The progressive collapse behavior of precast floor-to-floor connections using longitudinal and transverse ties

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ABSTRACT. This paper involves a fundamental study of a numerical method for progressive collapse resistance design of floor-to-floor joints in precast cross-wall structures. It presents a 3D numerical study of a floor-to-floor system with longitudinal and transverse ties. The model is also used to derive the post-bond behavior and the mechanism of forming catenary action concerning the bond behavior in precast cross-wall structures. The obtained results indicated the adequacy and applicability of the code specifications in British Standard, Euro Codes, and DoD 2013. Discrepancies in the tie-force between the numerical results and codified specifications have suggested an inappropriate use of the current TF method, hence, an improved model based on the numerical results has been proposed to address this concern. To the authors' best knowledge, this is the first numerical study to investigate the behavior of floor-to-floor joints following the removal of wall support in typical precast cross-wall structures when considering bar fracture and pull-out failure mode.

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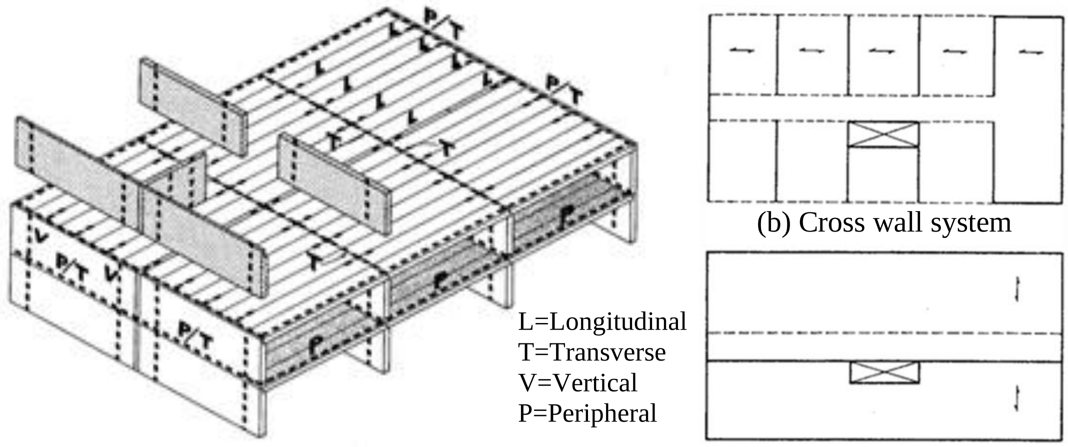
1. Introduction

The precast concrete cross-wall building is a modern construction method and well adapted for high-rise housing; as most building components are prefabricated in factories, they are precision engineered and facilitate a fast-track construction (**Fig. 1**). This method of construction has been developed for residential buildings such as multifamily housing, hotels, military barracks, and student residences.

As defined by the Portland Cement Association [1-9] the term "large-panel" concrete structure is used to describe a building system consisting of vertical wall panels together with precast concrete floors and roofs (**Fig. 1**). In the usual arrangement, a wall that is perpendicular to the longitudinal axis of a structure is referred to as the cross wall and a wall that is parallel to the longitudinal axis is termed the spine wall. In the cross-wall system, floor/roof slabs are typically one-way hollow-core precast concrete slabs and only cross-walls carry the floor loads.

The construction method employs a series of transverse, vertical, and longitudinal ties, designed to meet the criteria against progressive collapse based on provisions of building codes (**Fig. 1a**). These ties allow cantilever behavior, and beam actions for wall panels, membrane/catenary actions and horizontal suspension actions for floors and vertical suspension actions for wall/floor junctions [10, 11].

Literature review relating to the progressive resistance of structures indicates that compared with the numerical and experimental studies on the progressive resistance of steel/RC structures, the behavior of multi-story precast concrete cross wall following a wall removal had limited research and attention in the last two decades. Since the Ronan Point event makeshift provisions and recommendations have been incorporated into codes; most of them discriminate against precast concrete cross-wall construction and codes recognized precast concrete marginally, in which most codes provided only one-half page about Precast Concrete structures.



(b) Cross wall system

(a) Overall tie requirements (c) Spine Wall System

Fig. 1 Typical ties arrangement in precast concrete cross wall structure [6]

Following the partial failure of a precast concrete building based in London, Ronan Point apartment, in 1968, the British Standards for Concrete Structures started to incorporate provisions to deal with the problem of progressive collapse [12]. In 1976 the U.K. building regulation required that all buildings must be designed to resist disproportionate failure by tying together building elements, adding redundant members, and providing sufficient strength so that structures are strong, ductile, and capable of redistributing load [13]. The Fifth Amendment of British Standard has introduced two important concepts for the first time: "*key element design*" *and* "alternate *load path*". These two concepts have been employed by the current British standard and many International Codes [14].

After recent terrorist attacks on buildings in the world, several U.S. government agencies have published their design requirements for preventing progressive collapse [15, 16, 27, 18]. As each agency has adopted different performance objectives for buildings subject to abnormal loads, it indicates that the design methods to provide resistance to progressive collapse have not been standardized through these documents. However, to prevent progressive collapse most Codes and Standards have adopted two methods i.e. indirect i.e. "tie force-TF" and direct i.e. "the alternate load path (ALP) " method [12, 19].

The tensile tie force (TF) method is one of the main design approaches for preventing progressive collapse; whereby an indeterminate structure is analyzed statically by assuming a failure mode for a locally simplified determinate structure. To establish catenary action and prevent progressive collapse following the removal of a load-bearing wall, the TF method was established in the BS8110-1:1997 for the first time after the well-known Ronan Point event. Then BS EN 1991-1-7:2006 employed an identical method and formula. DoD UFC-04-023-03 (2005) has, also, directly employed the provision of the British Standard with a possibility of amendment by performing further investigations [20].

The tie force approach provides a mechanism that allows slabs to span over a removed load-bearing wall support. It is emphasized that there is no theoretical justification for which the tie force method can enable elements to bridge over removed wall supports in all circumstances [21]. On the other hand, although the tie force method's requirements are given by codes of practice, there is no specific provision to provide ductility; hence further difficulties might exist again in relying on catenary and membrane action.

In the alternate load path (ALP) method (design for load case "local failure") a degree of local failure is acceptable. Still, by providing redundant and alternate load paths to bridge over the failed members, progressive collapse will be prevented. In this method following the removal of a critical element from the structure due to abnormal loading, the structure should be capable of redistributing the gravity loads to the remaining undamaged structural elements. In this approach to analyze the structures, linear elastic static, linear elastic dynamic, nonlinear static, and nonlinear dynamic approaches can be used. In this study, the ALP method is used to propose a new TF method and evaluate the adequacy of the TF method in British Standard, Eurocodes, and DoD 2013.

The prescriptive tie requirements in the codes may have proven adequate in engineering practice, but are not fully scientifically justified; therefore, substantial efforts are still needed to improve understanding at a fundamental level of how the mechanism of post-collapse resistance is developed through these tie provisions. It seems the efforts to address all questions regarding the progressive collapse phenomenon are still a matter of life and need for systematic research and development of regulation. With today's high interest in added security measures, the structural robustness and progressive collapse of structures must be considered in the context of twenty-first-century evaluations of acceptable risk and abnormal loads [21]. This need has also been supported by several researchers in the last decade.

Dusenberry indicated the necessity of a better understanding of the mechanism by which progressive collapse can be resisted [22]. To verify the efficiency of the TF method, the UK Building Research Establishment (BRE) has performed some quarter-scale tests [23]. The investigations on several bombing attack sites in the UK indicated that most of these structures remained intact and damage was only limited to localized areas, hence Moore concluded that the current UK regulation is capable of successfully enhancing structural robustness. To show the adequacy of five current codes in the USA i.e. ASCE7-02, ACI 318-02, and GSA 2000/2003/PBS an evaluation of three famous collapsed buildings i.e. Ronan Point, Murrah Building, and WTC1&2 was performed by Nair [24]. The results indicate that all three studied structures are approximately susceptible to progressive collapse. Abruzzo et al. [25] conducted a nonlinear dynamic analysis on a five-story concrete building that met the ACI's integrity requirement and DoD 2005 tie-force provision. The result indicated that those regulations significantly underestimated the tie strength requirement, and the structure was remarkably susceptible to progressive collapse following removal of one column support.

To evaluate the adequacy of the tie force method in DoD 2005, Li et al. [26] conducted a numerical study on two different frame structures of three and eight stories by considering normal seismic load, the DoD alternate load path method (AP), and the TF design method. The susceptibility of the two structures in the case of normal seismic design and current tie strength design was quite identical. The numerical results revealed that the current tie force method cannot provide a safeguard to progressive collapse for all RC structures with different numbers of stories and experience damage in different locations; accordingly, an improved TF method was proposed.

This conclusion has been further confirmed by the latest edition of the DoD 2013 design criteria published in 2013, in which the required tie force has been increased significantly compared to the previous recommendations quoted in the British Standard [12, 15].

Another criticism of the TF method has been raised by several researchers who believe that it does not take into account the behavior of the structure as a whole [27-31]. In addressing this issue, they proposed that a global analysis of progressive collapse should be performed by considering the loss of stiffness in local regions. However, to do so, the key step is to characterize the real behaviors of the local regions, such as joints in the cross-wall structures, during the progressive collapse.

A comprehensive numerical model on the behavior of floor-to-floor joint systems following the removal of wall support was developed and verified by the authors using pull-out and full-scale tests from which key influencing factors were identified [32, 33, 34]. These papers focused on developing a model for global analysis of precast structures subject to increasing vertical loading and notional removal wall support. The developed model was found to be able to trace a complete and stable tie force-vertical deflection history with good accuracy and different bar sizes, embedment lengths, and slab lengths. Moreover, the results confirm that both bar fracture and pull-out failure modes can be effectively simulated by the developed model. Furthermore, the developed model can be used to analyze floor-to-floor system using longitudinal and transverse ties. To the best of the author's knowledge, this is the first study that allows the rigorous determination of a realistic behavior of floor-to-floor system following the removal of wall support.

It is generally accepted that precast concrete structures are more vulnerable to progressive collapse compared to cast-in-place concrete structures, while they have been subjected to less attention than conventional RC structures. To fill this knowledge gap and investigate the capacities and resistance mechanisms of unbonded post-tensioning precast concrete beam-column sub-assemblages with different connection configurations for preventing progressive collapse, an experimental study considering eight half-scaled specimens was conducted. The results clearly indicate that the failure mechanisms are significantly different from those of conventional RC frames and precast concrete structures with cast-in-place connections [35], which agrees well with the results of experimental studies conducted by authors [33].

