

## Seismic performance of sub-assembly of a demountable precast concrete frame building

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**ABSTRACT:** The conventional reinforced concrete and existing precast concrete buildings are generally either fully or partially monolithic in form. Because of this, these buildings are to be demolished instead of dismantling and reuse of undamaged building components either at the end of a buildings life span or when the building is in an irreparable damage state after an earthquake. Also, these types of buildings which are in repairable damage state after an earthquake require considerable downtime to repair in addition to the repair cost to fully restore their functionality. This will induce substantial seismic losses contributed by direct repair cost, and more significantly by the downtime (i.e. occupancy interruption). For these reasons, the authors are working on a new building system using standard precast concrete elements and steel connections which is industrialized, flexible, and demountable. The proposed building system can also be considered as a low loss building system because of quick repair/replace of damaged building components with new ones and thereby minimizing the seismic losses due to occupancy interruption. In this paper, the seismic performance of the steel connections (using steel angle/tubes, steel plates and threaded rods/bolts) between the precast concrete beams and columns are investigated under quasi-static cyclic loading tests. The details of the experimental test-setup, overall hysteresis behaviour and damage state of the tested beam-column sub-assemblies are reported.

### 1 INTRODUCTION

As conventional reinforced concrete (RC) and precast concrete buildings are either fully or partially monolithic in form, they have to be demolished either at the end of the building's life span or when it is decided to construct a new building or building has suffered an irreparable damage after an earthquake. The demolition process of a building is environmentally very unfriendly and causes extensive wastage of building materials. It is reported that construction and demolition waste (CDW) amounts to 17% and 40% of total landfill waste in New Zealand and Australia respectively (Crowther 2005; Storey et al. 2005). Especially demolition of concrete buildings requires huge amount of energy, it consumes around 275 mega joule (MJ) per ton and the crushing of concrete consumes another 85 MJ/t (Reinhardt 2012). Demolition of a RC building is usually very time consuming, and requires careful planning to avoid any danger to nearby structures. At the same time, the conventional RC and precast concrete buildings which are in repairable damage state after an earthquake require considerable downtime to repair in addition to the repair cost to restore the full functionality. This will induce substantial seismic losses contributed by direct repair cost, and more significantly by the downtime (i.e. occupancy interruption) (Comerio & Blecher 2010).

For these reasons, the authors are working on the development of a new building system using standard precast concrete elements and steel connections which is industrialized, flexible, and demountable. The proposed building system can also be considered as a low loss building system because of quick repair/replace of damaged building components with new ones and thereby minimizing the seismic losses due to occupancy interruption. In this paper, the seismic performance of the steel connections

between precast concrete beam and column using steel angle/tubes, steel plates and threaded rods/bolts are experimentally investigated. The full details of the overall building system and details of connections between other precast elements (i.e. floor-beam, floor-floor, column-column, and column-foundation ) are outlined elsewhere (Aninthaneni & Dhakal 2014).

**2 PROPOSED STEEL CONNECTIONS TO CONNECT PRECAST BEAM AND COLUMN**

The schematic layouts of the two types of demountable precast concrete beam-column connections reported in this paper are shown in Figure 1. In Type-1 connection configuration, precast concrete beam and column are connected using stiffened steel angles and threaded rods/bolts as shown in Figure 1a. In Type-2 connection configuration, precast concrete beam and column are connected using steel angles and embedded steel plates as shown in Figure 1b. The only difference between Type-1 and 2 connection is that the Type-2 connection has additional embedded steel plates to transfer the shear force.

The precast concrete beams and columns are provided with steel ducts encased in concrete through which threaded rods are passed and bolted. The threaded rods on the column side can also be accommodated along the beam side faces if the column width is at least 75 mm more than the beam width, otherwise only on top and bottom beam faces. The bolts are pre-tensioned so that the initial resistance to the lateral load is achieved through the frictional resistance developed between the steel connection and precast concrete elements, and when the lateral load exceeds the frictional resistance, then further resistance to lateral load is developed through shearing of the bolts, and bearing resistance of concrete.

