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Preliminary results from the testing of a precast hollowcore floor slab building

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ABSTRACT: Concern has been raised about the expected performance of many of New Zealand's recently and presently constructed precast concrete buildings during a severe earthquake. The intent of this paper is to give some insight into the results from a recently completed experimental programme undertaken at the University of Canterbury. One particular concern is the detailing used to attach the precast hollowcore floor units to the perimeter frame. Results show that if a large earthquake were to hit New Zealand, precast buildings would not perform well compared to cast-insitu systems—collapse of floors may be expected.

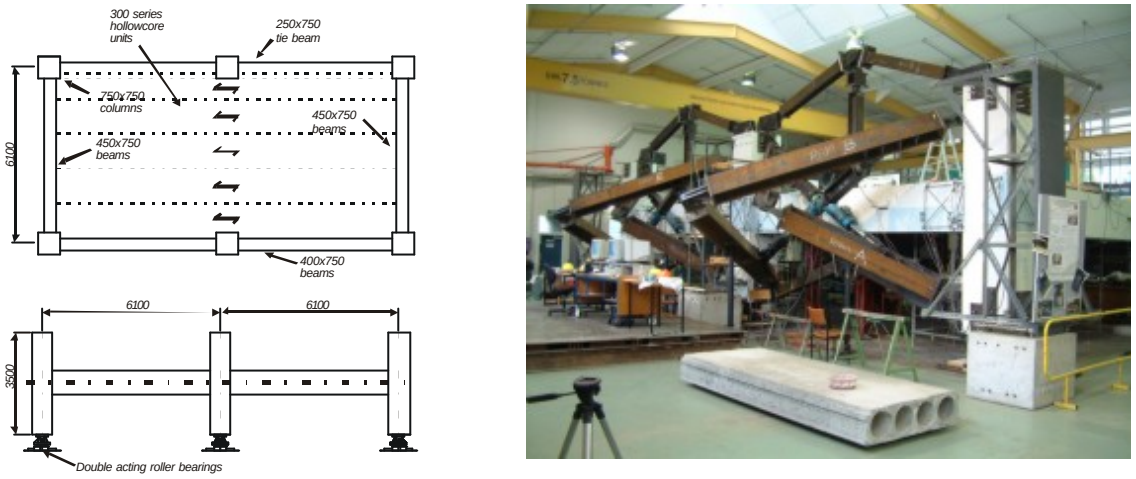
1 SPECIMEN DETAILS

This test specimen represents a corner section from lower storey in a typical precast concrete building. The specimen consists of 750mm deep beams with a bay length of 6.1m. The overall dimensions are approximately 12m long by 6m wide. The precast floor units being tested are 300mm series hollowcore units. These units are topped with a 75mm cast insitu topping. The hollowcore units span past the central column and are seated on the two end beams with a nominal seat width of 50mm. This detail is commonly used in New Zealand to make the design more economical. The actual seat length is 20mm on the east beam and 40mm on the west beam. These provided seats are considered to be representative of the range of seat width adopted in the field over the past two decades. Figure 1(a) shows the super-assemblage dimensions.

The load/ displacement is applied to the super-assembly through the columns in the form of a horizontal shear force to the top and bottom of the column. The loading frame set up is shown in Figure 1(b) for both the longitudinal and transverse loading directions. The two main loading frames are the diagonal frames. A set of secondary loading frames (that resemble an arrow shape) is provided to enforce displacement compatibility of the adjoining stories. The secondary frames ensure the drift angle on each column is the same.

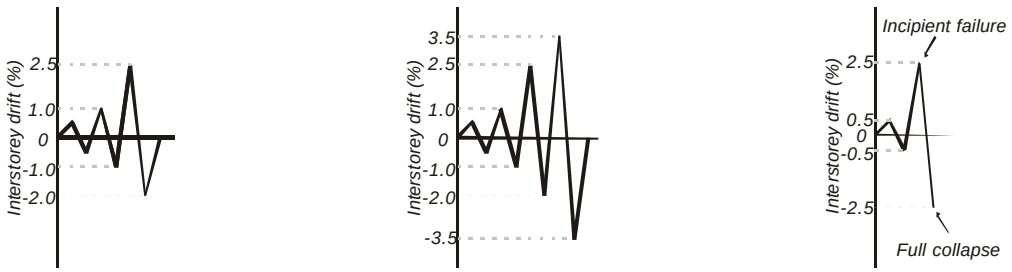
2 LOAD PATTERN APPLIED

Three different displacement histories were applied to the super-assemblage. Each of the three histories corresponds to the three different loading phases. These histories are shown in Figure 2. The background to the determination of these displacement histories is explained in companion paper Matthews et al (2003).



