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SEISMIC PERFORMANCE OF BEAM-COLUMN SUB-ASSEMBLIES OF A DEMOUNTABLE PRECAST CONCRETE FRAME BUILDING

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Abstract

Structural elements in conventional frame buildings are connected using “wet joints” (i.e. cast in-situ joints) to emulate the behaviour of a monolithic reinforced concrete (RC) frame building in transfer of loads to the foundation. Because of cast in-situ joints, the inherent advantage of precast building over RC building in-terms of speed of construction is limited due to the requirement of formwork and curing time to erect other precast elements. Also, because of wet joints the structural system of a conventional precast concrete building is either fully or partially monolithic in form, which requires these buildings to be demolished instead of dismantling and reuse of undamaged components. Buildings damaged to a reparable state after an earthquake require considerable downtime to repair in addition to the cost to restore its functionality. This will induce substantial seismic losses contributed by direct repair cost, and more significantly by the downtime (i.e. occupancy interruption). Also, there is limited research work carried out in the development of fully “dry and rigid (i.e. moment-resisting)” beam column connections for a precast concrete frame building. For these reasons, the authors are working on a frame building system using standard precast concrete elements and dry connections, which is quick to construct and easy to demount. The proposed building system can also be considered as a low downtime building system because of quick repair/replacement of damaged components (mostly lateral load resisting elements); thereby minimizing the seismic losses due to occupancy interruption.

In this paper, details of different steel angle connection configurations between precast concrete beams and columns which make the overall building structural system demountable are reported. The philosophy in load transfer mechanism between the beam and the column is also explained. The effect of the variable parameters of the connection like beam edge distance, fill material between the connection and the precast concrete elements, condition of ducts on the beam side (i.e. grouted or un-grouted), and bolt configuration on the column side on seismic performance of beam-column sub-assemblies is investigated. The sequence of observed modes of failure in the beam with increasing lateral drift is reported. Based on the investigated parameters, the best demountable steel angle connection configuration that emulates the structural performance of the monolithic RC frame is identified and recommended for implementation.

Keywords: Demountable; design to dismantle; deconstruction; precast concrete; dry beam column connection

1 Introduction

The seismic performance of a precast concrete frame building primarily depends on type of lateral load resisting structural system, which fall into two broad categories; either “equivalent monolithic” or “jointed” system [1, 2]. The difference between these two systems lies in the type of connection between the precast concrete beams and columns. The equivalent monolithic systems are achieved with the use of “wet joints” (i.e. cast in-situ joints) between precast concrete beams and columns either in or away from the plastic hinge region [3]. In jointed systems, the precast beams and columns are connected using un-bonded post-tensioned tendons [4]. The jointed systems do not emulate the behaviour of the monolithic systems, but the overall system can be designed as “ductile”. Also, the jointed systems offer the distinct advantage of self-centering with minimal residual deformation. There is another category of jointed system in which the precast beams and columns are connected using ductile connectors, and the behaviour is classified as “ductile” with pinching hysteresis [5].

In the past two decades, many researchers have tried to develop an equivalent monolithic frame system with the use of fully "dry joints" (i.e. similar to the connections in the steel buildings), and to the authors' knowledge this has not been very successful [6, 7]. Because of this, the structural elements in an equivalent monolithic precast concrete frame buildings are still being connected using "wet joints" (i.e. cast in-situ joints) to transfer loads to the foundations, which limits the inherent advantage of precast building construction in-terms of construction speed (i.e. due to the requirement of formwork and curing time before erecting other precast elements). The building structural system with wet joints turns into either fully or partially monolithic in form, enforcing these buildings to be demolished either at the end of the building's life span or when it is decided to construct a new building at the site or the building has suffered irreparable damage after an earthquake or for any other reasons. The demolition process of a concrete building is environmentally unfriendly and causes extensive wastage of building materials (i.e. concrete and steel). It is reported in the literature that Construction and Demolition Waste (CDW) amounts to 17% (this percentage will be much higher after 2010 and 2011 Canterbury earthquakes) and 40% of total landfill waste in New Zealand and Australia respectively [8, 9]. Demolition of concrete buildings requires great amount of energy, it consumes around 275 MJ/t and the crushing of concrete consumes another 85 MJ/t [10]. Demolition of a concrete building is usually time consuming as well, and requires careful planning to avoid any danger to nearby structures. At the same time, conventional precast concrete buildings which are in a reparable damage state after an earthquake, require considerable downtime to repair in addition to the repair cost to restore its functionality. This will induce substantial seismic losses contributed by direct repair cost, and more significantly by the downtime (i.e. occupancy interruption) [11].

The issues with a precast concrete frame building with "wet joints" can be summarized to; to a certain extent construction speed is limited, building needs to be demolished rather dismantle and reused, and damaged building components in an earthquake cannot be replaced. These issues can be addressed with a precast concrete frame building system with use of fully removable "dry joints", which makes the building demountable and replaceable at any stage. The other advantage of a demountable concrete building in seismic regions is "upgradability" of the building system with the replacement of critical elements (i.e. beams) with new elements of higher strength or addition of extra elements (i.e. steel braces) to the lateral load resisting frames. There has been limited research work carried out in the development of demountable concrete building systems. The very few efforts available in literature deal mostly with gravity loaded systems in non-seismic regions [12-14]. There is no explicit research work carried out to investigate the seismic performance of a demountable frame building system.

