OVERVIEW OF RETROFIT REQUIREMENTS AND TECHNIQUES FOR PRECAST CONCRETE FLOORS

Brooke, N.J.[1], Büker, F.[2], Bull, D.K.[3], Elwood, K.J.[4], Henry, R.S.[5], Hogan, L.[6]

ABSTRACT

Despite development of detailed assessment procedures based on extensive past research, little guidance has previously been available regarding how to retrofit deficient precast floors. Additionally, prior to the ReCast floor project there had been little previous experimental validation of retrofit techniques for precast concrete floors. This paper outlines philosophical requirements affecting design and detailing of retrofits for precast concrete floors, and summarises the efficacy of known retrofits techniques for floors constructed using hollow-core, double tee, flat slab, or rib and infill precast units.

1 INTRODUCTION

The use of precast concrete elements to construct the floors of buildings became almost ubiquitous in New Zealand during the 1980s and through much of the 1990s (Bull 1999; fib Task Group 7.3 2003; CCANZ 2004). Hollow-core floor units were particularly commonly used, along with still-substantial extents of double tee, rib and infill, and flat-slab units. However, failures of precast concrete floors during the Northridge earthquake (Norton et al. 1994) and subsequent research at the University of Canterbury over an approximately 15 year period around the turn of the millennium revealed that then-typical hollow-core floor detailing was prone to poor seismic performance (Herlihy 1999; Jensen 2006; Liew 2004; Lindsay 2004; Matthews 2004; Woods 2008). Around the same time concerns were also raised regarding performance of some other precast floor configurations (Hare et al. 2009), with these concerns later confirmed and amplified by observed performance of floors of buildings in Christchurch (Cattanach and Thompson 2013; Cooper et al. 2012; Corney et al. 2014) and Wellington (Brunsdon et al. 2017; Henry et al. 2017; MBIE 2017, 2018).

Drawing on the extensive research undertaken at the University of Canterbury, detailed guidance on assessing the behaviour of existing precast concrete floors was first published for hollow-core floors (Fenwick et al. 2010), with less comprehensive guidance also produced for double tee floors (Hare et al. 2009). More recently these guidelines have been refined and extended to cover all major precast flooring types (MBIE et al. 2018), based in part on further research undertaken at the University of Auckland (Corney 2017) and understanding gained from observations in Wellington following the Kaikoura earthquake. While some areas of uncertainty remain about the performance of existing precast concrete floors, these guidelines permit thorough assessment of floors by practicing engineers.

In contrast to guidance on assessment of precast concrete floors, relatively little information has been published that provides a clear basis for the design of retrofit solutions for precast concrete floors. Available information largely comprises advice dispersed between various research reports (e.g. Jensen 2006; Liew 2004) and the useful, but non-specific and now rather dated, draft summary of retrofit techniques published by the 'Precast Concrete Floors Overview Group' (PCFOG 2009). This lack of guidance reflects the fact that few, if any, retrofit techniques for precast concrete floors have previously been robustly investigated through analyses and experiments.

The ReCast Floors project (Brooke et al. 2019) was established in 2018 with the primary aims of refining understanding of the behaviour of precast concrete floors and validating and documenting design methods for retrofit methods for such floors. The project is principally funded by BRANZ from the Building Levy, with additional financial support received from EQC via the UC Quake Centre, Concrete New Zealand Learned Society, and

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[1] *Managing Director, Compusoft Engineering Limited*
[2] *PhD Candidate, The University of Auckland*
[3] *Technical Director, Holmes Consulting*
[4] *Professor, The University of Auckland*
[5] *Associate Professor, The University of Auckland*
[6] *Lecturer, The University of Auckland*

QuakeCoRE. Various initial outputs from the project (e.g. Henry et al. 2018; Parr et al. 2019; Sarkis et al. 2019; Puranam et al. 2019; Corney et al. 2021; Puranam et al. 2021) have so far largely documented aspects of the performance of existing floors.

The purpose of this paper is to collate retrofit techniques for precast concrete floors that have been proposed in various forums, and to summarise what (if any) improvement can be gained from each technique. Details are also provided regarding the extent of analytical or experimental validation that has occurred for each technique, available sources of more detailed guidance, and other information as may be relevant. It is intended that this paper and companion papers supersede the retrofit guidance provided in Chapter 8 of the Precast Concrete Floors Overview Group report (PCFOG 2009).

The paper is divided into a number of sections. The first of these provides discussion on the design philosophies that need to be considered during design of retrofits for precast concrete floors. Retrofits addressing hollow-core floors are summarised in section 3, followed in section 4 by those specifically applicable to floors constructed using double tees units considered. Brief discussion of retrofit techniques for flat slab and rib and infill floors is presented in section 5.

This paper does not consider or address retrofit techniques and requirements for floor diaphragms.

1.1 UNIT DESIGNATIONS

Precast floor units in certain locations are more prone to damage during an earthquake. Notably this includes, with reference to Figure 1:

- 'Alpha' units that are immediately adjacent to a parallel beam, wall, or other structural element, and
- 'Beta' units that are supported at or adjacent to an intermediate column or a wall.

Beta units are notably not addressed at all in the current Appendix C5E assessment guidance (MBIE et al. 2018), but their susceptibility to damage has now been well documented both in the laboratory (Büker et al. 2022c) and in earthquake-damaged buildings (Mostafa et al. 2022; Siddiqui et al. 2019).

Concerns regarding the behaviour of alpha and beta units are most commonly raised in relation to hollow-core floors. However, other precast floor units in these locations may also be more prone to damage.

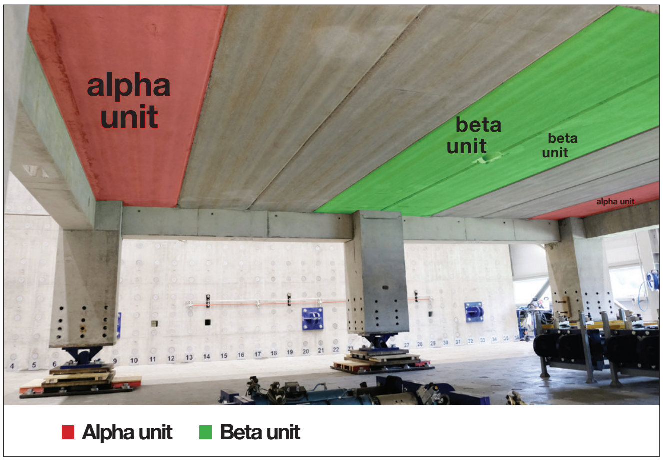


Figure 1: Underside of the 'super-assembly' specimen with alpha and beta units identified

2 RETROFIT PHILOSOPHY

Design of retrofit measures for precast concrete floors requires consideration of both the design philosophies that are generally applicable to all retrofits, as well as factors that are more specific to precast concrete floors. These philosophies are summarised in the following sections.

2.1 PERFORMANCE EXPECTATIONS AND REQUIREMENTS

Assessment of existing buildings in New Zealand is underpinned by *The Seismic Assessment of Existing Buildings: Technical Guidelines for Engineering Assessments* (MBIE et al. 2017 - the Guidelines). The Guidelines also consequently have an important role in setting the performance expectations for retrofits of structures, notwithstanding that the Guidelines contain relatively far less information pertaining to retrofit than assessment.

The Guidelines require earthquake scores (of building elements) and the consequent earthquake rating of a building to be expressed relative to the ultimate limit state (ULS) seismic demand used to design an equivalent new building on the same site, with these demands being tied for some purposes to those in effect on 1 July 2017. The resulting values are expressed as %NBS, i.e. the a percentage of New Building Standard calculated in accordance with the Guidelines. As a result of this assessment framework, it is typical that retrofits for structures (including precast concrete floors) are designed with the aim of increasing the earthquake scores of elements (and hence the earthquake rating of a building) to a specific %NBS value.

Section A10 of the Guidelines provides high level guidance regarding improvement of seismic performance, i.e. retrofit. Importantly, Section A10.2.4 notes that retrofit works must comply with the Building

Code (MBIE 2019). This is required by Section 17 of the Building Act (New Zealand Government 2004) because installing retrofits constitutes building work. In considering the requirement for compliance with the Building Code it is important that this not be confused with a requirement that the earthquake scores of retrofitted elements be 100%NBS. Rather, in summary, retrofit elements are required to resist demands associated with the ULS demands (factored where appropriate by the targeted %NBS rating, refer section 2.2) with design completed in accordance with Verification Method B1/BM1 or other appropriate Standards. Engineers are recommended to review Section A10.2.4 of the Guidelines, but for precast concrete floor retrofits the following points are pertinent:

- Demands on retrofit elements should be compared to the dependable strength of the element, i.e. the nominal strength, calculated using specified lower characteristic material strengths, multiplied by an appropriate strength reduction factor as specified in the relevant Standard, e.g. NZS 3101 or NZS 3404 (SNZ 2007, 2017).
- Anchors should typically be designed using provisions for seismic performance category C2 (CEN 2018), as described in section 2.5.
- Design of retrofit seating measures should be based on the required seating length specified in Chapter 18 of the New Zealand Concrete Structures Standard, NZS 3101 (SNZ 2017), with further guidance provided in a companion paper (Büker et al. 2022b).
- The earthquake scores of unretrofitted components (or for failure modes not affected by retrofits) should be assessed by relevant assessment guidance (MBIE et al. 2017, 2018) when determining the earthquake rating of the building after retrofit.

There are notable differences between the procedures used in design and assessment for determining the required seating length. At face value, the multiplier applied to drift demands for assessment (×2) appears greater than for design (×1.5/Sp) for some structures. However, other conservatisms in the design procedure are expected to result in the seating length required for design being larger than that required for assessment.

Engineers should clearly identify whether the design approach they adopt follows Verification Method B1/VM1, and hence is deemed to comply with the Building Code, or whether use of unreferenced Standards and guidance means that the retrofit design constitutes an Alternative Solution. This fact should be clearly communicated to reviewers and the building consent authority.

2.1.1 Targeting of retrofits

Retrofit measures should be well targeted to avoid unnecessary expense. For precast floors, this means that retrofit requirements should be determined on a unit-by-unit basis, or at least for groups of similar units. For example, the retrofits required for units within the elongation zone (MBIE et al. 2018) may be different to those outside that zone, and alpha and beta units typically warrant separate consideration. It may well be the case that the required retrofits varied markedly over the height of a building due to variation of drift demands at different levels.

It is also important to consider not just the critical mode of behaviour that defines the earthquake score for a particular precast unit, but the hierarchy of limiting behaviours that are expected as drift demand increases. Taking, for example, a hollow-core floor unit, assessment might show (with reference to section 3 for definitions) that LOS is expected at 1.2% drift and PMF at 1.8% drift, with NMF and WSF not anticipated. Installation of supplementary seating (section 3.1) would address the LOS. Whether this retrofit was sufficient would depend on the target drift capacity (i.e. %NBS). If the target drift capacity was (e.g.) 2%, then supplementary seating alone would not be a sufficient retrofit. As noted in section 3.1, supplementary seating does not address PMF. While it might be appealing to think that the expected occurrence of LOS at 1.2% drift would 'protect' against the occurrence of PMF, experimental evidence shows that this cannot be relied on. Consequently, for this example, retrofit would be required to address both LOS and PMF.

Consideration should also be given to non-structural items proximate to the precast floor. While retrofit is likely to disrupt existing services and fitout, the configuration of these items may inform the decision between alternative retrofit techniques that offer similar structural outcomes but different levels of disruption.

2.1.2 Expected performance of retrofits

Beyond simply achieving a selected %NBS target, Section A10.2.1 of the Guidelines emphasises the importance of understanding the requirements and expectations of the building owner. This may also necessitate improving the owner's understanding of the likely outcomes of earthquake shaking of various intensities so that their expectations are realistic. This is certainly an important step in retrofit of precast concrete floors. Engineers should ensure that owners understand that:

- it is typically not possible to retrofit precast concrete floors to achieve more than life safety objectives during strong earthquake shaking
- Damage that may be difficult or impossible to repair is likely to occur even if a floor has been retrofitted, and
- Such damage could occur even during relatively low intensity shaking, i.e. shaking below that associated with the retrofitted %NBS.

