# NUMERICAL SIMULATION OF THE SEISMIC BEHAVIOUR OF PRECAST PRE-STRESSED HOLLOW-CORE FLOOR SEATING CONNECTIONS

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## Abstract

Recent earthquakes have highlighted some issues with the seismic performance of relatively modern reinforced concrete buildings. In particular, the early use of Precast Pre-stressed Hollow-Core (PPHC) floors saw the use of support connections that are susceptible to undesirable failure modes when subjected to earthquake-induced deformations. In the past twenty years, extensive experimental research programs in New Zealand have helped identify some of the main vulnerabilities of these floors and led to the development of support connection detailing capable of accommodating larger earthquake demands.

While improved connection details have been developed, concerns regarding the seismic performance of buildings containing PPHC slabs were raised following the 2016 Kaikōura Earthquake. The earthquake dynamic characteristics caused high drift demands on some multi-story moment frame buildings resulting in damage to floor diaphragms, in particular. Furthermore, post-earthquake observations evidenced inconsistencies between the observed damage conditions and the damage states identified by past research, bringing into question the seismic assessment of buildings with this type of floor system, the residual capacity of the floors once the damage has been sustained, and the effectiveness of existing retrofit techniques.

This paper presents a comprehensive three-dimensional finite element (FE) modelling approach for PPHC floor seating connections, validated against experimental data collected from previous research programs in New Zealand. Material constitutive laws for concrete and steel, interaction behaviour for the concrete-concrete interface, and interaction behaviour for the concrete-steel bond are presented. The results of finite element modelling, that incorporates nonlinear fracture mechanics, have been used to numerically predict the drift values expected to cause the following failure modes: loss of seating, flexural positive and negative moment failures. The results of this research are helping inform methods for assessing and improving the seismic performance of PPHC floors in New Zealand.

*Keywords: finite element model, seismic behaviour, hollow-core slab, precast concrete, fracture mechanics*

## 1 Introduction

Buildings with precast concrete floors comprise a large percentage of the commercial building stock in New Zealand cities. Anecdotal evidence, based on post-earthquake inspections of buildings, suggests that over 60% of commercial floor area in Wellington falls into this category [1]. Observations suggest the proportion in Christchurch would have been similar until the earthquakes of 2010 and 2011, and there is no reason to think the situation is any different in Auckland and other major centres. New Zealand's extensive use of precast floors in regions of high seismicity is unusual, with in-situ floors more commonly used internationally. Consequently, and in contrast to most other deficiencies found in existing buildings, limited international research appears to be available regarding the adequacy of existing precast floors.

The partial collapse of precast concrete flooring components in Statistics House during the Kaikōura earthquake, together with several other buildings in Wellington damaged beyond economic repair, sparked serious concerns about the seismic performance of precast floors [2]. Of particular importance was the presence of damage to the floors that were inconsistent with the failure modes identified during previous research. This has brought into question the seismic assessment of buildings with these floors, the residual capacity of the floor once the damage had been sustained, and the effectiveness of the existing retrofitting techniques.

To help address these concerns, a comprehensive campaign of three-dimensional finite element (FE) analyses is underway. The modelling approach aims to numerically predict the drift values expected to cause loss of seating and flexural moment failures in the seating connections of Precast Pre-stressed Hollow-Core (PPHC) floor systems. The FE models developed will be first validated against data collected both from past and ongoing experimental programs and will then be used to parametrically investigate key aspects of the performance of PPHC floors. This paper presents advances in the initial modelling phases and describes the modelling approach, main assumptions, and initial results compared against experimental data.

## 2 Vulnerability of hollow-core floors and overview of research phases

Experimental research into the behaviour of PPHC floors has been undertaken during the last 25 years in New Zealand providing a significant basis for understanding the response of precast floors to earthquake-induced actions. Large scale floor tests at the University of Canterbury [3, 4, 5] highlighted two actions that were found to cause the most damage to the precast concrete floor unit support connections, which are illustrated in Fig. 1. First, as a building deforms when subjected to lateral earthquake loading, the support beams rotate relative to the floors which remain approximately horizontal. This relative rotation between the PPHC floor unit and its support causes forces to be developed in the continuity reinforcement placed in the in-situ topping [6]. Second, in ductile frames, the plastic hinges that form in the beams elongate during cyclic loading as a result of inelastic straining of the beam reinforcement and dislodgment of aggregate at tensile crack locations that cause these cracks to not fully close during load reversal. The frame dilation due to this axial elongation in the beams oriented parallel to the precast concrete floor units causes the floor unit seating to be reduced with possible fall-out [7].

