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Integrity and Collapse Resistance of Structural Concrete Floor Systems— Report

Reported by ACI Committee 377

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Integrity and Collapse Resistance of Structural Concrete Floor Systems—Report

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Integrity and Collapse Resistance of Structural Concrete Floor Systems—Report

Reported by ACI Committee 377

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Special acknowledgments to S. Cheruku,* J. Hurff, Y. Jia, M. Joyner, Z. Peng, and A. Trygestad for their contributions to this report.

This report provides a summary of the present understanding of collapse-resisting mechanisms of reinforced concrete and post-tensioned flat plates, flat slabs, slabs with beams, joist floor systems, and precast floor systems following initial local failures.

Keywords: beam growth; catenary action; collapse resistance; collapse-resisting mechanisms; column removal; compressive membrane action; emulative connection; flat plate; flat slab; floor systems; joist floor; post-punching capacity; post-tensioned; precast; punching; structural integrity.

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

Progressive or disproportionate collapse of structures happens with low probability but may cause large-scale—relative to the extent of the initial damage—structural failure. The following are some notable collapse examples that occurred previously in different types of reinforced concrete (RC) buildings during different abnormal events:

Ronan Point building in the United Kingdom in 1968—This 22-story precast concrete wall panel building suffered collapse when the partial collapse of one floor due to an accidental explosion propagated down to the rest of the floors.

Alfred P. Murrah Building in Oklahoma City, OK, in 1995—The explosion of a truck bomb destroyed an exterior column supporting a transfer girder and the blast may have also destroyed two adjacent exterior columns (Corley 2004). The loss of these columns led to the collapse of nearly half the RC frame building and 168 deaths.

A flat-plate office building in Jackson, MI, in 1956—The collapse occurred when concrete placement on the fourth floor resulted in punching failures at the second floor (Feld 1964).

A flat-plate structure in Boston, MA, in 1971—The punching failure of a slab-column connection at the roof level propagated all the way to the ground level, resulting in the collapse of two-thirds of the building. Construction overload, poor material properties in cold weather, and inappropriate positioning of slab top reinforcement precipitated this failure (King and Delatte 2004).

L'Ambiance Plaza in Bridgeport, CT, in 1987—The 16-story building collapsed during construction. The lift-slab method was used for construction with the floors supported by steel columns. Several potential causes of the collapse were identified, but the triggering collapse mechanism was never established (Martin and Delatte 2000). However, the anomalous design details of the slab unbonded post-tensioning tendons was considered a major factor in the propagation of the collapse after it had begun (Poston et al. 1991).

The aforementioned incidents indicate that, even though collapse takes place with low frequency, disproportionate structural collapse may cause severe consequences. Given that collapse is defined as loss of vertical-load-carrying capacity of structure, floor systems are critically important in providing structural integrity and redistributing gravity loads away from failing components. Once collapse is initiated, often due to column loss, the remaining structural system should be capable of resisting the progression of the collapse. The way in which the floor system redistributes the loads varies with the type of collapse initiation and the type of floor system. Current integrity provisions are intended to aid in redistributing gravity load after an initial failure.

This report is meant to assist engineers and practitioners in addressing an increasing desire by clients demanding designs beyond the integrity requirements in ACI 318.

1.2—Scope

The report seeks to summarize our present understanding of collapse initiations, collapse-resisting mechanisms, collapse-resisting strengths of floor systems, and to discuss the role of current integrity provisions in providing resistance to collapse.

This report is designed to be used by engineers, practitioners, and researchers. It will be used to improve understanding of how concrete floor systems redistribute gravity loads following the onset of initial failure. This report can also be used by educators to describe the mechanics of collapse resistance and direct them to the applicable code provisions currently in place.

Each chapter details possible causes of collapse initiation, collapse-resisting mechanisms, collapse-resisting strengths of floor systems, and discussion of relevant code requirements. Chapters 3 and 4 mainly correspond to cast-in-place concrete structures without and with beams, respectively. The focus of Chapter 5 is on precast concrete structures. The report also presents relevant integrity provisions in ACI 318, and CSA A23.3, and the New York City Building Code (2014).