For these reasons, the authors are working on the development of a low to medium rise precast concrete frame building system which is industrialized, easy to erect/construct, and demountable. This building system can also be considered as a low downtime building system because of quick repair/replacement of damaged building components with new ones and thereby minimizing the seismic losses due to occupancy interruption. The building frame structural system is comprised of precast concrete elements (i.e. foundations, columns, beams, floor slabs, and non-structural wall panels) and steel connections. The structural system is similar to conventional frame building system with perimeter lateral load resisting frames and internal gravity load resisting frames, except that all precast elements are removable/replaceable. Steel braces can also be added to the lateral load resisting frames because of the steel connections between the precast concrete beams and columns. Preliminary schematic details of the overall building structural system and the connections are outlined elsewhere [15].

In this paper, details of different steel angle connection configurations between precast concrete beams and columns which make the overall building structural system demountable are reported. The seismic performance of beam column sub-assemblies with different steel angle connection configurations like beam edge distance, different types of fill material, condition of ducts on beam side (i.e. grouted or un-grouted), and the arrangement and number of bolts on the column side are experimentally investigated under quasi-static cyclic loading. The main objectives of the experimental test program are;

(i) to check if an equivalent monolithic RC frame system can be achieved using precast concrete beams and columns connected using fully removable "dry joints" (i.e. steel angle connection).

(ii) to demonstrate the demount-ability and replace-ability aspects of the proposed building system by replacing the damaged beam with a new one of similar capacity at the sub-assembly level.

2 Components details of the proposed steel angle connection

The component details of the demountable steel angle connection between precast concrete beam and column are illustrated in Fig.1. The steel angles are welded with gusset plates to increase capacity and stiffness of the connection. The precast concrete beams and columns are cast with steel ducts through which threaded rods/bolts are passed and bolted. The precast beams are provided with horizontal stirrups in the connection region to resist the shear from the threaded rods (similar to the supplement reinforcement provided around the embedded anchors in a pedestal to transfer shear). To avoid any visible projections to enhance the architecture of the frame, the backing plates can be flush into the columns. To eliminate the requirement of temporary support during the erection of the beams, the precast concrete columns can be embedded with small rebate (i.e. either steel insert or corbel).

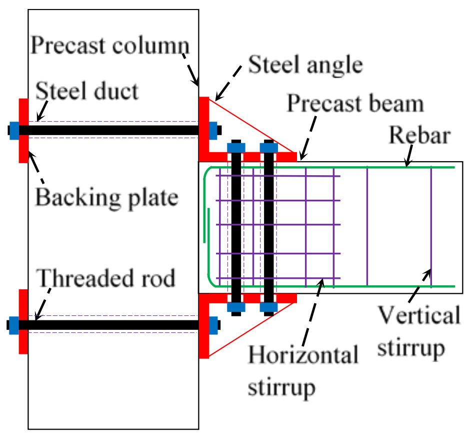
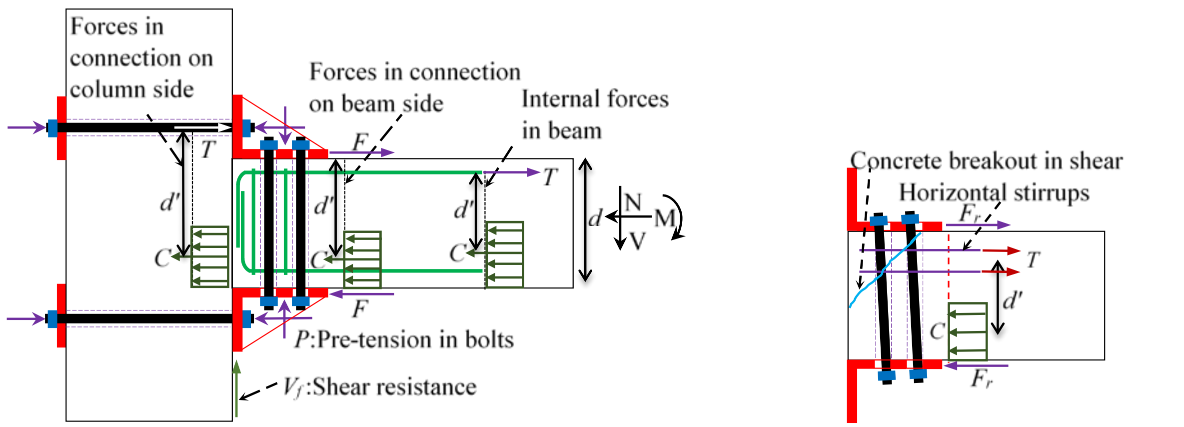


