

Reinforced concrete seating details of hollow-core floor systems

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ABSTRACT: Recent earthquake engineering research has raised concerns of the seismic performance of precast prestressed concrete hollow-core floor systems. Experimental research showed that with simple detailing enhancements, significant improvement in the seismic performance of hollow-core floor systems can be expected. The present experimental research aims at validating several new detailing enhancements. Based on previous research findings, the present super-assembly experiment included the following details: (i) a reinforced connection that rigidly ties the floor into the supporting beam, (ii) an articulated topping slab portion cast onto a timber infill solution that runs parallel to the hollow-core units and edge beams; (iii) specially detailed supporting beam plastic hinge zones reducing potential damage to the hollow-core units; (iv) Grade 500E reinforcing steel used in the main frame elements; and (v) mild steel deformed bars in the concrete topping in lieu of the customary welded wire mesh. The full-scale structure was cyclically tested in both the longitudinal and transverse directions to inter-storey drifts of $\pm 5\%$. Observations show extremely positive results with minor damage incurred by the hollow-core flooring and the overall performance dictated by the performance of the moment resisting frame. Recommendations for the forthcoming revision of the New Zealand Concrete Standard, NZS 3101, are also made.

1 INTRODUCTION

The collapse of the hollow-core units during testing by Matthews (2004) and Matthews et al (2003a,b) flagged issues over the performance of existing precast concrete frame structures with hollowcore flooring structural systems. A continuation of that research by Lindsay (2004) and Lindsay et al (2004a,b) demonstrated that different structural details could be implemented in new structures and these could be expected to behave adequately in a seismic event. This research project is a further continuation of previous work done by Matthews (2004) and Matthews et al (2003a,b) on existing structures and Lindsay (2004) and Lindsay et al (2004a,b) on new structures.

The popularity and widespread use of precast concrete is recognised in New Zealand design standards where there is specific reference to precast concrete flooring support conditions. Amendment No. 3 to the current New Zealand Concrete Design Code NZS3101:1995 provides two details for the connection of hollowcore floor units to reinforced concrete frame supporting beams. While Lindsay (2004) reported on the performance of the first of these, the second solution specifies a reinforced connection that rigidly ties the floor into the supporting beam, but to this point remains untested in any large-scale three-dimensional experiments. This research experimentally investigates the effectiveness of this solution and its adequacy for inclusion into the upcoming revised New Zealand Concrete Standard and use in New Zealand construction practice.

This paper initially provides an overview of the test specimen details and experimental set-up. The second part of this paper provides the visual and instrumental observations of the testing. The results and overall effectiveness of the design changes and new features are discussed and concluding remarks are made.

2 SUPER-ASSEMBLAGE DESIGN DETAILS AND CONSTRUCTION

2.1 Hollow-core seating details

The seated connection detail used for this experiment is the second detail prescribed in Amendment No. 3, NZS 3101:2004. The connection features two of the four hollow cores reinforced and filled with concrete, and is diagrammatically shown in Figure 1(a). Grade 300 D12 reinforcing was used at 300mm centres for the starter bars, which is lapped with the diaphragm reinforcing. In the two reinforced cores, Grade 300 R16 bars were placed close to the bottom of the cores. To prevent concrete entering the two non-filled cores of each hollow-core unit, a stiff backing board was used in place of the more conventional end plug. This is to help ensure that the rotation of the floor units relative to the beam occurs at the critical section at the beam-to-floor interface and that the relatively brittle hollow-core unit does not experience high rotational demands. This substitutes for the conventional use of end plugs, which create a concrete key part way into the cores, which, under relative rotations, causes prying and splitting forces within the unreinforced webs of the hollowcore. The hollow-core unit was seated on a low friction bearing strip and the seat widths were 50mm and 75mm at the east and west ends of the test structure respectively. The code amendment prescribes a 75mm seating, but it was decided to investigate the effect of a shorter seat width for cases when hollow-core units arrive on site short, due to drying or elastic shrinkage, indicative of real construction practice.