If an engineer is required to perform a comprehensive progressive collapse analysis, including the determination of collapse initiation scenarios, the design guidelines provided by the General Services Administration (GSA) or the Department of Defense (DoD) should be used. Furthermore, this report can be used to supplement current material-specific GSA and DoD guidelines with the approval of the authority having the jurisdiction over the project.

This report is developed based on analytical and experimental studies conducted on RC elements and structures with uniaxial concrete compressive design strength of approximately 3 to 8 ksi (21 to 55 MPa). Application of the collapse-resisting mechanisms discussed herein to high-strength concrete or high-strength reinforcement requires further study.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A_{sb}	=	area of bottom slab and beam reinforcement passing through each face of column
A_{sm}	=	area of integrity reinforcement required in each principle direction
F	=	beam vertical load-carrying capacity
f_y	=	specified yield strength of reinforcement
L	=	span length of beam or one-way slab
L_1	=	governing bay length
L_n	=	length of clear span measured face-to-face of supports
ℓ_1	=	center-to-center span length
ℓ_2	=	center-to-center span width (perpendicular to ℓ_1)
ℓ_x	=	clear span length in the short direction
ℓ_y	=	clear span length in the long direction
M	=	moment at right end of beam
M'	=	moment at left end of beam
P	=	beam axial force

V_{se}	= post-punching resistance of a slab-column connection
T_x	= force per unit length in the reinforcement in the short direction
T_y	= force per unit length in the reinforcement in the long direction
t	= outward lateral movement at beam end due to support flexibility
$w_{y,\ell}$	= slab capacity as predicted by yield-line theory
Δ	= generalized deflection
ϵ_x	= strain in the short direction
ϵ_y	= strain in the long direction
ω	= uniform floor load per unit area
ω_u	= factored uniform floor load per unit area

2.2—Definitions

Please refer to the latest version of ACI Concrete Terminology for a comprehensive list of definitions. Definitions provided herein complement that resource.

beam growth—tendency in concrete elements to grow in length as flexural cracking and yielding occur, which, if restrained, results in the development of axial compressive force or compressive membrane action.

collapse—partial or complete loss of vertical load-carrying capacity of a portion of or the entirety of the structure.

disproportionate collapse—collapse that is characterized by a pronounced disparity between the original cause and the ensuing collapse of a large (relative to the initial damage) part or the whole of a structure.

emulative design—design method in precast construction to provide continuity between adjacent flooring elements by using splices or field topping with continuous reinforcement over the connection, which emulates cast-in-place construction.

failure—partial or complete loss of load-carrying capacity of an individual element or component.

progressive collapse—spread of an initial local failure from element to element, resulting eventually in partial or complete collapse of a structure.

CHAPTER 3—REINFORCED CONCRETE AND POST-TENSIONED FLAT PLATES AND SLABS

3.1—Introduction

Reinforced concrete (RC) flat plate and flat slab floor systems are two-way framing systems composed of an RC slab directly connecting to columns. Drop panels, shear caps, or column capitals, which are the characteristics of flat slab construction, may be used at the columns to resist heavier loads or provide longer spans. The slab may be reinforced with nonprestressed two-way reinforcement or unbonded post-tensioning reinforcement. The slab may have steel decking; however, the role of decking is not considered in this report. This section does not cover floor systems that have intermediate beams. Floor systems may have perimeter beams; however, this chapter only pertains to the interior portion with no beams. Floor systems with nonprestressed reinforcement, prestressed reinforcement, or

both, are considered. Floor systems with interior beams are discussed in Chapter 4.

3.2—Collapse initiation and resisting mechanisms

3.2.1 Collapse initiation—The collapse of flat plate or flat slab floor systems may be initiated by: 1) punching shear failure of the slab around a column; 2) failure of a column; or 3) flexural failure of the slab. Progressive collapse may occur if:

a) Subsequent punching shear failures occur near surrounding columns

b) Slab(s), supporting columns, or both, fail, which may lead to impact on the floor below, thereby causing additional failures, depending on the floor at which the failure initiates

c) Tendons in unbonded post-tensioned (PT) floors fail, resulting in the loss of tension throughout the length of the tendons covering multiple spans and leading to failure across those spans

Previous collapses have been primarily due to Case (a) such as the Sampoong Department store (1995), Commonwealth Avenue, Boston, MA (1971); Skyline Plaza/Bailey's Crossroads in Virginia (1973); and Harbour Cay Condominiums in Cocoa Beach, FL (1981). As such, Case (a) is the focus of this report and current integrity provisions. Case (b) was also observed in the Murrah building as a result of the Oklahoma City bombing (Sozen et al. 1998).