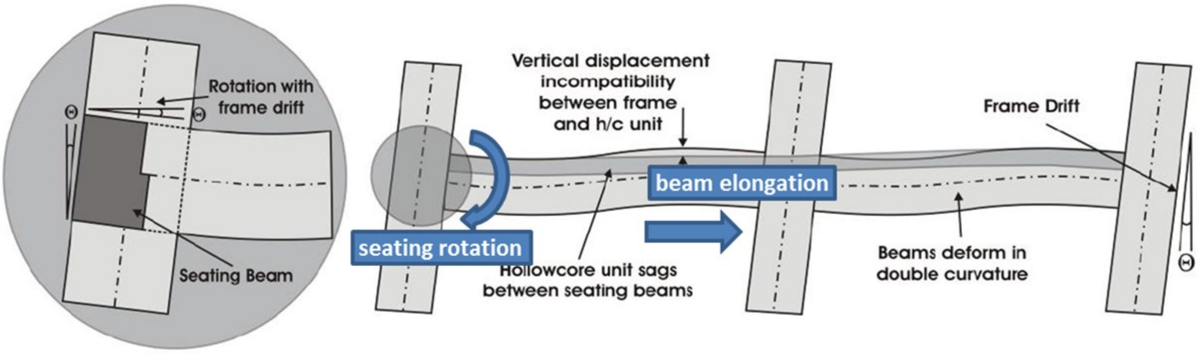


Fig. 1. Beam elongation and relative rotation between floor units and support (Courtesy of: Elwood, 2018).

The performance of PPHC slabs is complex and cannot be assessed accurately from the projected inter-story drift alone. Consideration needs to be given to local displacements and structural actions induced into the individual floor units, as it is these which are likely to cause brittle failure. Consequently, to perform numerical analyses into PPHC floor systems, the modelling approach being followed is fragmented into phases of increasing modelling complexity: unit, sub-systems, and systems (see Fig. 2). The first phase allows validation of the PPHC unit material models and examination of the web-shear strength of the units. The subsystem model helps to fulfil the objective of identifying the drift levels that cause loss of seating (LoS) or flexural moment failures (NMF) in PPHC seating connections. Ultimately, the results from the first two phases can be used to study the post-cracking behaviour of PPHC floor diaphragms at the system level.



 

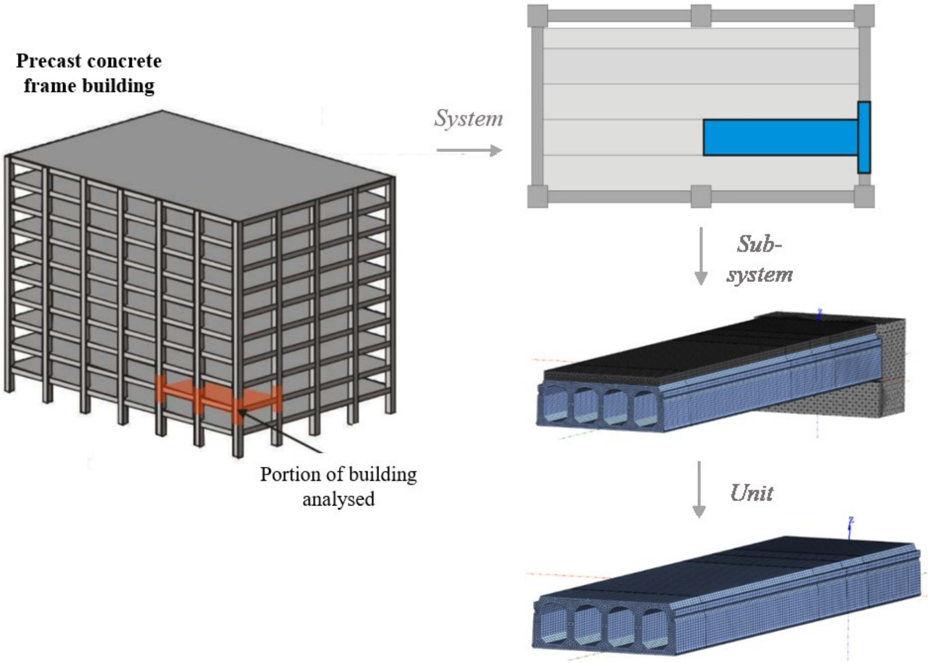


Fig. 2. Framework for the FE modelling campaign proposed for PPHC floors.