Fig.1-Components of the proposed removable steel angle connection

3 Philosophy in load transfer mechanism

The basic philosophy of the proposed steel angle connection in transfer of load between the beam and column is explained briefly herein. The bolts are pre-tensioned so that frictional resistance is developed between the steel connection and precast concrete elements. The amount of pre-tension in the bolts is decided based upon the grade of concrete, grade of fill material (if any) between the connection and the precast concrete elements, and grade of the threaded rods/bolts. The schematic and qualitative representation of transfer mechanism of the internal forces (i.e. moment (M), shear (V), and axial (N)) between the precast concrete beam and column is shown in Fig.2. In Fig.2, "P" represents the pre-tension applied to the bolts, "T" is tension force either in the longitudinal rebar's or in the bolts or in the horizontal stirrups, "C" is the compressive force either in an idealized internal concrete stress block or on the column due to bearing of the connection or on the beam due to bearing of the bolts, "F" is the frictional resistance between the connection and the beam due to bending moment, "F_r" is the reduced frictional resistance due to reduction of pre-tension in the bolts (because of loss of contact area due to spalling of concrete) and frictional coefficient (i.e. kinetic/dynamic friction coefficient is less than static friction coefficient), and "V_f" is shear resistance between the connection and the column.

When the frictional resistance is higher than the horizontal interface force due to the design actions, there is no slip between the connection and members and the resistance of the connection can be calculated using internal stress distribution shown in Fig.2a. In such cases, the behaviour of the frame is similar to a conventional precast concrete frame that uses wet joints (i.e. yielding of reinforcement and cracking of concrete). When the induced design force exceeds the frictional resistance, the behaviour of the connection comes from a combination of slip and conventional frame behaviour and the resistance of the connection can be calculated using stress distribution shown in Fig.2b. The capacity of the sub-assembly shown in Fig.2 is equal to the minimum of moment capacity of the beam or capacity of the connection (i.e. shear/bearing/tensile capacity of the bolts and steel elements of the connection) or any other possible mode of failure of the beam (concrete edge breakout in shear). Analytical equations to calculate the internal resistance of the connection and the members will be reported elsewhere.



(a) No slip between the connection and the beam (b) After slip between the connection and the beam

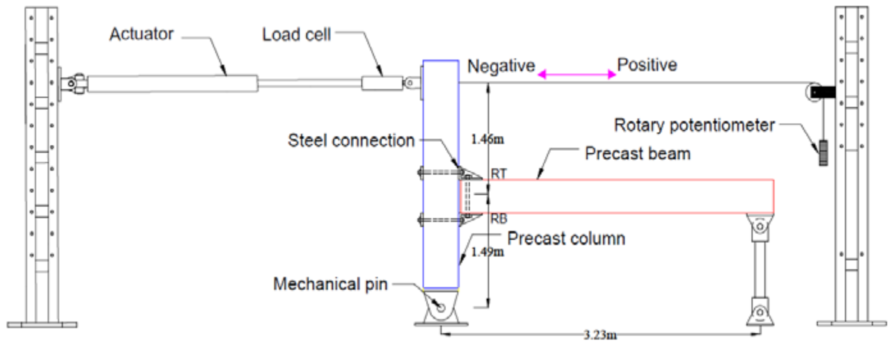
Fig.2- Load transfer mechanism between precast beam and column

One of the major governing design parameters that controls the behaviour of the proposed connection is the amount of pre-tension in the bolts which is limited to 70% of its tensile yield capacity. Also, other criteria such as the grade of concrete (compressive stress induced by the pre-tension has to be less than 25% of concrete strength), and infill material strength can also limit the pre-tension force in the bolts. The amount of pre-tension applied to the bolts connecting the connection and the beam is limited to a value such that the slip is avoided until the beam reaches 2/3rd of its yield moment capacity. The design criteria for the total amount of pre-tension in the bolts on the column side is such that any vertical slip due to the shear force induced at the ultimate stage of loading is minimal.

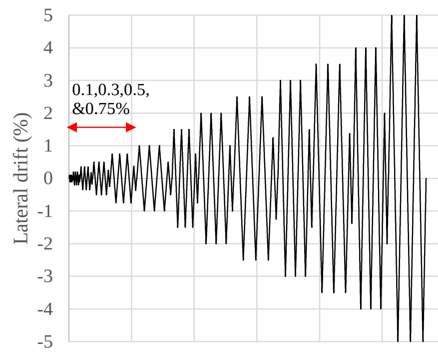
4 Overview of the experiments

4.1 Test setup details

The test setup of a corner beam-column subassembly of a frame building and instrumentation details are schematically shown in Fig.3a. In the figure, “negative” and “positive” represent the directions of loading, and the notations “RT” and “RB” represent the locations (e.g. right top, right bottom) of spring potentiometers to record the slip between the steel connection and the precast concrete members. It is important to note that the recorded slip is the actual movement of the connection from the initial position. In case of un-grouted connection, it is a combination of the actual slip until bolts do not bear against the ducts and deformation of the steel ducts (due to transfer of shear from the bolts to the ducts), whereas in grouted connection it only due to the deformation of the steel ducts. All specimens were subjected to unidirectional quasi-static cyclic loading as per ACI loading protocol [16], which is shown in Fig.3b. Beams of length 3.23 m and column of height 2.95 m are chosen such that the setup results in a loading pattern approximately representing half of the bay length and storey height of typical RC frame buildings.



