

EXPERIMENTS ON THE SEISMIC PERFORMANCE OF HOLLOW-CORE FLOOR SYSTEMS IN PRECAST CONCRETE BUILDINGS

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SUMMARY

Recent earthquake engineering research undertaken at the University of Canterbury has aimed at determining whether New Zealand designed and built precast concrete structures, which incorporate precast concrete hollow-core floor slabs, possess inadequate seating support details. A full scale precast concrete super-assemblage was constructed in the laboratory and tested in two stages. The first stage investigated existing construction and demonstrated major shortcomings in construction practice that would lead to very poor seismic performance. This paper presents the results from the second stage that investigates the efficiency of improved construction details on seismic performance. The improved details consist of a simple (pinnedtype) connection system that uses a low friction bearing strip and compressible material for the supporting beams together with a 750mm wide timber infill between the perimeter beams and the first precast floor unit. Test results show a marked increase in performance between the new connection detail and the existing standard construction details, with relatively small amounts of damage to both the frame and flooring system at high lateral drift levels. The results show that interstorey drifts in excess of 3.0% can be sustained without loss of support of the floor units with the improved detailing The overall performance of the super-assembly is determined in terms of the hysteretic performance and the fragility implications in terms of the drift damage are classified. Recommendations for future design and construction are made based on the performance of the super-assemblage test specimen and a probabilistic fragility analysis.

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INTRODUCTION

This research has followed on from recent work completed at the University of Canterbury's Department of Civil Engineering by Matthews [1]. Overall, the performance of the precast, prestressed concrete floor system in the Matthews (Stage 1) test was poor while the perimeter moment resisting frame behaved well. The testing completed by Matthews showed premature failure of the flooring system can be expected for design basis earthquakes in New Zealand, due to inadequate seating details and displacement incompatibilities between the frame and floor. Outlined in Matthews' thesis are several areas highlighted for future research that have been addressed in the second stage of the testing programme, including:

- Improving the seating connection detail between the precast, prestressed hollow-core floor diaphragm and the perimeter reinforced concrete moment resisting frame,
- Stopping the central column from displacing laterally out of the building due to an insufficient lateral tie into the building. It was because of this lack of interconnection that the floor slab tore longitudinally due to displacement incompatibility in the Matthews test. The central column was therefore no longer restrained and was able to translate freely outwards, and
- Isolating the first hollow-core unit spanning parallel with the perimeter beams from the frame due to displacement incompatibility. This displacement incompatibility was caused by the units being forced to displace in a double curvature manner due to being effectively connected to the edge of the perimeter beam, when hollow-core units are not designed for such displacement profiles.

SUPER-ASSEMBLY REPAIR

The entire experiment set-up was based on the Matthews [1] testing rig with connection modifications to improve the performance of the hollow-core units. The building was a two-bay by one-bay section of a lower storey in a multi-storey precast concrete moment resisting frame. The floor units were pretensioned precast hollow-core and were orientated so that they ran parallel with the long edge of the building, past the central column. The buildings origin along with the layout and dimensions are shown in Figure 1.



Figure 1. Origin, Layout and dimensions of the Stage 2 (Lindsay) super-assembly.

Following the Matthews testing the super-assemblage's frame was cracked but relatively undamaged compared with the collapse of the flooring system. It was decided to repair and re-use the existing structure; the remaining floor sections were removed, the concrete in the transverse beams was removed and the damaged plastic hinge zones in the southern longitudinal beam were removed and reconstructed. As the existing reinforcing bars were being re-used, the bars were heat treated to restore ductility and reduce internal stresses. This was done by heating the reinforcing steel to a temperature around the critical transition point (~750-850°C) and allowing it to cool (Oberg et al. [2]).

NEW CONNCTION DETAILS

Seating Connection Details

The seating connection between the hollow-core unit and the supporting beam consisted of replacing the plastic dam plug in the ends of the unit with 10mm of compressible material fully across the end of the unit and seating the unit on a low friction bearing strip. This detail is shown in Figure 2 along with how the floor unit is expected to rotate relative to the beam. The low friction bearing strip allows the floor unit to slide as designers had previously assumed it would. The compressible material is assumed to reduce the compression forces applied at the bottom of the unit under negative moments as well as restricting concrete from entering the cores of the units. If the large compression force forms between the bottom of the unit and the face of the supporting beam it is transferred at a relatively flat angle to the topping concrete. A perpendicular principle tensile force then forms, causing splitting of the webs at very early stages of the test.