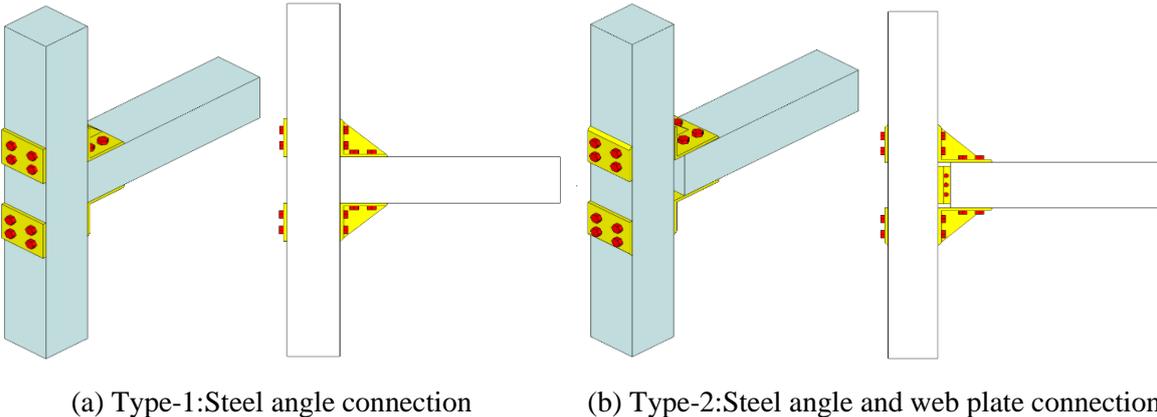


Figure 1. Tested types of steel connections for a demountable precast concrete building

**3 EXPERIMENTAL TEST SETUP, PRECAST SPECIMENS AND CONNECTION DETAILS**

**3.1 Test setup details**

The overall test setup with a specimen and instrumentation details are schematically shown in Figure 2. In the figure, “negative” and “positive” represent the directions of loading, and the notations “LT”, “LB”, “RT”, and “RB” represent the locations (e.g. right top, left bottom etc.) of spring potentiometers to record the slip between the steel connection and the precast concrete beam. An internal precast concrete beam-column sub-assembly with the proposed steel connections is subjected to quasi-static cyclic loading as per ACI loading protocol (ACI 2001), which is shown in Figure 3a. For a corner beam-column test assembly, the Figure 2 has to be visualized without one beam either left or right side of the column. The details of sub-assemblies location (i.e. internal or corner) for different tests are mentioned in Table 2. Three main aspects that will be assessed through this experimental test program are; (i) to demonstrate that emulation of the behaviour of a monolithic reinforced concrete (RC) building system can be achieved using precast concrete elements connected using standard demountable steel connections conforming to the acceptance criteria for moment resisting frames, which is shown in Figure 3b , (ii) to demonstrate the demount-ability and replace-ability aspect of the proposed building system by replacing the damaged beam with a new one of similar capacity at sub-assembly level, and (iii) to

demonstrate the upgradability aspect of the proposed building system by replacing beam of higher strength in place of lower strength beam at sub-assembly level.

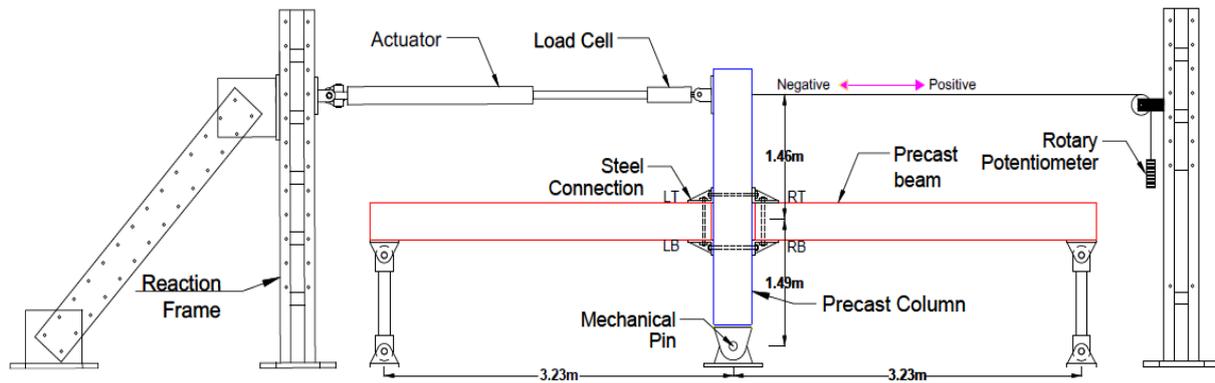
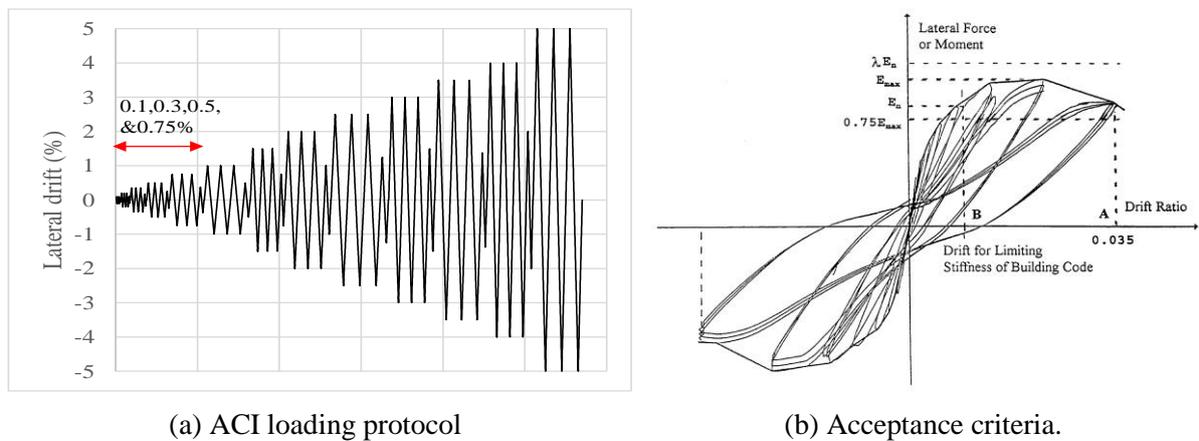