(a) Plan and elevation (b) Super assemblage load frame set up

Figure 1. Super assemblage dimensions



(a) Phase I-Longitudinal loading (b) Phase II-Transverse loading (c) Phase III-Longitudinal loading

Figure 2. Loading histories applied to the super-assemblage.

3 EXPERIMENTAL RESULTS

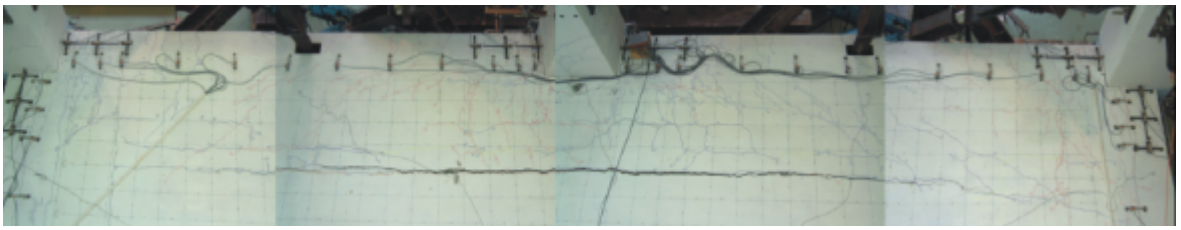
3.1 Phase I: Longitudinal Loading

The results of loading the super assemblage are summarised in Figure 3.

As testing progressed the seating detail used to attach the hollowcore floor units to the supporting beams started to crack and show signs of distress from an early stage. The first sign of damage occurred at an interstorey drift of 0.35%, and at a drift of 0.5% (μ=1) this level of damage would be sufficient to cause some economic loss to the building.

Overall the specimen behaved well up to interstorey drifts of ±1.0% (μ=2). However significant cracking in the topping slab developed. As the drift increased this led to a tear forming within the floor slab at a drift of 1.9%. This tear was due to the elongation within the beam causing the central column to translate outwards and taking the first hollowcore unit with it. The reinforcing mesh in the topping slab between the first and second hollowcore units fractured. The tear can be seen in Figure 3(a).

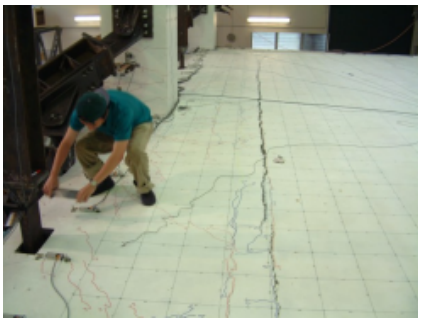
At the completion of the –2.0% cycle, the entire seating for the hollowcore units were damaged (Figure 3(b)), with some of the units dropping 10mm. There was also significant web splitting within the first hollowcore unit. It is considered that when compared to real dynamic earthquake loads, the test was possibly unconservative. This is because if any live load or vertical accelerations had been concurrently applied to the building it would be questionionable as to whether the floor would remain standing. At the completion of the –2.0% drift cycle the central column displaced transverse to the direction of loading by 25mm. This also caused the first floor unit to rise 12mm relative to the rest of the floor. The extent of the crack propagation is shown in Figure 3(c) and (d). The translation was not only due to the elongation of the main beam. The newly formed inverted L shaped beam (beam plus the adjacent floor acting as a flange) contributed to some of this displacement as it tried to bend about it sloping principal axes. Since the central column is no longer tied to the floor slab it is possible for the column to fail under buckling. The reason for this is that the floor tear could occur over several floors of the building greatly increasing the columns effective length and hence reducing its load carrying capacity.



(a) Longitudinal tear that formed at 1.9% drift.

(b) Entire seat has been lost at the completion of the Phase I (-2.0% drift)

(c) Vertical offset after the tear formed (-2.0% drift)



(d) Longitudinal view of floor slab tear at the end of Phase I (–2.0% drift)

Figure 3. Observed damage during Phase I testing

3.2 **Phase II: Transverse (Short direction) Loading.**

Since the major crack had formed within the floor diaphragm it changed the expected performance of the super assemblage during the transverse (out of plane) loading. Initially the expected performance of the diaphragm was for the perimeter beam to rotate relative to the floor units. This was not the case, the first hollowcore unit actually lifted as the beam rotated.

