NUMERICAL SIMULATION OF FLOOR JOINT BEHAVIOR FOR PROGRESSIVE COLLAPSE RESISTANCE DESIGN IN PRECAST CONCRETE CROSS WALL STRUCTURES

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ABSTRACT

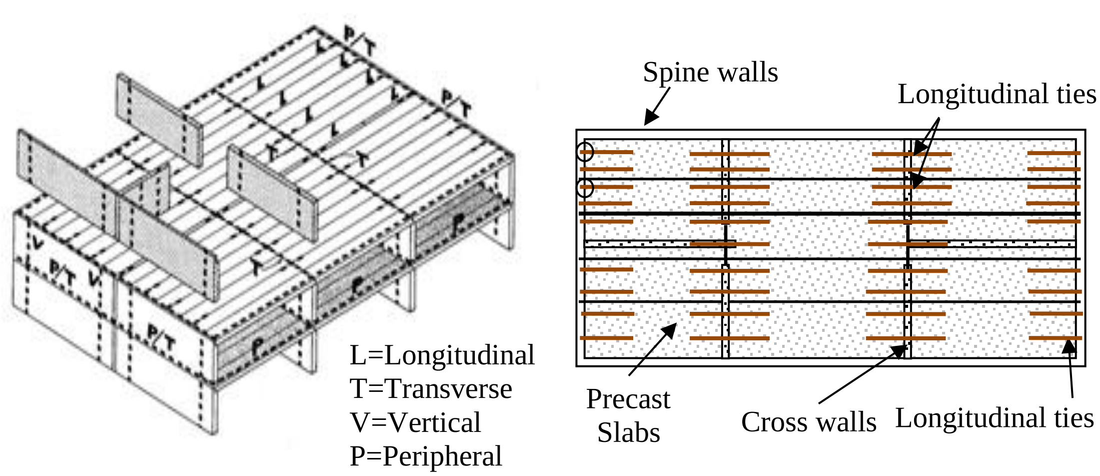
In the design of precast concrete cross wall structures, the prevention of progressive collapse often relies on four types of mechanisms, i.e. catenary, cantilever, vertical, and diaphragm actions that the undamaged structures can provide. The study reported in this paper includes a numerical simulation of the catenary action in concrete floor-to-floor assemblies. This load path is facilitated through longitudinal ties embedded in the cast in-situ grout contained in the keyways of floor slabs. A series of three-dimensional finite element models of the pullout behaviour of strand in keyways were developed. This was followed by another series of three dimensional non-linear analyses to simulate the behaviour of floor-to- floor connections in the absence of underlying wall support using the "alternate load path" method. To simulate the steel-concrete interaction, an element called as "translator", a type of interfacial connector built in ABAQUS, is employed. The numerical results were very close to the experimental results undertaken by Portland Cement Association (PCA). Discrepancies in the tie force between the numerical and codified specifications have suggested an underestimate of TF method; hence an improved TF method has also been proposed to address this deficiency.

Keywords: Cross wall, Progressive collapse, Tie force method, Alternate load path, Translator.

1 INTRODUCTION

As is defined by Portland Cement Association[1], the term of "large-panel" concrete structure is used to describe a structural system consisting of vertical wall panels together with precast concrete floors and roofs. Large panel buildings are featured as wall panels being used as the load-bearing structure. In the general arrangement, a wall that is perpendicular to the longitudinal axis of the structure is called as the cross wall and that is parallel to the longitudinal axis is called as the spine wall. In the cross wall system, floor/roof slabs are typically one way hollow core precast concrete slabs, and only cross walls are load bearing (Fig. 1).

Following the partial failure of a precast concrete building based in London, Ronan Point apartment, in 1968, the British Standards[2] for concrete structures started to incorporate provisions to deal with the problem of progressive collapse. Portland Cement Association[1] conducted a series of comprehensive investigations to form an underpinning knowledge basis supporting the stipulated minimum detailing requirements to ensure the development of an alternative load path (ALP) in the event of any local damage. These attempts led to a tie-force (TF) design method adopted in British Standard, for the first time, being known in the world. This method is mainly of prescriptive nature that requires the inclusion of internal, peripheral, and vertical ties to provide different "alternative load paths", e.g. catenary, cantilever, vertical and diaphragm actions, in a loss of underlying wall support (Fig. 1). These prescriptive tie requirements may have proven adequate in engineering practice but are not scientifically justified, so substantial efforts are still needed to improve the understanding, at a fundamental level, of how the post-collapse resistance mechanism are developed through these tie provisions. This need has also been echoed by a number of researchers in the last decade.



(a) General arrangement of ties (PCA, 1979)

(b) Plan view and longitudinal ties

(c) Precast cross wall structure

(d) Examples of longitudinal ties (Courtesy of Bison Manufacturing Limited)

Fig.1 Examples of precast concrete wall construction

Dusenberry[3] indicated the necessity of a better understanding of the mechanism how the progressive collapse can be resisted. To show the adequacy of codified methods for the progressive collapse, an evaluation on three well known collapsed building cases was performed by Nair[4] based on five current codes and standards. Results revealed that almost all three studied structures are susceptible to progressive collapse. Abruzzo et al.[5] has also indicated an inadequacy of the TF method to prevent progressive collapse of structures. The necessity of developing an improved TF method has also been recommended by DoD[6]. To investigate the efficiency of TF design method, Li et al.[7] 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.[8]. The numerical results revealed that the current tie force 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.

It is to be noted that the present TF method has not taken into account the effect of bond behaviour of tie bars and the surrounding grout. Such behaviour is influenced by many factors such as strand-grout interface characteristics, stress-slip relationship and the material properties of tie bars/strands e.g. diameter, elastic modulus, and embedment length. Accordingly, it can be considered as an overly simplified method.

Since bond is the key factor in the analysis and design of RC structures and it governs most RC performances, not just progressive collapse, reliable and viable bond behaviour modelling remains a challenging issue. To date, a large body of work to simulate the bond-slip behaviour has been presented in the published literatures, but for the post-bond-behaviour and the mechanism of forming catenary action in relation to the bond behaviour in precast cross wall structures is still limited. The first step of the present study is to fill up this gap. To this end, a 3D reinforcement concrete model with a spring element and a contact surface appears to be the best approach.