3.2.2 Collapse-resisting mechanisms—Collapse can be resisted for Case (a) in 3.2.1 if the slab near surrounding columns has sufficient punching or post-punching capacities; for Case (b), if the slab has sufficient tensile capacity to prevent floor collapse or the floor below is able to withstand the impact of a collapsed floor; and for Case (c), if the remaining tendons and conventional reinforcement are able to redistribute and carry the load. The amount of load transferred to the adjacent columns can be estimated by using static analysis for the case with the removed column. Analysis of these mechanisms is generally conducted under the following load combination ($1.2D + (0.5L \text{ or } 0.2S)$) per U.S. General Services Administration (2016) and UFC 4-023-03. Dynamic effects may need to be considered. The U.S. General Services Administration (2016) and UFC 4-023-03 analysis guidelines include dynamic load amplification. Research (Qian and Li 2012, 2014; Orton and Kirby 2013; Peng et al. 2018) has found the dynamic increase factors in the guidelines to be conservative.

3.2.3 Collapse resisting mechanism strengths

3.2.3.1 Punching capacity—The progression of collapse can be limited by resisting punching failure at the surrounding slab-column connections. The punching capacity of the connections can be evaluated by code provisions such as those in ACI 318 and *fib* MC2010. In addition, research conducted by Muttoni (2008) and Broms (2016) specifically consider the deformation limits of the connection. Unbalanced loading due to an initial punching shear failure or loss of a column should be considered in the surrounding connections. Dynamic effects at the connection may need to be considered.

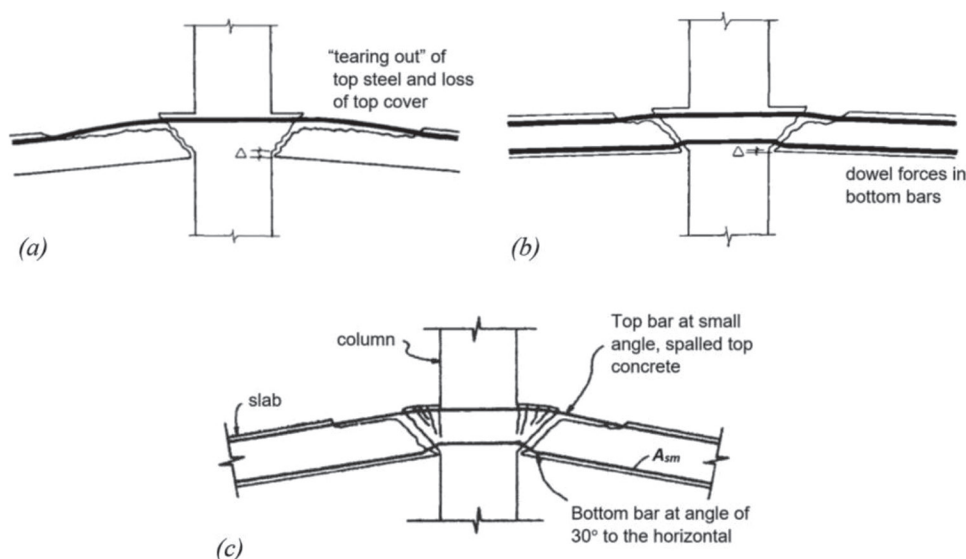


Fig. 3.2.3.2a—Geometry of deformation after punching shear failure in slab: (a) without bottom integrity bars; (b) with continuous bottom integrity bars (Hawkins and Mitchell 1979); and (c) inclination of bottom bars (ACI 352.1R).

3.2.3.2 Post-punching capacity—In the case of slab punching shear failure at one column location, resistance to progressive collapse can, in part, be provided by the remaining post-punching capacity of the slab-column connection. That is, if the failed connection can provide some post-punching capacity, then the amount of load redistributed to the surrounding connections is reduced (Keyvani et al. 2014; Peng et al. 2018). In addition, the post-punching behavior enables more toughness in the system and the possibility of resisting dynamic loads (Peng et al. 2018). Post-punching capacity in surrounding connections may also be able to limit the progression of collapse.