(a) Experimental test setup



(b) ACI loading protocol

Fig.3- Experimental test setup to assess beam-column sub-assemblies and loading protocol

4.2 Details of precast concrete beam and column specimens

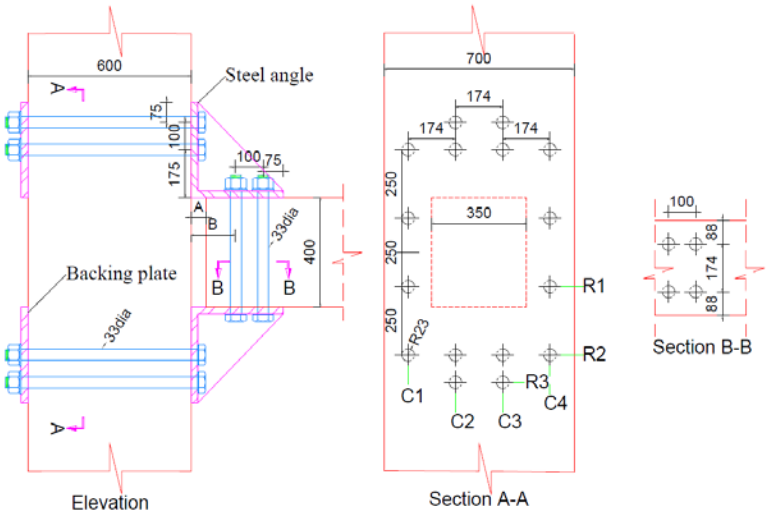
The precast concrete sub-assemblies were designed as per capacity design principle to ensure "strong column-strong connection-weak beam", so that the lateral capacity of the sub-assembly is limited by the capacity of the beam. The material properties, cross-sectional dimensions, reinforcement details, and nominal yield capacities of the precast concrete specimens are reported in Table 1. Horizontal stirrups of 10 mm diameter were spaced at 50 mm along the beam depth in the connection region to increase the beam edge breakout resistance (refer Fig.1 for the layout). The precast specimens were 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 (as shown in Fig.4), this was required in order to cater for different bolt configurations without changing the column given the limitation of available torque wrench size in the laboratory. The column was intentionally made bigger and stronger than required because the same column was used with beams of higher capacities in another series of tests (which is not reported here). In practice, the column size can be reduced conforming to building code provisions by ensuring "strong column-weak beam".

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

Details	Dimensions (b×d)	Rebar details	Stirrup details	Yield moment (kN-m)
Beam	0.35 m×0.4 m	Top & bottom: 4 Nos-25 mm	2 Leg-10 mm -150 mm c/c 5 No-2 Leg-10 mm*	319
Column	0.7 m×0.6 m	12 Nos-25 mm+4 Nos-32 mm	5 Leg-10 mm -150 mm c/c	756
Note: 40 MPa grade of concrete is considered in the calculation of yield capacity of beams, but of the actual concrete strength varied by ± 5 MPa among specimens (this has been neglected because of little effect on the yield moment capacity). The yield strength of 10 mm, and 25 mm was 586 MPa, and 517 MPa respectively. * represents the horizontal stirrups provided in the connection region of the beam (refer Fig.1 for qualitative details for the arrangement)				

4.3 Steel connection configuration details

The typical layout details of the tested connection configuration are shown in Fig.4a, note the dimensions are shown in millimeters. All the steel connections were made using 25 mm thick 350 grade (i.e.Fy=350 N/mm^2) steel plates except the gussets were made using 10 mm thick plate. The actual size of the connection, and the arrangement and number of bolts in each test can be identified from Fig.4a and Table 2. Threaded rods of 33 mm diameter and grade 8.8 steel (i.e.Fy=685 N/mm^2, and Fu=915 N/mm^2) were tightened using a torque wrench to the values reported in Table 2. As there is no direct way to measure the pre-tension in the bolts, the torque was converted to equivalent pre-tension using the approximate formula $T=0.2PD$, where T is the torque, P is the pre-tension in bolts, and *D* is the diameter of the bolt. A typical test sub assembly (here Test 6) with steel angle connection is shown in Fig.4b.



(a) Steel angle connection configuration details

(b) Test 6 setup: sub-assembly

Fig.4-Tested steel angle connection configuration details (refer along with Table 2) and actual sub-assembly test setup

The precast concrete member's surfaces had to be level to have uniform contact between the connection and precast members. As the precast specimens finished surface were not perfectly level and legs of steel angle were not exactly perpendicular, there was a gap between the connection and the members as shown in Fig.5a. To have a uniform contact surface between the steel connection plates and the precast member surfaces to transfer pre-tension force effectively, any likely gap was filled using fill material, namely; D60 hardness natural rubber sheet of 3 mm thickness, dental plaster with compressive strength of 60 MPa, and Sika grout with compressive strength of 50 MPa. The specimens with these different fill materials were tested to understand their effect on transfer of loads and hysteresis behaviour. As an example, levelled beam surface with grout (Test 6) is shown in Fig.5b.

