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Recommendations for Reducing Progressive Collapse Potential in Flat Slab Structural Systems

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Recommendations for Reducing Progressive Collapse Potential in Flat Slab Structural Systems

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Abstract. Flat slab structural systems are amongst the oldest methods to transfer gravity loads and contribute to resistance of lateral forces. The popularity of the system is owed to the ease and speed of construction compared to other floor systems. Flat slabs also offer flexibility where light and heavy partitions may be placed freely anywhere on plan. Properly designed flat slabs possess considerable ability to transfer vertical loads through membrane action. However, in the event of loss of primarily load-carrying system, the potential for progressive collapse is relatively high. This is due to the inherent lack of alternate load path to transfer gravity load and mitigate progressive collapse. Compared to interior columns, Loss of corner and exterior columns produces higher local demands on the slab and increases the potential for global progressive collapse in the panels at the vicinity of lost corner columns. This paper discusses the dominant failure modes associated with progressive collapse in flat slab construction. The paper also explores options for geometric and structural design of flat slab system that will enhance resistance for progressive collapse with better life safety. The following measures for reducing progressive collapse potential are discussed: 1) use of enhanced local resistance approach for corner and penultimate columns, 2) use of edge beams along perimeters to increase stiffness and improve load transfer, 3) detailing requirements for flat slab system to decrease the potential for punching shear and where continuity and anchorage of bottom reinforcement through columns was noted as critical in improving collapse resistance; 4) selecting columns configurations with improved resistance to punching shear and buckling, 5) addressing the inherently lower collapse resistance in corner slab panels through judicious selection of shear wall locations incorporated with perimeter exterior beams to form a load path.

1. Introduction

Flat slabs or flat plates are old, but popular structural floor systems. The popularity of this structural system is due to the speed of construction, which has become a critical consideration in many projects. The relative simplicity of formwork compared to slab-beam floor systems contributes to the quality of concrete. Flat slabs offer flexibility to structural designers in terms of distribution of superimposed dead loads, without the constraints associated with beam locations in slab-beam systems.

The primary goal of the progressive collapse resistant design is to ensure that a safe load path exists such that a localized failure, such as slab-column connection, or loss of a column, does not lead to disproportionate collapse.

Emphasis in this paper is on the loss of support due to column collapse or due to punching shear leading to subsequent punching shear failures and collapse of large area of the floor slab. Effect of bottom reinforcement on mitigating the consequences on punching shear failure and behaviour of edge panels after punching shear failure are discussed.



Amongst the notable flat slab failures is the 1997 progressive collapse of 120 tons section of the top floor of the Pipers Row Multi-Storey Car Park in the United Kingdom [1]. The collapse was triggered by punching shear failure. The parking structure was constructed using the flat slab method. The exact location where punching shear started is not clear, but it was concluded that the initiation of punching shear increased shear and flexural demand on edge columns, as shown in Figure 1. Corrosion of reinforcing bars was also observed in various locations of the collapse floor. The extensive sagging of the central part of the slab led to pulling on edge columns and subsequent column failure as well.



Figure 1. Collapse of large area of flat slab due to punching shear [1]

2. Failure modes in flat slab systems

Some of the factors affecting punching shear include: Some of the tensile strength of concrete, the aspect ratio of the two dimensions of a column, the aggregates used in the concrete, and the flexural stiffness of the slab-column connection.

The punching shear strength of concrete, excluding shear reinforcement, is defined by as the smaller of equations 1, 2, or 3.

$$v_c = 0.33\lambda\sqrt{f'_c} \quad (1)$$

$$v_c = 0.17(1 + \frac{2}{\beta})\lambda\sqrt{f'_c} \quad (2)$$

$$v_c = 0.083(2 + \frac{\alpha_s d}{b_0})\lambda\sqrt{f'_c} \quad (3)$$

Where,

β is the ratio of long side to short side of the column; The value of α_s is 40 for interior columns, 30 for edge columns, and 20 for corner columns; f'_c = specified compressive strength of concrete in MPa.

The two-way shear strength provided by concrete ranges from a maximum of approximately $0.33\lambda\sqrt{f'_c}$ around the corners of column with aspect ratio β of 2.0 or less and decreases down to $0.17\lambda\sqrt{f'_c}$ for much larger column aspect ratio. It is therefore advisable to use columns with aspect ratios not exceeding 2.0 to avail maximum punching shear capacity from concrete without shear reinforcement.

