Designing of Slab-Column Structures Due to Additional Loads

Miroslaw Wieczorek
Faculty of Civil Engineering, Silesian University of Technology, Gliwice, Poland
miroslaw.wieczorek.box@gmail.com

Abstract. The introduction of Eurocodes in the designing practices led to the change of methodology of the floor reinforcement, particularly for slab-column floors. Until now, the most important decisive factor was the determination and arrangement of reinforcement due to the limit states; any other problems were treated as additional information. At present designers - constructors of reinforcement in slab-column structures have to follow Eurocodes (PN-EN 1990:2004, PN-EN 1991-1-1:2004, PN-EN 1991-1-7:2008) in taking into account first and foremost the following threats accidental loadings and fires, as they influence further stages of the designing process. The article presents the most important design guidelines for slab-column structures due to accidental loads.

1. Introduction
Since the mid-twentieth century, monolithic slab-column systems have been increasingly used in residential, industrial and service construction, which is due to the undoubted advantages of this type of construction. However, in proportion to their popularity, the number of failures and building disasters of such systems, caused by various factors, also increases. With many advantages of slab-column constructions, one of the significant disadvantages is their lower resistance to exceptional loads, compared to longwall buildings or typical mullion-transom frameworks. Considering the issue of slab-column structure safety in the aspect of progressive collapse, it should be clearly emphasized that the Polish ([1-4]) and European ([5]) standards as well as other security concerns only threats caused by human errors, at any stage of life (existence) of the structure. These provisions do not include intentional actions aiming at the destruction of the object - i.e. acts of terrorist or suicidal nature.

The issue of the occurrence and impact of exceptional loads in Polish standards has so far been included practically only in the form of indirect construction regulations in PN-B-03264: 2002 [1]. Only the introduction of new PN-EN 1990: 2004 [6] and PN-EN 1991-1-1: 2004 [2] standards, and in particular PN-EN 1991-1-7: 2008 [3], set more explicit requirements. In this respect, irrespective of the design regulations currently contained in PN-EN 1992-1-1: 2008 [4], this topic is discussed much more in the Annex to the British Standard BS 8110 Part 1: 1997 [5] and in the Swiss Standard [7], as well as the design guidelines of Model Code 1990 [8] or Model Code 2010 [9]. The most detailed information related to exceptional loads, together with security guidelines against terrorism, can be found in the US regulations: Unified Facilities Criteria (UFC) [10] and Progressive Collapse Analysis and Design Guidelines for New Federal Offices Buildings and Major Modernization Projects [11]. General provisions are summarized in the International Building Code 2009 [12]. The listed provisions focus on defining preventive actions, marginalizing computational issues and analysing the situation...
after an accident. According to the authors, the apparent lack of correlation between individual provisions is not so much due to the novelty of the problem, but mainly due to the lack of adequate research understanding.

2. Problems addressed in standard regulations

The introduction of Eurocodes in the designing practices led to the change of methodology of floor reinforcement, particularly for slab-column floors. Until now, the most important decisive factor was the determination and arrangement of reinforcement due to limit states; any other problems were treated as additional information.

At present designers - constructors of reinforcement in slab-column structures have to follow Eurocodes (PN-EN 1990:2004 [6], PN-EN 1991-1-1:2004 [2], PN-EN 1991-1-7:2008 [3]) in taking into account first and foremost the following threats: accidental loadings – progressive collapses and fires because as they influence further stages of the designing process.

Only then is the reinforcement determined, arranged, depending on the ultimate limit state, and checked, and, if needed, enlarged, due to limit states of serviceability (scratches, deflections). As far as the scope of protection against damage caused by unintentional actions is concerned, the information contained in this bulletin is going to concern only slab-column structures, as they exhibit the lowest resistance to accidental loadings.

If slab-column structures meet the requirements of the correct designing against progressive collapse, these requirements will also be met in the case of other types of structures. As far as types of damage are concerned, the discussion will focus on local errors in single locations.

These means of protection can obviously concern multiple damage locations, however, this is a secondary effect. Particular attention should be paid to the fact that protection ought to secure us from two types of damages at a time:

- a) damages induced by mechanical (force) factors,
- b) damages induced by technological factors.