Since the side of the first hollowcore was adequately bonded to the beam, it caused the crack within the soffit of the first hollowcore unit to open some more. This meant that the condition of the hollowcore unit degraded as the transverse loading preceded. The first sizeable piece of concrete fell out around 2.0% drift. This is shown in Figure 4(a). Once the piece of concrete had fallen out of the hollowcore unit it was possible to look at the internal damage within the first hollowcore unit adjacent to the perimeter beam. Extensive damage could be seen to have occurred. Figure 4(b) and (c) shows the extent of the damage. The width of web crack within the hollowcore unit was approximately 25mm. At this stage, the webs of the first hollowcore unit had split halfway along the unit (approximately 6m). For some time now one small triangular shaped piece of concrete was holding the first floor unit up. This small section of concrete could not be relied upon to hold every time. This can be seen in Figure 4(e) at the top of the picture.

It should be noted that the hollowcore unit dropped some 30mm at this stage as shown in Figure 4(d). Figure 4(e) shows the first floor unit after a large piece of floor fell out. At this stage, the floor had dropped by some 60mm.

(a) Hollowcore damage

(b) Internal damage looking towards the seat

(c) Internal damage looking away from the seat

(d) Damage to the underside of the west hollowcore.

(e) Damage at 3.0% drift.

Figure 4. Observed damage during Phase II testing

3.3 **Phase III: Final Longitudinal Loading**

Eventually there was sufficient damage within the first hollowcore unit to allow the entire bottom section to drop as shown in Figure 6 (a) and (b). This failure occurred at a interstorey drift of 2.0%. These photos look very similar to those taken following the 1994 Northridge earthquake (Norton et al 1994).

Upon further loading, to the –2.5% drift amplitude, the remainder of the floor failed when the design live load was applied. Again, the photos of this failure (Figure 6(c)-(f)) were very similar to that seen in Northridge.

One major point to note is that even though the floor failed, the perimeter frames beams, columns, and beam column joints remained relatively undamaged. Clearly, significantly extra attention is required to be paid to the hollowcore seating details to ensure that class of precast floor system performs at a level that is not inferior to than that of the structural frame.

4 DISCUSSION

4.1 Seating detail performance.

One major difference between the expected seating performance and the observed performance was the way in which the floor unit moved relative to the beam it was seated. In design it is customary to assume that hollowcore units would slide relative to the beam; this was not the case in the experiment. There was enough bond/friction to cause the end of the unit to fracture rather than slide, as shown in Figure 7.

(a) Failure of the first hollowcore unit

(b) Close up looking at the seat damage

(c) The frame after the floor collapsed

(d) Collapsed floor units

Figure 5. Observed damage during Phase III testing.

(e) The remaining topping and ends of the hollowcore units after collapse.

(f) The delaminated topping on the failed unit.

Figure 6. cont. Observed damage during Phase III testing.

A Technical Advisory Group (TAG) has been formed to discuss these experimental results from the testing programme at the University of Canterbury. TAG recommended a new connection detail that is expected to perform better than the details currently used. The new detail consists of replacing the dam plug in the end of the unit and placing some compressible material approximately 10mm thick across the end of the unit. The unit will also be placed on a bond breaker, in the form of a low friction (PTFE or equivalent) bearing strip. A sketch of the proposed detail is shown Figure 8 along with the expected improved (damage-free) performance.

Attaching a low friction bearing strip allows the floor unit to slide as previously assumed. The compressible material is added to reduce the compression force applied to the bottom of the unit. The combination of the two additions allows the beam to rotate relative to the floor unit without fracturing the end of the floor unit allowing the connection detail to work as assumed.

The thickness of compressible material required is determined by multiplying the thickness of the topping and hollowcore by the maximum expected interstorey drift of the structure. For example, a 300 series hollowcore unit with a 75mm topping requires the compressible material to be at least 13mm thick if the maximum expected interstorey drift is 3.5%.

The initial results from the testing of a sub assemblage using this modified connection detail look promising. Unfortunately at the time of writing these results are not available for any further discussion.

