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Experimental and numerical studies of precast connection under progressive collapse scenario

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Abstract. Progressive collapse in a structure occurs when load bearing members are failed and the adjoining structural elements cannot resist the redistributed forces and fails subsequently, that leads to complete collapse of structure. Recently, construction using precast concrete technology is adopted increasingly because it offers many advantages like faster construction, less requirement of skilled labours at site, reduced formwork and scaffolding, massive production with reduced amount of construction waste, better quality and better surface finishing as compared to conventional reinforced concrete construction. Connections are the critical elements for any precast structure, because in past, major collapse of precast structure took place because of connection failure. In this study, behavior of four different precast wet connections with U shaped reinforcement bars provided at different locations is evaluated. Reduced 1/3rd scale precast beam column assemblies having two span beam and three columns with removed middle column are constructed and examined by performing experiments. The response of precast connections is compared with monolithic connection, under column removal scenario. The connection region of test specimens are filled by cast-in-place micro concrete with and without polypropylene fibers. Performance of specimen is evaluated on the basis of ultimate load carrying capacity, maximum deflection at the location of removed middle column, crack formation and failure propagation. Further, Finite element (FE) analysis is carried out for validation of experimental studies and understanding the performance of structural components. Monolithic and precast beam column assemblies are modeled using non-linear Finite Element (FE) analysis based software ABAQUS. Actual experimental conditions are simulated using appropriate boundary and loading conditions. Finite Element simulation results in terms of load versus deflection are compared with that of experimental study. The nonlinear FE analysis results shows good agreement with experimental results.

Keywords: precast concrete; wet connection; progressive collapse; U-shaped reinforcement; nonlinear finite element analysis; concrete damage plasticity

1. Introduction

One of the major causes for failures of many high profile structures took place, around the world, is extreme loading effects generated due to hurricane, flood, earthquake, explosion and terrorist attacks on buildings. This type of event imposes abnormal loading on the building structure. Generally, members of building are not able to resist this type of abnormal loading and results into failure. One of the mechanisms of failure during such event is referred to as "Progressive Collapse". Progressive collapse is a situation where local failure of a primary structural component leads to the collapse of adjoining members, which in turn leads to spread of collapse. Progressive collapse is defined as "the spread of an initial local failure from element to element resulting in the collapse of an entire structure or a disproportionately large part of it" (ASCE/SEI 7-05 2006). It is a chain reaction failure of building members to an extent disproportionate to the original localized damage (Ellingwood *et al.* 2007). Progressive collapse of building structures is initiated when one or more vertical load carrying members are seriously damaged or collapsed during any of the abnormal events. Once a local failure takes place, the building's gravity load transfers to neighboring members in the structure. If these members are not properly designed to resist and redistribute the additional load that part of the structure fails. As a result, a substantial part of the structure may collapse, causing greater damage to the structure than the initial impact. Thus it is necessary to provide sufficient redundancy, ductility and continuity, which helps the structure to find alternate paths for load distribution during undesired failure event and thus to reduce progressive collapse (GSA 2003, GSA 2013, UFC 2013).

Now a day, there is an increasing trend towards construction of buildings using precast concrete elements. Precast concrete elements is produced by casting of concrete elements in a reusable mould or form which is then cured in controlled environment. Subsequently they are transported to the construction site and placed at proper

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location. Mainly there are two types of connections used for precast construction such as dry connection and wet connection. In dry connection, the connection is carried out by anchor bolts, couplers, welding of reinforcement etc. In wet connection, small amount of cast-in-place concrete is required after connecting the reinforcement, either at beam column junction or away from junction. Precast construction is effective to achieve rapid construction within short period of time with high quality standards. Using precast construction technology, construction cost can also be reduced. Precast structures should be analyzed as a monolithic one and the joints in them are designed to take the forces, an equivalent discrete system. In addition members shall be designed for handling, erection and impact stresses that might occur during handling and erection. The main advantages of precast construction is high quality control, increased speed of construction, reduced on-site formworks & labours, good aesthetics and efficiency etc. Because of such advantages, the precast concrete construction is considered as sustainable construction alternative being adopted world-wide including India. In precast concrete construction, connections are critical elements of the structure, because in past, major collapse of precast structures took place because of connection failure. (Parastesh *et al.* 2014, Choi *et al.* 2013, Joshi *et al.* 2005). Therefore, it is important to study the performance of precast connections under column removal scenario.

Numerous experimental studies and numerical simulation have been carried out by different investigators, to evaluate progressive collapse potential of reinforced concrete and steel structures. Also, many researchers have performed experiments and numerical analysis, to study behaviour of precast connections subjected to monotonic lateral loading or reverse cyclic loading. However, studies to investigate behaviour of precast beam column assemblies under progressive collapse scenario is not reported in great detail. Authors have reported experimental and computational studies conducted on two full scale precast concrete moment resisting beam column assemblies under a column removal scenario, which was extracted from perimeter frame of 10-storey prototype building (Main *et al.* 2015, Lew *et al.* 2017, Main *et al.* 2014). Individual precast elements were connected using steel link plates, which were welded to steel angles, embedded in precast beams and steel plates, embedded in precast columns. During experimental studies, load was applied beyond the ultimate load capacities of assemblies, to study the behaviour and failure modes of precast beam column assemblies. From the results of experimental studies, it was found that precast beam column assembly designed and detailed for seismic design requirement was having higher ultimate capacity. It was also observed that, failure modes of both precast beam column assemblies were almost identical with development of flexural action and Compressive Arch Action (CAA). However, any significant development of catenary action was not seen in precast beam column assemblies. Authors have also carried out computational studies and FE models were developed using LS-DYNA software, to evaluate and compare performance of precast beam column assemblies observed during experiments (Bao *et al.* 2017). From the comparison of experimental and numerical analysis results, it is observed that FE models were able to predict behaviour and failure modes of specimens, observed during experimental studies. Further, influence of various parameters such as initial gap between precast beam and column, span length, ductility of welded reinforcement bars in heat affected zone near the welds etc., on behaviour of precast beam column assemblies were investigated.

Kang and Tan (2015) have evaluated behaviour of precast concrete beam column assemblies under progressive collapse scenario. Experiments were conducted on number of reduced (one-half) scale precast beam column assemblies. Individual precast elements were connected by cast-in-place concrete topping above beams and at beam column junctions. Effectiveness of two different types of detailing i.e. lap splicing and 90˚ bend of beam bottom longitudinal reinforcement bars and effect of longitudinal reinforcement ratio, on development of CAA and catenary action were investigated. In addition of joint detailing and longitudinal reinforcement ratio, effect of end columns size and influence of boundary conditions were also investigated (Kang and Tan 2016, Kang and Tan 2017). Authors have also investigated behaviour of precast assemblies, when engineered cementitious composites (ECC) was used as structural toppings above beams and at beam column junction, in place of cast-in-place concrete (Kang *et al.* 2015). From the results of experimental studies, it was observed that CAA and catenary actions are developed in precast assembly having both the type of detailing when rigid boundary conditions are provided. Further it was observed that, by increasing size of end columns, capacities at CAA and catenary action is significantly enhanced.