Regardless of the reason of the damage, the design must prevent the loss of stability of the structure as a whole in case of a local damage – so that it does not fold like a house of cards. Such stability should be ensured by e.g. proper tying walls or passage cores. It has been assumed that in the discussed situations such ties had been introduced. Such an approach is consistent with the strategies of designing members and the entire structures.

3. Structure designing strategies in the light of standard requirements

According to standard guidelines, the strategies of designing buildings exhibiting higher resistance to accidental loads can be divided into two groups:

1) Indirect methods, ensuring structural redundancy, i.e. minimal deformations of the structure upon its failure damage. The tie force method is the basic method of this group.

2) Direct methods, where individual load-bearing members are designed to carry accidental loads acting in any way. These include the key element method and alternate path method with the tensile membrane action discussed below.

According to the recommendations of the standard PN-EN 1991-1-7:2008 [3], not all buildings require an identical degree of protection against the actions of accidental loads. European regulations [3] divide the protection of buildings into four levels, depending on the consequence class.
Depending on the classification of a building or the level of protection, a proper method should be used in the designing process. According to [13] and following [3], to protect buildings against the actions caused by undefined factors, the following strategies have been recommended:

A. Consequences class 1
In this case, no additional actions are necessary, as long as the building is designed and utilized according to the provisions of Eurocodes (PN-EN 1990 ÷ PN-EN 1999).

B. Consequences class 2a (lower risk group)
For buildings belonging to this group, apart from adhering to the provisions of Eurocodes (PN-EN 1990 ÷ PN-EN 1999), it is necessary to ensure: effective horizontal ties for frames and effective anchorage of floors to walls.

C. Consequences class 2b (higher risk group)
For buildings belonging to this group, apart from adhering to all the provisions of Eurocodes (PN-EN 1990 ÷ PN-EN 1999), it is necessary to ensure: effective horizontal ties for frames, effective anchorage of floors to walls and vertical ties in all supporting columns and walls.

Also here, the additional protection is mentioned in structural recommendations from Eurocodes for reinforced concrete structures, thus the above list is only aimed at emphasizing the significance of these elements. Optionally, in this class, it is permitted [3] to check the building in order to determine if upon the notional removal of each supporting column, or any "nominal section of the wall", or any beam supporting a column, the building remains stable and that the scope of damage does not exceed a certain limit. It is assumed that in each version of the analysis only one element is removed. A more detailed description of this problem can be found in the next chapter.

D. Consequences class 3
A systematic risk analysis is required taking into account both foreseen and unforeseen loadings. This concise statement is a consequence of the basic assumption, stating that the provisions of Eurocodes guarantee the safety of the structure only at a consequences class 2. It is obvious that for buildings included in consequences class 3, the conditions for buildings included in consequences class 2b must be at least met. By default, also additional protection exceeding the one required in class 2b should be introduced, however, it is not exactly defined in Polish standard nor in administrative rules.

4. Elements of structural protection of buildings against accidental actions in the light of standard regulations

4.1. Tie force method
The tie force method is used e.g. in American guidelines [10]. For reinforced concrete buildings, the guidelines are based on the British standard on concrete [5]. The method has also been introduced to Eurocodes [3]. According to the method, in the load-bearing structure of a building, ties between its individual bearing elements are developed by ensuring reinforcement continuity and structure ductility. The task of the ties is to enhance the redistribution of internal forces in case a bearing element of the building becomes damaged. Usually, the typical structural elements of a building, designed according to conventional procedures, are used for this purpose. We distinguish: horizontal ties (peripheral, internal, connecting edge columns, and corner) and vertical ties.

4.1.1. Horizontal ties. All ties must be geometrically straight, and changes of their direction in order to bypass openings and similar discontinuities require using substitute bearing systems enabling a proper load transfer. Horizontal ties should be provided: around each storey at the floor and roof level
and internally at the floor levels in two right angle directions to tie the columns and walls securely to the structure and the remaining elements of the building.

The task of horizontal ties is to prevent detachment, displacement or separation of vertical bearing elements. This type of ties in the form of peripheral tie beams has been traditionally used in the Polish building industry for dozens of years in all types of structures, including the skeleton, wall, ceramic and concrete structures, as well as in hall structures. The exception includes buildings (obviously apart from historical ones) where distinct tie beams cannot be found. The use of continuous ties located as close as possible to the edges of the floors and rows of columns is required.