Figure 2. Experimental test setup to assess precast concrete beam-column sub-assemblies



(a) ACI loading protocol

(b) Acceptance criteria.

Figure 3. Loading protocol and parameters to assess acceptance criteria (ACI 2001)

### 3.2 Precast concrete beam and column specimen details

The material properties, cross-sectional dimensions, reinforcement details, and nominal capacities of precast specimens are reported in Table 1. The beam length of 3.23 m and column length of 2.95 m is chosen such that they approximately represent half of the length in typical frame buildings (i.e. distance between the point of contra-flexure). The precast specimens are cast with steel ducts of 50 mm diameter with 2 mm wall thickness to accommodate the threaded rods. In these tests, the column width had to be increased to 0.7 m (which is substantially more than the beam width); this was required in order to cater for different bolt configurations as shown in Figure 4 given the limitation of available hydraulic jack/torque wrench size in the laboratory. The capacity of the column is intentionally made higher than the required capacity because the same column is to be used to demonstrate the upgradability aspect with use of beam of higher capacity. In practice, the column size can be reduced conforming to building code provisions.

**Table 1. Details of precast concrete beam and column specimen's**

Details	Precast concrete beam	Precast concrete column
Dimensions (b×d)	0.35 m×0.4 m	0.7 m×0.6 m
Grade of concrete	30 MPa	30 MPa
Grade of rebar's	500 MPa	500 MPa
Longitudinal rebar details	Top & bottom: 4 Nos -25 mm diameter	16 Nos -25 mm diameter
Stirrups details	2 Leg-10 mm dia-150 mm c/c	5 Leg-10 mm dia-150 mm c/c
Nominal capacity	310 kN-m	950 kN-m

### 3.3 Steel connection configuration details

The details of the Type-1 and Type-2 connection configurations are shown in Figure 4, note that dimensions in Figure 4 are in millimetres. In Type-1 connection configuration, the precast beams are connected to precast column using stiffened steel angles of size 350×340×25 mm and 350 grade steel, and the threaded rods of 33 mm diameter and grade 8.8 steel. In Type-2 connection configuration, precast beams and column are embedded with steel plates, and are connected using steel angles used in Type-1 connection and the web plates are connected using 16 mm diameter bolts of grade 8.8. As the precast specimens' finished surface were not perfectly level and legs of steel angle are not exactly perpendicular, to have a good contact surface between the steel connection and the precast specimens, either D60 hardness natural rubber sheet of 3 mm or dental plaster or both were used, as reported in Table 2. Another reason to use rubber sheet is to prevent damage to the column because of the bearing of steel angle under lateral load. The threaded bolts are tightened using a torque wrench to the values listed in Table 2. As there is no direct way to measure the pre-tension in the bolts, the torque is converted to equivalent pre-tension using the formula  $T=0.2PD$ , where  $T$  is torque,  $P$  is pre-tension in bolts, and  $D$  is the diameter of the bolt.