Of particular note is that most retrofit techniques for precast concrete floors do not prevent failures from occurring, but instead are expected to reduce the life safety hazard to an acceptable level. Most retrofit techniques are more specifically "catch" systems that are expected to result in the floor unit displacing vertically but not collapsing. The term catch system is used at points through this paper in reference to such systems. As elaborated on in section 2.3, it is generally recommended to install a compressible layer between precast units and catch system retrofit components to reduce the suddenness of the expected vertical displacement associated with engagement of the retrofit.

2.1.3 Validation of retrofit techniques

As noted in the introduction to this paper, little robust investigation of retrofit techniques for precast floors existed prior to the ReCast Floors project. This paucity of information was a key motivator for the project.

As outlined elsewhere in this paper, and detailed in other companion papers (Büker et al. 2022a; b; c), several retrofit techniques have been extensively investigated by the ReCast Floors project. These investigations have comprised both analytical and experimental aspects, with experimental investigations further bifurcated between sub-assembly and super-assembly tests (Elwood and Hogan 2022).

Retrofit techniques that have been shown to perform satisfactorily in appropriate experiments and that have a sound analytical basis can be considered to be robustly validated. A number of such retrofit techniques are identified in this paper, in some cases with caveats about performance limitations and/or remaining uncertainties.

Inevitably, specific project circumstances may require consideration of retrofit techniques that have not been robustly investigated. Engineers should be cautious before adopting such techniques. The behaviour of precast floors and their retrofits is dependent on complex three-dimensional interaction of various components that are difficult to replicate from either analysis or sub-assembly testing alone. Alternative retrofit techniques must be based on sound engineering principles, with reference to experimental testing where possible and appropriate. Particular care is required to ensure that three-dimensional effects are adequately understood.

2.2 DESIGN ACTION COMBINATIONS

Building retrofits must be designed to resist appropriate combinations of design actions, with the combinations and actions requiring consideration generally following the requirements of the Structural Design Actions Standard, NZS 1170 (SNZ 2011).

In contrast to many retrofits that have their design governed by the seismic actions they are required to resist during an earthquake, precast floor retrofits are often primarily intended to provide reliable load paths for gravity actions both during and after a damaging earthquake.

Where precast floor retrofits are required to provide reliable load paths for gravity actions after the floor is damaged by an earthquake or other event, design should be consistent with the approach specified in AS/NZS 1170.0 for ULS gravity strength checks. AS/NZS 1170.0 requires that two gravity combinations be considered:

$$E_d = 1.35G \qquad (1)$$

$$E_d = 1.2\text{G} + 1.5Q \qquad (2)$$

Where E_d is the design action effect, G is the permanent action (i.e. 'dead load') and Q is the imposed action ('live load'). The area reduction factor, ψ_α, is not included in this combination because it is required to be taken as 1.0 for one-way slabs such as precast concrete floors.

Focussing on Equation 2, a reasonable argument can be made that in some circumstances it is unrealistic for the full imposed action, Q, to act on a damaged floor during or (for temporary use) after an earthquake. In recognition, Equation 2 may be altered to:

$$E_d = 1.2\text{G} + 1.5\psi_E Q \qquad (3)$$

Where ψ_E is the combination factor for earthquake actions.

Before adopting Equation 3 as the basis of a retrofit design, engineers should consider the following factors:

1 It would be inappropriate to adopt the reduced demand suggested by Equation 3 for areas of floor that provide emergency egress routes, and consequently could be expected to be heavily loaded in the aftermath of an earthquake. Identification of such egress routes may be challenging, particularly for open plan spaces.

2 Engineers should be cautious about suggesting that a precast floor could remain usable after it had sustained damage that was sufficient to cause retrofits to be engaged. Notwithstanding, adoption of Equation 3 would be inappropriate if there was a desire to protect the potential for such post-damage usage. Where Equation 3 was used to design retrofits, it would be critical that careful post-earthquake inspection was undertaken to ascertain whether retrofits had been engaged.

Where retrofits are designed to resist seismic actions induced during an earthquake, design should consider the typical earthquake design action combination specified by AS/NZS 1170.0 but with the earthquake actions factored to reflect the targeted %NBS rating, i.e.:

$$E_d = G + \psi_E Q + \psi_{\%NBS} E_u \tag{4}$$

Where $\psi_{\%NBS}$ is a non-conventional notation introduced to reduce the ULS earthquake demand to reflect the targeted %NBS score.

For retrofits where the performance target is based on drift or displacement, the design demand should be based on the peak Maximum Considered Earthquake (MCE) drift as defined in the Concrete Structures Standard (SNZ 2017) factored to reflect the targeted %NBS rating. Thus the design drift demand should be:

$$\psi_{\%NBS}\,\delta_{peak.MCE} = \frac{1.5}{S_p}\,\psi_{\%NBS}\,\delta_{ULS} \tag{5}$$

Where S_p is the structural performance factor used in the assessment, δ_{ULS} is the ULS interstorey drift for the structure, and $\delta_{peak.MCE}$ is the peak MCE interstorey drift demand for the structure.

It is not generally necessary to design retrofits to accommodate the 2.5% maximum ULS interstorey drift permitted by the Design Actions Standard (SNZ 2016), the exception of course being if analysis indicated that this level of drift occurred in the structure when it responded to the targeted %NBS level of shaking.

2.3 DEFORMATION COMPATIBILITY

For many retrofits it is necessary for designers to ensure that provided clearances are adequate to prevent unwanted interaction between the floor, retrofit, and/or superstructure of the building. Detailed guidance on the location and magnitude of clearances required for certain retrofit techniques can be found in other companion papers (Büker et al. 2022a; b). The principles and approaches outlined in these papers can be used as guidance for detailed consideration of other retrofit techniques.

Addressing differential vertical movement of retrofits and the floor is a particular challenge:

- If clearance is not provided, excessive demands could be induced on the floor or the retrofit. However,
- If clearance is provided consideration may be required regarding the potential for dynamic impact forces to be generated as a retrofit engages.

Avoiding the uncertainty of dynamic impact forces presents a strong argument for preferring retrofit techniques that are installed in contact with the floor. However, in many instances this is not practical due to the risk of generating problematically large forces due to deformation (in)compatibility. For such situations, i.e. where it is necessary to use a catch system as defined in section 2.1.2, it is considered that retrofit design could be undertaken without specific consideration of dynamic impact forces provided a deformable layer is included that provides a load path between the precast floor unit and the retrofit. Commentary on where such deformable layers are recommended are made in various parts of sections 3 and 4. The presence of the deformable layer will obviously result in forces being induced on the floor and retrofit when the building displaces laterally. While the magnitude of these forces will be less than if there was direct contact between the (e.g. steel) retrofit and the precast unit, it is critical that their impact be considered. An example of how to do so can be found in a companion paper (Büker et al. 2022b).

2.4 FAILURE HIERARCHY OF RETROFIT HARDWARE

The key concern with precast concrete floors is the brittle nature of most of the failure modes that they are susceptible to. Both for this reason, and in accordance with common structural engineering practice, it is highly desirable that retrofits for precast floors be designed so that they exhibit a ductile failure if subjected to demands larger than those anticipated during design.

The most dependable way of ensuring ductile behaviour is to employ a capacity design approach so that the force demands on potentially brittle components of a retrofit are limited by yielding of ductile components. Most commonly, this means that the strength of retrofits should be limited by yielding of steel components. Specific guidance appropriate to different retrofit techniques is given elsewhere in this paper and in companion papers, but given their brittle failure modes it is generally considered undesirable for either embedded anchors or fibre reinforced polymer (FRP) components to be the 'weakest link' that limits the strength of a retrofit.

Unless a secondary load path such as hanger bars can be provided, ductile behaviour is unlikely to be achievable where the failure mode involves loss of support. In such circumstances the required resilience should be provided by designing to accommodate peak MCE displacement demands as outlined in the previous section, which is consistent with the requirements for precast unit seating in the Concrete Structures Standard, NZS 3101 (SNZ 2017)

2.5 CONSIDERATION OF ANCHORAGE DESIGN FOR RETROFITS

Many retrofit techniques for precast floors require anchorage of new components to existing concrete elements. Chemical epoxy anchors are generally the recommended method for achieving this, though in some circumstances (Büker et al. 2022a) screw anchors may be beneficial.

Design of anchors should be undertaken in accordance with Chapter 17 of the Concrete Structures Standard (SNZ 2017) or another appropriate Standard. Chapter 17 makes reference to superseded guidance for design of post-installed anchors (European Organisation for Technical Approvals (EOTA) 2013). It is recommended that anchor design be undertaken using the replacement European Standard (CEN 2018), though this would result in the design representing an Alternative Solution. Irrespective of the guidance used, anchors will generally need to be designed using provisions for seismic performance category C2 as defined in the aforementioned European Standards.

Particular attention must be paid to anchors that are positioned in regions where plastic deformations are expected to occur during an earthquake:

- Anchors installed within half the beam depth away from the column are expected to be affected by substantial cracking and should not be relied on to resist shear and tension demands. Some consideration of anchors located in such locations may be warranted in specific situations (Büker et al. 2022a).
- For epoxy anchors installed less than one beam depth away from the column, the cover concrete should not be relied on and should be discounted from the effective embedment depth. This limitation is sufficient for reversing plastic hinges forming at the column face. Special attention is required where plastic hinges may form in other locations, for example where gravity dominance or reinforcement detailing results in potential plastic hinges forming within the beam span.

Where feasible, more robust anchorage can be achieved by drilling through an element and anchoring a threaded rod on the far side. This solution can be particularly appealing for internal support beams where the floors supported on each side require a seating extension retrofit.

Generally, where anchors are used to connect steel components to existing structural elements it will be beneficial to provide horizontally slotted holes in the steel components to simplify installation and mitigate potential clashes with existing reinforcement. Slotted holes are also recommended in regions where elongation of the supporting element could occur.

2.6 ASSESSMENT OF EXISTING RETROFITS

While not strictly related to design of new retrofits, comment is warranted on the subject of assessing the capacity of retrofits that already exist in a building.

For a variety of reasons, existing retrofits may not comply with the recommendations outlined in the preceding sections, other parts of this paper, and the more detailed companion papers (Büker et al. 2022a; b).

As with other aspects of assessment of existing structures, it is reasonable to assess existing retrofits on a less onerous basis than is suggested for the design of new retrofits. For aspects of retrofits such as the length of existing supplementary seating this can readily be achieved by applying the methods used for assessment of precast concrete floors (MBIE et al. 2018).

Existing anchors, for instance used to fix previously installed seating angles to beams, present a particularly problematic aspect to assess. Except when recently installed, existing anchors are likely to have been designed without adequate consideration of the deleterious impact of concrete cracking on anchor performance (Eligehausen et al. 2006). More fundamentally, it will often be the case that no (or limited) information is available regarding the anchors and epoxy used in a building. There is currently no clear basis on which to determine an appropriate “probable” anchor strength for use in assessment. Some guidance on assessment and testing of anchors is available (MBIE et al. 2018), but further work to produce a more comprehensive methodology is required.

2.7 CONSIDERATION OF EXISTING DAMAGE

It is not unusual to encounter existing damage when assessing and retrofitting precast concrete floors. Cracking of topping concrete is routinely encountered, while damage to precast floor units or support ledges is less common but not unusual. Damage to both the topping concrete and precast floor units can be present in floors irrespective of whether the floor has been subject to strong earthquake shaking. Examples of damage to precast units that can occur in the absence of earthquakes includes:

- Spalling at the ends of units or of the support due to interaction between the unit and the seating ledge.
- Transverse cracking of hollow-core planks close to supports, which is particularly common where thermal strains cause repeated movement of the unit and may be an indicator of weak bond to the prestressing strand. The transverse cracks may extend further into the webs of the hollow-core unit and thus compromise the gravity capacity of the unit. Consequently it is essential that inspection with a borescope camera be undertaken where existing transverse cracks are identified.
- Longitudinal cracking of hollow-core planks. Longitudinal cracking may be associated with web cracking, particularly if the longitudinal crack is observed to cross between adjacent voids. It would be prudent to undertake borescope investigations of longitudinal cracks, with inspections focussing on the regions proximate to the supports where shear demands are highest and prestress lowest.
- Cracking of the flanges of ‘flange hung’ double tee units proximate to the supports, and
- Damage caused by corrosion of prestressing strands or other reinforcement.

Guidance on inspecting precast concrete floors is available in Appendix C5E (MBIE et al. 2018). Guidance on identification of web cracks can be found in a companion paper (Büker et al. 2022c).