When the unit is seated on a low friction bearing strip, the seat length becomes very important; it must be placed back from the face of the beam so that as the unit tries to rotate it does not dig into the bearing strip. The draft 2003 amendment to the New Zealand Concrete Structures Standard NZS3101:1995 (Standards New Zealand [3]) has amended the required seat length of hollow-core floors to span/180 or 75mm based on Matthews' recommendations. A seat length of 75mm was used in this test.



Figure 2. Seating connection detail with expected performance.

Lateral Connection to the Perimeter Frame

This connection consisted of moving the first unit away from the perimeter beam and replacing it with a 750mm timber infill with 75mm insitu concrete topping (Figure 3(a)). The infill allows a more flexible interface between the frame and southern hollow-core unit. Some cracking is expected in this interface due to the displacement incompatibility but it is anticipated that the more flexible interface will accommodate this while allowing the beam to deform in double curvature and the hollow-core unit in single curvature, (Figure 3(b)) leaving the floor essentially undamaged. Ductile reinforcing mesh was used in the topping to aid in the performance of the floor by helping to ensure that any damage and cracking would not result in such an early failure of the floor system.



(a) First hollow-core unit to perimeter frame connection.



(b) Expected displacement incompatibility between the hollow-core floor units and the perimeter frame.

Figure 3. Displacement incompatibility and connection between hollow-core unit and perimeter frame.

Diaphragm Tie Reinforcement

The New Zealand Concrete Structures Standard, NZS3101:1995 (Standards New Zealand, [4]), requires that columns shall be tied at each level of the floor system and be capable of resisting 5% of the maximum total axial compression load on the column. NZS3101 specifies that these bars should be placed at an angle close to 45° to the beam. However, this would contribute to the overstrength actions of the perimeter beams, through flange action, therefore the drag bars were placed perpendicular to the longitudinal beams. Two YD20 (f_y=500MPa) drag bars were required by design and were post-installed into the central column, spanning 5m into the floor.

TEST SET-UP

The super-assemblage loading was conducted in drift control. Displacements were applied to the specimen through the form of horizontal shear forces to the top and bottom of each column. The load frame set-up design is explained in Matthews [1]. Three different displacement histories, corresponding to different phases of loading, were applied to the super-assemblage as shown in Figure 4.



Figure 4. Loading histories applied to the super-assemblage.

EXPERIMENTAL RESULTS

Phase I: Longitudinal Loading

The super-assemblage performed well in this phase of loading. The yield drift was determined to be 0.5%. The key results are shown in Figure 5. Diagonal cracks in the infill appeared at +1.0% and extended in the second $\pm 1.0\%$ cycle reaching the infill/hollow-core interface and running along the interface for almost the entire floor length, except around the central column where the drag bars appeared to tie the infill and floor together. By the end of Phase I this crack was 2mm wide with a vertical displacement of 2mm, in the west end (Figure 5(c)). A crack in the south corner of the first unit (ref Figure 1 for layout) developed at +2.0% and extended into the second core of the unit. There was 10mm of hollow-core pull-off in the $\pm 2.0\%$ cycle with the low-friction bearing strip sliding out in some places instead of the unit sliding on the bearing strip (Figure 5(a) and (b)). Some spalling occurred in the later cycles of this phase on the seat of the first unit due to the unit bearing on the unreinforced cover concrete. The economic consequences to an owner of a building with damage like this may become an issue. However, the cracks are considered to be repairable with the only permanent damage being the residual interstorey drift of the building (about 0.8% drift).



(a) Corner crack in first hollow-core unit at +2.0%



(b) Bearing strip sliding out at +2.0%



(c) Crack at hollow-core/infill interface at ±2.0%

Figure 5. Damage to super-assemblage after Phase I loading.