2.2 Lateral connection and diaphragm reinforcing

The infill detail used in this testing is shown in Figure 1(b). A 75mm thick concrete topping was cast on a 750mm wide timber plank infill running between the first hollowcore unit and the perimeter beams. The starter bars from the perimeter beams extended 600mm into the topping above the first hollowcore unit. It was hoped that the longer starter bars and use of individual reinforcing bars instead of mesh would increase the ductility capacity at the infill-hollowcore interface and reduce the risk of fracture from occurring. The reinforcement used in the diaphragm topping slab was individual reinforcing bars (instead of cold drawn or ductile mesh) which was used to provide a higher level of ductility in areas of high deformation and ensure that the topping reinforcement did not fracture. For this experiment, Grade 300 D12 reinforcement was used at 300mm centres in both directions. The use of D12 bars is beneficial for the fact that it is the same as the starter bars used, and all lapping is between the same reinforcing. The current standards as stipulated in the amendment to NZS 3101:1995, state that starter bars should extend to the larger of either 20% of the hollowcore span, or the development length (l_d) plus an additional 400mm, which in some cases can be a considerable length. In the experiment, the starter bars and diaphragm bars were the same, in effect satisfying the curtailment requirements for starter bars.

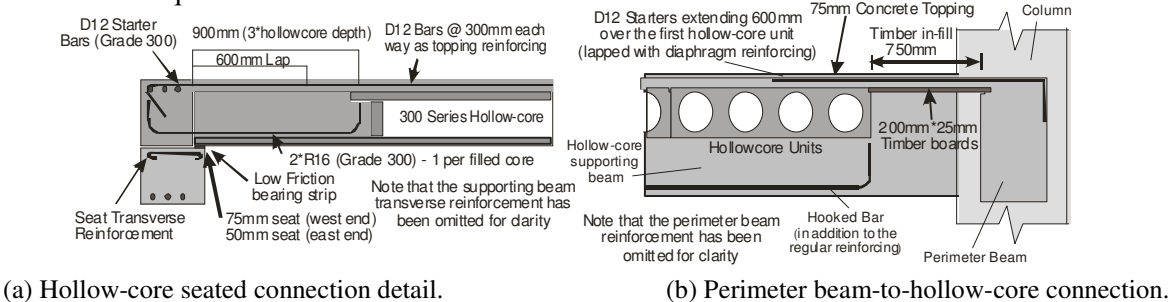


Figure 1. Hollow-core connection details.

2.3 Super-assembly construction

The full-scale super-assembly specimen was a two-bay by one-bay structure designed as a lower storey corner section of a multi-storey precast concrete moment resisting frame building. The pretensioned flooring system ran parallel to the longitudinal perimeter beams (east-west), past the central column, and were seated on the transverse beams. The frame dimensions were identical to the original Matthews rig to maintain the same loading set-up and to be able to compare results. The columns were 750mm square in section and spaced at 6.1m centres with an inter-storey height of 3.5m. The perimeter beams were 750mm deep by 400mm wide, and the transverse seating beams were 750mm deep by 475mm wide. The super-assembly was constructed in a similar fashion as it would be done on a construction site. Plan and elevation layouts of the super-assembly are shown in Figure 2.