Anecdotal evidence indicates that car park ramps constructed using hollow-core planks may be particularly susceptible to non-earthquake damage, and therefore warrant particularly careful inspection.

If the performance of a precast floor is expected to be significantly affected by existing damage, it is critical that these effects be taken into consideration when designing retrofits for the floor.

3 RETROFIT TECHNIQUES FOR HOLLOW-CORE FLOORS

Numerous retrofit techniques have been proposed for hollow-core floor units, including those summarised in earlier guidance (PCFOG 2009) and additional techniques conceptualised or developed since. These techniques are discussed in the following sub-sections, and are intended to address one or more the deficiencies that commonly affect hollow-core floor units, namely (MBIE et al. 2018):

- Loss of support **(LOS)**
- Positive moment failure **(PMF)**
- Negative moment failure **(NMF)**

While positive moment failure is identified in the assessment guidance (MBIE et al. 2018) as a single category, two criteria are used to define whether a unit is expected to experience positive moment failure, namely wide opening of a transverse soffit crack proximate to the support or presence of a transverse soffit crack along with web cracking. Web cracking is most commonly observed in alpha and beta units (Büker et al. 2022c) and may be caused by torsion or incompatible displacements. Different retrofit measures may be required depending on which of these failure sub-types are expected. In this paper, PMF will be used to refer to failure due to wide opening of a crack proximate to the support, while web splitting failure **(WSF)** will be used to refer to failures arising from web cracking.

Aspects of the positive moment response of hollow-core floors remain relatively poorly understood. As detailed in a companion paper (Büker et al. 2022c), recent experimental testing has shown that:

- Soffit cracks can form proximate to, but away from the face of the support. Units with cut outs to accommodate columns are particularly vulnerable to soffit cracks initiating at the corner formed by the cut out. In addition, poor strand-concrete bond can contribute to the likelihood of soffit cracks away from the support. However, further research is required to develop techniques to identify units with poor bond, and in any case anecdote suggests soffit cracks can exist without excessive bond degradation.

- Positive moment cracks can propagate at a shallow angle. It is not apparent that the possibility of shallow angle propagation can be excluded for any particular hollow-core unit.

Additionally, soffit cracking of units approximately 300 mm from the support and away from any unit cutouts has been observed in earthquake damaged buildings (Brunsdon et al. 2017; Henry et al. 2017). Attempts to reproduce such damage in the laboratory have been unsuccessful to date (Corney et al. 2018), and it remains uncertain whether such cracking is a result of earthquake shaking or other causes.

The uncertainties about the position and inclination of positive moment cracks must be accounted for when designing retrofit measures to address PMF.

Particular caution is required when assessing whether retrofit is required to address NMF. The potential for NMF is sensitive to the strength of reinforcement crossing the interface at the end of a hollow-core unit. This strength can be materially affected by common differences between the as-drawn and as-built configuration of a hollow-core floor, including:

- Starter bar configurations that differ from those shown on the drawings,
- Mesh reinforcement that is anchored over the support beam, rather than terminating prior to the end of the hollow-core unit, and
- "Paperclip" reinforcement used as a remedial measure where construction issues resulted in a unit having no effective seating (Bull 1999).

Various retrofit techniques for hollow-core floors are summarised in the following sections. Generally, retrofit techniques for hollow-core floors do not address all of the potential failure modes that can afflict hollow-core units. An overall summary of which failure modes are addressed by each retrofit technique discussed can be found below in Table 1.

Table 1: Summary of hollow-core retrofit techniques

Retrofit technique	**Refer section:**	**Failure type**			
		LOS	**PMF**	**NMF**	**WSF**
Supplementary seating	3.1	✓	✗	✗	✗
Strongback supports	3.2	✓	✓	✓	✓(1)
Cable catch system	3.3	✓	✓	✓	✗(2)
Supplementary negative moment reinforcement	3.4	✗	✗	✓	✗
Supplementary positive moment reinforcement	3.5	✗	✗(3)	✗	✗
Supplementary transverse reinforcement	3.6	✗	✗(3)	✗	✓(4)
Catch beams	3.7	✓	✓	✓	✓
Release of negative moment restraint	3.9	✗	✗	✗(5)	✗
Core filling	3.8	✗	✗	✗	✗

Notes to table

(1) Strongback supports address WSF within the length of the strongback. Other measures are required if WSF beyond the strongback requires addressing as described in a companion paper (Büker et al. 2022a).

(2) The cable catch does not necessarily address WSF, but can do depending on the configuration of cross beams used for the diverters.

(3) Supplementary positive moment reinforcement in conjunction with supplementary transverse reinforcement and supplementary seating may be able to address PMF, but this combination has not been validated experimentally.

(4) The ability of supplementary transverse reinforcement has not been validated experimentally.

(5) Table content reflects release of negative moment restraint by drilling/cutting concrete. Cutting of reinforcement at ends of hollow-core unit can prevent NMF.

While not included in Table 1 or discussed in the following sections, replacement of hollow-core units may be a viable retrofit technique in some circumstances, whether for particularly vulnerable units or for entire floors. Depending on floor span and demands, appropriate replacement floor systems may include rib and infill floors or steel beams with composite flooring. Other systems may also be structurally viable, but problematic from an access and/or construction perspective.

3.1 SUPPLEMENTARY SEATING

Supplementary seating refers to the technique of extending the seating of precast units by fixing new structural elements to the face of the existing seating ledge, typically by use of epoxy anchors. Detailed recommendations for the design of supplementary seating can be found in a companion paper (Büker et al. 2022b). The new seating element is most commonly a steel section but can also be made of another material such as concrete. Supplementary seating:

- Is an effective method of mitigating LOS.
- Cannot, on its own, mitigate NMF, PMF, or WSF.

Previously it was considered that installation of supplementary seating would mitigate PMF. This is no longer considered to reliably be the case for two reasons. Experimental testing has shown that positive moment cracks can form away from the face of the beam, i.e. beyond the length of supplemental seating. Additionally, even where positive moment cracks form at the beam face, testing has shown that positive moment cracks can propagate at a shallow angle as shown in Figure 2. The resulting slender unreinforced concrete section cannot be relied on to transfer the full weight of the floor to the supplementary seating.

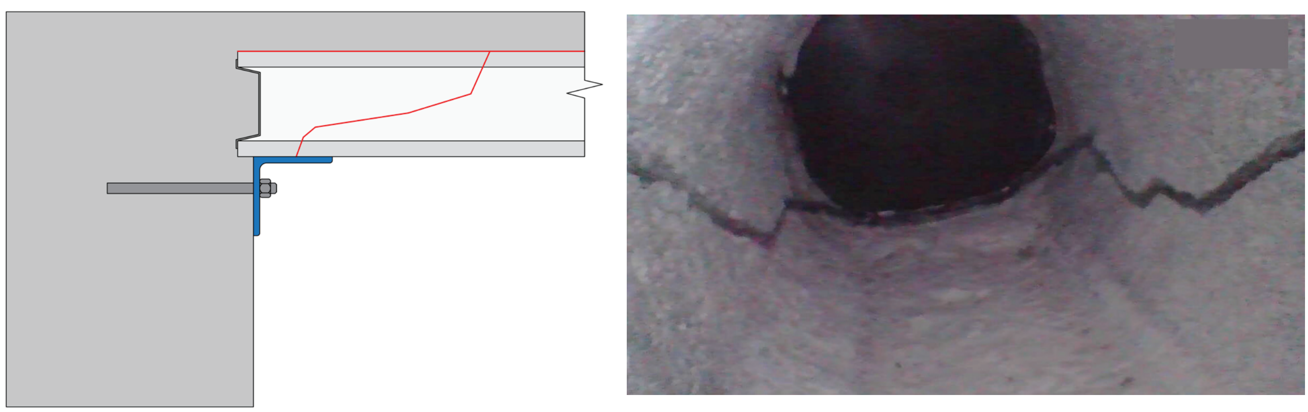
While installation of supplementary seating has long been adopted as a measure to mitigate LOS, recent consideration outlined in a companion paper (Büker et al. 2022b) shows that design of supplementary seating is more challenging than had been previously understood due to the need to preclude potentially brittle failure of anchors.

Previous guidance (PCFOG 2009) suggested that rolled hollow sections (RHS) were the preferred form of supplementary seating, and that angle or channel sections were not recommended. This is no longer the case; either can be used. Provided the stiffness of the chosen section is appropriately considered in the design process, the choice of supplementary seating type can be made based on practical considerations related to ease of installation.

The companion detailed design guidance (Büker et al. 2022b) suggests two alternative approaches to the design of supplementary seating, as shown in Figure 3 :

1. Supplemental seating hard up against the soffit, which requires the post-installed anchors to be capacity designed, or
2. Supplemental seating set down from the soffit with a compressible layer provided between the supplemental seating and the floor unit.

For either configuration, it is recommended that the anchors be installed in slotted holes and that the supplemental seating span at least 75% of the width of the hollow-core unit as shown in Figure 4. It is also essential that the ability of the supported webs to resist the shear demand on the unit be checked.



(a) (b)

Figure 2: (a) Schematic and (b) borescope images showing potential shallow angle of positive moment cracks

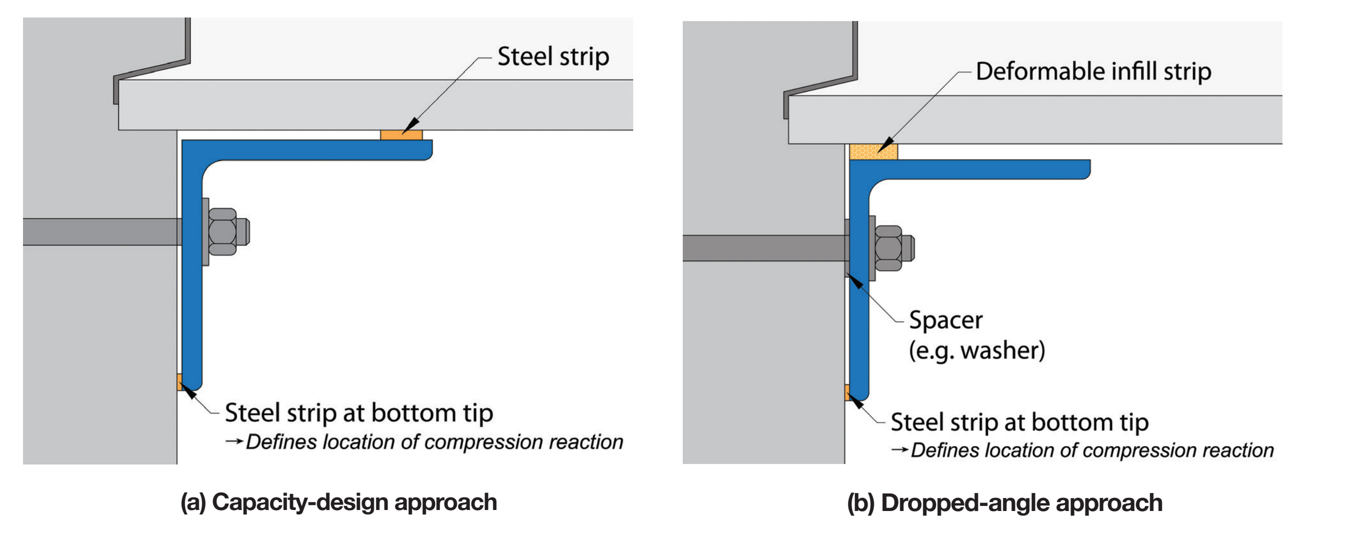


Figure 3: Detailing of the two recommended design approaches for supplementary seating angles (Büker et al. 2022b)