Due to high cost of full scale experimental studies, a reliable numerical modelling by using FE software packages provides an ideal alternative to extend the current knowledge on the behaviour of floor-to-floor joint system following the removal of wall support and to identify the key influencing factors. The aim of this study is to develop a computer FE model with a particular attention to the post bond-failure behaviour of tie bars/strands in the floor-to-floor joints of cross wall structures considering these influencing factors and use the obtained results to evaluate the adequacy of current TF method as recommended by most codes of practice. To this end, a 3D reinforcement concrete model with a spring element and a contact surface appears to be the best approach. Two types of modelling are carried out, i.e. the pullout performance of the ties in grout and the floor-to-floor joint subjected to a uniform and line load exerted by upper walls. The results obtained from the modelling will reveal the full history behaviour of the pull-out force against slip relationship including the pre-failure phase, the ultimate load, and the post-failure phase so that failure mode caused by high local slip can be identified.

2 Tie force method

The Tie force TF method requires that in each direction ties should be designed to carry a tensile force of P (kN/m) equal to the greater of the following two values (BS 8110-11, 1997):

$$P_1 = \frac{(g_k + q_k)}{7.5}\frac{l_b}{5}F_t \tag{1}$$

$$P_2 = F_t \tag{2}$$

where:

$(g_k + q_k)$ is the sum of the characteristic dead and imposed floor loads (in kN/m^2) ;

F_t is the lesser of (20 + $4n_o$) or 60 kN/m, where n_o is the number of storey;

l_b is the length of the floor span.

In implementing the TF method that is adopted in most codes or standards, an indeterminate structure is usually simplified to a determinate one by introducing hinges at connections, by which the minimum tie forces can be calculated. Based on the calculated results of tie force, sufficient tie arrangements are made to provide sufficient strength to establish overall structural integrity, continuity and redundancy. This method is suitable for the hand calculation and inevitably results are rather approximate. Recently, with the advancement of computer tools, an “alternative load path” method has become more popular. In this method, following the removal of a critical element, the structure should be capable of redistributing loads to the remaining undamaged structural elements.

3 Catenary action mechanism

According to the current code specifications, in order to prevent the progressive collapse for building structures, four types of alternative load path should be provided, i.e.

- catenary action of floor-to-floor system,
- cantilever and beam action of wall panels,
- vertical suspension of wall panels, and
- diaphragm action of the floor plans.

In this study, only the catenary action of floor-to-floor systems (Fig. 2) is considered, so it is assumed that all other load paths have been effectively provided. If an underlying wall support is suddenly removed due to an abnormal load, in order to bridge out the load exerted by the upper walls and hence retains the structural integrity, a continuity requirement at the floor-to-floor joints must be provided so that an alternative load path can be found (Fig. 2a). Unlike the normal service condition, a much larger deformation in the affected zone is allowed. Therefore, the ductility of these connections is essential to satisfy the deformation demand. In precast cross-wall constructions, these requirements can be facilitated by the tie strands/bars embedded in the cast in-situ grout placed in keyways and the side edge gap of floor slabs (Fig. 2b). After an underlying wall support is removed, the grout will be crushed immediately under the increased loads and these ties will experience tensile forces and develop large deflection in floor slabs. This process forms a catenary action mechanism.

An equilibrium equation of the catenary system can be derived by taking moments about the side support in the free body diagram of the half system shown in Fig. 2d.

$$F_l = \frac{(wl_b + \alpha q)b_p l_b}{2\delta_s} \tag{3a}$$

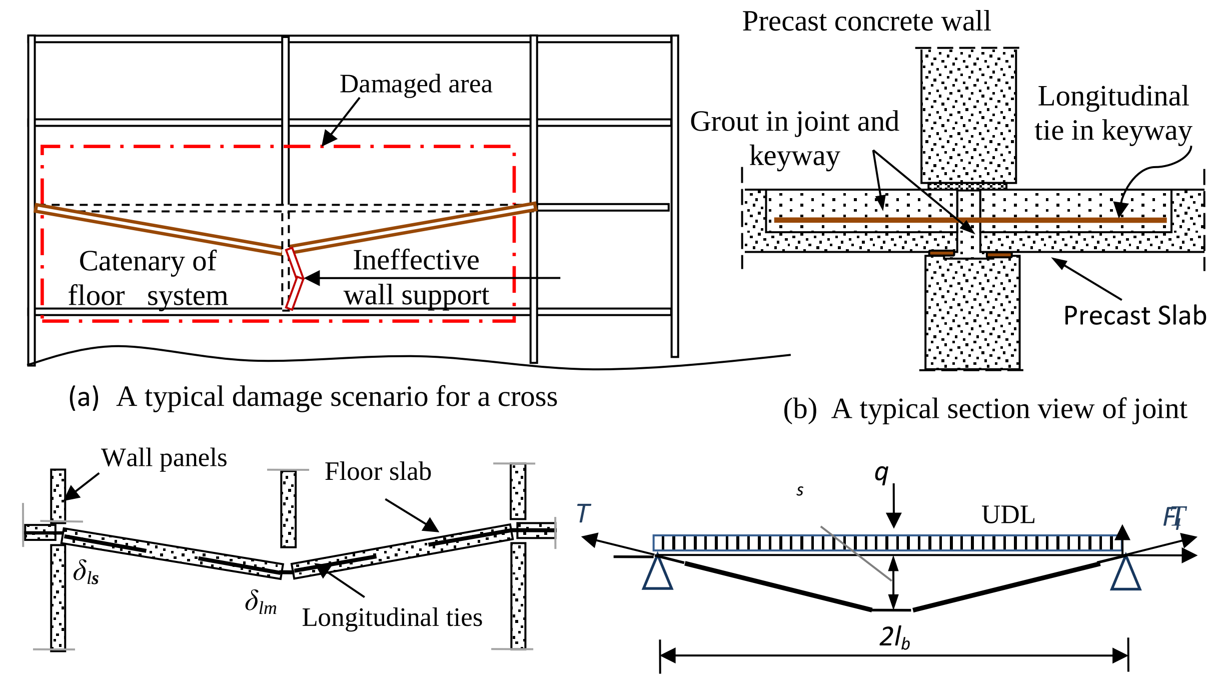
Let $q = wl_b$, $$F_l = (1+\alpha)\frac{wb_p l_b^2}{2\delta_s} \tag{3b}$$

where:

w	=	Uniformly distributed load (including dead and imposed loads)
b_p	=	Spacing of ties
l_b	=	Floor span length
F_l	=	Force in the longitudinal tie joining adjacent slabs
δ_s	=	Vertical displacement at the middle wall support
q	=	Line load exerted by the upper wall
α	=	Percentage increase of the line load considering the number of storey (see Table 1).