Maximum shear stresses in an interior column may be calculated using equation 4 proposed by ACI318-14. Maximum shear stress is assumed to occur at a critical perimeter located a distance $d/2$ from the face of the support as shown in figure 2, where, d , is the effective depth of tension reinforcing steel.

$$v_{u,AB} = v_{ug} + \frac{\gamma_v M_{sc} C_{AB}}{J_c} \quad (4)$$

where, $\gamma_v = 1 - \gamma_f$, and $\gamma_f = \frac{1}{1 + \left(\frac{2}{3}\right)\sqrt{\frac{b_1}{b_2}}}$

v_{ug} is the factored shear stress; M_{sc} is the factored slab moment resisted by the column and determined at the centroidal axis $c-c$ of the critical section; d is the effective depth, and J_c = property of assumed critical section analogous to polar moment of inertia, given by

$$J_c = \frac{d(c_1+d)^3}{6} + \frac{(c_1+d)d^3}{6} + \frac{d(c_2+d)(c_1+d)^2}{2}$$

The distance d is the effective depth of concrete reinforcement c_1 and c_2 are the column dimensions shown in Figure 2.

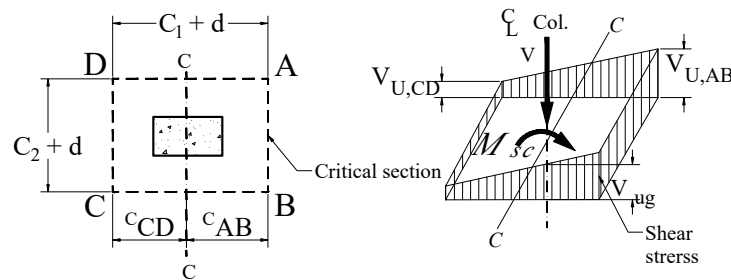


Figure 2. Assumed shear stress distribution in interior column (ACI318-14 [2])

3. Post-punching behaviour

Punching shear failure is presumed to occur when the shear stress v_u exceeds the shear capacity ϕv_c . If punching shear failure starts in one or more columns, a secondary load path, such as structural integrity reinforcement, must be available in order redistribute loads and prevent progressive failure.

Hawkins and Mitchell [3] suggested that the best defence mechanism against progressive collapse in flat slab floor systems is to ensure they are capable of forming tensile membrane through effectively continuous bottom reinforcing steel in the slab-column connection. They concluded that corner panels are not as capable of developing tensile membrane compared to interior panels with well anchored steel at its edges.

One progressive collapse philosophy is to design, and detail reinforced concrete slabs so that in the event of a punching shear failure, the slabs would be able to develop a secondary load carrying mechanism that can sustain a load equal to or greater than the two-way punching shear failure load. Mitchell and Cook [4] suggested using continuous bottom steel through the column to provide post punching shear resistance at slab-column connections, and well detailed steel to transform the reinforced concrete slab into a tensile membrane after large deflections had occurred.

3.1 Role of flexural reinforcement after punching shear failure in flat slabs

Studies have shown, that subsequent to punching shear failure, the top reinforcement "rips-out" of the top surface of the slab and becomes ineffective in carrying the load as shown in Figure 3a Hawkins and Mitchell [3]. However, in regions away from supports, the top reinforcement does not rip-out of the slab. A slab-column connection without bottom reinforcement properly anchored into the column would therefore have negligible post punching shear resistance, which would result in failure of the slab and would likely cause progressive collapse of the structure. In contrast, bottom reinforcement well anchored into supports does not rip-out of the slab and thus provides significant dowel action, providing some post-punching shear resistance as shown in figure 3b. If these bottom bars are both well anchored and effectively continuous, then they will be capable of hanging the damaged slab off of the columns as tensile membrane action develops.

The importance of bottom reinforcement in mitigating punching shear failure requires that they continuous to have appropriate tensile capacity. Fibre Reinforced Polymer (FRP) composites offer sustainable alternatives to steel reinforcing bars. They possess higher tensile capacity and are naturally corrosion resistant compared to traditional reinforcing steel. Mohamed and Khattab [5] reviewed experimental data on punching shear resistance of flat slab floors reinforced with FRP bars as main reinforcement.

