USE OF REINFORCING BARS AS TIES TO PREVENT PROGRESSIVE COLLAPSE OF PRECAST CROSS WALL STRUCTURES

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ABSTRACT

To prevent the progressive collapse of the concrete building structures, the catenary action mechanism should be established in beams and slabs so that an alternate load path can be found. To examine the adequacy of British concrete design codes requirements on the conditions of catenary action and the Tie Force (TF) method for the design of progressive collapse, 4 full-scale floor joint tests of precast concrete cross wall buildings were designed and carried out.

Test specimens are the full-scale representatives of the precast hollow core floor slabs used in the typical cross wall building structures. The test assembly resembles a 2.075m long floor structure made of 1.2 m wide floor units and supported on the cross walls. Each assembly includes two or three keyways, where the longitudinal steel bars are provided as connection ties. The concrete grout used in the test specimens were specified with strength of 20 MPa to 30 MPa. The size of tie bars were designed to be 10 mm or 12 mm and the embedment length ranged from 250mm to 350mm.

Two conclusions can be drawn from the experimental investigation, namely, (1) there is a significant discrepancy on the behaviour between the floor assembly systems and the continuous RC beams following the removal of the middle support; and (2) the current TF method adopted in the British Standard has significantly underestimated the tie force requirement.

Key word: Progressive Collapse, Precast Cross Wall, Catenary, Tie Design, Tie Force Method

1. INTRODUCTION

Following the progressive failure of a precast concrete building, Ronan Point apartment, in London in 1968, the British design code for concrete structures [1] started to incorporate the provisions to address the problem of the progressive collapse. To gain a systematic understanding of this problem and hence to reduce the risk of progressive collapse, a 3-year research was conducted by Popoff [2]. Portland Cement Association (PCA) [3] also conducted a series of comprehensive investigations to form an underpinning knowledge basis supporting the stipulated minimum detailing requirements in the event of any local damage in the precast concrete structures. These attempts led to a tie-force (TF) design method adopted in the British concrete design code [1], which was a first of its kind in the world. Since then, the Eurocodes [4] adapted quite similar method and DoD 2005[5] has directly used the provisions of the British code. TF method, which is mainly of a prescriptive nature, requires the inclusion of internal, peripheral, and vertical ties to provide different alternative load carrying mechanisms, e.g. catenary, cantilever, vertical suspension and diaphragm actions, in the event of the loss of underlying wall supports. These prescriptive ties requirements may have proven to be adequate in engineering practice but are not fully scientifically justified, so through experimental, theoretical and numerical efforts are still needed to improve the understanding, at a fundamental level, of how the mechanism of post-collapse resistance is developed through these tie provisions in the absence of wall supports. These needs have also been supported by a number of researchers in the last decade.

Dusenberry [6] indicated the necessity of better understanding of the mechanism how the progressive collapse can be resisted. The UK Building Research Establishment (BRE) verified the adequacy and reliability of TF method by conducting a series of quarter-scale tests which led to further amendments in the current guidance [7]. To show the adequacy of codified methods for the progressive collapse, an evaluation on three well known collapsed building cases was performed by Nair [8] based on five current codes or standards. Results revealed that almost all three studied structures are susceptible to progressive collapse. Abruzzo et al. [9] has also indicated an inadequacy of TF method to prevent progressive collapse of structures. The necessity of developing an improved TF method has also been recommended by DoD (2005) [5]. According to the analytical study on a single beam, Rudi [10] showed that the tie rules are effective for the progressive collapse when the class C steel bars are used .To investigate the efficiency of TF design method, Li et al. [11] also conducted comprehensive numerical studies on two reinforced concrete (RC) structures of 3 and 8 stories, respectively; results were verified by the experimental work of Yi et al. [12]. The numerical results revealed that the current TF method cannot provide safeguard to progressive collapse for all RC structures that have different number of stories and experience damages in different locations; accordingly, an improved TF method was proposed. Based on the numerical assessment results of the disproportionate collapse, Gerasimidis et al. [13] suggested that a structure could respond better if damage is distributed in two adjacent elements rather than in only one element. Finally, based on the latest knowledge related to the design practice for the progressive collapse, DoD (2013) [14] has provided a significant revision to TF method in DoD (2005) [5] and British standard [1].

Compared with the experimental studies on the catenary action of steel structures, limited experimental studies are available on assessing the catenary action in RC structures [15-17]. This is specially the case for the multi-storey precast concrete cross wall buildings [18]. PCA's [3] experimental studies are the only published work on the performance of cross wall structures in terms of the catenary action and the progressive collapse by considering the floor joint behaviour. The previous research mostly focused on the behaviour of walls during the progressive collapse [18, 19]. To the best of the authors' knowledge, this is the first study that allows a rigorous determination of the realistic behaviour of precast concrete floor slab joint systems with the catenary actions following the removal of wall supports by considering the effect of tie bars in the keyways.

This paper presents the results of the experimental study on the mechanism of preventing the progressive collapse in the precast concrete cross wall structures under an accidental event such as an explosion or bombing attack. The concept that was studied is based on the TF method specified by the BS Standard which is commonly used in the current cross wall structures design in the UK. The key to TF method is to develop the catenary action mechanism that is expected to dissolve the energy arose from the accidental event and divert the loads to the undamaged structure. To this end, the longitudinal ties must be placed in the floor joints [1, 4, 5, 10, 11]. After a wall support is removed, the grout will be crushed very soon under the suddenly increased loads and these ties will mainly experience tensile forces and develop large deflection at the floor slab joints. This process forms a catenary action mechanism.