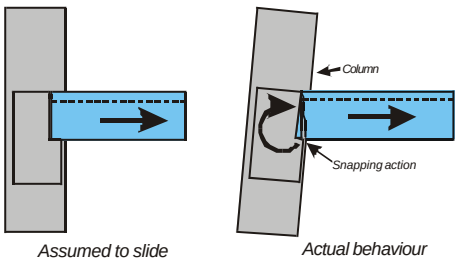


Figure 7. Assumed versus actual hollowcore to beam performance.

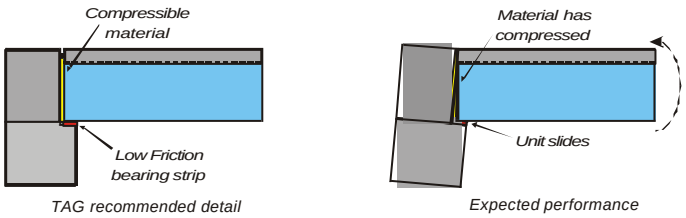


Figure 8. Recommended detail and assumed performance.

4.2 **The performance of the first hollowcore unit adjacent to the frame.**

A hollowcore floor unit is designed to act as a simply supported one-way floor system. The first unit placed adjacent to the perimeter frame does not act in this manner as it is adequately tied not only at its ends but also along its entire length. This leads to the unit being displaced in a quasi-two way manner as the hollowcore unit is forced to undergo the displaced shape of the perimeter beam. This displacement incompatibility between the double curvature of the perimeter beam and the simply supported hollowcore unit causes the hollowcore unit to fail (Figure 8). Since the hollowcore unit has no redundancy in its design the unit fails through web splitting and the bottom half of the hollowcore unit drops.

If the unit was not tied along its length, in other words the hollowcore unit was not forced to undergo the displaced shape of the perimeter beam, then the unit would most probably perform better. If the unit is then detached from the perimeter beam then there are problems with the transfer of the inertia forces from the diaphragm to the perimeter moment resisting frame. This area requires a lot of additional attention.

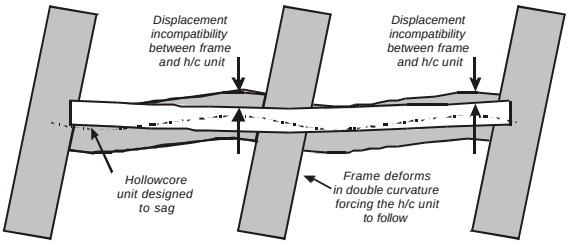
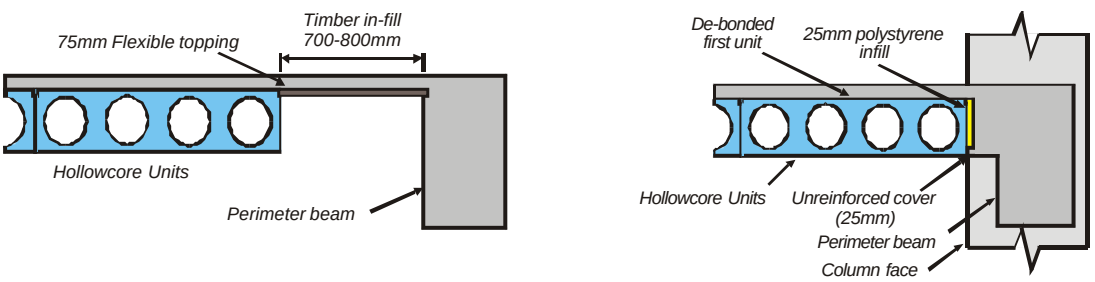


Figure 9. Displacement compatibility between the frame and the hollowcore floor units.

Changing the way in which the first hollowcore unit is connected to the adjacent perimeter beam should allow the unit to perform in the manner in which it was intended by design—that is, a one-way slab. Two different connection details are proposed. The first involves a timber infill that allows a more flexible interface (Figure 10(a)). Damage is expected within this infilled section leaving the first hollowcore unit undamaged. The second is a detail that would need to be used if the bottom of the hollowcore unit is left exposed for an architectural finish (Figure 10(b)). This requires the unit to be butted against the perimeter beam. Placing some polystyrene against the edge of the first hollowcore unit and applying a bond release agent to the top of the first hollowcore unit, so that the topping does not bond to the unit, should allow the hollowcore unit to be de-bonded from the perimeter beam and permit the differential movement between the beam and slab as depicted in Figure 9.