Failure of one connection or column leads to additional loads due to the load redistribution to the surrounding connections. Prevention of progressive collapse assumes that the surrounding connections were not already overloaded (due to differences in design or capacities) and can withstand the additional load. If the surrounding connections were close to their capacities, then progression of collapse may occur.

The post-punching capacity is primarily provided by structural integrity reinforcement, as shown in Fig. 3.2.3.2a. The structural integrity reinforcement provides an alternate load path immediately following punching. The top reinforcement is often not effective due to its tendency to rip out of the concrete cover. Bottom bars, often used for structural integrity reinforcement, are well anchored and resist load due to their inclination. Structural integrity bars can offer resistance until they either rupture due to high tensile load in the bar, rip out of the concrete (in the case of top bars or bottom bars in thin slabs), or pull out due to insufficient embedment length. An example of a load-deflection curve showing post-punching capacity can be seen in Fig. 3.2.3.2b. Post-punching capacity is the residual strength available in the slab-column connection after punching occurs.

Previous research to estimate the post-punching capacity of slab-column connections includes work by Hawkins

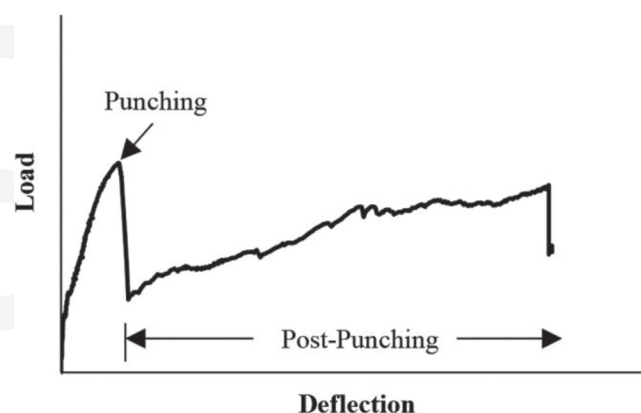


Fig. 3.2.3.2b—Post-punching response of isolated slab column test.

and Mitchell (1979), Melo and Regan (1998), Habibi et al. (2012), Ruiz et al. (2013), and Orton et al. (2016). Habibi et al. (2012) considered the following equation to determine the post-punching resistance of a slab column connection

$$V_{se} = 0.5 \sum A_{sb} f_y \quad (3.2.3.2)$$

where V_{se} is the post-punching resistance; $\sum A_{sb}$ is the total area of bottom slab (integrity) reinforcement on all column faces; and f_y is the yield strength of the reinforcement. The equation assumes the top bars (due to ripping out) and concrete (due to punching) are ineffective and assumes the slope of the bottom bars draped over the support at an angle of 30 degrees with respect to a horizontal line (Fig 3.2.3.2a(c)). The equation also assumes the bars are able to develop their yield strength at the connection. Therefore, the equation is the sum of the vertical projection of the tensile yield force of bottom bars where the 0.5 factor comes from the vertical projection of a force at 30 degrees to the horizontal ($\sin(30) = 0.5$). The vertical capacity for the bars should resist the force applied at the connection. Equation (2.2.3.2) is consistent

with provisions in [CSA A23.2](#) and [ACI 352.1R](#). [Habibi et al. \(2012\)](#) reports that using this equation produced reasonable predictions of the post-punching capacity with an average actual-to-predicted ratio of 1.07.

3.2.3.3 Slab response—After an initial punching failure or loss of a column, loads once carried by that connection are redistributed to surrounding columns. This requires that the slab is able to carry and redistribute the load, and the surrounding slab-column connections are able to transfer loads into the columns. If the slab is not able to carry the loads, then it could fail and may fall to the floor below, triggering a progression of collapse (Case (b) in 3.2.1). To redistribute these loads, two possible mechanisms are available based on the deflection of the slab.