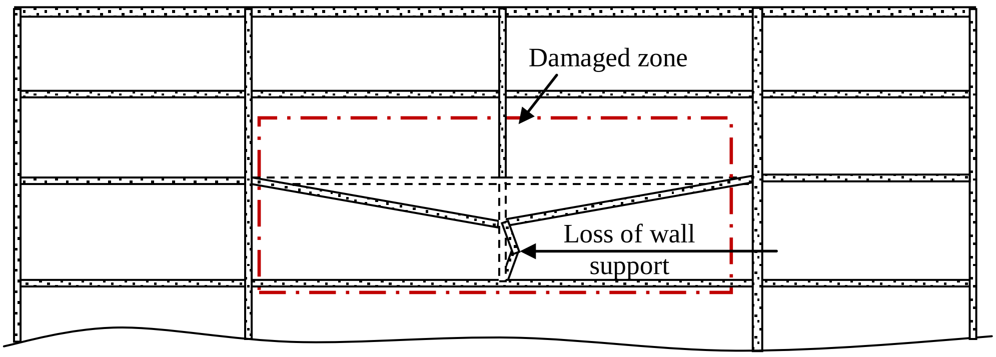
An experimental investigation programme studying the pre- and post- failure behaviour of the floor joints with longitudinal ties was carried out. The test specimens were the full-scale representatives of precast hallow core slabs used in a typical cross wall structures. The strains in the tie bars and the vertical deflection at the joint were recorded by using pre-instrumented sensors, e.g. strain gauges and LVDTs during the loading process.

2. GEOMETRY AND PROPERTIES OF TEST SPECIMENS

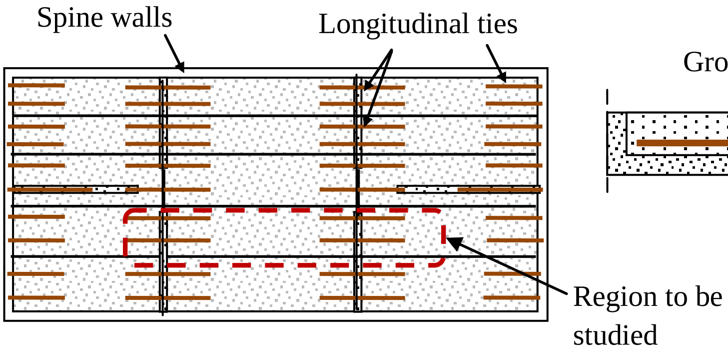
In the multi-story precast concrete cross wall structures, the applied load is carried by the one way precast slabs simply supported on vertical wall panels (Fig 1). In the case of a damage from the supporting walls (Fig. 1a), the floor joint above the removed wall is the most critical element to redistribute the applied loads to the undamaged parts of the system. Immediately after the removal of the joint support, the axial restraints at both sides will introduce a compressive arch action which can enhance the resistance of the system [15, 23]. This arch action soon disappears and the system turns into the flexural action once the deflection grows at the mid-joint. Under the flexural action, the joint grout fails shortly due to its low strength. While the deflection continues increasing, the system will develop a catenary action with the presence of axial restraints at both end and tie bars mainly experience axial forces.

The test assembly was designed to represent a portion of floor system that is affected by the loss of wall support. It includes one pair of floor units spanning on two adjacent spans in the longitudinal direction (Fig. 2), which consists of two hollow-core planks with the dimension of 2000 x 1200 x 150mm. This assembly provides two or three keyways, where straight steel bars can be placed as ties (Fig. 2 and 3). The precast floor slabs used in the test were provided by Bison Ltd. with a standard size in their product range.

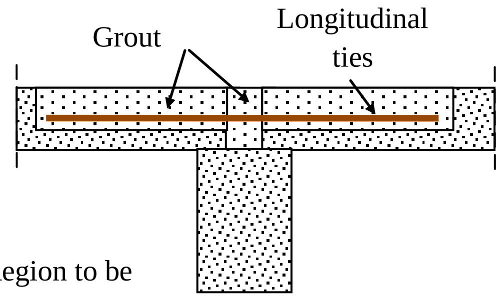
In the test assembly, the adjacent floors and walls were replaced by two braced steel frames (Fig. 2). The lateral stiffness of the support system has been chosen to represent the real stiffness of a typical cross wall structure. In the test, two possible failure modes of tie bars are expected to attain, i.e. bar fracture and pull-out. To this end, two grout strength were adapted, that is, 30 and 20 MPa, respectively. According to the pullout test results conducted in this study, appropriate embedment length of tie bars was selected to introduce these two failure modes (Table 1). FT1 and FT2 were designed to develop the bar fracture failure mode, while FT3 and FT4 were designed to establish the pullout failure mode. To study the contribution of concrete grout at the joint gaps during the loading process, FT4 specimen was specially designed without any cast-in-situ concrete grout in the gaps (Fig. 4). The average compressive strength of cube f_{ck} and the flexural strength of prism f_t were measured for the grout concrete based on the corresponding standard material tests from three specimens on the days of tests (Table 1). The slump of concrete was between 90-110 mm, which fell in the range of the normal design limit