To investigate the progressive collapse resistance of prestressed precast concrete (PC) frame structures, a comprehensive numerical and analytical investigation was conducted by Li et al. [36]. The results indicate that increasing the top reinforcement ratio of beams or setting additional shear reinforcement at joint regions can effectively increase the vertical resistance of the beam at both compressive arch action (CAA) and tensile catenary action (TCA) stages. Decreasing the length of the bonded segment of steel strands could improve the structural ductility in the process of progressive collapse but cannot influence the variation of vertical resistance of the beams at both the CAA and TCA stages [36]. The same results have been obtained by authors [33].

Most studies on progressive collapse indicate that general RC beam behavior can be divided into three stages; (1) flexural action, (2) compressive arch action, and (3) catenary action [34]. The results indicated that similar to the other studies, beams collapsed at the deflection-to-span length ratio of 17% and the maximum capacity to plastic stage capacity was 1.4. According to the results of a half-scale three-story RC frame, Xiao et. Al. [35] showed that the structures designed based on ACI 318-08 and ASCE – 2002 can resist progressive collapse following internal or corner column removal, while for two-column removals, the structures cannot resist progressive collapse.

To study the effect of slabs on progressive collapse behavior, five one-third-scale beams with and without slabs were tested under quasi-static loading [36]. The results showed that the specimens with slab increased the progressive collapse resistance by 146% under flexural action. Furthermore, they observed that the seismically RC beams and the increase in beam height improve the collapse resistance of the beams. According to the two one-third-scale RC flat plates under two corner column removal scenarios, Ma et al. [37] concluded that the ultimate progressive collapse resistance at the post-failure stage to the first peak load ratio was 0.71 and 1.2.

To explore the efficiency of bolted steel plate beam-column joints in a precast frame for progressive collapse, an experimental study on three half-scale specimens was conducted by Al-Salloumn et al. [38]. The precast frame with bolted steel plate at the joint provided the collapse load 29 times of specimen without strengthening. The result of an experimental study on two one-third-scale precast and one cast-in-place RC frame under column removal scenarios indicated that precast frames with appropriate joint detailing can provide a progressive collapse resistance similar to that of cast-in-place frames [39]. To investigate the efficiency of connection types (pinned and weld) in mitigating progressive collapse, five one-third-scale beam-slab substructures, two precast, and three cast-in-place RC tests were performed by Qian and Li [40]. The results showed that all specimens did not experience any sudden collapse. Unexpectedly, the specimens with welded connections experienced brittle failure, while the specimens with pinned joints experienced ductile failure with large rotational capacity.

A comprehensive review of the progressive collapse of RC structures based on experimental studies has been conducted by Alshaikh et al. [41]. The main focus was on beam–column and beam–slab sub-assemblies, planar frame structures, and large-scale buildings. Various variables were taken into account; (1) alternate load path, boundary conditions, research method, additional reinforcing, seismic detailing proposed by codes, and contribution of the RC slabs. The review indicated that there are significant discrepancies between the results of experimental investigations conducted in the last decade.

Furthermore, a comprehensive review of the progressive collapse resistance of precast RC structures has been conducted by Ibrahim et al. [42]. The review includes the latest advancements in strengthened schemes of precast concrete beam-column and beam-column-slab connections. The review clearly shows that preventing progressive collapse in precast structures is still a serious challenge.

In this paper, to study the adequacy and applicability of the TF method in British Standard, Eurocodes, and DoD 2013 extensive parametric 3D analysis of floor-to-floor systems has been conducted, using the developed method proposed by authors [43]. The results obtained will improve the understanding of the mechanism of how tie bars will contribute to the resistance of loads, for structures subjected to local damage. To do so, a comprehensive progressive failure mechanism of the longitudinal and transverse tie bars has been displayed. Discrepancies in the tie force between the numerical results and codified specifications have suggested an underestimate from the TF method, which may lead to an unsafe design, hence an improved model based on the numerical results has also been proposed to address this concern.

2. Research Significance

Current design practice in the UK, EU, and USA is mostly based on the descriptive method to specify the tie design to address the progressive collapse in precast concrete construction. Although these provisions have proved to work well, the real performance following a localized failure has not yet been fully understood. To further investigate this, the developed model will focus on various levels of details, which include the bond failure modeling on a single steel bar, the post-failure behavior of floor-to-floor connections, and the behavior of a typical building unit. It is worth mentioning that by using computational methods and the performance provisions, in contrast with the descriptive provisions, it is possible to deal directly with the structure's behavior under given loads. Furthermore, the above literature review indicates that the current research does not take into account the behavior of the precast structure as a whole. In addressing to this issue, a global analysis of progressive collapse has been conducted by considering the loss of stiffness in local regions. However, to do so, the key step is to characterize the real behaviors of the local regions, such as joints in the cross-wall structures during the progressive collapse.

It is generally accepted that the FE is a robust structural analysis approach, but the separation, collision with other elements, and falling simulation would be relatively difficult, and the analysis cannot follow the procedure for the entire collapse. To address this issue, the failure mechanism is automatically defined through the developed model by the authors [43]. Also, the maximum collapse area can be obtained following the progressive collapse of structures.

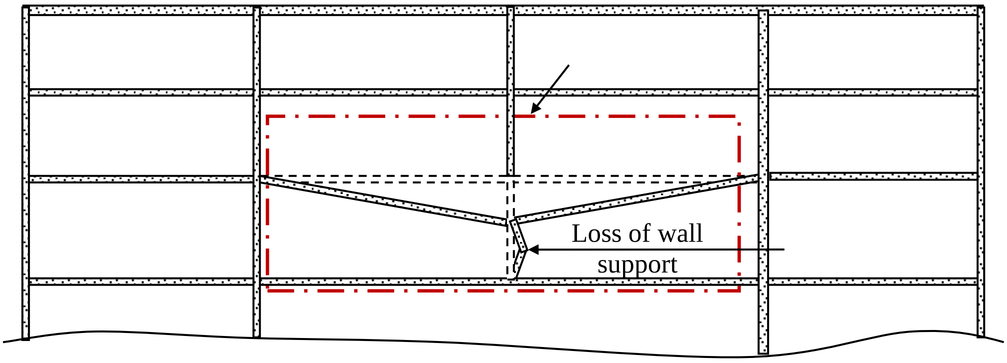
In this study, to overcome the inadequacy of the current TF method in both national and international codes of practice, a new TF method to analyze precast concrete structures is proposed.

3. Alternate load paths

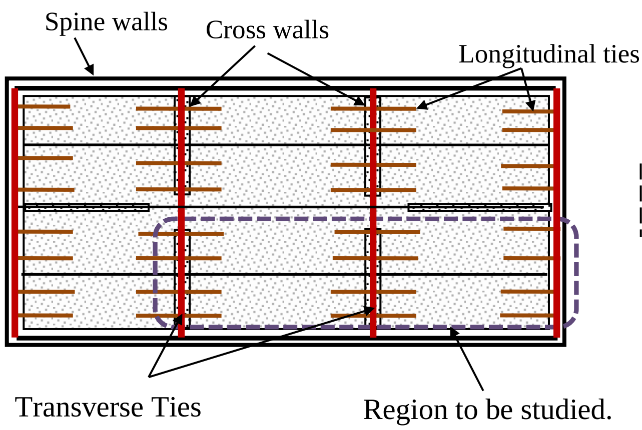
Following the removal of wall support in the precast cross-wall structure due to abnormal loading, a new load path must be provided which is defined as an "alternate load path". To prevent progressive collapse, the new load path should be capable of bridging over loads from the damaged elements or area to the remaining undamaged structure. In the precast cross-wall structures this can be achieved through tying the whole structure together in both horizontal and vertical directions (**Fig.1** and **2**). Alternate load paths can be established through various mechanisms i.e. catenary action of the floor-to-floor system, cantilever and beam action of wall panels, vertical suspension of wall panels, and diaphragm action of the floor plans [7]. In this paper catenary and cantilever action is taken into account and it is assumed that the other two load paths have been effectively provided.

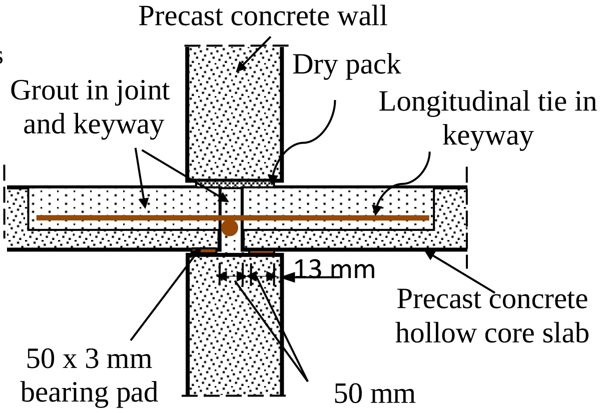
3.1 Catenary action

This mechanism can sustain gravity loads through tensile force in the ties while the structural elements are carrying excessive deflection. In precast cross-wall structures, when underlying wall support is suddenly removed due to an abnormal load (**Fig. 2a**), to bridge out the loads exerted by the upper walls and retain structural integrity continuity at the floor-to-floor joints must be provided so that an alternate load path can be found. Unlike under normal service conditions, a much larger deformation in the affected zone is allowed. Therefore, the ductility of these connections must be sufficient to satisfy the deformational demand. In precast cross-wall constructions, these requirements can be facilitated by longitudinal ties (**Fig 2**) embedded in the cast in-situ grout placed in the keyways of floor slabs. After a wall support is removed, the grout will be quickly crushed under the increased loads and these ties will experience tensile forces and develop a large deflection for the floor slabs. This process forms a catenary action mechanism [1-9, 23].