Table 1 The percentage increase of the line load with the number of storey (α) (BS 8110-11:1997)

Storey No.	1	2	3	4	5	6	7	8	9	10
α%	0	17	33	50	67	83	100	117	133	150



(a) A typical damage scenario for a cross

(b) A typical section view of joint

(c) Damage affected

(d) Idealized catenary system

Fig. 2 Catenary action mechanisms facilitated by longitudinal ties

Based on the compatibility condition of deformation in Figure 2d:

$$\delta_l = \sqrt{l_b^2 + \delta_s^2} - l_b^2 \tag{4a}$$

$$\delta_l = l_b \ \sqrt{1 + (\delta_s / l_b)^2} - 1 \tag{4b}$$

$$\frac{\delta_l}{l_b} = \frac{1}{2}\left(\frac{\delta_s}{l_b}\right)^2 \qquad\qquad if \left(\frac{\delta_l}{l_b}\right) \ll 1 \tag{5}$$

where δ_l represents the increase in the length of each floor slab, which consists of the extension of ties at both ends of the floor slab. If we use δ_{ls} and $\delta_{lm}$to represent the extension experienced at the side and middle supports of one of the affected floor slabs, we have

$$\delta_l = \delta_{ls} + \delta_{lm} \tag{6}$$

During the development and evolution process of the catenary system, the tie force will reduce with the increase of vertical deflection as indicated by Eq. (3). The increase in deflections facilitated by the extension of floor-to-floor joints including the elongation of tie strands and the slip of the strand out of the surrounding grout. The failure of the catenary system occurs when the extension reaches a certain level. The corresponding deflection at the joint has often been set as the failure criteria. At failure, the tensile force in the tie has usually reduced below the yield stress and therefore most extension is provided by the slip due to the pullout action.

To analyze a catenary action mechanism, the following assumptions are considered:

- The local damage, and thereof the initial failure of slab, will not affect the ability of system to develop a catenary action mechanism.
- After establishing the catenary action, a static behaviour of system can be assumed.
- In the event of the removal of the underlying support walls, sufficient transverse, vertical, and peripheral ties have been provided, so the whole structure remains stable.
- All extension demand is provided by the elongation of longitudinal ties and the slip between the longitudinal ties and surrounding grout.
- Adequate longitudinal continuity has been provided to establish catenary action for the floor-to-floor system.

4 FEA Modelling

PCA[1] conducted pullout tests for strands embedded in the grout filled in the keyways of 41 precast concrete blocks to study their pre- and post-bond behaviours[1]. The blocks were cut from the precast hollow core slabs and have the following dimensions, i.e. 1000 or 600 mm in width, 200 mm in height, and a variable length with an aim to study the effect of embedment length (see Fig. 3). Two strand sizes are considered in the test, i.e. 9.5 mm (3/8 in) and 12.7 mm (1/2 in). In order to seek an appropriate numerical approach to model the grout-steel interface, these tests were reproduced by using a commercial FEA package, ABAQUS (ABAQUS, 2006), as a preliminary phase of study. In this stage, 3D models of steel bar in the grouted keyways of concrete slab block were generated.

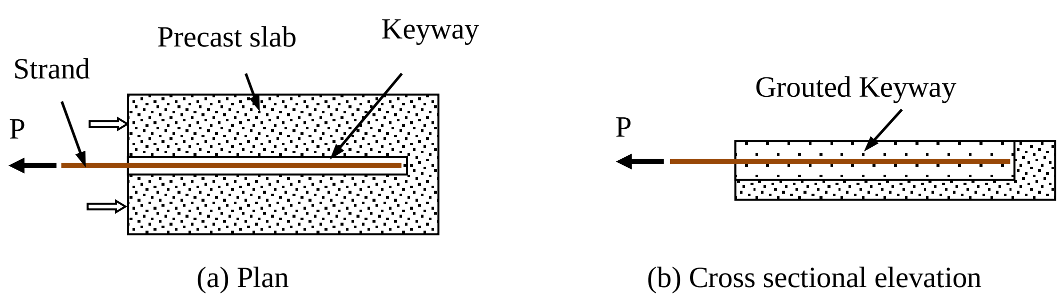


Fig. 3 Illustrative diagram of the pullout test

Once the modelling approach to treat the grout-steel interface has been verified by test results, it was also adopted in the modelling process of the full-scale floor-to-floor joint tests, also carried out by PCA[1]. In these tests, the systems consisting of two hollow core precast concrete slabs of full width, which were connected through two or three strands placed into two keyways (Fig. 4), was subjected to uniform and central line load to imitate the load exerted by upper walls. All longitudinal ties were seven wire strands, which are placed symmetrically into keyways in the middle and side joints.