Qian *et al.* (2016) tested three dimensional (3-D) precast structures with different connection detailing, to evaluate progressive collapse performance. The effect of different types of slab to beam connections and beam to column connections were investigated by performing experiments simulating column loss scenario. From the results of experimental studies, it was observed that for precast structures with welding connection, brittle failure of weld was observed. Authors had further concluded that precast structures with adequate connection detailing showed better ductility as compared to monolithic structure, along with development of flexural action, CAA, tensile catenary action and compressive & tensile membrane actions in precast beams and slabs. Elsanadedy *et al.* (2017) have carried out an experimental studies on two reduced (one-half) scale precast specimens under middle column removal scenario. Connections between individual precast elements were provided through dowel bars projecting from column corbel and welding of cleat angles embedded in precast elements at the time of casting. Results of experimental studies indicated that, ultimate load carrying capacity and ductility of specimens with precast connections were inferior as compared to specimen with monolithic connection. Authors have also developed nonlinear Finite Element (FE) models using LS-DYNA software, to predict behaviour of precast beam column assemblies under column removal scenario. Further, authors have investigated effectiveness of bolted steel plates when provided at beam column junction of precast assembly under column removal scenario (Al-Salloum *et al.* 2018). Based on results of experimental studies, authors have found that strengthening of precast RC beam-column connections using bolted steel plates was effective in increasing the collapse load of precast assembly under progressive collapse scenario.

Qian and Li (2018a) tested scale precast beam-slab substructure under progressive collapse scenario and compared their performance with monolithic counterpart. Two different types of dry connections i.e. welded connections and pinned connections were considered for the study. Experiments were conducted on reduced 1/3rd test specimens under pushdown loading conditions to investigate effectiveness of connections to mitigate progressive collapse. From the results of experimental studies, it was concluded that, welded connection is not suitable dry connection, due to brittle failure, which prevented development of CAA despite larger initial crack strength and initial stiffness. Qian and Li (2018b) have also investigated three different reduced 1/3rd scale precast beam-column-slab substructure under progressive collapse scenario. Authors have found that dry bolted connections used in precast structure exhibits poor progressive collapse resistance. Thus, to improve the performance of precast substructure, strengthening schemes using Glass Fiber Reinforced Polymers (GFRP) strips were adopted and found improved resistance of test specimens. Further authors have investigated progressive collapse performance of one-storey, two-bay large scale frame floor subassemblies under pushdown loading conditions (Qian and Li 2019). Two precast building subassemblies were tested and their performance is compared with monolithic subassembly. Qian *et al.* (2019) evaluated progressive collapse resistance of eight different reduced scale (one-half) precast beam column sub-assemblies by performing experiments. Authors have studied behaviour of precast beam column assemblies for parameters such as types of dry connections, prestressing force in strands and location of beam column assemblies. From the results of experimental studies, different pattern of development of CAA and tensile catenary action was observed as compared to conventional RC beam column assemblies. It was found that, catenary action was initiated during the initial phase of testing, simultaneously with CAA, due to provision of unbonded post-tensioned strands.

Nimse *et al.* (2014, 2015) investigated performance of different reduced scale precast beam column connections under progressive collapse scenario. Authors have studied behavior of dry and wet precast connections provided at beam column junction by adopting different connection detailing and observed that precast connections having adequate connection detailing performs similar to that of monolithic connection. Authors have also reported performance of different dry precast connections provided at beam column junction under column removal scenario and its comparison with monolithic connection (Joshi and Patel 2016, Patel *et al.* 2015). From the results of experimental studies, it was observed that performance of precast dry connections was inferior as compared to monolithic connection under column removal scenario, which can be enhanced through adequate connection detailing between precast elements. Patoliya *et al.* (2017) carried out nonlinear FE analysis to predict behaviour of precast beam column assemblies having dry connections provided away from beam column junction under column removal scenario and showed good correlation between results of experimental studies and numerical analysis.

Progressive collapse assessment of precast concrete connections were carried out using Applied Element Method (AEM). Ehab *et al.* (2016) have developed detailed model using AEM, for different dry and wet precast connections under column removal scenario and validated with available experimental results.

The main objective of the present study is to evaluate behavior of wet connection of precast beam column assembly under column removal scenario and to compare its performance with monolithic connection. Precast beam column assemblies are constructed by providing wet connections either at the beam column junction or away from beam column junction. U-shaped reinforcement bars projecting from the either side of precast beam or column element are overlapped together and subsequently empty spaced within connection region are filled with cast-in-place micro concrete. Experiments are carried out on reduced (1/3rd) scale precast beam column assemblies by applying monotonic vertical load at the location of removed middle column.

2. Experimental Investigation

A (G+5) storey building is considered as shown in Fig. 1. Structural design of RC building is carried out according to IS: 456 (2000) with lateral loading due to earthquake and wind load. Modelling and analysis of building is carried out using ETABS software. Precast building considered for the study is assumed to be located in seismic zone-III (having moderate seismic intensities) with importance factor 1 (residential building) and soil type II (medium soil conditions), according to seismic provisions of IS: 1893 - Part-I (2002). Self-weight of structural components is considered as dead load apart from floor finish of 1.5 kN/m^2, Live load of 3 kN/m^2 and 1.5 kN/m^2 is considered on floors and roof, respectively. Brick walls of 230 mm thickness are considered at entire exterior perimeter frames. Different load combinations are considered for design of structure as suggested by Indian standard code of practice for plain and reinforced concrete IS: 456 (2002) and U. S. General Service Administration (GSA) guidelines. Structural components of test specimen are designed for the governing load case which is 1.5 (Dead Load±Earthquake Load). The connection between different precast elements is designed according to Precast Concrete Institute (PCI) handbook (2010) and British Standard (BS) 8110 (1997). Ductile detailing with response reduction factor R=5 (special moment resisting frame) is considered for design and detailing of specimens having monolithic and precast connections as per codal requirements of IS: 13920 (1993).

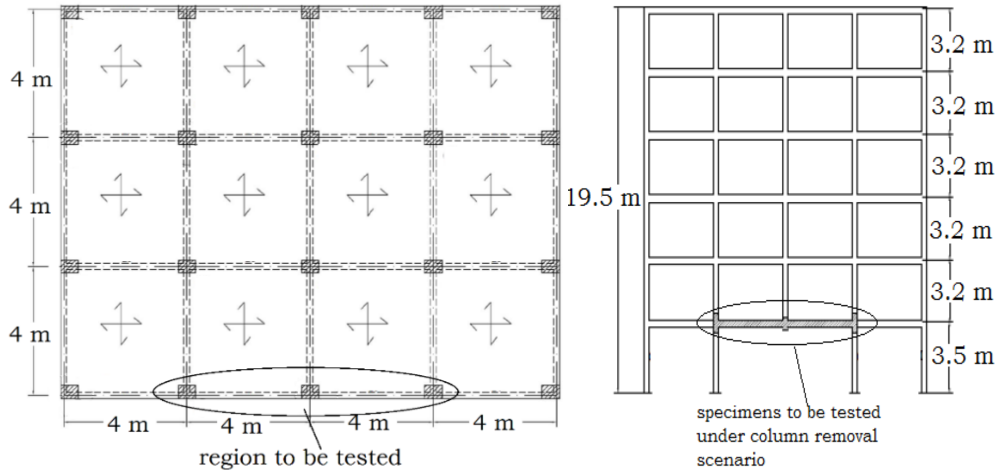


Fig. 1 Plan and elevation of building

Table 1 Geometric properties of prototype frames and scaled down specimens

Specimen	Beam Length (mm) (centre to centre)	Beam Size (mm)		Column Size (mm)		Reinforcement Ratio at Middle Junction	
		Width	Depth	Width	Depth	Top	Bottom
Prototype Frame	4000	300	400	400	400	0.84% 3-20ø	0.84% 3-20ø
Reduced (1/3rd) scale specimen	1283	100	135	135	135	0.84% 2-8ø	0.84% 2-8ø