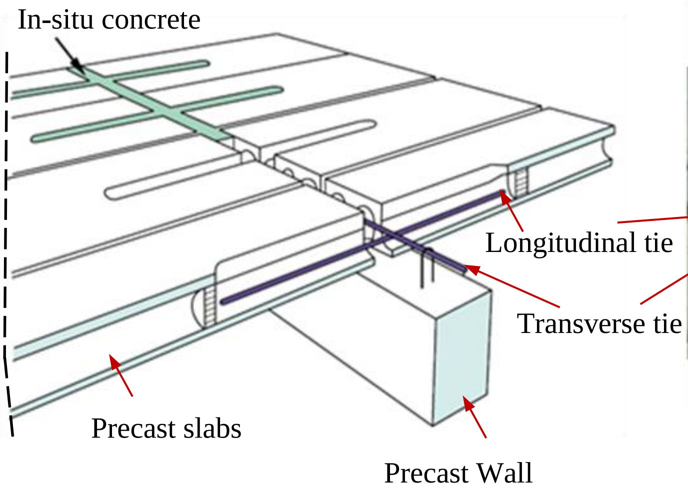
(a) Section view of a precast cross wall building subjected to the wall damage

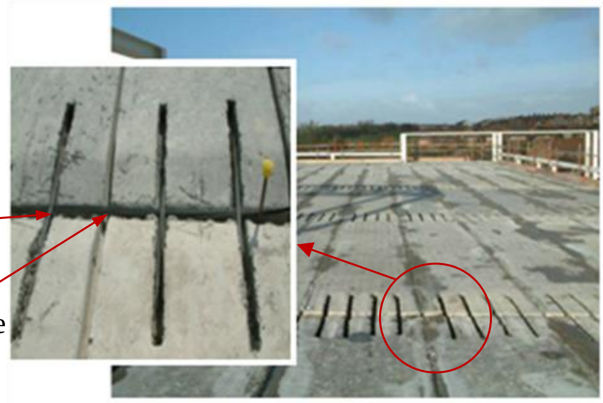




(b) Plan view of floor system with longitudinal and transverse ties

(c) Typical section view of floor joint





(Courtesy of Bison Ltd)

(a) Internal floor ties within hollow core units

(b) Examples of internal ties

Fig. 2 Precast floor-to-floor systems in a typical cross-wall structure

3.2 Cantilever action

The cantilever action of the remaining wall panels above the removed element is the main load path to transfer the loads. This mechanism is achieved through sufficient tensile continuity by employing transverse ties in the horizontal connections between successive wall lifts and proper resistance to prevent overturning of the cantilever must be provided. Horizontal ties which lie in horizontal connections, transverse to the span of the floor elements, are called transverse ties (**Fig. 2**). Transverse ties allow the development of a cantilever action of a wall panel above a lost support. If these ties can provide sufficient shear and tensile strength, the entire wall acts as a monolithic cantilever [7].

4. Description of the selected structure

The designed precast slab by Bison Ltd was selected as the source of information for the subject of the feasibility study presented in this paper. The studied specimens are a full-scale, realistic representation of simply supported concrete floor slabs in a precast concrete cross-wall building. The study is conducted assuming two different tie bars i.e. (1) specimens with longitudinal ties (**Fig. 3**); and (2) specimens with longitudinal and transverse ties (**Fig. 4**).

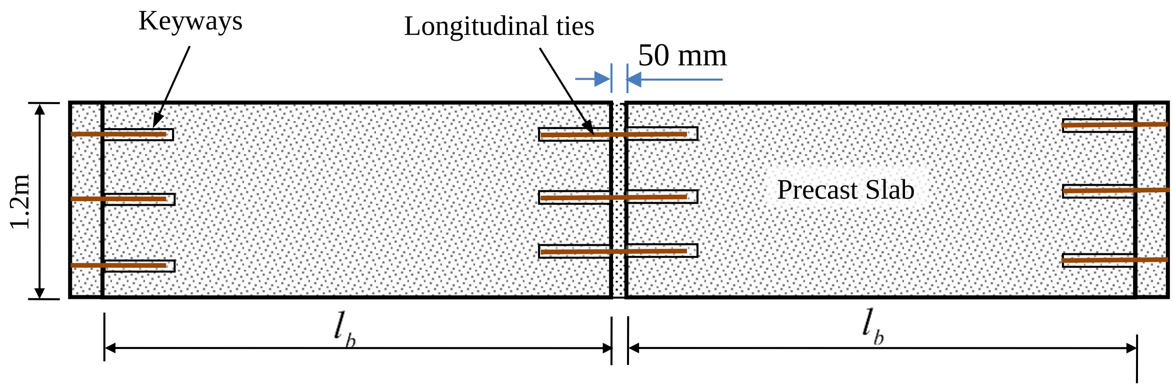


Fig. 3 Floor-to-floor system facilitated by longitudinal ties.

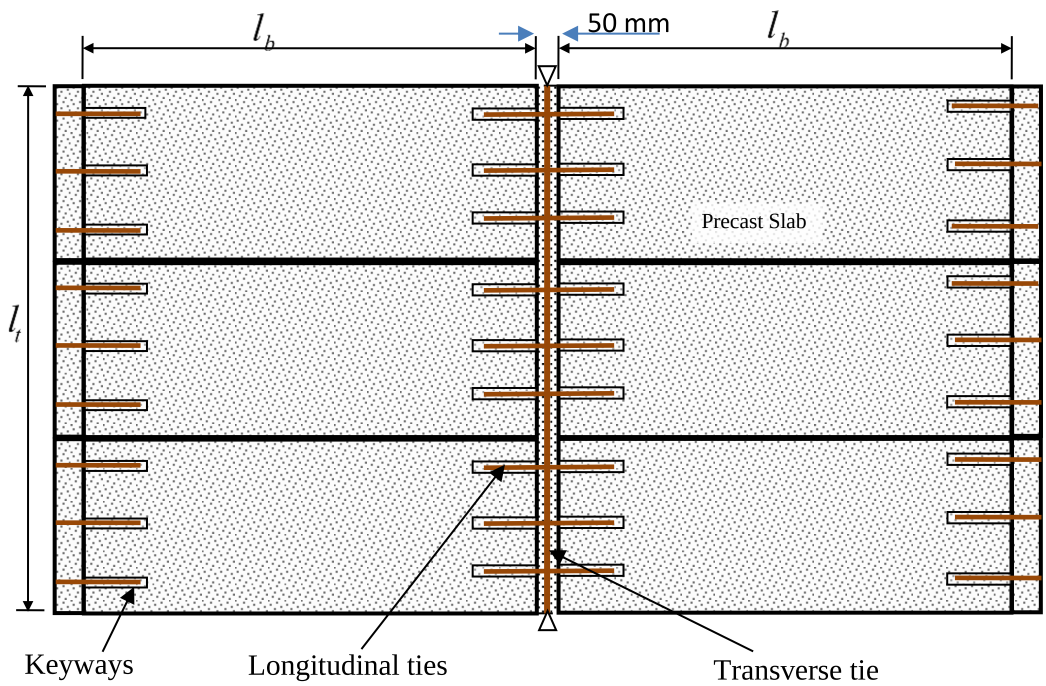


Fig. 4 Floor-to-floor system facilitated by longitudinal and transverse ties

4.1 Floor-to-floor system using longitudinal ties

For the first case, at the floor-to-floor junction, longitudinal ties are placed within the keyways. The section of the analyzed system consists of one full width of a precast concrete slab each containing two/three keyways. A two-span continuous slab system is modeled for different span lengths of 2, 4, and 6m using reinforcement bars in the longitudinal direction (**Fig. 3**). The diameter of the reinforcement bars in all specimens is 12 mm. The embedment length of ties is 400 mm and 250mm for bar fracture and pullout failure mode, respectively. Also, the compressive strength of 30 and 20 MPa was assumed in specimens with bar fracture and pullout failure mode, respectively [33].

4.2 Floor-to-floor system using longitudinal and transverse ties

Assuming the 2D behavior of the structure, considered in most of the studies, is not able to provide a clear understanding of post-collapse and the mechanism of forming catenary action. To simulate the actual performance, in the second set of analyses both longitudinal and transverse ties are taken into consideration using 3D modeling (**Fig. 4**). The properties of the specimens for bar fracture failure mode are shown in **Table 1**.

Table 1 The properties of specimens for bar fracture failure mode [33]

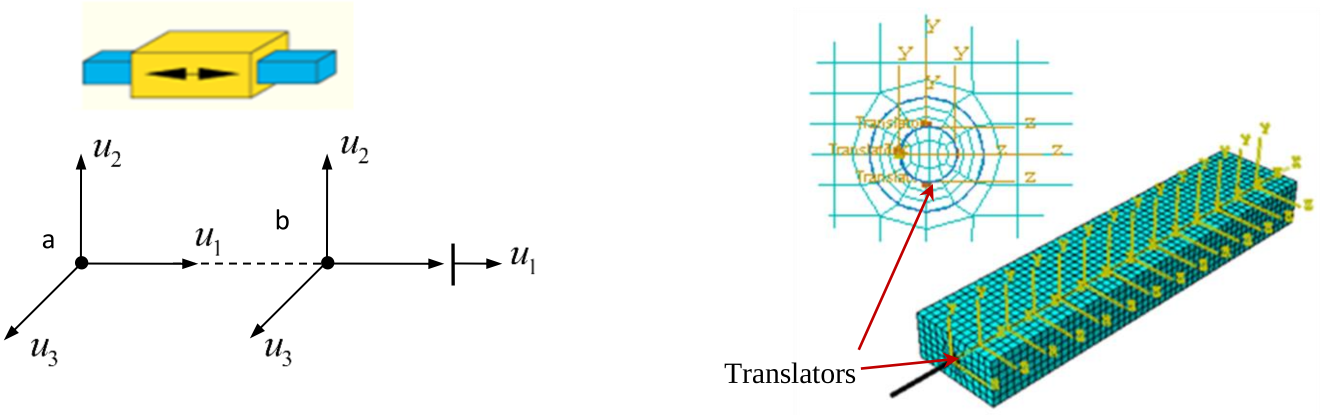
	Longitudinal Axis				Transverse Axis		f_{ck}**	f_t**
No.	**Length (m)**	**Bar Diameter**	**Number of ties/slab**	**Embedment length (mm)***	**Length (m)**	**Bar Diameter**	(MPa)	(MPa)
LTF1	2	$\phi 12$	2	400	2.4	$\phi 18$	30	4.07
LTF2	4	$\phi 12$	2	400	2.4	$\phi 18$	30	4.07
LTF3	4	$\phi 12$	2	400	4.8	$\phi 24$	30	4.07
LTF4	6	$\phi 12$	3	400	4.8	$\phi 24$	30	4.07
LTF5	6	$\phi 12$	3	400	7.2	$\phi 36$	30	4.07

5. Finite element model technique

The fundamentals of the developed numerical model conducted by the authors [43] are summarized and presented in the following sections.