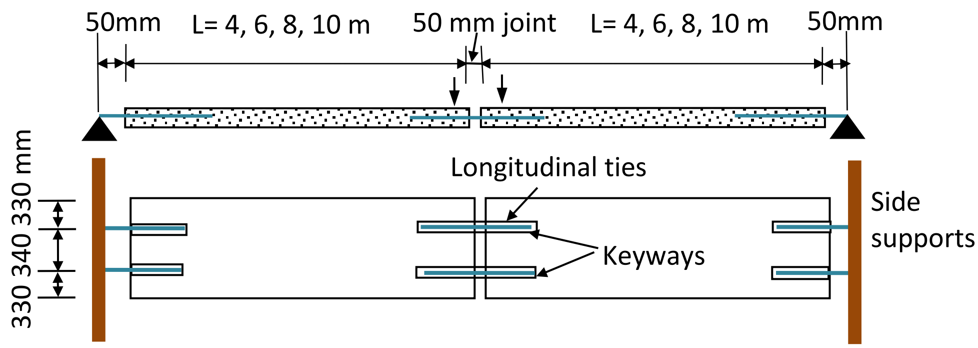


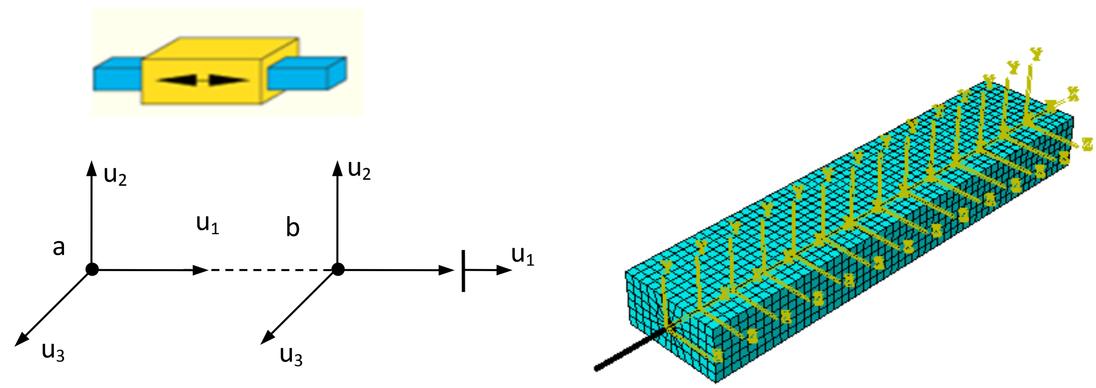
Fig. 4 Illustrative diagram of the full scale floor-to-floor joint tests

4.1 Modelling techniques for the bond-slip behaviour

Since bond is the key factor in the analysis and design of RC structures and it governs most RC performances, not just progressive collapse, seeking a technically reliable and economically viable bond modelling technique remains a challenging issue. To date, numerous research papers have been published on studying the bond-slip behaviour between the tie and the surrounding grout, a large proportion of which were carried out by numerical modelling. In these papers, a wide range of modelling techniques has been adopted to simulate such behaviour. Bresler and Bertero[9] first introduced a layer wise model. Since, in practice, bond only occurs in the concrete zone near the reinforcement surface, to distinguish the inelastic deformation and fracture damage in this zone from the bulk concrete, the concrete is divided into two zones: an inner boundary layer and an outer layer. It was assumed that both zones have a linearly-elastic isotropic behaviour but with different material properties. Reinhardt et al.[10] later introduced a “slip layer ” , which was divided into two layers with thicknesses equal to the bar diameter and the outer zone of concrete, respectively. The steel bar was assumed to be elastic. The nonlinearity of concrete layer was described by an elastic-softening law in the tension zone, and an elastic-plastic law in the compression zone. The chosen element for the steel bar can exactly represent the shape of ribbed bar. From the 80’s onward, a variety of new FE element types emerged, which were applied successfully to simulate the bond-slip relationship. Relevant work has been reviewed in by CEB-FIP[11].

An alternative treatment is to assume a negligible thickness of the interface layer, and thus bond problem fells into a category of “contact issue ” . A useful review was presented by Keuser and Mehlhorn[12] in respects of this type of work. In this group of models, the normal stress between the steel bar and concrete and the bond-slip behaviour was modelled by using a double spring with one movement in the longitudinal axis and the second in the perpendicular direction. The spring does not have dimension and the relevant stiffness is calculated based on the bond-slip characteristics. The bond strength is a structural behaviour rather than just a material property, and hence Darwin and McCabe[13] proposed a full scale reinforced concrete model and simulate the interface layer by using a 3D interface link, which acts as a contact-slip element.

It can be seen that a large body of numerical work to simulate the bond-slip behaviour up to the failure stage has been completed, but endeavours on the post-bond-behaviour and the mechanism of forming catenary action in relation to the bond behaviour is still limited. This type of scarcity will be addressed in the following study. To that end, a 3D reinforcement concrete model with a translator element and a contact surface appears to be the best approach. The translator element is a special type of FEA element that has been built in ABAQUS programmes. It has two nodes, which can be attached two substrates (see Fig. 5a). Like other types of contact elements, it can be assigned a force together with corresponding relative displacement between these two nodes. It also can receive slot constraints and align them in the local direction u_1 as shown in Figure 5a. This connector dictates kinematic constraints by combining connection types with the options of SLOT and ALIGNS.



(a) Translator element (ABAQUS 2006) (b) Implementation of translator element

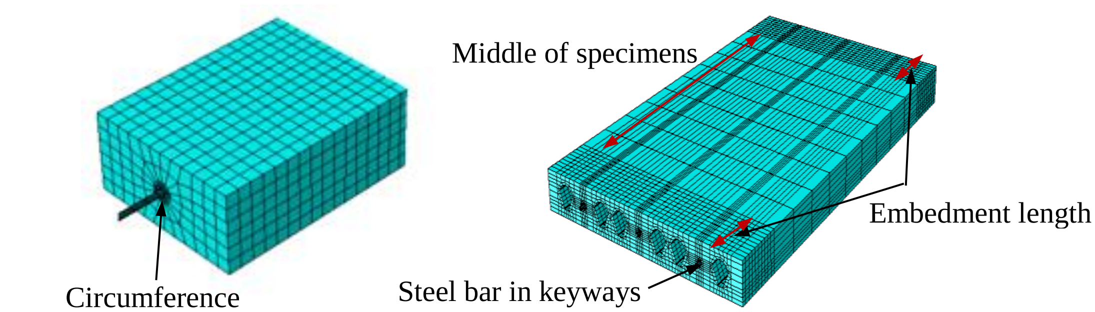
Fig. 5 FE Modelling of steel-concrete interaction