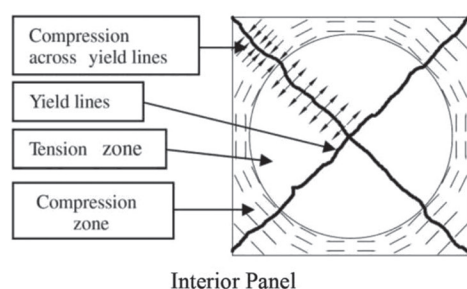
3.2.3.3.1 Slab response for deflections approximately less than slab thickness—At low levels of deflection, loads in the slab are resisted through:

a) Shear and flexure in slab

b) In-plane compression caused by in-plane slab growth due to flexural cracking and yielding that increases shear and flexural capacity

Prediction of the response can be achieved through elastic slab deflection estimates such as those in [Chang and Hwang \(1996\)](#) or the crossing beam analogy in [ACI 435R](#) for slabs still in the elastic range. For deflections beyond the elastic range, plastic analysis methods can be used. Using finite element models with crossing beams that have distributed or concentrated flexural-axial interacting plasticity, the slab growth tendency and in-plane compressive action can be captured ([Sasani and Kropelnicki 2008](#); [Bazan 2008](#); [Sasani et al. 2011](#)).

3.2.3.3.2 Response for deflections approximately greater than slab thickness—At larger vertical deflections (greater than slab thickness), the slab can resist loads through the tensile/compression membrane action illustrated in Fig. 3.2.3.3.2. The interior of the slab exhibits tensile forces whereas the edges form a compressive ring. At this level of deformation, the response of a simply supported slab is essentially the same as a fully restrained slab because of the ability of the slab to form its own in-plane edge restraint by forming a compression ring around its perimeter ([Mitchell and Cook 1984](#)). [Hawkins and Mitchell \(1979\)](#) developed an iterative method to determine the tensile membrane response of a laterally restrained slab under a circular deformed shape.



The model assumes only the load-carrying capacity of reinforcing bars is considered; thus, the resistance is due only to the tensile net formed by the reinforcing bars. It is further assumed that all the bars yielded and their stress-strain relationship is elastic-perfectly plastic. The floor load per unit area can be calculated by

$$\omega = \frac{2T_x \sin \sqrt{6\varepsilon_x}}{\ell_x} + \frac{2T_y \sin \sqrt{6\varepsilon_y}}{\ell_y} \quad (3.2.3.3.2a)$$

and the deflection at the center of the slab by

$$\delta = \frac{3\ell_x \varepsilon_x}{2 \sin \sqrt{6\varepsilon_x}} \quad (3.2.3.3.2b)$$

where ℓ_x and ℓ_y are the clear span lengths in the short and long direction, respectively; and T_x and T_y are the forces in the reinforcement in the x- and y-directions, respectively, corresponding to strain ε_x and ε_y . The iterative method assumes a strain ε_x , finds the corresponding ε_y , and then uses Eq. (3.2.3.3.2a) and (3.2.3.3.2b) to calculate the load and deflection. The process is continued until rupture of the reinforcement can be predicted. [Orton et al. \(2013\)](#) modified these equations based on a parabolic deflected shape in a rectangular slab and developed relationships that estimate the slab deflection and the corresponding uniformly distributed load as a function of reinforcing bar tensile strain.

The edge spans and corner span are more vulnerable than the interior span to damage following failure (loss) of an exterior or corner column ([Yi et al. 2014](#)). Equations (3.2.3.3.2a) and (3.2.3.3.2b) would not apply in these cases. As shown in Fig. 3.2.3.3.2, the resistance of the edge panel could be determined through one-way action in the slab and the resistance of a corner panel from action between the two penultimate columns. It is important that reinforcing bars need to be continuous to develop yield strength along their entire length.

3.3—Current integrity provisions

ACI 318 integrity provisions do not specifically address the collapse-resisting mechanisms previously mentioned, but are intended to enhance the ability of a structure through

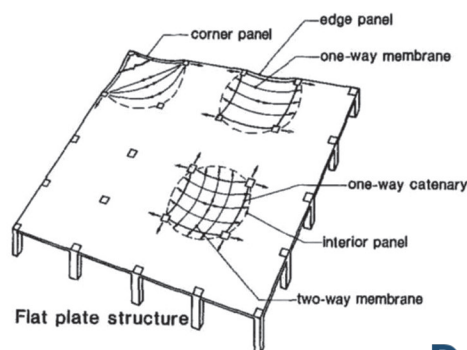


Fig. 3.2.3.3.2—Membrane action possible in flat plate structure ([Dat and Tan 2013b](#); [Mitchell and Cook 1984](#))

strength, redundancy, ductility, and detailing of reinforcement to redistribute stresses and maintain overall stability if localized damage or overstress occurs. This is often achieved by tying members together and providing post-punching resistance through structural integrity reinforcement.