5.1 Bond model

In interaction modules, various methods and elements to simulate the contact surface have been presented by ABAQUS [44], such as contact, constraints, and connector elements. Since bond slip is a function of the load (stress) versus displacement (slip), the elements that can couple a relative displacement with a force should be considered. Moreover, the connector needs to be able to specify damage mechanisms with different damage evolution laws. In this study, to simulate bond slip the translator is used (**Fig. 5**). To simulate a bar steel-concrete interface general contact algorithm with hard contact and frictionless behavior is applied.



(a) Translator element

(b) Implementation of translator element

Fig. 5 Translator as a type of connector

mesh size		
Circumference at interface	Embedment length	Middle of block
2.3mm	15 mm	150 mm

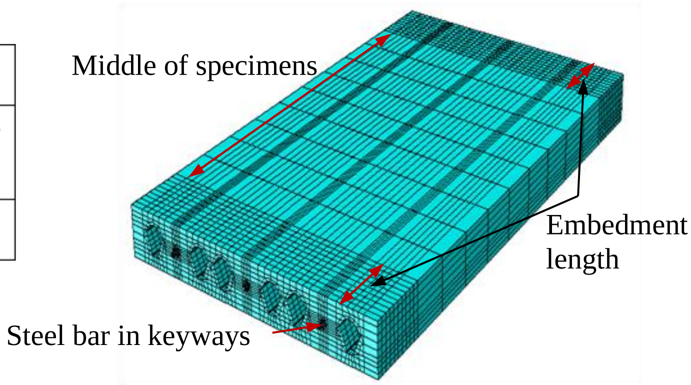


Fig. 6 Close-up view of the mesh configurations of slab

5.2 Mesh description

Both concrete and steel were modeled by the 8-node solid element with linear reduced integration. The fine meshes in reinforcement elements result in a smooth interaction at the interface between steel and concrete (**Fig. 6**). In this study emphasis has been placed on the high level of reliability and accuracy of the model rather than the efficiency of calculation. The model was discretized in such a way that the mesh density varied at different locations where the stress distributions were different.

5.3 Boundary condition

The boundary conditions for both systems are shown in **Fig. 7**. The lateral stiffness of side supports is simulated using 6 springs/slab and the side reinforcement bars were assumed to be fully bonded to the supporting slabs (**Fig. 7**). It is assumed that the lateral forces due to arch action or catenary action are transferred directly to the adjacent shear walls parallel to the longitudinal axis. Due to symmetrical boundary conditions, only one-fourth of specimens with longitudinal and transverse ties are taken into account.

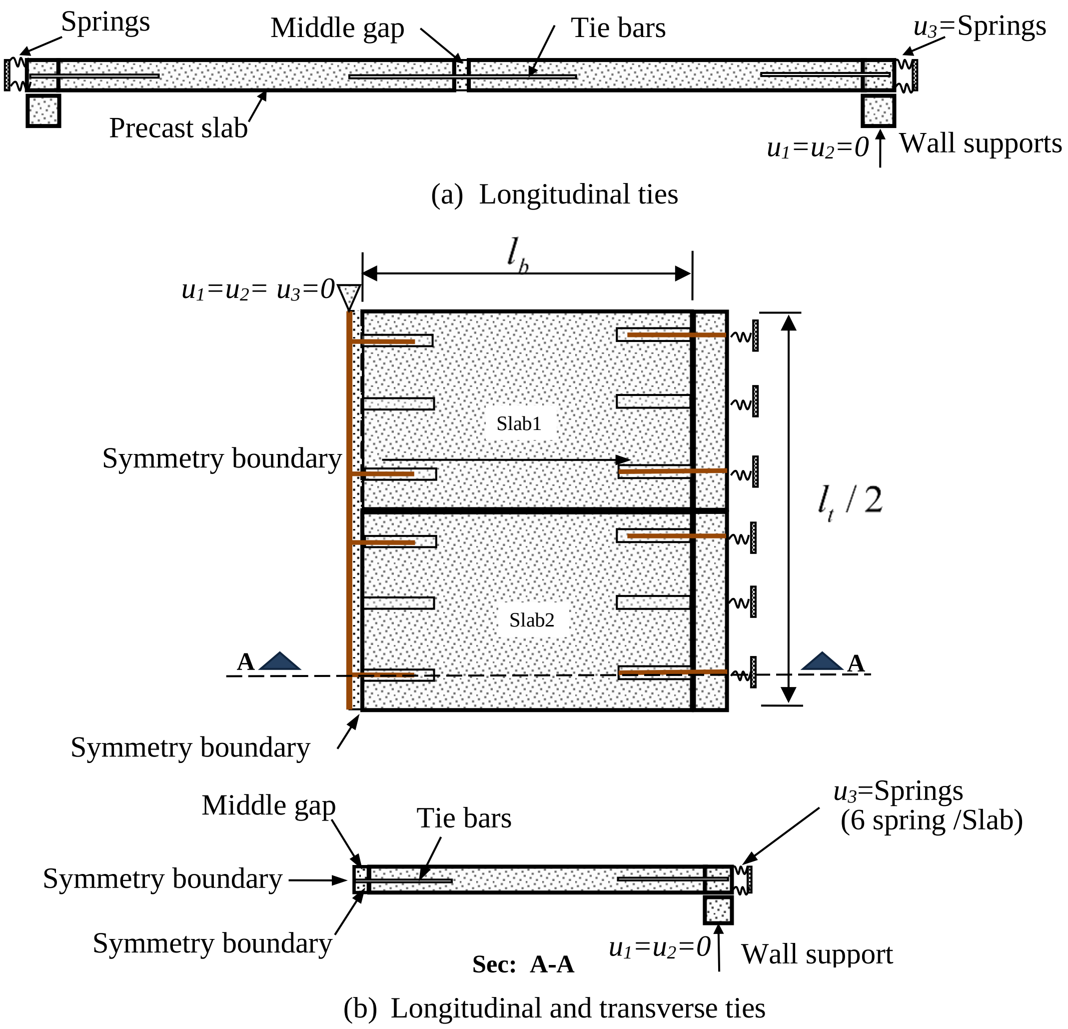


Fig. 7 Boundary condition

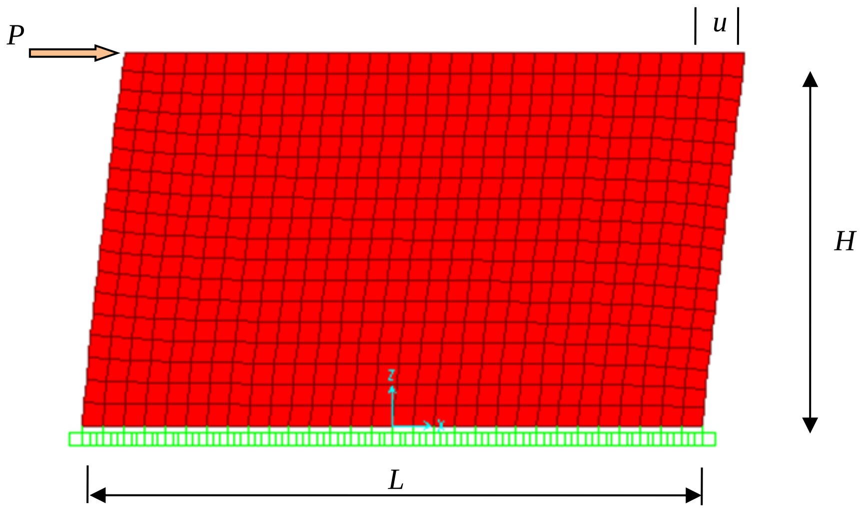
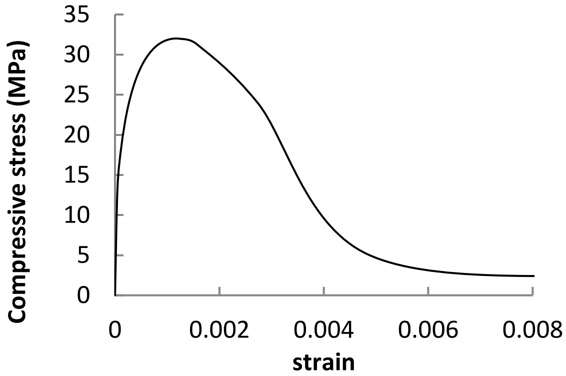


Fig. 8 Stiffness of shear walls, L=2m, 4m and 6m - H=3.5 m

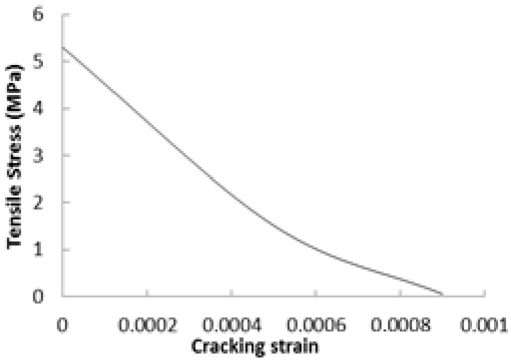
To define lateral spring stiffness, it is, also, assumed that only the lateral wall directly close to the system provides the lateral support, and the contribution of the other walls is neglected. According to SAP 2000 analysis, the stiffness of shear walls with a length of 2m, 4m, and 6m is 691E6N/m, 1925E6 N/m, and 3245E6 N/m, respectively (**Fig 8**). The thickness of the shear wall was assumed to be 250 mm. The lateral supports are simulated using six springs/slabs at each support. Assuming a cross wall of 2, 4, and 6 m as lateral support, the corresponding spring stiffness will be 12E7 N/m, 16E7 N/m, and 18E7 N/m, respectively. As the response of the system was not considerably sensitive to a small alteration of spring stiffness, in the following analyses the spring stiffness of 15E7 N/m was applied for all specimens.

