TORSIONAL CAPACITY ASSESSMENT OF PRECAST HOLLOW-CORE FLOORS

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ABSTRACT

Estimation of the torsional capacity of precast hollow-core floor units is required as a step for assessing the expected seismic performance of these floors according to the seismic assessment procedure followed in New Zealand. Due to limited research on the torsional behaviour of hollow-core units, there are multiple uncertainties regarding the accuracy of the procedure currently used to assess the torsional capacity of hollow-core floors. This paper discusses the basis and limitations of the procedure adopted in the New Zealand assessment guidelines (C5) for quantifying the torsional capacity of hollow-core floor units, describes potential implications of the limitations of the available methodology on the assessed torsional capacity accuracy and potential impact on other hollow-core seismic failure modes, and provides a twist-limits chart as a simple tool to assess the torsional deformation capacity of typical hollow-core unit depths based on the methodology available in the current assessment guidelines. The twist-limits chart provides a useful assessment tool for engineers, but must be used with due consideration of the limitations of the torsional capacity assessment methodology discussed herein. Informing the judgement of how and when to use this twist-limit information in the assessment process, given the limitations and uncertainty of conditions in the field, remains a key challenge for future research on the seismic assessment of precast floors.

1 INTRODUCTION

The lack of shear reinforcement in precast hollow-core floor units due to the extrusion process is the primary reason for the susceptibility of these floors to sustaining severe damage under seismic demands, as described in Fenwick et al. (2010). This lack of shear reinforcement makes hollow-core floor units inherently vulnerable to brittle failure due to the application of torsional demands. Consequently, the torsional capacity of hollow-core units depends significantly on the relatively low angles of twist they can resist before torsional cracking occurs, as once torsional cracking is initiated the flexural and shear strength of the hollow-core floor unit are compromised.

Various studies have been conducted on hollow-core floor units subjected to flexural and shear actions (Walraven and Mercx 1983; Yang 1994; El-Sayed et al. 2019; Michelini et al. 2020). However, little research has been conducted on the behaviour of hollow-core floor units undergoing torsion or shear-torsion actions. Previous research regarding the torsional performance of hollow-core floor units included experimental testing of bare hollow-core units (i.e. without topping) subjected to pure torsion (Pajari 2004a) and shear-torsion demands (Pajari 2004b). The experiment results were then used to validate and calibrate a finite element model (Broo et al. 2007), which was then used in a parametric study where the capacity of 200 mm and 400 mm hollow-core units with different shear and torsion demands were investigated. Then both the experimental and numerical results were compared with the analytical methodology available in the European standard for hollow-core design EN-1168 (British Standards Institution (BSI) 2005) by Broo et al. (2005).

Although previous work has provided some basis for understanding the torsional performance of hollow-core floors, it does not provide enough information regarding the torsional behaviour of hollow-core units in a seismic event in which the torsional demands are induced to the units from the supporting structure through the connection rather than an eccentric gravity load. There is considerable uncertainty about the torsional performance of hollow-core units in floors under seismic demands. This uncertainty is primarily due to limited research and a general lack of information regarding the torsional response of these floor units and lack of information on how different floor to support connections affect the torsional demand induced to the units. Due to the current limited state of knowledge, the current structural concrete design standard, NZS3101:2006, simply contains a caution on using these units where appreciable twisting may occur (clause C9.4.3.6) (Standards New Zealand 2017).

Moreover, many buildings incorporating hollow-core floors, especially those constructed in areas of high seismicity, need to be assessed for their seismic capacity. The torsional capacity of these floor units has to be assessed as part of the seismic assessment procedure (MBIE et al. 2018; Puranam et al. 2021).

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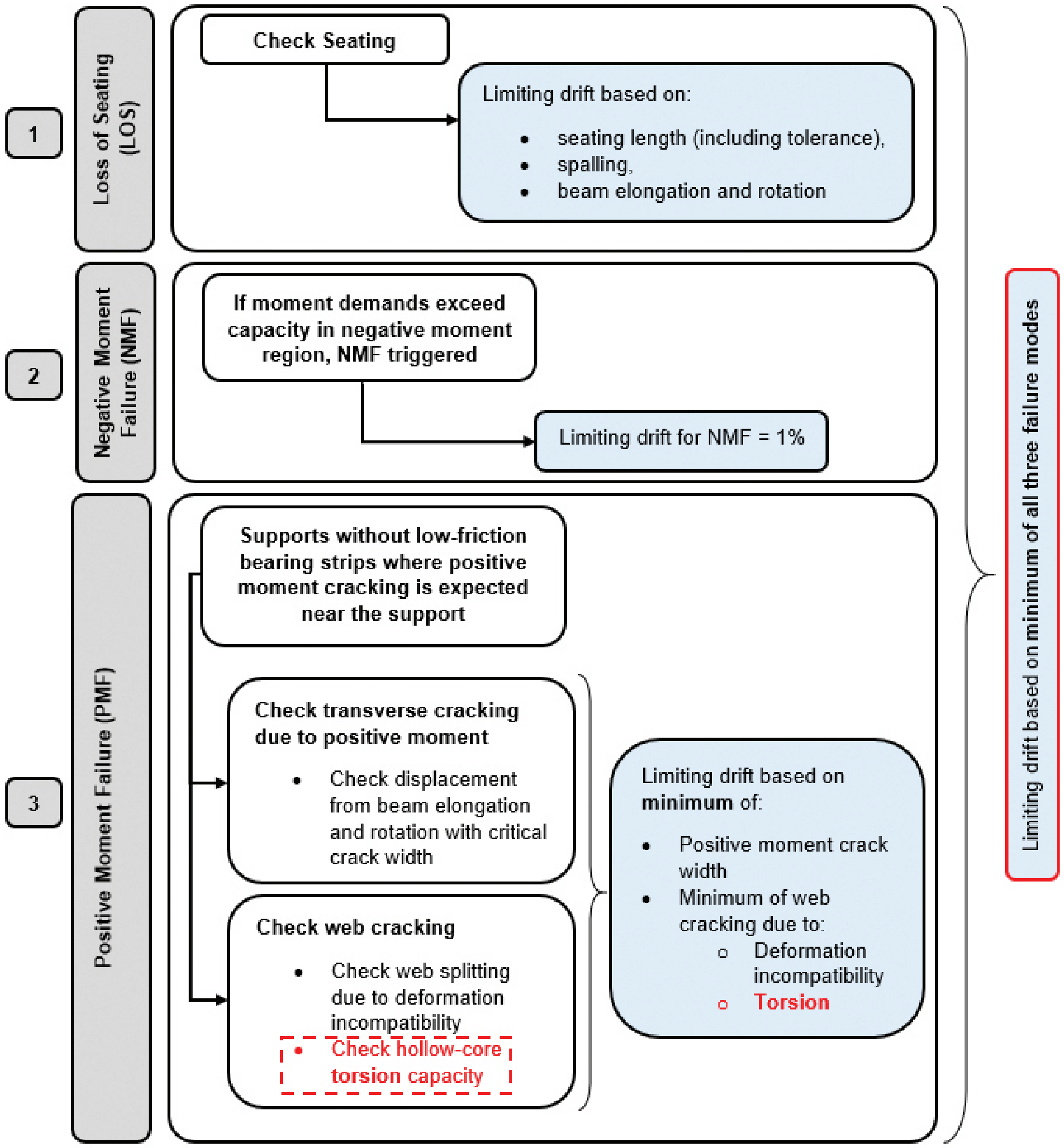

Figure 1: Summary of hollow-core floors assessment procedure according to the New Zealand Seismic Assessment Guidelines (C5) modified from MBIE et al. (2018)

Furthermore, it was noted that hollow-core torsional capacity can result in a low seismic rating of the floor in areas of low seismicity, which can lead to consideration of retrofit. This realisation highlights the importance of better understanding the torsional behaviour of hollow-core floor units as it affects rated performance in low seismic zones, possibly triggering retrofit when the remaining hollow-core failure modes scores were satisfactory.