4.2 Mesh description and boundary conditions

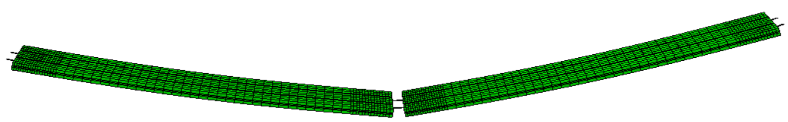
Both concrete and steel were modelled by the 8-node solid element with reduced-integration. The model was discretized in a way that the mesh density varies at different locations where stress behaviours are different. Three locations have been chosen to apply different mesh densities in the hollow core concrete labs, and they are the steel-grout interface, zones within the embedment length and the middle of the block (Fig. 6). A mesh size convergence analysis was carried out to determine the optimal meshing pattern. Table 2 presents three mesh trials with various mesh sizes at the circumference of the steel-grout interface, along the embedment length and the middle of the block. The results of slip and tie-force were examined for the convergence check. Table 2 indicates that meshing trials B and C yield very close results and hence trial B has been chosen for the following modelling work.

Table 2 Mesh properties for pullout and full scale models

Mesh trial	Number of element or mesh size				
	Circumference at interface	Embedment length	Middle of block	Slip ratio	Tie force ratio
A	8 elements	50 mm	150 mm	1	1
B	16 elements	25 mm	150 mm	1.09	1.08
C	32 elements	12.5 mm	150mm	1.10	1.08

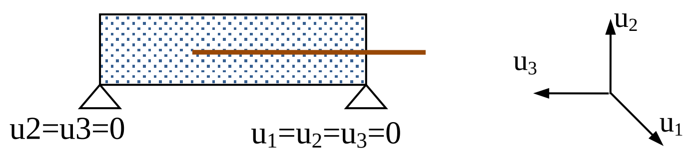


(a) Mesh configuration of a typical pullout model (b) Close-up view of the mesh configurations of floor-to-floor joint

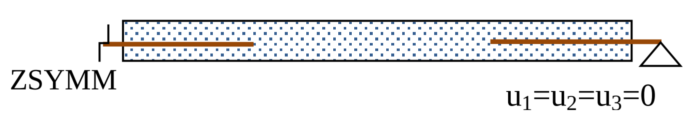


(c) Model of the floor-to-floor system

Fig 6 Finite element mesh pattern



(a) Model for concrete blocks



(b) Model for Full Scale floor- to-floor

Fig. 7 Boundary conditions for pullout and full scale model

The boundary conditions applied in the concrete block and floor-to-floor joint models are displayed in Figures 7 (a) and (b). In pullout case, only one degree of freedom of two end nodes remained free, i.e. the longitudinal movement for the left end node. In the latter case, the right end node remains the same but the middle point has been allocated a symmetry boundary conditions as only the right-hand side half is included in the model.

4.3 Material properties

As stated in section 3, the catenary action occurs at the post-bond-failure stage, at which both the concrete and steel reinforcement have been unloaded. As a result, the stress in both materials will be below the yield level, so elastic material properties were employed. One of the key challenges in the modelling is to define an appropriate and efficient bond-stress relationship. The damage initiation criteria and the damage evolution laws are also important to simulate the degradation behaviour of the bond-slip relationship.

To determine the non-linear property of the translator elements, the pullout test of concrete block tests were used to derived the force-slip relationship. The measured results from the test were pullout force and overall displacement. It was assumed that the stiffness for translator along the embedment length is uniform. According to the pullout test results and using four translators at the interface and with an interval of 100 mm along the embedment length, the translator properties were defined as shown in Figure 8.

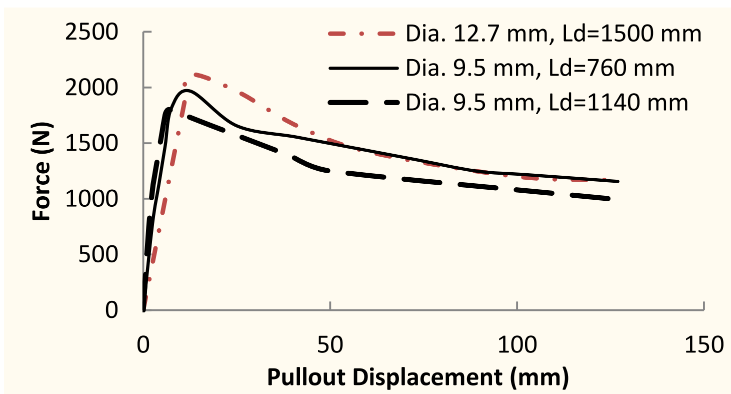


Fig. 8 Translator properties for different strand size and embedment length, four translators at the circumference/100 mm along embedment length

4.4 Analysis solution strategy

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

4.5 Verification of models

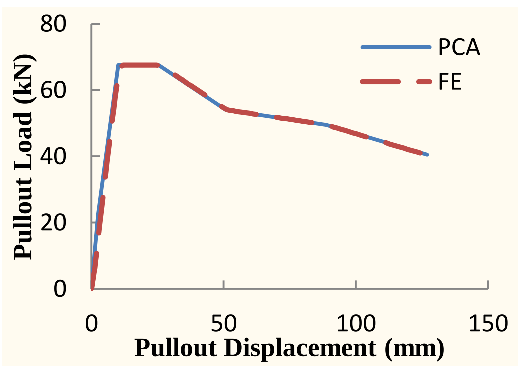
The FE models were validated by comparing pullout and full scale floor-to-floor joint tests undertaken by PCA[1]. The PCA experimental study was performed on a wide variety of strands embedded in the keyways of precast concrete slabs of different geometry and material properties. In the present study, in order to validate the FE modelling, two pullouts and two full scale floor-to-floor joint test results are used (Tables 3 and 4). The pullout load versus pullout displacement for pull-out test specimen CP1 and CP2 are obtained from the FE modelling. They are compared with the corresponding experimental studies as presented in Figure 9. As can be seen from the Fig.9, both sets of results agree extremely well in the entire loading range, which indicates the accuracy of FE modelling to simulate the pullout behaviour of strands in the grouted keyways.