3.3.1 Role of integrity reinforcement—Post-punching capacity can be provided by integrity reinforcement composed of continuous bottom reinforcement. In the case of initial punching failure, these bars may be able to reduce the amount of load redistributed to surrounding connections through their ability to provide post-punching capacity. The post-punching capacity in turn may be able to resist the progression of collapse as found in Peng et al. (2018).

3.3.2 Code integrity reinforcement

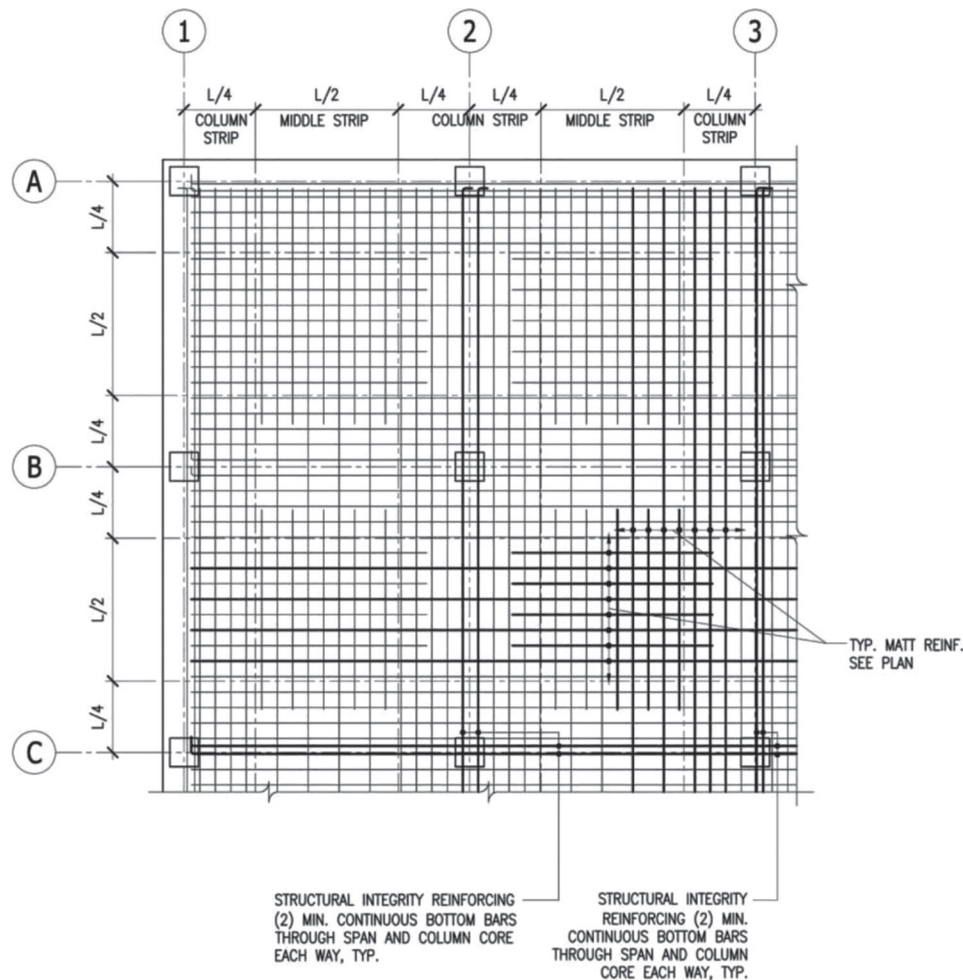
3.3.2.1 ACI 318-19 Section 8.7.4.2.1 requires that all bottom bars or deformed wires within the column strip be continuous or spliced using mechanical or welded splices in accordance with 25.5.7 or Class B tension lap splices in accordance with 25.5.2. In addition, **ACI 318-19** Section 8.7.4.2.2 requires that at least two bars pass through the

region bounded by the column longitudinal reinforcement. These provisions have been in place since ACI 318-89. The two continuous bottom bars through the column are intended to provide the post-punching capacity indicated in Eq. (3.3.2.1). However, ACI 318 does not specify the minimum area of integrity reinforcement, which could lead to undersized bars being used that are unable to provide the required post-punching capacity. Figure 3.3.2.1 shows the bottom integrity reinforcement requirement based on ACI 318.