Currently, the New Zealand seismic assessment guidelines (C5) consider the susceptibility of hollow-core floor units to sustain torsional damage only when web cracking due to torsion is accompanied by transverse cracking through the bottom flange of the hollow-core unit. Hence, torsion is only considered under the positive moment failure check, as shown in Figure 1, instead of treating torsional damage as a potential failure mode itself. It is worth noting that, currently, checking if a low-friction bearing strip was incorporated in the connection detail is considered a binary check that decides whether or not a unit is deemed susceptible to Positive Moment Failure (PMF) and consequently torsion as well. This binary check is due to the assumption that a low-friction bearing strip suppresses transverse cracking of the bottom flange of the unit.

Torsional demands can be imposed on one or more hollow-core units in floors in multiple situations, more relevantly, where the seismic response of a building could cause twisting (torsion) of these units about their longitudinal axes due to differential rotations of the supporting structure at each end of the unit. Some examples of when significant twisting of the units might occur include but are not limited to:

- One end of a unit supported on a link of an eccentrically braced frame.
- One end of a unit supported on a cantilever beam in buildings with moment resisting frames that do not have corner columns (a and d in Figure 2).
- One end of a unit supported on a coupling beam.
- One end of a unit supported on a shear wall and the other end supported on a frame (b in Figure 2).
- One end of a unit is slanted or supported on a skewed supporting element (c in Figure 2).
- Units supported within the plastic hinge zone of frames with staggered columns, where one end is supported on a column and the other on a beam's plastic hinge (e in Figure 2).
- One end of a unit is supported within the plastic hinge region of a seismic frame, and the other end of the unit is seated intra-span of a gravity frame resulting in different end rotation demands.

This paper provides a twist-limits chart and table as a simple tool for assessing the torsional deformation capacity of different hollow-core unit depths according to the available assessment procedure described in the seismic assessment guidelines C5 (referred to as the 'Yellow Chapter' within the Structural Engineering community in New Zealand) (MBIE et al. 2018). Furthermore, the basis of the assessment methodology adopted in assessment guideline C5 and the assumptions used to determine the torsional capacity are described in this paper. Limitations of the torsional assessment procedure and how these limitations might impact the torsional capacity assessment of the floor are highlighted so that prudence can be practised as necessary when assessing the torsional capacity of these floors.

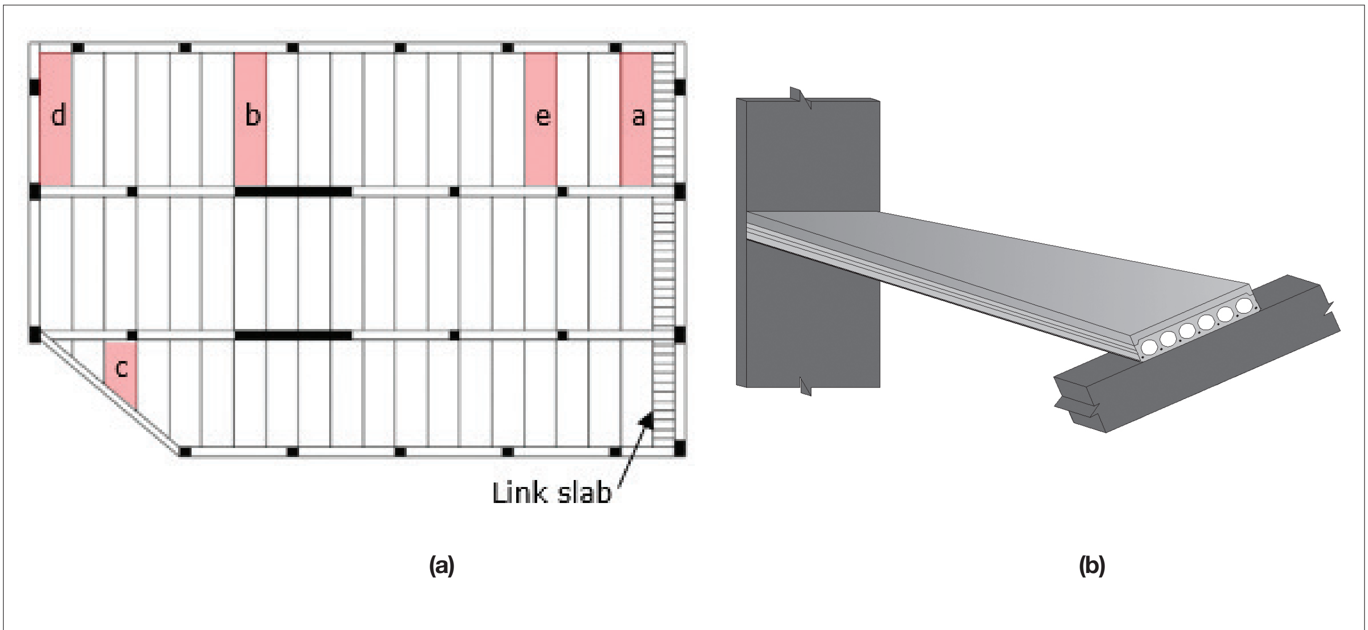

Figure 2: (a) examples of when torsional demands can be induced into a floor unit under due to the seismic response of the building (b) illustrative example

2 HOLLOW-CORE TORSIONAL CAPACITY ASSESSMENT

2.1 BACKGROUND

The torsional limit state for hollow-core units is governed by the twisting angle of the unit about its longitudinal axis, where the critical twisting angle (capacity) should be used to calculate a maximum allowed inter-storey drift at which that critical twist is reached in the unit. The approach adopted in the assessment guidelines C5 uses the principle of stationary total potential energy to relate the unit torsional stiffness to the applied torque, where the limiting twist angle is deduced by equating the work done by the external forces (Equation 1) and the internal strain energy due to the shear force generated in the unit (Equation 2).

$$W = \frac{1}{2} * T * \theta \qquad (1)$$

$$U = \frac{1}{2} * F * \gamma \qquad (2)$$

Where,

U Strain energy (due to shear)

W Potential energy of the load (work done by torque)

T Torque

θ Angle of rotation of the hollow-core unit about its longitudinal axis

F Shear force due to the shear flow generated in the unit as a result of the unit twisting/torsion

γ Shear strain

Whilst shear demands from gravity loads are assumed to be uniformly distributed along the hollow-core unit webs as shown in Figure 3a, when a unit is subjected to torsion the shear stresses are primarily generated in the perimeter of the section. Hence it is reasonable to analyse the section as a thin-walled tube in which torsion is resisted by a shear flow as shown in Figure 3b. Bredt's thin tube theory, as explained in Collins & Mitchell (1997), is used to theoretically estimate the hollow-core torsional cracking moments. The expression used in the assessment guidelines C5 is given by (Equation 3).

$$q = \frac{T}{2A_{\circ}} = \tau * t \qquad (3)$$

Where,

τ Shear stress

q Shear flow

$A_{\circ}$ Area enclosed by the centreline of the tube cross section

T Torque

t Tube wall thickness

Using thin tube theory includes multiple simplifications and assumptions, which are summarised below:

- All calculations assume an uncracked section.
- Assume that the hollow-core unit is subjected to pure torsion (i.e. not accounting for the interaction of gravity shear forces).
- Use of thin tube theory implicitly implies ignoring the section distortions (i.e. shear deformation perpendicular to the torsional twisting).
- The intensity of the shear stresses varies across the thickness of the assumed tube. Since the tube is thin, it can be assumed that the shear stress is constant across the thickness of the tube.