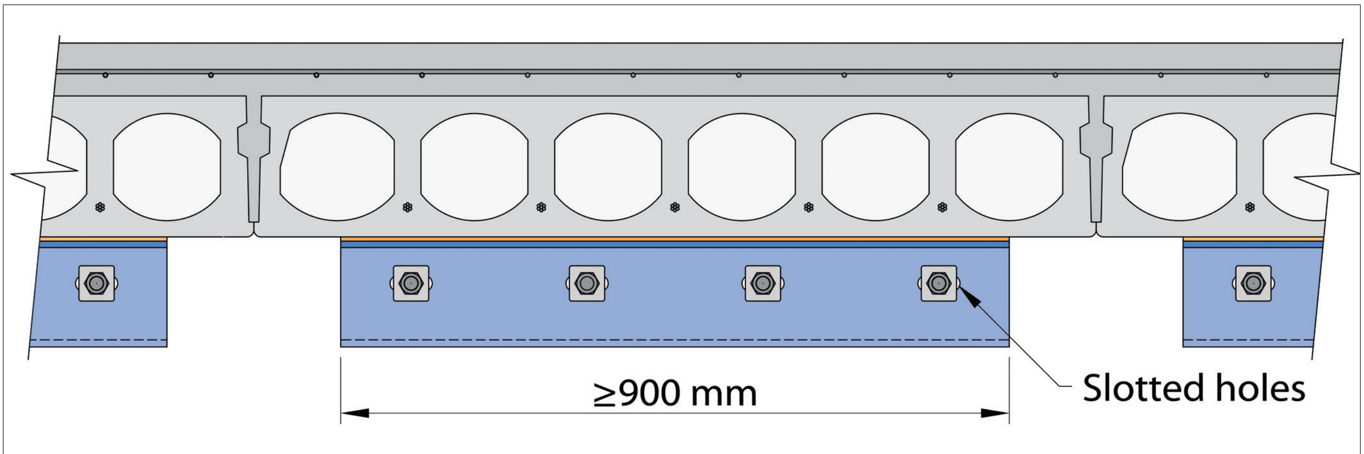


Figure 4: Elevation showing recommended supplemental support configuration

As outlined in the companion paper (Büker et al. 2022b) and assessment guidance (MBIE et al. 2018), supplementary seating elements can induce negative moment demands in the hollow-core units. Lateral drift of the building will cause this retrofit configuration to interact with the floor and induce a force in the hollow-core units that will increase the negative moment demand and may increase the propensity for NMF (Liew 2004; Parr et al. 2019). The significance of these forces needs consideration and is particularly significant for stiff supplementary supports (Parr et al. 2019). For new installations, guidance is provided in the companion paper. It is also important that the impact of previous supplementary seating retrofits that have been installed in contact with the soffit of the floor is accounted for during seismic assessments. Rational analysis accounting for the probable stiffness of the supplementary seating should be undertaken to quantify the significance of this interaction. This analysis should also account for the likelihood that the position of the reaction between the supplemental seating and the floor unit will be uncertain unless a steel strip (Figure 3a) or similar approach has been used to provide a defined reaction point.

3.2 STRONGBACK SUPPORTS

Strongback supports are a newly developed retrofit technique that utilise a number of steel beams to provide an alternative load path from the span of a hollow-core unit to the support beam. As shown in Figure 5 steel elements (in the case shown, I-beams) are fixed to a hollow-core unit and supported by a seating angle at the support beam. Further discussion of strongback configurations can be found in a companion paper (Büker et al. 2022a).

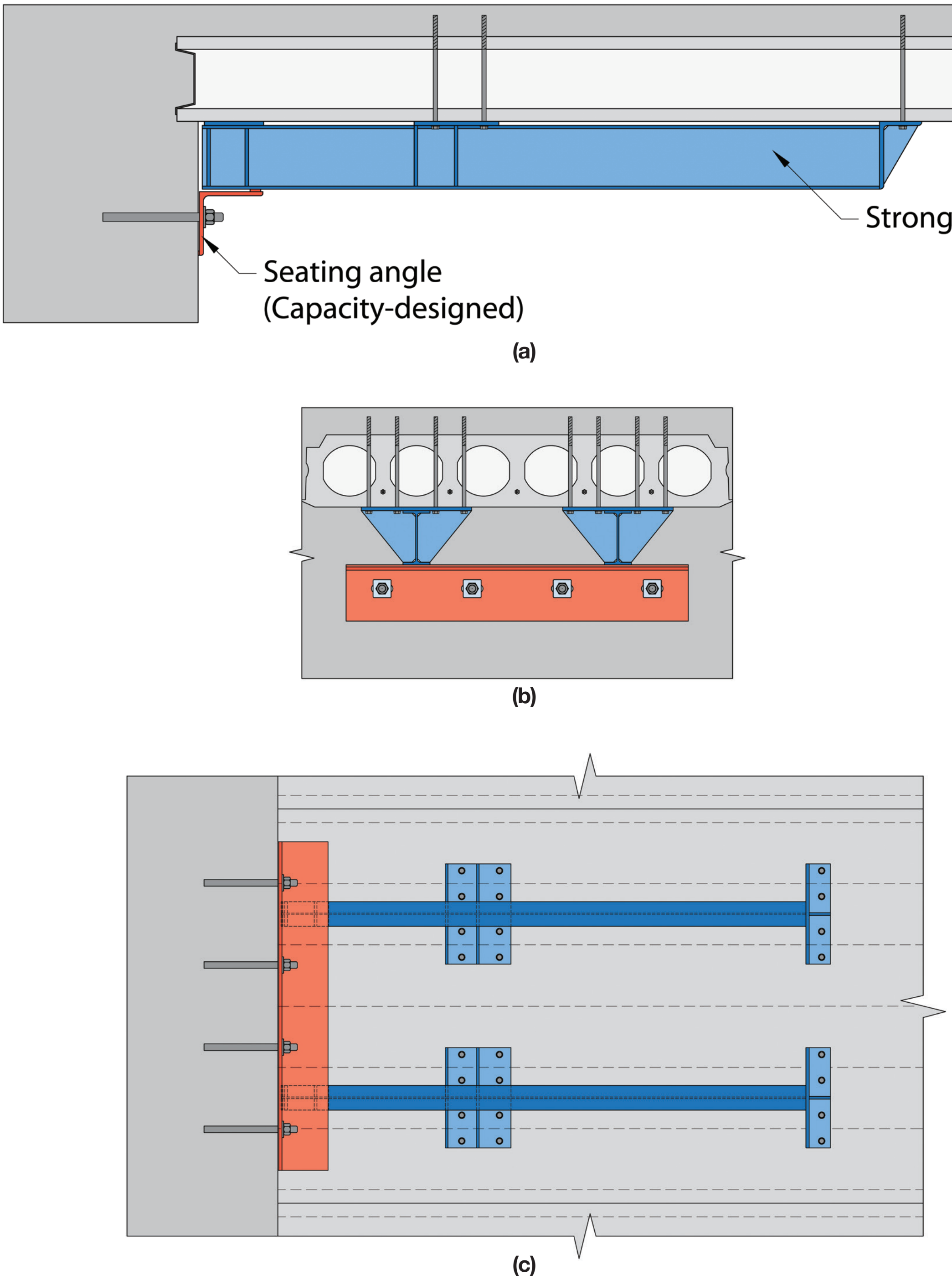


Figure 5: Strongback retrofit showing (a) side elevation, (b) cross section, and (c) plan view of soffit

Strongback supports:

- Can effectively address LOS, PMF, and NMF.
- Can address WSF within the length of the strongback, but other measures are required if WSF beyond the strongback is expected.

The performance of strongback supports has been verified by testing of a super-assembly specimen with strongback supports installed (Büker et al. 2021, 2022c). Detailed guidance for the design of strongback supports is available in a companion paper (Büker et al. 2022a).

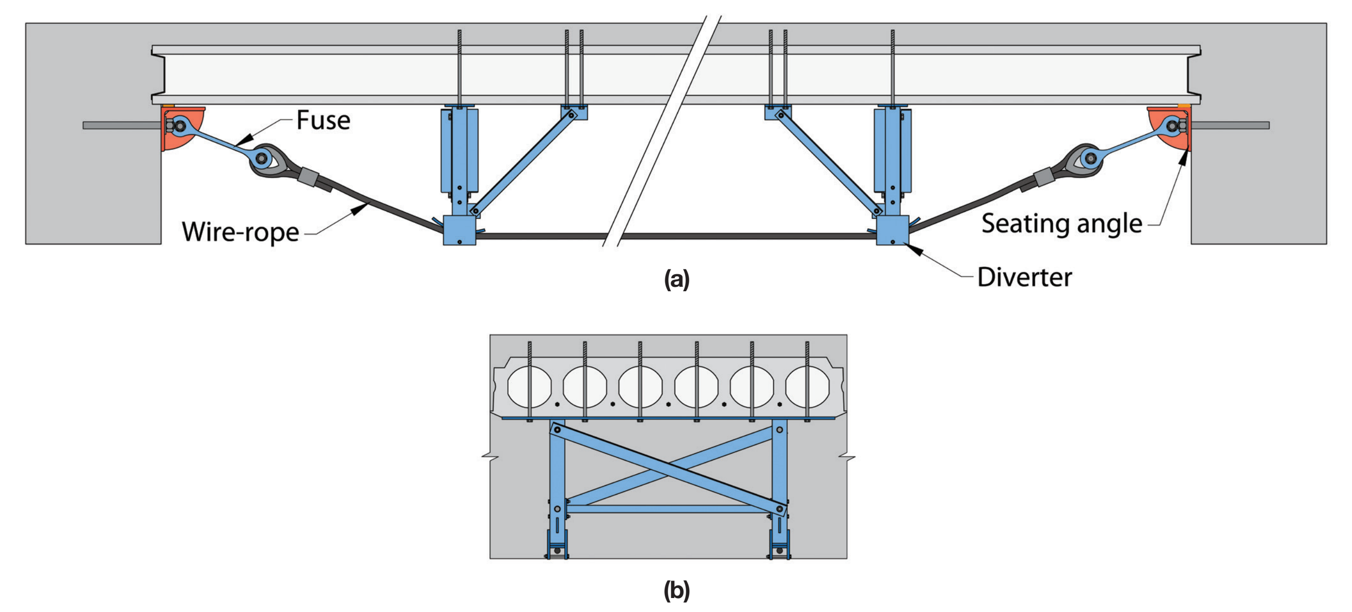


Figure 6: Cable catch system showing (a) side elevation and (b) cross section through diverter

3.3 CABLE CATCH SYSTEM

The cable catch system is a newly developed retrofit technique for hollow-core floors. Two wire ropes, spanning underneath and parallel to a hollow-core unit, are deflected by diverters located approximately one meter from each end of the unit. The wire ropes are connected to stiffeners welded to steel brackets that are anchored to the beams, which also act as supplementary seating and should be designed using the dropped-angle approach. The cable catch retrofit is shown schematically in Figure 6. Although the wire rope is shown to be taut in Figure 6, it is necessary to install the cable slack to allow for frame dilation. If sufficient slack is not provided elongation of the frame could induce upward forces on the diverters that would potentially 'break the back' of the hollow-core units. However, this must be finely balanced; too much slack would result in excessive drop of the floor before engagement of the retrofit.

The cable catch system is capable of mitigating LOS, PMF, and NMF. It does not necessarily address WSF, but can do provided the strength of a potentially detached soffit is considered when determining the configuration of cross beams used for the diverters.

The viability of the cable-catch system has been demonstrated during testing of a large scale super-assembly as detailed elsewhere (Büker et al. 2021, 2022c). As well as demonstrating the efficacy of the retrofit, this testing also confirmed that engagement of the cable-catch system occurs in conjunction with relatively large displacements and increased vertical flexibility of the floor.

While effective, the cable catch system is complex and is considered unlikely to be attractive as a practical solution. Consequently, there are no plans at the date of this publication to produce detailed design guidance for the cable catch system. Design can be undertaken from first principles, including those outlined in section 2. Aspects of the cable catch system that require particular consideration include:

- Determining the length of cables to ensure that sufficient slack is available to accommodate elongation of parallel beams,
- Design of the embedded anchors of the seating angle to resist the tensile forces in the cable. If possible, fastening of the seating angle should be achieved by drilling through an element and anchoring a threaded rod on the far side, and
- Providing a 'fuse' in the system is recommended to enable capacity design of the embedded anchors. The fuses used in the tested configuration also functioned as an efficient means of connecting the cables to the steel brackets.

3.4 SUPPLEMENTARY NEGATIVE MOMENT REINFORCEMENT

Supplementary negative moment reinforcement refers to the technique of installing additional reinforcement in or on the topping concrete close to the ends of hollow-core units. The intent of this technique is to increase the negative moment capacity of the floor at and beyond the end of the existing starter bars and consequently to prevent NMF.