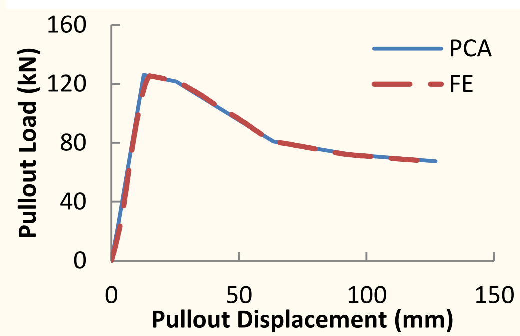
Table 3 The Properties of pullout test specimens undertaken by PCA (1975-1979)

ID	Strand size φ (mm)	l_d/φ	Embedment length (l_d)	α
CP 1	9.5	120	1140	8
CP 2	12.7	120	1500	0

Table 5 Slab details from floor-to-floor joint tests

ID	Dimension (mm)	Strand diameter(mm)	l_b/φ
FT1	150x1000x6300	9.5	152
FT2	150x1000x6300	9.5	110





Φ 9.5mm, l_d =1140 mm (b) Φ12.7 mm, l_d =1500 mm

Fig 9. Pullout load versus displacement for PCA and FE results

The results of the tie force in the strands at the mid-span vs. the central vertical deflection from the full scale floor-to-floor joints tests modelling for FT1 and FT2 are presented in Fig. 10. The comparison in Fig. 10 reveals that the FE modelling provides a good estimate in terms of both peak load and ascending or descending phases. The slight discrepancy can be attributed to the measuring errors from full scale test procedure and inherent errors associated with the assumptions introduced in the modelling process.

In the experimental work, the tie force has been calculated based on the measured strain results. Strain gauges were attached to the steel strands in two discrete points at the loaded end. In the test, the grout in the middle joint gap can provide contribution to the stiffness of the system prior to the crushing. This happened when $\delta_s \approx 250mm$[1]. In the FE modelling, the grout was not included in the model. This explains why the stiffness from tests was slightly higher that the numerical modelling before the central deflection reached 250mm (see Fig. 10). However, after the grout crushing, FE and experimental results show a close agreement.

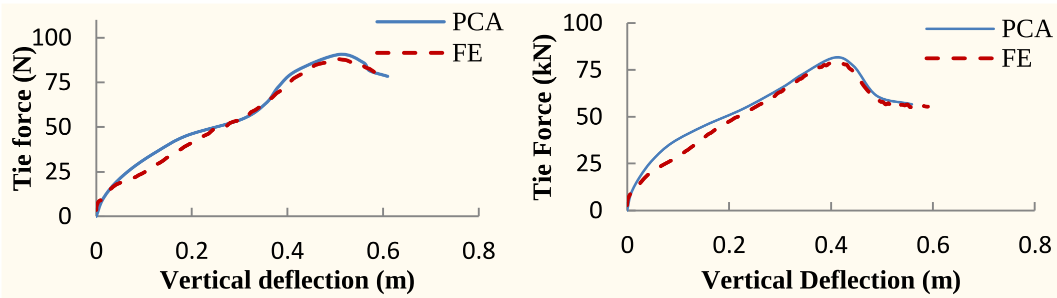


Fig.10 Experimental and FE result of tie force-vertical deflection, FT1 and FT2

The developed model has been confirmed to be able to capture a complete tie force vs. vertical deflection history with good accuracy for different bar sizes, embedment lengths, and slab lengths. From both full-scale test and FE modelling, during the descending phase in the tie force vs. deflection curves, the ties undergo stable pullout damage until the pullout displacement becomes excessive. According to the experimental study, the safe region to establish catenary action is when $5\% \le \delta_s / l_b \le 15\%$[1]. Hence, based on Eq. (4), the upper limit for pullout displacement can be defined as $\delta_l / l_b = 0.56\%$. Similarly, the corresponding limit for the tie force can be obtained by using tie force versus vertical deflection graphs. Both components are added to the translator's properties in the failure option.

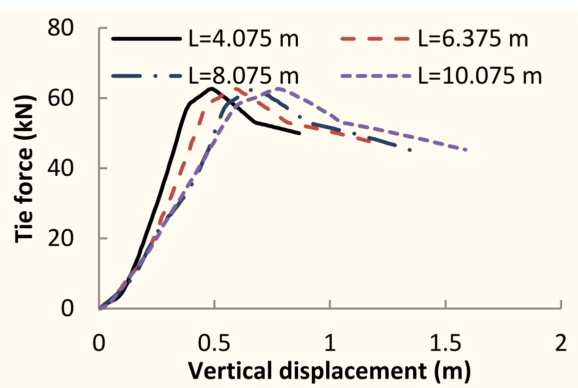
5 Parametric study

Based on the full scale experimental study, PCA[1] suggested that embedment length, bar size, concrete strength, slab length, number of keyways and surface load has major effect on floor-to-floor system behaviour, hence they can be considered as main variable. Due to wide range of variables, to keep the experimental and FE study manageable, the parametric study is limited to floor-to-floor system subjected to uniform surface load only, as this type of loading always occur after removing wall supports due to explosion.

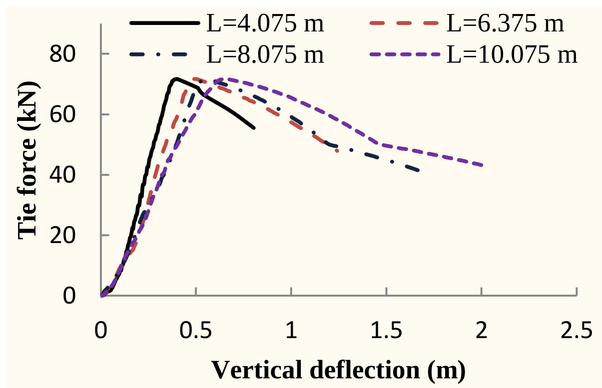
To design the parametric study, the material and geometric properties of floor-to-floor system that can notably affect their behaviour can be defined as influencing parameters. Among these, the slab length, l_b, tie spacing, b_p, embedment length, l_d, and bar size, Φ, are identified as the most important geometric variable. The Translator properties which can be representative of compressive strength of concrete and bond stress of interfacial between steel and concrete can be identified as material variable. A two-span continuous slab system is modelled for different slab lengths of 4m, 6.375m (the same as PCA test specimens), 8m, and 10m. The diameter of strands is 9.5, and 12.7mm. The embedment length of strand in grout (keyways) has been considered to be nearly the same as experimental study of PCA on full scale floor-to-floor structure i.e. 1.5m, 1.14m, and 0.76m. The above figures are taken in a view of resembling PCA's experimental data.

