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PROTECTION OF LARGE PANEL BUILDINGS AGAINST PROGRESSIVE COLLAPSE

ABSTRACT

With the large panel buildings being widely used, effective protection of buildings against the destructive effects of extraordinary loads becomes an important issue. Therefore, the article delves into the history of developing standards that regulate design basics of buildings made of large-scale elements.

The article deals with characteristic properties such as spatial stiffness by using a system of rigid transverse and longitudinal walls. The author emphasizes that a characteristic feature of the design of large panel buildings is the presence of monolithic joints between prefabricated elements in the ceiling and wall slabs. The article identifies the components that play an important role in ensuring sufficient reliability of large panel system buildings.

The article stipulates methods of accounting for accidental loads in designing large panel system buildings and covers application methods. The article covers the guidelines for protecting large panel buildings against progressive collapse.

The authors outline that the local destruction of a load-bearing wall in the zone adjacent to the outer edge is extremely dangerous due to a progressive collapse, and that is why it is important to take into account progressive collapse in structural design.

The article contains basic design recommendations for protecting large-panel buildings against the effects of extraordinary loads.

In conclusion, the authors opine that the methodology of protecting large panel buildings against the effects of a progressive collapse may be used for assessing the state of a structure damaged due to warfare.

In the case of war damage to large-panel buildings occurring in Ukraine, due to the extent of the destruction, the article recommends developing procedures aimed at reliability assessment with partial use of the methods described in the article. On top of that, the catalog of strengthening solutions for each large panel system should be developed and available in case if reconstruction is needed.

KEYWORDS: large panel buildings, spatial stiffness, monolithic joints, reliability, accidental loads, progressive collapse

ЗАХИСТ ВЕЛИКОПАНЕЛЬНИХ БУДІВЕЛЬ ВІД ПРОГРЕСУЮЧОГО ОБВАЛЕННЯ

АНОТАЦІЯ

У зв'язку з широким використанням велико-панельних будівель ефективний захист будівель від руйнівного впливу екстремальних наванта-

жень стає важливим питанням. Тому в статті розглядається історія розробки стандартів, які регулюватимуть принципи проєктування будівель з великих елементів.

У статті розглядаються такі характерні властивості, як просторова жорсткість за допомогою використання системи жорстких поперечних і поздовжніх стінок. Автор підкреслює, що характерною особливістю проєктування великопанельних будівель є наявність монолітних швів між збірними елементами в плитах стелі і стін. У статті вказані компоненти, які відіграють важливу роль для забезпечення достатньої надійності великопанельних будівель при їхньому проєктуванні.

У статті розглядаються методи обліку навантажень при виникненні аварії під час проєктування великопанельних будівель і сфера застосування цих методів. У статті конкретизовані та проаналізовані настанови щодо захисту великопанельних будівель від прогресуючого обвалення.

Через локальне руйнування несучої стіни в зоні, прилеглій до зовнішнього краю, що є надзвичайно небезпечно через можливість прогресуючого обвалення, автор наполягає на тому, що локальному руйнуванню необхідно приділяти особливу увагу при проєктуванні будівлі. У статті містяться основні проєктні рекомендації щодо захисту великопанельних будівель від впливу екстремальних навантажень.

У висновках автор висловлює думку, що методологія захисту великопанельних будівель від наслідків прогресуючого обвалення може використовуватися для оцінки технічного стану об'єкта, пошкодженого в результаті бойових дій.

У випадку пошкодження великопанельних будівель внаслідок війни в Україні, в залежності від масштабу руйнувань, рекомендується розробити послідовні принципи оцінки надійності об'єктів з частковим використанням методів, описаних у статті, а також розробити каталог рішень щодо посилення великопанельної системи будівлі у випадку можливості проведення реконструкції.