Major challenges faced during experimental investigations of full scale building or part of building in laboratory are; capabilities of test facilities, handling of massive specimens and their accommodation in limited space of loading frame for testing. To accommodate the test specimen within the available testing facilities, 1/3rd scale is chosen for test specimens, in this study. As experiments are conducted on scaled down specimens, concrete with small size aggregates (10 mm down) is used, to avoid size effects due to material property (Yu and Tan 2013). Further, during the experimental studies, structural behaviour is mainly dominated by flexural action and Compressive Arch Action (CAA), rather than shear. From the literature survey, it is found that while testing of RC components in flexure reduced scale up to one-quarter (1/4th) is permitted. Therefore, size-effect of test specimens would not govern significantly and 1/3rd scaled test specimens can reasonably reflect the structural behaviour of full scale prototype structure in real life (Yu and Tan 2013). Many literatures have also reported experimental studies carried out on 1/3rd scaled test specimens (Su *et al*. 2009, Qian and Li 2013, Vidjeapriya and Jaya 2013, Qian and Li 2018a, Qian and Li 2018b).

For reduced (1/3rd) scale specimen, geometric dimensions of structural elements are factored by (1/3rd), without changing percentage of longitudinal reinforcement bar in bridging beams (Yu and Tan 2013). Geometrical dimensions of structural elements and percentage of longitudinal reinforcement bars, for both prototype frames and scaled down specimens are presented in Table 1. Longitudinal reinforcement detailing of beams of full scale prototype frame consists of 3 nos. of 20 mm diameter reinforcement bars at the top as well as bottom. The ratio of beam longitudinal reinforcement for prototype frame is 0.84%. In case of reduced scale test specimen, geometric dimension of beam and column is reduced to 100 mm×135 mm and 135 mm×135 mm, respectively. However, reinforcement ratio of longitudinal bars is kept similar to that of prototype frame. Reinforcement ratio of top and bottom longitudinal reinforcement bars is 0.84% for reduced scale specimen, by providing 2 nos. of 8 mm diameter reinforcement bars at top and bottom each. The clear cover for reduced scale specimens is considered as 15 mm.

This paper reports, performance of four different 1/3rd scaled precast beam column assemblies having wet connections and comparison of results with monolithic beam column assembly. Reinforcement detailing of precast test specimens is similar to that of monolithic test specimen as shown in Fig. 2. Two columns of 900 mm height and 135 mm×135 mm cross section are provided at the ends of specimen for providing sufficient anchorage for the longitudinal reinforcement, to simulate continuity of reinforcement. The beam is having cross section dimension equal to 100 mm×135 mm. Closely spaced stirrups are provided at the beam ends near the junction. Similarly, closely spaced stirrups are provided at column ends to avoid crushing of concrete.

In monolithic connection (MC), the longitudinal reinforcement of the beam consisted of two 8 mm diameter bars at top and bottom of the beam. The shear reinforcement consisted of 6 mm diameter two legged stirrups spaced at 80 mm *c/c* for beams. The column reinforcement arrangement consisted of four 10 mm diameter longitudinal bars and 6 mm diameter stirrups provided at spacing of 75 mm *c/c* which is reduced to 50 mm *c/c* near the ends of concrete column, to avoid crushing of concrete column at the ends. Reinforcement detailing for