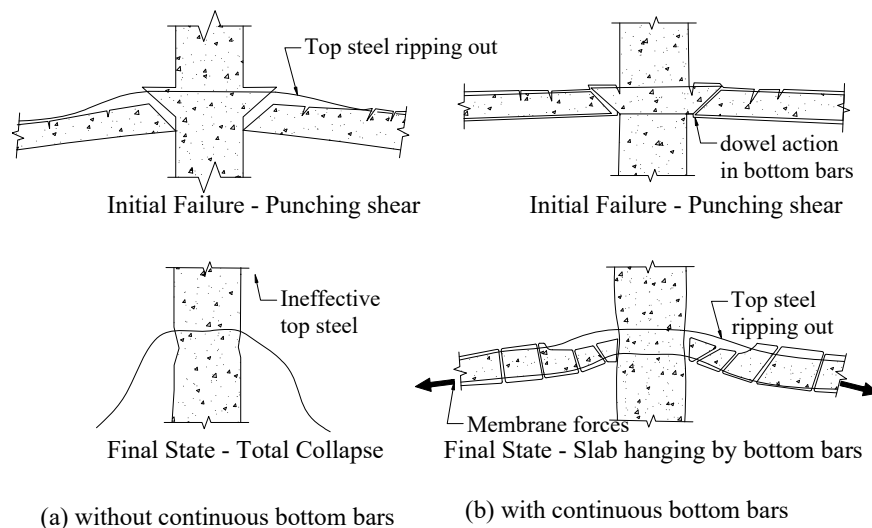


Figure 3. Role of continuous bottom bars in mitigating punching shear failure

3.2 Membrane action in flat slab versus slab-beam systems

If a two-way slab panel is supported on four sides by stiff beams, it is likely to develop two-way tensile membrane that *hangs off* the beams as shown in Figure 4a. In this type of slab-beam systems, each panel can form independent tensile membrane. Similarly, an interior panel can form a tensile membrane as it is horizontally restrained by the adjacent regions of the structure. An edge or corner panel is capable of providing its own horizontal in-plane restraint provided, that the free edges are supported by beams, which are capable of preventing significant vertical displacements along the panel sides, where tensile membrane response is expected. For development of membrane response, it is essential to provide sufficient bottom reinforcement in the slab that is well anchored into the stiff supporting regions [4].

In a flat slab or slab-beam systems, when a properly detailed interior panel is loaded at ultimate conditions, a two-way membrane system is formed that is supported by one-way catenary systems running along column lines as shown in Figure 4. This system keeps the floor slab together as long as the bottom reinforcing bars are continuous and properly spliced in internal panels, and well anchored to columns or edge beams at external panels.

When an edge or corner panel is subjected to ultimate loading, flat slab and slab-beam systems behave differently. In Figure 4a, two-way membrane system formed properly in edge/corner panel due to the presence of internal and edge beams. However, as shown in Figure 4b, edge panels in flat slab systems do not have the ability to form two-way membrane capacity. In this case, only one-way membrane, which spans between adjacent edge panels can occur, taking advantage of the available horizontal in-plane restraint of these adjacent panels. In corner panels of flat slab systems, the post-failure resisting mechanism involves folding of the slab across the corner and the development of one-way catenaries diagonally across the corner of the slab seeking the edge restraint offered by adjacent panels.

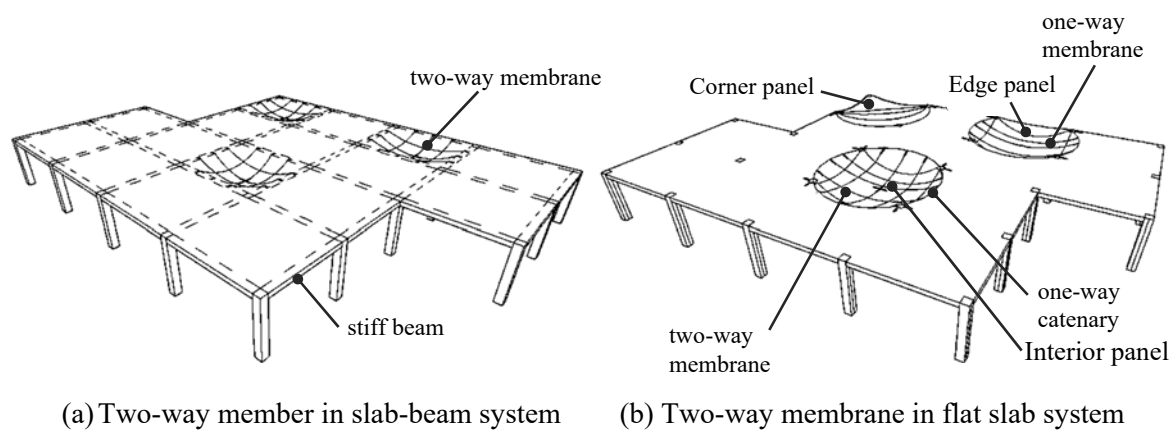


Figure 4. Catenary and two-way membrane in flat slab and slab-beam systems

4. Detailing of bottom reinforcement and development of membrane action in corner panels of flat plate and flat plate with edge Beams