## 3 Nonlinear finite element analyses

### *3.1 Experimental database*

A detailed discussion of all available experimental tests is beyond the scope of this paper; nonetheless, a summary of the experimental database sourced and the key test results used for the development of the benchmark FE model are presented below.

At the unit level, six full-scale tests were performed on 200 mm deep PPHC units fabricated by a local precast company using the extrusion method. The 200 mm deep units were selected since this has been the most commonly used precast floor unit in New Zealand. All specimens were tested in a three-point bending test where all geometric properties remained the same across the tests with the exception of the shear span. Table 1 summarizes the test results, which have been used to calibrate the material properties and fracture mechanics of the FE model for the PPHC units.

Table 1. Summary of 3-point shear bending test results.

| Specimen | Shear span (mm) | Aspect ratio | Failure load (kN) | Deflection at failure (mm) | Failure mode |
|---|---|---|---|---|---|
| **HC1** | 300 | 1.5 | 245 | 2.2 | Web-shear |
| **HC2** | 300 | 1.5 | 248 | 2.1 | Web-shear |
| **HC3** | 500 | 2.5 | 199 | 3.4 | Web-shear |
| **HC4** | 500 | 2.5 | 190 | 3.3 | Web-shear |
| **HC5** | 700 | 3.5 | 203 | 6.5 | Flexural-shear |
| **HC6** | 700 | 3.5 | 204 | 7.1 | Flexural-shear |

NMF refers to the failure of a hollow-core floor due to the exceedance of the strength at the top of the section followed by propagation of cracking through the depth as shown in Fig. 3. Such failure typically manifests at the end of the starter bars provided to connect the floors to the supporting structure. The occurrence of a NMF is dependent on a number of factors that affect the magnitude of the moment transferred to the connection, including the strength of the starter bars and the possible presence of 'hairpin' reinforcement or other additional sources of strength (see detail in Fig. 3).

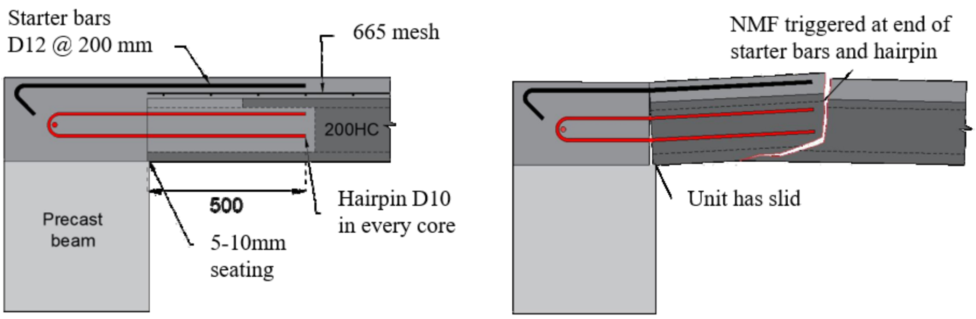


Fig. 3. Seating connection hairpin detail with expected failure

Past sub-system experimental testing was carried out by [3, 5, 7, 8] mainly focused on studying improvements into the design of seating connections. More recently, an experimental investigation has been initiated with the aim of identifying the impact of seating connection details on 200mm deep HC units that would lead to unfavourable failure mechanisms such as LoS and NMF and validating retrofit options to remediate existing vulnerable buildings containing these floor systems. Table 2 shows the test matrix for the most recent experimental testing being carried out at the moment of writing, including the loading and retrofit used on each test and the triggered failure mode.

Table 2. Testing matrix for recent sub-system PPHC testing [9]

| Test | Loading | Retrofit | Observed failure mode |
|---|---|---|---|
| **A1** | Rotation | Un-retrofitted | LoS |
| **A2** | Rotation | Flexible seating angle | NMF |
| **A3** | Rotation | Stiffened seating angle | NMF |
| **A4** | Rotation | Flexible seating angle | NMF |
| **A5** | Rotation | Stiffened seating angle + NMF bars | NMF |
| **A6** | Rotation + Elongation | Offset stiffened angle | LoS |
| **B1** | Rotation | Un-retrofitted (hairpin detailing) | NMF |
| **B2** | Rotation + Elongation | Un-retrofitted (hairpin detailing) | NMF |
| **B3** | Rotation | Un-retrofitted (short starter bars) | NMF |

### *3.2 Proposed numerical approach*

To investigate the behaviour of PPHC units, three-dimensional FE models have been created using the software Midas FEA [10], and then calibrated against full-scale test results. Experimental results and section characteristics for specimens HC1 and HC2 have been adopted (see Table 1). Mean material properties are obtained from material characterization testing into the hollow-core concrete. Fig. 4 shows the detailed solid FE model for the 200 mm depth specimen in both longitudinal and transversal directions. Concrete has been modelled employing 6-node brick elements.