(a) Gap: beam and connection

(b) Grout on beam surfaces

Fig.5-Gap and fill material between the connection and the beam

Table 2-Summary of total number of tests, bolts arrangement and tensions, and fill material to level specimens surface

Test reference	Gap and location of first duct		Layout of bolts on column side	Ducts on beam side	Threaded rods		Fill material to provide uniform contact area	
	A (mm)	B (mm)			Torque (N-m)	Tension (kN)	Beam to connection	Column to connection
Test 2	NG	165	R2-3&C2-3	UG	2000	300	RS+ DP	RS+ DP
Test 3	NG	165	R2-3&C2-3	UG	2250	340	DP	DP
Test 5	40	125	R2&C1-4	UG	2250	340	G	RS
Test 6	NG	125	R2&C2-3	GD	2250	340	G	RS
Note: "UG" represents ducts are un-grouted, "GD" represents ducts are grouted on beam side only, and "NG" represents no gap or gap less than 2 mm between the beam and column. Fill material notation "RS", "DP", and "G" represents rubber sheet, dental plaster, non-shrink grout respectively.								

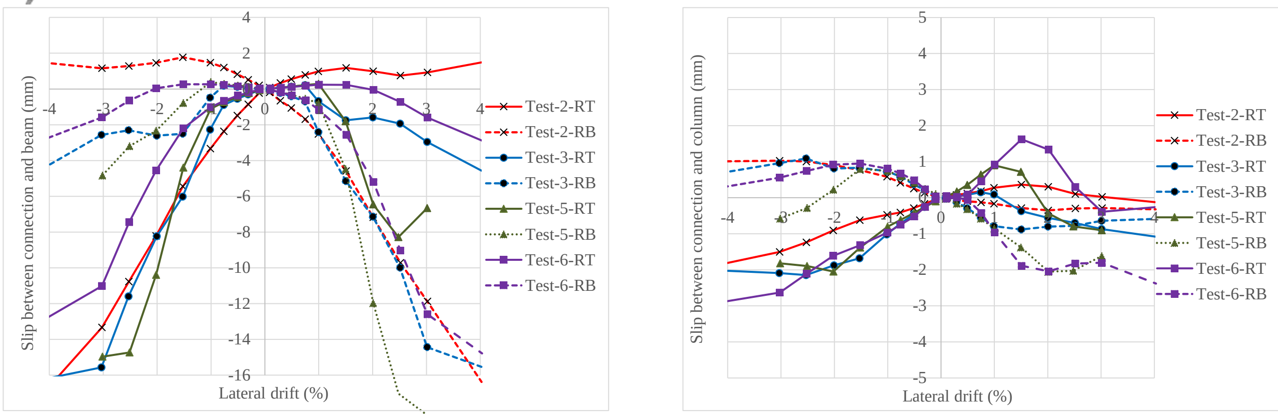
5 Results and discussions

As mentioned before, the precast concrete beams and column were designed as per capacity design principles to ensure "weak beam-strong connection-strong column". So, the nominal lateral capacity of the sub-assembly is limited by the capacity of the beam. The nominal yield capacity of the beam from the section analysis is 319 kN-m, which is equivalent to nominal lateral strength of 105 kN for a corner beam-column sub-assembly with dimensions as shown in Fig.3a. Note that in the calculation of nominal lateral strength, it is assumed that the other modes of failure, such as edge failure in the beam due to the slip between the connection and the member will not occur before the beam reaches its yield capacity.

Four experimental tests (Test 2, 3, 5, 6) were carried out to evaluate the efficiency of precast concrete sub-assemblies with different steel angle connection configurations under quasi static cyclic loading. The parameters that were varied in these tests are beam edge distance (i.e. distance from first duct location to beam edge), the gap between the beam end and column, fill material between the connection and precast concrete members, arrangement and number of bolts on column side, and the condition of the ducts on beam side. Actual details of the connection configuration in different tests can be identified using Fig.4a and Table 2.

5.1 Slip between the connection and precast concrete members

In all tests, it was observed that the cyclic behaviour of the sub-assemblies were predominantly governed by the initiation and extent of movement (i.e. slip) of the connection from the initial position. The measured slip between the connection and the precast members is shown in Fig.6. It can be observed from Fig.6a that the initiation and rate of slip depend on the type of fill material; this is because of different friction coefficients for different fill materials. It was observed that the rate of movement of the connection on beam side increases rapidly at higher lateral drift. This was due to; (i) loss of pre-tension in the bolts (because of loss of contact area), (ii) the reduction of friction coefficient (i.e. kinetic friction is less than static friction), and (iii) easy deformation of the ducts after beam edge failure. The measured slip on the column side was much lesser compared to the slip on the beam side. In Test 2, 3, and 5, four bolts were used to connect the connection to the column, whereas in Test 6 only two bolts were used (refer Fig.4a for details). It was observed that with change of arrangement and number of bolts on the column side, there was no major difference in the measured slip between the connection and the column. This was because in all tests the frictional resistance was higher than the shear force developed at the column and the connection interface.