Phase II: Transverse Loading

Very little new cracking occurred in the early stages of the transverse loading. This was because the transverse beams were pre-cracked from the longitudinal loading and these cracks simply opened during transverse loading. Key behaviour photos are shown in Figure 6. In the $\pm 1.0\%$ cycle a crack (2mm at this stage, opening to 6mm at $\pm 2.0\%$) opened up in the ends of both of the transverse beams about 1.0m from the column face as indicated Figure 6(a). It appeared that the weight of the hollow-core units caused the transverse beams to sag, accentuated by the cracked section, and in turn formed a uni-directional hinge at about 1.0m from the column face at both ends. The north side corner of the fourth hollow-core unit at both ends formed a corner crack at -1.0% drift that progressed up from the bottom of the unit to run along the web (Figure 6(b)).



(a) Large crack forming in transverse beam, 1.0m from column face at +1.0%.



(b) Corner crack in north side of fourth hollow-core unit developing into a web-split at +1.0%.

Figure 6. Significant damage in the transverse loading cycle

Phase III: Longitudinal Re-Loading

Early in the Phase III testing, beam spalling at the west end of the first hollow-core unit left the unit with almost $\frac{3}{4}$ of its length with at least 20mm of seat spalled off (Figure 7a) and (b)). At +2.25% drift, on the way to +3.0% drift, the first crosswire of mesh fractured at the hollow-core/infill interface about 2m west of the central column. This first fracture was followed by nine others on the way to +3.0%. Once the mesh had fractured it could be seen that the fracture was due to two mechanisms. Firstly, the tear was due to the floor diaphragm restraining the frame from elongating causing a transverse tension force as the beam tries to translate outwards instead; this produced a horizontal east-west dislocation between the infill and topping of the first hollow-core unit (15mm) once the mesh fractured (Figure 7(d)) as well as accentuating the transverse north-south displacement (i.e. crack width, 10mm) (Figure 7(c)). Secondly, the tear was due to the displacement incompatibility. This caused a vertical offset of 10mm once the mesh had fractured (Figure 7(e)). The crack was 3m long at this stage but the central column was still adequately tied into the building. The transverse beams showed significant amounts of torsion due to degradation of the PHZs accentuated by the hollow-core load eccentricity.

A large section of the unreinforced seat of the fourth hollow-core unit at the west end began to drop away showing the necessity of reinforcing the seat to tie it to the beam. The load carrying capacity of this seat/cover concrete was lost at 3.0% drift (first cycle). The concrete fell out during the second +4.0% cycle (Figure 7(f)). It was during these $\pm 4.0\%$ cycles that the PHZs showed some sign of distress with large sections of cover concrete falling off. The first main longitudinal bar fractured at +3.56% in the second $\pm 4.0\%$ cycle (Figure 7(g)) in the west PHZ in the southern beam with the remaining bars in that PHZ fracturing in the following cycles. A final +5.0% cycle was performed and during this cycle further seat damage was observed along with the main bars and topping mesh fracturing. A photograph of the infill section of the floor at the end of test is shown in Figure 7(h). It was at this stage that life safety became a concern, enough of the hollow-core seat had been damaged to question the stability of the floor diaphragm and nine main bars had fractured in total leading to concern about the stability of the frame elements.



(a) Underside of first hollow-core unit, west end, +3.0% drift.



(c) Crack width of 10mm in places after mesh fractures (+3.0%)



(d) Displacement of 15mm after mesh fractures (+3.0%)



(b) 45-50mm of seat exposed of first unit, west end, +3.0% drift



(e) Vertical offset of 10mm after mesh fractures (+3.0%)



(f) Section of unreinforced seat fallen out at +4.0%



(g) Fractured main bar at +3.56% on 2nd cycle to +4%



(h) Floor damage at end of test (5.0% drift).

Figure 7. Damage in Phase III testing

HYSTERETIC PERFORMANCE

The base shear versus interstorey drift hysteresis plots for Phase I and III are shown in Figure 8(a) and Phase II in Figure 8(b). In Phase I, the hysteresis loop has a little pinching arising from a self-centring effect due to the PHZ cracks not opening and a large part of the deformation occurring at the beam/column interface which acted almost like a self-centring rocking connection. The maximum positive base shear was 1390kN while the maximum negative was 1320kN which both occurred in the first cycle to $\pm 2.0\%$. It can be seen that in the second cycle of loading very little loss in base shear capacity was observed. The overall theoretical base shear capacity was determined to be 1220kN at 2.0% drift onwards, once the entire floor had been activated. The theoretical mechanism assumes that as the interstorey drift increases more of the starter bars along the transverse beam are activated by flange action and these contribute to the negative moment capacity of the exterior hinges, up to a drift of 2.0% when all of the starters have been activated. The interior hinge capacity is made up of a contribution from the infill slab in the form of a yield line mechanism and activated mesh as well as the longitudinal beam bar capacity.