 

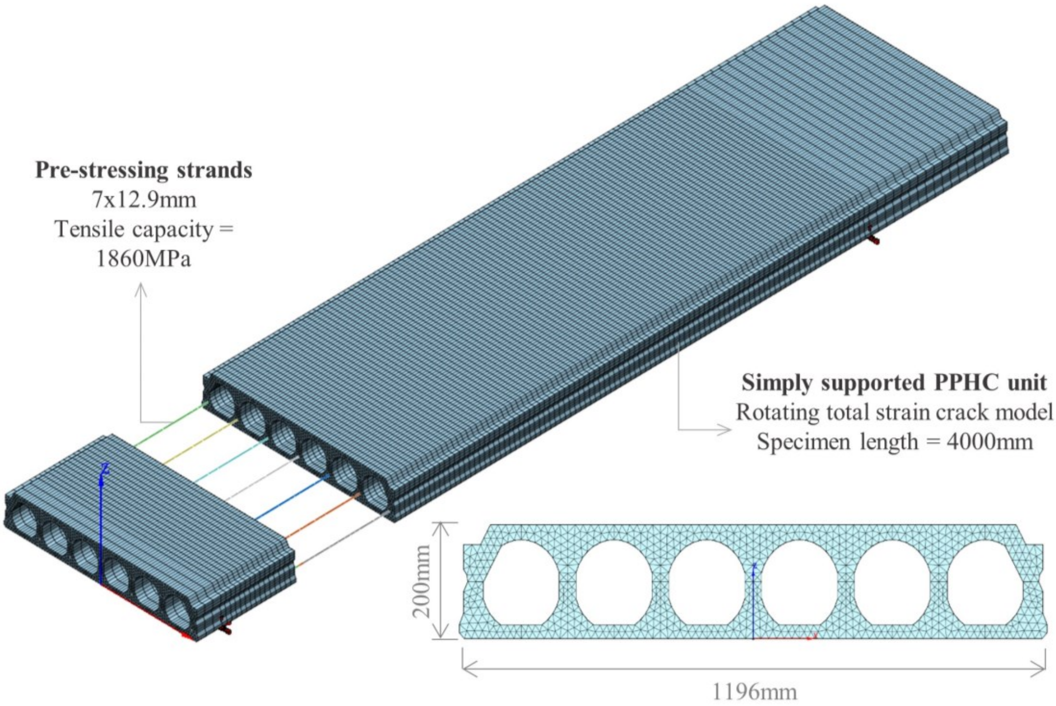


Fig. 4. Detailed solid FE model developed for PPHC units.

The total strain crack model, developed along the lines of the modified compression field theory, originally proposed by Vecchio and Collins [11] and then extended to the three-dimensional case by Selby and Vecchio [12], was adopted to allow the development of a brittle web-shear failure mechanism. A rotating total strain cracking constitutive model was assumed, in which the directions of the cracks are presumed to continuously rotate depending on the changes in the axes of the principal strains. The algorithm for the rotating crack model is relatively simpler than for its counterpart, a fixed crack model, providing a superior convergence due to the fact that, at each loading step, this model is unrelated to the previous cracking conditions [11].

The total strain crack model used is classified under the smeared crack model. Therefore, only one stress-strain relationship for tensile behaviour including cracks and one for the compressive behaviour is considered. In the case of the hollow-core concrete, the constitutive model was assumed to adopt a brittle function and Thorenfeldt et al. model [13] for uniaxial tensile and compressive behaviour, respectively (Fig. 5a,b), whereas for the cast-in-situ concrete it was assumed the model from Hordijk and Reinhardt [14] (Fig. 5c) for tensile behaviour. Confinement effects were neglected, while full shear retention and lateral crack effect [15] were potentially accounted for through the selected rotating strain crack model.