(a) Section view of a cross wall subjected to the wall damage



(b) Plan view of floor system with longitudinal ties



(c) Typical section view of floor joint

Fig. 1 Precast floor systems of a cross-wall structure

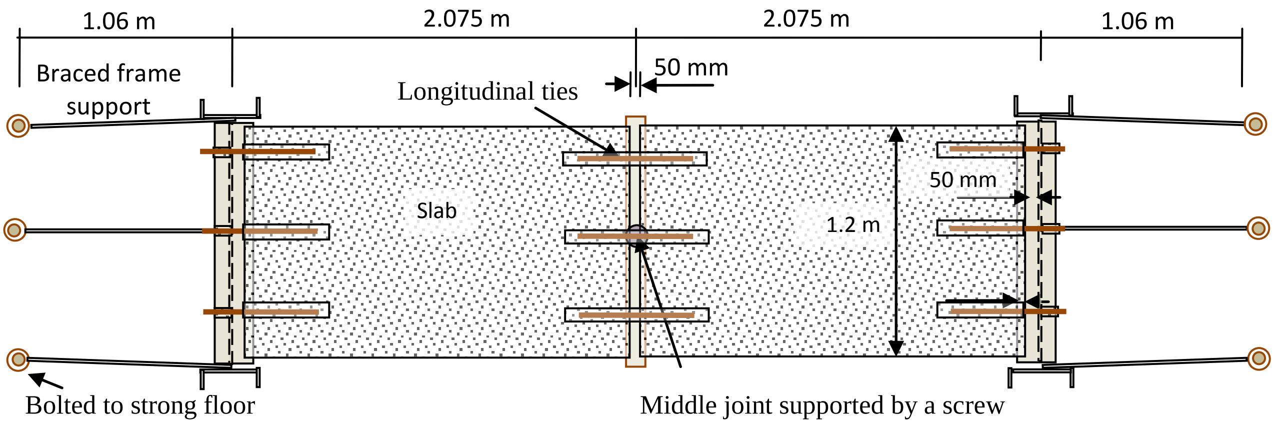


Fig. 2 Plan view of the test assembly

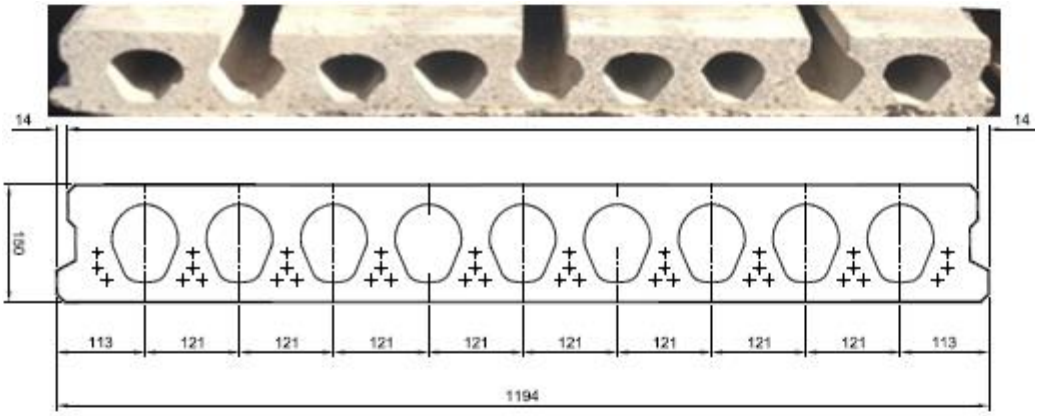


Fig. 3 Precast concrete floor units used in the full scale test specimens (courtesy of Bison Ltd.)

Table 1 Details of test components

Tests No.	Ties							No. of Ties	Grout	
	d_b (mm)	l_d^*	l_b/d_b	Bar ratio	ε_u %	f_y (MPa)	f_u (MPa)		f_{ck} (MPa)	f_t (MPa)
FT1	$\phi 10$	350	35	0.087%	14.90	515	616	2	30	4.07
FT2	$\phi 10$	350	35	0.087%	13.36	515	614	3	32	4.53
FT3	$\phi 12$	200	16.7	0.126%	14.15	545	667	2	23	3.14
FT4	$\phi 12$	250	20.83	0.126%	15.98	545	671	2	18	-

$^*l_d$ the embedment length of the bars

(a) Concrete at the joint gaps FT1, FT2, FT3

(b) Specimen without cast-in-suit concrete in the joint gaps - FT4

Fig. 4 Cast-in-suit concrete at the floor joint gaps

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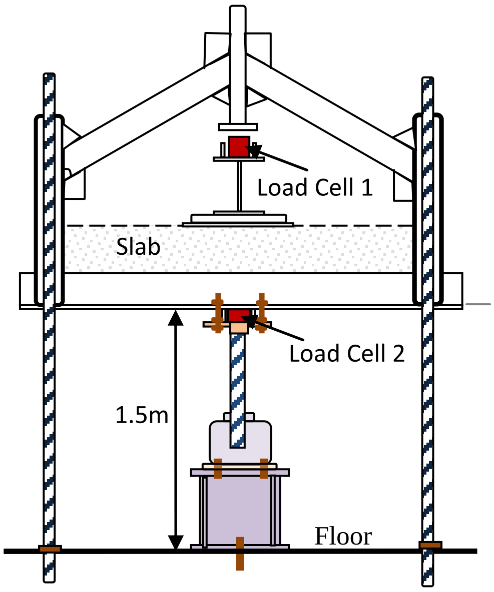
3. TEST SETUP AND INSTRUMENTATION

Fig. 5 shows the overview of the test setup and loading devices. A braced steel frame consisting of beam supports, two columns and three bracings provides the axial and vertical restraints at the slab ends. As shown in Fig 5, the precast slabs rest on the steel beam and are horizontally restrained at both ends by anchoring tie bars to the supporting frame. The loading device is so designed to resemble the scenario that an underling support wall is gradually removed. In reality, on the loss of the supporting wall, the floor may experience an impact line load from the upper wall. It will then lead to the chain reaction of the collapse of the upper floors. In reality, the damage and the resulting load path diversion occur in an extremely short duration and hence are of the dynamic nature. In this study, the dynamic effect is not considered, but a quasi-static case will be examined to reveal the failure process and the mechanisms.