(b) Hysteresis loop for Phase II loading

Figure 8 Hysteresis loops for the three phases of testing

The reason that the overall calculated base shear in Phase I and III was lower than the experimental one was because, in the theoretical calculations, compensation was made for the effect that the heat treatment had on the bars but the exact effect is not known due to the fact that the bars were heat treated in-situ and not tested. Therefore an assumption was made as to the effectiveness of the heat treatment and the resulting yield stress and hinge locations.

Phase I loading appeared to have no effect on the performance of the super-assemblage in Phase II. The hysteresis loop had less pinching than Phase I which is because the transverse beams were reconstructed entirely and therefore there was more distributed cracking and less of a self-centring rocking connection effect at the face of the columns in the hinges as seen in the southern hinges. The maximum positive base shear was 920kN which occurred in the first cycle to +2.0% drift. The maximum negative base shear for Phase II was -970kN which occurred in the first cycle to -3.0% drift. The theoretical mechanism for Phase II assumes that relocated positive moment hinges are at the nominal moments because the hinges are forming in sections of the beams that have not pre-yielded. In the areas where the bars have pre-yielded compensation was made for the effect that the heat treatment had on the bars. Starter bars in the interface between the infill and perimeter beam are also activated in the negative drift direction. This mechanism predicts a positive base shear of 880kN and a negative of -970kN which agree with the experimental data.

The difference in base shear capacities between the positive and negative base shears in Phase II is due, in part, to the non-symmetrical reinforcing layout. On the northern side of the super-assemblage the floor is not tied to the tie beam with starters therefore these can not be activated in a negative hinge moment cycle. A crack line also forms at the beam/infill interface through the starter bars. These reasons also account for why the base shear in Phase II is considerably less than Phase I as well as the use of the nominal yield stresses for the positive moment hinges in Phase II due to these hinges forming in steel that was not pre-yielded and subsequently heat treated.

It should also be noted that although hysteresis loops are beneficial in determining the overall capacities of test super-assemblies their usefulness is limited when looking at the performance of individual elements. As can be seen by the graphs in Figure 9, the overall comparison of the two super-assemblies would be that similar overall base shears were observed, the Stage 1 (Matthews [1]) super-assembly was slightly stiffer than Stage 2 (Lindsay) and the Stage 2 super-assembly was loaded to higher drifts and therefore underwent more plastic deformation. What is not known is that the hysteresis loops are dominated by the performance of the perimeter concrete frame, and in Stage 1 the overall performance of the super-assembly was vastly inferior to the Stage 2 testing due to premature failure of the hollow-core flooring system. Therefore hysteresis loops should be used only to determine the overall capacities of systems and the individual performance should be assessed in a different manner.





(b) Stage 2 hysteresis loop for Phase I and III

Figure 9. Hysteresis loops for Stage 1 and 2 for comparison of performance.

FRAGILITY ANALYSIS

As was shown above, the use of hysteresis loops in categorising and assessing the performance of a system is not adequate. In Stage 1, there was little evidence to indicate the poor performance of the hollow-core floor system. Assessment of the frame performance alone is not satisfactory in determining the damage state and account needs to be made for the performance of all of the elements in the system.

An investigation has been undertaken by Matthews that determined the expected interstorey drift demand on the class of structure tested in this programme. The findings were, in terms of the expected (median) drift;

or

$$\widetilde{D}_D = 2.0 (F_v S_1)_D \tag{1a}$$

$$\tilde{D}_{D} = 2.0(PGA)_{D} \tag{1b}$$

in which \tilde{D}_D = the median (50th percentile) drift <u>demand</u> as a percentage of the storey height, $(F_v S_I)_D$ = one second spectral acceleration for tall structures (above four stories) and $(PGA)_D$ = peak ground acceleration for low rise structures (up to four stories). From Equation (1a) it follows