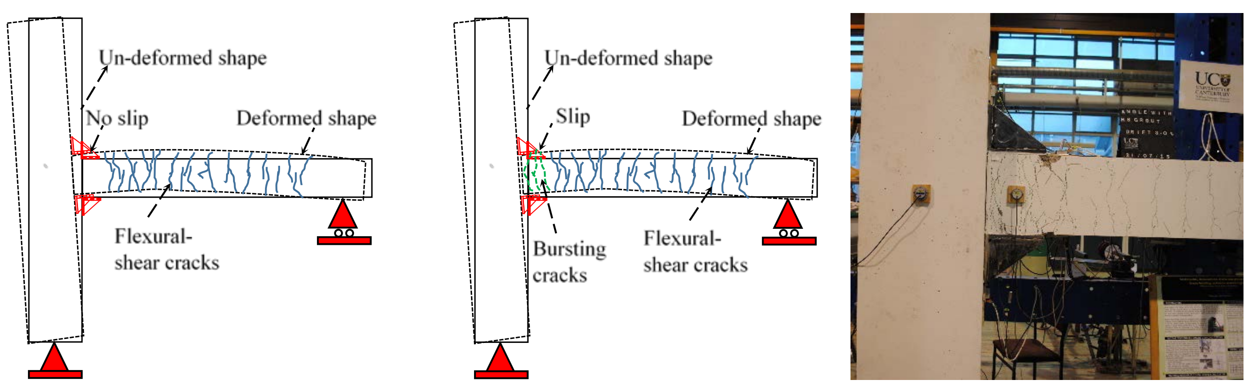


(a) Slip envelopes between the connection and the beam (b) Slip envelopes between the connection and the column

Fig.6-Slip envelopes between the connection and the precast concrete elements

5.2 Modes of failure

In all tests, similar modes of failure (damage) were observed in the beam, which is schematically shown in Fig.7a and 7b. The initial damage to the beam was due to the spread of flexural shear cracks until the slip was minimal (no contact between the bolts and the ducts) as shown in Fig.7a. Thereafter, when the slip exceeded the clearance between the bolts and ducts, the bolts started to bear against the ducts which induced bearing stress (i.e. bursting stress) into the beam edge concrete resulting in bursting cracks and spalling of concrete in the connection zone, which is shown in Fig.7b. At higher drifts, vertical shear crack in front of the connection was observed; this was due to the ingress of the connection into the beam. The typical observed damage to the beam with steel angle connection at 3.0% lateral drift (here Test 6) is shown in Fig.7c.



(a) Damage to the beam until no slip (b) Damage to the beam after slip (c) Typical observed damage to beam

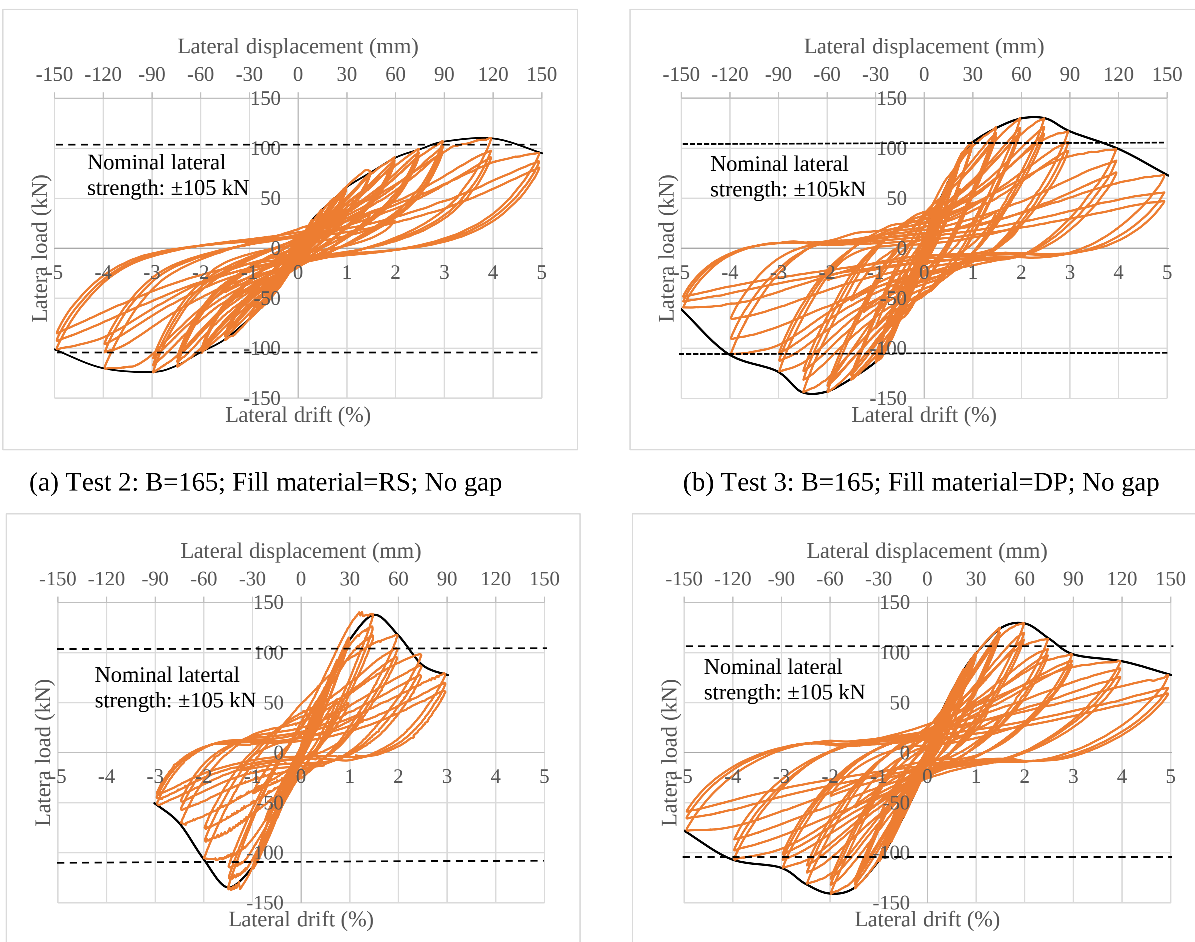
Fig.7-Modes of failure in the beam until no slip and after slip, and typical observed damage to beam at 3.0% lateral drift