ACI 352.1R has recommended the following equation to be used to size integrity bars

$$A_{sm} = \frac{0.5\omega_u \ell_1 \ell_2}{\phi f_y} \quad (3.3.2.1)$$

where $\phi = 0.9$; ω_u is the factored uniform floor load (but not less than two times the dead load); ℓ_1 and ℓ_2 are the center-to-center span length and width, respectively, in the perpendic-



NOTES:

1. ALL MIDDLE STRIP WIDTH VARIES ACCORDING TO COL. SPACING.
2. BACKGROUND LINES SHOW TYP. MAT REINF. BOTTOM.
3. AT INTERIOR SUPPORT REGION ALL BOTTOM BARS SHALL BE CONTINUOUS WITH CLASS B LAP, MECHANICAL OR WELDED TENSION SPLICES
4. REINFORCEMENT SHALL RESIST THE MAXIMUM OF (1.2DL+1.6LL) OR (1.4DL) NOT LESS THAN THE REQUIRED FOR TEMPERATURE AND SHRINKAGE REINFORCEMENT.

Fig. 3.3.2.1—Bottom integrity reinforcement requirement in ACI 318.

ular directions; and A_{sm} is the area of integrity reinforcement required in each principle direction.

This equation is essentially equivalent to Eq. (3.2.3.2), where the term $w_u \ell_1 \ell_2$ can be considered equivalent to the shear demand V_{se} in Eq. (3.2.3.2). Equation (3.3.2.1) is also equivalent to the equation provided in [CSA A23.3](#) and has been demonstrated to be adequate to provide enough post-punching capacity to carry the factored loads in the slab in [Hawkins and Mitchell \(1979\)](#) and [Mitchell and Cook \(1984\)](#).

For post-tensioned (PT) slabs, [ACI 318-19](#) Section 8.7.5.6.1 requires that at least two tendons with 1/2 in. (12.7 mm) diameter or larger be placed in each direction passing through the region bounded by the column longitudinal reinforcement or anchored within the region. If tendon layout constraints make it difficult to provide structural integrity reinforcement, bonded reinforcement can be provided. [ACI 318-19](#) Section 8.7.5.6.3.1 gives the required area of bonded reinforcement. [ACI 352.1R](#) further recommends that for post-tensioned slabs, continuous bottom bonded reinforcement be provided in accordance with Eq. (3.3.2.1).

3.3.2.2 [CSA A23.3](#) assumes that structural integrity reinforcement is capable of yielding and, after punching, is inclined at an angle of 30 degrees from the horizontal. Consistent with Eq. (3.3.2.1), [CSA A23.3](#) requires a minimum area of bottom reinforcement connecting the slab to the column on all faces to be

$$\sum A_{sb} \geq \frac{2V_{se}}{f_y} \quad (3.3.2.2)$$

where V_{se} is the shear transmitted to the column due to specified loads but not less than twice the self-weight of the slab. The reinforcement area requirement from [CSA A23.3](#) is equivalent to the area recommended in [ACI 352.1R](#).

3.3.2.3 The New York City Building Code (2014) Section 1916.2.1 requires all floor slabs to have a continuous mat of bottom reinforcement in two directions with steel reinforcement not less than the amount required for temperature and shrinkage reinforcement. Section 1916.2.3 requires tension ties composed of the slab bottom reinforcement be provided at each level that develops a tension force equal to the maximum of (a) or (b):

(a) Three times the unfactored dead load (considering self-weight of structure only)

(b) One and a half times the factored load using the load combinations of $(1.2DL + 1.6LL)$ or $1.4DL$

For transfer elements only, in place of (a) and (b), the slab tension tie reinforcement should be anchored at all supports.