Table 2 Materials' Properties [33]

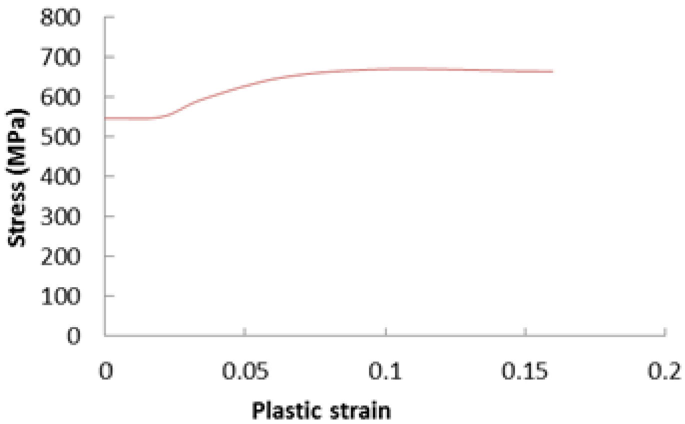
	Elastic Modulus (GPa) (N/m^2)	Specimen	Poisson's Ratio	Density (kg/m^3)
Concrete	30.5 E09	all	0.2	2500
Steel	210.0 E09	all	0.30	7800



(a) Compressive hardening-CDP Model

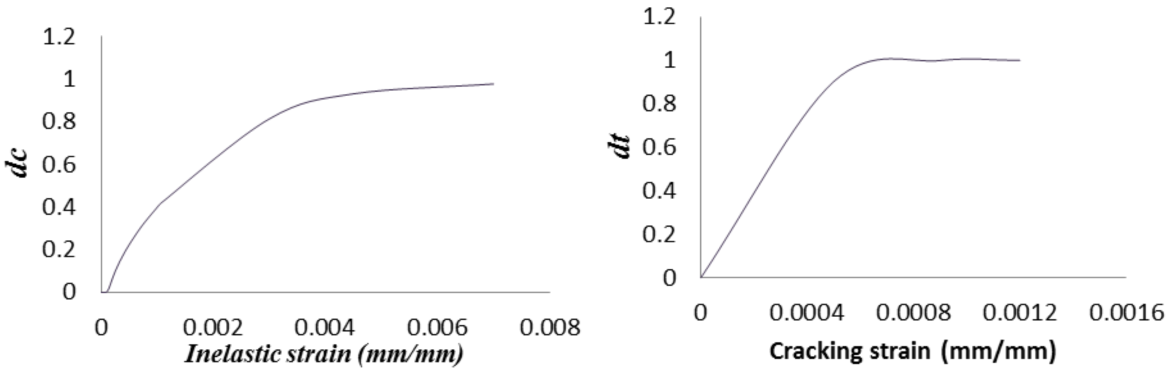


(b) Tension stiffening-CDP Model



(c) Plastic model-reinforcement bar

Fig. 9 Properties of materials in CDP and plastic model [43]



(a) Compression (b) Tension

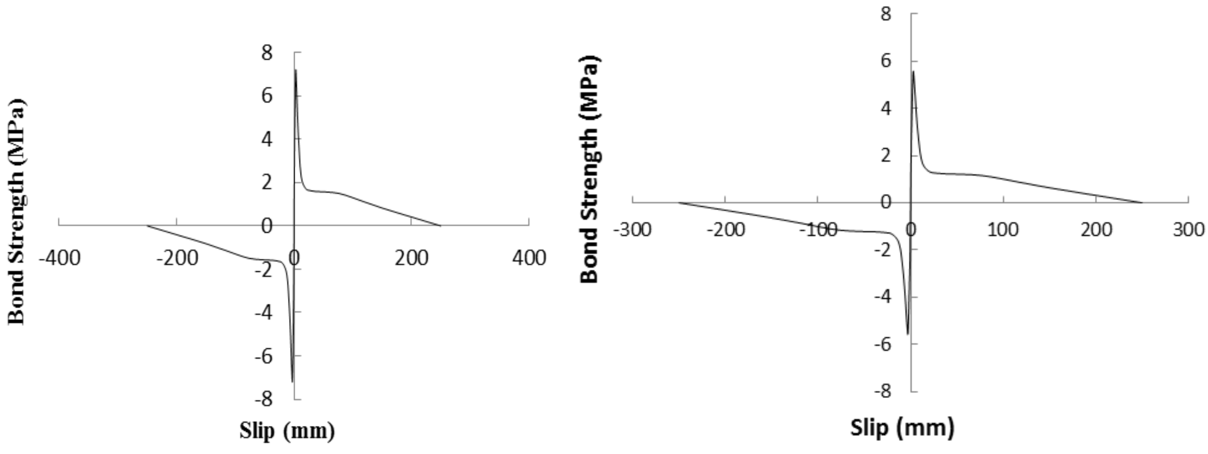
Fig. 10 Damage parameters for the CDP model [43]

5.4 Material properties

The concrete was modeled using concrete damage plasticity (CDP) available in ABAQUS, which can be used for both static and dynamic analyses and in all types of elements. Also, the plastic model was used to model the nonlinear behavior of reinforcement bars. The input data for the CDP and plastic model in ABAQUS in the form of the stress-strain curve for tensile or compressive behavior of concrete and reinforcement bar in different specimens is displayed in **Fig. 9** and the relevant elastic material properties of specimens are shown in **Table 2**. The damage variables of d_c and d_t considering the compressive and tensile strength of various specimens are shown in **Fig. 10**.

5.5 Translator Property

According to the pull-out test results [33] the bond-slip relationship was defined as shown in **Fig. 11**. The translator force is defined using the bond model (**Fig. 11**) as $F_{TR} = Csu/n$, where F_{TR}, C, s, and n are the translator force, the circumference of the bars, the translator spacing, the bond strength, and the number of translators in each section, respectively.



(a) Bar fracture failure mode (b) Pull-out failure mode

Fig. 11 Bond-slip relationship to define translator property [33]

6. Analysis solution strategy

The translator element is only available in ABAQUS/Explicit, and the contact condition and other discontinuous problems can be readily formulated in the Explicit module. Hence it is used in this study to perform a non-linear quasi-dynamic analysis.

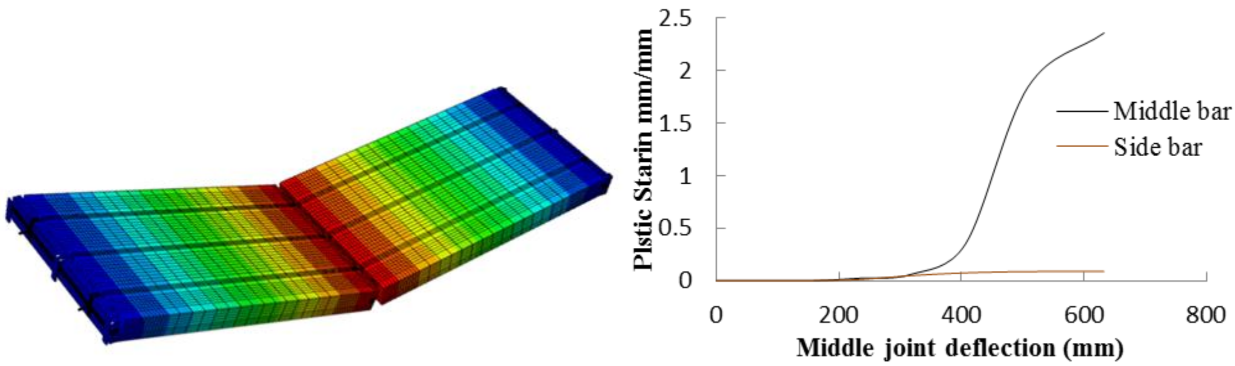
7. Wall support removal analysis

According to the embedment length of tie bars into keyways of the precast slab, in a floor-to-floor joint two kinds of collapse mechanisms; (1) bar fracture, and (2) pull-out failure mode governs the behavior of this system. To develop a generic analysis and design guideline, both failure modes are taken into account; considering various slab lengths and different numbers of longitudinal ties at the joints. In the first set of analyses only longitudinal ties into keyways of precast slabs are taken into account, followed by a series of 3D analyses considering both longitudinal and transverse ties at the joints with different slab lengths and bar sizes.

8. FEA Results

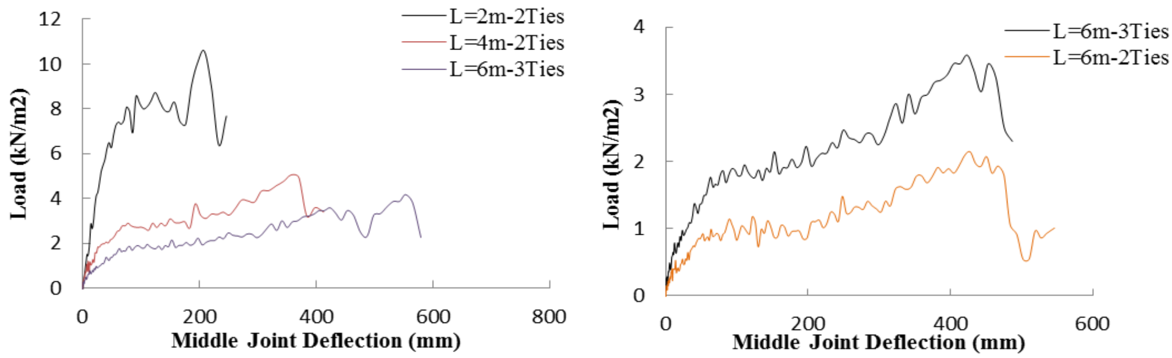
8.1 Floor-to-floor system using longitudinal ties

To study the behavior of a floor-to-floor system and to provide an initial data set about the contribution of ties in progressive collapse resistance, three specimens with slab lengths of 2, 4, and 6 m are analyzed (**Fig. 12**). To make results manageable, in all specimens the translator's properties, bar size, and embedment length are kept constant and only the slab length and load is taken into account as a variable. **Fig. 13a** shows that, at the failure, the strength of the system is relatively in proportion to the slab length; which confirms the TF method's provision in the codes of practices. The result indicates that the system collapses by bar fracture of the tie bars at the middle joint and at a deflection/slab length ratio of around 9% i.e. $\delta_s / l_b = 9\%$. Furthermore, **Fig. 13** shows that the second peak strength to initial strength ratio is around 1.4 which is less than the reported experimental study on the conventional RC structures [26, 31].