4.1.2. Vertical ties. The basic statement for vertical ties says that [3] "each column and wall should be tied continuously from the foundations to roof level". Here, it must be emphasized that vertical ties are anchored to the roof structure. This is necessary, as in case of fire the roof becomes elevated and the walls lose the support on the upper edge, which changes the pattern of their action against the free-standing cantilever. In case of buildings with a skeleton structure which should also include slab-column structures, columns and walls carrying vertical actions should be capable of resisting an accidental design tensile force [3]. The tensile force adopted in calculations should equal the largest permanent and variable load reaction applied to the column (wall) from a single storey. This is equal to taking into account the cutting of the column (wall) below.

4.2. Alternative path method
The alternative path method is recommended in the Department of Defense (DoD) [10] and General Services Administration (GSA) [11] guidelines for designing and renovating government and administrative buildings in the USA. In case of some buildings, the EC1 standard [3] allows to use it also as an alternative for the tie force method. The method focuses on analysing the building’s response after the removal of one main bearing element. The result of analyses conducted with the alternative path method is the determination of the scope of damage that can follow as a result of losing a bearing element.

The admissible limits of the local damage impact may vary depending on the type of building. Applying the smaller value of the following is recommended [3]: 15% of the floor surface or 100 m² on each of the two adjacent storeys (Figure 1). If removing one supporting element could cause damage greater that described above, such elements must be designed as so-called key elements (discussed below).

![Figure 1. The scope of damage upon removal of one column in a storey, based on [3]: a) horizontal projection, b) section: 1 - notionally removed column, 2 - damage of floors on two adjacent storeys, the area not exceeding 15% of the floor projection.](image)
4.3. Key element method
A key element should be capable of transferring accidental design action $A_p$. The $A_p$ action can be applied both horizontally and vertically, assuming that it simultaneously acts in one of these directions. The accidental action is applied both to the key element and any attached components, with regard to the strength of such components and their connections. The recommended $A_p = 34 \text{kN/m}^2$. According to the author, the concept of key elements which will not become damaged is derived from the English regulations drawn up on the basis of experiments following the analysis of Ronan Point collapse in London (http://en.wikipedia.org/wiki/Ronan_Point). However, the use of the concept might be seriously limited. Let us imagine a monolithic skeleton structure, where binding joists and columns are to be treated as key elements. In this case, the floors would require a bottom and an upper load of 34 kN/m$^2$. It would lead to an impossible over dimensioning of the entire structure. Similarly, if we assume the external column of the skeleton structure with lightweight curtain walls to be the key element, connected to the column only with ties transferring the wind load, almost every single column will meet the conditions for a key element. This approach can be used, for example, for checking the walls of stiffening cores which should, as a rule, be constructed as key elements. However, it needs to be emphasized that the load of 34 kN/m$^2$ should be treated as applied independently and separately on each storey, not on the entire height, which stems from the assumptions discussed before. As can be inferred from the above, when determining individual elements as key ones, a reasonable approach must be used, as both standard and administrative regulations lack any guidelines or recommendations on this matter.

5. Accidental loads
5.1. Combinations
Previously according to [14], and nowadays according to [6], accidental loads include: loads caused by vehicles, seismic activity, explosions; fires, and many more types, not discussed further. Design accidental loads $A_d$ form a part of design combinations of accidental loads, for which the design value of action effects $E_{d}$ is taken into account only if there is an actual possibility that they will occur. In any of such situations the following combinations of actions must be considered [6]:

$$E_{d} = E\left\{ \sum_{j=1}^{J} G_{k,j} \cdot P \cdot A_{d} \cdot \left( \psi_{1,1} \text{ lub } \psi_{2,1} \right) Q_{k,1} \cdot \left\{ \sum_{j=1}^{J} \psi_{2,j} Q_{k,j} \right\}, \ldots, J \geq 1; i > 1 \right\}

(1)

The choice between $\psi_{1,1}$ and $\psi_{2,1}$ is made according to a reliable design situation, where:
- $\psi_{1}$ - Factor for the frequent value of a variable action ($\psi_{1} \leq 1$) – according to [6] tab. A1.1,
- $\psi_{2}$ - Factor for the quasi-permanent value of a variable action ($\psi_{2} \leq 1$) – according to [6] tab. A1.1,
- $P$ - Relevant representative value of a prestressing action.