(a) Perspective view of test set-up



(b) Loading device used in the test

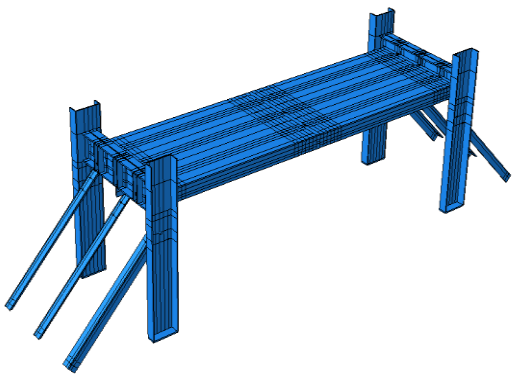
Fig. 5 Floor joint test specimens

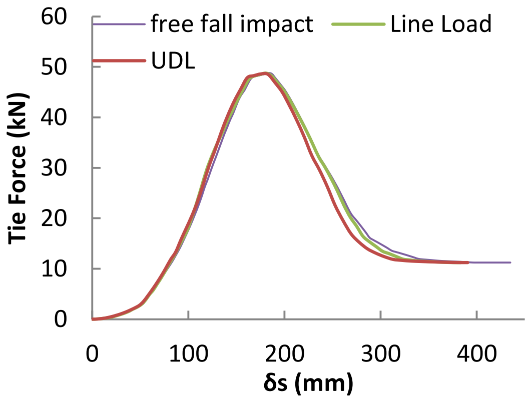
To introduce the collapse in a static manner, a screw jack is considered to support the slabs at the middle joint so that it can be moved down to mimic the removal of the wall support in a slow way (Fig. 5b). Astaneh-Asl et al. [21] used the same concept to simulate the column removal in a steel frame structure, dividing the middle supporting columns into two parts so that lower section can be pulled down. As only the gravity load due to the self-weight of slabs was present in the full scale tests, applying imposed load in the UDL form turned out to be challenge. A line load was then applied at the middle join instead. This loading arrangement agrees with those adopted in the previous studies [3, 12, 15, 20, 21]. The loading device has been design to facilitate this type of action in this test programme by using a screw jack with a stroke length of 600mm (Fig. 5b). The load was applied in a displacement control method up to bar fracture or pullout.

When the screw jack initially moved down, the central support reaction is reduced; with the jack continuing moving down to the point of the zero contact with the slab, the top loading unit touches the upper loading mechanism and starts to exert downward load to the slab. This process will carry on until the failure of test assembly. The loads and deflection was recorded by the load cell and the LVDT during the process.

3.1 VALIDATION OF LOADING METHOD APPROXIMATING SUDDEN REMOVAL OF SUPPORT WALL

The jack can apply downward load by moving with a constant velocity of 11 mm/minute. Assuming under an accidental event, the floor joist experiences a free fall impact action, the main concern is whether the tie force and the joist deflection would be the same as the case where the system is subjected to steady movement, in particular, for the relationship between the tie force and the deflection of the joint? To examine this, a series of finite element analyses were performed for the test designs considering various loading cases including the free fall action, the UDL load and the line load applied by the screw jack at the middle joint. The ABQUS software was used for the numerical analysis by considering the same boundary condition as in the test set-up (Fig. 6a). Results clearly shows that using screw jack to pull the slabs produces the same tie force in the reinforcement at the mid-joint as the free fall drop (Fig. 6b). A similar observation was obtained by Astaneh-Asl et al. [21] in their study on the composite floor for the progressive collapse resistance.





(a) TABAQUS model (b) Tie-force deflection relationship

Fig. 6 Tie force- vertical deflection under various loading applications

3.2 INSTRUMENTATION OF SPECIMENS

To monitor the tie force and vertical deflection in the full scale tests, several instruments were placed on the carefully chosen critical locations in the specimens. The details, locations and numbers of instruments are presented in Fig. 7. Several strain gauges were attached on the reinforcement bars to examine the stress variation of tie force. A linear variable differential transformer (LVDT) was used to measure the vertical deflection at the mid-span point of the specimen (Fig. 7). To monitor the restraining forces at the side supports due to the large vertical deflection, in FT2 and FT3 specimens, strain gauges were also used to measure the forces in the bracing (Fig. 7). The applied load was measured by two load cells as shown in Fig. 5b.

