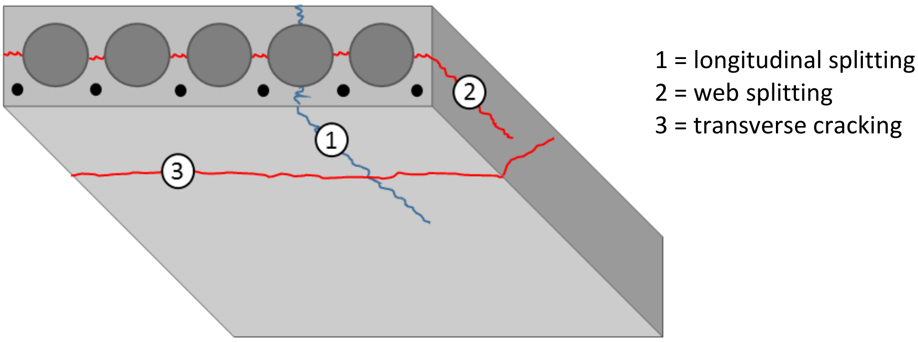


Figure 11: Types of cracks found in hollowcore units.

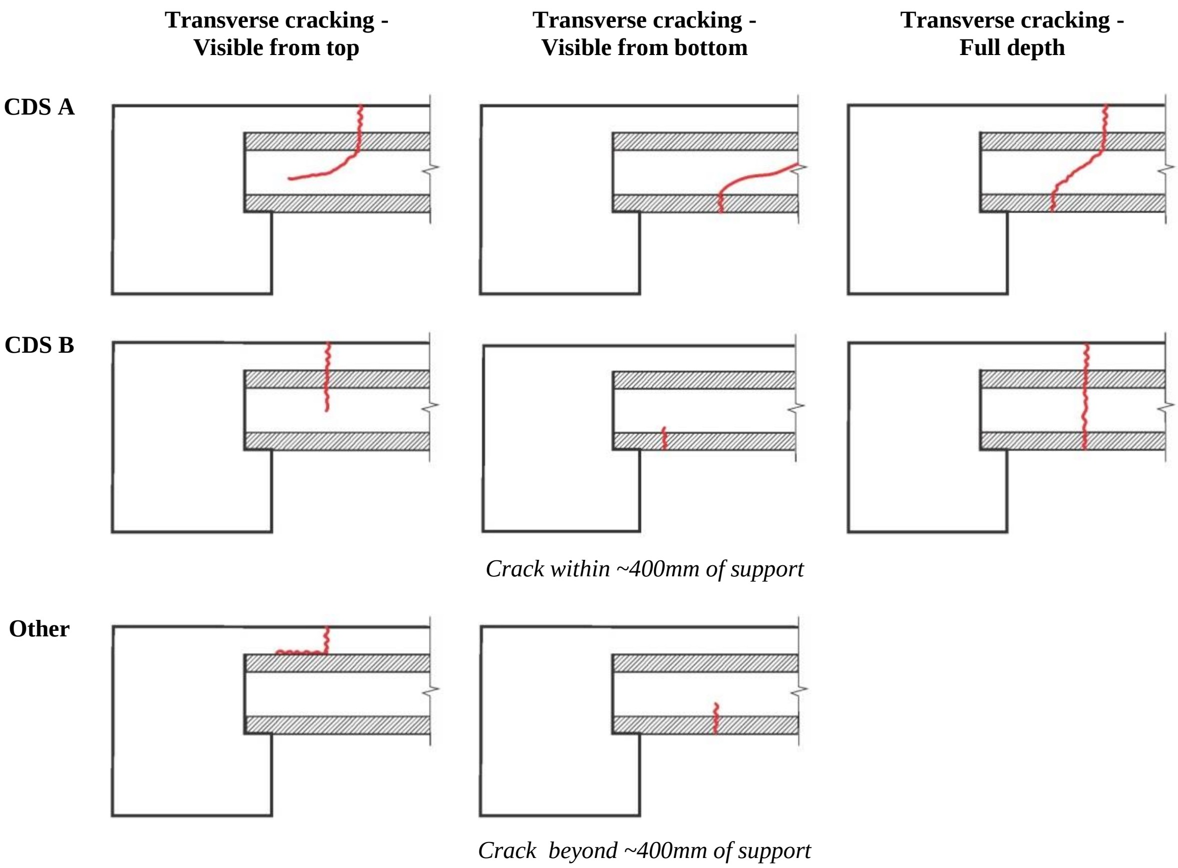
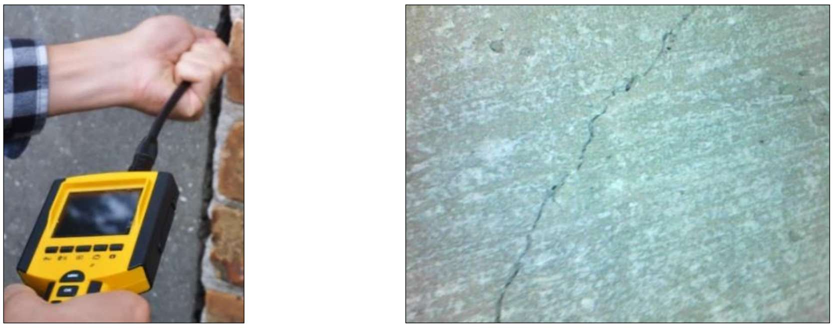