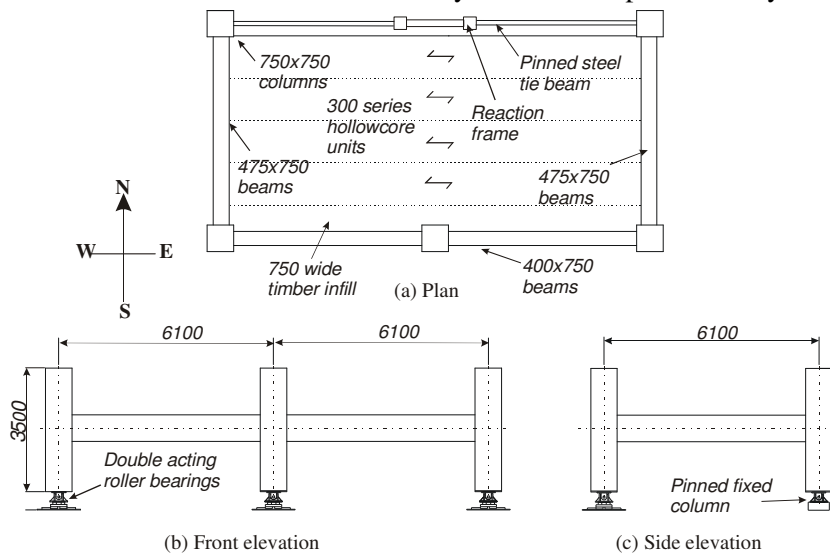


Figure 2. Details and geometry of the super-assembly

The original testing (Matthews, 2004) was, in part, a retrospective look at the structural details of existing buildings whereas this experiment was aimed at validating new construction solutions. With this in mind, it was decided to use Grade 500E seismic steel throughout the frame, which is currently the most commonly used grade of steel in New Zealand construction practice. During previous testing (Lindsay, 2004), the hollowcore units seated on the potential plastic hinge zones of the supporting beams suffered damage due to the high deformation occurring in these zones. To restrain the deformation demand on the floor units, a hooked bar was placed adjacent to the longitudinal steel (as shown in Figure 1) within the beam to force the hinge to occur close to the column face. Another negative feature from the performance of the previous experiments was the spalling of the ledges that supported the precast floor units, sometimes called the “seat”. To overcome this, transverse reinforcement was placed within the seat to tie the seat concrete back into the transverse beams. Part of the support detail was a low friction bearing strip between the bottom of the precast unit and the top of the ledge. A second generation of low-friction bearing strip was used which featured longer ‘teeth’ to both ensure the strip stayed fixed to the ledge and reduce the effects of seat surface roughness.

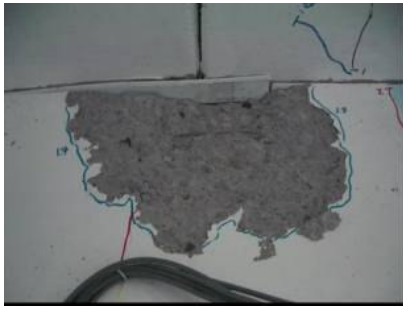
3 TEST SET-UP

Loading of the super-assembly was undertaken by inter-storey drift control. A self-equilibrating primary loading frame was used to apply equal and opposite shear forces to the top and bottom of the columns. A secondary loading frame was used to ensure the columns displaced parallel to each other and in a realistic manner. A full account of the test loading configuration can be found in Matthews (2004). The experiment consisted of three phases comprising: (i) longitudinal loading of two completely reversing cycles to inter-storey drifts of $\pm 0.5\%$, $\pm 1\%$ and $\pm 2\%$; (ii) transverse loading of two completely reversing cycles to $\pm 0.5\%$, $\pm 1\%$, $\pm 2\%$ and $\pm 3\%$; and (iii) longitudinal re-loading of an initial reversing cycle to $\pm 2\%$ followed by two completely reversing cycles of $\pm 3\%$, $\pm 4\%$ and $\pm 5\%$.

4 EXPERIMENTAL RESULTS.

4.1 Phase I: longitudinal loading

Photographs of the key damage sustained are shown in Figure 3. From the early stages of the experiment, diagonal torsional cracks appeared at the ends of the transverse beams and continued to extend and widen throughout this phase of testing. Cracking propagating diagonally outwards from the longitudinal beams appeared over the infill slab from the onset of testing and continued to extend into the second hollowcore unit, arching towards the south central column. Damage in the plastic hinges was confined to one major crack at the beam to column interface, and another significant crack around 300mm from the column. Some instances of spalling of seat cover concrete were evident at +1%. Following the 2% cycles, the spalling had not extended but had worsened in a few areas, as can be seen in Figure 3(a). The damage to the hollow-core floor units themselves was minimal. Damage was confined to a single crack at the beam to floor interface, as shown in Figure 3(b), hairline hollow-core soffit cracks, and a web crack across the side and bottom of the hollow-core unit immediately next to the infill strip was observed at +2% propagating at 45 degrees from the seat to the topping. Beam elongation was illustrated both by the residual crack openings and by the sliding of the floor units out from the supporting beams, as could be seen by the exposure of the unpainted sections of the soffit of the precast floor units. It also showed that the bearing strips were working as intended: the strip staying fixed to the ledges while the hollow-core units slide across the top of the strips. The residual drift after the $\pm 2\%$ cycles was around $\pm 1.1\%$ and the structure had suffered moderate, but repairable damage.



(a) Spalling of seat concrete after Phase I loading.



(b) Continuity crack after Phase I.

Figure 3. Damage to the super-assemblage from Phase I longitudinal loading.