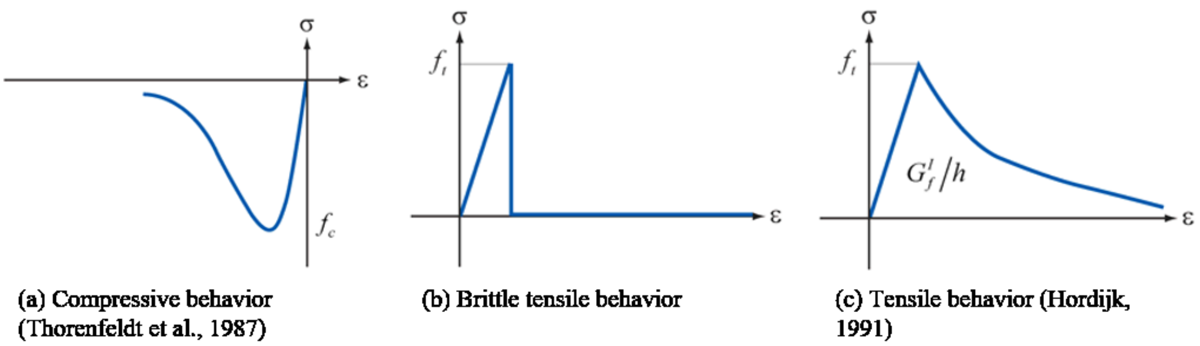


Fig. 5. Constitutive models used to represent the compressive and tensile behaviour of precast concrete (a, b) and cast-in-situ concrete (a, c).



 

The classical Von Mises yielding criterion with strain hardening was used for the pre-stressed steel strands, represented as embedded elements. To represent the strands-concrete interaction an equivalent parabolic pre-stress distribution was selected according to the work presented by Yang [16]. Therefore, no interface elements were introduced to represent strands-concrete interaction, since it is implicitly captured by the equivalent pre-stress distribution. A pre-stress loss ratio of 12% was implemented, in accordance with the values estimated using the New Zealand Concrete Standard NZS3101:2006 [17] and the manufacturer records.

When the tendons are tensioned and released, inward slippage at the wedges takes place thus allowing the tendons to slacken. A slippage of 5mm is considered to happen in the model, provoking tension losses in the tendons in the vicinity of the anchorages.

Fig. 6a shows a general 3-D view of the modelled hollow-core seating connection and the meshing layout. The main components of the sub-system model are the hollow-core slab, the cast-in-situ concrete beam, and topping, all modelled using 6-noded 3-D solid elements. All reinforcement was modelled using 1-D embedded elements (see Fig. 6b). The classical Von Mises yielding criterion was used to model the ductile mesh, starter bars, and the hairpin bars, whereas the beam's reinforcement was assumed to remain elastic.

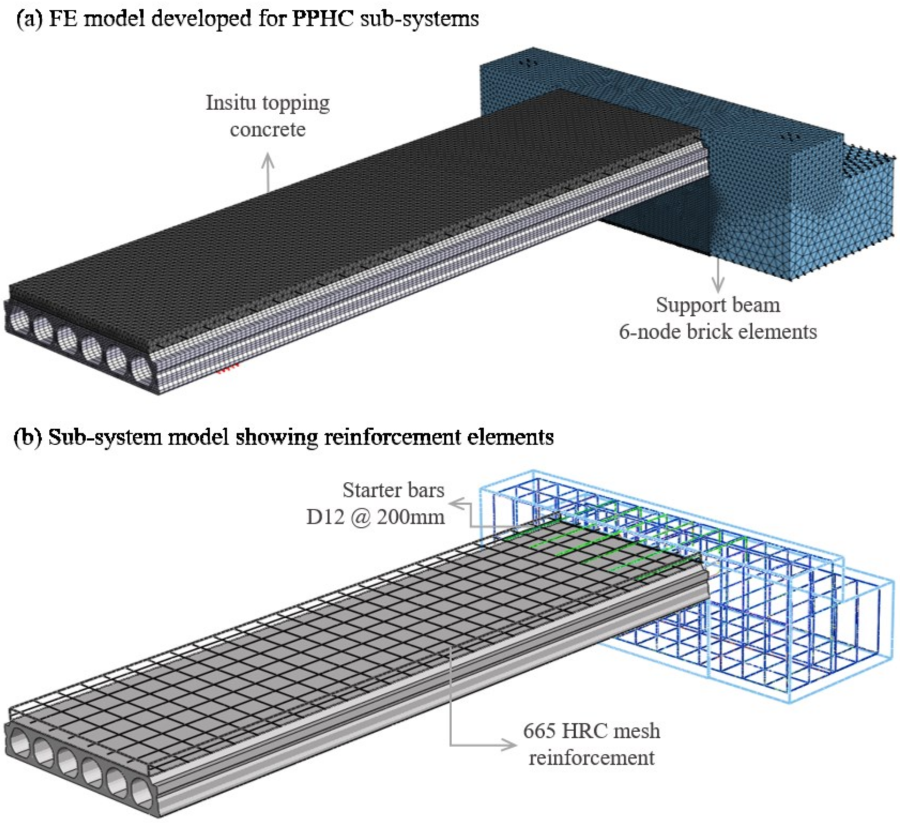


Fig. 6. PPHC seating connection FE model.