Figure 12: Cross sections of observed crack patterns in hollowcore floor units.



(a) Borescope camera (b) Image inside cell of hollowcore

Figure 13: Borescope camera example.

(a) Narrow crack (<0.5 mm) (b) Wider cracks

Figure 14: Diagonal cracking in webs of hollowcore floor unit.

The second and third rows in Figure 12 show damage patterns which are generally of lower risk, compared with CDS A; however, cracking at underside near the support is also of concern. Such damage, along with possible retraction of the prestress strand from the end of the unit, can reduce the bond between bottom flange prestress strand and concrete leading lower shear strength at the end of the hollowcore unit. Inspection of the end of the hollow core unit for any signs of strand retraction was typically not possible due to casting of beam concrete and topping slab over hollowcore unit end, hence without extensive removal of concrete it was not possible to determine if the strands had retracted.

Away from the floor diaphragm corners, transverse cracking in hollow core was less prevalent. When observed, transverse cracking was typically observed in individual units (i.e. not continuing through multiple units). Transverse cracks were sometimes found in floor units randomly distributed across a floor diaphragm, both adjacent to exterior seismic frames and near interior gravity frames.

Figure 15 shows examples of transverse cracking within 100 mm of beam support. This unit was located adjacent to a column offset in the supporting frame. Column offset resulted in localised deformation concentration in beam and thus higher stresses on the floor units.

Several buildings were also found where single transverse cracks in the hollowcore units occurred at approximately 300 mm from the support (gravity or seismic frames) as shown in Figure 16. The location of crack on top surface typically corresponded to the end of the starter bars and in many cases the crack was found to extend vertically for the full depth of the unit. While past research has shown that negative moments at supports may lead to cracking at the end of the starter bars, this cracking was previously found to extend into a diagonal crack in the unit web with significant concern for remaining gravity load support [15], in contrast to the vertical crack observed here. The exact cause of the observed vertical cracking pattern through the depth of the unit is unknown. Initially there was some suspicion this cracking could be due to temperature or shrinkage effects on the restrained unit. However, a review of 11 parking garages in Auckland (i.e., where no earthquake damage is present) did not lead to the identification of any similar cracking patterns, suggesting that this transverse cracking of units in Wellington is most likely earthquake-induced. Given the distance of the transverse crack from the support (typically 300 mm), the bond for bottom flange prestress strand is likely sufficient to sustain gravity loads, but further widening of the crack in aftershocks was still a concern. It is noted that typical retrofits of hollowcore floor systems in New Zealand [3] only provide support for units within approximately 100 mm of the supporting beam and would not prevent collapse of units failing at such cracks at approximately 300 mm from the support. Further research is needed to understand the cause of this cracking pattern, the capacity in future aftershocks, and potential retrofit techniques to address this failure mode.

In some buildings more complex damage patterns were discovered including diagonal cracking across hollowcore units, as shown by the examples in Figure 17. These cracks either occurred near the corners of the buildings where diagonal cracking in the topping propagated through the hollowcore unit, or at irregularities in the floor plan or structural system layout.