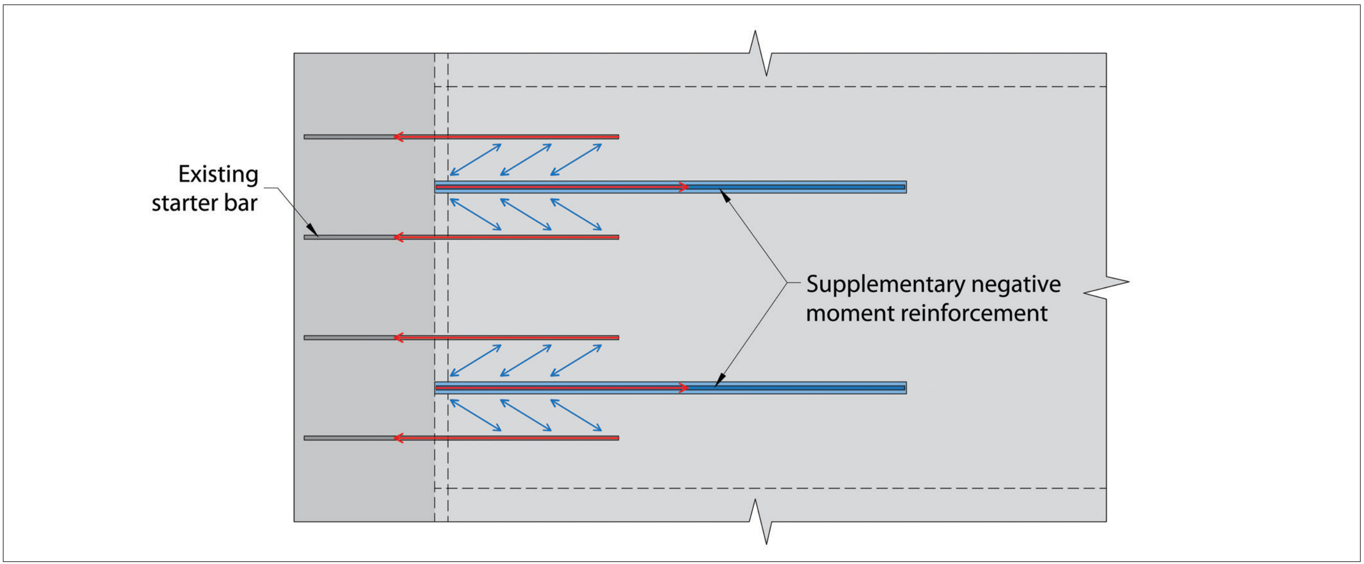


Figure 7: Example configuration of supplementary negative moment reinforcement (adapted from Parr et al. 2019)

Supplementary negative moment reinforcement can suppress NMF, with this being demonstrated in sub-assembly testing (Parr et al. 2019). In isolation, supplementary negative moment cannot prevent LOS, PMF, or WSF.

Supplementary negative moment reinforcement can consist of steel reinforcing bars chased into the topping concrete, FRP sheets or strips fixed to the topping concrete, or other configurations that provide the required tensile load paths. Testing of supplementary negative moment reinforcement has focussed mainly on solutions employing additional reinforcing bars, though an FRP configuration using FRP sheets worked satisfactorily in a sub-assembly test (Büker et al. 2020).

Supplementary negative moment reinforcement can be designed from first principles. Two key factors must be accounted for during design:

a. The supplementary negative moment reinforcement must not extend across the interface between the end of the hollow-core unit and the support beam, as this would increase the negative moment demand.

b. Forces must be able to reliably transfer between the existing starter bars and the supplementary negative moment reinforcement. Where discrete bars or FRP strips are used, consideration should be given to ensuring that this transfer can occur via a non-contact lap splice even if the existing starter bars are unfavourably positioned. As noted in section 3 it should not be assumed that starter bars are located precisely as indicated by drawings. Scanning to identify the actual locations may be beneficial.

Additionally, if FRP is being used as supplementary negative moment reinforcement it is important that FRP sheets not span across the interface between adjacent hollow-core units. If sheets do span between units this creates a risk that differential movement of the hollow-core units will cause the FRP to delaminate from the topping concrete. This restriction should not be assumed to apply if FRP is used for diaphragm strengthening, where different considerations may apply.

The use of steel reinforcing bars is generally preferable because, in comparison to FRP, these are more ductile, less reliant on bond of epoxy, and simpler to protect from wear and tear. Additionally:

- Design guidance for FRP (ACI Committee 440 2017) states that FRP sheets must not be anchored on a concrete surface that is expected to crack. This calls into question the ability of the FRP sheets from achieving full bond capacity within the region of topping concrete proximate to the end of a hollow-core unit.
- The use of steel reinforcement as a negative moment strengthening measure is particularly recommended over FRP products in regions where diagonal cracking in the topping of the floor is expected (e.g. in the vicinity of columns). This is because the ductile behaviour of steel reinforcing bars would be especially beneficial when subjected to local shear deformations as can occur across diagonal cracks.

3.5 SUPPLEMENTARY POSITIVE MOMENT REINFORCEMENT

Supplementary positive moment reinforcement refers to reinforcement, typically FRP sheets, that is applied to the soffit of hollow-core units. The intent of the reinforcement is to prevent positive moment cracking away from the

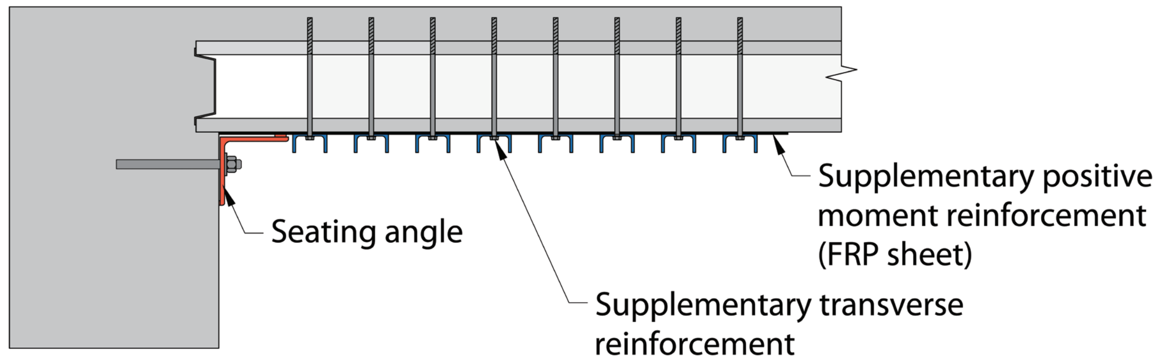


Figure 8: Example configuration of supplementary seating and supplementary positive moment and transverse reinforcement

support face. However, the reinforcement cannot prevent cracking close to the support face as sufficient bond length is required on either side of a potential crack in order for the supplementary reinforcement to be effective. Hence, supplementary positive moment reinforcement on its own is not considered an effective retrofit technique. However, used in conjunction with other techniques, structural mechanics suggest it may be effective as a means of addressing PMF, though this has not been validated experimentally. The additional measures required comprise:

- Supplementary seating (section 3.1) to prevent failure if a positive moment crack forms at the face of the support, i.e. beyond the point where post-installed supplementary positive moment reinforcement can suppress cracking, and
- Supplementary transverse reinforcement (section 3.6) of the end regions of the hollow-core to prevent failure if the crack from the support face propagates at a shallow angle as discussed in section 3.1.

Figure 8 shows an example of the configuration of retrofits envisaged. It must be emphasized here that this combined retrofit technique has not been tested in the laboratory. Further testing as outlined in section 2.1.3 is required to confirm that the combination of retrofit techniques described above can reliably prevent PMF. Until such testing is completed, the use of strongback supports (section 3.2) is recommended to address PMF.

As with supplementary negative moment reinforcement, FRP used as supplementary positive moment reinforcement should not span between hollow-core units.

3.6 SUPPLEMENTARY TRANSVERSE REINFORCEMENT

The brittle, unreinforced nature of the webs is one of the key vulnerabilities of hollow-core units. If correctly detailed, and depending on the nature and quantity provided, supplementary transverse reinforcement can:

a. Increase the shear strength of the hollow-core unit,
b. Prevent propagation of web cracks, as demonstrated during recent testing (Büker et al. 2022c), and
c. Prevent collapse of the bottom flange of a hollow-core unit in the event of WSF by providing a secondary load path to the topping and top flange.

Supplementary transverse reinforcement is likely to be an effective means of mitigating WSF, but this has not yet been experimentally validated. Ongoing research is likely to produce relevant results in the near future (Mostafa - In preparation).

Supplementary transverse reinforcement in conjunction with supplementary positive moment reinforcement and supplementary seating is also likely to provide an effective means of addressing PMF, though as discussed in the preceding section the efficacy of this combination has not been tested experimentally and strongback supports are recommended to address PMF. Supplementary transverse reinforcement is also an important part of the strongback support described in section 3.2 and a companion paper (Büker et al. 2022a).

Previous guidance (PCFOG 2009) recommended supplementary transverse reinforcement comprising either steel rods or FRP extending through holes drilled through the full depth of the floor. While the use of FRP supplementary transverse reinforcement is feasible, there appear to be no reasons that suggest it would be preferable to steel reinforcement and the anchorage

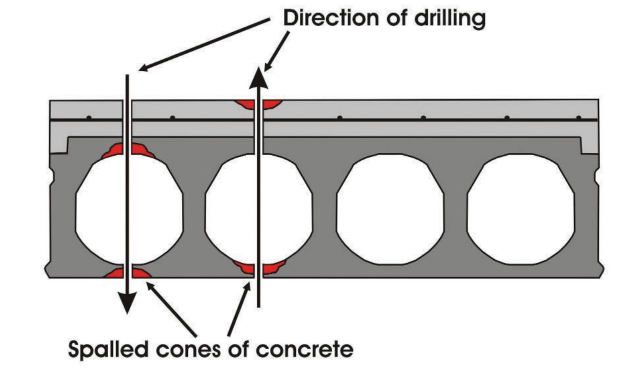


Figure 9: Illustration of the propensity for drilling through hollow-core units to cause spalling (Jensen 2006)

of FRP may be considered less reliable. The process of drilling holes for installation of either option risks damaging the hollow-core unit by causing concrete to spall as shown in Figure 9. Jensen (2006) investigated the process of drilling through hollow-core units, and recommends drilling from the underside on the basis that spalling of the topping concrete is less problematic than spalling of the unit near the strands and to minimise cosmetic impacts at the soffit. Drilling from the underside is also preferable as it simplifies alignment of the drilled holes with the voids of the hollow-core unit.

It remains critical that supplementary transverse reinforcement extend through the soffit of a hollow-core unit to ensure that the reinforcement engages with flexural tension forces in the soffit. Recent testing (Büker et al. 2022c) has shown, however, that satisfactory performance can be obtained through use of screw anchors installed from beneath and extending into, but not through, the topping concrete. As recommended in previous guidance (PCFOG 2009), supplementary transverse reinforcement could be tensioned to induce compression in the hollow-core webs, which can delay or arrest crack formation.

It is recommended that anchorage of supplementary transverse reinforcement at the soffit be achieved by use of steel sections that span across the hollow-core webs (Figure 10), rather than by washers as indicated in previous guidance. Steel plates or channel sections would both be viable options. The use of such cross beams provides an improved load path for compression struts through the webs, and also mitigates the risk of damage to the hollow-core unit that can occur when drilling holes in which to install supplementary transverse reinforcement (Jensen 2006).

Detailed design of supplementary transverse reinforcement depends on the purpose of the reinforcement:

- Where the purpose of supplementary transverse reinforcement is to prevent collapse in the event of WSF, the provided reinforcement must be sufficient to support the weight of the lower half of the hollow-core unit. The spacing of reinforcement must be such that the lower half of the unit can span between the reinforcement locations without failing. It is recommended that a minimum of two rows of reinforcement should be provided.
- Where the purpose of supplementary transverse reinforcement is to increase the shear strength of hollow-core units, design should be undertaken following the applicable provisions of the Concrete Structures Standard (SNZ 2017). In order to be effective as shear reinforcement the minimum spacing of reinforcement along the span of the hollow-core unit should not exceed half the effective depth of the floor (i.e. $s \leq d/2$).

