
Seismic Performance of Fired Clay Rib and Hollow Block Infill Floors

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ABSTRACT

Prior to the mid-1970s, rib and infill floors in New Zealand were predominantly constructed from fired clay tiles with prestressed wires grouted in place and hollow clay tile infill blocks topped with cast-in situ concrete.

Compared to modern precast rib and timber infill systems, fired clay rib and infill systems exhibit several potential vulnerabilities. These include a heightened risk of global or local dislodgement of infill blocks from the floor soffit, uncertainty in load path reliability between the grouted prestressed wires and clay ribs, and potential weaknesses at the clay-concrete interface. Damage or cracking in rib supports during seismic events may compromise gravity load capacity, particularly if there is bond loss in the prestress wires.

While potential vulnerabilities have been identified, there is limited experimental testing and real-world observation of the seismic performance of these floor systems. Due to concerns regarding the robustness of clay rib infill systems, current New Zealand assessment guidelines adopt a conservative approach by prescribing drift limits of 2% to mitigate the risk of positive moment failure and 1% for negative moment failure, where the latter is identified as a governing failure mode.

This experimental program investigates structural performance of fire clay rib-and-infill systems under seismic drift. A specimen, including partial frames, representative of historic standard detailing were tested under cyclic loading to investigate drift capacity, gravity load paths, and positive and negative moment failure. Test setups were designed to replicate support conditions inside and outside beam elongations zones with varied imposed loading.

1 INTRODUCTION

The Stahlton floor system is a prestressed flooring system that was introduced to New Zealand in the 1960s and originally utilised fired clay components. In its early form, the system comprised prestressed ribs constructed by arranging 300 mm by 150 mm fired clay tiles along a prestressing bed, with prestressing wires placed within grooves formed in the tiles. These wires were tensioned and subsequently grouted into position. Once the grout had achieved sufficient strength to maintain the prestress, the wires were released and the ribs were cut to the required lengths [1].

The completed prestressed ribs were then installed on temporary supports, spanning between beams or load-bearing walls. Hollow fired clay infill blocks were positioned between the ribs to achieve the desired floor depth, depending on span and loading requirements. A cast in situ concrete topping was then applied to complete the floor system.

In comparison with contemporary precast rib systems and timber infill floor construction, fired clay rib and infill systems present a number of inherent vulnerabilities that warrant careful consideration. One of the primary concerns is the increased susceptibility of the hollow infill units to local or global dislodgement from the soffit, particularly under dynamic loading conditions. The brittle nature of fired clay, combined with limited mechanical interlock, can result in reduced robustness relative to modern systems.

Additional uncertainty arises in relation to the reliability of the load transfer mechanism within the prestressed ribs. Specifically, the interaction between the grouted prestressing wires and the surrounding clay units is not always well defined, raising questions about the consistency of force transfer and long-term bond performance. The interface between the fired clay components and the cast in situ concrete topping also represents a potential plane of weakness, where differential material properties and limited composite action may lead to debonding or slip.

These vulnerabilities are of particular concern under seismic loading. Cracking or damage at the supports of the prestressed ribs can significantly reduce their ability to sustain gravity loads, especially in cases where bond degradation or loss of anchorage occurs in the prestressing wires. Such mechanisms may lead to a reduction in overall system redundancy and an increased likelihood of partial or progressive collapse.

Despite the identification of several potential vulnerabilities, there remains a notable lack of experimental data and documented field observations regarding the seismic performance of fired clay rib and infill floor systems. The limited availability of testing results and post-earthquake assessments introduces significant uncertainty in evaluating their behaviour under seismic actions, particularly with respect to failure mechanisms, deformation capacity, and residual load-carrying performance.

In the absence of a comprehensive evidence base, current New Zealand assessment guidelines [2] adopt a conservative approach by recommending the application of a reduction factor of 1.5, similar to hollow-core units, when estimating the percentage of New Building Standard (%NBS) for this type of floor system.