(a) Deflection of specimen (b) Plastic strain vs. middle joint deflection

Fig. 12 Failure mode



(a) various slab lengths and ties at the joints

(b) slab length of 6 m with 2 and 3 ties at the joints

Fig. 13 Load versus middle joint deflection in floor-to-floor system

Fig. 13b shows that the strength of the specimen with 3 ties is more than the specimen with 2 ties by 67%; which is slightly higher than the rate of increasing the number of ties. It is agreed extremely well with the full-scale test result [33]. In practical analysis and conservatively it can be assumed that the progressive collapse resistance of a floor-to-floor system is increased in proportion to the number of ties or area of the tie bars. Furthermore, the results indicate that the collapse was initiated by bar fracture at the middle bars (**Fig. 12b**), which shows the same behavior as the experimental study [33].

Experimental studies on RC structures indicate that following column removal and before catenary action, top bars at the middle joints and bottom bars at the supports are under compression, while following large deflection they are under tension and contribute to the catenary action mechanism [30, 31]. To investigate the contribution of top bars at the middle joint and bottom bars at the supports, one specimen with a length of 2 m is analyzed with a new arrangement of tie bars (**Fig. 14**). The result shows different behavior with conventional RC structures.

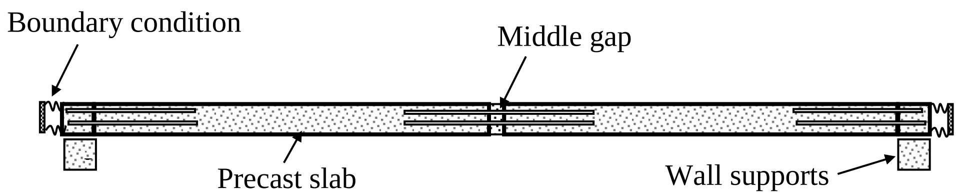


Fig. 14 Floor-to-floor joint with top and bottom tie bars

Comparing **Fig. 13a** and Fig. **15** indicates that the behavior of the system with top and bottom bars is similar to the specimens with one bar layer at the joints up to the plastic stage, followed by an increase in the strength of the system up to failure by 50%. It can be explained based on the strain and stress distribution along the depth of the slab. As the cross-section of reinforcement bars is relatively small, the natural axis is very small i.e. < 3mm. Furthermore, the top bar is located 35mm from the top of the slab, hence the top bars at the middle joints do not experience any compressive force and they will be under tension up to failure with different rates (**Fig. 15**). The result indicates that the system collapsed at $\delta_s / l_b = 11\%$, which is slightly higher than specimens with bottom bars only.

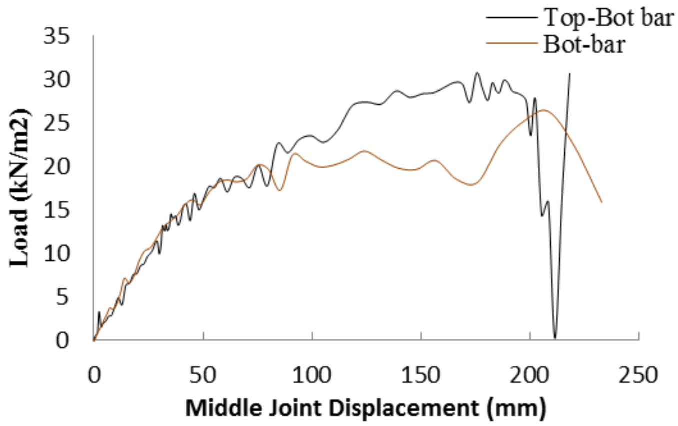


Fig. 15 Load versus middle joint deflection for slab length of 2 m using tie bars at the top and bottom bars and only tie bar at the bottom of the joint

8.2 Floor-to-floor joint analysis using longitudinal and transverse ties

To make the results manageable, in all analyses the cross-section area of the reinforcement bars in the longitudinal and transverse directions is assumed to be relatively identical. Furthermore, the translator property, embedment length, and compressive strength of the specimens are kept constant and the same as the specimens with longitudinal ties.

Prior to removing the wall support, the floor-to-floor system acts as a one-way slab; while following the removal of wall support the behavior of the system considering longitudinal and transverse ties approximately represents two-way slab behavior.

Fig. 16 shows the failure mode of specimen LFT1. **Fig. 17** indicates that the system reached maximum capacity at $\delta_s / l_b = 8.4\%$. The result, also, shows that at the deflection around 200 mm the strain is more than the yield or fracture strain in all bars, hence it can be considered as the failure point i.e. $\delta_s / l_b = 10\%$ and $\delta_s / l_t = 8.3\%$ (**Fig. 18**). It shows the same ductility as the specimens with the longitudinal ties only (**Fig. 13a**). The flat behavior in **Fig. 18** shows the redistribution mechanism of load.

Fig. 18 indicates that prior to the collapse and in the ascending stage, the load sustained by the longitudinal ties is more than the load sustained by the transverse tie; while before failure, the load is redistributed and both ties sustain identical load. The progressive failure of tie bars is rated from 1-5 and displayed in **Fig. 19**.

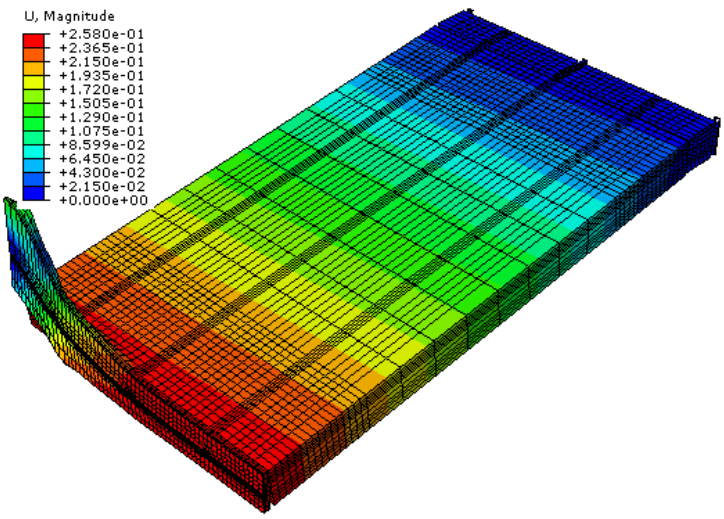


Fig. 16 Failure mode of specimen-LFT1

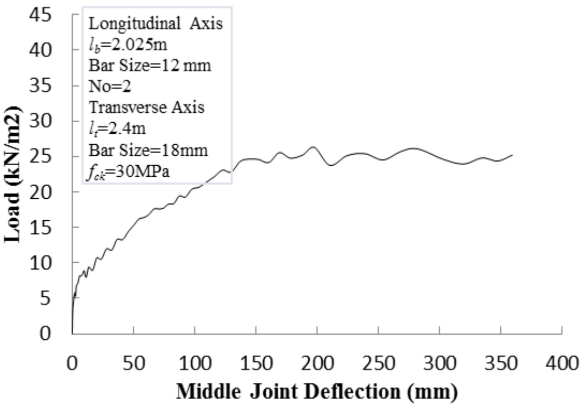


Fig. 17 Load versus middle joint deflection - LTF1

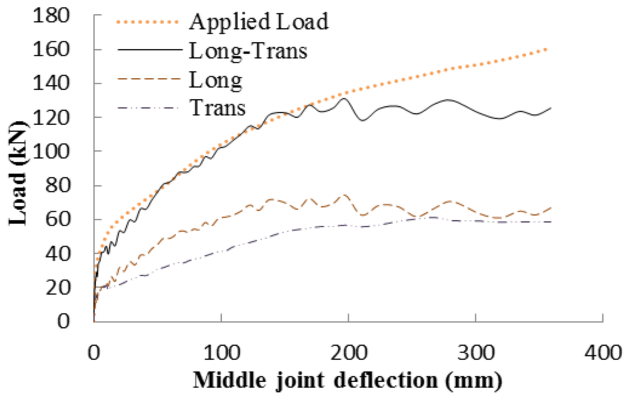


Fig. 18 Load sustained by longitudinal and transverse ties versus middle joint deflection - LTF1

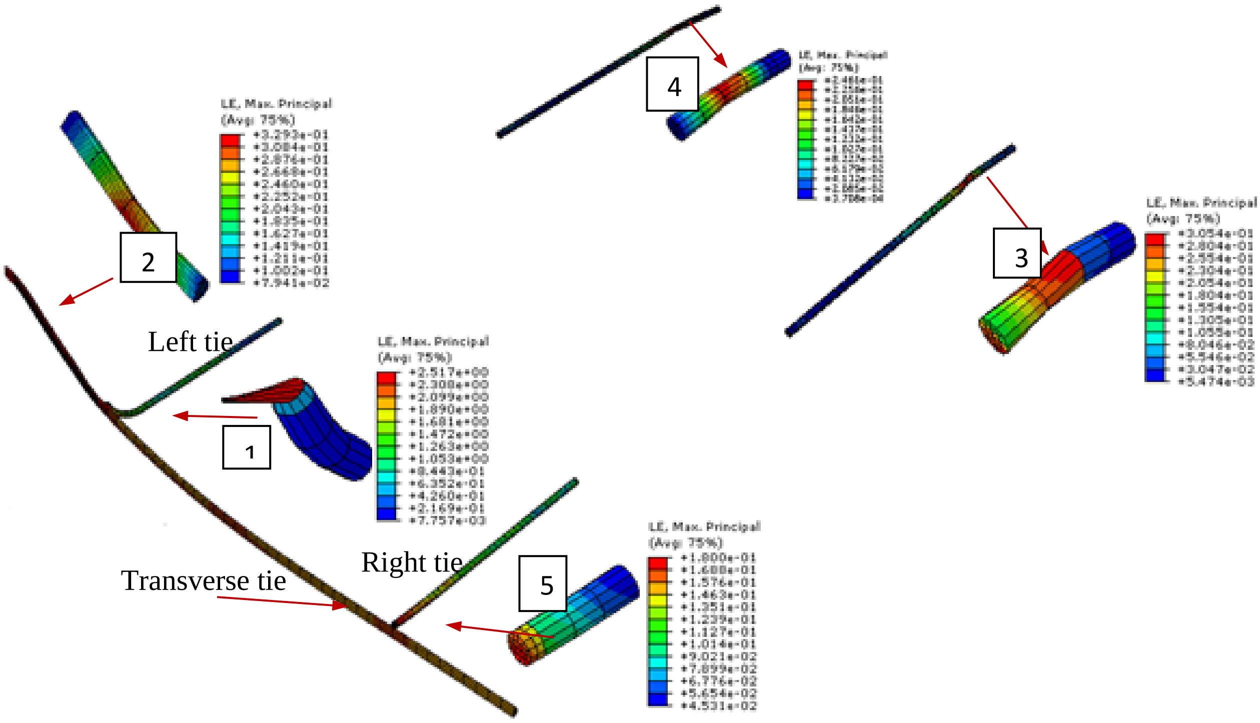


Fig. 19 Progressive failure in the longitudinal and transverse ties; strain in different ties at middle joint deflection of 455 (LFT1)