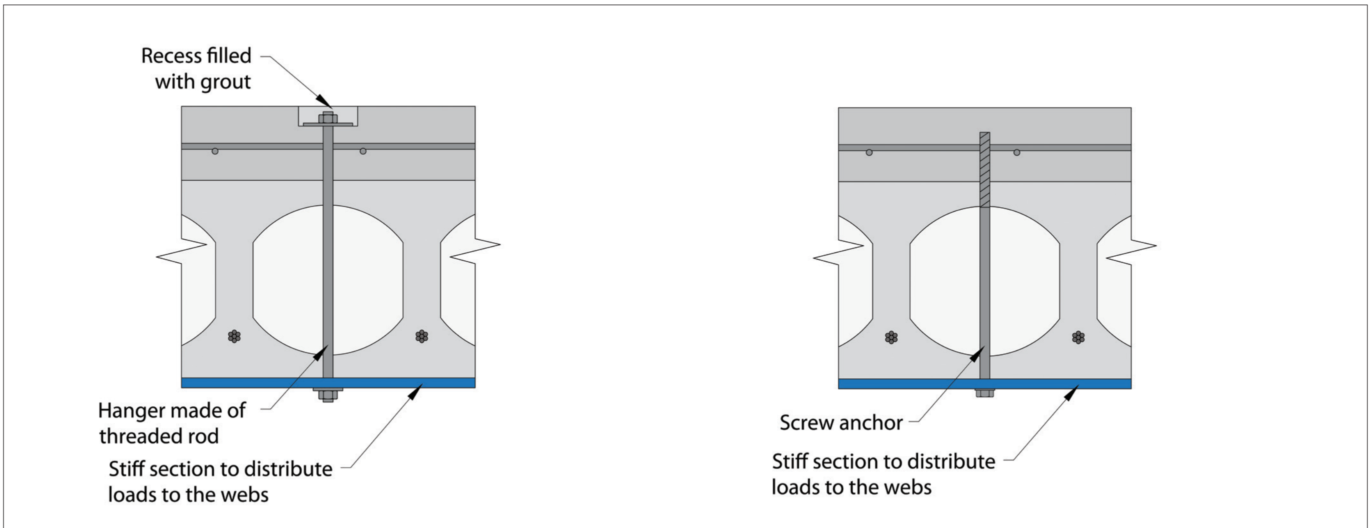


Figure 10: Steel supplementary transverse reinforcement configurations using screw anchors (left side) or threaded rods (right side)

3.6.1 Ineffective supplementary transverse reinforcement

Two forms of supplementary transverse reinforcement are identified in previous guidance (PCFOG 2009) as NOT being reliable retrofit techniques. These are:

a. Supplementary transverse reinforcement anchored into (filled) cores, and

b. Supplementary transverse reinforcement installed at the interface between adjacent units.

These retrofit techniques are still considered ineffective and are not recommended. They do not satisfactorily address LOS, PMF, NMF, or WSF.

The first technique is afflicted by the issues pertaining to infilling of cores that are discussed in section 3.8. Notably, the termination of the supplementary transverse reinforcement within the core means there is still no reliable load path between the top and bottom flanges of the unit should web cracking occur. This is elaborated on in section 3.8.

While supplementary transverse reinforcement at the interface between units would be expected to extend through the full depth of the floor, the widely spaced reinforcement is unlikely to adequately support the central parts of the bottom flange of the hollow-core unit. While this could conceptually be addressed by providing cross beams between the supplementary transverse reinforcement, it seems unlikely such a solution would be preferable to installing more closely spaced reinforcement. At most, this retrofit approach can only be used to try to stop a soffit from falling after failure since the wide spacing does not enable the supplementary transverse reinforcement to improve the shear resistance of the middle webs or limiting web crack propagation as observed with screw anchors (Büker et al. 2021).

3.7 CATCH BEAMS

Various retrofit techniques for hollow-core floors can collectively be described as catch beams. These include:

- 'Simply supported' catch beams illustrated in Figure 11. This configuration is similar to that described in previous guidance (PCFOG 2009), but has the notable addition of a deformable layer between the unit and catch beam. For this retrofit configuration it is important that the connection to the support beam is detailed to act as a (nominally) pinned support to avoid excess prying of the anchors. The ability of the adjacent hollow-core unit to support additional demands also requires careful checking,
- Catch beams designed to cantilever from adjacent primary structure (Figure 12), and
- Catch beams spanning directly between primary structural elements, such as the example shown in Figure 13 installed diagonally across the end of an alpha unit. To accommodate elongation of the supporting beams, such beams must be provided with the ability to rotate in the horizontal plane and to elongate. This has previously been achieved by provision of a sliding support at one end as indicated in Figure 13, or by use of telescoping steel hollow sections to provide elongation in conjunction with a support at one end that is detailed as a pin in the horizontal plane.

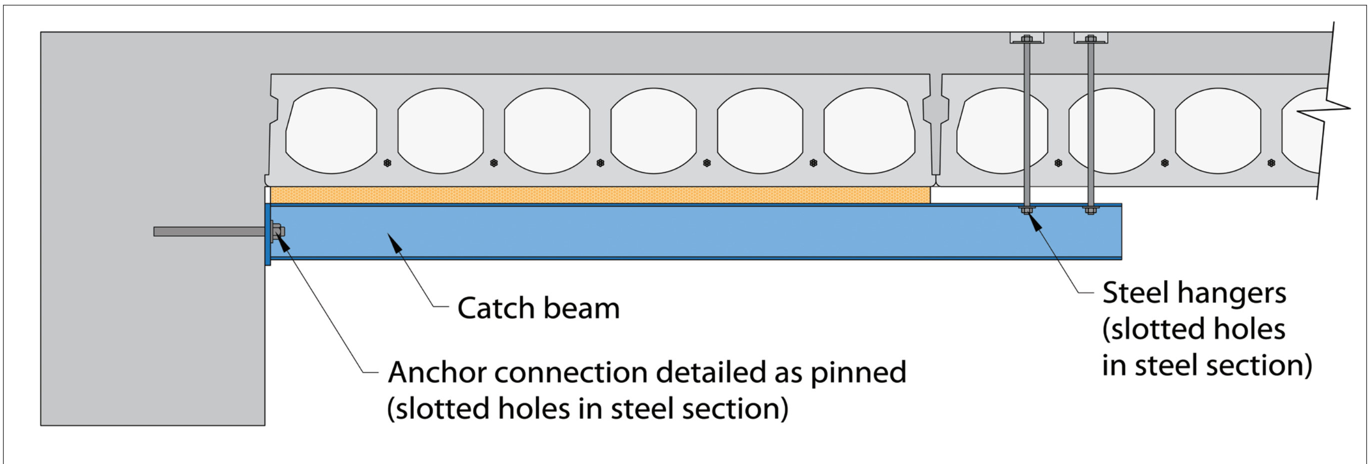


Figure 11: Simply supported catch beam

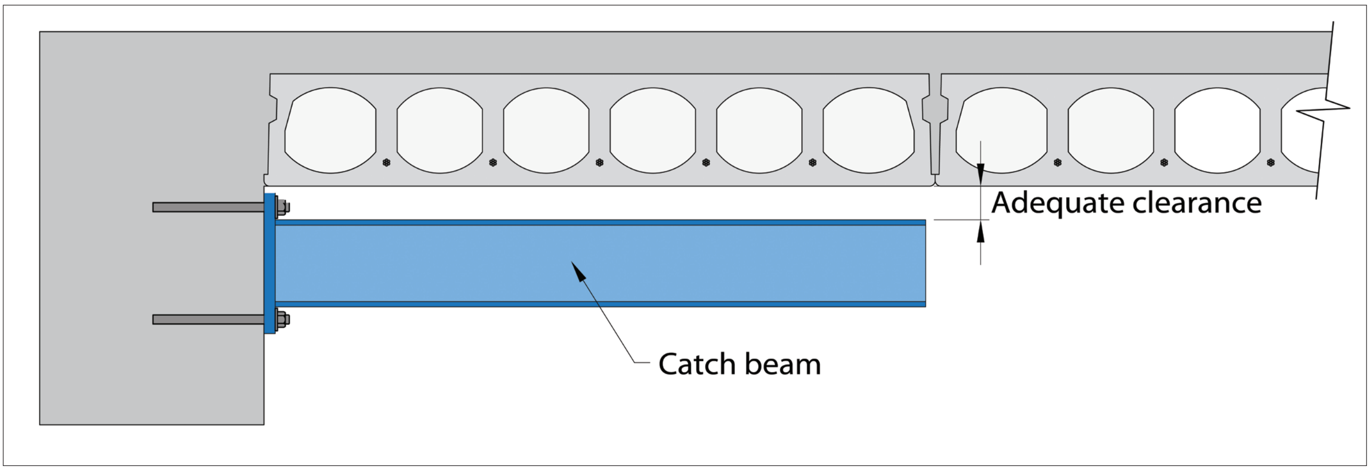


Figure 12: Cantilever catch beam

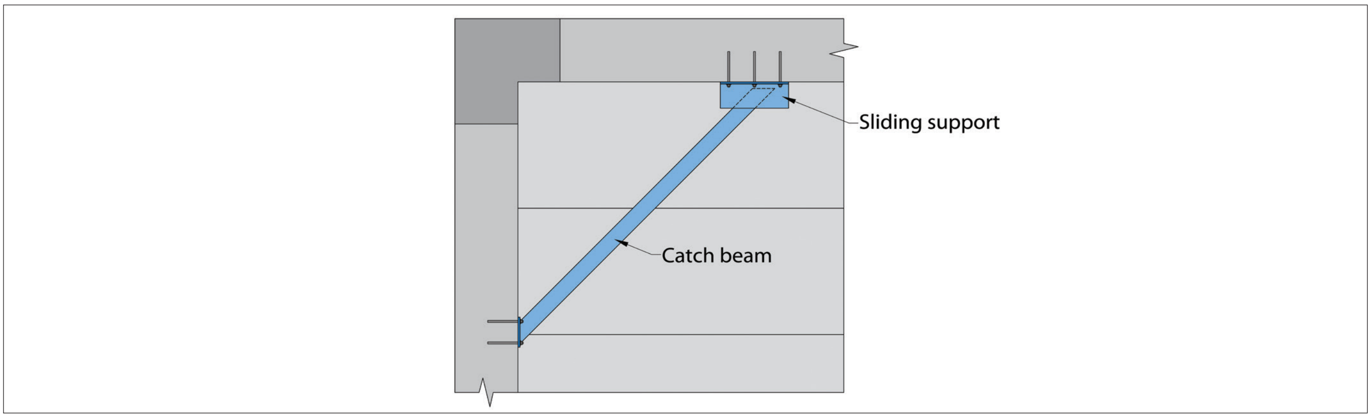


Figure 13: Diagonal alpha catch bracket with sliding support

Provided they are appropriately positioned, catch beams can address LOS, PMF, NMF, and WSF. As indicated by the name, catch beams function as a catch system as defined in section 2.1.2.

Key aspects of the design of catch beams that require consideration include:

1. The strength of the catch beam and the structure that supports it must be sufficient to support the ULS demands resulting from the weight of hollow-core unit(s) required to be caught. ULS demands should be determined based on the design action combinations outlined in section 2.2.
2. The spacing of catch beams must be sufficient so that the hollow-core unit (or parts thereof) can span reliably between the catch beams. This can also be achieved through a combination of catch beams and supplementary transverse reinforcement.
3. Compatibility of displacements and rotations between the catch beams, hollow-core units, and primary structure must be carefully considered. This can include both vertical movements, horizontal movements (e.g. due to beam elongation), as well as relative rotations between the support structure and the floor. The impact of forces arising from deformation (in)compatibility on the hollow-core unit and retrofit anchorages must be assessed.
4. If displacement compatibility is accommodated by providing clearance between the beams and the hollow-core units, consideration should be given to the potential for increased demands due to dynamic impact. It is recommended that a deformable layer be placed between the catch beam and the hollow-core unit, which removes the need to explicitly consider impact forces as outlined in section 2.3. For catch beams a relatively soft deformable layer such as polystyrene is likely to be appropriate.
5. The catch beam must be robustly supported. Particular attention is required for anchors used to fix the beams to the support structure. As outlined in section 2.5, these cannot be relied on to resist forces when installed in regions expected to experience substantial plastic deformation. This unreliability is not reflected in catch beam configurations suggested in previous guidance (PCFOG 2009 - refer Figure 8.14).