КЛЮЧОВІ СЛОВА: великопанельні будівлі, просторова жорсткість, монолітні шви, надійність, навантаження при аварії, прогресуюче обвалення

INTRODUCTION

During the use of buildings, in special situations, in addition to the standard permanent and variable loads included in the calculations, loads of accidental character may also occur. In most design situations, it is impossible to clearly determine both the type, size and frequency of occurrence of extraordinary loads. Therefore, effective protection of buildings against their destructive effects becomes an important issue. Despite difficulties, it is necessary to ensure that a building structure meets the minimum level of safety by adopting an appropriate combination of influencing factors during static-strength calculations and thus already taking the influence of additional accidental loads into account. Furthermore, satisfying design requirements developed on the basis of the results of experimental research and theoretical analysis is mandatory.

The issue of protecting structures from the effects of special loads applies to all types of structures. However, due to the limited degree of monolithicity and the fact that the load-bearing structures include simply supported systems, prefabricated structures are characterized by their lower ability to redistribute internal forces and are therefore more susceptible to a progressive collapse in typical monolithic structures.

THE BASIC PRINCIPLES OF DESIGNING THE STRUCTURE OF LARGE PANEL BUILDINGS

Standard setting regulations

Constructing buildings from large scale prefabricated elements was a new technology in Poland in 1960s, which is why textbook studies on designing such structures [1], [2] were considerably ahead of standard setting provisions [3].

During the early stages of the evolution of large panel buildings, two international organisations were working vigorously to develop international guidelines regarding the principles of designing buildings made of large scale elements [4]: CEB-FIP (European Committee for Concrete - International Federation for Prestressing) and CIB (International Council for Research and Innovation in Building and Construction), especially in the CIB W23 "Wall Structures" sub-committee; representatives of Poland actively participated in these efforts as well.

The first national standardisation deliverable regarding the designing of large panel system buildings was industry standard BN-74/8812-01 [3], approved in 1974.

In order to make the structure adequately resistant to local damage and limit the scope of potential damage, the PN-B-03264:1999 standard for designing concrete structures [5], harmonized with PN-EN 1992-1-1:2008 [6], emphasized the need to ensure that buildings are sufficiently cohesive by systematically connecting ceilings to walls and columns. The rules of shaping the reinforcements of specific components and their joints specified in these documents were generally consistent with the requirements of standard [3].

In the period of designing large-panel construction systems, the existing standards and guidelines allowed most of the technical problems that occurred during the development of these systems to be solved. In the cases that involved the use of entirely new solutions, which had not been tested practically and were not covered by existing standards or instructions, many experimental studies were carried out (Fig. 1), primarily in the Building Research Institute (Instytut Techniki Budowlanej). They generally concerned structural strength issues, but they also included the

Figure 1 - Studying the horizontal joint of the Wk-70 system (A. Pogorzelski, 1982)

verification and evaluation of insulation and acoustic properties of building envelopes. Additional analyses were focused on the issues of production, transport and storage of prefabricated elements [7].

Characteristic properties

The basic rule of designing large panel system buildings was to give them adequate spatial stiffness by using a system of rigid transverse and longitudinal walls which pass through vertically through the entire building and, usually, through its monolithic, underground part.

The walls constituted vertical partitions whose main task was to take over loads (mainly vertical ones) and transmit forces to the foundations of the building. In addition, due to their significant resistance to plane strain, the external walls prevented the twisting of the building's spatial structure in the course of bending. Therefore, the design assumptions posit that under the influence of wind, the cross sections of the structure move in parallel.

Transverse and longitudinal walls, which were the main components of the building's spatial structure, were treated as supports fixed in the monolithic, underground part of the building or in the ground.

Ceilings in large panel system buildings were treated as rigid horizontal partitions in the calculations (when taking over horizontal forces they act as beams walls bent in their plane), which was equivalent to the assumption that the horizontal contour of the cross section of the building structure would not change in the case of its strain.

A characteristic feature of the design of large panel system buildings is the presence of monolithic joints between prefabricated elements in the ceiling and wall slabs. These joints are locations where the strength properties of the slab changes, places in which its strains caused by internal forces induced by external influence on the structure, concrete shrinkage, temperature changes or incorrect filling of the joints are concentrated. The internal forces, both perpendicular (to the plane of the joint) and tensile, as well as parallel and shearing may cause cracks to appear in the joint.