4.2 Phase II: transverse loading

A selection of photographs showing the behaviour of the test specimen under transverse Phase 2 loading is shown in Figure 4. During the early stages of testing, the structure exhibited very little new damage with only the cracks caused by Phase 1 loading opening wider. In a similar fashion to the longitudinal beams, the rotation experienced by the transverse beams was concentrated at or near the column face rather than being distributed over a conventional plastic hinge zone length (due to a different reinforcement configuration). Figure 4(a) and (b) show the damage in these zones, where crack widths were approximately 15mm at 3% drift. There was little new damage to the floor and topping slab in general, although elongation of the transverse frames was clearly apparent with large openings between the column and topping slab and the top of beams under negative moments. At -3% drift, the topping slab was pulling away from the corner columns approximately 25mm. Vertical deformations of the floor also became apparent through vertical displacement of the supporting beams. This movement can be accounted for by shear deformations at the ends of the transverse beams. The large deformation occurring at the northern end of the east transverse beam resulted in a crack propagating from the seat through the northernmost filled core of the northernmost hollow-core unit and into the topping slab, as shown in Figure 4(c). This damage did not worsen during the rest of the experiment. At the completion of the transverse Phase II loading to $\pm 3\%$ the residual drifts were roughly $\pm 1.6\%$ and the structure had suffered moderate damage but was still in a repairable state.



(a) Damage to the south end of the west transverse beam at +3%. (b) Damage to the north end of the west transverse beam at +3%. (c) Hollow-core web cracking and damage at the north east corner after Phase II testing.

Figure 4 Notable damage from Phase II transverse testing.

4.3 Phase III longitudinal re-loading

Figure 5 shows some key photographs of the Phase III longitudinal re-loading of the test specimen. During the initial stages of loading, no major new damage occurred. A 1m long soffit crack, 1-2mm wide, running along the unit, appeared at the west end of the southernmost unit underneath one of the filled cores. However no more damage to the hollow-core or beam seats was witnessed throughout the remaining testing. At -2.4% drift, compression crushing of the top concrete of the south centre column occurred. The torsional response of the transverse beams was worth noting. While the front frame and north columns were inclined (east-west), the transverse beams appeared to remain vertical, acting to minimise the relative rotation imposed on the seating connection. Cracking at the ends of the transverse beams showed between 2mm and 5mm of lateral movement and evidence of torsional hinging. At $\pm 4\%$ drifts, significant amounts of concrete had become loose and fallen from the plastic hinge zones of these beams, exposing several of the reinforcing bars. Figure 5(b) is indicative of the damage in the plastic hinges and although the structure was still stable and maintained load carrying capacity, the damage in some areas became irreparable and major components would need to be replaced for further structural use. Prior to the first cycle to -4% drift, buckling of the compression bars at the bottom of the western beam of the south frame was observed, as illustrated in Figure 5(c). On the accompanying cycles with an opposite bending moment the bars, now in tension, did not straighten completely. During the final cycle of loading to $\pm 5\%$ drift (at -1.14% and then at -0.36% , unloading from -5%) the inside and outside top reinforcing bars respectively in the eastern beam of the south frame fractured. Although, the future load-carrying capacity of the structural system was jeopardised, life-safety of the structure was still maintained at the $\pm 5\%$ drift limit; only two reinforcing bars had fractured ensuring that the frame remained stable and the lack of damage to the hollowcore units and seating support mitigated the major life safety concerns.



(a) Super-assembly specimen at +5% drift. (b) Damage of the southeast plastic hinge at +5% drift. (c) Longitudinal beam bar buckling.

Figure 5. Damage during and after Phase III loading.

5 DISCUSSION OF RESULTS

The overall behaviour of the super-assembly was positive and the specimen performed well up to inter-storey drifts of $\pm 5\%$ when the longitudinal reinforcing bars in one of the beams fractured due to low cycle fatigue. The hollow-core flooring sustained little damage and the overall performance was dictated by the behaviour of the frame, as against the failure of the floor (Matthews, 2004). The following provides comments on the primary areas of interest and significant features of the test.