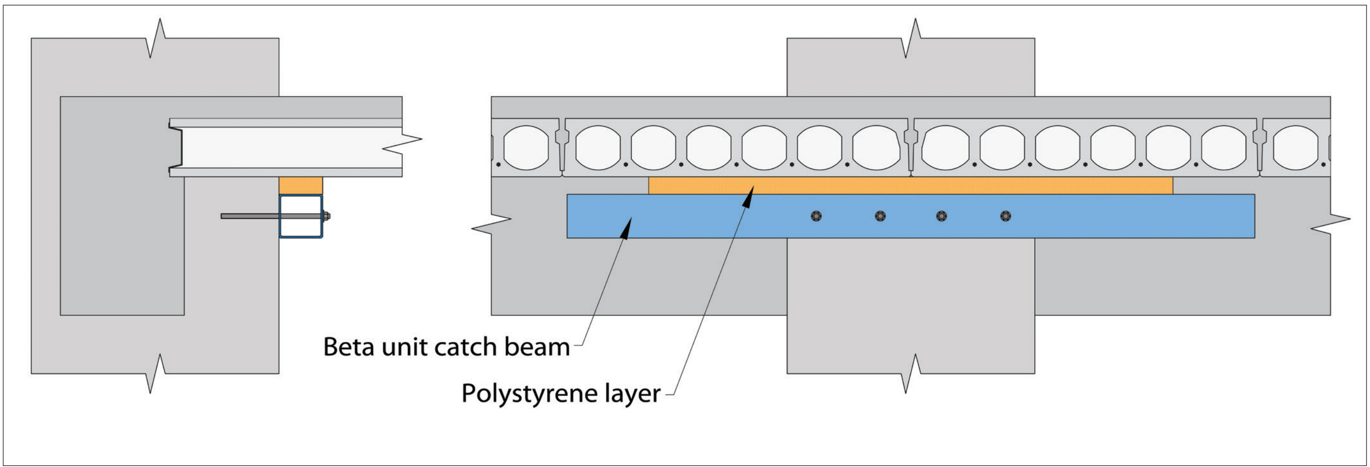


Figure 14: Example beta unit catch beam configuration

Provision of catch beams for beta units may pose particular challenges, in many cases requiring the catch beam to cantilever from the column that interacts with the beta unit. An example configuration is shown in Figure 14 that was demonstrated to be effective during super-assembly testing (Büker et al. 2022c). This catch beam extended to approximately 400 mm beyond the face of the support beam. As an interim recommendation until understanding of beta unit behaviour is refined, it is recommended that beta unit catch beams be detailed to support the beta unit a similar distance into the span, i.e. so that they extend to approximately twice the unit depth from the support beam face.

3.8 INFILLING OF CORES

Infilling of cores at the ends of hollow-core units is NOT considered an effective means of improving the performance of hollow-core floors.

Despite being identified as ineffective since at least 2009 (PCFOG 2009), discussion regarding retrofit techniques for hollow-core floors regularly turns to the concept of infilling cores. It is typically proposed that this will increase the shear strength of the hollow-core units, or otherwise enhance performance.

The previously-expressed rationale for deeming core infilling to be ineffective (PCFOG 2009) remains sound. However, this inadequacy warrants elaboration. Concerns about the approach are twofold:

- The presence of infilled cores will increase the flexural strength and stiffness of the end region of the hollow-core unit, which is likely to increase vulnerability to negative or positive moment failure outside the filled region.
- As illustrated in Figure 15a, for this infill to effectively increase the shear strength of the end of the unit it must contribute to resisting vertical tensile forces that have to anchor effectively into the top and bottom flanges where the flexural tension and compression forces are located. The infill material itself is not prestressed, and so depending on its composition it is unlikely to be able to reliably resist tension forces. Irrespective of its composition, the infill material is separated from the hollow-core unit by a cold joint. Even in favourable circumstances cold joints have greatly reduced tensile capacity compared to concrete (Torres et al. 2016). This reduction is likely to be even more significant in relation to the inside of a hollow-core unit that is likely to be old at the time that core infilling is undertaken, smooth/unroughened, and dusty.

The ineffectiveness of core infilling as a means of retrofit can be demonstrated by reference to previous testing that considered core infilling as a solution for new hollow-core (e.g. Liew 2004). For example, the test shown in Figure 15b clearly shows the clean separation of the core infill from the hollow-core unit. While the filled core in the unit shown in Figure 15b contained 'paperclip' reinforcement, this does not detract from the observation that no effective force transfer could occur across the cold joints at the top and bottom of the infill.

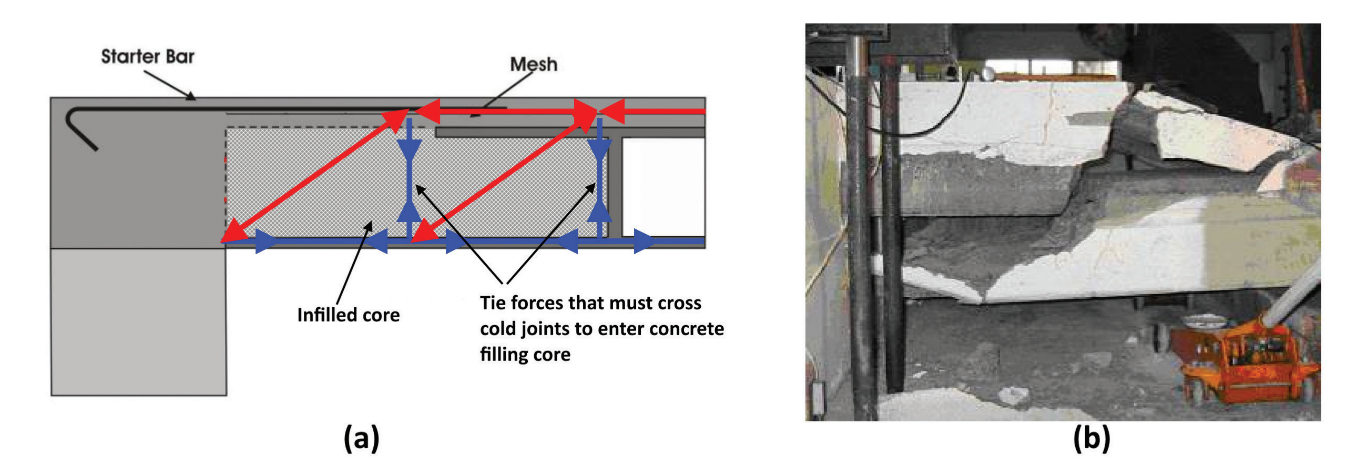


Figure 15: (a) Illustration of necessity of tension forces crossing cold joints at top and bottom of core and (b) failure during testing showing debonding of hollow-core unit from infilled core (adapted from Jensen 2006)

The discussion above applies equally irrespective of whether the proposed infilling material comprises plain concrete, fibre reinforced concrete, or other alternatives such as fibre reinforced polymer (FRP) tubes.

3.9 RELEASE OF NEGATIVE MOMENT RESTRAINT

In contrast to previous guidance on retrofit of hollow-core floors (PCFOG 2009), based on current knowledge it is concluded that:

- NMF cannot be prevented by inducing a crack in the topping concrete (and any core infills) at the end of a hollow-core unit, and that
- Cutting of reinforcement crossing the interface at the end of the hollow-core unit can be an effective retrofit technique provided that diaphragm capacity is maintained.

Inducement of a crack through the concrete at the end of a unit is not sufficient to prevent NMF. Inducement of the crack does not preclude yielding and strain hardening of reinforcement crossing the crack, and hence does not materially reduce the negative moment that can develop at the end of the unit and that dictates the propensity for NMF (Parr et al. 2019).

Cutting starter bars and other reinforcement that cross the end of the hollow-core unit can be a valid retrofit strategy for NMF as it reduces restraint and therefore negative moment demand at the end of the unit and consequently at the end of the starter bars. However, it also has the side effect of reducing the strength of the floor diaphragm. The impact of this reduction requires detailed consideration.

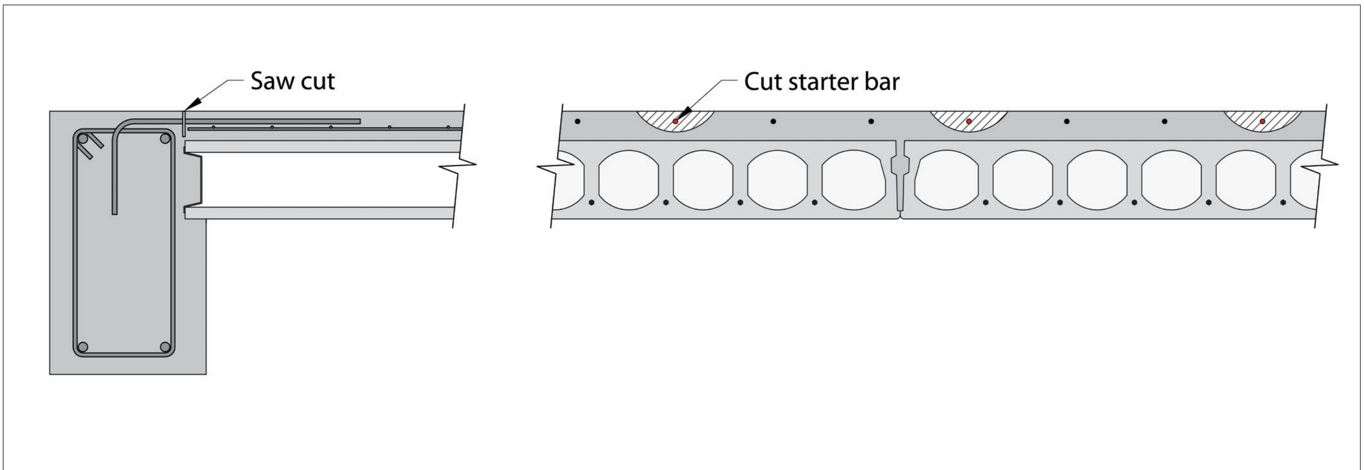


Figure 16: Illustration of cutting starter bars

4 RETROFIT TECHNIQUES FOR DOUBLE TEE FLOORS

In comparison to hollow-core floors, double tee floors are susceptible to fewer modes of failure. As outlined by the assessment guidance (MBIE et al. 2018), double tee floors:

- Are generally susceptible to loss of support (LOS) failure, and
- May be susceptible to failure of the flange support where units are supported by "loop bar" (i.e. "pigtail") hanger details.

Hare et al. (2019) identified that there was no rational method for determining the reliable capacity of loop bar double tee supports. Despite further research since (Jenkins 2022), this remains the case.

Both of the above noted failure modes occur at the end of the unit and result in the unit dropping relative to the support beam. Consequently, similar retrofit techniques, outlined below, are applicable to both failure types.

It is essential that retrofit techniques for double tee units provide concentrated support to the webs of the unit because it is the webs that transfer the shear forces arising from gravity demands to the supporting structure. Techniques that provide remedial support only via the flanges of units are unlikely to effectively support the double tee unit after failure of the original load path. The flanges of double tee units are slender, and are not likely to have sufficient reinforcing to support the full weight of the floor or accommodate transfer of the web shear demands into the support.

Details regarding recent experimental investigations of double tee retrofit techniques along with more detailed design recommendations are expected to be described in a future paper (Henry and Jenkins - In preparation).

4.1 SUPPLEMENTARY BRACKETS

Supplementary brackets are a common retrofit technique for double tee floors that could be implemented for units that are originally either web or flange supported. As suggested by the name, the technique provides bracket supports at the end of each web of a double tee unit. Generally the supplementary brackets are installed as a catch mechanism with an initial gap between the web and bracket, i.e. they prevent collapse rather than preventing failure of the original support detail. An example support detail is shown in Figure 17.

Supplementary brackets for double tee units are typically more substantial than supplementary seating used for hollow-core floors (section 3.1). More substantial supports are required due to the concentration of support reactions in the two webs of the double tee unit, which contrasts with relatively even distribution of reactions across the width of hollow-core units.

When support beam rotation occurs that induces a negative moment in the connection of a flange hung double tee unit, there is potential for the webs of the double tee to bear against the face of the supporting beam and cause LOS due to prying. The tendency for this to occur is reduced or removed by the presence of a gap between the beam face and the end of the web. It is imperative that retrofit techniques do not encroach on this gap, as doing so would increase the chance of LOS occurring. Consequently, supplementary brackets should be installed such that components are located either under the double tee unit, or around the web as shown in Figure 17.