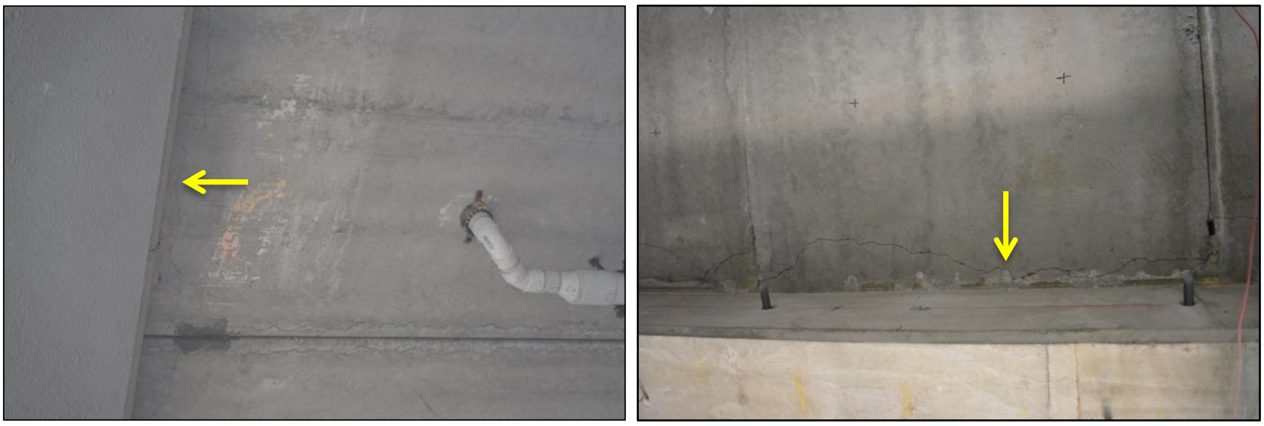


Figure 15: Transverse cracking of hollowcore floor units close to support.

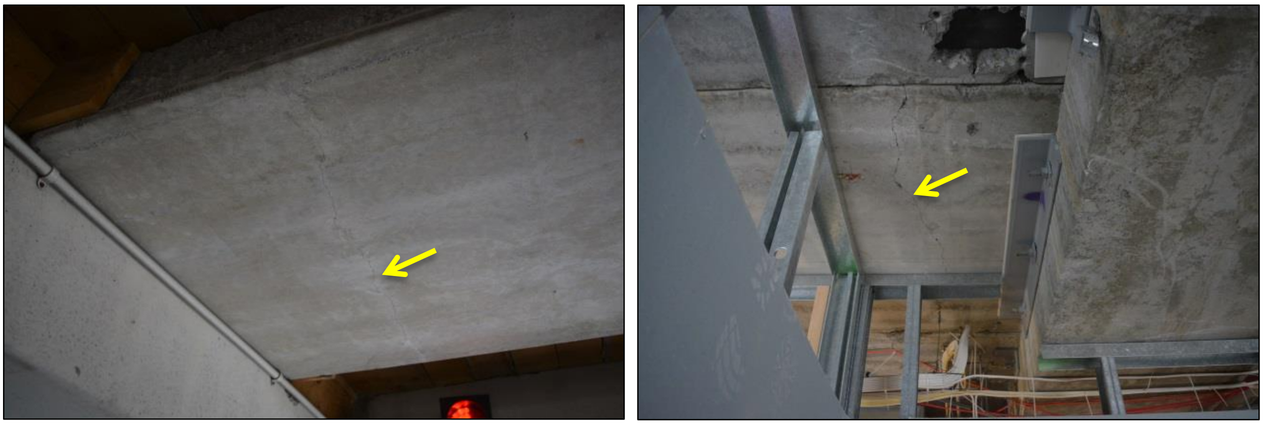
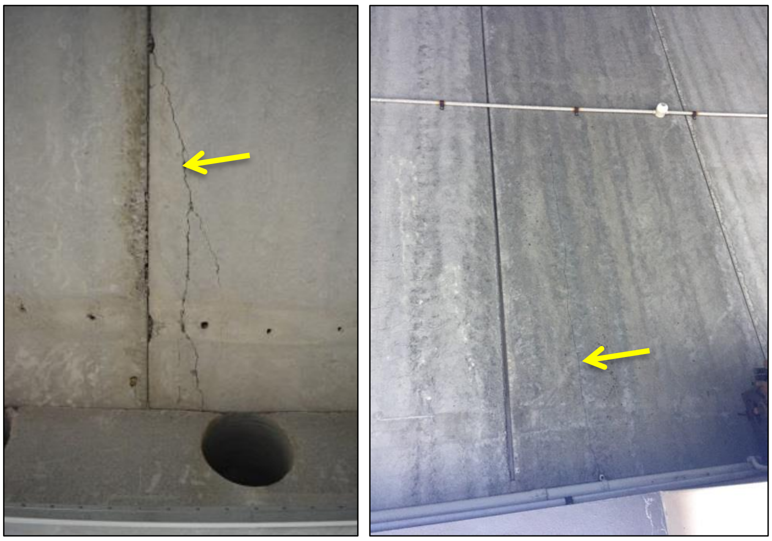


Figure 16: Transverse cracking of hollowcore floor units away from support.