(a) Timber infill solution (b) Architectural finish solution

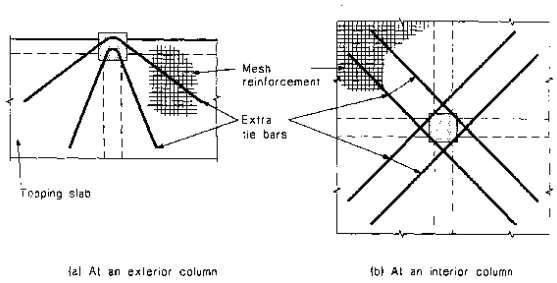
Figure 10 Two fixes that should allow the first hollowcore unit to be separated from the perimeter beam.

4.3 **Extra diaphragm tie reinforcement.**

During the experiment a longitudinal tear formed within the floor diaphragm due to the overloading of the diaphragm reinforcement as floor-frame set up displaced. This tear within the floor now affects the column effective length. If such a tear occurred over several floors in a real multi-storey frame then the column may become unstable.

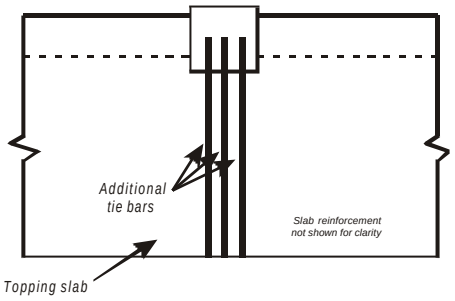
Another scenario that it not usually considered is that columns at lower levels within buildings need to be adequately tied to the floor diaphragm. These columns need to be tied because as a building displaces in an earthquake all the bottom columns must hinge at ground level. This means that several of the edge columns are being dragged across by floor diaphragm. If the provided tie force is insufficient then the diaphragm will tear due to this displacement incompatibility.

The New Zealand Concrete Standard, NZS3101:1995, (Standards New Zealand 1995) states, "additional tie reinforcement must used to tie the column to the floors at each flooring level. The magnitude of the tie force is equal to the larger of 5% of the maximum total axial compression load on the column or 20% of the column shear force induced by the lateral design forces." The draft joint Australian and New Zealand Structural Design Actions Standard (Standards New Zealand 2003) requires that "all parts of the structure shall be interconnected. Connections shall be capable of transmitting 5% of the value of ($G+Y_cQ$) for the connection under consideration."



As recommended by NZS3101:1995

(Standards New Zealand 1995)



(b) Preferred method

Figure 11. Recommended tie details.

As specified by the NZS3101 the bars should be placed at angles close to 45^0. This does help tie the column in but also contributes to the perimeter beams overstrength actions. The bars would be better placed transverse to the perimeter beam. These two comparisons are shown in Figure 10.

5 PERFORMANCE IMPLICATIONS

Buildings in New Zealand are commonly designed for displacements of up to ±2.0% interstorey drift. This is for the so-called 10% in 50 year earthquake (500 year return period). However, there is a worldwide trend to use the 2% in 50 years as the principal design event (≈2500 year return period). For New Zealand seismicity, this would lead to interstorey drifts in excess of 3.5% for the present building stock.

The experiment conducted as part of this research has demonstrated that for present design basis earthquakes considerable damage to precast flooring systems may be expected and lead to irreparable damage. However, should a larger event occur, such as a maximum credible-like event (2% in 50 years) complete collapse of the precast floor is possible. This violates the life-safety intent of the present design codes.

It is concluded that further work is required on three fronts:

(1) For existing structures retrofit measures need to be explored to enhance floor seating and strength. Provision of limiting interstorey drift may also be considered such as the use of structural walls and/or damping devices.

(2) For structures to be designed in accordance with the present design codes, design drifts should be limited to about 1.2% to ensure life-safety can be maintained. This may have severe economic implications because in order to limit the drifts heavier and stronger structures will result.

(3) For future structures, considerable work needs to be undertaken if precast flooring systems are to remain a viable design option. Particular attention needs to be paid to 3D effects and the seating details. In summary a new Damage Avoidance Design (DAD) philosophy needs to be developed for seismic resistant building structures.

6 **ACKNOWLEDGEMENTS**

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