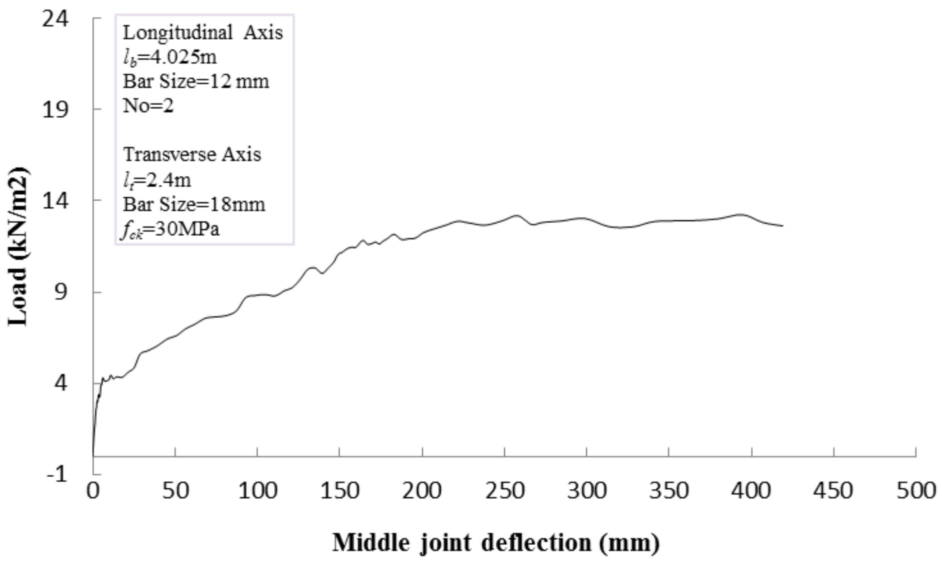


Fig. 20 Load versus middle joint deflection - LTF2

The strength of the specimen with a slab length of 4 m and width of 2.4 m is shown in **Fig. 20**. The tie properties remain the same as specimen LFT1. The results indicate that the progressive failure follows the same failure mechanism of LFT1 and the system collapses at a deflection around 385 mm i.e. $\delta_s / l_b = 9.6\%$ and $\delta_s / l_t = 16.05\%$. Comparing **Fig. 20** and **18** indicates that the progressive collapse resistance of specimen LFT2 is approximately half of that of LFT1. As the same property in the transverse direction was applied in both specimens, it can be concluded that the strength of the system is in inverse proportion to the slab length in the longitudinal direction.

To study the effect of the properties of the transverse tie on the behavior of the system, in specimen LFT3, the transverse tie with a length of 4.8 m and bar size of 24 mm was used; while the properties of the specimen in a longitudinal direction remain the same as LFT2. The failure mode of the specimen is shown in **Fig.21**, which indicates that at the collapse the maximum deflection of the middle slab is more than four times the maximum deflection in the side slabs.

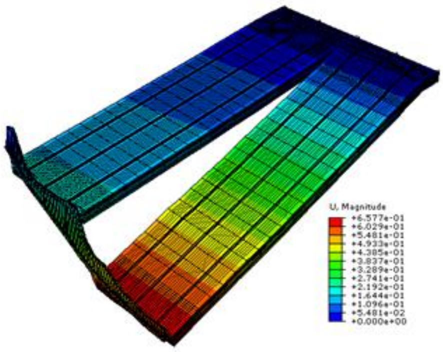


Fig. 21 Failure mode of specimen LFT3

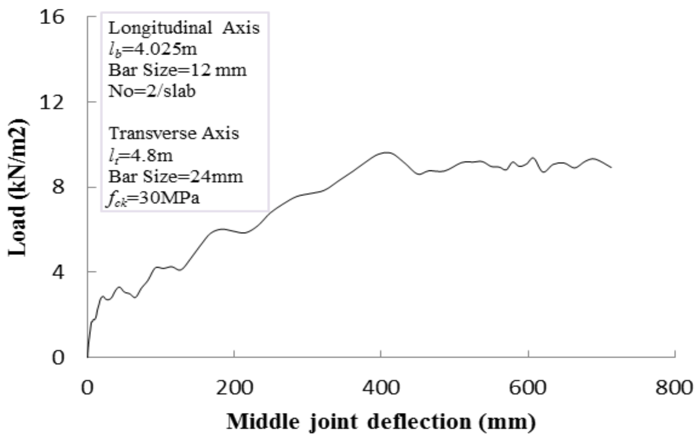


Fig. 22 Load versus middle joint deflection - LTF3

Fig. 22 shows that the strength of the system is less than specimen LFT2 by 23% and maximum capacity is induced at $\delta_s / l_b = 10\%$ and $\delta_s / l_t = 8.3\%$, which is similar to specimen LFT1. It indicates that to provide the same strength, the cross-section of the tie needs to be increased in proportion to the length of the specimens in the corresponding directions. The result of the stress-deflection relationship indicates that the system collapsed at a deflection of around 550 mm i.e. $\delta_s / l_b = 13.67\%$ and $\delta_s / l_t = 11.45\%$ which is slightly more than other specimens.

According to the TF method, the tie forces are increased in proportion to span length; hence in specimen LFT4 with a longitudinal length of 6 m the number of ties is increased by 50% compared to LFT3 i.e. 3 ties/slab, but the same cross-section of the transverse tie was applied, which is less than the cross-section are of the longitudinal ties by 50% .

Fig. 23 indicates that similar to other specimens at the yielding stage the load sustained by the longitudinal ties is higher than the transverse ties by more than 50 %, while at certain deflection ties in both directions carry the same load. **Fig. 24** indicates that the system reached maximum capacity at $\delta_s / l_b = 8.33\%$ and collapsed at $\delta_s / l_b = 13.61\%$ and $\delta_s / l_t = 17.08\%$, which shows slightly more ductility than other specimens.

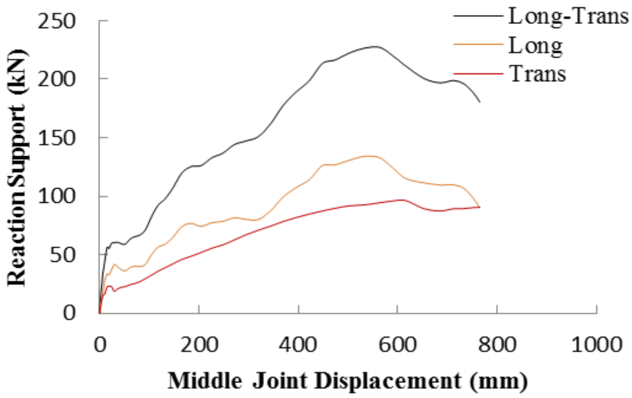


Fig. 23 Reaction supports versus middle joint deflection - LTF4

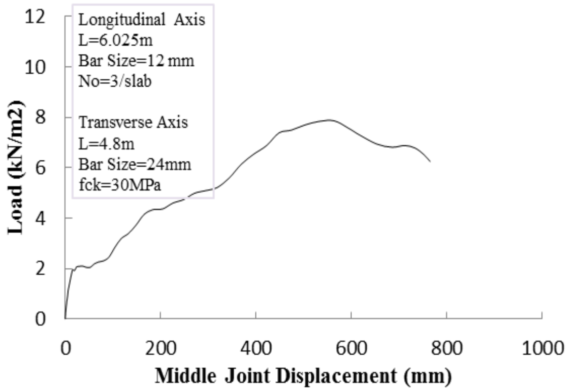


Fig. 24 Load versus middle joint deflection - LTF4

Comparing results between LTF3 and LFT4 indicates that, with the same properties of transverse ties, the strength of the system is decreased by 25% (**Fig. 22** and **Fig. 24**), hence it can be concluded that to provide the same level of progressive resistance, the cross-section of both longitudinal and transverse tie needs to be increased in proportion to the length of the longitudinal ties.

In the specimen LTF5, the span in the transverse direction is increased by 50% compared to LTF4, hence to provide relatively the same tie cross-section/span length ratio the bar size of 36 mm is used as a transverse tie. **Fig. 25** shows that the progressive collapse resistance of specimens reaches its maximum capacity at a deflection/length ratio of around 8.1% and collapses at $\delta_s / l_b = 13.27\%$ and $\delta_s / l_t = 11.05\%$, which is slightly less than deflection/span length ratio of the LTF4. Comparing **Fig. 24** and **Fig. 25** again confirms that as long as the cross-section of the ties is increased in proportion to the length of the specimens in the relevant direction, the specimens are capable of providing the same capacity.