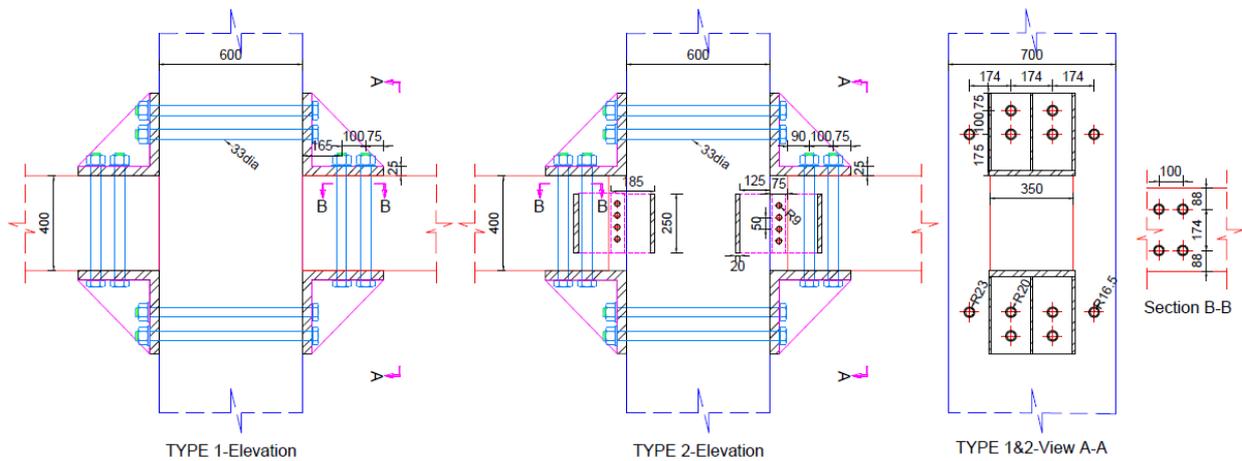


Figure 4. Type-1 and Type-2 connection configuration details

**Table 2. Sub-assembly location, connection type, pre-tension in each bolt, and contact surface**

Test No	Sub-assembly location	Steel connection type	Threaded rods/ bolts		Beam to connection contact surface	Column to connection contact surface
			Torque (T)	Pretension		
1	Corner	Type-1	2000 N-m	300 kN	Rubber sheet+dental plaster	Rubber sheet+dental plaster
2	Corner	Type-1	2250 N-m	340 kN	Dental plaster	Rubber sheet
3	Internal	Type-2	1600 N-m	240 kN	Rubber sheet	Rubber sheet
4	Corner	Type-2	2250 N-m	340 kN	Dental plaster	Rubber sheet

## 4 RESULTS AND DISCUSSIONS

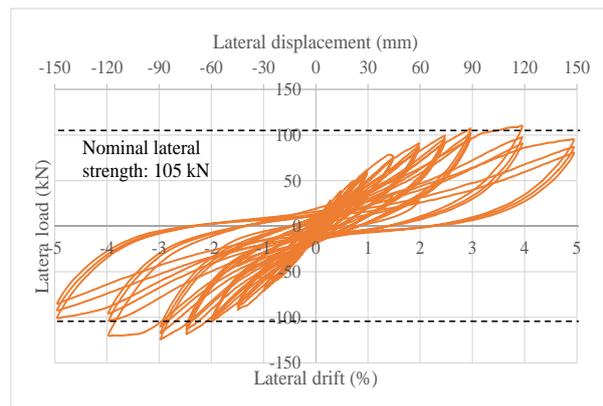
The precast concrete beam-column sub-assemblies are designed as per capacity design principles to ensure weak beam strong connection and stronger column. So, the nominal lateral capacity of the sub-assembly is limited by the capacity of the beam. As mentioned before, the nominal capacity of the beam is 310 kN-m which is equivalent to nominal lateral strength of 105 kN for the corner beam-column sub-assembly and 210 kN for the internal beam-column assembly. The experimental results of four precast concrete beam-column sub-assemblies are reported here, out of which three are corner sub-assemblies location in a frame building.

#### 4.1 Test-1: Type-1 connection with rubber and dental plaster as contact surface

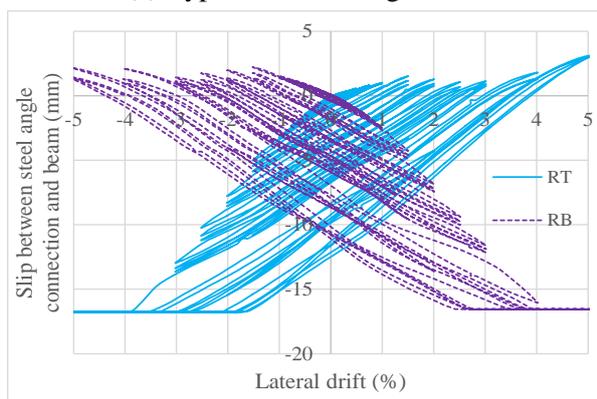
The corner beam-column sub-assembly with Type-1 connection, and rubber and dental plaster in between the connection and the precast members is shown in Figure 5a. The overall hysteresis behaviour of the sub-assembly is shown in Figure 5b, it is clear that the system is able to achieve its nominal lateral capacity of 105 kN in both directions of loading at higher lateral drift of 2.5%. The overall hysteresis loop can be classified as “pinching” type, this is because of the rubber sheet between the steel connection and the beam which makes the sub-assembly requiring less lateral load at higher drift to bring the sub-assembly to the un-deformed position. Even though each bolt is pre-tensioned to 300 kN (assuming that there is no error in converting applied torque to equivalent pre-tension), the slip between the connection and the beam cannot be avoided, which is evident from Figure 5c. The position of the connection started to shift from the initial position towards the column side in subsequent loading cycles after 1.5% lateral drift. The initial behaviour of the precast concrete sub-assembly using the Type-1 connection is similar to monolithic reinforced concrete sub-assembly under cyclic loading up-to 1.5% lateral drift. The initial damage to the sub-assembly is due to the crack formations on beam top and bottom faces, yield of longitudinal rebars, and the spread of the cracks along the beam length. Thereafter, at higher lateral drifts the slip between the connection and the beam exceeded the clearance between the bolts and the ducts, and the bolts started to bear against the steel duct which induced bearing stress (i.e. bursting stress) into the concrete resulting bearing and direct tensile split cracks, which is shown in Figure 5d. At larger drifts because of the ingress of the connection either downwards or upwards into a beam, vertical crack passing throughout the beam cross-section is observed.