Corner columns and corner flat slab panels are particularly vulnerable to punching shear failure and subsequent progressive collapse. Experimental studies confirmed, that detailing of bottom reinforcement in corner panels of flat slab system is critical in determining the load-carrying capacity and the response subsequent to punching shear failure [6, 7, 8]. The following details were shown to significantly increase the initial failure load, as well as the final collapse, compared to panels when these details were not implemented: 1) bottom reinforcement extended to within clear cover distance from free edge of the slab, 2) every second bottom bar is lap spliced with bottom bars of adjacent panels in order to develop the tensile capacity of these bars, 3) increased area of bottom bars that are anchored to the corner column and continuous past other columns. The same studies confirmed, that the largest load-carrying capacity of the same corner panels was exhibited when the edge of the slab is stiffened by spandrel beams, while maintaining continuous bottom reinforcement that is anchored on edge columns.

The current ACI318-14 recognizes the significance of anchoring slab reinforcement that end at an edge or spandrel beam. Section 8.7.4.1 states that “Where a slab is supported on spandrel beams, columns, or walls, anchorage of reinforcement perpendicular to a discontinuous edge shall satisfy (a) and (b): (a) Positive moment reinforcement shall extend to the edge of slab and have embedment, straight or hooked, at least 150 mm into spandrel beams, columns, or walls (b) Negative moment reinforcement shall be bent, hooked, or otherwise anchored into spandrel beams, columns, or walls, and shall be developed at the face of support”.

4.1 Torsional stresses in edge beams

The positive effect of edge beams in stiffening corner panels and decreasing deflections was mentioned earlier in this paper. Edge beams also support the development of two-way membrane action that mitigates collapse in the event of initiation of punching shear failure. Edge beams however, are susceptible to torsional shear stresses. If torsional moment, T_u , exceeds the torsional cracking moment, ϕT_{cr} , edge beams are likely to crack rendering them ineffective in supporting anchored slab top and bottom reinforcement. The authors recommend proportioning edge beams to resist all applied loads, but also to have stiffness sufficient for edge beams not to crack under compatibility torsion, T_u . For non-prestressed concrete system, the cracking torsion is given by equation 5.

$$0.33\lambda\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right) \quad (5)$$

Where, A_{cp} is the area enclosed by outside perimeter of concrete cross section (mm^2), p_{cp} = outside perimeter of concrete cross section (mm).

5. Design and detailing recommendations for bottom reinforcement in flat slab

As discussed earlier in this paper, continuous bottom reinforcement passing through the column reaction area is necessary to ensure concrete slab remains connected to the column after punching shear failure. Equation 6 was suggested as the minimum bottom reinforcement area passing through the column reaction area [4].

$$A_{sb} = \frac{0.5\omega_s l_n l_2}{\phi f_y} \quad (6)$$

Where,

A_{sb} = minimum area of effectively continuous bottom bars in direction l_n placed through reaction area of supports; ω_s = the load to be carried after initial failure, assumed to be the larger of the total service load acting on the slab or twice the slab dead load; l_n = clear span, in the direction being considered, measured face-to-face of supports; l_2 = distance measured from the centreline of the panel on one side of the catenary to the centreline of the panel on the other side of the catenary; f_y = specified yield strength of non-prestressed reinforcement; ϕ = capacity reduction factor (=0.9 for tension).

ACI352.1R-11 [9] proposes the same formula for the minimum amount of continuous bottom slab reinforcement passing within the column core in each principal direction. This minimum reinforcement is intended to mitigate the consequence of punching shear failure and reduce the potential for progressive collapse. This requirement is more stringent than ACI318-14 [2] 8.7.4.2.2 which requires as integrity reinforcement that two of the column strip bottom bars or wires in each direction to pass within the region bounded by the vertical reinforcement of the column and must be anchored at exterior supports.