$$\left(F_{\nu}S_{1}\right)_{C}=0.5\widetilde{D}_{C} \tag{2a}$$

$$(PGA)_C = 0.5\tilde{D}_C \tag{2b}$$

where \tilde{D}_{c} = expected drift <u>capacity</u> of the structure. Analysis conducted by Matthews [1] showed that the distribution of drift outcomes is lognormal with a coefficient of variation of $\beta_D = 0.52$. When combining distributions, to give an overall composite distribution, Kennedy et al [5] showed that by using the central limit theorem the coefficient of variation for a lognormal distribution can be found from:

$$\beta_{C/D} = \sqrt{\beta_C^2 + \beta_D^2 + \beta_U^2} \tag{3}$$

where β_c = coefficient of variation of the capacity, taken herein as $\beta_c = 0.2$ (Dutta, [6]); and $\beta_u =$ dispersion parameter to account for modelling uncertainty, taken here $\beta_U = 0.2$. Applying (3 gives $\beta_{C/D} =$ 0.60. By using a lognormal cumulative distribution that can be described by a lognormal variate ξ_{β} (where the median = 1 and the lognormal coefficient of variation, $\beta_{C/D} = 0.60$), the distribution of ground motion demands needed to produce a given state of damage can be found by

$$F_{\nu}S_{1} = 0.5\widetilde{D}_{c}(DS)\xi_{\beta} \tag{4}$$

or

$$PGA = 0.5\tilde{D}_{c}(DS)\xi_{\beta}$$
⁽⁵⁾

where $\tilde{D}_c(DS)$ = the expected value (in this case, the experimentally observed drift) for a given damage (5) state (DS). The state of damage after an earthquake is typically quantified by a colour-coded or numerical format. Both of these are outlined in Table 1 and Table 2 respectively, along with the drift classification of the test super-assembly under both of these systems.

Tag Colour	Description of damage	Classification (Interstorey drift)	
	level	Floor	Frame
Green	No Damage, building occupiable	1.0%	1.0%
Yellow	Moderate levels of damage. Building can be entered to remove belongings.	2.0%	2.0%
Orange	Heavy damage. Building can be entered for brief periods to remove essential items only	2.25%	3.0%
Red	Near collapse. Building can not be entered	4.0%	4.0%

Table 1. Definition of colour coding used to classify building damage following an earthquake and the interstorey drift classification for the super-assemblage investigated by Lindsay.

Damage State	Description of	Post-earthquake utility	Classification (Interstorey drift)	
	Damage	of structure	Floor	Frame
1	None (pre-yield)	Normal	-	-
2	Minor/Slight	Slight Damage	1.0%	1.0%
3	Moderate	Repairable Damage	2.0%	2.0%
4	Major/Extensive	Irreparable Damage	2.25%	4.0%
5	Complete Collapse	Irreparable Damage	-	-

 Table 2. Definition of damage states used to classify building damage following an earthquake and the interstorey drift classification for the super-assemblage investigated by Lindsay (Mander, [7]).

Figure 10 shows the fragility curves for the floor and frame performance when classified under the colourcoded and numerical schemes for the two stages of testing. On each of the graphs the 10% in 50 years, Design Basis Earthquake (DBE), $F_{\nu}S_{I} = 0.40g$ for Wellington, New Zealand is shown, as well as the 2% in 50 years, Maximum Considered Earthquake (MCE), $F_{\nu}S_{I} = 0.72g$, for Wellington, New Zealand. If the structure is classified in terms of the critical element (floor or frame) then it can be seen that if the damage to the two structures (Matthews and Lindsay) was classified in terms of colour-coding under a MCE then in Stage 1 72% of structures would be expected to be red tagged or have collapsed (Figure 10(a)), whereas in Stage 2 only 23% would be expected to sustain damage such that the building could not be entered (Figure 10(b)). Both of these performances are dictated by the performance of the floor. Under a DBE, in Stage 1, every building would still sustain some form of damage to the floors whether it be moderate (5%), heavy (60%), near collapse (27%) or total collapse (8%) (Figure 10(a) and (c)). In Stage 2, 65% would sustain no damage allowing immediate occupancy and 29% would sustain moderate damage to the floors while only 5% of floors would be red tagged. No buildings would collapse from inferior floor or frame performance (Figure 10(b) and (d)).