6 FE results

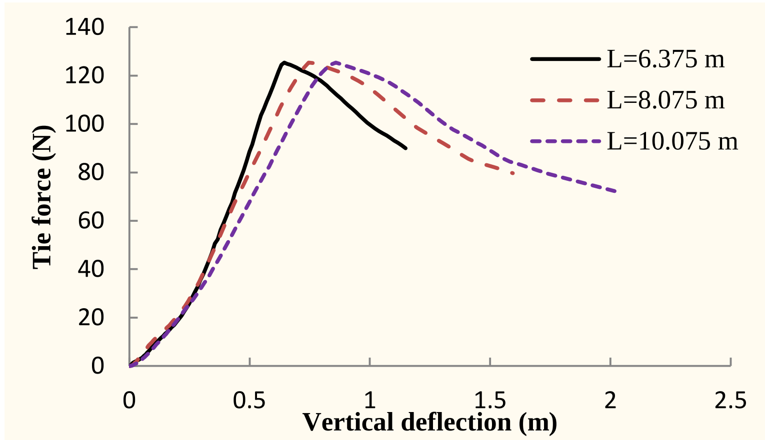
To show the efficiency of bar size and embedment length on establishing full catenary action, the floor-to-floor assemblies with various floor span lengths, bar sizes and embedment lengths are analyzed. Figure 11 presents the tie force in 9.5mm and 12.7 mm strands versus the vertical deflection for four different slab spans. Three embedded lengths are considered, i.e. 760mm and 1140mm, and 1500 mm which render l_d/Φ 80 and 120, and 120, respectively. Figure 11a and b shows that although the embedment length for strand size of 9.5 mm is increased by 50%, the maximum tie force shows only 15% increase. However, the ductility is different for various span length of slab. Figure 11c shows similar results for a 12.7 mm strand with ld/φ = 120. The shortest slab span (4.075m) has been removed as it is considered inappropriate for this strand size. From Figure 11c, it can be seen that the maximum tie force has increased to just over 120kN, increased by almost 74%. The respective vertical deflection at the maximum tie force is increased by 40- 45%. It can be seen that the tie diameter has a more significant effect on the strength of floor-to-floor joint system than the embedment length. Figure 11 also show that the maximum tie force is a constant regardless the loading span, which confirms that it is the pullout behaviour that governs the tie force-deflection behaviour. However, the ultimate deflection varies significant with the slab span. From design perspective, the necessary condition of forming the catenary action is adequate ductility, i.e. ultimate deflection. Therefore, the ability of strands to provide efficient pullout displacement must be considered as a significant factor. Hence the load-vertical deflection relationship in the post-bonding failure stage needs to be considered in the tie design. In the TF method, however, it is the tie strength requirement that has to be met.



(a) Φ 9.5mm, l_d=760 mm



(b) Φ9.5mm, l_d=1140 mm

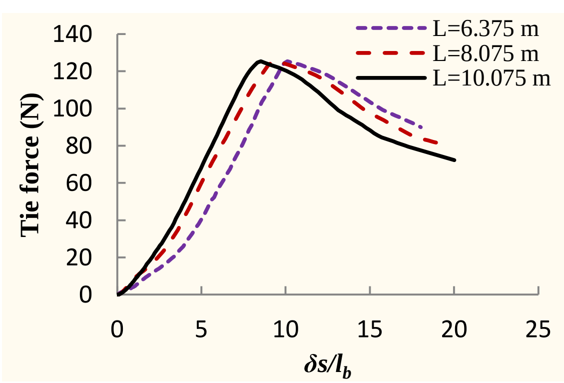


(c)Φ 12.7, l_d =1500 mm

Fig. 11 Tie force versus vertical deflection for deferent floor spans

For $\delta_s / l_b = 15\%$, which is the limit of safe catenary mechanism established by PCA[1], according to Eq. (5) pullout displacement δ_l at each end is $\delta_l / l_b = 0.56\%$. It shows that for higher span length, catenary action requires more pullout displacement. However, as pullout behaviour governs the ductility behaviour of the system, it can be seen that in descending phase, for a specific δ_s / l_b value, the tie forces demand is reduced while the span length is increased. The same conclusion can be obtained through FE analyses (Fig. 12).

The tie force demand for different bar size, and embedment length at $\delta_s / l_b = 15\%$ versus different span lengths is shown in Figure 13, which indicates that in the TF method, tie force is increased with the increasing of span length, and whereas FE results suggest tie force requirements reduce with increasing span length.



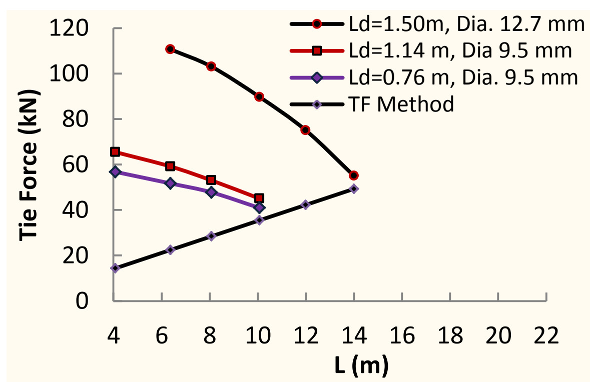
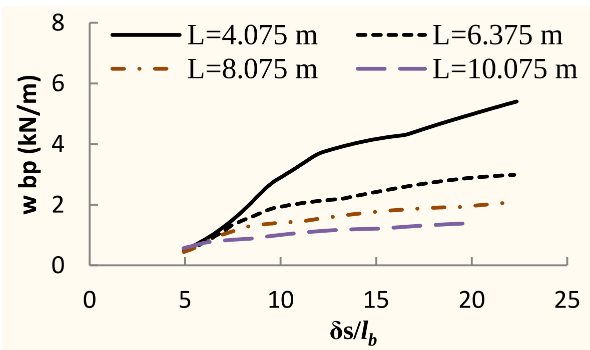