This beam or slab bottom reinforcement should be distributed around the column perimeter and should be extended on all sides of the column into the adjacent slab for at least one-third of the span length. Where reinforcing bars cannot be extended beyond the column (for example, at slab edges and openings), they should be hooked or otherwise developed within the column. Figure 3.3.2.3a shows the structural integrity reinforcement. For more information, refer to [Gilsanz et al. \(2016\)](#). Figure 3.3.2.3b shows the bottom integrity reinforcement requirement based on the New York City Building Code (2014).

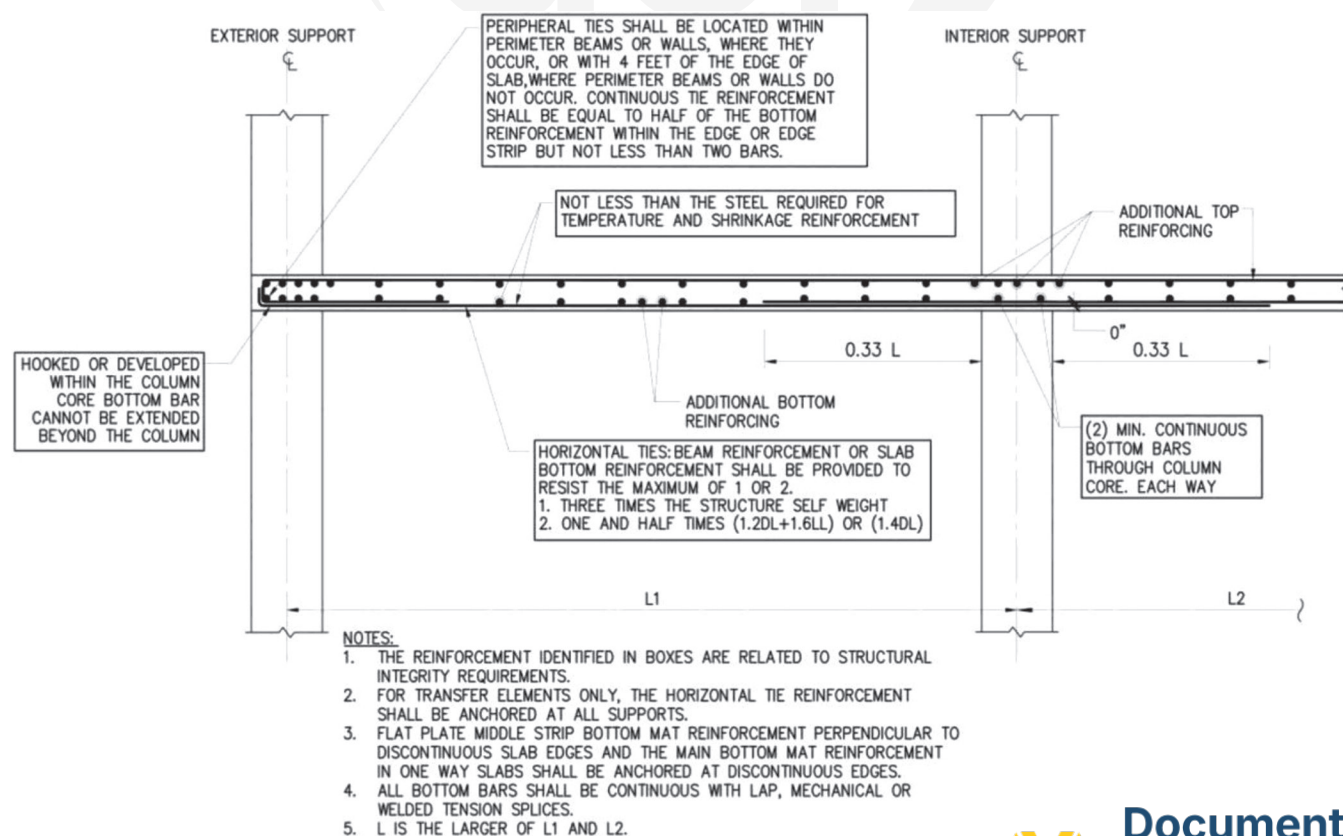


Fig. 3.3.2.3a—Integrity reinforcement requirement in the New York City Building Code (2014).