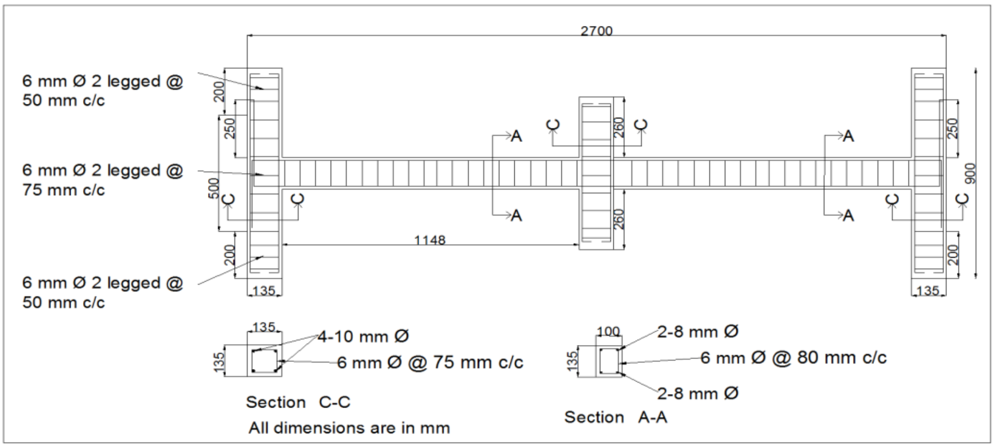


Fig. 2 Reinforcement detailing of monolithic specimen

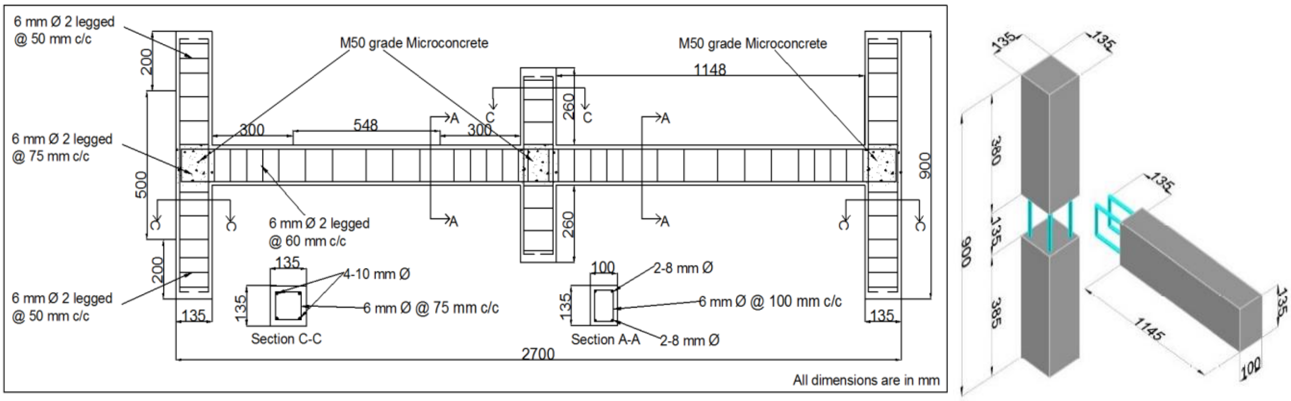


Fig. 3 Reinforcement detailing of wet connection-1

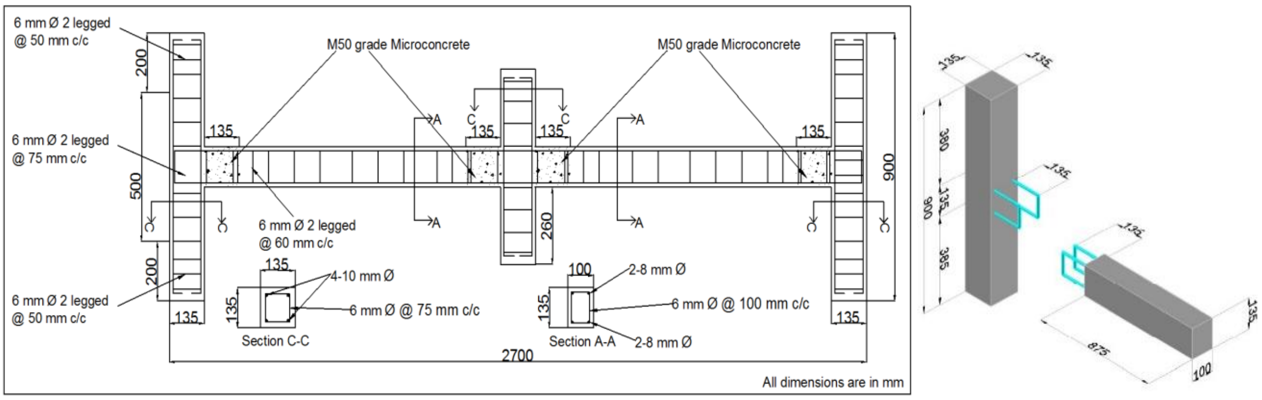
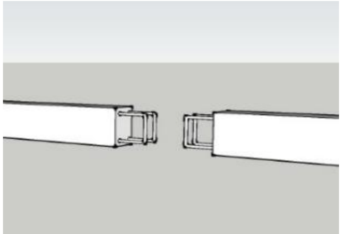


Fig. 4 Reinforcement detailing of wet connection-2

precast wet connections are almost similar to that of monolithic connection as shown in Fig. 2, except detailing at the connection region between two precast elements.

In wet connection-1, connection is provided at beam column junction within column. Column is cast separately with gap of 135 mm. The column gap is filled with cast-in-place micro concrete, after inserting U-shaped reinforcement bars projecting from the beams into that gap as shown in Fig. 3. In wet connection-2, connection is carried out at the face of column. Precast concrete column is cast separately with U-shaped reinforcement bars projecting from it. Similarly, precast concrete beam is cast separately by keeping projection of U-shaped reinforcement bars as shown in Fig. 4. These U-shaped reinforcement bars projecting from precast beam as well as column elements are overlapped together and empty spaces are, subsequently, filled with cast-in-place micro concrete, such that connection is provided at the face of the column.

In wet connection-3 & 4, connection is provided away from the beam column junction by overlapping U-shaped reinforcement bars projecting from beams on opposite sides as shown in Fig. 5. In wet connection-3, connection region having length of 300 mm and micro concrete is used to fill connection region whereas in wet connection-4, connection



* Length of Connection region
Wet connection -3 --- 300 mm
Wet connection -4 --- 100 mm

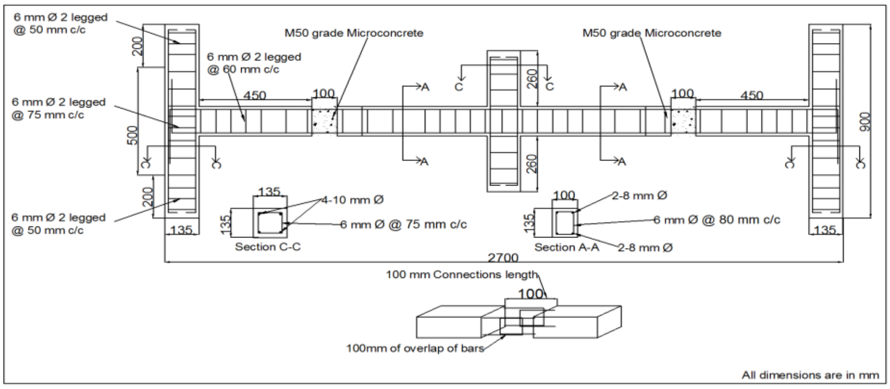
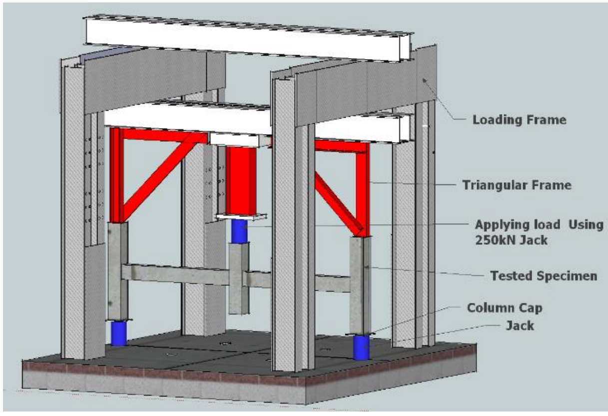


Fig. 5 Reinforcement detailing of wet connection-3 & 4

is placed over 100 mm with additional stirrup is used for better confinement of concrete between overlapped U-shaped reinforcement bars within connection zone. Also, micro concrete with addition of polypropylene fibers (1% by weight) is used to avoid brittle failure of micro concrete, as experienced during wet connection-3.

M25 grade of concrete with characteristic compressive strength as 25 N/mm^2 is used for casting of test specimens. Steel of Fe500 grade with 0.2% proof stress 500 N/mm^2 is used. Material test includes testing for compressive strength of concrete. Because of reduced scale and small cross section, aggregate of size 10 mm are used. Specific gravity of coarse aggregates (10 mm) and fine aggregate used for casting are 2.71 & 2.66 respectively. Sand of Zone II and OPC 53 grade cement is used. Mix design of concrete is carried out as per Indian standard IS: 10262 (2010). Wood Plastic Composite (WPC) formwork is used for casting of monolithic as well as all precast specimen.

Schematic diagram of test setup is shown in Fig. 6. The specimen considered in present study is a substructure frame with two span beam and three columns at first floor level of prototype building. In order to consider continuity effect of beam in specimen, reinforcement of beam are embedded into two end columns and end of columns are restrained from horizontal and vertical movement. To simulate exact condition as in prototype building during experiments, two triangle frames are fabricated to prevent horizontal movement of end columns. These triangle frames are attached with the existing loading frame that enables them to transfer the load from column to existing loading frame. End columns are restrained vertically by providing equal reactive force through hydraulic jack at bottom of it. Total four caps are fabricated, two caps are attached with triangle frame and two caps are placed on hydraulic jack, to maintain the position of column. After erection of triangle frame, bottom hydraulic jack and caps, specimen is placed in the position. Leveling of the test specimen is ensured to avoid development of cracks due to level difference. The gap between hydraulic jack and top of the specimen is filled with spacer plates. The load is applied at the top of the removed middle column with the help of hydraulic jack of capacity 250 kN till the complete failure of specimen takes place. LVDT are used to measure vertical displacement of the middle column and the middle span of beam which is attached with Data Logger.



(a) Schematic diagram

(b) Actual test setup

Fig. 6 Schematic diagram and actual test setup

Column removal scenario in real life situation is dynamic in nature. However, the guidelines such as General Service Administration (GSA) specifies threat independent approach for progressive collapse resistant design of building by considering removal of critical elements. As per guidelines, static load combination of 2(DL+0.25LL) is considered while designing building for progressive collapse resistance. Factor '2' accounts for dynamic condition of loading. On similar basis, static loading condition is considered for the experimental study representing threat independent approach of progressive collapse. Reinforcement detailing of specimen is also based on design specifications given by GSA guidelines.