This paper presents an experimental study undertaken to investigate the behaviour and evaluate the drift capacity of a fired clay rib and hollow block infill floor system.

2 EXPERIMENTAL PROGRAM AND TEST SETUP

2.1 Prototype and Specimen Configurations

The specimen configuration for this study was based on an existing building in Wellington constructed in 1969. In the absence of shop drawings, and to develop a clear understanding of the detailing of the Stahlton floor system used in this building, a destructive investigation was undertaken. This involved cutting and removing one of the ribs, followed by hydro demolition to expose the reinforcement details, as shown in Figure 1. As shown in Figure 1b, the hollow block infill is oriented perpendicular to the ribs and is therefore partially filled with concrete during construction. This may increase the seismic mass of the structure but can also provide a degree of mechanical interlock between the hollow blocks and the concrete topping.



a) Extracted prestressed rib



b) Close-up view of the rib section



c) Close-up view of the hollow-block infill section

Figure 1: Extracted rib from an existing building in Wellington

For the first and last 20% of the rib length, 4.7 mm stirrups are placed in the grooves at 200 mm spacing, followed by 800 mm spacing, as is shown in Figure 2.

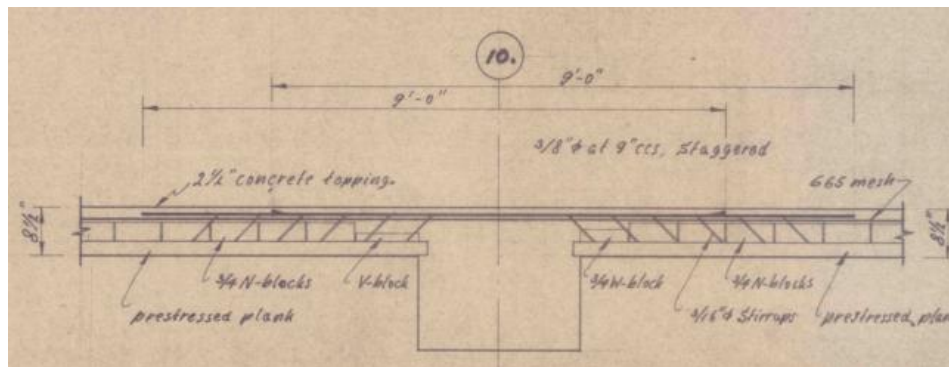


Figure 2: Typical longitudinal section of the ribs

The test setup consists of a central concrete frame with cantilevered concrete beams supporting two precast concrete ribs either side of the central parallel beam, referred to as the north bay and south bays. This allows the test to capture elongation attributed to the support locations and rotation of the support beam. Figure 3 to Figure 5 on the following pages illustrate the general test setup.

The key parameters for this testing program are the drift ratio and the elongation of the concrete frame. Considering the limitations of the structural lab, a 5100mm span length was selected for the specimens. A column height of 1500mm was selected to represent the point of inflection for a typical storey height. It is important to note the beam length and column height are not critical to the investigation.

Based on the span length of the specimen, an approximate scale of 2/3 was adopted to determine the dimensions of the beams and columns. The column dimensions are 450x450, the dimensions of the support beam are 500(H)x400(W), and the dimensions of the parallel beam are 500(H)x350(W). All dimensions are in millimetres. The test setup was included a horizontal actuator with a 165mm stroke which is able to produce more than a 5% drift ratio. It is worth noting that modern design codes limit the maximum plastic rotation demand for beams to 0.025 radians for a Life Safety/ULS performance criterion.

The floor system consists of 160 mm deep ribs with 100mm hollow block infill and a 60 mm concrete topping. The rib detailing is consistent with that used in the building.

The size of the mesh, the reinforcing starter bars, and the overlap between the mesh and starter bars are key parameters considered in this test program. Since a shorter starter bar length increases the risk of negative moment failure, the starter bar extension into the slab is designed based on the building's perimeter beams and set to 600 mm.