The current ACI318-14 [2] code emphasizes the significance of bottom reinforcement details discussed by Mitchel and Cook [4] in terms of their effect in resisting progressive collapse. Structural integrity requirements of section 8.7.4.2 state, “*All bottom deformed bars or deformed wires within the column strip, in each direction, shall be continuous or spliced with full mechanical, full welded, or Class B tension splices*”. The tension splice is Class B splices is defined in ACI318-14 section 25.5.2.1, as $\ell_{st} = 1.3 l_d$ (but not less than 300 mm).

Structural integrity reinforcement provisions for column strips exceed the proposition of the study by Mitchel and Cook [4]. Therefore, ACI318-14 provisions are like to ensure the development of catenary action along support lines. In addition, provision of rigorous tension lap splices for column strip reinforcement mitigates the risk of collapse associated with damage of support.

Figure 5 shows recommended ACI318-14 detailing for top and bottom reinforcement in flat slab. The recommended for top reinforcement, shown as dotted lines in Figure 5, are intended primarily to ensure tension bars have adequate tensile development length. However, continuity is required by ACI318-14 for bottom reinforcement in both column strip and middle strip zones. In middle strips, 50% bottom reinforcement bars are spliced by 150 mm while the remaining 50% of the bottom reinforcement stops $0.15 l_n$ of the support/column line, where l_n is the clear span. Column strip bottom enjoy the most rigorous requirements for continuity where bars are required to be spliced using class B splices, mechanical splicing, or welding of bars. This is likely prescribed to ensure the one-way catenary action to develop between columns.

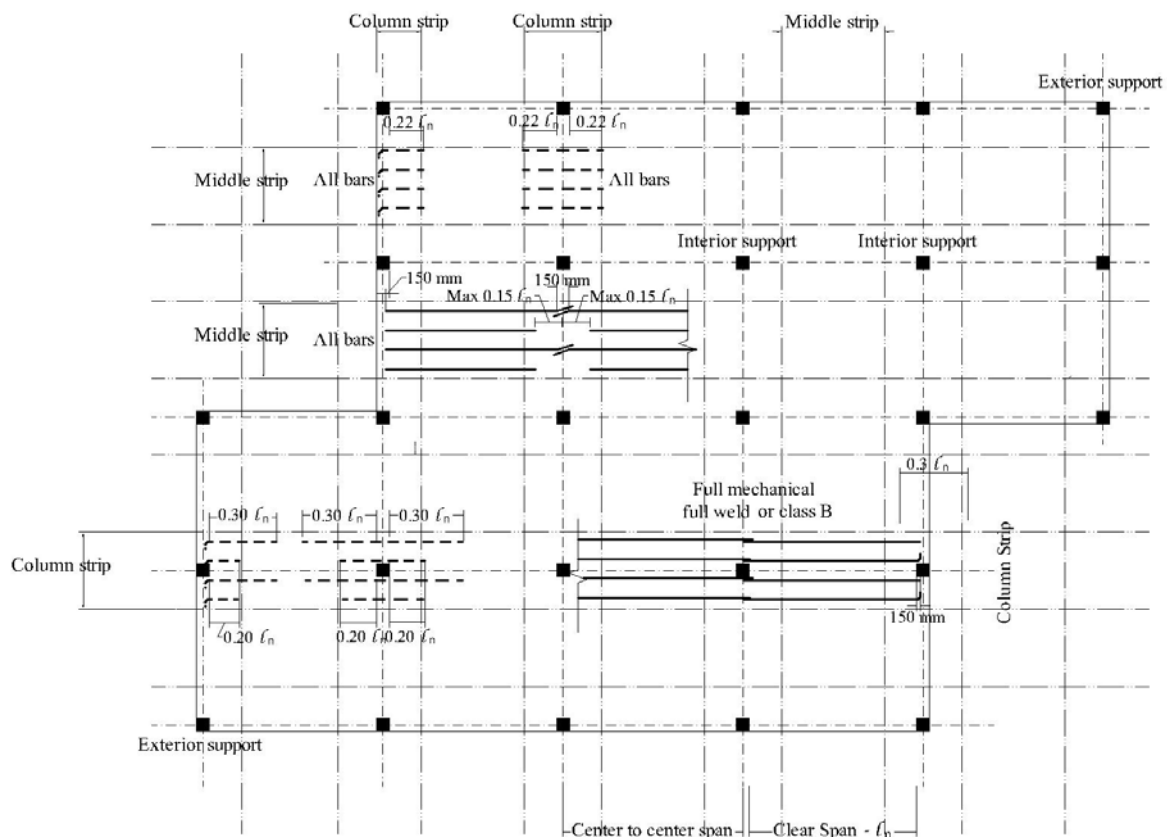


Figure 5. Top and bottom reinforcement including integrity reinforcement at selected locations of flat slab based on ACI 318-14 [2]