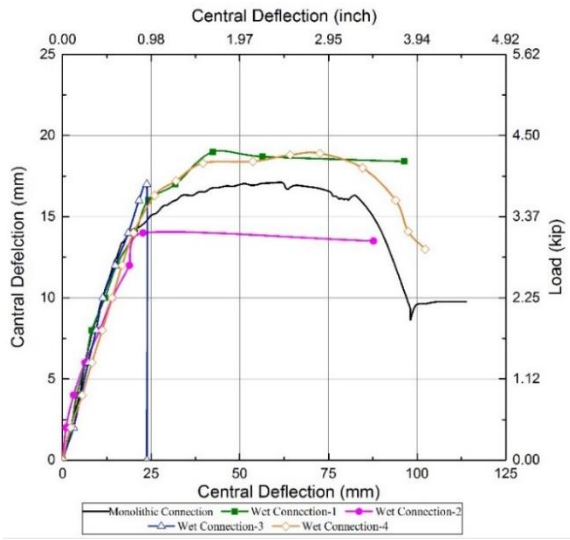


Fig. 7 Load versus central deflection for test specimen

3. Experimental results

For monolithic connection (MC), the minor cracks are observed after load of 10 kN at end beam column junction. These small cracks are initiated at the top surface of beam which was subjected to tension and further propagated in downward direction. As any significant major cracks are not developed till load of 13 kN, load versus central deflection curve is essentially linear up to that point. Upon further increase in load, tension cracks are also developed at right side of middle beam column junction, at the bottom surface of beam at a load of 14 kN. As cracks are developed on right side of middle junction, deflection of beam towards that side is slightly more as compared deflection recorded on the other side. More number of flexural tension cracks are developed on the top surface near beam column junction at both extreme ends along with widening of minor cracks within load range of 14 kN to 17 kN. As load increases, these cracks are further propagated till complete failure of test specimen. Hinge formation by means of concrete crushing is also observed at the top surface near middle junction and bottom surface at the extreme beam column junction at ends. Ultimate load carrying capacity of monolithic test specimen is observed as 17.09 kN and after this point specimen is not capable to resist further load but deflection is still continued, as bottom longitudinal reinforcements are contributing in resisting axial tensile force developed in the beam. Rupture of bottom longitudinal reinforcement bar at middle beam column junction is observed at a deflection around 85 mm to 90 mm with corresponding load value of 14 kN. Rupture of bottom longitudinal reinforcement bar results into sudden drop of load which is seen from Fig. 7. Rupture of longitudinal reinforcement bars near the middle beam column junction as shown in Fig. 8(a), which indicates contribution of bottom reinforcement bars in resisting axial tensile force developed within the beam and initiation of catenary action. The load versus vertical deflection at the location of removed middle column for all test specimens is shown in Fig. 7.

Fig. 8(a) Failure pattern of monolithic connection

Fig. 8(b) Failure pattern of precast wet connection-1

For wet connection-1 from the curve, it is seen that, response of load versus deflection is linear up to load of 16 kN, as any major cracks are not observed till this point. At load of 16 kN, diagonal shear cracks are developed within connection region at extreme beam column junctions as shown in Fig. 8(b). After this point, stiffness of specimen is decreased with increase in load. Upon further load application, stiffness of specimen is reduced significantly, as connection loses fixity and cracks are propagated within connection regions. The diagonal shear cracks developed within connection region due to lack of stirrups provided at beam column junction. As load increases further, cracks formed in micro concrete gets widen and also concrete crushing is observed at bottom of end column. Maximum load resistance capacity for this specimen is observed as 19 kN, with corresponding deflection equals to 42.5 mm. Also de-bonding between precast concrete of beam and cast-in-place micro concrete at beam column junction is also observed. However, after this point, specimen continues to deflect and deforms up to 96.4 mm, as reinforcement bars are contributing in resistance of axial tensile forces developed within the specimen, without resisting any further load.

For wet connection-2, the ultimate load resistance capacity is observed as 14 kN, with corresponding deflection equals to 22.7 mm. However, specimen undergoes deflection upto 87.7 mm, without increase in load resistance. It clearly indicates that, after load of 14 kN, stiffness of specimen remains constant, as it is not contributing in load resistance, but deflection of the specimen is still increasing. During the initial phase of loading i.e., up to load of 6 kN, specimen behaves as fixed beam with corresponding central deflection equals to 6.4 mm, in symmetrical manner. At a load of 6 kN, shear cracks

Fig. 8(c) Failure pattern of precast wet connection-2

are initiated near right side of middle beam column junction as shown in Fig. 8(c) and connections starts loosing fixity. At a load of 14 kN, micro concrete at connection region is failed near extreme beam column junction on right side, due to expansion and widening of shear crack. Load resisting capacity of specimen could be enhanced by providing stirrups within connection region, which provide better confinement to micro concrete. After this point, specimen could not resist any further load, but deflection is still continued as reinforcement bars are contributing in resisting axial tensile forces developed within the specimen. During later phase of loading, specimen deforms in asymmetric manner and larger deflection is observed on right side of middle beam column junction, as more cracks are developed within connection on that side.

The ultimate load carrying capacity for wet connection-3 is observed as 17 kN with corresponding deflection equals to 23.8 mm. The load versus deflection response of the specimen is completely linear till failure load. First crack is observed at a load of 6 kN around extreme beam column junctions. However, specimen fails from the connection region and any significant failure is not observed at beam column junctions. Neither tension cracks are developed nor are concrete crushing and hinges formed at beam column junctions, unlike other precast connections. Therefore, any significant reduction in stiffness is not observed from the load versus deflection curve, as specimen could not undergo large deformations and suddenly collapsed. At load of 11 kN, cracks are initiated in micro concrete within connection region. This cracks are propagated further in upwards direction, as load increases and suddenly at load of 17 kN entire cruciform shape of precast element is collapsed due to brittle failure of micro concrete as shown in Fig. 8(d). It is clearly observed that, forces could not transferred from one precast element to another precast element and as results it fails from connection region. This type of abrupt failure could be avoided by providing additional stirrup within connection region, around overlapped portion of U-shaped reinforcement bars, which could give better confinement. Further, brittle failure of micro concrete could be avoided by using fibres in the mix, which contributes to arrest propagation of cracks.

A curve of load versus central deflection at the location of removed middle column is linear up to load of 14 kN, for wet connection-4 as shown in Fig. 7. After this point

Fig. 8(d) Failure pattern of precast wet connection-3

Fig. 8(e) Failure pattern of precast wet connection-4

stiffness of the specimen decreased slightly, as widening of tension cracks formed near beam column junction are started. As load increases further, stiffness of specimen decreased rapidly, due to propagation of flexural tension cracks and concrete crushing in compression zone at beam column junctions. At a load of 18.9 kN, with corresponding deflection equals to 72.6 mm, fracture of one of the top longitudinal reinforcement bar is observed near middle beam column junction, due to which load is suddenly dropped, which is evident from the curve of load versus deflection. After this point, specimen could not resistance further load and maximum load resistance capacity of specimen is observed as 18.9 kN. However, deflection of specimen is still continued and maximum deflection is recorded as 102.3 mm, where fracture of another top longitudinal reinforcement bar is observed near middle beam column junction, which indicates initiation of catenary action. Fracture of longitudinal reinforcement bars are occurred mainly due to development of axial tensile forces in the specimen, at higher value of deformation.