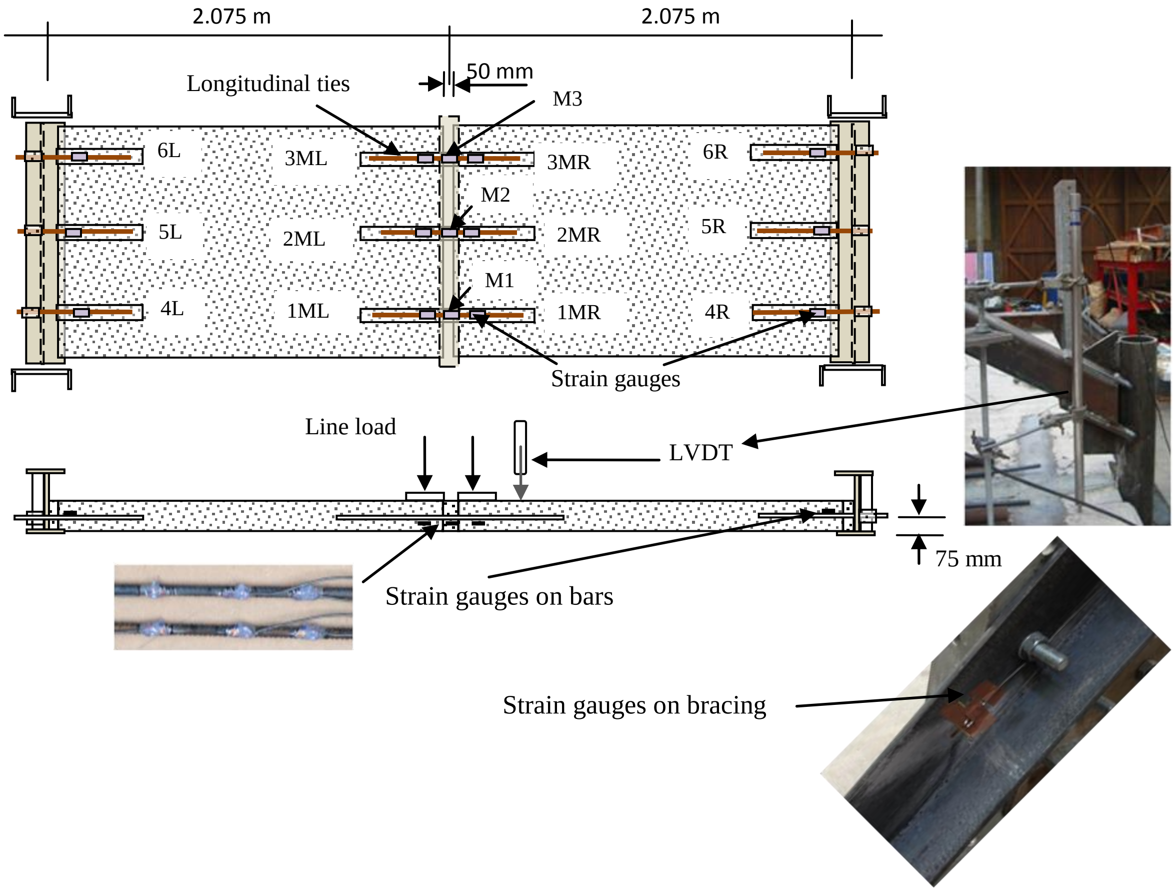


Fig. 7 Layout of instrumentations

4. Test results

Test results are presented at two levels, i.e., at the floor structure and the reinforcement bars.

4.1 FLOOR STRUCTURE LEVEL RESULTS

A total of four full scale tests were performed concerning different bar sizes, embedment lengths, compressive strengths, the number of ties and the presence of concrete grout at the floor joints. The first two tests were conducted to investigate the behaviour of system with the embedment length more than the anchorage length to introduce the bar fracture mechanism at failure and concerned the effect of the number of ties at the floor joints. Specimen FT3 and, FT4 were conducted to investigate the behaviour of system concerning the pullout failure mechanism. The detailed information of the test samples are shown in Table 1.

The screw jack was lowered at a constant rate until the tie bars at the middle joint fractured in FT1 and FT2 or failed due to pulling out in FT3 and FT4. The final deflection patterns of all specimens were very similar (Fig. 8). The experiment studies for the single-span RC beams showed that they would be collapsed by the bar fracture at the ends [11, 12, 16]; while in this study, no bar fracture was observed at the specimen ends.

Fig. 8 Failure pattern of test assemblies

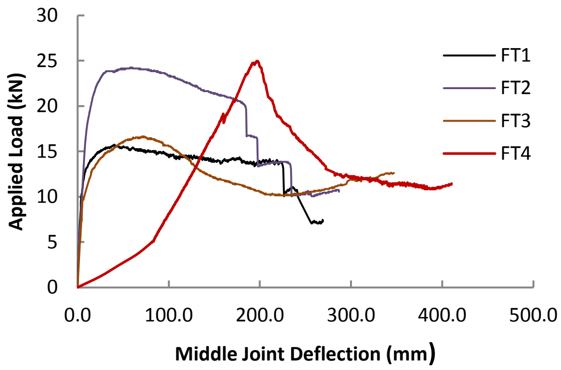


Fig. 9 Applied load versus middle joint deflection

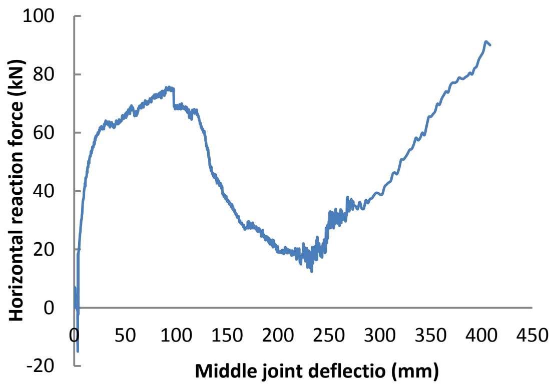


Fig. 10 Horizontal reaction force vs. middle joint displacement for FT3

Test results indicate the key differences in the two failure mechanisms. The failure patterns of FT1 and FT2 are approximately similar. Prior to the development of the catenary action, the middle bars fractured at the deflection around 200 mm, i.e. 10% of span length and the rotation of the slabs was 6.3^{o} (Fig. 8 and 9). However, for specimens FT3 and FT4, no bar failure were observed even at the vertical deflection around 400 mm, i.e. 20% of span length (Fig. 9). In the case of RC beams, even very well detailed RC beams will fail at this level of deflection [11, 12, 15].