In reinforced concrete, the interaction between the reinforcement and the concrete is governed by secondary transverse and longitudinal cracks in the vicinity of the reinforcement. This behaviour has been modelled with a bond-slip mechanism where the relative slip of the reinforcement and the concrete is described by the 1-D interface element. The constitutive laws for the selected bond-slip model are mostly based on total deformation theory, which expresses the traction (resistance against slip) as a function of the total relative displacements. The relationship between the normal traction and the normal relative displacement is assumed to be linear elastic, whereas the relationship between the shear traction and the slip is assumed to follow the polynomial relation proposed by Dorr [18].



 

A composite interface model was used for the interaction between precast and cast-in-situ concrete. This model combines cracking, shearing and crushing, being able to simulate fracture, frictional slip, as well as crushing along the material interfaces. The two-dimensional interface element of zero-thickness is modelled based on multi-surface plasticity, comprising a Coulomb friction model combined with a tension cut-off.

In the FE models developed, all calculations were performed via displacement control, adopting the Newton–Raphson iteration scheme with an energy-normalized convergence criterion. The FE model has been first calibrated according to fracture mechanics and the expected failure mechanisms, and has been further validated by comparing the numerical results with test data providing load-deflection behaviour, ultimate load, and the failure mechanism.

## 4 Preliminary modelling results and discussion

### *4.1 PPHC model and material validation*

The proposed numerical approach is first validated by focusing on a single PPHC unit, and for this purpose, specimens HC1 and HC2 (Table 1) have been taken as a reference. The accuracy of the unit FE model predictions is then tested for the remaining specimens and analysed in comparison with the experimental data. The adopted modelling approach provides a consistent match with experimental test results. Principal tensile and compressive strains, crack pattern and shear stress distribution at failure are in close agreement with the collapse mechanism observed by the experimental damage pattern at the shear span of the unit tested in bending. An inclined crack emerges from both principal tensile strains and numerical crack patterns. Simultaneously, an inclined compressive diagonal strut arises, resulting in the failure mode that finally developed, also confirmed by a cut-off in the shear stress flow. Fig. 7a displays the shear force-displacement relationship from the testing of specimens HC1 and HC2 as well as the results from the FE prediction, together with the flow of shear strains in the shear span of the member under ultimate conditions in Fig. 7b.