The recommendations contained in [6] point out the items to which attention should be paid so that the combinations of actions for accidental design situations could concern: either the accidental action only $A_d$ or either the situation following the accidental event ($A_d = 0$). It is recommended that [6] "in fire situations, irrespective of the effect of temperature on the material properties, the Ad value expresses the calculation value of the indirect fire effect".

5.2. Types of accidental loads included in the standards
5.2.1. Loads according to PN-EN 1991-1-7:2008. The basic statement for vertical ties says that [3] "each column and wall should be tied continuously from the foundations to roof level". Here, it must be emphasized that vertical ties are anchored to the roof structure. This is necessary, as in case of the fire, the roof becomes elevated and the walls lose the support on the upper edge, which changes the pattern of their action against the free-standing cantilever.

The scope of accidental actions imposed on buildings discussed in PN-EN 1991-1-7:2008 [3] has been limited to the following loads: imposed by the impact (vehicles and helicopters and rail
and ship traffic), caused by internal explosions and imposed by a local destruction of unknown cause. A thorough description of the methodology of load determination can be found in PN-EN 1991-1-7:2008 [3] and in [15] [16].

5.2.2. Loads according to PN-EN 1991-1-2:2006. Because of the drastic action exerted both on the structure and on people present inside the building, fire load is undesirable and dangerous. The action of fire – higher temperature should be considered especially while designing all parts of buildings and other civil engineering works with a great possibility of starting a fire resulting mainly from the purpose of a given room.

PN-EN 1991-1-2:2006 [17] contains the following requirements that have to be met in order to limit the risk of fire: "The construction works must be designed and built in such a way, that in the event of an outbreak of fire:

- load bearing resistance of the construction can be assumed for a specified period of time,
- the generation and spread of fire and smoke within the works are limited,
- the spread of fire to neighbouring construction works is limited,
- the occupants can leave the works or can be rescued by other means,
- safety of rescue teams is taken into consideration."

5.2.3. Loads according to PN-EN 1991-1-6:2007. PN-EN 1991-1-6:2007 [18] concerning the loads during execution concerns the following topics: impact on construction elements, impact due to equipment falling on the structure and impact due to falling of people.

6. Protection of slab-column structures – general description

Here we are going to discuss protection in case of:

a) destruction of a floor slab in the span (overload – acting load),
b) destruction of a single (any) supporting column,
c) destruction of a support zone, which must be considered in two variants: destruction of a support zone due to overload or destruction of a support zone due to premature removal of formwork or in frozen concrete.

A situation will be considered in which the skeleton structure is stiffened by cores or walls. Furthermore, it is assumed that the floors constitute a horizontal, practically non-deformable shields.

6.1. Protection of a floor slab in the span against destruction – overload

6.1.1. Internal field. The destruction of the span zone of slab-column structures can comprise one or more inter-column fields. Assuming the tendon systems are the method for securing the span zone of a slab-column structure against a progressive collapse, the way in which the span reinforcement is arranged becomes crucial. Thus, when analysing slab-column floors under accidental loads, the possibility of transferring the loads on spans only in one direction, \(x\) or \(y\), must be considered. Hence, the tendon system should be capable of transferring (Figure 2) the force of the value [8] both in \(x\) and \(y\) directions,

\[
F_x = 0.8 \cdot (g_k + q_k) \cdot l_{x,net}^* \cdot l_{x,net}
\]

\[
F_y = 0.8 \cdot (g_k + q_k) \cdot l_{y,net}^* \cdot l_{y,net}
\]

where:

- \(l_{x,net}, l_{y,net}\) – span values in the clear openings of columns for \(x\) and \(y\) directions respectively,
- \(l_{x,net}^*, l_{y,net}^*\) – width along which the load is gathered on the tendon.

6.1.2. Outermost field. To begin with, a neglected problem of destroying a slab resting on an outermost or corner column must be considered (Figure 4a, Figure 4b). The slab part being transferred into a tendon formally loses its axial stiffness, simultaneously transferring the \(F\) force from the span onto the supports.
The $F$ force transferred inside the structure (Figure 3 – support $A$) should be neutralized by the roof shielding, provided that the reinforcement has been properly anchored. The problem may lay in taking over the tendon anchorage force $F$ on the external support (Figure 3 – support $B$). The force must be transferred by the column onto adjacent storeys (upper and lower). Of course, the force will partially be taken over by the surrounding floor slab, but to an extent hard to evaluate. If, however, the damage is done to two parallel strips of the slab, the entire force is transferred onto outermost columns. The arrangement of these forces is presented in (Figure 4c).