ACI318-14 emphasis on continuity and anchorage of bottom reinforcement bars is evident. A slab-column connection transferring significant moment can develop tensile cracking on the bottom surface of the slab adjacent to the column, which would lead to a brittle failure without the presence of anchored bottom reinforcement.

5.1 Combined Action of Top and Bottom Reinforcement in Flat Slab

The top reinforcement in regions close to vertical supports is considered ineffective since it rips out of the top surface of the slab. However, in regions away from supports the top reinforcement does not rip-out of the slab. In addition, this reinforcement has a significant overlap with the bottom bars and therefore participates in the transfer of tension between bottom reinforcing bars in adjacent panels. Analyses indicate that standard slab reinforcement details in regions away from supports, with a minimum amount of bottom steel in the middle regions of the slab, together with the proposed requirements for effectively continuous bottom bars anchored into the columns should provide a secondary defence mechanism capable of preventing progressive collapse. Mohamed [10] provided a general review of a general system to tie flat slab structural system horizontally and vertically to ensure provision of alternate load path to mitigate progressive collapse in the event of loss of vertical support.

6. Case Study

Progressive collapse resistant design typically follows the successful design and check of structural system in accordance to the local code. This section demonstrates using a case study, how damage and loss of load-carrying capacity of columns leads to significant rotations in slabs at columns around the damaged column. The same situation may occur when a slab fails by punching shear and all reinforcement rips off leading to slab going through the column at the failure location.

6.1 Model description

Model of a 220 mm thick flat slab, without any beams, is created and analysed using the finite element software SAFE developed by Computers and Structures, Inc., CA, USA. Columns are located at centre-to-centre of 6.00 meters in one direction and 5.00 meters in the perpendicular direction, as shown in Figure 6 (a). The 28-day compressive strength of slab and columns is $f'_c = 30 \text{ N/mm}^2$. All columns are 500 mm x 500 mm modelled with 3.00 m above and below slab level. Total service dead load including self-weight is 4 kN/m^2 and service live load is 6 kN/m^2 . The floor slab is designed and checked based on ACI318-14 code and punching shear stress was calculated based on equation 4 and the shear strength capacity was calculated based on equation 1, 2, or 3. The ratio of the punching shear stress demand to punching shear capacity (D/C) is shown on Figure 6 (b). D/C is less 1.00 for all columns, indicating safety of floor slab against punching shear at all column locations.