If the damage is classified in terms of different damage states, then for a MCE, in existing structures with conventional precast floor seating details (Stage 1), only 2% of structures would be expected to sustain slight or repairable damage. The remaining 98% of the structures would be demolished due to irreparable damage or collapse, of these some 32% of floors would be expected to partially or entirely collapse leading to possible loss of life (Figure 10(e)). Even under a DBE, 92% of structures would sustain irreparable damage with 8% leading to possible loss of life (Figure 10(e) and (g)). In this research, testing a structure with the proposed seating details (Stage 2), under a MCE, 71% of the buildings would sustain repairable damage to the frame or floor, of the remaining 29% irreparable damage, 23% of floors and 5% of frames would sustain major damage (Figure 10(f) and (h)). Under a DBE, 94% of buildings would sustain repairable damage to the floor and frame with 5% of the remaining 6% of floors sustaining heavy irreparable damage while none of the remaining 6% of frames sustains heavy irreparable damage (Figure 10(f) and (h)). This is almost a complete reversal of the damage states identified during Stage 1 testing and shows the improved performance due to the enhanced details.

As can be seen, in Stage 2, the performance of both the frame and floor are very similar whereas in Stage 1 the performance of the floor is vastly inferior to the performance of the frame and therefore the overall performance is dictated by the poor performance of the floor. The findings from Stage 2 adhere to the expectations of ductile structures designed and detailed in accordance with the principles of capacity design as well as meeting the target objective that the confidence interval at the onset of irreparable damage under a DBE exceeds 90%. It is clear that similar conclusions can be drawn whether the building damage is rated by the colour-coded or damage state format.



(a) Matthews' fragility curve for the floor performance when rated according to colour coding



(c) Matthews' fragility curve for the frame performance when rated according to colour coding



(e) Matthews' fragility curve for the floor performance when rated according to damage states







(b) Stage 2 fragility curve for the floor performance when rated according to colour coding



(d) Stage 2 fragility curve for the frame performance when rated according to colour coding



(f) Stage 2 fragility curve for the floor performance when rated according to damage states



performance when rated according to damage states

(h) Stage 2 fragility curve for the frame performance when rated according to damage states

Figure 10 Fragility curves for both stages of testing using both colour-coded and numbered format for quantifying building damage.

DISCUSSION AND RECOMMENDATIONS

The performance of the hollow-core unit was significantly better than the test by Matthews (Stage 1) [1] who investigated existing construction practice that was found to perform at a level far below expectations. This follow-up investigation by Lindsay (Stage 2) demonstrated satisfactory overall performance with the structure maintaining life safety throughout the test.

Performance of Hollow-core Seat Connection

It is clear from the photos (Figure 5(b) and Figure 7(a & b)) that the low friction bearing strip did not perform how it was designed to. The bearing strip has teeth on one side and is smooth on the other allowing the hollow-core unit to slide on the smooth surface. In this case there was not enough bond/friction between the toothed surface and the floor supporting seat and in some places the bearing strip slid with the floor unit instead. By designing a bearing strip that has bigger teeth, to grip the beam better, or bonding the underside of the bearing strip to the beam should stop this movement.

From the initial analysis of the results it is evident that the compressible backing board did not actually compress much more than approximately 1mm. This is because in the early stages of testing the compression strut and rotation of the beam and hollow-core are small as well as the occurrence of elastic elongation of the perimeter beams and therefore the backing board will not compress. After yielding of the super-assemblage, beam elongation of the longitudinal beams has occurred meaning that the rotation of the hollow-core unit, which would cause compression of the backing board in simplified two-dimensional tests that do not consider beam elongation, does not compress the backing board. However, a baffle of some sort is required to stop concrete from entering the cores and therefore isolate the floor units from the beam. The authors recommend a thinner and not necessarily compressible backing board but one that is still robust enough to resist the pressure of fresh concrete. The need to reinforce the seat of the hollow-core has become evident (Figure 7(a) and (f)). Reinforcing the seat with an additional longitudinal bar and stirrups would prevent large sections of the unreinforced seat from spalling off.

Performance of Infill Slab between Perimeter Frame and First Hollow-core Unit.