The relationship of the applied load and the middle joint vertical deflection is presented in Fig. 9 for all four specimens. The sudden drop in the applied load as observed in FT1 and FT2 is due to the bar fracture in sequences. The applied load and corresponding displacement at several critical points of curves are listed in Table 2. During the initial loading stage, a combination of flexural and compressive membrane action governs the behaviour of system. As expected, this phase was short-lived and followed by the visible crack at the mid and the side supports. Figure 9 shows that FT2 /FT1 strength ratio is 1.62, which indicates that the strength of system is roughly proportion to the number of tie at the joint.

To measure the horizontal reaction of specimens following the removal wall supports, two strain gauges were attached on the inclined bracing angles. Results indicate that the maximum horizontal support reactions are approximately similar to the maximum tensile force of ties at the side joints (Fig. 10). Furthermore, results also show that, for the pullout failure mode, the reaction force will rise after the trough, which confirms the establishment of the catenary mechanism (Fig. 10).

Table 2 Applied load and middle joint deflection at critical points

Test	Critical deflection at the middle joint (mm)				Loading resistance		
	At the first peak	1st rebar fracture	2st/3st rebar fracture	2nd increase in loading capacity	Ultimate bending moment resistance (kN.m)	Peak load due to flexural action (kN)	Peak load due to catenary action (kN)
FT1	40	227	241	-	6.14	15.65	-
FT2	33	185	198/225	-	9.15	23.85	-
FT3	77	-	-	270	5.98*	16.65	13.62**
FT4	198	-	-	380	-	24.75	11.45**

*Considering maximum pullout force
**Due to the head of crew jack reaching the maximum travel, the test was stopped at the catenary zone

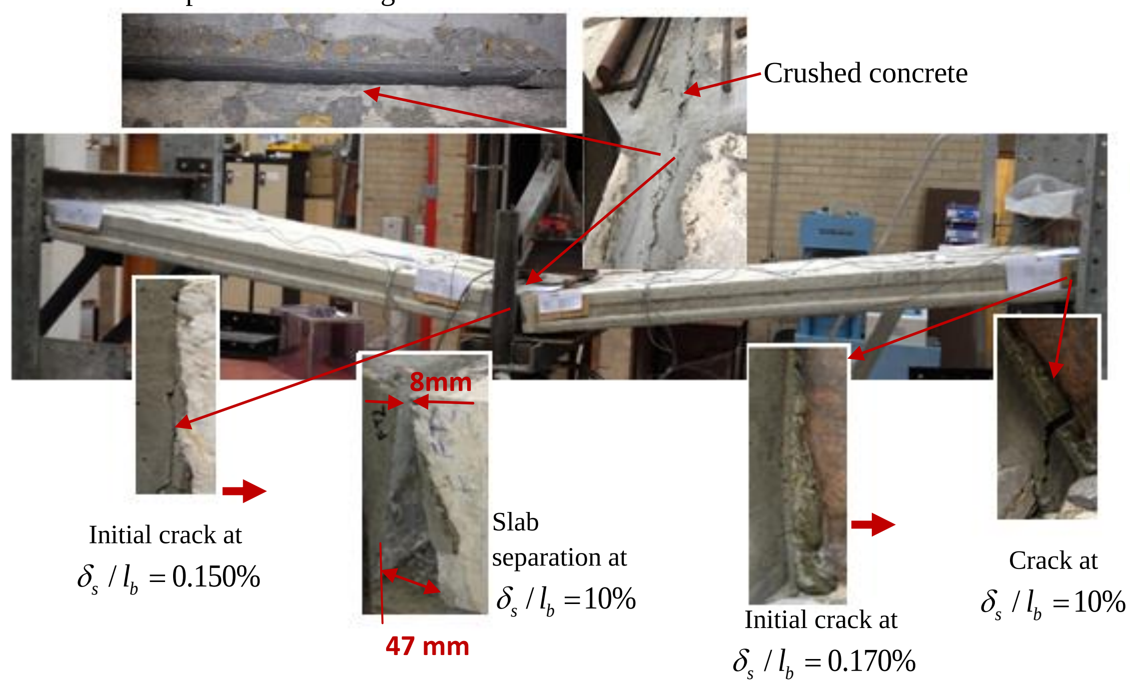


Fig. 11 Failure mode at the middle and ends joints for FT2

The interesting results was that, in all specimens, only one crack developed at the interface between the precast slab and cast in-situ grout at the middle joint gap, which was widen very rapidly with the increase in deflection (Figs. 11 and 12). This has suggested that the tie bars reaches the yield point at a very early stage, i.e. at the deflection around 1% of slab length. Multiple tension cracks in the lower concrete grout were observed at the side supports at the vertical deflection of 4.06/5.16/3.25 mm for FT1, FT2 and FT3, respectively. The failure patterns clearly indicate that following the initial crack at the middle joint, floor slabs act as two rigid bodies connected by reinforcement bars at the joints (Figs. 8, 11, 12).