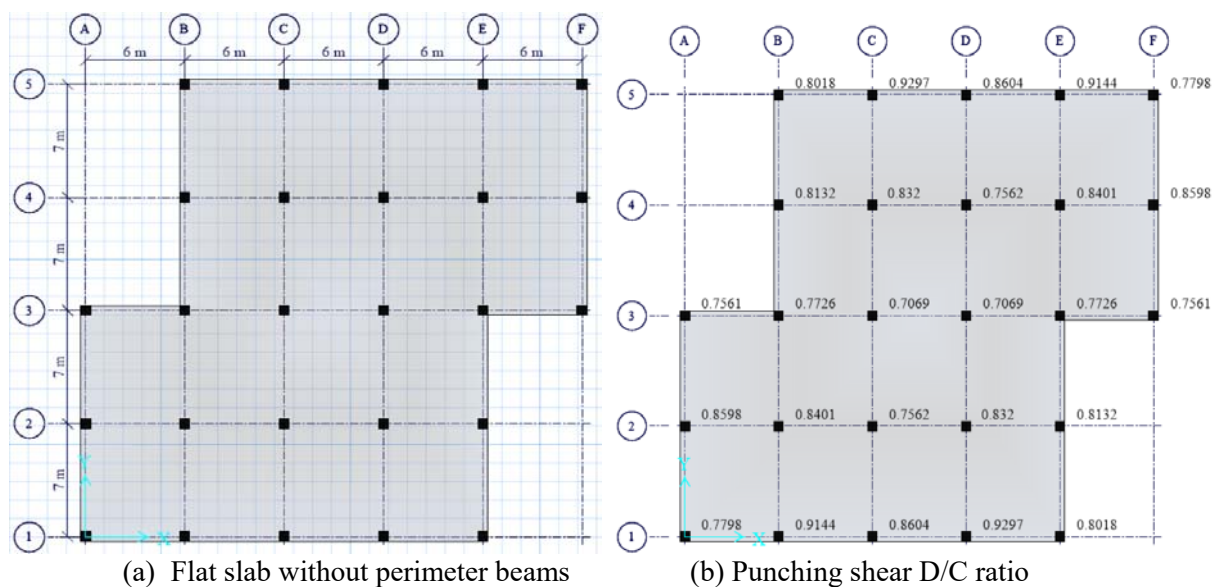


Figure 6. Flat slab geometry and D/C ratios for gravity loading in case study

6.2 Column removal scenarios and two-way shear redistribution

In order to understand the effect of localized punching shear failure in flat slab on triggering progressive collapse, analyses were conducted where support is notionally removed and punching shear D/C is calculated in each column location. Support at re-entrant corner E3 is assumed to be lost due to punching shear failure and the floor slab was reanalysed statically along with determination of shear D/C ratios. The resulting punching shear D/C ratio is compared to the control case in Figure 6b and percentage increase/decrease is calculated. Highest increases in punching shear D/C occurred in slabs at columns D3, E2, E4, and F3, which are closest to E3 within a circle centred at E3 as shown in Figure 7(a). Similarly, removal of support at D3 creates the highest increase in punching shear D/C in the slab at locations C3, D2, D4, and E3, which are closest to D3 within a circle centred at D3. Punching shear at supports within the critical punching shear stress circle may exceed 1.0 leading triggering of further. Therefore, punching shear failure in a slab may increase punching shear D/C at support locations nearest the punching shear failure location, triggering further failures. In the absence of properly continuous bottom reinforcement, punching shear failure and one or more locations may lead to progressive collapse in floor slab.

Figure 8(a) and (b) demonstrate loss of support in columns F4 and F5 respectively. This loss of support may be caused by column collapse or punching shear failure, but in the absence of properly continuous bottom reinforcement.

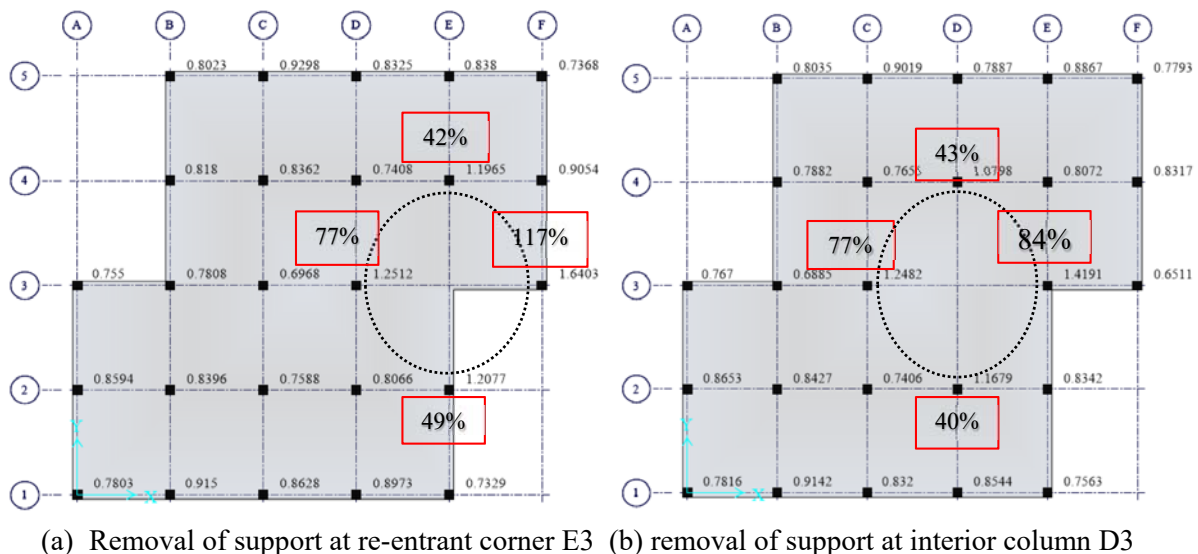


Figure 7. Increase in shear stress D/C at locations nearest the removed column

A critical punching shear stress failure is formed that is centred at the location of column F4 and F5 respectively. At column locations within the critical punching shear stress failure circle, the flat slab fails due to punching shear. In Figure 8(a), punching shear capacity was exceeded in the flat slab at locations E4, F2, and F5. In Figure 8(b), punching shear capacity is exceeded at locations E5, F2, and F5. The failures were initiated by localized failures, therefore, classify as progressive failures. In the absence of properly continuously bottom reinforcement progressive collapse of the portion of the slab is likely, precipitated by punching shear failures at various locations of the slab.