5.1 Hollow-core seated connection

The beam-to-floor connection detail performed well and the super-assembly structure was able to sustain inter-storey drifts up to $\pm 5\%$ without loss of support of the floor. At the conclusion of testing, there was minor diagonal web cracking in the hollow-core units at the eastern end. A camera that could be placed into four of the hollow-core units could not detect any internal cracking of the particular cores observed and only minor soffit cracking was observed under the filled reinforced cores. The single crack that formed along the beam-to-floor interface in the timber infill link slab concrete and lack of any other cracking demonstrated the objective to centre the rotation on this plane and restrict the rotational demand on the floor units themselves. The bearing strip was also adequate and allowed the floor units to slide whilst staying on the seat. The improved friction resistance and better grip associated with the newer generation of bearing strip seems to have greatly improved the performance of the bearing strip. The minimal amount of spalling of the edge of the seats was also a very positive feature of the experiment owing to the beneficial effects of torsion of the supporting beams, the improved bearing strip performance and the transverse reinforcement of the seat. It must also be noted that there was no difference in behaviour between the 50mm and 75mm seating ends of the structure. However, the specification of a minimum of 75mm seat allows for elastic and drying shortening of the hollow-core units that occur and specifications should adhere to minimum seat widths of 75mm or more. Consideration of the construction tolerances are in addition to the minimum 75 mm seat width.

5.2 Timber infill connection and diaphragm performance

The inclusion of the timber infill connection isolating the longitudinal beams from the floor system performed well during the Lindsay (2004) and Lindsay et al (2004) experiment but it did suffer from a few shortcomings. The provision of longer starter bars and use of conventional reinforcing instead of ductile mesh for this investigation proved to be successful. The presence of the longer starter bars, which were terminated 600mm over the first floor unit ensured that a longitudinal crack or tear did not form at the interfaces between perimeter beam and infill slab, and the infill slab and first hollow-core floor unit. The cracking in the infill only showed very small signs of vertical displacement at the higher drifts during Phase III loading. The crack pattern extended through the topping concrete over the first floor unit and into that above the second floor unit indicating that the longer starter bars were more suitable and able to distribute the forces over a larger area. Figure 6 illustrates the cracking pattern and damage to the infill following the completion of testing. The use of conventional reinforcing within the diaphragm also proved to be successful. The combination of the longer starter bars and conventional reinforcement throughout the topping concrete ensured that no major cracks appeared, and therefore the diaphragm steel was not exposed to high ductility demands.



Figure 6. Floor damage after the completion of testing.

5.3 Supporting beam detailing enhancements

To overcome some of the deficiencies observed during the previous experiments (Matthews, 2004; Matthews et al, 2003; Lindsay, 2004; Lindsay et al, 2004), detailing improvements including a hooked

bar to promote column face hinging and seat transverse reinforcement were implemented. The minor damage that occurred within the hollow-core units at the southern corners of the structure, point to the added hooked bar performing as designed – restricting the plastic hinge in the beams to the column faces, inhibiting plastic deformations from progressing under the first hollow-core unit. That action was detrimental in the previous work (Matthews 2003, Lindsay 2004). The hinging in this current investigation was confined to a small area, approximately within half the beam depth from the column face for all of the plastic hinges within the structure, so it is difficult to say that the hooked bar was solely responsible. However, the unit seated on the northeastern corner plastic hinge did experience web cracking, which can be seen in Figure 4(c), which further emphasises the fact that it is preferable to seat hollow-core floor units away from areas of high deformation, and infill-type isolation details should be used over plastic hinge zones of beams.

5.4 Transverse beam torsion

One of the significant features of the present experiment was the degree of torsional twist evident under longitudinal loading on the two transverse beams. Figure 7 shows diagrammatically the behaviour observed. Five inclinometers were installed on the western transverse beam as a means of gauging the amount of torsion experienced. Figure 8 presents the torsion, in terms of percentage drift as a function of time for both Phases I and III loading. It can be seen that during early stages of testing, the rotation of the beam followed that of the corner columns - rigid rotation, which would be expected. In the latter stages of Phase I the beam rotation was similar to the columns for positive drifts but less for negative drifts, which shows the effect of the eccentric loading of the flooring units that rotate the beam in a positive direction. Figure 8 also shows that the transverse Phase II loading had had a marked effect in that beam rotations for Phase III were noticeably reduced with respect to the column signifying that the beams were remaining essentially upright and the flooring horizontal. There is an apparent tendency towards positive drifts accounting for the eccentric floor support. For example, at the first cycle to +4% the beam was at an average +1% inclination, and the first cycle to -4%, the beam rotation remained at around +0.1%. The fact that all five of the inclinometers, which were distributed evenly along the beam, show similar behaviour indicates that the torsion was occurring at the ends of the beams and that torsional hinges were present. The presence of torsional hinges had important effects on the behaviour of the structure. The positive implication of the torsional hinging was that the relative rotation between the flooring units and supporting beam was small and this can help explain the excellent performance of the seated connection and lack of damage.