As noted, supplementary brackets should generally be designed as a catch system with a gap provided between the soffit of the double tee web and the support face of the supplementary bracket. The gap is required to prevent trapping the web in the retrofit and to avoid unpredictable large demands being placed on the

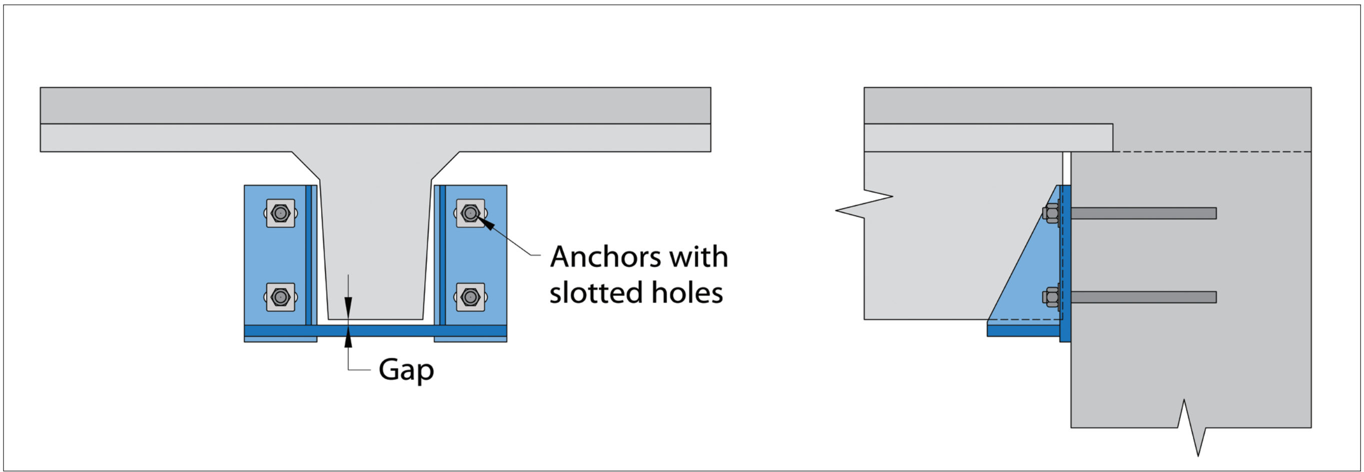


Figure 17: Example supplementary bracket

bracket due to relative displacement between the bracket and double tee when support beam rotation occurs.

Assuming that the supplementary bracket is designed so that it does not contact the double tee unit when the building sustains lateral drifts, it is not necessary to use capacity design to determine demands on the anchors for the bracket. Instead, they should be sized to resist the design action combinations discussed in section 2.2, with appropriate consideration given to dynamic impact forces that could arise if the double tee unit is able to drop onto the supplementary bracket. It is recommended that a deformable layer be placed between the bracket and double tee soffit to avoid the need to consider dynamic impact forces as outlined in section 2.3. Design of the deformable layer should follow the principles outlined for hollow-core supplementary seating in a companion paper (Büker et al. 2022b).

As with supplementary supports for hollow-core, horizontally slotted holes may be beneficial to simplify installation and mitigate potential clashes with existing reinforcement.

4.2 ARTICULATING HANGER SUPPORTS

Articulating hanger supports comprise a steel beam installed beneath the web soffit that is attached to the supporting structure at one end and to the web of a double tee unit via a pair of pinned links at the other end. Parallel flange channel (PFC) sections are likely to be appropriate for the steel beam. The links made from steel flats should be connected by rods inserted through the double tee web and the PFC and as long as practical to attach towards the top of web section. An example of the detail is shown in Figure 18. It is important that the connection of the PFC to the support beam is detailed to behave as a pin so that prying forces on the anchors are avoided.

The intent of the detail is that elongation demands can be accommodated by articulation of the links. Occurrence of this articulation requires a vertical clearance to be provided between the PFC and the double tee unit soffit which will close up as the links project at an angle due to elongation demands. The need for this gap means that the articulating hanger support acts as a catch system as defined in section 2.1.2.

An alternative detail has been proposed but not tested. The suggested difference involves placing a deformable plastic shim or other deformable layer on the PFC near to the end of the double tee web and with a nominal gap between the shim and web soffit. This gap would be sized so that the floor becomes supported when the hanger reaches its maximum extension.

If the magnitude of elongation is sufficient, the gap between the PFC and double tee soffit will close and the articulating hanger can bind as illustrated in Figure 19. Binding would be likely to overstress the anchors connecting the articulating hanger to the support structure. To mitigate against this risk it is recommended that:

- The potential elongation used when dimensioning the articulating hanger should be based on the peak MCE drift as discussed in section 2.2, and
- The demands on the anchors should be protected by a capacity design approach if practical.

Hanger retrofits of this type had been installed in Statistics House prior to the Kaikoura earthquake, but not in locations where double tee units lost support (MBIE 2017, 2018). Consequently the real-world performance of the retrofits was not tested by the Kaikoura earthquake.

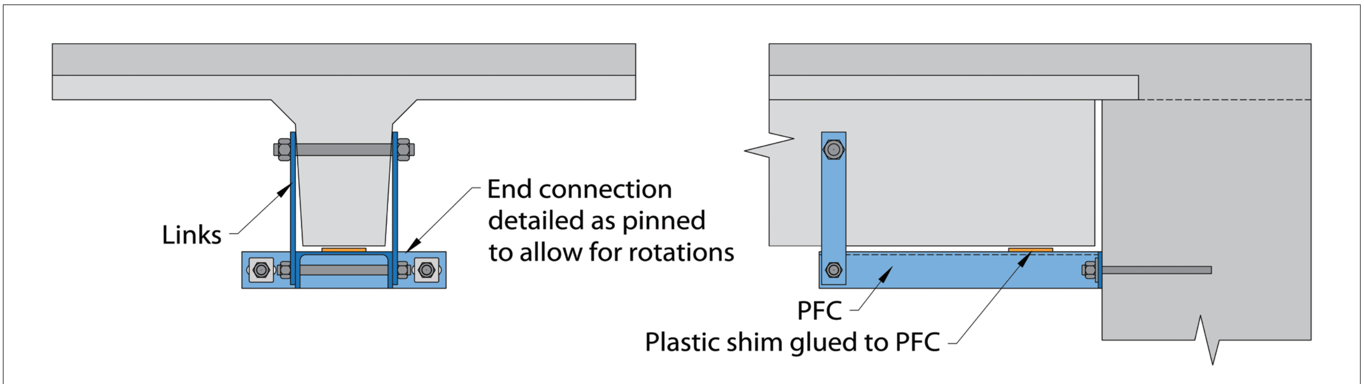


Figure 18: Example of articulating hanger configuration

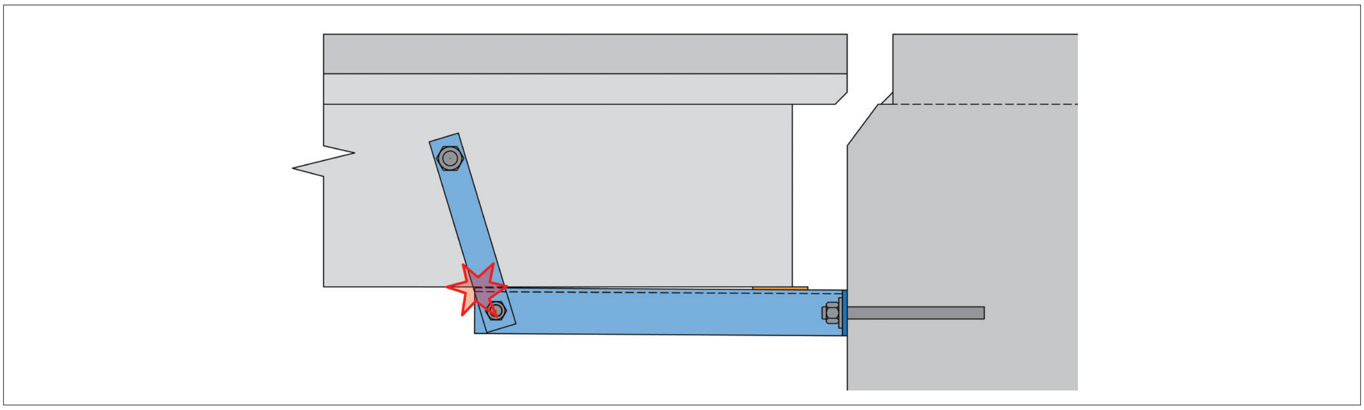


Figure 19: Illustration of potential for hanger binding (adapted from Jenkins 2022)

4.3 RETROFITTED CAZALY HANGER

Instead of providing a catch system for double tee units, it may be possible to provide an improved support system by retrofitting steel elements in a configuration similar to a Cazaly hanger (PCI Industry Handbook Committee 2010). The typical configuration of a Cazaly hanger is shown in Figure 20. Modifications would be required to enable this to be post-installed rather than cast in to the double tee unit:

- A hollow section, channel, or flat plate would need to be fixed in or on top of the topping concrete,
- A tie would be fixed at or near the soffit of the double tee unit (potentially externally mounted), and
- Rods, or other vertical ties, would be required to connect the top and bottom elements. These would logically be placed either side of the double tee web through holes cored in the flange.

Although a retrofitted Cazaly hanger presents a viable retrofit solution for flange hung double tees, it should be noted that no such solution has been designed or tested to validate the approach.

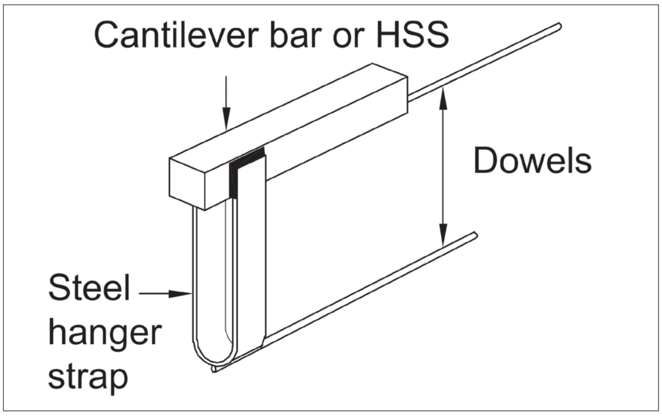


Figure 20: Cazaly hanger (PCI Industry Handbook Committee 2010)

5 RETROFIT TECHNIQUES FOR FLAT SLAB AND RIB AND INFILL FLOORS

In addition to the hollow-core and double tee floors discussed in the preceding sections, retrofit techniques may also be required for precast floors constructed using flat slab precast units or precast ribs with timber infills.

No specific research has been undertaken to investigate retrofit techniques for these other types of floors. However, it is reasonable to consider that:

- Flat slab floors have similarities to hollow-core floors, albeit flat slab floors are typically more robust due to the absence of voids and slender unreinforced webs which precludes WSF, and
- Rib and infill floors have similarities to double tee floors, but are vulnerable to a wider range of failure modes. Notably, rib and infill floors may be vulnerable to PMF if the ribs are 'trapped' at the supports.

In light of the comments above and the absence of specific research, it is recommended that retrofit techniques for hollow-core and double tee floors be adapted for use with flat slab and rib and infill floors respectively. For rib and infill floors care should be taken to ensure that retrofits extend far enough to mitigate PMF if a floor is vulnerable to this behaviour.

6 CONCLUSIONS

Design and detailing of efficient retrofits for precast concrete floors requires that retrofits be tailored to reflect the specific behaviour of individual precast units within the floor. It is unlikely to be efficient to assume that the same retrofit is applicable across an entire floor or building. Design of effective retrofits requires consideration of many factors, notably including the failure mode(s) expected to affect the unit considered and deformation (in)compatibility between the floor unit, retrofit components, and other elements of the existing structure.

In relation to hollow-core floors:

- Retrofits that have been shown experimentally to address one or more failure modes comprise supplementary seating, strongback supports, the cable catch system, supplementary negative moment reinforcement, catch beams, and cutting of starter bar reinforcement.
- Supplementary transverse reinforcement is expected to be an effective retrofit technique, but has not yet been experimentally validated.
- Supplementary positive moment reinforcement has not been demonstrated to be an effective retrofit. It may be useful in conjunction with other measures, but this has not been experimentally validated.
- Infilling of cores and inducement of cracking at the ends of units are not effective retrofit techniques.

Double tee floors may be effectively retrofitted using supplementary brackets or articulating hangers. Post-installed Cazaly hangers may also be an effective retrofit, but this has not been demonstrated experimentally.

Retrofit techniques for flat slab and rib and infill floors would be broadly similar to those for hollow-core and double tee floors respectively.

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