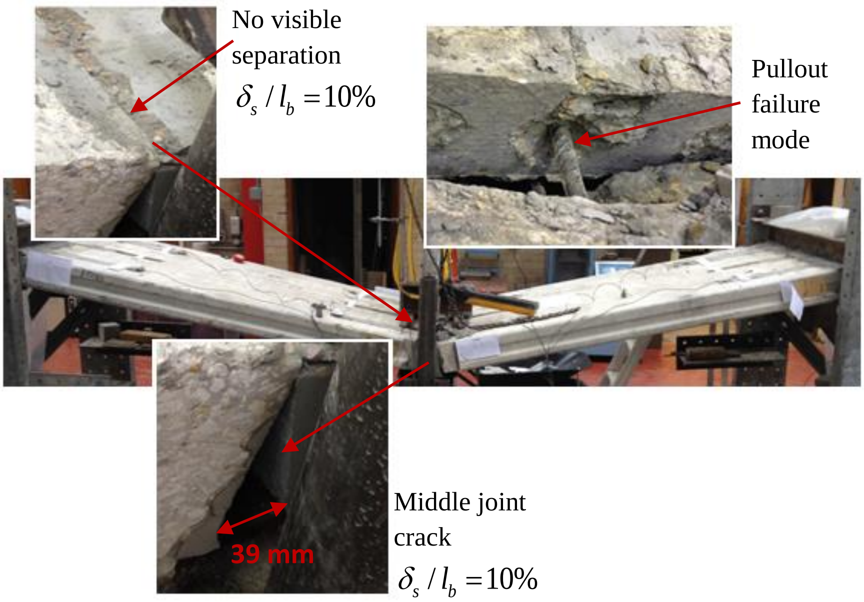


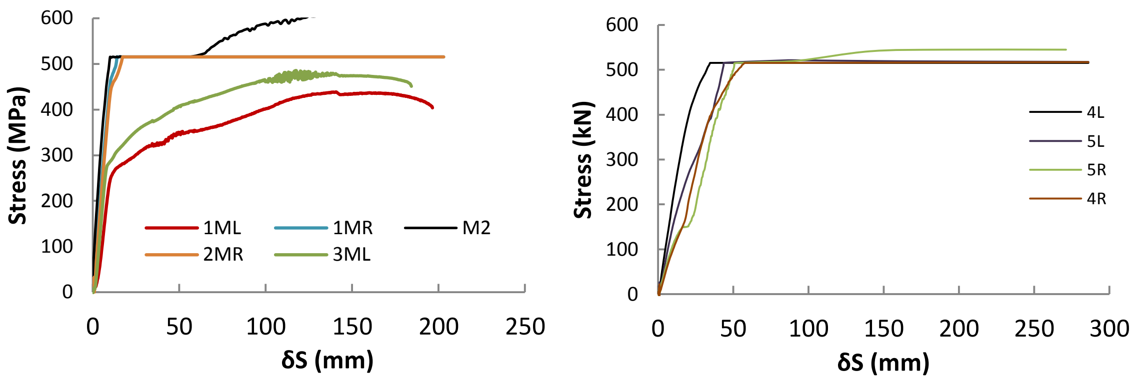
Fig. 12 Failure mode at the middle and side joints for FT3 at the deflection of 400 mm

Figure 11 and 12 illustrates the typical ultimate failure patterns near the middle and side joints for FT2 and FT3, respectively. Grout concrete is crushed in the zone with a depth of 4-5 mm, and then the middle bars fractured and a very wide crack penetrated through the entire slab depth. Slabs rotate as a rigid element without experiencing much flexural deformation. It was observed that, the crack width in FT3 was smaller than FT1 and FT2 and the corresponding slab units separated at different loading levels at the middle joint. While FT1 and FT2 separated at the deflection of 200mm (Fig. 11), no visible separation was observed in FT3 at the same deflection level (Fig 12). This difference can be attributed to that fact that, following the peak strength, the components of the applied force in parallel to the slab surface tends to push the slab units slabs toward the middle joint with the increase in the mid-joint deflection. FE analyses results indicate that the slip at the middle joint following the peak pullout force decreases with the increase in vertical deflection [22], but it increases at the side joints with the increase in deflection.

As expected, the behaviour of FT4 specimens displays remarkably discrepancy to the first three tests. The test results of FT4 indicate that the peak load occurs where $\delta_s / l_b \approx 10\%$ while for other specimens it was around 1% (Fig. 9). The absent results in Table 2 was that, the peak load of FT4 is greater than FT2, while the tie strength at the joints of FT2 specimen is more than FT4 by 48%, which confirm that the loading capacity of system is jointly related to the tie strength and the middle joint deflection.

4.2 REINFORCEMENT BAR LEVEL RESULTS

In all specimens, side tie bars were connected to the test frame through anchor bolts. To remove any rigid slip between the bars and anchor blots, before placing tie bars into keyways an axial force of 4 kN for 10 mm, or 6 kN for 12 mm bars was applied using hydraulic jack. The tie bars were placed into keyways at the 75 mm above the bottom surface of the slab or 40 mm above the lower surface of keyways. Near the end of all bars, a spacer was employed to fix the bars in the designated positions during casting and vibrating. To measure the slip and elongation, the reinforcement bars were marked at the free concrete surface. For all specimens, the gap width was 50 mm. To capture the fracture strain, one strain gauge was attached at the middle point of the joint gap and two gauges were attached over the embedment length. The measured strain was converted to stress by using stress-strain graph taken from the standard bar tests.