(a) Type-1: Test configuration



(b) Lateral load vs displacement



(c) Slip between steel connection and beam



(d) Damage to beam at 2.5% lateral drift

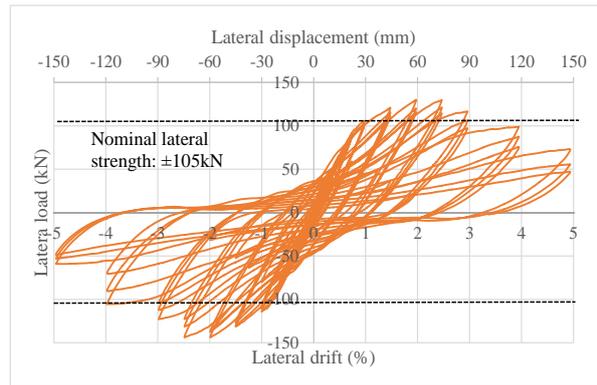
Figure 5. Experimental details and results of Test-1 with Type-1 connection

#### 4.2 Test-2: Type-1 connection with dental plaster as contact surface

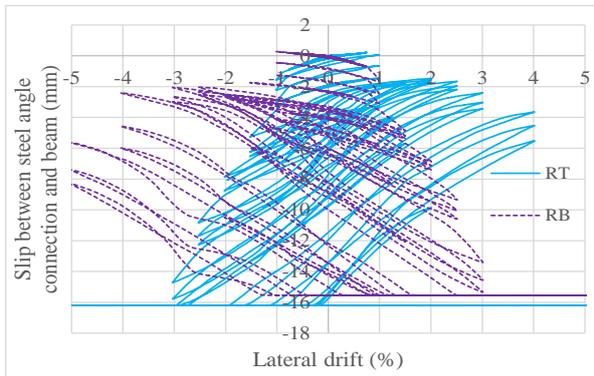
The Test-2 configuration which is shown in Figure 6a is the same as Test-1 configuration except that there is no rubber sheet in between the connection and the beam and another difference is that each bolt is torqued to 2250 N-m, which is equivalent to 340 kN. The lateral load versus lateral drift of the sub-assembly is shown in Figure 6b, it is clear that the system has reached its nominal lateral strength of 105 kN at 1% lateral drift. By comparing Figures 5b and 6b, it is obvious that the overall hysteresis of the sub-assembly with and without rubber sheet is quite different. The sub-assembly without rubber sheet is able reach its nominal capacity at a lower drift. Also at higher drifts the sub-assembly without rubber sheet suffered significant strength degradation compared to the sub-assembly with rubber sheets. The sub-assembly with rubber sheet did not undergo strength degradation up-to 4% lateral drift whereas the sub-assembly without rubber sheet suffered significant strength degradation after 3% drift. The slip between the connection and the beam is less than 2 mm and up-to 1% drift cycles the steel connection returned to initial position when the column was brought back to the un-deformed position. After 1% drift onwards when the lateral load exceeded the frictional resistance between the connection and the beam, slip gradually increased and the position of steel connection started to shift towards the column side in subsequent loading cycles, which is shown in Figure 6c. The damage to the beam is due to the spread of flexural cracks up-to 1.5% lateral drift after that the slip exceeded the clearance between the bolts and ducts, and the bolts started to bear against the ducts which induced bearing stress (i.e. bursting stress) into the concrete resulting spalling of concrete in the connection zone, which is shown in Figure 6d. At higher drift ratios, a big vertical crack was observed passing throughout the beam cross-section, this is due to the ingress of the connection either downwards or upwards.



(a) Type-1: Test configuration



(b) Lateral load vs displacement



(c) Slip between steel connection and beam



(d) Damage to beam at 2.5% lateral drift

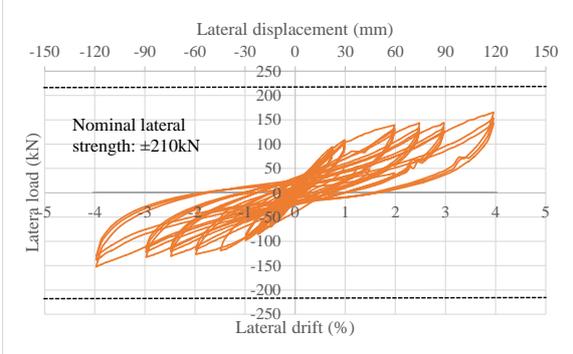
Figure 6. Experimental details and results of Test-2 with Type-1 connection

