



Assessment of reinforced concrete buildings with hollow-core floors

A. Puranam, F. Bueker & K.J. Elwood

University of Auckland, Auckland.

ABSTRACT

The 2016 Kaikoura Earthquake highlighted the vulnerabilities of existing reinforced concrete (RC) frame buildings with precast floor systems to seismic demands. Since then, the NZ engineering community has investigated causes of damage, identified key failure modes, and proposed assessment guidelines (NZ Guideline, 2018) considering that the drift at which precast floors fail is likely to be smaller than the limiting drift of the reinforced concrete moment frames. This paper presents a short summary of the guidelines to estimate drift capacity associated with different failure modes of precast hollow-core floor units. The guidelines were used to study a dataset of existing buildings with hollow-core floor units in Wellington and the results suggested that the most likely mode of failure was loss of seating.

1 INTRODUCTION

Construction of buildings with precast floors in New Zealand began in the early 1970's, and was supported by the construction boom in the 1980's, around the same time flexible or "ductile" framing was gaining momentum (Brunsdon, 2017). These buildings consisted of floor systems made of precast slab units (hollow-core, double-tee, rib and infill, flat slab) seated on ledges on supporting beams. A topping slab with light reinforcement was cast in-situ (Figure 1a) and connected the precast units to supporting beams through "starter" bars. When subjected to earthquake demands, buildings deform and inter-story drift demand is accommodated by rotation of the support beams and elongation of the beam parallel to the span of the floor units (Figure 1b). In this scenario, precast floors can be vulnerable to damage in the form of unseating of the unit or failure near support connection, depending on the flexibility of the structure and the detailing at the supports.

Damage to precast systems observed after the 1994 Northridge Earthquake (Iverson and Hawkins, 1994) led to a large amount of research (both at the component-level and the system-level) in New Zealand in the 90's and 2000's (Fenwick et al., 2010). The results of these research studies led to vast improvements in the standards for design of precast systems in New Zealand (NZS 3101: 2006, 2006).

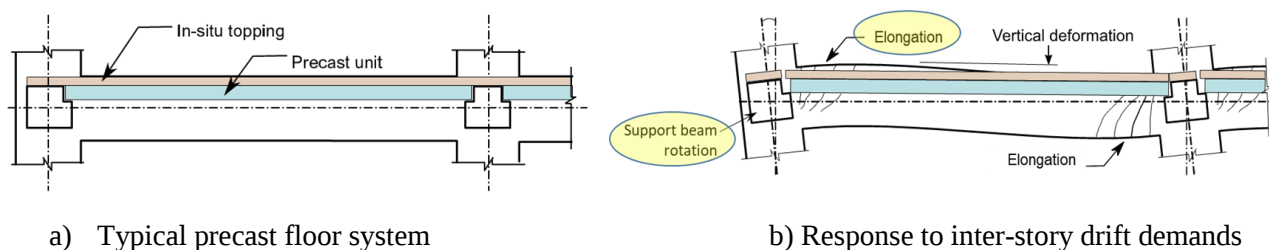


Figure 1: Precast floors (Fenwick, 2010)

Nevertheless, damage observed after the 2016 Kaikoura Earthquake (Henry et al, 2017) brought to attention the urgent need to assess and retrofit the large number of existing buildings with precast floor systems. The engineering community responded to this need by studying the observed damage, re-examining past test data, and initiating research programs to investigate retrofit and repair options. One product of this effort is a technical proposal to revise Section C5 (NZ Guideline, 2018) of the “Guidelines for Detailed Seismic Assessment of Buildings not identified as potentially Earthquake Prone (EPB)” including Appendix C5E on assessment of buildings with precast floor systems.

Appendix C5E was developed considering that a combination of flexible framing and soft soils can lead to large drift demands which exceed the drift capacity associated with failure of precast units. Appendix C5E provides detailed guidelines to identify drift capacities associated with plausible failure modes in different types of precast floor systems including hollow-core, double-tee, rib and infill, and flat slab. Drift demands can be estimated as a function of the stiffness/ flexibility of the structure and the intensity of the ground motion. Comparing the demands with capacities obtained using Appendix C5E may help identify retrofit options. This paper presents some highlights of the guidelines for assessing hollow-core floors. The new guidelines are also used to study a dataset of existing buildings with hollow-core floors in Wellington (DBH, 2006) for an understanding and distribution of controlling failure modes.