5.3 General observations

The hysteresis behaviour of the sub-assemblies with different steel angle connection configurations is shown in Fig.8. It is clear from the figure that the hysteresis behaviours of the sub-assemblies were very much dependent on the varied connection parameters. In general, the overall hysteresis loop of the sub-assemblies with steel angle connection can be classified close to “pinching” type, and the extent of pinching depends on the varied parameters. The pinching behaviour is due to slip and the concentrated damage in the beam edge, which results in less energy dissipation when compared to monolithic RC frame sub-assembly. By comparing Fig.8a to 8d, the overall hysteresis behaviour of the sub-assemblies depends on the sequence of modes of failure in connection region; rebar yielding after beam edge failure or beam edge failure after rebar yielding. As can be seen in Fig.8, all sub-assemblies were able to reach their nominal yield capacity of 105 kN at around 1% lateral drift except Test 2, , in which the sub-assembly achieved its nominal yield capacity at 3% lateral drift. This delay was mainly due to early sliding of the connection, which can be observed from the slip envelope shown in Fig.7a.

5.4 Effect of varied parameters on the structural performance

The comparison of load displacement backbone envelopes of the tested sub-assemblies is shown in Fig.9. The stiffness of the Test 2 sub-assembly is smaller than that of other tested sub-assemblies. This was because in Test 2 the rubber sheet between the connection and members deformed and isolated the steel connection. By comparing load displacement backbone envelopes from Tests 3, 5, and 6, it can be concluded that dental plaster and grout have similar material and frictional properties. For practical use, the dental plaster is not suitable as it sets too quickly (its setting time is 5 minutes), and the recommended fill material, if required to be used between the connection and precast concrete members, is "non-shrink grout".



(a) Test 2: B=165; Fill material=RS; No gap

(b) Test 3: B=165; Fill material=DP; No gap

(c) Test 5: B=125; Fill material=G; Gap

(d) Test 6: B=125; Fill material=G; No gap

Fig.8- Hysteresis behaviour of the sub-assemblies with different steel angle connection configurations

The provided beam edge distance in Tests 2 and 3 was approximately "6d", whereas in Tests 5 and 6 it was "5d", where "d" is the rebar diameter. By comparing the backbone envelopes of Tests 3 and 6, it can be concluded that beam edge distance affects the strength degradation rather than the nominal lateral strength. It would have also affected the nominal lateral strength if the horizontal stirrups were not provided in the connection zone to resist the shear transfer from the bolts. By comparing the backbone envelopes of Tests 5 and 6, the sub-assembly with the gap between the beam and the column seems to have suffered higher rate of strength degradation compared to the sub-assembly without gap between the beam and the column. This was because the gap between the beam and the column allows for an early failure of beam edge¸ thereby resulting in loss of bond between the rebar and surrounding concrete, which eventually leads to strength degradation. In Tests 3 and 5, the steel ducts on the beam side were not grouted, whereas it was in Test 6. By comparing the backbone envelopes from Tests 3 and 6, it can be concluded that the condition of the ducts on beam side does not have a major effect on load-displacement behaviour as long as the amount of pre-tension in the bolts is same. This was because the ultimate mode of failure in both cases was beam edge failure.

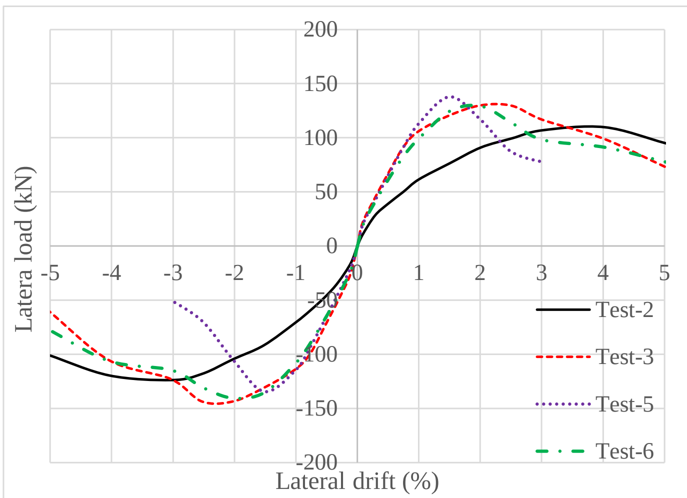


Fig.9-Comparison of load displacement backbone envelopes of tested sub-assemblies