**4.3 Test-3: Type-2 connection with rubber sheet at the contact interface**

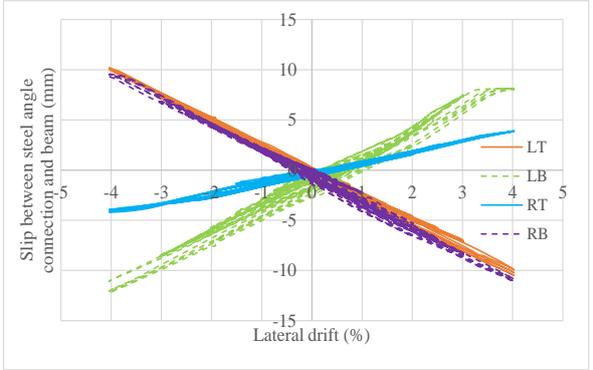
The overall test setup of internal beam-column sub-assembly with Type-2 connection is shown in Figure 7a. The sub-assembly behaviour under quasi-static loading is shown in Figure 7b in terms of lateral load and drift. It is clear that the sub-assembly with the Type-2 connection did not achieve its nominal lateral strength of 210 kN. The maximum lateral capacity that the sub-assembly could reach is only 165 kN, which is 20% less than the nominal lateral strength. Because of the rubber sheet in between the connection and the precast members, the overall hysteresis loop is close to pinching behaviour. The identified reasons for the sub-assembly with Type-2 connection not achieving its nominal lateral strength are: (i) as the slip between the connection and the beam is high at smaller lateral drifts (which is shown in Figure 7c), the bolts started to bear against the ducts which induced the bearing stress into concrete resulting in beam edge failure, (ii) to accommodate the web plate connection, a gap of 75 mm had to be provided between the beam end face and the column face; thereby leaving only 65 mm of concrete is behind the first row of ducts which caused early beam edge failure, which is shown in Figure 5d, and (iii) when the beam longitudinal rebars are stressed either in tension or compression, the bent portion of the bars towards the beam end face are forced to move outwards, because there is no beam end face protection (i.e. steel end armour) and no bursting reinforcement to protect the beam end failure, which caused early end cover spalling. Comparing Figures 5c and 7c indicates that the slip between the connection and the beam of corner and internal sub-assemblies under quasi-static loading is quite different. When internal sub-assembly is unloaded to the initial position the connection also returns to the initial position, which is not the case with the corner sub-assembly. In case of the internal sub-assembly, the connections being symmetrical with respect to the column, and connected to each other by the threaded rods on the column side, when the column is returned to the initial position, the connections on both sides of the column have to move equally and have to return to the initial position as well. In case of the corner sub-assembly when the column is drifted, because of the slip between the connection and the beam has exceeded the clearance between the bolts and the ducts, which results in change of shape of the duct from circular to oblong (i.e. oval shape), and when the column returned to the initial position, the connection will equilibrate at another position which is not the initial position.



(a) Type-2: Test configuration



(b) Lateral load vs displacement



(c) Slip between steel connection and beam



(d) Damage to beam at 2.5% lateral drift

Figure 7. Experimental details and test results of Test-3 with Type-2 connection

#### 4.4 Test-4: Type-2 connection with repaired beam and dental plaster contact surface

The aim of Test-4 is to prove that the sub-assembly with Type-2 connection in Test-3 wasn't able to achieve its nominal lateral strength because of early beam edge failure, which is because of no beam edge protection. To prove this, the beam from the sub-assembly of Test-3 which is shown in Figure 8a is repaired by armoring with steel plates on all sides of the beam end. The steel armour is anchored to the beam by welding the steel studs connecting the steel plates and longitudinal bars at beam end and side faces, and also by welding beam end face steel plate around the embedded web-plate. Thereafter, the damaged portion of the beam is filled up with Sika high early strength (HES) grout as shown in Figure 8b. The corner sub-assembly with the repaired beam and Type-2 connection is shown in Figure 9a. It is clear from Figure 9b that the sub-assembly with the beam end protection with steel armour was able to achieve its nominal lateral strength of 105 kN. Since the beam was already damaged in the previous test, the sub-assembly showed significant strength degradation after 2.5% lateral drift. The slip between the connection and the beam under quasi-static cyclic loading is similar to other corner sub-assemblies, which is shown in Figure 9c. The physical condition of the repaired beam at 2.5% drift is shown in Figure 9d.