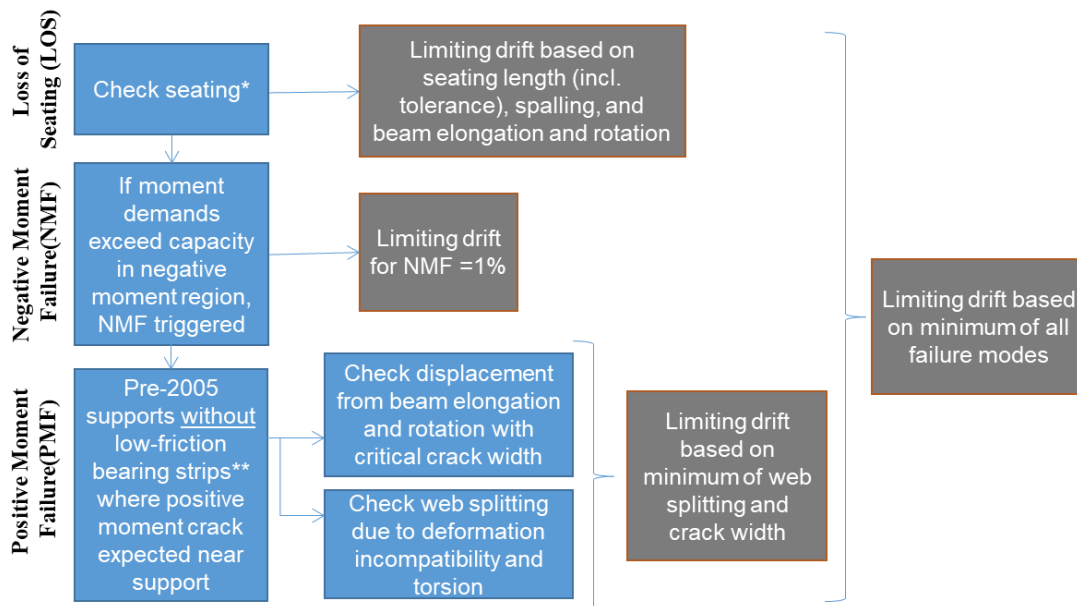
2 ASSESSMENT GUIDELINES FOR RC BUILDINGS WITH HOLLOW-CORE FLOORS

The guidelines for assessment of existing precast concrete floor systems (Appendix C5E) were produced by a group of researchers, practitioners, and policymakers and were published in late 2018 (NZ Guideline, 2018). Since then, a summary of Appendix C5E has been presented in a series of seminars by Concrete NZ (Concrete NZ, 2018). This section of the paper highlights some portions of Appendix C5E and the Concrete NZ seminar notes related to hollow-core floors. Readers should note this is only a summary and that they are recommended to refer to Appendix C5E (NZ Guideline, 2018) for complete information.

An overview of the procedure for assessing hollow-core floors is shown in Figure 2. Three plausible failure modes (LOS, NMF, and PMF shown in Figure 3) are considered and the corresponding story drift capacities are to be estimated. The smallest of the calculated drift capacities controls. In practice, this value may then be compared with plausible drift demands to evaluate need for or type of retrofit.

As mentioned in Section-1, precast floor systems have to accommodate deformation at supports caused by: (1) elongation of beams parallel to the units and (2) rotation of the support beam. The guidelines provide expressions to calculate this deformation as a function of story drift ratio. Limiting drift associated with loss of seating (LOS) is then estimated as the story drift ratio at which the calculated deformation caused by beam elongation and support rotation exceeds the available seating length. Similarly, limiting drift associated with positive moment failure (PMF) (in units which do not experience torsion or vertical deformations) is the drift ratio at which the calculated deformation exceeds a threshold value equal to diameter of pre-stressing strand.

Negative moment failure (NMF), on the other hand, is considered to be triggered if moment demand exceeds the capacity in the region near the end of the starter bars. If the demand exceeds the capacity, limiting drift for this mode of failure is set at 1%. The following sections summarize the methods to estimate deformations at supports and drift capacities related to the three failure modes.



* Seating need not be checked if supplemental support can be provided by two anchored R16 bars

** Supports with low-friction bearing strips, positive moment crack can be assumed to be suppressed.

Figure 2: Determining limiting drift capacity of hollow-core floors (Concrete NZ, 2018)

