Designing Slab-Column Constructions due to the Possibility of a Progressive Collapse Caused by a Punching

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Abstract. When designing slab-column structures, one of the most important elements is an adequate protection of the support zone. The main problem when designing this zone is the possibility of a punching. This phenomenon is the main cause of failures, damages and progressive collapse of slab-column structures. The article presents standard methods for calculating support zones after a punching. The calculations were verified by a few experimental researches.

1. Introduction
The destruction of the support zone has been the original cause of the majority of slab-column structure collapses. According to literature, the destruction of the support zone is most often caused by human or technological factors:
   a) Human causes include e.g. the destruction of a column by a car crash, explosion in a single room (excluding cascade explosions), cracking of reinforcing steel induced by manufacturing error in the process of rolling, etc. In all these cases, it is assumed that concrete exhibits the intended strength and we can rely on adhesion forces between the reinforcing steel and concrete. In this case, when threatened with collapse, the load of the structure will equal the operational load.
   b) A technological cause means a drastic reduction of concrete strength owing to the supplier's mistake, freezing of concrete, or the structure striking too soon after concreting. In all these cases, adhesion forces between reinforcing steel and concrete cannot be relied on. In such a situation, it can be assumed that the damage is done at the erection phase. In this case, the load of structure in the hazardous zone corresponds to the implementation load: dead load plus stored materials.

Hence this zone deserves particular attention.

The common feature of almost all collapses listed in literature is the fact of visible top reinforcement detached from the floor slab which fell down. It is obvious and must be taken for granted that in the last phase of the support zone destruction the reinforcement is practically non-existent. This concerns a situation when the load is applied axially (Figure 1a1 and 2b1), but also a situation when the load is applied eccentrically (Figure 1a2 and 2b2). Only a bottom reinforcement (properly selected and designed) is able to prevent the progression of the collapse. The basic condition here is that the reinforcement passing over the column is continuous, or at least its continuity is maintained in a way allowing the reinforcement to break away but never be pulled out when impacted by force.
Figure 1. Symmetric image of the support zone action following its failure by punching:
   a) in the case of anchoring of the bottom reinforcement in the column; b) in the case of arranging the reinforcement over the column trace (1- axial load; 2- eccentric load).

2. Recommendations of foreign standards

The basic problem is the determination of the demand for bottom reinforcement protecting against the progression of the collapse in the column zone. The problem is not addressed directly in PN-EN 1991-1-7:2008 [1], PN-EN 1992-1-1:2008 [2] nor in PN-B-03264:2002 [3]. Because [2], apart from the requirement of arranging two continuous rods in each of the two perpendicular directions, containing no other guidelines below, the authors discuss the requirements of foreign regulations. The problem is mentioned in Model Code 1990 [4] (currently in Model Code 2010 [5]) and Canadian [6] and Swiss standards [7]. Owing to the significance of the problem, the materials are discussed here fairly broadly.

2.1. Canadian Standard Association A23.3 [6]

The total area of the bottom surface of reinforcement connecting the slab to the column at all surfaces of the column perimeter should equal:

\[ A_{sb} = \frac{2V_{se}}{f_y} \]  \hspace{2cm} (1)

or

\[ A_{sb} = \frac{2V_{se}}{f_{py}} \]  \hspace{2cm} (2)
where:

- \( A_{sb} \) - minimum area of bottom reinforcement crossing one face of the periphery of a column and connecting the slab to the column to provide structural integrity,
- \( V_{se} \) - shear transmitted to column due to specified loads, but not less than the shear corresponding to twice the self-weight of the slab,
- \( f_y \) - yield strength of non-prestressed reinforcement,
- \( f_{py} \) - yield strength of prestressing tendons.

It is required that the calculated reinforcement consists of at least two bars or two tendons extended above the section of the column in every direction of the slab spans length. Moreover, CSA A23.3 [6] requires the fulfillment of one or more conditions:

- bottom reinforcement should be extended, allowing the overlapping with the bottom reinforcement in adjacent spans at the length of \( 2l_d \),
- when the reinforcement reaches the external edge of a corner column, it must be extended and bent or mechanically anchored in order to fully utilize the yield point of the reinforcement.

If the bottom reinforcement against progressive collapse passing over the column trace has the form of pre-stressed tendons, the minimum field of the total pre-stressing steel surface should be calculated based on the formula (2).