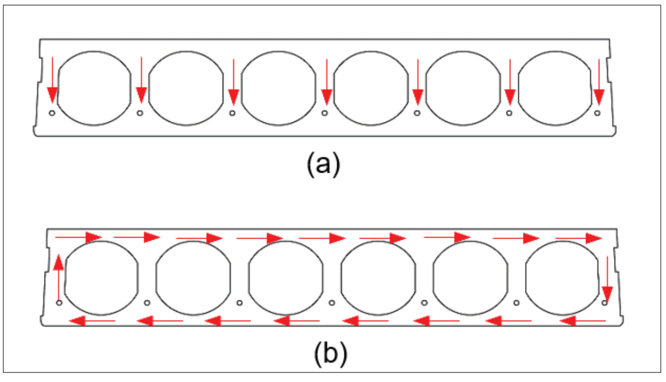

Figure 3: (a) sketch showing uniform shear stress distribution among hollow-core webs due to gravity shear (b) sketch showing shear distribution in hollow-core due to torsion

Under the assumption of pure torsion, torsional cracking is assumed to occur when the principal tensile stress at the critical point reaches the tensile strength of concrete. Therefore, using Mohr's Circle, the maximum shear stress the section can withstand before cracking is defined by (Equation 4).

$$\tau_{cr} = f_{ct}\sqrt{1 + \frac{f_{pc}}{f_{ct}}} \qquad (4)$$

Where,

f_{ct} Concrete tensile strength

f_{pc} Precompression stress at the at the neutral axis in the critical section

τ_{cr} Cracking shear stress

The cracking torque, τ_{cr}, is calculated by substituting the cracking shear stress obtained from Equation 4 into Equation 3.

$$\tau_{cr} = 2 * q * t * A_{\circ} \tag{5}$$

The external work done (Equation 1) should be a function of the cracking angle of twist, θ_{cr}, where it will be equated with the internal shear strain energy to obtain the cracking angle of twist, as shown in (Equation 6). Appendix B discusses the derivation of the equations used herein for elaboration.

$$\theta_{cr} = \frac{2*U}{T_{cr}} \tag{6}$$

The critical section for assessing the torsional capacity of the unit will be within the transfer length of the prestressing strands. It is difficult to exactly calculate the value of longitudinal stress applied to the critical section as the longitudinal stress will depend on the strands' prestress and on the bending moment, which varies continuously during an earthquake. Consequently, for practical purposes, the conditions leading to torsional cracking can only be assessed to be in a likely range of structural actions (Bull et al. 2009). Therefore, the longitudinal stress at the critical section is assumed to be equivalent to the precompression stress applied on the section due to one-third of the effective prestressing force from all the prestressing strands after long term losses have occurred as a best estimate.

Torsional cracking in a concrete member without torsion reinforcement can result in the collapse of the member. However, as shown in Figure 4, two out of four pure torsion tests conducted by Pajari (2004a) showed that the hollow-core units sustained between two and four times the twist corresponding to torsional cracking before an abrupt and severe drop in torsional resistance (torsional strength capacity) took place. Based on these test results, Fenwick et al. (2010) recommends that the twist at torsional failure, θ_f, which is deemed to be accompanied by loss of gravity carrying capacity be taken as:

$$\theta_f = 2.50\theta_{cr} \tag{7}$$

It should be noted that the limited tests available focused on torsional failure or loss in torsional strength capacity, whereas assessing the torsional capacity of a hollow-core unit when subjected to seismic demands is in fact a compatibility torsion issue where we are ultimately interested in the potential loss of gravity support, not loss in torsional strength capacity. Furthermore, it is worth mentioning that all four tests failed by cracking the top flange at an angle of 45° with the longitudinal axis of the unit. This failure mode is not expected to occur in typical building applications due to the presence of gravity loads and the presence of a continuous concrete topping. Instead, a brittle shear-tension or strand anchorage failure would be expected. Brittle shear failure would be expected as the floor bending moment decreases the tensile stresses in the top flange, and prestressing strands reduce the tensile stresses in the bottom flange. At the same time, one of the outermost webs will be subjected to shear stresses due to the combination of gravity shear and torsion shear (Figure 3). These stresses, in combination with the thickness of the outermost webs relative to the top flange with concrete topping, indicate that a web shear failure should be expected to occur.

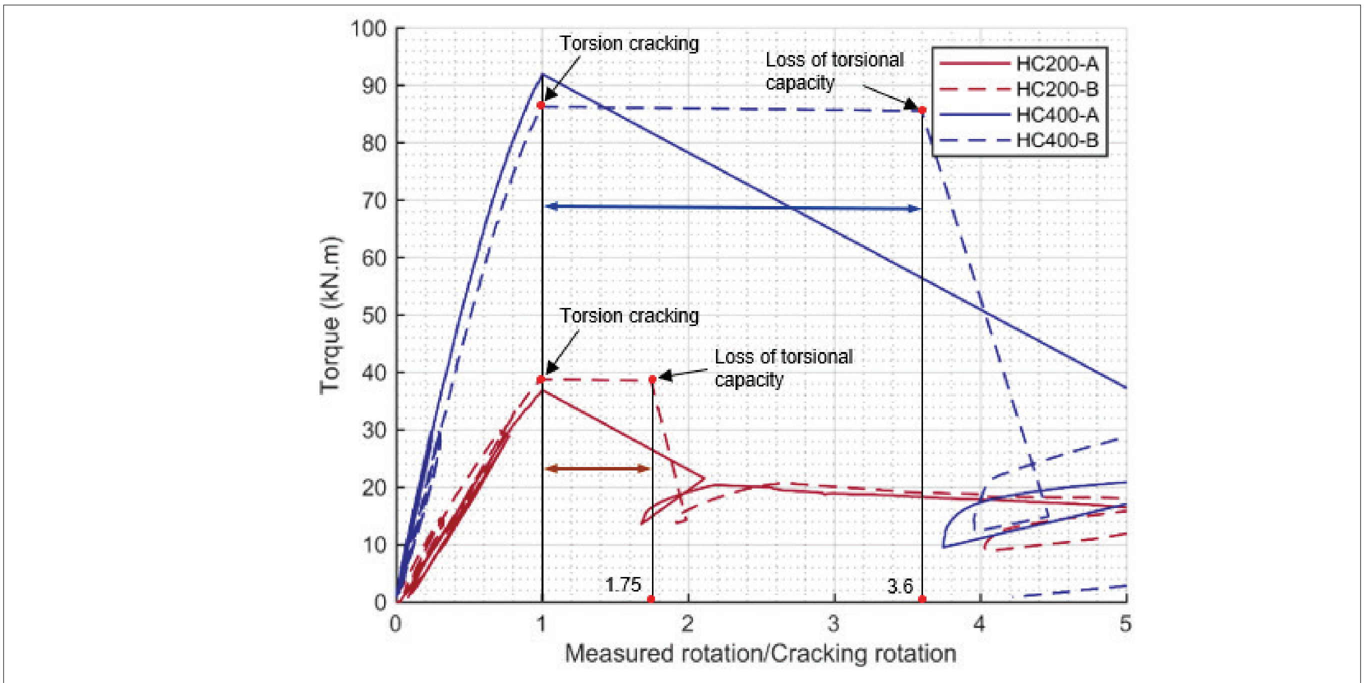

Figure 4: Pure torsion tests results modified from Pajari (2004a) showing two HC200 tests and two HC400 tests results

2.2 SUMMARY OF ASSESSMENT APPROACH

The current analytical assessment methodology considers two potential scenarios of how a hollow-core unit might resist torsional demands due to deformation compatibility under seismic actions. The first scenario assumes that a torsional shear flow can develop in the outside perimeter of the unit, as illustrated in Figure 5a. For this "equivalent tube" scenario, the torsional deformation capacity can be taken as 2.5 times the twist corresponding to the nominal cracking torque, θ_{cr}, (eq. 7). The second scenario assumes wide longitudinal cracks under the voids of the hollow-core unit due to bending of the supporting beam, as illustrated in Figure 5b, and the unit is assumed to be effectively separated into a series of I-beams linked by the concrete topping. The assessment guidelines (C5) assume the maximum torsional rotation (twist) sustained by the "top flange only" scenario, is determined based on design limits for a beam without torsional reinforcement as specified in cl. 7.6.1.2 - NZS 3101:2006-A3. For the top flange only case, the thickness where shear flow due to torsion is generated, t_w, should be calculated according to eq. 8 (Collins & Mitchell (1997)):

$$t_w = \frac{3A_c}{4p_c} \qquad (8)$$

Where,

p_c Section circumference (external perimeter) of the assumed tube cross section

A_c Area enclosed by the external perimeter of the assumed tube cross section

The calculations for the limiting twist for both scenarios are summarised below (Figure 6 and Figure 7). The assessment guidelines (C5) and Fenwick et al (2010) indicate that the controlling scenario is the case with the larger limiting twist, which may seem illogical to most engineers. Below we challenge the validity of the top flange only scenario, and recommend that engineers can focus on the equivalent tube scenario.