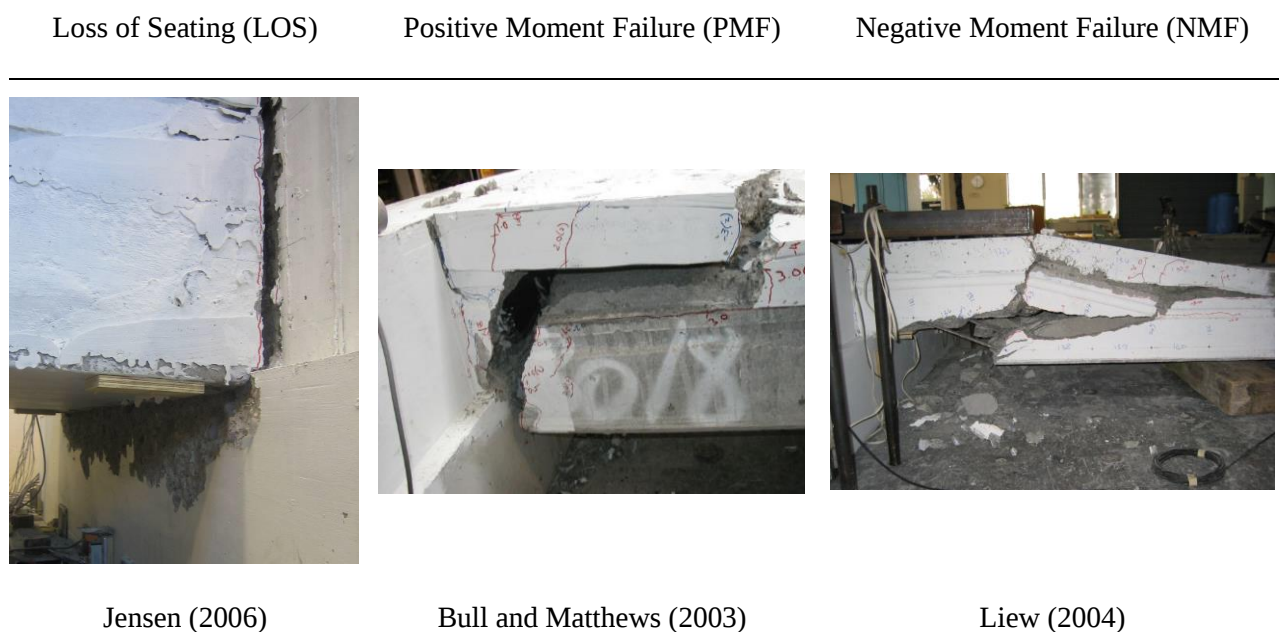


Figure 3: Observed failure modes in hollow-core floors

2.1 Estimating deformation at supports

Total deformation at supports (δ_{tot}) is estimated as a function of story drift ratio considering the location of the unit (Figure 4). Effects of beam elongation (δ_{el}), and support beam rotation (δ_{r1} , δ_{r2} , δ_{el_unit}) are considered in “elongation zones”. For units not within elongation zones, only the deformations caused by rotation of support beam (δ_{r2} , δ_{el_unit}) are considered.

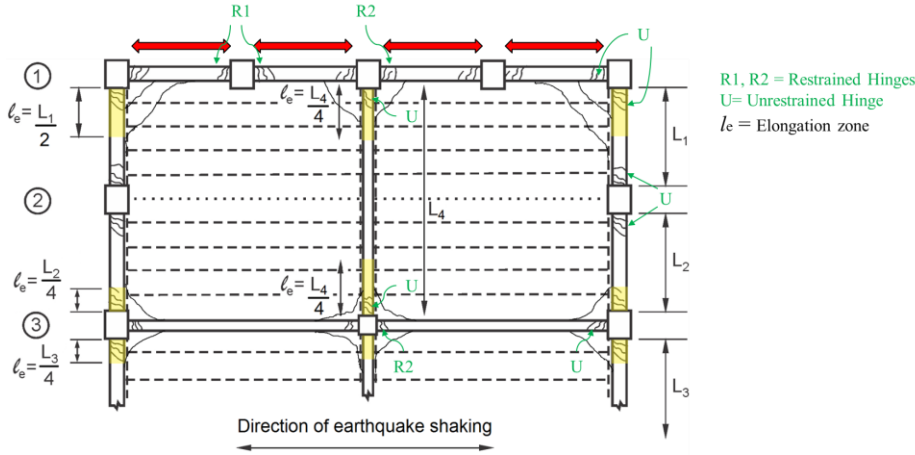


Figure 4: Locations of floor units and hinges (NZ Guideline, 2018)

At locations where not much restraint is provided by the floor slab and the framing system (unrestrained hinges, Figure 4), elongation at mid-depth of beams parallel to the floor units can be estimated as:

$$\delta_{el} = \max(0.005h_b, 2.6 \frac{\theta_p}{2} (d - d')) \leq 0.036h_b \quad (\text{for reversing plastic hinges}) \quad (1)$$

$$\delta_{el} = \max(0.005h_b, \frac{\theta_p}{2} (d - d')) \leq 0.036h_b \quad (\text{for unidirectional plastic hinges}) \quad (2)$$

where h_b is beam depth, d and d' are the distances from the outermost fibre in compression to the centroid of tension and compression steel, and θ_p is plastic rotation in beam plastic hinge caused by story drift ratio (θ). For restrained hinges, elongation at mid-depth of beam may be taken as half the value of unrestrained hinges.

Movement between the precast unit and the support ledge caused by rotation of support beam is estimated as:

$$\delta_{r1} = \left(\frac{h_b}{2} - h_L \right) \theta \quad (\text{within the elongation zone}) \quad (3)$$

$$\delta_{r2} = h_L \theta \quad (\text{outside the elongation zone}) \quad (4)$$

where: h_L is the depth of unit and topping and θ is beam rotation.

Plastic strain in the starter bars will cause additional movement of the unit on the support ledge:

$$\delta_{el_unit} = 1.3 \frac{\theta_p}{2} (h_L - d') \leq 0.018h_L \quad (5)$$

where: $\theta_p = (\delta - \delta_e) * \frac{L}{(L-2s)}$, δ is total drift, δ_e is elastic drift, L is unit length, s is distance from column centreline to face of ledge, h_L is depth from seating to top of beam, d' is cover to centreline of starter bars.