Failure pattern of precast wet connection-4 revealed that, it behaves exactly similar to that of monolithic connection. Flexure tension cracks are developed within tension zone near beam column junction and perfect hinges are formed by means of concrete crushing within compression zone near beam column junction as shown in Fig. 8(e). Any significant failure is not observed at connection region, which indicates that forces are effectively transferred from one precast element to other precast elements through connection. Any major failure and crack development is not observed within connection region, as experienced in wet connection-3, which indicates that additional stirrups provided within connection region enhances its performance. Also, contribution of

Table 2 Comparison of maximum load and deflection

Specimen	Maximum load (kN)	Ratio of Maximum load (Precast connection to monolithic connection)	Deflection corresponding to maximum load (mm)	Ratio of deflection at maximum load (Precast connection to monolithic connection)	Maximum Deflection (mm)	Ratio of maximum deflection (Precast to monolithic connection)
Monolithic Connection	17.09	-	58.30	-	114.00	-
Precast Wet Connection-1	19.00	1.11	42.50	0.73	96.40	0.85
Precast Wet Connection-2	14.00	0.82	22.70	0.39	87.70	0.77
Precast Wet Connection-3	17.00	0.99	23.80	0.41	collapse of specimen	-
Precast Wet Connection-4	18.90	1.11	72.60	1.25	102.30	0.90

Table 3 Summarization of failure modes observed in test specimens

Test Specimen	Near Middle Junction	Near End Junctions
Monolithic Connection	Flexural tension cracks at bottom surface of left side beam, concrete crushing at top surface, Rupture of bottom longitudinal reinforcement bar on left side of middle beam column junction	Flexural tension cracks at top surface of beam, concrete crushing at bottom surface
Precast Wet Connection-1	Flexural tension crack at bottom surface of beam on right side, failure at interface of cast-in-place micro concrete and normal concrete	Flexural tension crack at top surface of beam, diagonal shear cracks within beam column junction
Precast Wet Connection-2	Diagonal shear crack within connection region on right side, crushing of concrete on top surface	Diagonal shear cracks within connection region
Precast Wet Connection-3	For precast wet connection-3, any significant failure near middle or end beam column junctions is not observed, as specimen collapsed due to brittle failure of micro concrete	
Precast Wet Connection-4	Flexural tension cracks at bottom surface of left side beam, concrete crushing at top surface, Rupture of bottom longitudinal reinforcement bar on left side of middle beam column junction	Flexural tension cracks at top surface of beam, concrete crushing at bottom surface

polypropylene fibers in arresting crack initiation and further propagation is observed. However, a minor crack is observed at connection region post peak loading.

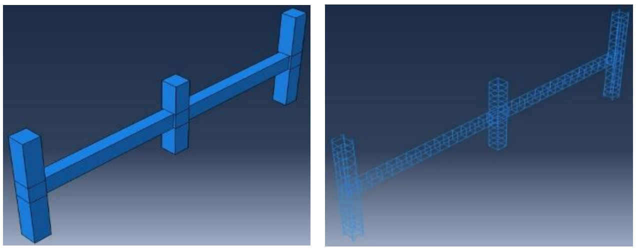
Comparison of maximum load carrying capacity, deflection corresponding maximum load and maximum deflection at which load is stopped and specimen is removed from the test set-up, for all the specimens considered for the study, is presented in Table 2.

From the graph of load versus central vertical deflection and failure pattern of the test specimens, it is observed that specimens undergoes different structural mechanism such as flexural action and Compressive Arch Action (CAA). Flexural action is observed in the specimens, during the initial phase of loading, when specimens behaves as fixed beam with almost symmetrical deflection of beams on both the sides of middle beam column junction. However, during later phase of loading, rotation is observed at middle column stub, as any rotational restrain is not provided. Subsequently, CAA is observed in the test specimens, upon further increasing the load. As, CAA has attained its capacity, vertical load applied at the location of removed middle column starts decreasing with increase in central vertical deflection, because of material and geometrical nonlinearities.

For monolithic connection and precast wet connection-4, rupture of longitudinal reinforcement bars are observed after attaining peak load at CAA, which indicates that process of development of catenary action is started. However, catenary action is not developed to its full extent, which is reflected from failure pattern of test specimens. For other precast wet connections, cracks are mainly initiated at the interface between the region filled with cast-in-place micro concrete and normal concrete and further propagated within the connection region. Failure at the interface reveals loss of bond between micro concrete and normal concrete after attaining peak load. The behaviour of precast wet connections considered for the study can be further enhanced by achieving proper bonding between two different materials. After attaining capacity at CAA, wet connections are not able to resist further load, but deflection is still continued in the specimen. As load increases, after peak loading, diagonal shear cracks developed within connection region gets widened and connection gradually loses fixity. Therefore, further application of load is stopped and specimen is removed from the test setup. As a result, any cracks are not observed within beam length away from the junction, which indicates that catenary action has not been developed fully in precast connections provided at beam column junction. Smaller size of test specimen and support conditions are also governing factors which prevented significant development of catenary action.

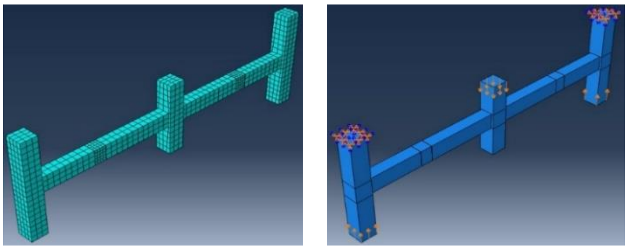
Primary failure modes for all the test specimens observed during experimental studies are summarized in Table 3.

In this study, progressive collapse resistance of monolithic and precast beam column assemblies are evaluated without considering effect of existence of slabs or transverse beams. Typical test specimen, considered for the



(a) Modelling of concrete (b) Modelling of steel reinforcement

Fig. 9 Modelling of concrete, micro concrete and steel reinforcement for monolithic connection



(a) Meshing of specimen (b) Boundary condition of specimen

Fig. 10 Meshing and boundary condition for wet connection-4

study is having two span beam and three columns with removed middle column, essentially in one plane only. However, effect of slab or beam in transverse direction will provide lateral restrain to longitudinal beam and would certainly result into improved progressive collapse resistance of beam column substructure. Provision of slab or transverse beams would increase ultimate load carrying capacity of the test specimen. Also, it would play an important role in developing tensile catenary action and tensile & compressive membrane action in addition to flexural action and CAA.

Though, typical test specimen is having only two span beam, continuity is considered partially through providing adequate anchorage of longitudinal reinforcement bars of beam into column. However, results of present experimental studies exhibits that, any significant catenary action is not developed. Provision of short beam extension projecting from ends columns allows continuity of longitudinal reinforcement bars of beam. In such case, horizontal restraint applied to beam ends, would change failure pattern of the test specimen and likely to allow development of tensile catenary action to its full extent, which is effective in mitigating progressive collapse.

4. Numerical simulation

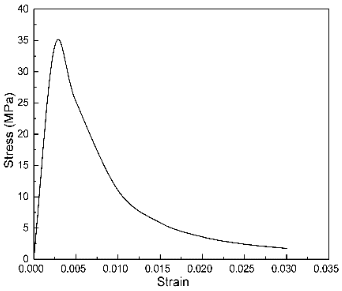
Nonlinear Finite Element (FE) analysis of monolithic and precast beam column assemblies under column removal scenario are carried out using ABAQUS software. The results of numerical simulation are compared with experimental results. Fig. 9(a) and Fig. 9(b) shows typical FE model of concrete portion and steel reinforcement bars of beam column assemblies. Concrete is modelled using first order 8-node linear brick elements with reduced integration (C3D8R) elements, while the steel reinforcement bars are modelled using 2-node linear 3-D truss elements (T3D2) elements. Typical meshing applied to beam column assembly is shown in Fig. 10(a), while Fig. 10(b) shows boundary conditions considered for FE analysis. Column base and column top are modelled by restricting the translational degrees of freedom in all the three directions, to simulate the actual boundary conditions considered during experimental studies. Load is applied at the top of the removed middle column in form of displacement increments in vertical downward direction. To simulate the force applied by the hydraulic jack at the bottom of end columns during experiments, a small amount of displacement in vertical upward direction is also specified at the base of both the end columns.