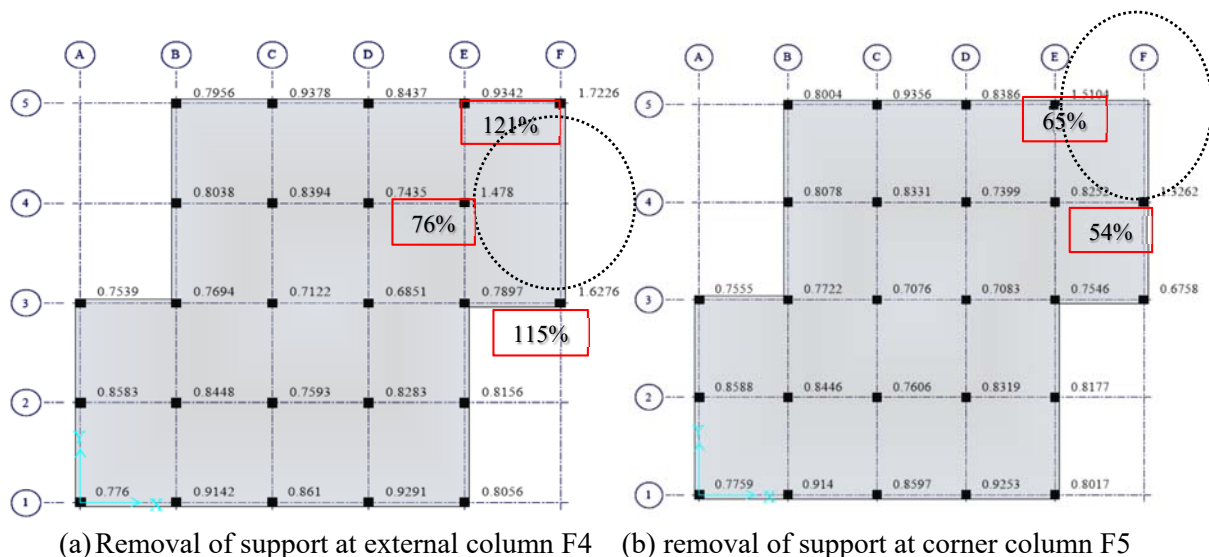


Figure 8. Increase in shear stress D/C at locations nearest the removed column

7. Conclusions

This paper discussed strategies for mitigation of progressive collapse in flat-slab structures. Punching shear was identified as major source of progressive collapse failure in flat-slab structures.

- In order to mitigate the consequences of punching shear failure, bottom reinforcement must be designed and detailed for continuity and anchorage to ensure the slab hangs over the column

through bottom reinforcing bars after top bars rip off concrete. ACI318-14 bottom reinforcement detailing requirements are adequate. However, the authors recommend the requirements of ACI352.1R-11 for minimum integrity reinforcement bars passing through columns. However, ACI352.1R-11 specifies minimum continuous reinforcement area for bottom bars, which requires caution to avoid congestion of reinforcement and subsequent weakening of concrete cast in this area.

- If practicable, consider the use of edge beams to stiffen perimeter of flat plate floors and help with development of two-way membrane action. Membrane action is essential for the response of the structure subsequent punching shear failure. Without or without spandrel beams, corner columns do not offer significant in-plane horizontal restraint to adjunct slab panels.
- A case study was used to demonstrate in the event of a support losing its load-carrying capacity, additional moments and shear forces are transferred to adjacent slab/column joints leading to possible punching shear failures. Therefore, loss of a column not only increases the unsupported span of the flat slab system but may also trigger a cascade of punching shear failures. In order to mitigate the potential progressive collapse, bottom reinforcement in general and integrity reinforcement within column in particular, must be continuous, properly lap-spliced, and adequately anchored at the edges of the slab or spandrel beam.

Acknowledgment

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