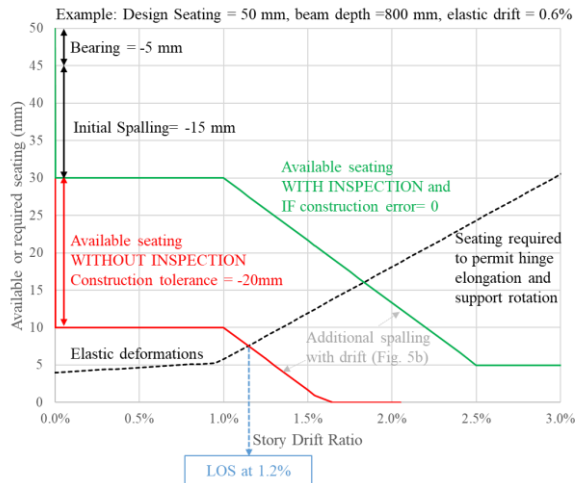
From Eq. 1-5, total movement of precast floors relative to the supporting ledge may be estimated as:

$$\delta_{tot} = \max[\Sigma \delta_{el} + \delta_{r1}, \delta_{r2} + \delta_{el_unit}] \quad (\text{for units supported within the elongation zone}) \quad (6)$$

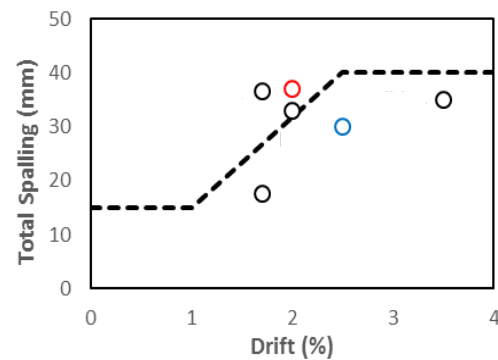
$$\delta_{tot} = \delta_{r2} + \delta_{el_unit} \quad (\text{for units supported outside the elongation zone}) \quad (7)$$

2.2 Loss of Seating (LOS)

The total deformation at support estimated using Eqs. 6 and 7 is compared with available seating length to determine the story drift ratio at which LOS is likely to occur (example shown in Figure 5a). Available seating length is estimated from the specified seating length considering: 1) construction tolerance (20mm if the seating is not inspected to confirm actual seat length), 2) seating required for bearing (minimum of 5 mm), and 3) spalling caused by rotation of the unit relative to the support (estimated using the approximation from test data shown in Figure 5b). For the example in Figure 5, LOS failure is estimated to occur at 1.2%.



(a) Sample calculation for LOS



(b) Spalling vs. drift from tests (courtesy S. Corney)

Figure 5: Drift capacity associated with loss of seating (LOS)

2.3 Positive Moment Failure (PMF)

Units placed on mortar pad or directly on concrete support ledge (i.e. pre-2005 code details without low-friction bearing strips) and poor bond to strand may be susceptible to transverse positive moment cracks (Figure 6). Limiting drift for failure in this situation (PMF) is taken as the smaller of the drifts causing: 1) the transverse crack width to exceed a critical value equal to strand diameter, and 2) cracking in webs because of incompatible displacements or torsion.

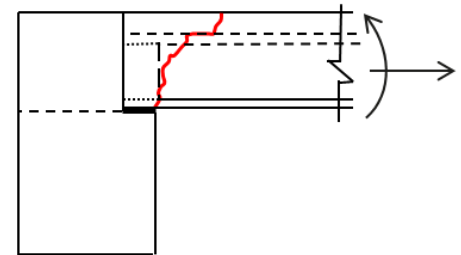


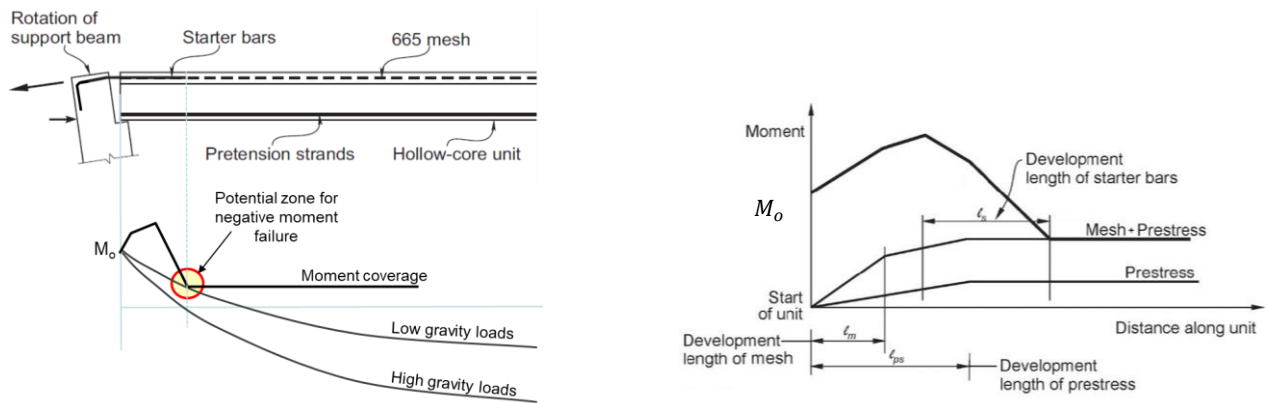
Figure 6: Positive moment crack

For units not subjected to differential movement or torsion, limiting drift is estimated as the story drift ratio at which the calculated support deformation from beam elongation and rotation (δ_{tot} from Eq.5, 6) exceeds the diameter of prestressing strand, typically 12.5 mm. Details to estimate limiting drift for cracking in web presented in Appendix C5E (NZ Guideline, 2018) are not discussed here for brevity.