Assuming for simplification that the width along which the loads are gathered reaches as far as half of the span and that the load is uniform, individual columns are going to be loaded independently on each storey with the following forces:

$$F = 0.8 \cdot \left(g_k + \gamma \cdot q_s\right) \cdot l_y \cdot l_z$$ (edge column - Figure 4a) \hspace{1cm} (4)

$$F = 0.4 \cdot \left(g_k + \gamma \cdot q_s\right) \cdot l_y \cdot l_z$$ (corner column - Figure 4b) \hspace{1cm} (5)

What has been discussed above is at the same time related to a very important recommendation – the load-bearing capacity of the reinforcement connecting the beams and columns will not exceed the necessary load-bearing capacity. Otherwise, it is obligatory to dimension the connection and elements converging therein as forces resulting from the load-bearing capacity of the reinforcement, not from acting loads. Of course, the tensile membrane action will in a way be supported by other reinforcements (e.g. upper, if not detached). These additional factors should be considered only as an unidentified reserve, not taken into account when dimensioning the reinforcement acting as a tendon in the state of failure.
The analysis contained in [19] has proven that the minimum value of total elongation $\varepsilon = 0.065$ implicitly assumed in [3] is lower than the required minimum value for class C steel (according to EC2 [4]). The requirement for class C is $\varepsilon \geq 0.075$. The actual elongatability is considerably greater.

The analysis presented in [19] indicated that the assumptions made in [3] with regard to tensile membrane action are safe. Lack of utilizing the full elongatability expresses additional influence, first of all, the limited freedom of the tendon elongation and its non-monotonic, more polygonal pattern.

6.2. Protection of a single (any) supporting column against destruction

In this part, we are going to discuss the loss of a load-bearing column. Usually, a few possibilities of a loss are considered as representative (Figure 5). While indicating columns the loss of which should be considered, it is crucial to take into account that the columns adjacent to the lost one will receive extra vertical and horizontal forces that should be carried by the structure. The loss of the column of course leads also to the regrouping of vertical forces in the remaining columns - these forces should be taken over by the columns, even keeping some reserve. As a standard, it is often assumed (in the case of simplified calculations) that the forces within columns are gathered from the adherent floor surface. This is a considerable simplification, especially with regard to the first internal supports.

A slab-column structure may be protected against a possible progressive collapse so that the displacements caused by the loss of columns are relatively small. In this case, the slabs are dimensioned as values of forces obtained with the assumption of a linear elasticity of the material, or elastic-plastic action of the structure is considered, with the possibility of the emergence of plastic hinges and more considerable deformations.

6.2.1. Destruction of an internal column. Considering the removal of an internal column (Figure 6) in case of uneven values of field spans, the sag of the tendon in the shorter direction should be assumed as the decisive factor $l_y < l_x$. With this assumption, forces and are determined based on the following formulae:

$$ F_x = \frac{1.6 \cdot \left[ k \cdot (g_k + \psi \cdot q_k) \cdot l_y \right]}{l_x^2} l_y $$

(6)

$$ F_y = 1.6 \cdot \left[ k \cdot (g_k + \psi \cdot q_k) \cdot l_x \right] l_y $$

(7)

assuming $l_x$ and $l_y$ as average values from cooperating spans. The reinforcement transferring forces $F_x$ and $F_y$ must be continuous.

When considering the loss of an internal column, transferring extra reactions caused by the elimination of the analysed column should be taken into account. It is important that, similarly to systems counted as elastic, the forces transferred onto the adjacent columns directly after losing the column are going to be higher than those resulting from the type of reaction transferred by the lost column. This stems from burdening the adjacent columns with end fixities found on these columns. This is especially important when plastic hinges emerge not in the column face but at a certain distance from the face. Values presented in Figure 7 should be treated as minimum values of extra forces. It is assumed that horizontal forces resulting from anchoring of tendons should be taken over by roof shielding in case an internal column is lost.
6.2.2. Destruction of an edge column. Losing an edge column is more dangerous to the structure than losing an internal column. If in case of an internal column the protection against the progress of collapse is guaranteed by two intersecting tendons, an outermost column is protected by practically one edge tendon. In case of a monolithic slab-column structure (Figure 8), the force $F_y$ within the edge tendon can be determined to refer entirely to the previous formulae based on the formula:

$$F_y = 1.6 \cdot \left[ (g_k + q_k) \cdot l_y \right] \cdot l_y^*$$  \hspace{1cm} (8)

where $l_y^*$ means the width along which the loads are gathered (Figure 8).