Figure 17: Diagonal cracks in hollowcore floor units.

Longitudinal Cracking of Hollowcore Floor Units

Topping cracks often propagate through the joints between units, but in some cases the position of hollowcore units relative to the beam hinges, and the limited capacity of unreinforced hollowcore unit to resist tension perpendicular to the unit, can result in longitudinal cracking along the length of the unit. Examples of longitudinal splitting cracks are shown in Figure 18. Some longitudinal cracks are isolated to the end of the unit, whereas others propagate almost the entire length. In addition, some mixed cases of longitudinal cracks propagating diagonally across units (through multiple cells) were observed. In general, longitudinal splitting cracks can be commonly found in buildings with hollowcore floor units. A survey of parking buildings in Auckland with hollowcore floor units identified several cases of longitudinal cracking, highlighting that longitudinal cracks can form due to loading other than earthquakes. Longitudinal cracks were not classified as a significant collapse risk as they do not significantly compromise the vertical load capacity of the hollowcore unit.



(a) Cracks only at ends

(b) Wide cracks along entire length

Figure 18: Longitudinal cracks in hollowcore floor units.

Concern has been expressed for the possible presence of longitudinally web splitting cracks, which, when accompanied by transverse cracking, can result in life safety hazards from falling portions of the hollowcore unit. Although longitudinal web splitting cracks were only reported in two TDE buildings, such damage is hard to detect as it may not be accompanied by surface expression cracks.

SEISMIC RETROFITS

A number of buildings in Wellington had previously been subjected to some degree of seismic strengthening 44% (28) of TDE buildings). Retrofits implemented for precast concrete floor diaphragms typically consisted of extended seating of hollowcore units using a steel angle or rectangular hollow section (RHS), beams to support vulnerable alpha units directly adjacent to frame, and additional ties across the floor diaphragm. Examples of such retrofits are shown in Figure 19.

The use of an angle or RHS to extend the seating length may help to prevent loss of support due to elongation and/or support beam spalling, but does not address the potential for failure of the precast unit. As observed in buildings in Wellington, transverse cracking in hollowcore unit can occur a significant distance away from the support (>100 mm) and beyond the extended seating. Additionally, if the steel angle or RHS is placed hard against the precast unit, the increased restraint and prying forces might actually increase the risk of transverse cracks developing in the unit.

SUMMARY AND FUTURE RESEARCH NEEDS

Observations of typical structural damage to Wellington concrete buildings as a result of the 2016 Kaikoura earthquake have been presented. The earthquake dynamic characteristics resulted in high drift demands on some multi-storey storey moment frame buildings in the period range of 1-2 seconds. Deformation demands resulted in damage to floor diaphragms, in particular, reduced seating for precast floor units and cracking in hollowcore units.

In general, it was found that detailing recommended by current design standards [12] for precast unit supports resulted in a reduction in the collapse risk, despite not necessarily preventing support damage for buildings with high ductility demands.

In many cases the observed damage aligned closely to damage patterns identified during prior research; however, some inconsistencies were also observed (most notably transverse cracking in hollowcore units approximately 300 mm from the support beam). The apparent randomness of damaged hollowcore locations in some buildings has made it difficult to identify the exact cause of damage. The severity of different types and locations of transverse cracking in hollowcore units and the residual load capacity is a topic that requires additional research and testing in order to provide improved guidance for damage assessment, repair, and retrofits.

In light of the observed damage, the seismic assessment of buildings with precast concrete floors should be further examined to determine if appropriate conservatism is being implemented when providing %NBS ratings. In particular, the identification of possible load paths for diaphragm forces after mesh fracture deserves further consideration.

Retrofits to precast units with insufficient seating and hollowcore (alpha) floor units adjacent to frames have already been implemented in several Wellington buildings. The objective of these retrofits was to prevent collapse of floor units. Despite damage to precast units described in this paper, the Kaikoura earthquake did not test the performance of these implemented retrofits as units did not drop onto supplemental supports. However, observed transverse cracking in hollowcore floor units beyond the typical supplemental support length (~100mm) calls into question the value of these retrofits for future earthquakes and highlights the need for research to understand the cause and significance of transverse cracking in hollowcore floor units observed in the Kaikoura earthquake.