2.4 Negative Moment Failure (NMF)

Negative moments develop at the support because of deformation compatibility as the supporting beam rotates away from the hollow-core unit. The discontinuity in strength near the end of starter bars may produce conditions in which the units are prone to negative moment failure where the starter bars terminate. Limiting drift for this failure is set to 1 % if the calculated moment demand exceeds the available moment coverage i.e. capacity (Figure 6). This is done considering two loading conditions (Figure 7) to estimate moment demand along the span and moment capacity at the end of unit (M_o). It should be noted that strong,

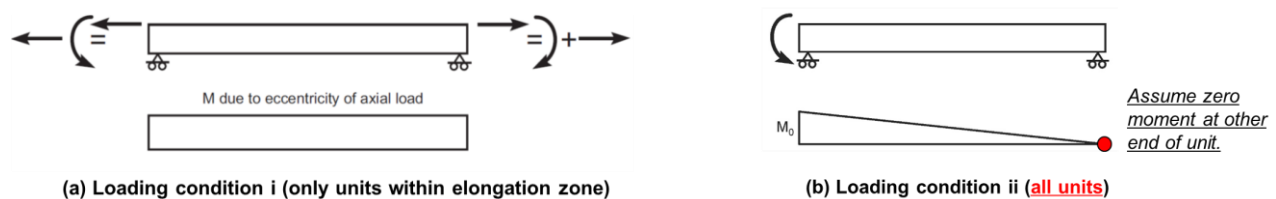
short starters are more likely to cause NMF than weak and/ or long starters. Also, counterintuitively, small gravity loads (or shorter spans) are more demanding than large gravity loads (as shown in Figure 6a).



(a) Potential zone for negative-moment failure

(b) Variation of moment coverage near supports

Figure 6: Negative moment failure in hollow-core units (NZ Guideline, 2018)



(a) Loading condition i (only units within elongation zone)

(b) Loading condition ii (all units)

Figure 7: Loading conditions to be considered for negative moment failure (Concrete NZ, 2018)

3 DATABASE OF EXISTING RC BUILDINGS WITH HOLLOW-CORE FLOORS

Hollow-core floors are estimated to comprise over 80% of the floor area of precast construction in New Zealand. Considering their widespread use in NZ, and results of tests at University of Canterbury, the Department of Building and Housing (DBH) conducted a study to determine the extent and usage of hollow-core floor systems in NZ and their vulnerabilities (DBH, 2006). The study by DBH produced a dataset of 112 RC buildings with hollow-core floors in Wellington. The guidelines discussed in Section 2 were used to study this dataset considering that the dataset was compiled using drawings and without identification of any retrofits. Some buildings may have been retrofit after compilation and as-built dimensions may vary.

A summary of the properties from the dataset is presented in Figure 8. Key observations include: 1) over 50% of the buildings have fewer than 6 stories, 2) approximately 50% of the buildings have RC frames classified as flexible or flexible-to-intermediate (i.e. may experience drifts larger than 1%), 3) 80% of the buildings have 200-mm deep hollow-core units, 4) over 80% of the buildings have floor spans not exceeding 10m, 5) in approximately 75% of the buildings, the floors span between two columns (single span) and about 20% span more than two columns (intermediate columns), 6) over 60% of the buildings have seating length not exceeding 50 mm, 7) typical spacing of external starter bars is between 200 and 400 mm, 8) over 60% of the cases have starter bar lengths not exceeding 600 mm, and 9) over 80% of the cases have a 65mm topping slab with 665 mesh.

Story drifts associated with three failure modes (LOS, PMF, NMF) were estimated for all buildings. For simplicity of computation or because of paucity of information available, the following assumptions were made: 1) beam depth = 800 mm, 2) beam plastic hinge length = 400 mm 3) column depth = 1000 mm, 4) HC depth = 200 mm, 5) elastic drift = 0.6%, 6) strand diameter = 12.5 mm, 7) topping = 65 mm with 665 mesh, 8) no intermediate column (i.e. single span), and 9) units are in elongation zone.