Apart from the intensity of the influencing factors, as a result of which internal forces appear within the structure, the shape and width of such cracks may also affect the effectiveness of the reinforcement which ensures that the building retains spatial cohesion, i.e. the peripheral reinforcement (tie beams) which surrounds the structural walls at the ceiling level and the supporting reinforcement of the ceilings anchored in these tie beams or spreading from one ceiling span to another. The tie beams and the support reinforcement connect prefabricated panels into ceiling and wall slabs, thus ensuring that the building has adequate spatial stiffness. These components also play an important role in the creation of the secondary load bearing structure above the potentially damaged part of the building and in compensating for the strains in the interface of walls bearing different loads, as well as in taking over the tensile forces which appear in the wall as a result of an uneven settlement of the building.

The adoption of such assumptions when designing large panel buildings was intended to ensure their sufficient reliability.

Accidental loads

When designing large panel residential and public buildings, the possibility of extraordinary loads caused by the following factors were taken into account:

- gas explosion in enclosed rooms,
- an impact caused by a wheeled transport vehicle if the building is located near an active traffic lane.

Until the development of the standard [3], accidental loads were included in the design on the basis of CEB/FIP recommendations [4]. Some of the requirements of these documents can be applied directly to extraordinary events caused by other factors, e.g., warfare.

METHODS OF ACCOUNTING FOR ACCIDENTAL LOADS IN THE DESIGN OF LARGE PANEL SYSTEM BUILDINGS

Method I

The first method consists in estimating the most approximate magnitude of the extraordinary load and including it in the design of a building structure. In such approach, cracks with significant gap widths and permanent deformations in close proximity to the occurrence of extraordinary loads, are allowed. However, the structure should remain unchanged and capable of transferring loads safely, as well as allow the evacuation of residents and rescue operations.

Method II

In the second method, the possibility of local destruction of the structure is permissible. The magnitude of the extraordinary load, as in the first method, is not determined but the extent of local destruction is estimated. The essence of the protection is to endow the structure with a capability to shape a secondary load bearing structure around the site of local destruction, capable of transferring loads in a changed static diagram. The secondary system may be composed of, in addition to the supporting elements of the structure located outside the place of local destruction, elements which are non-structural in the primary system (e.g. concrete curtain walls), provided that the load bearing capacity of these elements and the method of connection with the rest of the structure ensures their appropriate contribution in the operation of the secondary system. In the zone of the secondary structure, significant cracks and displacements of structural elements are allowed in relation to each other, provided that the safety of residents is ensured.

Scope of application of the methods

Method I, consisting in allowing for extraordinary loads in the design, leads to increasing cross-sections of structural elements and their reinforcement. For this reason, its application is recommended for protecting structures with limited capabilities of creating a secondary load bearing structure in the event of local destruction. This type includes skeleton frame structures. According to observations, an explosion in a building with a skeleton frame structure usually destroys the infill walls and, sometimes, the ceilings, while the columns have a chance to resist the force of the explosion. On the other hand, designing a skeleton frame structure that would be resistant to a progressing collapse, when one of the columns is destroyed, is possible only to a limited extent.

Wall structure buildings (large panel system buildings) should be secured using method II. In special cases of the construction system of these buildings, certain parts of the structure or the whole require being secured in accordance with method I.

Combinations of both methods are possible, i.e., transferring extraordinary loads up to a specific magnitude by the primary structure and creating a secondary structure under higher load conditions.

In both methods of securing a building against the effects of extraordinary loads, due to the lower probability of the occurrence of these loads, the safety of the structure should be verified for the typical strength properties of materials and the magnitude of loads.

In the analysis of the structure, all strength reserves can be taken into account, including the full plasticity of the reinforcement, consequences of large rotations and displacements.

In the case of method I, the extraordinary load is added to the normal load, and in the case of method II, the normal load is only the load impacting the structure, but depending on the secondary structure in question, this load can be increased by an appropriate dynamic coefficient.