If a near-edge corner column is lost (Figure 9), the spread of damages is limited by the load-bearing capacity of horizontal forces exhibited by the floor slab immobilizing the outermost column. The loss of an edge column, as presented in Figure 9, leads to the emergence of a horizontal force acting on a corner column. Such an effect has also been noted in the research of a reinforced 9300×9300×100 mm concrete floor slab resting on 16 supports [20]. The flexural destruction of a floor slab occurred due to the lack of corner slab stiffness. Attention should be paid to the fact that in an actual slab-column structure a corner column would probably become destroyed first.

**Figure 5.** Location of columns potentially threatened with destruction. (1- internal column, 2 - internal near-edge column, 3 - edge column within longer edge, 4 - edge column within shorter edge; 5- corner column, 6- near-corner edge column).

**Figure 6.** Markings to be taken into consideration in an internal column.

**Figure 7.** Extra forces taken over by columns adjacent to the lost column $V$ - forces inside the lost column.

**Figure 8.** Slab-column floor system – situation of the edge column (1 – lost column).
6.2.3. **Destruction of a corner column.** The loss of a corner column brings about the most difficult situation. When it comes to a monolithic slab-column structure, the proposal most widespread in reference works [19] [21] is the one based on two models (Figure 10). The proposal was adopted from the methodology used for slab-wall structures in emergency removal of a wall. As the tests on models [22-24] have proved, the actual destruction process is different.

![Figure 9](image1.png) **Figure 9.** The principle of taking over horizontal forces acting on a near-corner edge column in accidental loads (1- failure zone; $F_n$ - horizontal component of force in the tendon)

![Figure 10](image2.png) **Figure 10.** Forming a diagonal angle strut compressed in slab-column structures (described in the text), according to [21]

Based on the conducted research [23] [24] that the image of the destruction obtained during the tests and its pattern differed considerably from mechanisms of the model destruction originally assumed, i.e. the "cantilever" model or the "envelope-shaped" model according to [19] [21]. As a result of losing the corner support the slab diagonal took the shape of a slightly elevated inverted shelf. The obtained change in the shape of the element in the diagonal section (the diagonal section changed from rectangular to arch-shaped) significantly increased the arm of internal forces, which caused e.g. a much greater load capacity than originally expected.

7. **Conclusions**

Of the discussed methods of securing a slab-column structure against uncontrolled development of a disaster, two methods deserve attention: the "flex method" and the "tension method". The bending method ensures the highest level of the system security after removing the pole, and thus meets the requirements of the standard [3] on limiting the damage zone. The disadvantage of this method is the significant cost resulting from the need to increase reinforcement, even three times or increase the dimensions of the cross-sections of the elements. Structural protection of the structure requires relatively little additional reinforcement. Sometimes, it does not require additional reinforcement, but some interference in the way the structure is reinforced. The disadvantage of this solution is the admission of significant deformations in the zone of damage to the support element. If the damaged support element is a column, all columns above the removed one will move. In addition, this method does not complement the limitation of the damage zone resulting from the provisions of the standard [3]. Limiting the damage zone in the vertical system can only be achieved by introducing technical floors capable of taking over all loads in the event of removal of any pole. These load-bearing technical floors have found application in some very tall buildings.

It should be expected for economic reasons that in relation to the skeletal structures in the near future we will use, practically only methods of protection when adopting the membrane model of their work. This method, with all its shortcomings, limits the development of the catastrophe without requiring a significant increase in the reinforcement in the system. It should be remembered, however, that the condition for this model of the system to work in the event of removal of the column, it is necessary to ensure the proper connection of the rods to constitute the replacement string system. As already emphasized, these rods can break, but they can never be pulled out of the structure. Of course, the reinforcement intended for work as a tendon must be made of high toughness steel (C grade steel).
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