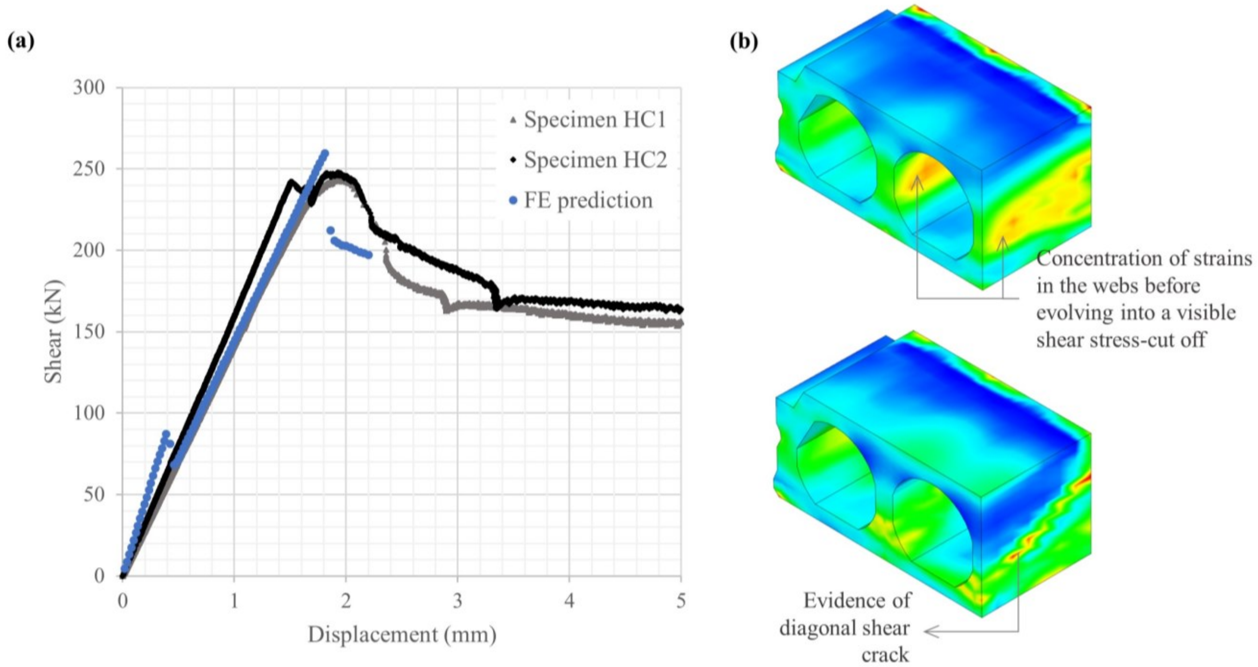


Fig. 7.Comparison of FE predictions and experimental results for PPHC units: (a) Force-displacement relationship (b) change in shear strain distribution at failure.



 

The effectiveness of the FE models developed, accounting both for geometrical and material nonlinearities, is considered satisfactory. An accurate agreement with experimentally observed shear strength and displacement capacities has been achieved. For these models, mean values for the material properties were assumed. A variation of these properties in the actual specimens could have an influence on the difference between numerical and experimental results, especially considering that PPHC units are characterized as having a large variation on their properties. Table 3 summarizes the results from some of the sensitivity studies conducted for the shear strength and displacement capacity of the PPHC units. It can be observed that the model is sensitive to the values of tensile strength of the concrete and elastic modulus of the concrete, whereas, parameters such as the level of pre-stress and compressive strength of the concrete have a small influence on the shear strength and peak displacement of the units.

Table 3. Sensitivity analysis results for shear force and displacement variation.

| Parameters | Difference between min & max values modelled | Resulting variation in predictions | |
|---|---|---|---|
| | | **Shear Force** | **Displacement** |
| **Compressive strength concrete** | 66.67% | 0.15% | 0.05% |
| **Tensile strength concrete** | 51.24% | 50.49% | 50.52% |
| **Elastic modulus** | 85.71% | 27.76% | 92.73% |
| **Applied pre-stress force** | 28.57% | 3.23% | 0.07% |

Fig. 9a and Fig. 9b present a comparison between the experimentally observed and numerically predicted web-shear failure mechanism for specimens HC1 and HC2 respectively. In both the experiment and the numerical model, the presence of flexural cracks was observed in the bottom of the PPHC units. The development of this secondary cracking mechanism was more evident in the specimens with a bigger shear span. Fig. 9c shows the results corresponding to specimen HC6 with a shear span of 700mm, in which the flexural cracks appeared before the formation of the critical diagonal cracking mechanism. The observed flexural cracks formed in the bottom of the unit towards the mid-span, matching the predicted cracking pattern which extended in the same direction.