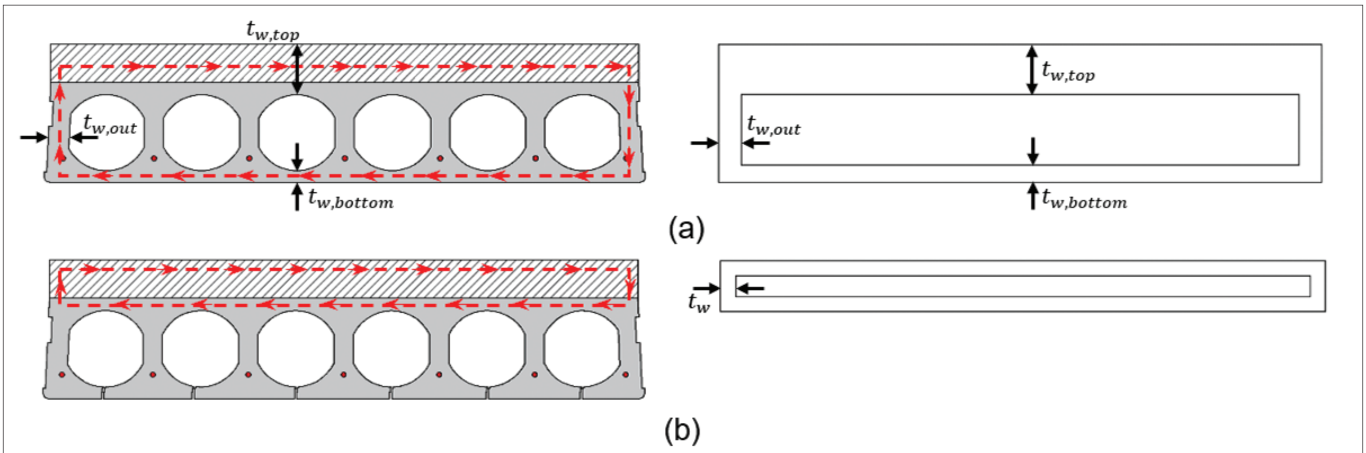

Figure 5: (a) equivalent tube section for assessing scenario one, (b) top flange only for assessing scenario two

SEISMIC PERFORMANCE OF NON-STRUCTURAL ELEMENTS: DISCUSSION BETWEEN AND SEAOC

The Structural Engineering Society of New Zealand (SESOC) and Structural Engineers Association of California (SEAOC) have been collaborating for the past seven years, in an effort to share knowledge between the two organizations. This has included publishing paired articles in both organizations' newsletters on topics of shared interest. Jan Stanway, Principal Structural Engineer at WSP, New Zealand, and SESOC member has recently led writing of an article regarding the current state of design practice for non-structural elements in New Zealand. To get feedback from the SEAOC community and provide a snapshot of California practice compared to New Zealand, SEAOC members active in non-structural component seismic performance and functional recovery were asked to provide their thoughts and reactions to the article. Part 1 of this paired article set is the article by Jan Stanway et. al. Part 2 presents excerpts from the responses provided by SEAOC members.

A copy of the article can be downloaded here: https://www.sesoc.org.nz/wp-content/uploads/2022/04/2022-02-22-SESOC-SEAOC-Paired-Article-Nonstructural.pdf

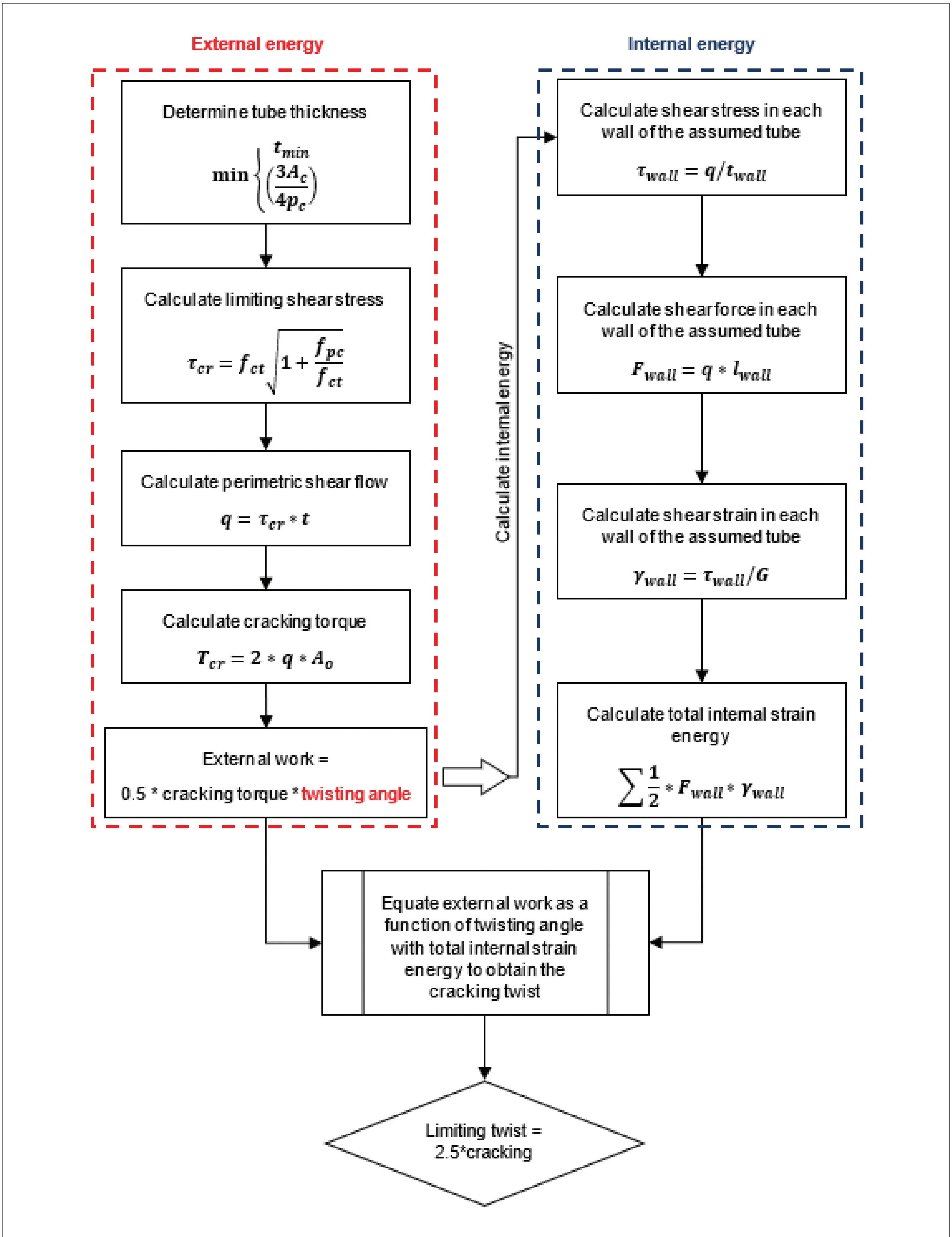

Figure 6: Flow chart summarising the torsional capacity assessment procedure for the equivalent tube case