Fig. 12 Tie Force vs. Vertical deflection/Span Length ratio relationship, Φ 12.7, l_d=1500 mm

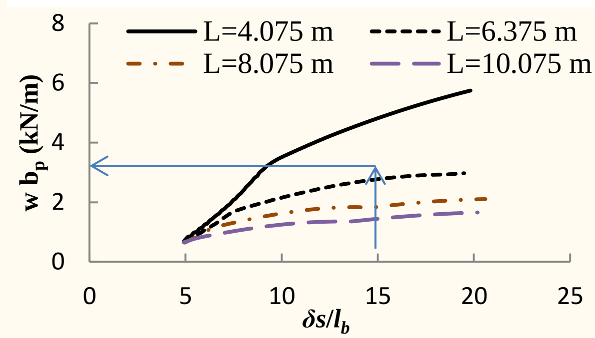
Fig. 13 Tie force requirement vs. span length for different strand size and embedment length

7 Proposed design method

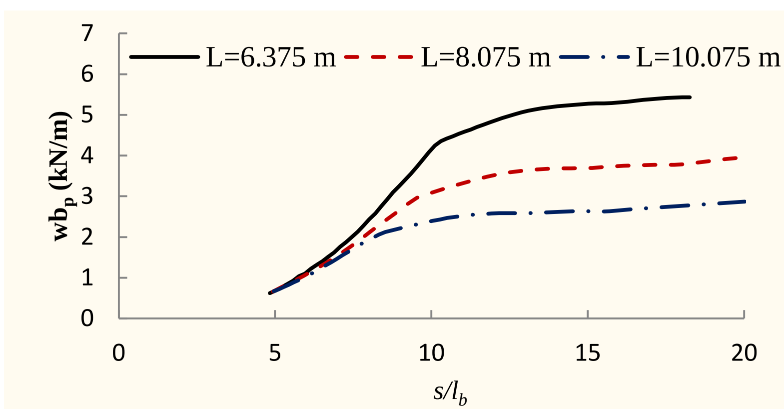
Based on FE analyses results, Figures 14 shows applied load versus δ_s / l_b for various strands diameter, embedment length, and floor spans. However, corresponding to each δ_s / l_b in the safe region e.g. $5\% \le \delta_s / l_b \le 15\%$ maximum uniform load on the floor sustained by the system can be derived. To develop a general deign method, according to PCA[1] experimental study maximum δ_s / l_b which catenary action would be established i.e. 15%[1] is taken into account. Figure 15 shows the strength of system i.e. wb_p versus span length for different embedment length and bar size which capable to provide efficient tie force and vertical deflection to establish safe catenary action mechanism a prevent progressive collapse following removal an underlying wall support.



(a) Φ 9.5mm, l_d =760 mm



(b) Φ9.5 mm, l_d =1140 mm



(c) Φ12.7 mm, l_d =1500 mm

Fig 14. Strength of system wb_p versus vertical deflection/floor span ratio for different span length, bar diameter and embedment length

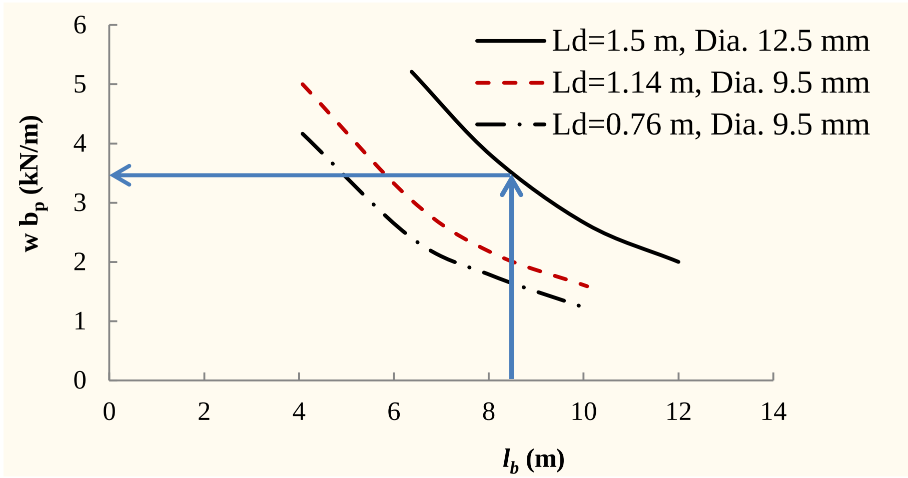


Fig 15. Floor-to-floor design graph

8 Conclusion

The tie force (TF) method is one of most common methods to design concrete structures for progressive collapse. Due to high degree of simplifications, this method is easy to use as compared to the FE method. However, it has been found that the design based on TF method will turn out to be unsafe for a certain range of floor span.

The pullout and full scale model were developed to reproduce laboratory tests. The interfacial behaviour between the steel and grout was modelled by using the translator elements built in ABAQUS software. The bond stress-slip relationship was established by using the pullout tests. The FEA method provides a more economic way to examine the bond behaviour of ties at the joint as well as the ductility of the floor assembly in the absence of underlying wall support. The modelling process is verified by comparing the results with test data carried out by PCA. Parametric analysis reveals that the bar diameter have more significant impact on the strength of floor-to-floor system than embedment length. Results also indicate that bond behaviour of tie governs the floor-to-floor system behaviour in catenary mechanism; hence the maximum tie force for different span length is identical if the tie configurations are the same.

The novelty of this study is that, from design perspective, it is the ductility rather than the tie strength should be considered in the progressive collapse design. Discrepancies in the tie force between the numerical and codified specifications have suggested that an underestimate from 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.

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