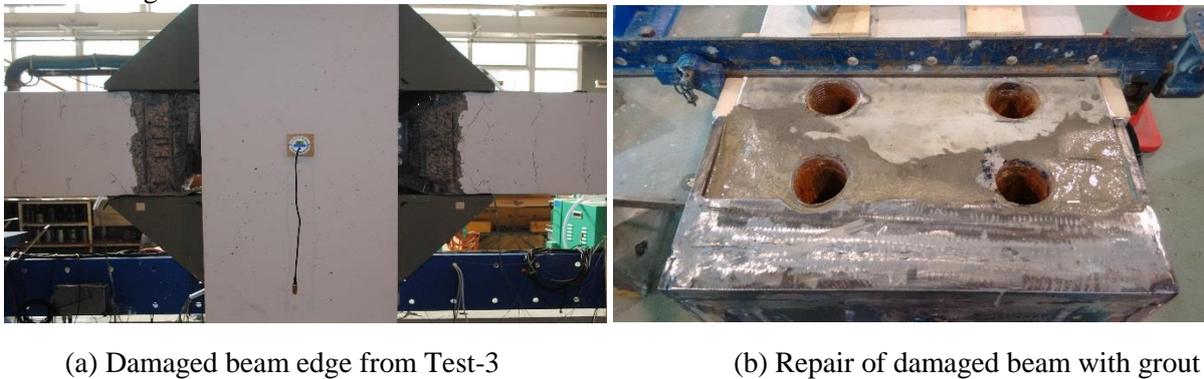


Figure 8: Damaged beam from Test-3 and repaired beam with steel armour and grout

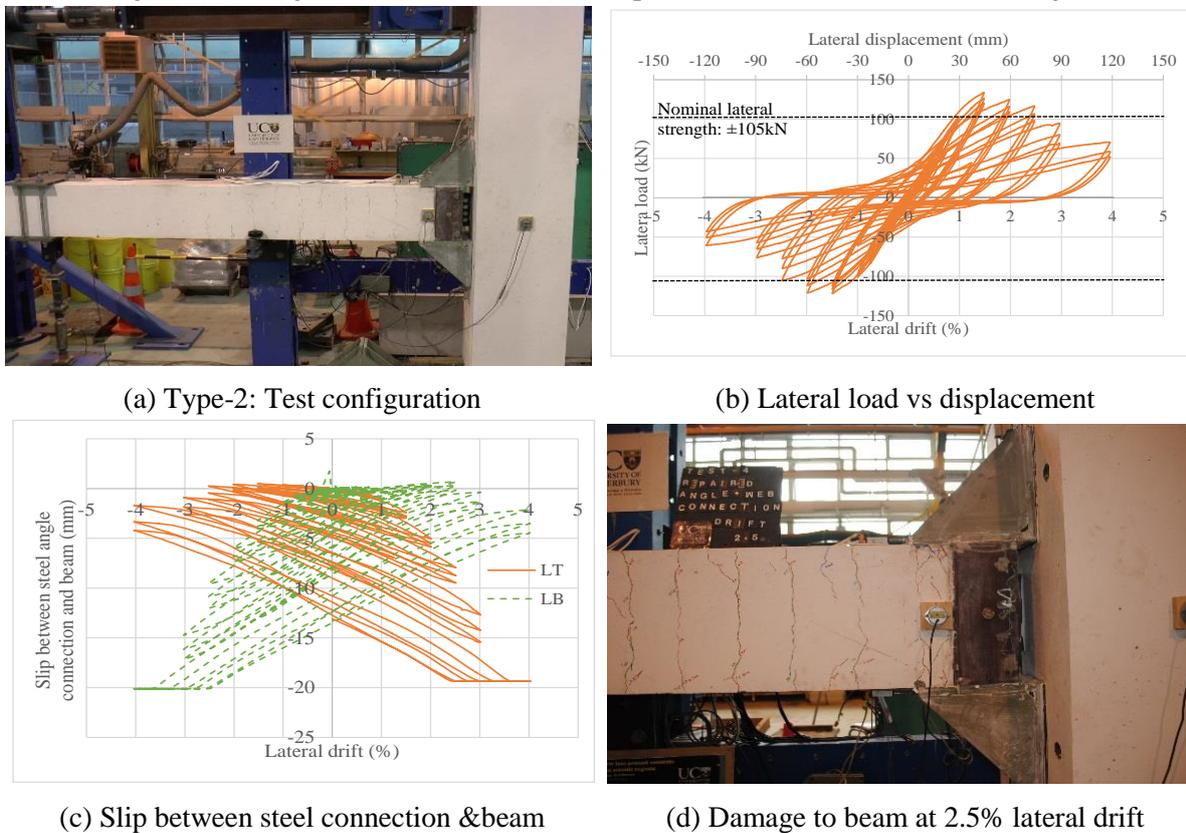


Figure 9. Experimental details and results of Test -4 with repaired beam and Type-2 connection

## 4.5 Summary of the test results

### 4.5.1 Nominal lateral strength:

The precast concrete beam-column corner sub-assemblies with Type-1 connection could achieve the lateral strength of equivalent monolithic RC beam-column sub-assembly which is 105 kN. The internal sub-assembly with Type-2 connection couldn't achieve its lateral strength of 210 kN because of early beam edge failure. It is proved by Test-4 that the sub-assembly with repaired beam with steel armour has achieved its lateral strength of 105 kN. It can be concluded that the sub-assemblies can be considered as moment resisting frames as per acceptance criteria shown in Figure 3b.