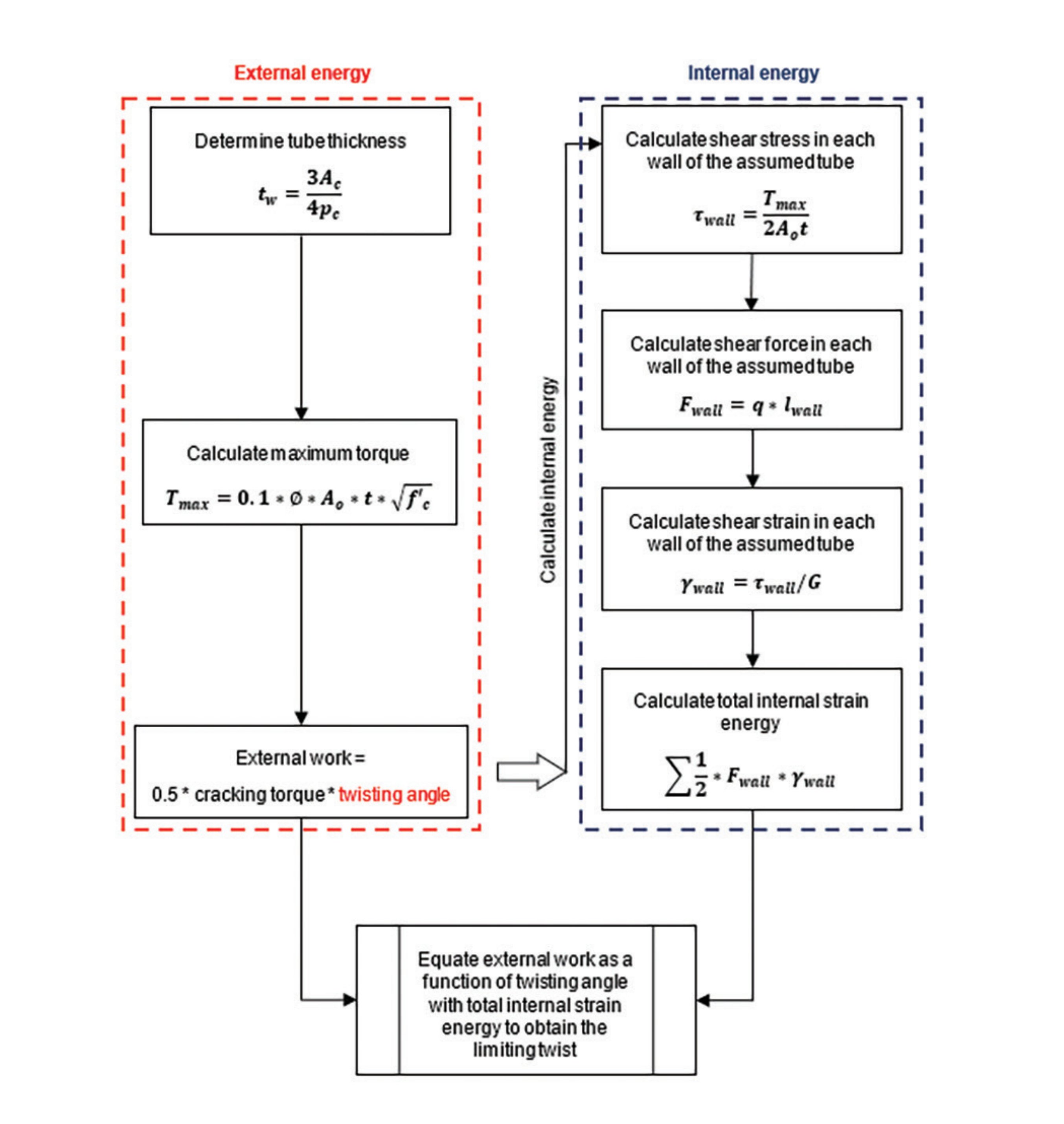

Figure 7: Flow chart summarising the torsional capacity assessment procedure for the flange only case

2.3 TORSION DEFORMATION CAPACITY ESTIMATES AND TWIST-LIMITS CHARTS

The torsional capacity of hollow-core units is sensitive to the actual material properties and geometry of the units, which varies with different manufacturers. Differences in parameters, such as floor depth, number of strands, strand height or web width can significantly affect the torsional capacity.

To facilitate the assessment of the torsional capacity of hollow-core units under the current methodology, the torsional capacities for different hollow-core depths and span lengths are plotted following the procedures described above. The plot produced herein is based on the average probable capacity from all the different manufacturers cross-sections found in Bull et al. (2009).

Probable material properties were used in assessing the expected hollow-core floor unit torsional capacity, as design values would not reflect the likely performance. Probable values for concrete strength are deemed to be significantly higher than the specified 28-day strength (MBIE et al. 2018). It is recommended to increase the specified compressive strength by a factor of 1.5 for a specified strength smaller or equal to 40 MPa and by a factor of 1.4 for a specified strength greater than 40 MPa (MBIE et al. 2018).

The concrete used in hollow-core units requires a high early strength (typically 30 MPa after 24 hours) to resist the prestressing force when releasing the prestressing strands and cutting the units to their specified lengths. Usually, the 28-day strength significantly exceeds the specified concrete strength and ranges between 50 MPa and 60 MPa (MBIE et al. 2018). Furthermore, the probable tensile strength of concrete was based on the equation $f_{ct} = 0.55\sqrt{f'c}$ found in C5.4.2.4 in the assessment guideline C5 (MBIE et al. 2018).

The probable effective prestress, f_{se},was taken as 80% of the probable initial prestress, accounting for losses. The prestressing strands are typically pretensioned to 65% of the ultimate strength of the strands. The ultimate strength of the strands used to produce the plot below was obtained from AS/NZS 4672.1:2007, where 12.7 mm-7 wire strands were assumed to be used. Finally, the shear modulus, G, was used to obtain the shear strain at each wall of the assumed tube section and was assumed to be 40% of the concrete using Young's modulus ($G = 0.4E$).

The equivalent tube case was found to always produce a larger limiting twist compared with the top flange only case. The top flange only case assumes a thin unreinforced section (topping and top flange of the unit) and uses the torsional limit of an unreinforced section (i.e. torque upon which torsion reinforcement is required) based on requirements for design. Such design provisions are used to identify when it is prudent in new design to include torsion reinforcement; they are naturally conservative and should not be used as a forensic tool to predict failure.

The indicative limits for different unit depths were calculated based on the analytical method currently available in the assessment guidelines C5. The length and depth of the unit are the only required information to use the indicative limits in Table 1 and the twist-limits chart in Figure 8.

It is worth noting that the principal stress at the critical point in the critical section depends on the longitudinal compression stress and on the bending moment at that location, which varies continuously during an earthquake. Consequently, for practical purposes, the conditions leading to torsional cracking can only be assessed to be in a likely range of structural actions (Bull et al. 2009). Calculated capacities for each hollow-core type (different manufacturers) with varying prestressing forces at the critical sections can be found in the table provided in Appendix C, and a detailed worked example can be found in the worked example by Büker et al. (2020).

The following assumptions were used in calculating the limits provided herein:

- In-situ topping was 75mm (charts are conservative for thinner toppings; thinner toppings allow slightly higher torsional deformation)
- Probable hollow-core concrete compressive strength was 58.8 MPa (1.4*42 MPa)
- Probable topping concrete compressive strength was 37.5 MPa (1.5*25 MPa)

Table 1: Indicative twisting limits for different hollow-core depths

Probable limiting vertical displacement per span length of unit (mm/m)			
	Cracking twist	Equivalent tube case	Top flange only case
200HC	1.2	3.0	0.2
300HC	1.0	2.4	0.2
400HC	0.8	2.0	0.2