FE model of monolithic beam column assembly consists of total of 2580 elements, out of which concrete is having 888 elements and reinforcement bars are having 1692 elements. Mesh size for concrete elements including micro concrete and steel reinforcement bars varies in range of 25 mm to 50 mm. FE models for precast wet connection-1 and precast wet connection-4 is having total of 2753 and 2676 elements, respectively. A mesh convergence study is carried out before finalizing mesh size. Several alternatives for different mesh density as well as types of elements are considered and analysis results in terms of load-deflection are compared. Based on convergence of analysis results, mesh size is chosen which exhibits reasonably close agreement between results of numerical analysis and experimental studies. The interface between inserted steel reinforcement bars and concrete is modelled using constraint “embedded region” available in ABAQUS by assuming perfect bond between concrete and steel reinforcement bars. Surface to surface interaction with friction coefficient of 0.5 is considered for the contact between normal concrete and cast in-place micro concrete.

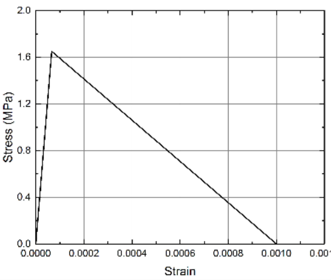
For linear properties of concrete of M25 grade, Modulus of Elasticity is considered as $5000\sqrt{f_{ck}}$ as specified by Indian standard (IS: 456, 2000), where f_{ck} is average characteristic compressive strength of cubes having size 150 mm×150 mm×150 mm. Modulus of elasticity ‘E’ for steel is considered as 2×10^5 N/mm^2. Poisson’s ratios are considered as 0.2 and 0.3 for concrete and steel, respectively. During experimental studies, mainly two types of failures have been observed i.e., concrete crushing at compression regions and development of flexural cracks at tension regions. Therefore, in this study, continuum based concrete damage plasticity (CDP) model is used to capture nonlinear behaviour of concrete, which assumes compressive crushing and tensile cracking as two main failure mechanism. The CDP model requires parameters such as dilation angle, flow potential eccentricity, viscosity parameter, the ratio of initial biaxial compressive yield stress to initial uniaxial compressive yield stress (f_{bo}/f_{co}) and the ratio of the second stress invariant on the tensile meridian to compressive meridian at initial yield (K). For the present study, these parameters are considered as commonly assumed values widely available in literature (Dere 2016, Kumar and Patel 2016, Sumer and Akta 2015,

Table 4 Parameters considered for CDP Model

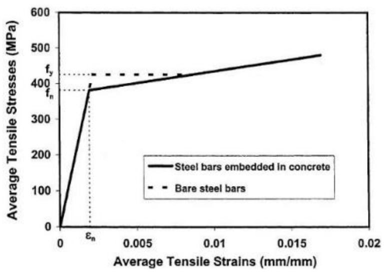
Dilation Angle	Flow potential eccentricity	f_{bo}/f_{co}	K	Viscosity Parameter
37.00	0.10	1.16	0.67	0.00



(a) Concrete under uniaxial compression



(b) Tension stiffening curve for concrete



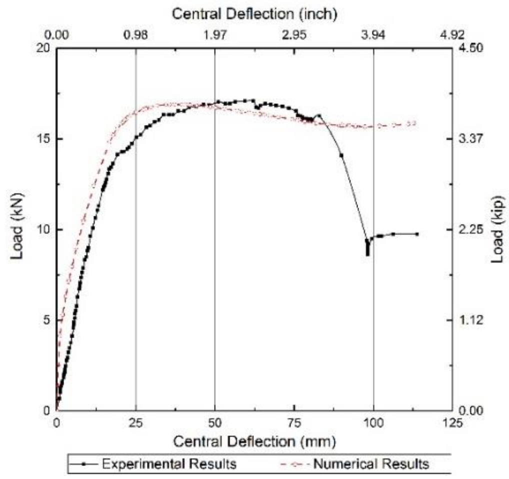
(c) Stress-strain relationship for steel

Fig. 11 Material properties of concrete and steel

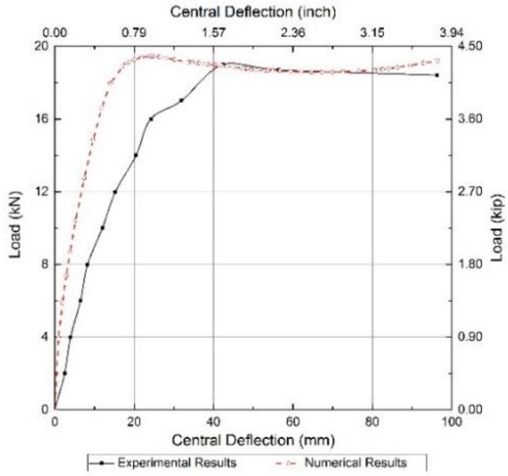
Singh *et al*. 2014). Table 4 shows values of different parameters considered in this study, for CDP model.

The CDP model also requires stress-strain relationship of concrete under uniaxial compression and uniaxial tension along with stress-strain relationship of steel. Stress-strain relationship of concrete under uniaxial compression is considered using model proposed by Saenz (1964) and Obaidat *et al*. (2010), which is shown in Fig. 11(a). Stress-strain relationship for concrete under uniaxial tension is assumed to be linear up to uniaxial tensile strength and beyond that it is derived as mentioned by Hu *et al*. (2010). Beyond uniaxial tensile strength, a simple descending line is used to incorporate tension stiffening phenomena as shown in Fig. 11(b). The default value of strain at which the tension stiffening stress is reduced to zero is taken as 0.001. The stress-strain relationship for steel reinforcement bars with yielding stress equals to 500 N/mm^2 is presented in Fig. 11(c), which is obtained based on mathematical formulation discussed by Wang and Hsu (2001).

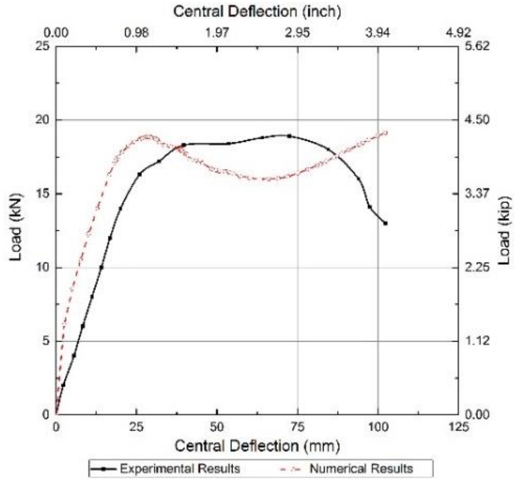
The curve of load versus central deflection for monolithic beam column assembly is closely matches up to