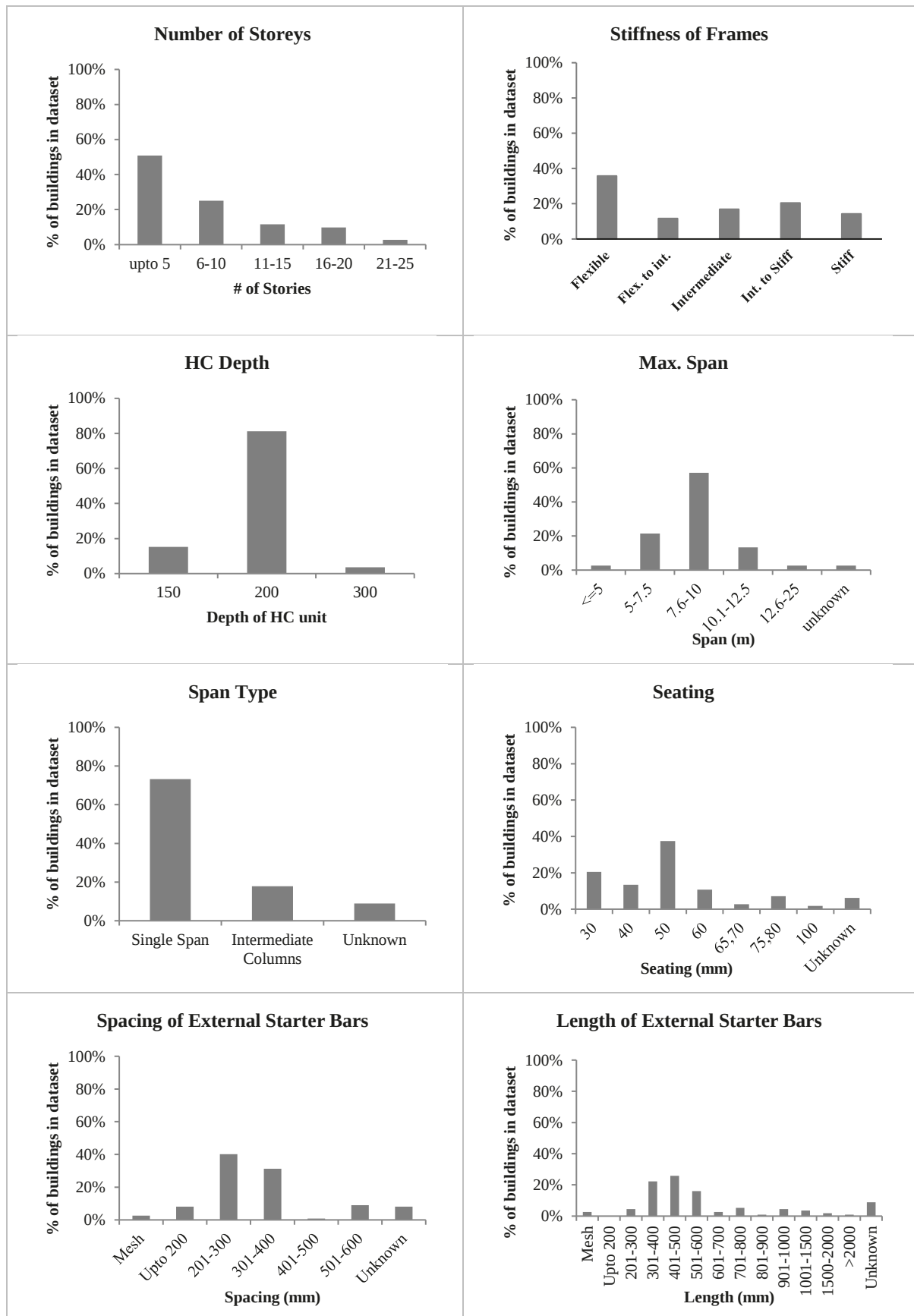
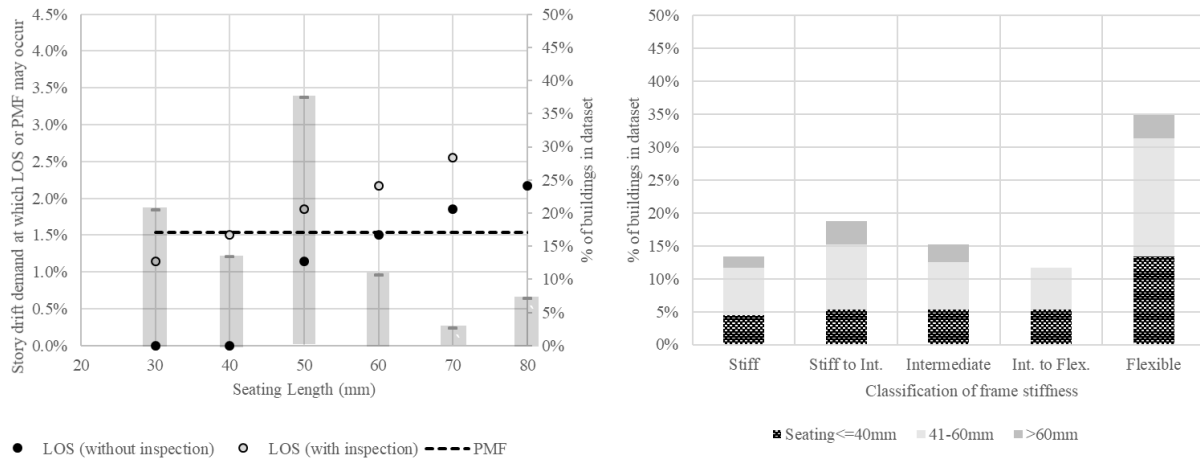