### 2.2. Model Code 2010 [5]

The problem of reinforcement protecting the slab-column structure after punching is more broadly discussed in the recent version of Model Code. It is required that the reinforcement consists of at least four crossing bars, two on each edge, located above the support field and properly extended at the side of the compressed slab. Pre-stressing tendons (cables) may also be considered as a protective reinforcement. The load-bearing capacity ensured by the existing reinforcement after punching should be calculated based on the formula (3).

\[
V_{Rd,int} = \sum A_k f_{yd} \left( \frac{f_t}{f_y} \right)_k \sin \alpha_{ult} \leq \frac{0.5\sqrt{f_{ck}}}{\gamma_c} d_{res} b_{int}
\]

where:

- \( A_k \) - the sum of the cross-sections of all reinforcement suitably developed beyond the supported area on the compression side of the slab or to well-anchored bent-up bars,
- \( f_{yd} \) - the design yield strength of the integrity bars,
- \( (f_t/f_y)_k \) - ratio of strength values defined in EC2 [2] and depend on the class of plasticity of reinforcement.

For design purposes, three ductility classes are defined in [5], which are defined by minimum specified values for the characteristic value of the ratio \((f_t/f_y)_k\) and the characteristic percentage total elongation at maximum force \( \varepsilon_{ok} \) as follows:

- Class A: \((f_t/f_y)_k \geq 1.05 \) and \( \varepsilon_{ok} \geq 2.5\%\),
- Class B: \((f_t/f_y)_k \geq 1.08 \) and \( \varepsilon_{ok} \geq 5\%\),
- Class C: \(1.35 \geq (f_t/f_y)_k \geq 1.15\) and \( \leq 1.35\) and \( \varepsilon_{ok} \geq 7.5\%\),
- Class D: \(1.35 \geq (f_t/f_y)_k \geq 1.25\) and \( \leq 1.45\) and \( \varepsilon_{ok} \geq 8\%\).

The characteristic value of the ratio \((f_t/f_y)_k\) corresponds to the 5% fractile of the relation between actual tensile strength and actual yield stress.

\( \alpha_{ult} \) - the angle of the integrity bar with respect to the slab plane at failure (after the development of plastic deformations in the post-punching regime). The permissible angle values \( \alpha_{ult} \) are given in Table 1

**Table 1.** Permissible values of \( \alpha_{ult} \) angle depending on the steel class.
| $\alpha$ | Type of Bars | Ductility Class |
|---------|--------------|----------------|
| $0^\circ$ | straight bars | A |
| $20^\circ$ | straight bars | B |
| $25^\circ$ | straight bars | B, C or D |
| $\alpha \leq 40^\circ$ | inclined or bent-up bars | B, C or D |

where $\alpha$ is the angle of the integrity bars with respect to the slab plane (before punching occurs – Figure 3b),

$\gamma_c$ - partial safety factor for concrete material properties,

$f_{ck}$ - the characteristic value of the cylinder compressive strength of concrete,

$d_{res}$ - the distance between the centroid of the flexural reinforcement ratio and the centroid of the integrity reinforcement (Figure 2a, Figure 2b),

$b_{int}$ - the control perimeter activated by the integrity reinforcement after punching which can be calculated as:

$$b_{int} = \sum \left( s_{int} + \frac{\pi}{2} d_{res} \right)$$

where the summation refers to the groups of bars activated at the edge of the supported area and $s_{int}$ is equal to the width of the group of bars (Figure 2).

**Figure 2.** Arrangement of reinforcement against a progressive collapse, according to [5]:

a) from straight bars, b) from bent bars (described in the text)

### 2.3. Recommendations for the American standard ACI 352.1R [8]

It should be emphasized that the awareness of the need for increasing the protection against progressive collapse is demonstrated in consecutive versions of the American standard on designing bottom reinforcement of column strips. Originally, it was required that at least 50% of the reinforcement should be conducted 70 mm beyond the edge lines of a column, then (1989) – that at least two continuous bars should pass through a clear opening of a column, and recently (since 2002), referring to a standard – that all rods in a column strip, resulting from calculations in the span cross-section, should reach the support axis with continuity maintained.

According to the recommendations [8], for increasing the resistance of the structure system against a progressive collapse, the reinforcement should be determined based on the scheme presented in Figure 3. When considering an internal connection without beams, the bottom reinforcement with maintained continuity passing above the column in each main direction should have a surface of at least:

$$A_{sm} = \frac{0.5 w_u l_1 l_2}{\phi f_v}$$

(5)
where:

\( A_{\text{sm}} \) - minimum area of effectively continuous bottom bars in each principal direction placed over the support,

\( w_u \) - factored uniformly distributed load, but not less than twice the slab service dead load,

\( l_1, l_2 \) - centre-to-centre span in each principal direction,

\( f_y \) - yield stress of steel,

\( \phi = 0.9 \).