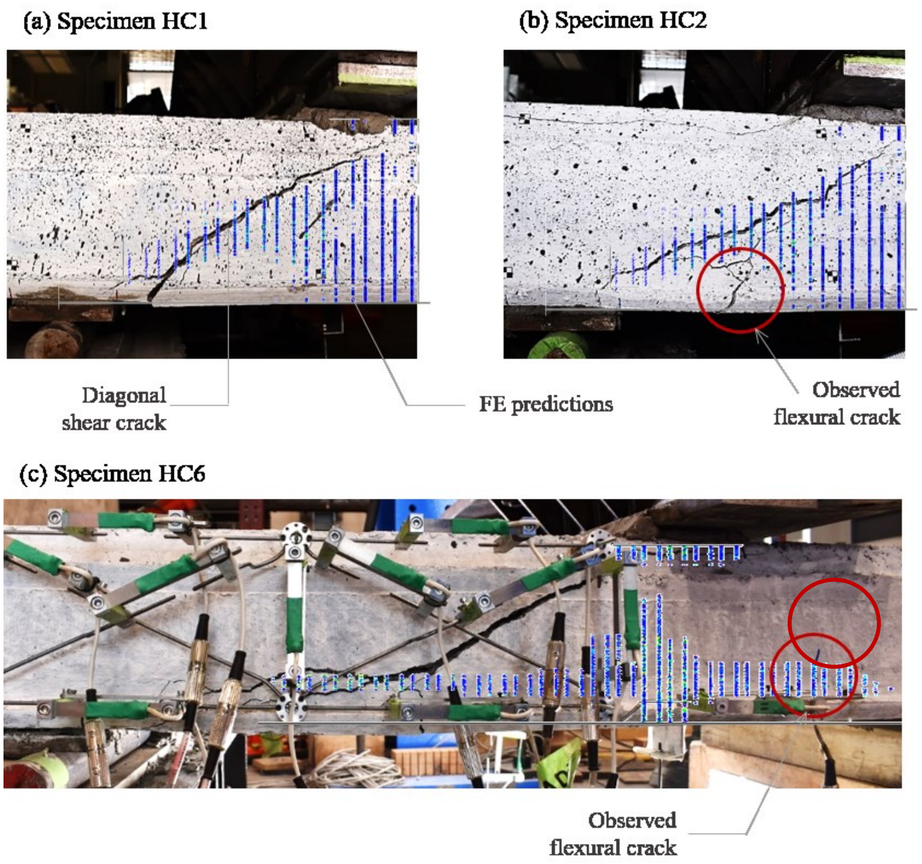


Fig. 8. Comparison between the experimentally observed and numerically predicted cracking mechanism for PPHC units.

### *4.2 PPHC sub-systems model*

For the initial calibration of the subsystem model against experimental results, specimen B1 (Table 2) has been used as a reference. Specimen B1 was a control case with a hairpin reinforcement detail (Fig. 6), no retrofit and with a vertical actuator used to impose rotational demands on the beam to hollow-core seating connection by rotating the hollow-core unit. The specimen did not display NMF; however, the progressive damage observed consisted in cracking at the end connection and cracking at the end of the starter bars in the topping concrete and then extending downwards and throughout the bottom half of the unit, as shown in red on Fig. 9. Fracture mechanics obtained from preliminary FE predictions show a consistent match with the experimental observations.



 

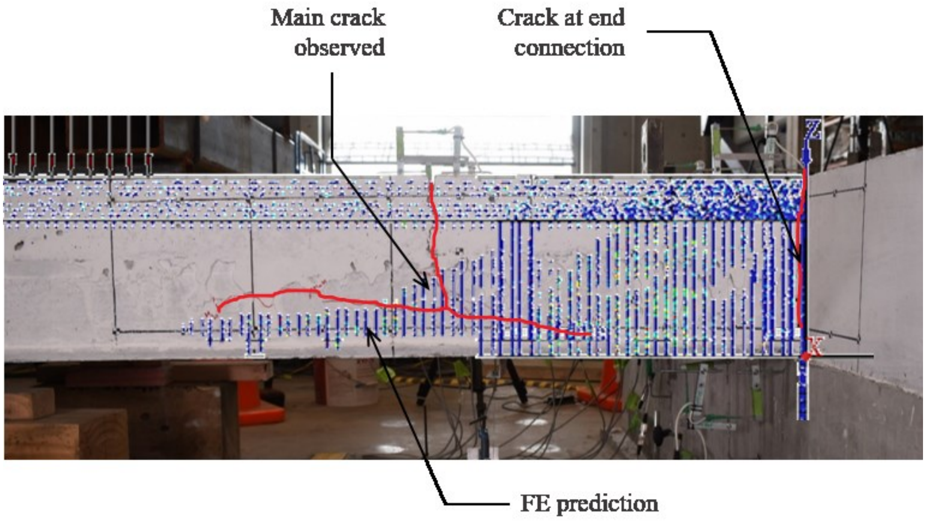


Fig. 9. Comparison between the experimentally observed and numerically predicted crack mechanics for PPHC sub-systems.

## 5 Conclusions

The recent Kaikōura earthquake has raised again the concern about the general lack of understanding of the behaviour of PPHC floor systems and instigated the need for the vulnerability of these floors to be further investigated. For this reason, a numerical investigation, organized into a campaign of nonlinear FE analyses, has been initiated.

A FE modelling approach for PPHC units was proposed with the aim of validating the material properties and behaviour of the New Zealand hollow-core units. This modelling approach, based on nonlinear failure mechanisms, has proved successful in predicting the formation of web-shear mechanisms in PPHC slabs. The outcomes from this first analysis phase provide the basis for the development of more complex FE models for PPHC slab connections and diaphragms.

To identify the drift levels that cause critical failure modes such as LoS and NMF, a second modelling phase is being carried out into the PPHC seating connections. Preliminary analyses have shown consistency between the experimental observations and FE predicted cracking mechanics. Numerical outcomes from these analyses will assist in studying the post-cracking behaviour of floor diaphragms at the system or global level. Furthermore, results from the overall modelling program may be directly applicable to revise and validate methods for assessing and improving the seismic performance of PPHC floors.

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