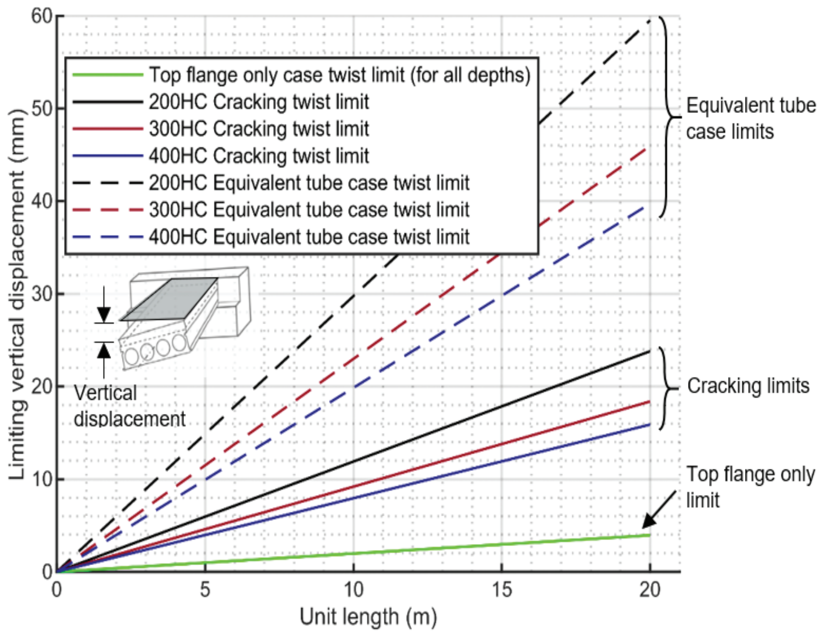

Figure 8: Torsion vertical displacement limits (deformation required to crack the unit due to torsion and deformation capacity of the unit according to both cases adopted in C5-equivalent tube case and top flange only case)

3 LIMITATIONS AND CONSIDERATIONS OF THE ADOPTED ASSESSMENT APPROACH

Most of the available information and existing research conducted on the seismic performance of hollow-core floors mainly focused on the effect of the supporting system rotation and elongation relative to the hollow-core units in the longitudinal direction (i.e. parallel to the unit direction). There has been very limited research investigating the effect of the deformations of the supporting system relative to the transverse direction of the units (i.e. perpendicular to the unit direction). Deformations in the transverse direction of the hollow-core units can cause potential problems in cases where lateral sway of a building can induce significant twist into hollow-core units. Furthermore, transverse deformations also play an important role in damage to units supported within the plastic hinge regions of intermediate columns (Beta units), as described in Mostafa et al. (2022a).

There is significant uncertainty in the available torsion methodology described herein; Broo et al. (2005) indicate that the torsional capacity of a unit subject to pure torsion might be overestimated (Figure 9) using the analytical methodology adopted in EN-1168, which is also adopted in the assessment guideline C5. Also shown in Figure 9, the shear-torsion interaction impact on the torsional capacity is not directly accounted for in the assessment guidelines methodology, potentially leading to an overestimation of torsional capacity.

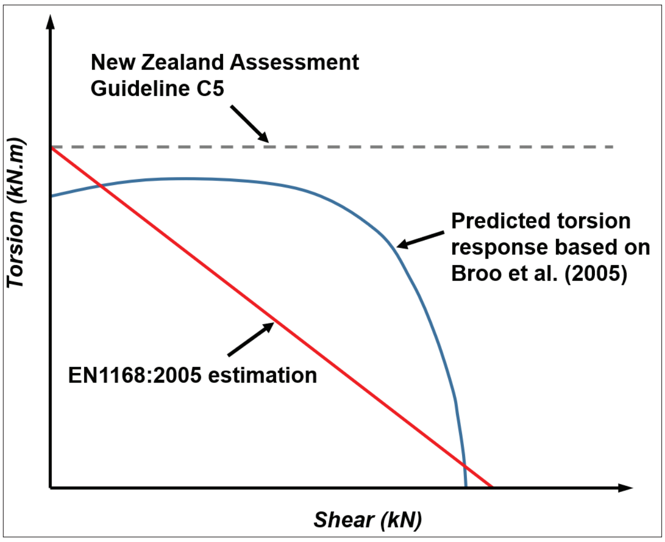

Figure 9: Schematic of shear torsion interaction estimation (Modified from (Broo et al. 2005))

On the other hand, the assessed torsional demand on the unit could also be overestimated, as the connection flexibility is not accounted for in the calculations. As twisting of the units due to the seismic response of the building is induced into the units through the connection, the connection generally acts as a fuse. The more flexible the connection is, the less twist (torsion) induced in the unit. Therefore, higher drift levels can be sustained by

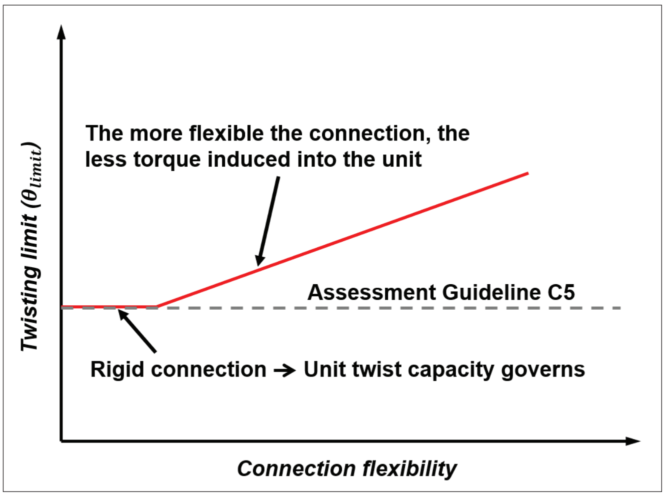

Figure 10: Schematic of the expected twist limit when accounting for different connections flexibilities

the units prior to torsional cracking. If the connection is considered rigid (e.g. filled and reinforced cores), then the connection will not dissipate deformations and the full torsional demand may be induced into the unit (Figure 10).

Table 2: Summary of current torsion assessment limitations

Limitation	Effect
• Based on the work done by Broo et al. (2005), the analytical method is unconservative for pure torsion. • Not accounting for shear-torsion interaction.	Overestimate torsional capacity
• The 2.5 factor for limiting twist is based on only two of the four tests used to develop the torsion assessment. More data points are required to verify this 2.5 factor.	Either underestimate or overestimate torsional capacity
• Not accounting for different connection flexibility on the torsional response of a hollow-core unit.	Overestimate torsional demand

Another limitation of the current torsion assessment methodology used in C5 is that only individual units are assessed. By not accounting for the contribution of neighbouring units to the hollow-core unit capacity, the potential risk of premature cracking of the webs of a unit subjected to torsion due to a compression strut that is generated in the unit from restraint by the neighbouring units, as described by Fenwick et al. (2010) and illustrated in Figure 11, is obscured.

The primary concern regarding the torsional response of hollow-core floor units is not that a unit will collapse due to torsion damage alone – this has not been observed in post-earthquake reconnaissance (Mostafa et al. 2022b; Henry et al. 2017). Instead, the concern is any potential impact torsional damage may have on other failure modes (e.g. negative moment failure (NMF) and positive moment failure (PMF)). Torsional damage will cause web cracking that, when combined with positive moment cracking or negative moment cracking, may reduce the drift capacity estimated for each individual potential failure mode.

Such impact is not well understood, and further large-scale testing of super-assembly systems incorporating units subject to high torsional demands are required to assess if there should be a reduction in the assessed drift at NMF or PMF due to damage caused by torsion. In the absence of such tests and given the generally conservative assessment of NMF and PMF using the assessment guidelines, it is not currently considered necessary to reduce the assessed drift capacity of units due to torsion damage beyond that provided by C5.