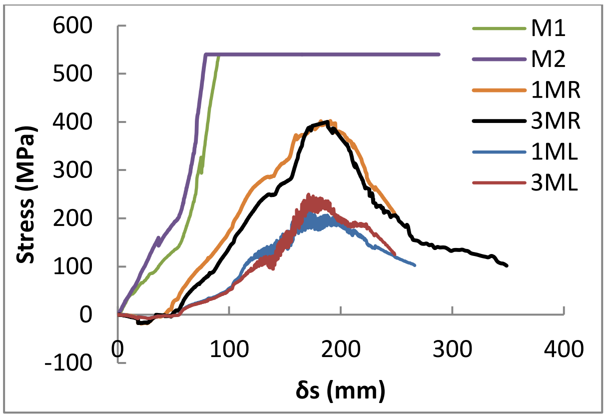
(a) Middle joint

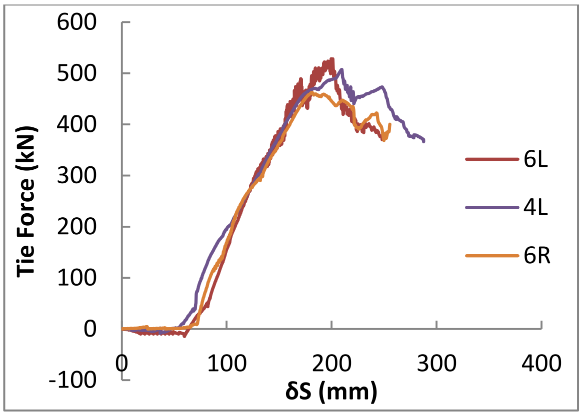
(b) Right and left support

Fig. 13 Stresses versus vertical deflection-FT2

The tensile stresses in the tie bars for the specimens experiencing bar fracture failure mode e.g. FT2, are shown in Fig. 13. For the same location of strain gauges, FT1 and FT2 led to the same stress-deflection curve, so only the results of FT2 are presented herein. In both specimens only one gauge was attached on the middle gap. The yield strain of reinforcement bars was 0.25%. Figure 13 indicates that, the middle ties experience yield stress at the deflection around 18 mm; while the side bars yield at the deflection around 50 mm, which indicates that the strain of reinforcement bars at the middle joints increase more quickly than the side bars (Fig. 13a, b). At this deflection of 205mm, the first longitudinal tie in FT2 fractured at the mid joint at the strain of 14.9%, but the longitudinal ties at the side supports did not fail and experienced plastic deformation without entering to the hardening stage (Fig. 13 b).

Figure 14 display the stress-deflection curves for FT4 experiencing the pullout failure mode. Results indicate that stress-deflection relationship follows the pullout behaviour of tie bars. The results show that, the stress from the strain gauges at the mid-floor joint increases from the very beginning of the test and the initial stress-deflection curve is almost linear up to the deflection reaching 50mm, which is followed by a dramatic increase in gradient up to the yield point. However, for the stress measured from the strain gauges attached within the embedment length, e.g. 1MR, 3MR, 1ML and 3ML, stresses are almost zero up to the deflection reaching 50 mm, which are followed by a linear increase up to the maximum stress. Prior to the development of the catenary action at the deflection of 50 mm, the total load was sustained by the screw jack located under the middle joint; accordingly the tensile force over the embedment length of bars was negligible. While lowering the middle joint, the tie bars were bent and the strain at the bottom surface of bars was induced at the middle gap (Fig. 14 a - M1 & M2). After the catenary mechanism is activated, the tie bars at the middle joint carried the total load indicated by the increase in strain of all gauges.





(a) Middle joint (b) Left and right support

Fig. 14 Stresses versus vertical deflection-FT4

The tie stresses in 4L, 6L, and 6R show that the maximum stress from the strain gauges attached on the steel bar near the end gap is similar to the maximum tie stress in pullout test results. This has suggested that the pullout behaviour of the reinforcement bars in the keyway dominates the floor system behaviour. It is noted that even if the steel bars at the middle and side joints experience yield at the gauges locations, the average stress is relatively smaller than the yield stress [22].

5. DISCUSSION

It is generally accepted that, through catenary mechanism the applied load is sustained by tensile force along the elements. Although, so far, there is no clear definition for the onset of the catenary action, it can be defined as the point where the axial force in the steel reinforcement at the compression zone turns from compression to tension [24]. Alternatively, the point of the re-ascending phase has been defined as the starting point of the catenary action by some researchers [15].

In the bar fracture mode, the middle joint ties cannot contribute in the catenary action, as they have already fractured before developing the catenary action; while the specimens with the pullout failure mode are capable of developing the catenary action mechanism. It is to be noted that, based on their experiment results, Yi et al. (2008) and Su et al. (2009) argued that the bottom reinforcing bars in the middle joint of RC beams can contribute in the catenary action, while Yu et al. (2010) obtained the opposite results. As in the RC beams structure, the ends are critical points, the bottom bars at the middle joint probably can contribute in developing catenary action, while in precast cross wall structures, the middle joints are the most critical point and any fracture will start from this point (Fig. 13b, 14a). This can be considered as the main difference between RC and precast structures.

6. CONCLUSION

The specimens with the bar embedment length more than the anchorage length result in the bar fracture failure mode prior to the development of the catenary mechanism, i.e. FT1 and

FT2, while the specimens with the embedment length less than the anchorage length result in the pullout failure mode and the full catenary action mechanism, i.e. FT3 and FT4.

In the case of using one bar in the keyways, specimens with the pullout failure mode will provide more strength and ductility than specimens with the bar fracture mode. Although the specimens with the pullout mode and without concrete at the gaps (i.e. FT4) provides higher capacity than other specimens, the load-deflection relationship shows that less energy will be absorbed by this system. However, specimen FT3 experiencing the pullout failure mode with the concrete at the gaps provides relatively less strength than FT4, but it can be defined as the best energy absorption system which is capable to provide catenary action mechanism.

Following the peak load, the strength of FT4 dramatically decreased, changing the failure mode from the pullout to the bar fracture, the load-deflection relationship will follow the same trend as FT1 or FT2; hence this system will provide considerable strength without significant ductility up to failure.

The main finding of this study is that it is the pullout behaviour which governs the floor system behaviour. There is clear difference compared with the reported test result on the RC structures.

7. REFERENCES

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