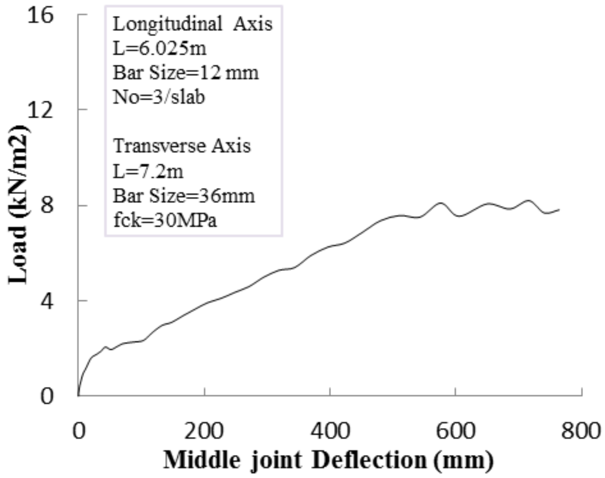


Fig. 25 Load versus middle joint deflection - LTF5

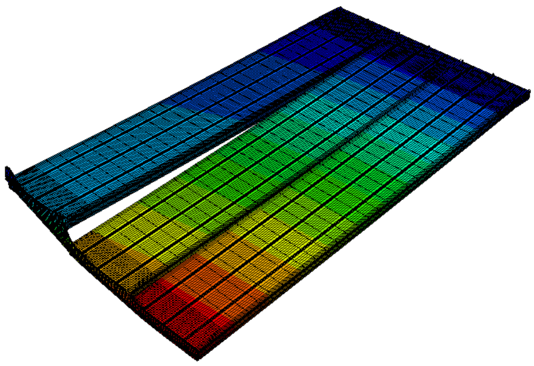
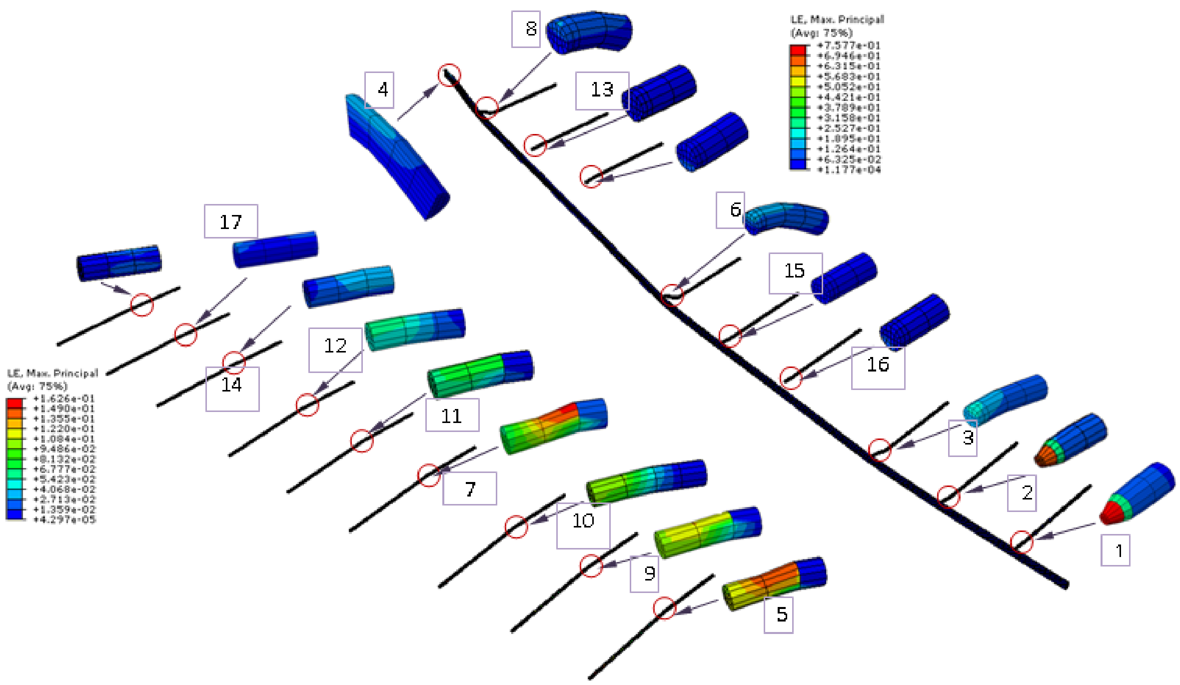


Fig. 26 Failure mode of specimen LFT5, $\delta_s / l_b = 8\%$



(a) Side ties (b) Middle ties

Fig. 27 Progressive collapse mechanism, $\delta_s / l_b = 8\%$ -LFT5

The overall failure mode of the specimen at the collapse is shown in **Fig. 26**. The result indicates that the failure mechanism is initiated by yielding in the middle longitudinal tie at the right-hand side of the middle gap **(Fig. 27)**. Also, as indicated in the specimen LFT1, the transverse tie started yielding underneath the longitudinal tie close to the side transverse tie at the support3, while in this specimen the yielding in the transverse tie is initiated at the top of the bar at the left support. The result indicates that the specimens with different tie arrangements show slightly different progressive failure mechanisms, but generally, it is initiated from longitudinal at the middle joint, followed by bar fracture of transverse and side ties.

Considering the maximum failure force corresponding to a strain fracture of 11% [33] the relationships between the tie force with load and slab lengths following over strength factor of 1.25 (DoD 2013) for each specimen is summarized in **Table 3**.

Table 3 Tie force based on bar fracture failure mode

	LTF1	LTF2	LTF3	LTF4	LTF5
$P_l\ (kN/m)$	$1.95wl_b$	$1.92wl_b$	$2.6wl_b$	$3.2wl_b$	$3.16wl_b$
$P_t\ (kN)$	$1.37wl_tl_b$	$1.36wl_tl_b$	$1.65wl_tl_b$	$1.33wl_tl_b$	$1.99wl_tl_b$

9. Improved tie-force method

The result of the tie force in the longitudinal and transverse ties based on the present study for bar fracture (**Table 3**), BS 8110-11(1997), and DoD 2013 is shown in **Fig. 28**. The dead and live loads are assumed to be 6.5 kN/m^2 and 3.5 kN/m^2, respectively. According to DoD 2013, the load combination of $2(1.2DL+0.5LL)$ is considered to calculate tie force in the present study and DoD 2013 and $DL+LL$ is applied in BS 8110-11(1997). Where DL and LL are dead and live load, respectively. **Fig. 28** shows that the result of the present study and DoD 2013 for specimens with a span length of less than 5 m is relatively conservative but for the specimens with a span length of more than 5 m underestimate the tie force. Furthermore, the results indicate that the load excreted from the upper floor has not been taken into account by DoD 2013 [16]. Also, the result shows that the current BS 8110-11(1997) [13] and BS EN1991-1-7 (2006) [19] significantly underestimated the tie force requirement.

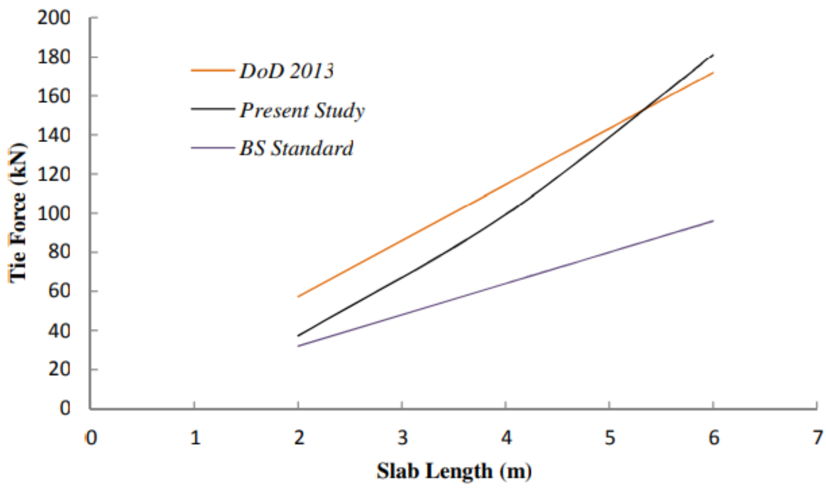


Fig. 28 Tie-Force versus slab length

The result of the analysis for bar fracture failure indicates that the relationship between tie forces in the longitudinal tie can be considered linear up to a slab length of 5m; while a high level of nonlinearity exists with the increasing of the slab length. According to the results, a nonlinear relationship between longitudinal tie force, load, and slab length for bar fracture failure mode can be defined as follows:

$$P_l = 1.39wl_b^{1.46} \tag{1}$$

Where:

P_l = Longitudinal tie force (kN/m)

$w =$ Load combination according to DoD 2013, kN/m^2; $w = 1.2D + 0.5L$
$D =$ Dead load (kN/m^2)
$L =$ Live load (kN/m^2)
$l_b =$ Slab length in the direction of longitudinal ties (m)

Table 4 The percentage of increase of the line load with the number of storey (**α**) [13]

storey	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
α%	0	17	33	50	67	83	100	117	133	150

To take into account the effect of load exerted from upper floor, Eq. (1) is modified when considering the current TF provision in BS 8110-11(1997) or BS EN1991-1-7 (2006).

$$P_l = 1.39(1+\alpha) w l_b^{1.46} \qquad \text{(Eq. 2)}$$

Where:
$\alpha =$ Percentage increase of the line load considering the number of stories (see **Table 4**)
Similar to the longitudinal ties, for the transverse ties a linear relationship exists for a span length of less than 5m; while the tie force is rapidly increased for a span length of more than 5m; which indicates that the tie force in both directions follows the same behavior. According to the FE result, the relationship between force in the transverse tie with load, slab length and span length may be proposed as follows:

$$P_t = 1.1(1+\alpha) w l_b l_t^{1.32} \qquad \text{(Eq. 3)}$$

Where:

$P_t =$ Transverse tie force (kN)

$l_t =$ Span length in the direction of transverse ties (m)

10. Conclusion

The overall behavior of specimens in bar fracture failure clearly indicates that up to yield capacity, the load sustained by longitudinal ties is higher than transverse ties by around 35-50%; while at the collapsing stage, both longitudinal and transverse ties carry the same load which indicates the redistribution of loads prior to collapse. The results show that the systems reach yield capacity at $\delta_s / l_b \approx 2\%$ and maximum strength is induced at $\delta_s / min(l_b, l_t) \approx 8\%$, followed by sudden collapse at $\delta_s / min(l_b, l_t) \approx 10 - 13\%$, which indicates that a short length of span in either longitudinal or transverse direction dominates the ductility of the system. The results indicate that, for specimens with the same length, a precast floor-to-floor system assuming bar fracture failure mode exhibits relatively less ductility compared to the conventional concrete structure.

The results show that, although the failure mechanism varies with the number of slabs in the direction of the transverse tie, generally the failure is initiated from the middle longitudinal ties followed by the side longitudinal ties at the middle joint. Subsequently, the transverse tie reaches its capacity followed by failure in the longitudinal ties at the side supports. The strength analysis of the specimens clearly indicates that following maximum capacity the progressive collapse resistance remains relatively constant up to the overall collapse of the system, which shows a redistribution mechanism. The result of load versus middle joint deflection indicates that the floor-to-floor joint exhibits similar behavior as to that of an RC structure with non-seismic detailing and low compressive strength [29]. The results, also, indicate that the strength of the system is in proportion to the length and cross-section area of longitudinal and transverse ties.

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