(a) Monolithic specimen



(b) Precast wet connection-1



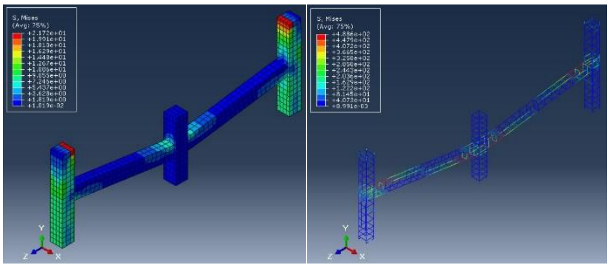
(c) Precast wet connection-4

Fig. 12 Comparison of experimental and numerical results

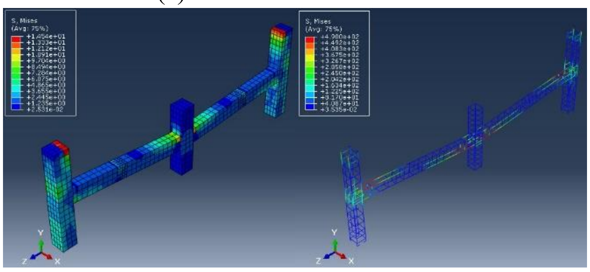
peak load, as shown in Fig. 12(a). However, difference between results, increased during later phase beyond peak load. One of the reason for this difference is assumption of perfect bond between concrete and steel reinforcement bars which is applicable up to lower strain values. During experimental studies, at large deformation and corresponding higher strain values, bond slip between

Table 5 Comparison of experimental and numerical results

Specimen	Maximum Load (kN)			Deflection corresponding to maximum load (mm)		
	Experimental	Numerical Simulation	% difference	Experimental	Numerical Simulation	% difference
Monolithic Connection	17.09	16.95	0.82%	62.05	38.16	38.5%
Precast Wet Connection-1	19.00	19.46	2.42%	42.50	24.19	43.08%
Precast Wet Connection-4	18.90	18.81	0.48%	72.60	28.73	60.42%

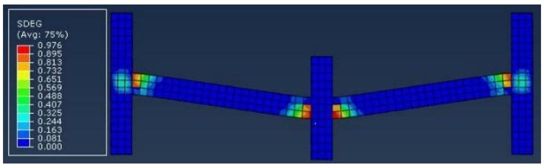


(a) Monolithic connection

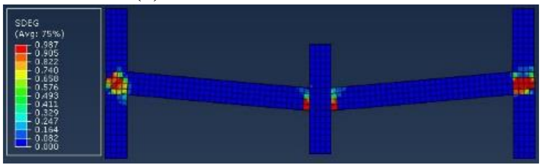


(b) Precast wet connection-4

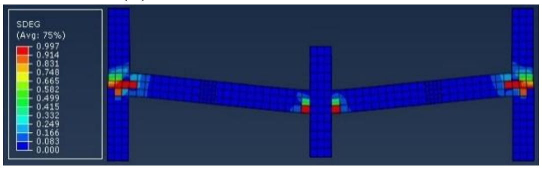
Fig. 13 Stress contour in concrete and in steel reinforcement bars



(a) Monolithic connection



(b) Precast wet connection-1



(c) Precast wet connection-4

Fig. 14 Scalar stiffness degradation contour of test specimens

concrete and steel reinforcement bars occurred, which is not captured in FE model considered for the study. The other reason is that, during experimental studies, middle column exhibits slight rotation due to cracking and rupture of bottom longitudinal reinforcement bars, at large deformations, which results in unsymmetrical deflection in left beam and right beam, however, bar rupture modeling is not considered in numerical analysis carried out in this study. Comparison of results obtained from experimental studies and numerical analysis for precast wet connection-1 and precast wet connection-4 are presented in Fig. 12(b) and Fig. 12(c), respectively. Table 5 shows the comparison between experimental results and results obtained through numerical analysis, in terms of maximum load and corresponding deflection, for monolithic connection as well as precast wet connections.

For precast wet connection-1, connection is carried out by inserting U-shaped reinforcement bars projecting from beam into gap provided in column. During experimental studies, formation of diagonal cracks are observed due to inadequate confinement reinforcement within connection region. Also, minor separation between cast-in-place micro concrete and normal concrete is observed, which lead to flexible behaviour of specimen. Similarly, for monolithic connection and precast wet connection-4, connection loses fixity with increase in load, due to crack formation, which relaxes bond between reinforcement bars and concrete. While, during numerical analysis, perfect bond is considered between reinforcement bars and concrete. Therefore, results of numerical analysis exhibits very stiff response with significant difference of deflection at the peak in comparison with responses observed during experimental studies.

In the present study, perfect bond is considered for interaction between concrete and steel reinforcement bars as reported in literature. As test specimen consists of large number of reinforcement bars, bond-slip behaviour is quite difficult to model. The assumption of perfect bond is about to capture the behaviour of specimen with reasonable accuracy up to low strain values, till width of crack is small. But, at large strain value or at higher deflection, some bond-slip takes place between reinforcement bars and concrete. Also bond-slip is likely to occur at the time of rupture of reinforcement bars, during experimental studies. Therefore, difference between experimental results and results of numerical analysis is increasing significantly after rupture of longitudinal reinforcement bars, particularly in case of monolithic connection.

Typical stress contours for concrete and steel reinforcement bars are shown in Fig. 13. Stress values for concrete varies from 14.54 N/mm^2 to 21.72 N/mm^2 in which maximum stress is observed in end columns. For steel reinforcement bars, stress varies from 488 N/mm^2 to 490 N/mm^2, with maximum stresses developed at beam column junctions. Fig. 14 represents typical contours of stiffness degradation, which shows good agreement with failure observed in test specimens during experimental studies. Contours of scalar stiffness degradation (SDEG) in a range of 0 to 1 indicates that 0 means element is having least damage and 1 means having maximum damage. For monolithic connection, SDEG contour exhibits maximum damage near the beam column junction which matches with the failure pattern of monolithic connection as shown in Fig. 8(a). For precast wet connection-4, maximum stiffness degradation is predicted at the beam column junction, without any significant failure within connection region, which is in line with failure pattern observed during experimental studies. Basically, scalar stiffness degradation indicates state of damage in elements and thus provide an insights into damage initiation and propagation. From the contours of scalar stiffness degradation, it is seen that developed FE models are about to capture behavior of monolithic and precast connections under progressive collapse scenario.

5. Conclusions

In present study, experimental investigation and numerical simulation of four different precast wet connections under progressive collapse scenario are discussed and their performance is compared with monolithic connection. For precast test specimens, connections are carried out at beam column junction and away from beam column junction. Based on this study following conclusion is derived.

• From the results of study, superior performance of precast wet connection-4 is observed when connection is provided away from beam column junction as compared to connection provided within or at the face of beam column junction. From the comparison of maximum load, it is observed that, precast wet connection-4 is having maximum load carrying capacity, which is 10.59% more as compared to monolithic connection.

• From the failure pattern of precast wet connection-4, where in connections are provided away from beam column junction, it is observed that, it behaves similar to that of monolithic connection with formation of flexure tension cracks and formation of hinges in form of concrete crushing. Any significant failure is not observed within connection region, for wet connection-4, which indicates that forces are effectively transferred from one precast element to other through connection.

• For wet connection-3, sudden collapse of cruciform shape element is observed due to brittle failure of micro concrete and insufficient shear reinforcement within connection region. While, from the failure pattern of wet connection-4, it is realized that, inclusion of poly-propylene fibres in micro concrete contributes in arresting crack initiation and further propagation within the connection region. Thus, addition of poly-propylene fibres in the micro concrete at connection region enhances ductile behaviour of specimen.

• Good agreement is observed between results obtained from experimental studies and numerical simulation, which indicates that finite element models adopted in present study are about to capture behaviour of precast connections under column removal scenario up to peak loading.

• Numerical simulation can be further implemented to evaluate behaviour of various types of precast connections by considering different parameters, in place of expensive and time consuming experimental studies.

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