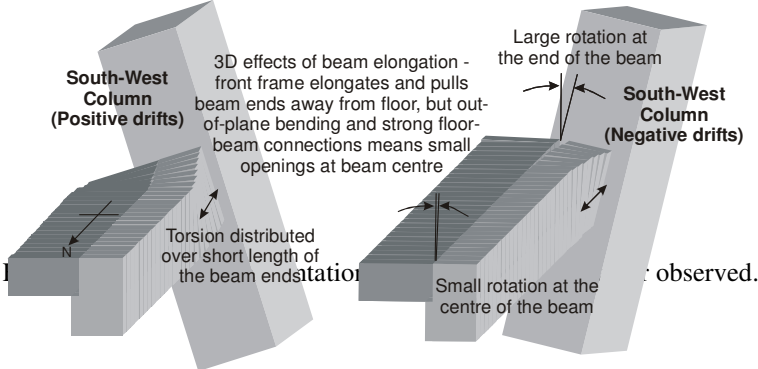


Figure 7. Schematic representation of the torsional behaviour observed

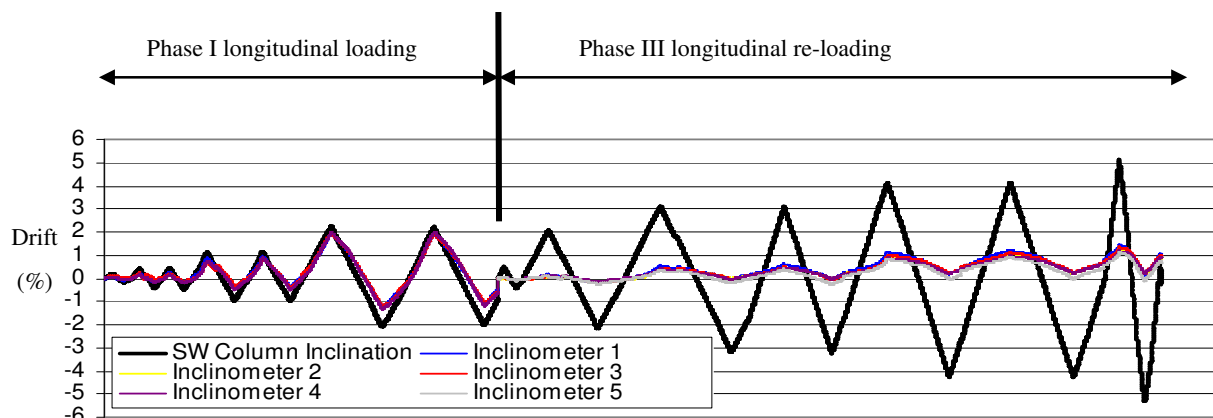


Figure 8. Torsion rotations during Phase I and III longitudinal loading.

6 CONCLUSIONS

The rigid floor-to-supporting beam connection behaved well up to structural inter-storey drifts of $\pm 5\%$, where damage to the hollow-core units and seating support was minimal. Previously, the details investigated herein were untested, and for that reason Amendment No. 3 to the current New Zealand Concrete Design Code NZS3101:1995 limited inter-storey drifts of this class of construction to 1.2%. In light of the good performance, this restriction should be removed. The use of longer starter bars and conventional reinforcing with the articulated timber infill slab connection performed well with no major cracking and tearing experienced. The displacement incompatibility that occurs between the floor units and the beams of the parallel frame was able to be accommodated and diaphragm action was maintained. This research has shown that new concrete frame structures with hollow-core flooring can be expected to perform well in an earthquake event and that life-safety would be maintained under a 2% probability seismic event in 50 years (a 2500 year return period event).

This research has successfully validated several detailing enhancements for hollow-core floor systems in new concrete structures. However, more research at both a sub-assembly level and in full-scale three-dimensional test rigs needs to be done to examine retrofit measures for existing buildings, and more broadly, to investigate the seismic adequacy of other precast flooring systems.

7 ACKNOWLEDGEMENTS

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