Figure 8: Summary of hollow-core dataset

Figure 9a shows the story drift capacities associated with LOS (dots) and PMF (dashed line), along with the seating provided for the buildings in the database. In addition to the assumptions stated above, a span length of 8m was assumed and the effects of torsion and vertical displacements were not considered. Figure 9a shows that LOS is likely to control when design seating length does not exceed 60 mm and no inspections are performed. This is the case for over 80% of the buildings in the dataset. It should be noted that small seating lengths are particularly critical in buildings with flexible frames as drift demand is inversely proportional to frame stiffness. Figure 9b indicates that ~ 30% of the buildings in this dataset were categorised as flexible with seatings of 60mm or less.



a) Drift associated with LOS and PMF

b) Seating length and frame stiffness

Figure 9: Statistics associated with LOS and PMF (Note: vertical bars represent % of buildings in dataset)

Possibility of NMF was assessed using the simplified tool shown in Figure 10. Values on the vertical axis represent the strength of the starter bars ($A_s f_o$) where A_s is the cross-sectional area of starter bars connecting each unit to the supporting beam and f_o is their probable yield stress multiplied by an over-strength factor of 1.25 (NZ Guideline, 2018). The horizontal axis represents the span of the unit. Threshold lines for the triggering of NMF are shown for two lap lengths (600- and 300-mm) and two loading conditions (Figure 7).

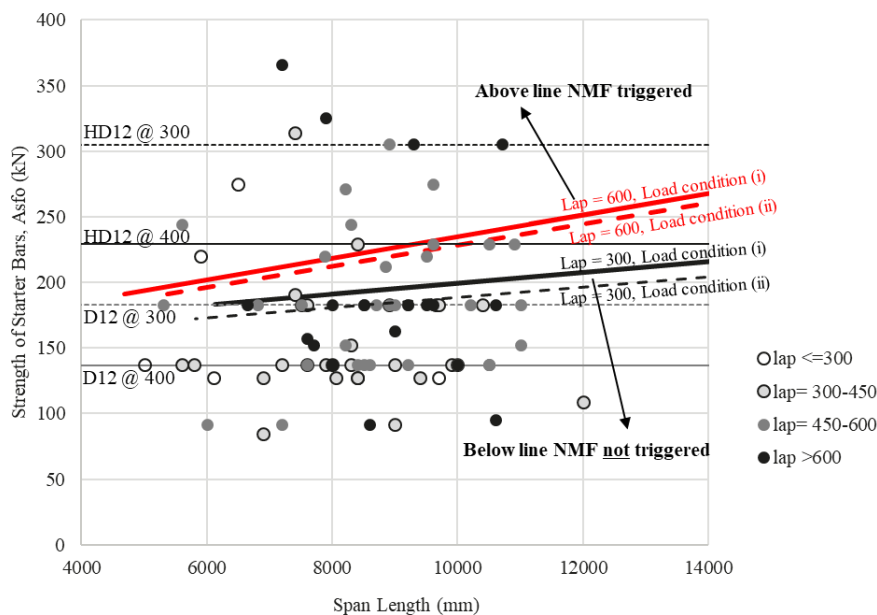


Figure 10: Assessing NMF (Note: threshold lines for selected lap lengths produced using Appendix C5E)

The circles represent buildings in the dataset assuming that the specified length of the starters is the lap length. NMF is assumed to be triggered for the points above the threshold line corresponding to their lap length and the drift capacity for these cases was taken as 1%. Figure 10 reinforces that NMF is more likely to occur in cases with stronger and/or shorter starter bars and shorter spans, but that NMF is anticipated in a small percentage of the overall dataset compared with LOS.

For the 112 buildings in the dataset, limiting drift capacity was estimated as the smallest of the values associated with LOS, PMF, and NMF. Due to missing information in the dataset, the drift capacity could not be calculated for LOS and NMF for 7 and 20 cases, respectively. For such cases, the limiting drift capacity was taken as the minimum for the failure modes for which drift capacity could be calculated.

Figure 11 shows the distribution of controlling failure modes and drift capacities for two scenarios: 1) without inspection and 2) with inspection of seating. If we assume no inspections are performed, LOS controls in approximately 70% of the cases (Figure 11a) and limiting drift capacities are smaller than 1.5% for approximately 85% of the buildings (Figure 11b). On the other hand, there is a shift in the most prevalent failure mode from LOS to PMF if we assume that inspections are performed (and there is no construction error) as evident from Figures 9a and 11a. In this scenario, drift capacities increase in some cases but are still smaller than 2% for all buildings. These results are dependent on the assumptions stated earlier, especially the depth of the supporting beam. Nevertheless, even the largest drift capacities shown in Figure 11b are concerning considering the permissible drift limits of 2.5% for structural frames in NZS 1170.5.