The transverse surface of the reinforcement \( A_{\text{sm}} \) may be, according to [8], decreased to two thirds of the value determined in the formula (5), if an edge slab-column connection is analysed, or to a half of the value, if a corner slab-column connection is analysed. If the calculated values \( A_{\text{sm}} \) in a given direction for adjacent spans differ, higher values should be applied in the designing process.

Bottom bars of \( A \) area may be classified [8] as continuous if they are:

1. overlapping beyond the border of the column at a distance of \( 2l_d \) from it, and the minimum length of the overlapping connection equals \( l_d \),
2. mechanically anchored right at the external corner surface in a way allowing the emergence of plastic stresses in the reinforcement.

![Figure 3. Connection model in the course of failure by punching, based on [8] (described in the text)](image-url)

In the above assumptions, we need to establish that the connection is acted upon by a design force for accidental situations \( V_{Ed} \), so the force determined at the least favourable configuration of variable loads while using increasing coefficients suitable for the accidental combination, determined on the basis of PN-EN-1990:2004 [9].

2.4. Comparison of test results with standard calculations

Based on the conducted research [10] [11] [12], analytical calculations [13] and numerical analyses [14] [15], it has been concluded that:

- In the course of the laboratory research, the proportion of the force loading the connection to the reinforcement load-bearing capacity, depending on the type of tested connection, was established at 2.5 to \(~2.9\). In standard formulae, e.g. (1), (3) and (5), the value of this proportion was assumed at \(~2.0\), so about 25%÷45% lower than revealed in the research.
- The difference between the values contained in the standards and the outcome of the research is probably caused by the omission of the extra bending which appears in the reinforcing bar in the point of slab-column connection. In tested situations, this impact indicates the decrease in the load-bearing capacity of these rods due to additional bending effects of around 10%÷20%, depending on the bar diameter.
- None of the standards brings satisfactory recommendations:
  - only Model Code 2010 [5] points at the difference in the load-bearing capacity resulting from the parameter of ductility of reinforcing steel. It is recommended that when using B class steel, the amount of steel should be increased by 32% in relation to class C. Moreover, using class A steel is practically ruled out. Unfortunately, the standard does not contain precise guidelines for edge or corner connections.
Providing more detailed guidelines for edge and corner connections is only contained in recommendations for the American standard [8].

All standards include the requirement of maintaining the continuity of the reinforcement in its entire length, so that the floor slab remains supported even after spalling the whole cover.

3. Remarks on the reinforcement over supports

3.1. Construction of reinforcement over supports

None of the given standards provides a precise location of the tying reinforcement. However, knowing that the reinforcement should cooperate in the situation of secondary structure when the column below had been destroyed, it becomes obvious that the tying reinforcement should be situated as low as possible. The reinforcement cannot be capable of breaking away from the structure and must resist the full force until detachment.

When designing - constructing horizontal ties, at the first stage of floor reinforcement external peripheral ties must be applied. The arrangement of this reinforcement and its laps (No. 1 Figure 4) must be conducted properly for tie beams. It applies especially to all concave corners (No. 2 Figure 4). If small parts (balconies, bays) protrude beyond the basic edge of the building, the tie beam reinforcement should not include these elements (No. 3 Figure 4). Obviously, the protruding elements should have their own independent peripheral reinforcement (No. 4 Figure 4).

Figure 4. Rules for arranging the peripheral tie beam reinforcement in slab-column structures, based on [16] (described in the text).

3.2. Reinforcement structure in spans

For the correct reinforcement of span zones, we need to assume that reinforcing bars passing over the columns should be arranged as the first (counting from the bottom). The remaining bottom reinforcement should be supported on it (Figure 5b1). This way of arranging the reinforcement, not increasing the cover too much, requires one layer of reinforcement to be deflected, which is not the most comfortable solution (Figure 5b2).

Sometimes a crossing reinforcement in two layers is applied (Figure 5b3). After the break-off of the cover, the whole reinforcement of one direction becomes somehow excluded from cooperation
(except for rods passing over the column). This situation of losing a cover is probable e.g. in case of a fire rapidly extinguished with water, advanced corrosion, etc.

![Diagram of reinforcement in slab-column floors](image)

**Figure 5.** The arrangement of reinforcement in slab-column floors:  
a) markings; b) ways of arranging the reinforcement (described in the text).

### 4. Conclusions

As for the plate-column structures subject to accidental loads, in particular, the removal of one of the columns, the onset of punching the available experimental material is only incidental.

The proposed calculation methods, including normally sanctioned ones, mainly result from theoretical considerations, in the absence of experimental verification, regarding both basic fragments and entire structures. With a small precision of calculation methods, the intuition and experience of the designer, in particular, in relation to the applied construction solutions, gain in importance.

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