5.5 Summary of test results and observations

It can be concluded from the experimental test results that the hysteresis behaviour of the sub-assembly with steel angle connection depends on the type of fill material between the connection and members, the gap between the beam and the column, the amount of pre-tension in the bolts (not studied in this paper), and to certain extent on the beam edge distance. The slip between the steel connection and members is crucial in deciding the hysteresis behaviour. The slip can be delayed by increasing the amount of pre-tension in the bolts or by increasing the number of bolts, thereby allowing the beam to develop its nominal capacity. The strength and stiffness degradation can be delayed by delaying the beam edge failure by increasing beam edge distance or by providing beam edge protection using steel armour plates, and eliminating any gap between the beam and column; thereby making the behaviour close to the monolithic RC frame. To summarize, the sub-assembly with steel angle connection (i.e. grouted or un-grouted) is able to achieve its hysteresis behaviour similar to the monolithic RC frame behaviour only if other modes of failure such as failure of the steel connection, beam edge failure in the beam due to the slip between the connection and the beam are delayed until longitudinal rebars in the beam yield.

5.6 Demountability and replaceability

One of the primary aims of the project is to assess whether the proposed connection between precast concrete beam and column is demountable (thereby making the whole building frame demountable). Another aspect is whether the damaged beam can be replaced with a new beam of either the same or a higher capacity without disrupting the weak beam–strong column/connection hierarchy. In the lab, the deconstruction sequence of the beam and the connection was exactly opposite of the erection sequence, which can be summarized as; (i) unbolting the connection, (ii) removing the connection components above the beam top surface, (iii) removing the damaged beam with the help of a crane, and (iv) removing the connection components below the beam surface. It was noticed that because of the sliding of the connection, the bolts on the beam side had moved from their initial position, which resulted in some difficulties in removing the bolts and steel angles. Also, for the steel angle connection configuration with grouted ducts on the beam side may require extra effort in removing the steel angles (as the holes are fully grouted). In general, from this experimental test program, it was realized that the beams can be demounted and replaced with reasonable effort at the sub-assembly level.

If the whole building has to be demounted for any reason, then demounting of the components (slabs, beams, columns) in a building can be done from the top to the bottom of the building with available lifting facilities (i.e. cranes or forklift). It is important to note that demounting of the components from a building without structural damage is not as easy as removing the beam from the sub-assembly in the lab. This is primarily due to the difficulty of access, and involves propping of the slabs and beams, and many other unexpected challenges. Again, if a building is in reparable damage state after an earthquake, then the exact process of demounting of the damaged components (say beams from level-2 of the building) after an earthquake and replacing with new ones will not be as easy as demounting the damaged beams and replacing with new ones in the laboratory at sub-assembly level. Before demounting the damaged beams, all the surrounding elements (i.e. precast floors and beams in surrounding bays) in that level need to be shored to the next level down. If the damaged components (say beams) are from the perimeter lateral resisting frame, they can be replaced using available lifting facilities without much complications, whereas if the damaged beams are from the internal gravity frames then the exact process of demounting and replacing them with new ones need to be worked out by accounting for the building orientation, configuration and accessibility. More details on the demounting process of components at building level will be reported elsewhere.

6 Conclusions

In this paper, details of different steel angle connection configurations between beam and column for a demountable concrete frame building system are reported. The experimental results of precast concrete beam-column sub-assemblies under quasi-static cyclic loading are presented. The performance of sub-assemblies with steel angle bolted connection predominantly depends on the fill material (i.e. rubber sheet, grout or dental plaster) between the connection plates and the member surfaces, beam edge distance beyond the ducts, the gap between beam end face and column face, and the amount of pre-tension in the bolts. Based on the experimental results, it can be concluded that emulation of the behaviour of a monolithic reinforced concrete (RC) frame building system can be achieved using precast concrete elements connected with the proposed steel angle connection. The recommended design parameters to be considered for the steel angle connection are: (i) minimum beam edge distance of 5 times rebar diameter, (ii) horizontal stirrups in connection region, (iii) non shrink grout as fill material (if required), (iv) no gap between the beam and column, (v) ducts on beam side can be either grouted or un-grouted, and (vi) enough amount of pre-tension in the bolts to avoid the slip of the connection before the rebar yields. From the experimental test programme, the steel angle connection between precast concrete beam and column can be considered as demountable connection, and the beam can be dismantled and replaced with a new one without much effort. Although beams were removable with steel angle connection, some minor problems were identified during the disassembly process such as; (i) difficulty in removal of misaligned threaded rods and steel angles due to sliding of the connection, and (ii) difficulty in removing the hardened grout from the holes in the steel angles.

Using the proposed steel angle connection to connect the precast concrete beams and column in a frame building results in the following advantages compared to conventional RC frame buildings; (i) the building does not need to be demolished; it can be demounted 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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