This element performed very well. Damage to the infill section was always anticipated but as can be seen from the photos the rest of the floor was essentially uncracked. Ductile reinforcing mesh was used in the topping to try and stop the fracture of the reinforcing crossing the damaged interface between the infill and first hollow-core unit. This ductile reinforcement did not perform as well as hoped. The reinforcing mesh fractured at an interstorey drift of 0.35% above that when it fractured in the Matthews test (2.25% vs. 1.9% drift). However, at that time the super-assemblage had undergone more than six times the plastic rotation than when the mesh fractured in the Matthews test. The authors recommend, however, that the mesh be substituted for simple deformed reinforcing bars (e.g. HD10 at 300 crs both ways: 5th percentile yield stress of 500MPa) and the starter bars from the perimeter beams run over this interface to lap with the topping reinforcement. This will increase the ductility of the damaged interface and lower the risk of fracture of the reinforcing across this joint.

Global Performance Issues

The failure of the longitudinal reinforcing bars can be predicted by low cycle fatigue theory (Dutta and Mander, [8]). As this failure is a function of material properties and overall plastic rotation it is not a parameter that can be altered and therefore becomes the defining failure point for the super-assemblage.

The damage to the corners of the first and fourth hollow-core units and cracking of the soffit of the units could be avoided. If the hollow-core units are not seated in the plastic hinge zones (PHZ) of the supporting beams they would not be forced to undergo the large deformations of the PHZ of the beams. Cracking of the soffit of the units should not occur and large sections of the corners of the units should not fracture.

By using the detail shown in Figure 11 on either side of all columns, the plastic hinge zones are forced into the area under the infill rather than underneath the hollow-core unit. The extra bar cast into the beams achieves this. The starter bars extend across the hollow-core/infill interface and are lapped with the HD10 topping reinforcement at 300 centres each way.



Figure 11. Recommended detail to reduce damage to hollow-core units

Overall System Performance

The hysteresis loops showed a small amount of pinching in the longitudinal loading cycles due to a rocking type connection that formed at the interface between the old and new concrete at the beam/column joint. The theoretical mechanism for the longitudinal loading assumes a progressive yield of the starter bars up to 2.0% drift when all of the starter bars are activated. The effect of the heat treatment on the reinforcing bars is not accurately known, this is because the bars were heat treated in-situ and were unable to be tested, and therefore a yield stress value was assumed. The performance of these bars could have implications on the overall performance of the super-assembly.

The hysteresis loops appeared to be well-formed and dissipated a reasonable amount of energy. However, as previously discussed, this can be misleading. Hysteresis loops are a good indicator of overall system capacities but the performance of the individual elements of the system needs to be investigated in order to assess system performance accurately.

Fragility Analysis Implications

By using fragility curves to assess individual elements of a system it is possible to determine the implications of the drift damage on New Zealand constructed buildings of this type. The analysis shows a vast improvement in performance of Stage 2 testing compared with Stage 1 and this is entirely due to the improved detailing. The results show that following a DBE (10% in 50 years) in Wellington, New Zealand, 94% of buildings would sustain damage to the floors that would be considered repairable and under a MCE (2% in 50 years) 71% of buildings would sustain damage, due to damage to the floors, that would probably be repairable. This is a vast improvement on the expected near total devastation of precast buildings of this type under a MCE following Stage 1 testing. The improved details mean that the floor system performs at a level not inferior to that of the frame. However, in both testing stages the performance of the super-assembly is governed by the performance of the floor system can be made and the ultimate limit of the structure can then be accurately determined by low cycle fatigue of the longitudinal reinforcing. Fragility analysis allows comparisons to be made between separate elements and drift limits to be placed on different performance levels.

CONCLUSIONS

The experiment conducted as part of this research has ensured that new precast concrete moment resisting frame buildings with precast, prestressed hollow-core floors can be expected to perform satisfactorily up to interstorey drifts well in excess of 3.0% with the details outlined above. These details also ensure only moderate economic consequences to the owner of the building under a 10% in 50 years; design basis earthquake, and that life safety is maintained under a 2% in 50 years: maximum considered event. The target objective that the confidence interval at the onset of irreparable damage under a DBE exceeds 90% is also achieved with the new details. The superior performance of the proposed future detailing practice when compared to existing practice was clearly demonstrated when the damage states are compared in a fragility analysis.

This research has also shown the necessity to test structures in the three-dimensional format to fully understand certain elusive secondary, three-dimensional effects that are present. It is concluded that further work is required to test the design recommendations outlined above in Figure 11. Further work also needs to be undertaken to develop retrofit measures for existing structures and to test further seating details for other classes of precast concrete floor systems in order to determine their performance under three-dimensional conditions.

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