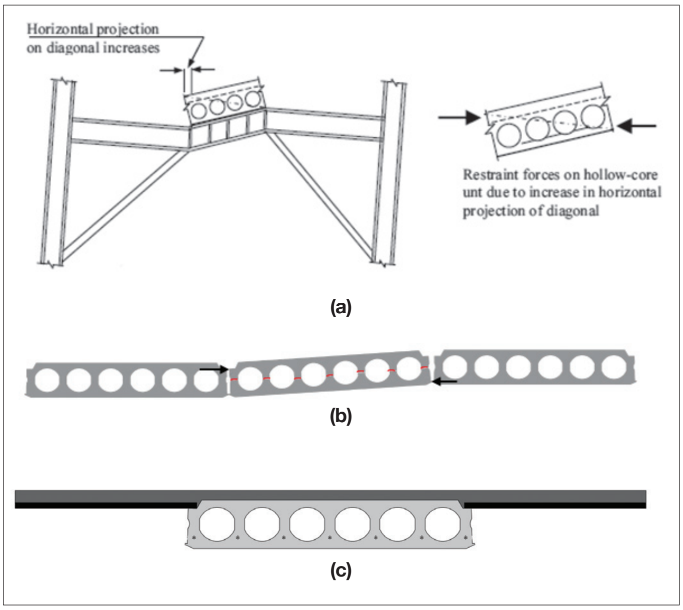

Figure 11: Example of where neighbouring units might affect unit capacity (a) twist of unit increases length of diagonal in plane of floor and restrain forces from neighbouring unit act on hollow-core (Fenwick et al. 2010) (b) schematic of potential contribution of neighbouring units when subjected to torsion (c) spaced unit will not have such contribution

Given the uncertainties listed in Table 2 and illustrated in Figures 9-11 regarding the ability of the assessment procedure to predict the torsional cracking of hollow-core units and the impact of this cracking on performance, it is recommended that in the case of a unit susceptible to high torsion demands due to its location or loading, as described in Figure 2, a conservative approach be taken when devising retrofit scope (i.e. for any units expected to undergo significant torsional demands, a retrofit be used which can provide gravity support even if a torsional crack were to form away from the support).

4 CONCLUSION

This paper provides a twist-limit chart and table as a simple tool for assessing the torsional deformation capacity of precast hollow-core floor units with varying depths according to the New Zealand seismic assessment guideline C5. It is recommended to only consider the equivalent tube case for assessing the torsional capacity of hollow-core floor units and not consider the flange only case for the following reasons:

- The equivalent tube case is stiffer and hence more likely to attract torsional demands leading to cracking.
- Based on post-earthquake reconnaissance observations, a hollow-core unit is unlikely to reach a damage state assumed by the top flange only scenario where longitudinal cracking between all the hollow-core unit webs completely loses interaction even due to aggregate interlock between webs leaving only the top flange and topping to provide torsional resistance.
- Selecting a limiting twist based on a design trigger for torsion reinforcement, as is suggested in C5 for the top flange only scenario is overly conservative.

Moreover, an extensive literature review was conducted to trace back the basis of the analytical methodology adopted in the seismic assessment guidelines C5 for assessing the torsional capacity of hollow-core floor units. Limited research on the torsional behaviour of precast hollow-core floor units was found to serve as the foundation of the methodology adopted in the assessment guidelines C5.

The lack of research data led to multiple uncertainties regarding the accuracy of the procedures currently in use. The primary limitations of the current torsion assessment procedure include the following:

- Not accounting for different connection flexibilities on the torsional response of a hollow-core unit can lead to an underestimation of the total drift at which a hollow-core unit will experience torsional cracking.
- The 2.5 factor used for the limiting twist (Equation 7) is based on only two test results that primarily focused on loss of torsional strength rather than the damage state leading to loss of gravity support. More tests are required to verify this number.
- The analytical method adopted in the assessment guidelines C5 to calculate the cracking torque can be unconservative according to Broo et al. (2005).
- Exclusion of any units with low-friction bearing strips from torsion assessment.

Despite these limitations, the guidance in C5 (modified by the above recommendation to ignore the top flange only scenario) is the best and simplest available approach to estimate the twist at torsional cracking and failure. But given the uncertainties and limitations described above, regardless of the assessed twist capacity, it is recommended that if a building with hollow-core floor units is being retrofitted, then, for any units expected to undergo significant torsional demands (even if support details have a low-friction bearing strip), a retrofit should be used which can provide gravity support even if a torsional crack were to form away from the support.

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Appendix A – Notations

π	Total potential energy
U	Strain energy (due to shear)
W	Potential energy of the load (Work done by torque)
T	Torque
θ	Angle of rotation of the hollow-core unit about its longitudinal axis
F	Shear force due to the shear flow generated in the unit as a result of the unit twisting/torsion
γ	Shear strain
τ	Shear stress
q	Shear flow
G	Shear modulus
E	Young's modulus
A_o	Area enclosed by the centreline of the tube cross section
A_c	Area enclosed by the external perimeter of the assumed tube cross section
p_c	Section circumference (external perimeter) of the assumed tube cross section
t_w	Thickness of the assumed tube cross section
f_{ct}	Concrete tensile strength
f_{pc}	Precompression stress at the neutral axis in the critical section
T_{cr}	Cracking torque
θ_{cr}	Cracking angle of twist
θ_f	Estimated failure angle of twist
F_{se}	Effective prestressing force on the section
f'_c	Probable concrete strength
Ø	Strength reduction factor, taken as 0.75 for torsion

Appendix B – Derivations and basis of the current assessment approach

The torsional capacity assessment methodology for hollow-core floor units adopted in the assessment guidelines C5 is based on the principle of stationary total potential energy (π), which is defined as the sum of the stored strain energy (U) and the potential energy of the load (W) where the potential energy of the external load is due to the torque induced into the unit (T) and the corresponding angle of rotation of the hollow-core unit about its longitudinal axis (θ), and the internal stored strain energy is due to the shear forces generated in the unit (F) and the corresponding shear strain (γ).

To be in equilibrium the total potential energy should be zero. Therefore, the energy from the work done by external forces should be equal to the internal strain energy.

$$\pi = W - U \tag{9}$$

$$W = \frac{1}{2} * T * \theta \tag{10}$$

$$U = \frac{1}{2} * F * \gamma \tag{11}$$

$$W = U \tag{12}$$

All the calculations used to satisfy equilibrium are based on Bredt's thin tube theory as explained in Collins & Mitchell (1997). As the wall of the tube is assumed to be thin, the shear stresses are considered to be constant across the wall thickness. If we consider the equilibrium in the longitudinal direction of the small element shown in Figure 12d, we conclude that the shear flow has to remain constant along the total perimeter of the section, Equation 13.

To relate the torsional demands applied on the element, T, to the shear flow, q, an element of area of length ds is considered, where ds is measured along the centreline of the tube. The total shear force acting on the element is the sum of $q.ds$ along the entire tube length, and the moment of this shear force about any point 'O' is the multiplication of the shear force by its lever arm, r_p. The torque produced by shear is obtained by integrating the shear flow along the entire length of centreline of the cross section as shown in Equation 14. The quantity $r_p * ds$ is equivalent to twice the area of the shade triangle shown in Figure 12c. Therefore, the integral $\int r_p * ds$ represents double the area enclosed by the centreline of the cross section, A_o.

$$q = \tau * t = constant \tag{13}$$

$$T = q \int r_p * ds \tag{14}$$

$$\int r_p * ds = 2A_o \tag{15}$$

$$T = 2 * q * A_o \tag{16}$$

$$q = \frac{T}{2A_o} = \tau * t \tag{17}$$

Under the assumption of pure torsion, torsional cracking is assumed to occur when the principal tensile stress reaches the tensile strength of concrete, f_{ct}. Therefore, the maximum shear stress the section can withstand before cracking, (Equation 25), is obtained using the stress Mohr-circle. Once cracking torque, T_{cr}, is calculated by substituting the cracking shear stress obtained from Equation 25 into Equation 17, internal shear energy, U, should be calculated and equated to external work done, W, as a function of the cracking angle of twist, θ_{cr}.