The topping mesh consisted of 665 mesh and extended over the top of the supporting beam due to the higher demand from negative bending moments, which was confirmed in the building under study.

Figure 6 shows the construction of the ribs and the specimens before carrying out the test.

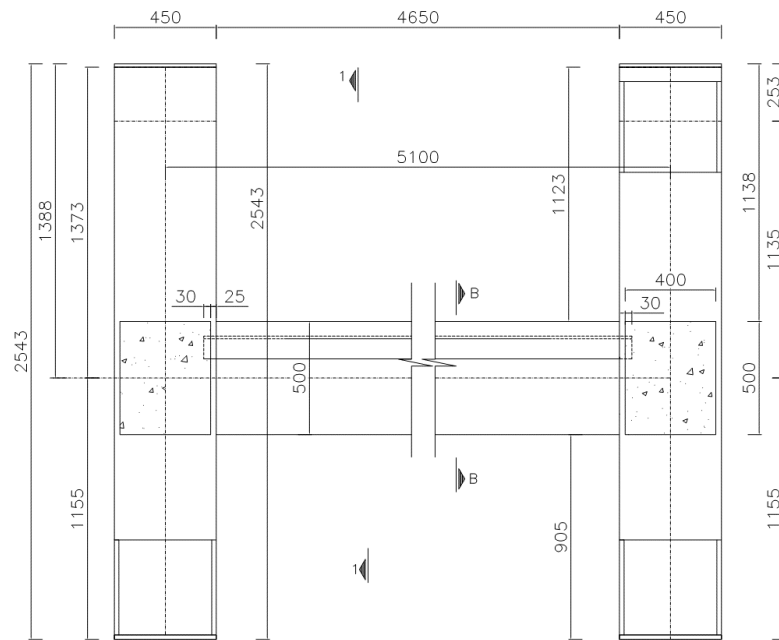


Figure 3: Side elevation of test setup (floor system parallel to page)

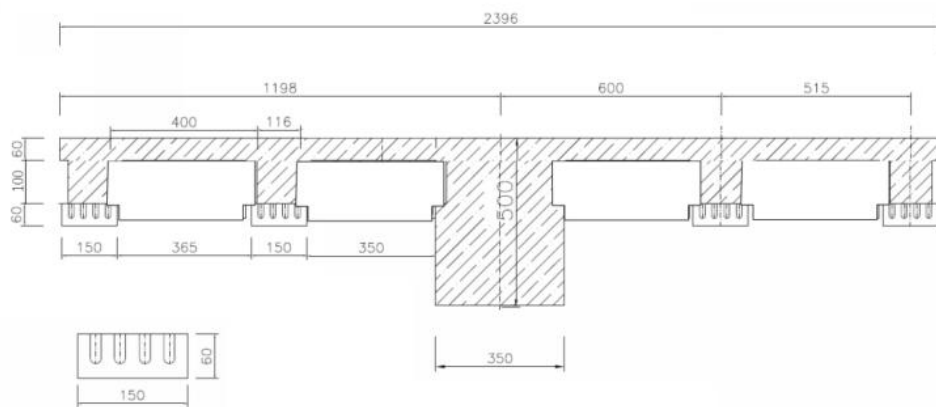


Figure 4: Floor Elevation

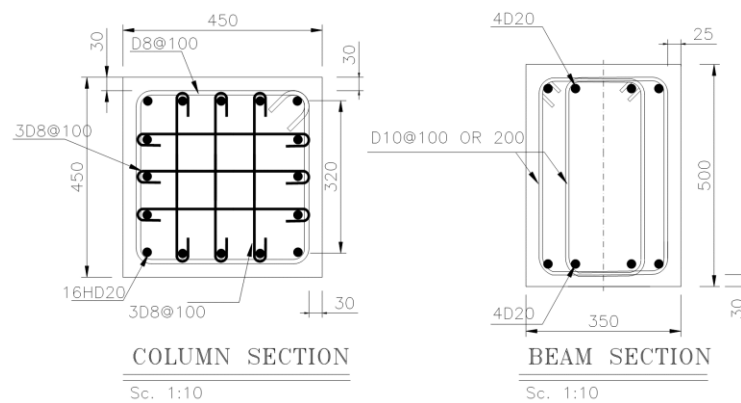


Figure 5: Typical beam and column detailing.