### 4.5.2 Deformation capacity:

The lateral deformation capacities of the sub-assemblies with Type-1 and Type-2 connections are similar to the monolithic RC sub-assembly behaviour. The beam-column sub-assemblies with Type-1 connection is subjected up-to 5% cyclic lateral drift, whereas the sub-assemblies with Type-2 connection is limited up-to 4% cyclic lateral drift because of significant strength and stiffness degradation. Based upon the experimental results, it can be concluded that the sub-assemblies with the Type-1 and Type-2 connections can be classified as ductile lateral structural systems.

### 4.5.3 Hysteresis behaviour:

The overall hysteresis behaviour of the precast concrete beam-column sub-assemblies with the use of the proposed demountable steel connections is very close to conventional RC frame hysteresis behaviour. The use of the rubber sheet between the steel connection and precast members has considerable impact on the lateral load versus deformation behaviour. The hysteresis loop is close to "pinching" type, thereby requiring less lateral load to bring the lateral system to un-deformed position. Without use of the rubber sheet between the steel connection and the precast members, the hysteresis loop is similar to typical monolithic RC sub-assembly behaviour.

### 4.5.4 Demount-ability and replace-ability:

One of the primary aims of the project is to assess whether the proposed steel connections between precast concrete beam and column can be considered as demountable connections so that the whole building system can also be considered as demountable. Another aspect is whether the damaged beam can be replaced with new beam of either same or higher capacity without modifying the capacity design principle of weak beam-strong column/connection. From this experimental test program, it is realized that the beams can be easily demounted and replaced without much effort at sub-assembly level. So if the whole building has to be demounted for any reason, then demounting of the beams in a building can be done from top to bottom of the building with available lifting facilities (i.e. crane or forklift). If a building is in repairable damage state after an earthquake, then the exact process of demounting of the damaged beams after an earthquake (i.e. say from 2 level) and replacing with new ones will not be the same and easy as demounting the damaged beams and replacing with new ones as done in the laboratory at sub-assembly level. Before demounting the damaged beams, all the surrounding elements (i.e. precast floors) in that level need to be shored to the bottom of the building. If the damaged beams are from the perimeter lateral resisting frame, it can be replaced with little effort and available lifting facilities, whereas if the damaged beams are from the internal gravity frames then the exact process of demounting and replacing with new ones needs more investigation.

## 4.6 Optimization and improvements of steel connections

### 4.6.1 Steel connection optimization

The main objective of the experimental tests is to prove that the emulation of behaviour of a monolithic RC frame system can be achieved using precast elements connected with demountable steel connections. As this is a concept proof test and first of its kind, the steel connections are designed with a high factor of safety so that any unexpected mode of failure in the connections are eliminated. In reality, the size of connections can be optimized. The cost of the connections can be substantially reduced by using

standard steel angles and by standardizing connections for different beam capacities.

#### 4.6.2 Possible improvements

It is realized that some improvements can be made in the connections and in the precast beams as listed below.

1. Thin steel plate can be embedded around the beams and columns in the connection zone, so that the contact surface will be more level. This will increase the contact area, and pre-tension to the bolts will induce uniform contact pressure providing better frictional resistance to the lateral loads.
2. Instead of using less number and large diameter of bolts, use of more number and smaller diameter of bolts will induce uniform contact pressure and thereby increase in frictional resistance.
3. It is realized that horizontal stirrups (i.e. bursting reinforcement) in the connection zone are effective to a certain extent in resisting the bursting stresses because of direct contact of bolts onto the steel duct and the concrete. To further improve the performance, the U-bar around the steel duct along with bursting stirrups can be used.
4. The use of U-shape longitudinal rebars as top and bottom reinforcement instead of separate bent-up rebar's as top and bottom reinforcement.

## 5 CONCLUSIONS

In this paper, details of two possible beam-column connections for a demountable concrete building system using precast elements are reported. The experimental results of the precast concrete beam-column sub-assemblies with Type-1 connection (i.e. steel angle) and Type-2 connection (i.e. steel angle and web plate) under quasi-static cyclic loading are presented. Based on the experimental results, it can be concluded that emulation of behaviour of a monolithic reinforced concrete (RC) building system can be achieved using precast concrete elements connected with steel connection Type-1 (steel angle) and Type-2 (steel angle and web plate).

With the use of the proposed steel connections to connect the precast concrete beams and column in a frame building there are many advantages when compared to conventional concrete frame buildings; (i) the building can be demounted instead of demolishing and the undamaged components can be reused, (ii) the damaged building components (i.e. mostly beams if the capacity design principles are followed in the building design) in an earthquake can be replaced with new ones in relatively quick time, thereby bringing the damaged building to the pre-earthquake stage, and significantly reducing seismic losses contributed by downtime required to repair and restore a damaged building.

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