$$\textit{Origin of Mohr's circle} = \frac{-f_{pc}}{2} \tag{18}$$

$$\textit{Radius of Mohr's circle} = \sqrt{\left(\frac{-f_{pc}}{2}\right)^2 + (\tau)^2} \tag{19}$$

$$f_{ct} = \textit{Origin} + \textit{Radius} \tag{20}$$

$$f_{ct} = \frac{-f_{pc}}{2} + \sqrt{\left(\frac{-f_{pc}}{2}\right)^2 + (\tau)^2} \tag{21}$$

$$(\tau)^2 = \left(f_{ct} + \frac{f_{pc}}{2}\right)^2 - \left(\frac{-f_{pc}}{2}\right)^2 \tag{22}$$

$$(\tau)^2 = f_{ct}{}^2 + \left(\frac{f_{pc}}{2}\right)^2 + f_{ct}.f_{pc} - \left(\frac{-f_{pc}}{2}\right)^2 \tag{23}$$

$$(\tau)^2 = f_{ct}{}^2 + f_{ct}.f_{pc} \tag{24}$$

$$\tau_{cr} = f_{ct}\sqrt{1 + \frac{f_{pc}}{f_{ct}}} \tag{25}$$

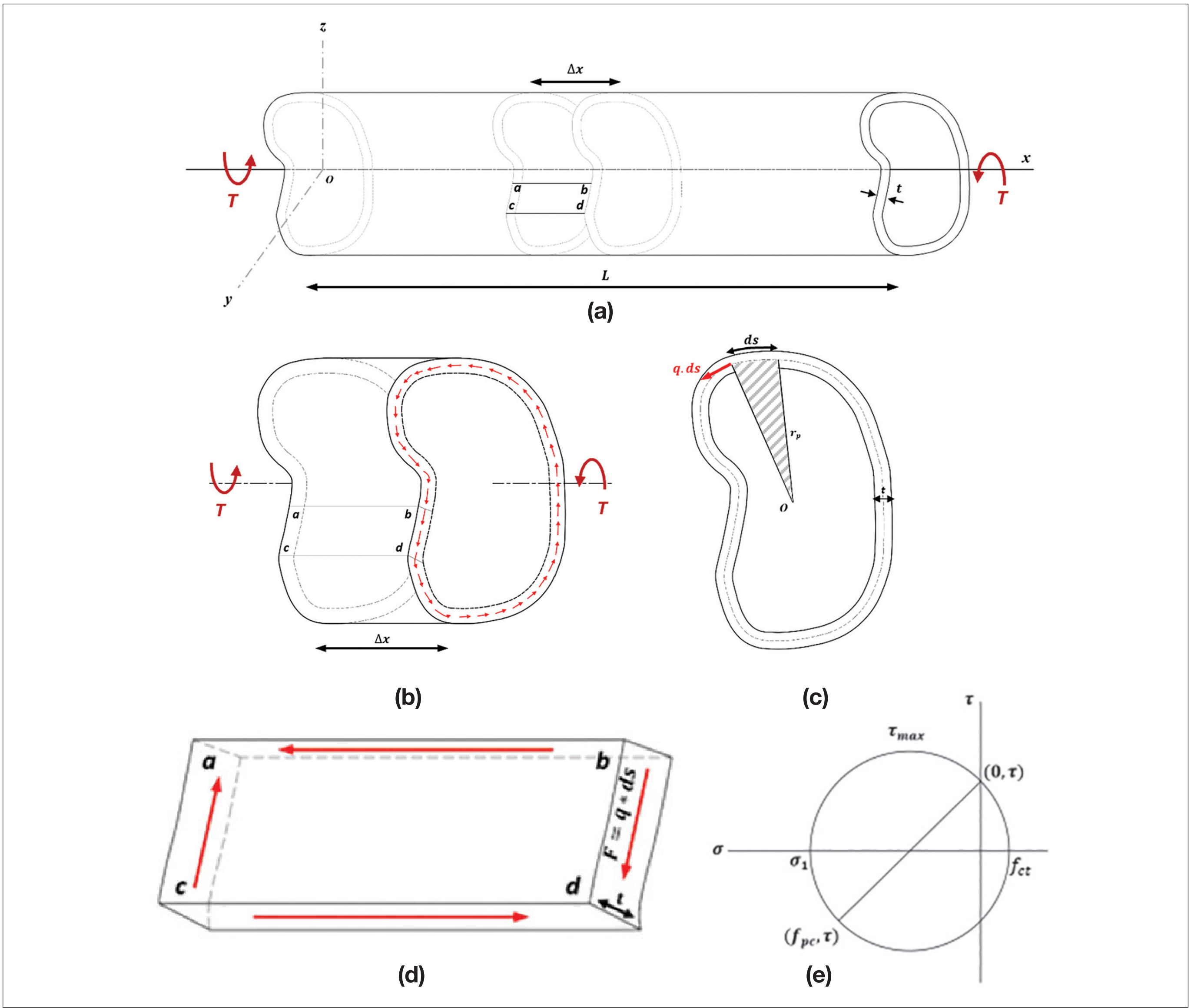

Figure 12: (a) Arbitrary thin tube element (b) Elemental section subjected to torsion with shear flow (c) Element section (d) Shear forces on an infinitesimal section (e) Stress state at the critical section

Appendix C – Torsion limits of different manufacturers

HC - Type			Prestressing Stress (Mpa)	Limiting rotaion per meter run (rad./m)			Limiting vertical disp. per meter run (mm/m)		
				Cracking twist	Scenario 1	Scenario 2	Cracking twist	Scenario 1	Scenario 2
Stahlton	Echo	200HC	0	0.0009	0.0019	0.0002	1.11	2.22	0.23
			3.5	0.0010	0.0021	0.0002	1.26	2.51	0.23
			7	0.0012	0.0023	0.0002	1.38	2.77	0.23
			3.50 (probable)	0.0010	0.0021	0.0002	1.26	2.51	0.23
		300HC	0	0.0007	0.0014	0.0002	0.86	1.72	0.22
			3.5	0.0008	0.0016	0.0002	0.97	1.94	0.22
			7	0.0009	0.0018	0.0002	1.07	2.14	0.22
			4.78 (probable)	0.0008	0.0017	0.0002	1.01	2.02	0.22
		400HC	0	0.0006	0.0011	0.0002	0.68	1.35	0.22
			3.5	0.0006	0.0013	0.0002	0.76	1.53	0.22
			7	0.0007	0.0014	0.0002	0.84	1.68	0.22
			4.11 (probable)	0.0006	0.0013	0.0002	0.78	1.56	0.22
	Elematic	200HC	0	0.0009	0.0019	0.0002	1.11	2.23	0.23
			3.5	0.0010	0.0021	0.0002	1.26	2.52	0.23
			7	0.0012	0.0023	0.0002	1.39	2.78	0.23
			3.56 (probable)	0.0011	0.0021	0.0002	1.26	2.52	0.23
		300HC	0	0.0007	0.0013	0.0002	0.80	1.59	0.20
			3.5	0.0007	0.0015	0.0002	0.90	1.80	0.20
			7	0.0008	0.0017	0.0002	0.99	1.98	0.20
			5.72 (probable)	0.0008	0.0016	0.0002	0.96	1.92	0.20
		400HC	0	0.0006	0.0012	0.0001	0.69	1.39	0.18
			3.5	0.0007	0.0013	0.0001	0.78	1.57	0.18
			7	0.0007	0.0014	0.0001	0.86	1.73	0.18
			4.69 (probable)	0.0007	0.0014	0.0001	0.81	1.62	0.18
	Spirol	200HC	0	0.0008	0.0017	0.0002	0.99	1.98	0.25
			3.5	0.0009	0.0019	0.0002	1.12	2.24	0.25
			7	0.0010	0.0021	0.0002	1.24	2.47	0.25
			5.44 (probable)	0.0010	0.0020	0.0002	1.19	2.37	0.25
		300HC	0	0.0006	0.0013	0.0002	0.76	1.53	0.23
			3.5	0.0007	0.0014	0.0002	0.86	1.73	0.23
			7	0.0008	0.0016	0.0002	0.95	1.91	0.23
			8.22 (probable)	0.0008	0.0016	0.0002	0.98	1.96	0.23
Stresscrete	Dycore	200HC	0	0.0008	0.0016	0.0002	0.95	1.89	0.24
			3.5	0.0009	0.0018	0.0002	1.07	2.14	0.24
			7	0.0010	0.0020	0.0002	1.18	2.36	0.24
			5.76 (probable)	0.0010	0.0019	0.0002	1.14	2.28	0.24
		300HC	0	0.0006	0.0012	0.0002	0.71	1.43	0.23
			3.5	0.0007	0.0013	0.0002	0.81	1.61	0.23
			7	0.0007	0.0015	0.0002	0.89	1.78	0.23
			6.72 (probable)	0.0007	0.0015	0.0002	0.88	1.77	0.25

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