(a) Steel angle at hollowcore floor unit support

(b) Steel RHS at hollowcore floor unit support

(c) Steel beams below alpha hollowcore unit

(d) Steel brackets across units in corner

Figure 19: Precast floor seismic retrofits.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the owners and engineers who kindly allowed them access to buildings to document damage. The authors also gratefully acknowledge Marta Giaretton, Mehdi Sarrafzadeh and Max Stephens for their assistance with the processing of data. The working group established to define the critical damage states is also acknowledged; working group members included: Ken Elwood, Nic Brooke, Des Bull, Alistair Cattanach, Rick Henry, and Peter Smith. Lastly, the authors appreciate Wellington City Council granting permission to publish high level data statistics related to the Target Building Assessment Programme. This is QuakeCoRE publication number 155.

REFERENCES

1 Bradley B, Wotherspoon L and Kaiser A (2017). “Ground motion and site effect observations in the Wellington region from the 2016 Mw7.8 Kaikoura, New Zealand earthquake”. *Bulletin of the New Zealand Society for Earthquake Engineering*, **50**(2): 94-105.

2 Brunsdon D, Hare J and Elwood K (2017). “Engineering assessment processes for Wellington buildings following the November 2016 Kaikoura earthquakes”. *Bulletin of the New Zealand Society for Earthquake Engineering*, **50**(2): 338-342.

3 PCFOG Committee (2009). *"Seismic Performance of Hollow-Core Floor Systems"*. Preliminary Draft, Department of Building and Housing, Wellington, NZ. http://www.nzsee.org.nz/db/PUBS/HollowCoreFloorSystems.pdf.

4 NZSEE, SESOC (2016). *"Wellington City Council Targeted Building Assessment Programme: Engineering Guidelines for Targeted Damage Evaluation following the November 2016 Kaikoura Earthquakes"*. New Zealand Society for Earthquake Engineering (NZSEE), and Structural Engineering Society (SESOC), Wellington, NZ. http://www.sesoc.org.nz/public_resources/Wellington-Targeted-Damage-Evaluation-Guidelines.pdf?285

5 Kestrel Group (2017). "*Wellington City Council Targeted Assessment Programme Following the Kaikoura Earthquake of 14 November 2016*". Technical Report, Kestrel Group, Wellington, New Zealand, 58 pp.

6 Marder KJ, Elwood KJ and Clifton GC (2017). "Post-earthquake residual capacity of damaged reinforced concrete buildings". *16th World Conference on Earthquake Engineering*, Santiago, Chile.

7 Fenwick R, Bull D and Gardiner D (2010). *"Assessment of Hollow-core Floors for Seismic Performance"*. Report 2010-02, Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, NZ. http://www.ir.canterbury.ac.nz/bitstream/handle/10092/4211/12626196_CNRE%20Assessment%20of%20Hollow-core%20Floors%20for%20Seismic%20Performance.pdf?sequence=1

8 Matthews J (2004). *"Hollow-core Floor Slab Performance following a Severe Earthquake"*. PhD thesis, Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, NZ.

9 Corney SR, Henry RS and Ingham JM (2014). "Performance of precast concrete floor systems during the 2010/2011 Canterbury earthquake series". *Magazine of Concrete Research*, **66**(11): 563-575.

10 MBIE (2017). "*Investigation into the Performance of Statistics House in the 14 November 2016 Kaikōura Earthquake*". Report prepared by the Ministry of Business, Innovation and Employment (MBIE), Wellington, NZ.

11 Hare J, Fenwick R, Bull D and Built R (2009). "Precast double tee support systems". *SESOC Journal*, **22**(1): 10-44.

12 SNZ (2006). *"NZS3101: Concrete Structures Standard"*. Standards New Zealand, Wellington, NZ.

13 SNZ (1995). *"NZS3101: Concrete Structures Standard (Amendment No. 3)"*, Standards New Zealand, Wellington, NZ.

14 Jensen J (2006). *"Experimental Investigation of Existing Hollow-core Seating Connections Pre and Post Retrofit"*. ME Thesis, Department of Civil and Natural Resource Engineering, University of Canterbury, Christchurch, NZ.

15 Woods L (2008). *"Significance of Negative Bending Moments in the Seismic Performance of Hollow-core Flooring"*. ME Thesis, Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, NZ.