a) Ribs Following Prestressing of Wires



b) Ribs During Groove Grouting



c) Specimen Prior to Concrete Pouring



d) Specimen During Test Setup Installation

Figure 6: Specimen construction

2.2 Test setup and loading protocol

Figure 7 illustrates the schematic test setup and loading system.

In this setup, the columns are hinged (or moveable) at the base to simulate inflection points that would be in an actual frame subject to lateral loading. The specimen was loaded with a horizontal hydraulic jack with a capacity of 2000kN along the E-W direction (parallel to the unit), with a stroke of $\pm 165\text{mm}$. The loading was controlled by displacement. No axial load has been considered as axial load on the frame is not expected to impact the lateral response of the specimen.

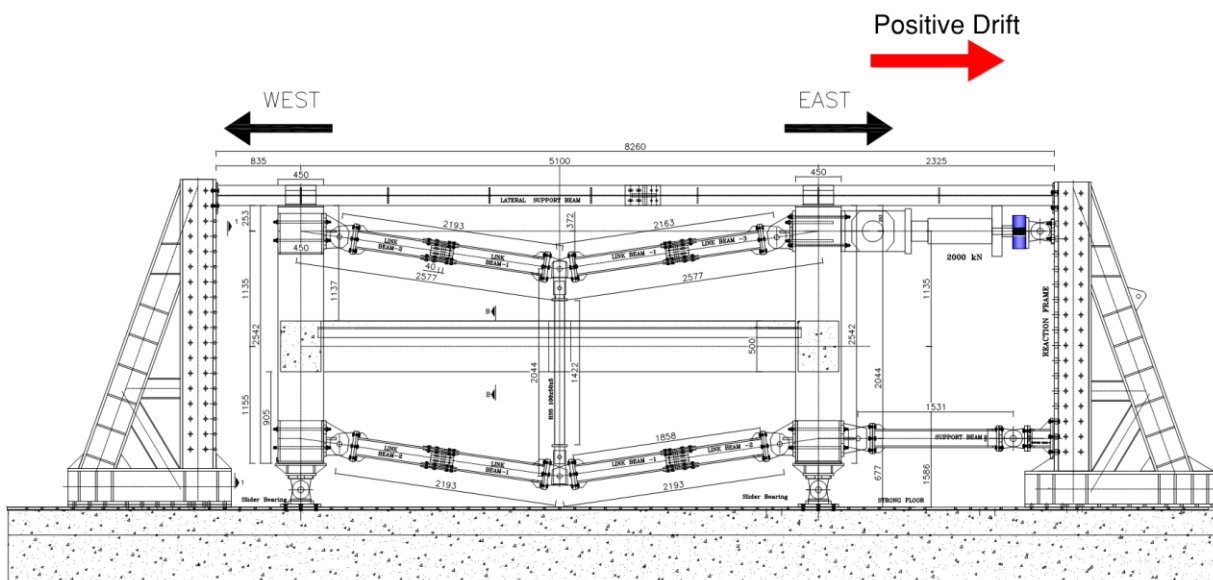


Figure 7: Test setup parallel to floor system.

The loading protocol has been designed based on the recommendations in ACI 374.2R13 [3]. A reverse cyclic lateral loading was applied to the specimens using the horizontal actuator. Figure 8 shows the loading protocol based on ACI 374.2R13. In this loading protocol, each step is two cycles up to 1% drift and 3 cycles above 1%.