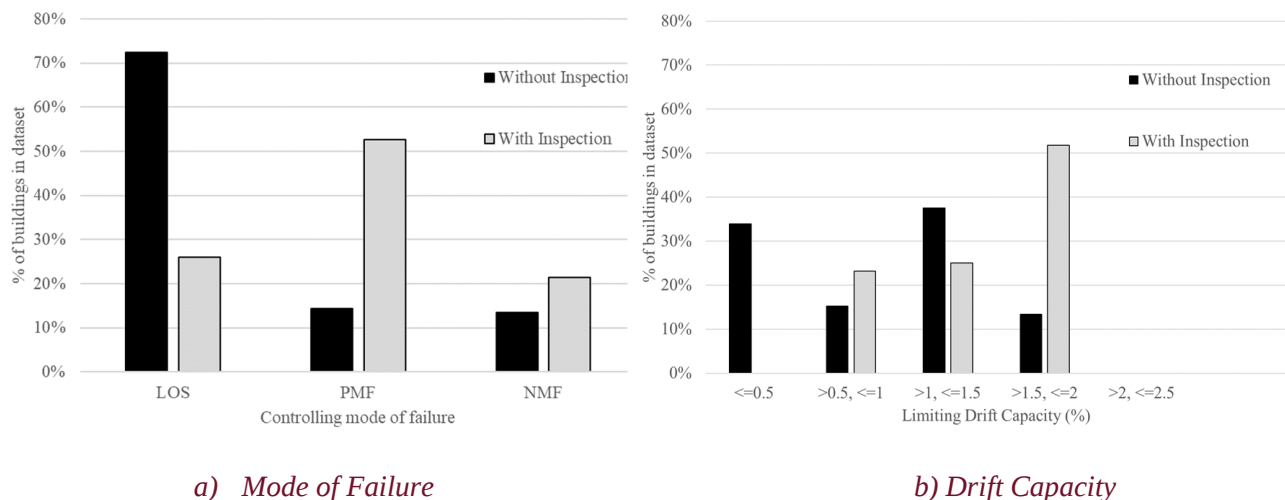


Figure 11: Distribution of controlling mode of failure and drift capacity

4 SUMMARY AND CONCLUSIONS

Highlights of new guidelines to assess buildings with hollow-core floors have been presented (NZ Guideline, 2018). Methods to estimate drift capacity associated with three failure modes (LOS, PMF, NMF) of precast hollow-core floor units were discussed. These methods were applied to a dataset of 112 building with hollow-core floors in Wellington to assess the most likely failure mode and associated drift capacity. The results of this study suggest that in over 70% of the buildings loss of seating (LOS) is likely to control when no inspections of the seating are performed. In all cases, limiting drift capacity for hollow-core floors was estimated to be smaller than 2% which is less than the drift limits for which structural frames are designed.

Note that the dataset used was compiled in 2006 without identification of retrofits. Some buildings may have been retrofit since the date of collection and hence would be expected to perform much better than suggested by the above results. Identification of un-retrofitted buildings with hollow-core floors should be considered a high priority.

5 ACKNOWLEDGEMENTS

The authors would like to acknowledge the support from Wellington City Council and the contributions of Jiatong Jiang and Kaiqi Li. The authors would also like to thank Des Bull for providing valuable critiques on the manuscript.

6 REFERENCES

- Bull, D. & Matthews, J. 2003. *Proof of Concept Tests for Hollow-core Floor Unit Connections*, Commercial Report, C2003-1, Dept. of Civil Engineering, University of Canterbury, Christchurch, New Zealand.
- Brunsdon, D., Hare, J. & Elwood, K. 2017. Engineering assessment processes for Wellington buildings following the November 2016 Kaikoura earthquakes, *Bulletin of the New Zealand Society for Earthquake Engineering*, Vol 50(2) 338-342.
- Concrete NZ. 2018. *Assessment of Existing Precast Concrete Floors*, Seminar Series 2018.
- Department of Building and Housing. 2006. *Hollow Core Floor Overview Report*. ISBN: 0-478-00000-0
- Fenwick, R., Bull, D. & Gardiner, D. 2010. *Assessment of hollow-core floors for seismic performance (2010-2)*. Christchurch, New Zealand: Dept. of Civil and Natural Resources Engineering, University of Canterbury. http://www.ir.canterbury.ac.nz/bitstream/handle/10092/4211/12626196_CNRE%20Assessment%20of%20Hollow-core%20Floors%20for%20Seismic%20Performance.pdf?sequence=1
- Henry, R.S., Dizhur, D., Elwood, K.J., Hare, J. & Brunsdon, D. 2017. Damage to concrete buildings with precast floors during the 2016 Kaikoura Earthquake, *Bulletin of the New Zealand Society for Earthquake Engineering*, Vol 50(2) 174-186.
- Iverson, J. & Hawkins, N. 1994. Performance of precast/prestressed concrete building structures during Northridge earthquake, *Precast concrete institute (PCI) journal special report*, Vol 39(2) 38-55.
- Jensen, J. 2006. *The seismic behaviour of existing hollowcore seating connections pre and post retrofit*, ME thesis, Dept. of Civil Engineering, University of Canterbury, Christchurch, New Zealand.
- NZ Guideline. 2018. *Part C5: Technical Proposal to Revise the Engineering Assessment Guidelines*. Retrieved from: <http://www.eq-assess.org.nz/dsa-non-epb/>
- Standards New Zealand. 2006. *Concrete Structures Standard, NZS 3101 2006, including amendments 1 and 2 (2008)*, Wellington, New Zealand.