Previous precast floor testing has found that a pulse type motion is critical for the governing failure modes of precast floors [4]. In order to account for this pulse type motion, the loading protocol started with a single cycle at 1% drift before continuing with the ACI 374.2R13 protocol, ignoring the initial 1% pulse. The test continued until failure occurred in either the main beam or the ribs.

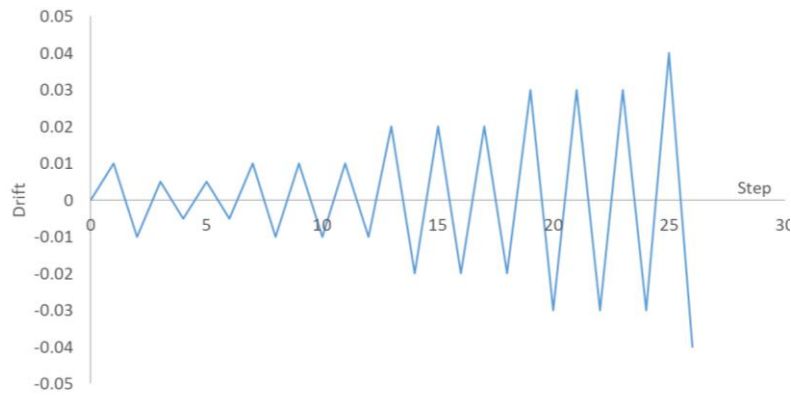


Figure 8: Loading protocol- Positive drift refers to the ram extension toward the east direction.

3 EXPERIMENTAL OBSERVATION

Flexural cracking of the beam was first observed at a drift ratio of -0.5% on the bottom face at the east side. A similar pattern of flexural cracking developed at a drift ratio of $+0.5\%$ on the west side, affecting both the bottom face and the web of the beam. At a $+1\%$ drift ratio, two distinct hairline cracks were observed on the top surface of the slab on the east side: one at the end of the starter bar and the other between the end of the ribs and the transverse beams. At the same drift level, hairline cracks also appeared on the top surface of the slab and at the beam–slab interface on the west side.

At a 2% drift ratio (both positive and negative), the cracks formed at the previous drift level (1%) widened. The crack width at the end of the starter bars on the east side reached approximately 2 mm , while the crack at the beam–slab interface on the west side widened to approximately 3.0 mm . Other cracks exhibited widths of less than 1 mm . At this stage, spalling of concrete at the rib supports initiated, and cracks also formed at the soffit of the hollow block infill tiles.

At a 3% drift ratio (both positive and negative), the damage progression observed at 2% drift continued. The main cracks on the topping surface widened to approximately $3\text{--}5\text{ mm}$, and rib pull-out of approximately $2\text{--}4\text{ mm}$ was recorded on both the east and west sides.

At a 3.5% drift ratio, failure of the slab mesh at the west end and at the ends of the starter bars occurred, accompanied by the development of negative bending in the ribs. As a result, separation of the clay plank in the ribs was observed, exposing the prestressing wires.

At a 4% drift ratio, the crack at the beam–slab interface on the east side widened to approximately $5\text{--}6\text{ mm}$, and a vertical offset of about 5 mm was observed in the topping. The test was terminated at this drift level due to safety concerns.



a) Rib Pull-Out Observed at 3% Drift



b) Cracking at the Bottom of the Infill Block and the Corner of the Rib Plank



c) Mesh Failure at 3.5% Drift (West Side)



d) Rib Negative Moment Failure and Prestressed Plank Separation at 3.5% Drift (West side)



e) Negative Moment Failure at Rib Ends (East Side)



f) Beam–Slab Interface Crack at 4% Drift (East Side)

Figure 9: Crack Pattern Observed at 3% to 4% Drift

4 REFERENCES

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