In the calculations, the permanent load, a long term variable load and 1/3 of the short-term variable load are adopted as the normal load, while the wind load is omitted.

GUIDELINES FOR PROTECTING LARGE PANEL BUILDINGS AGAINST PROGRESSIVE COLLAPSE

Protecting buildings according to Method I

Protecting buildings with the method I involves designing a structure capable of transferring extraordinary loads in addition to ordinary loads (permanent loads and long and short- term variable loads). The dynamic load caused by a gas explosion in a closed room can be replaced by a static force acting evenly on the walls and ceiling slabs. The value of this force depends on the strength and surface of the weakest partition or part of the partition in the room (window, door), which at the time of explosion is the first to be destroyed and acts as a valve, reducing the gas pressure on the remaining partitions and on the room where the explosion took place.

The magnitude of the horizontal forces caused by a vehicle impact is determined by the standard for the loading of structures with vehicles [6], where these forces range from 40 to 500 kN. For the calculations, it is assumed that the forces are applied at a height of 1.2 m above the level of the surrounding area.

Protecting buildings according to Method I

Local destruction of the structure is defined as the destruction of two walls conjoined in the corner of the room where the explosion occurred and when one of these walls is the outer wall [4]. Two cases of the local destruction zone range can be distinguished depending on the resistance of the wall to a uniformly distributed load (blast) less or more than 10 kN/m^2.

In the case of walls which can withstand a blast below 10 kN/m^2, the length of the destroyed section (l_w) is equal to the distance between the vertical stiffeners of the wall or between the stiffener and the free edge. However, in the case of walls which can resist a larger blasts, the length of section l_w can be assumed to be 3.6 m (for end walls or 1.8 m (for internal walls in cases where there is no weakness in the section of the wall adjacent to the estimated destruction zone in the form of a vertical joint, the edge of a door or window opening.

The secondary load bearing structure, located over a part of the locally damaged building, is shaped in accordance with the newly emergent conditions. Depending on the location of the zone of local damage to the load bearing walls, this may be a cantilever, tension rod or beam structure.

One of the most frequent cases of local damage in buildings with walls as the load bearing structure is the destruction of a fragment of internal walls.

Depending on the structural solution, there are two possible situations [8]:

- in the case of insufficient connection between the ceiling, tie beams and the wall the ceiling above the damaged part may fall off or hang from the tie beam acting as a tie rod. This situation is undesirable due to the necessity to transmit tensile forces through the wall above the destruction zone (Fig. 2a),
- in the case of an effective connection, the tie beam remains attached to the wall above it, and the loads above the destruction zone are transferred by the walls and tie beams, which undergo stretching (Fig. 2b).

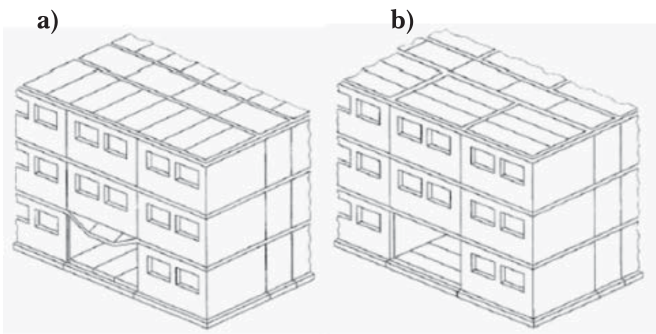

Figure 2 - Situations following the destruction of a section of the external wall

a) a building with a longitudinal load bearing structure (secondary beam structure)

b) a building with a transverse girder bearing structure

Without the central support (which is a part of internal load bearing walls), the ceiling may form a tie rod structure (Fig. 3b), in which the reinforcement placed along the span of the ceiling and the reinforcement of the tie beam transmit tensile forces together, thus forming a structure that spans in two directions; in addition to a relatively significant cross section of the reinforcement, it must have the correct shape in order to prevent the bars in the collapse zone from breaking off the concrete. When creating a secondary structure, the preferred solution is to connect the floor panels using a loop with threaded reinforcement (Fig. 3c) Stirrups connecting the upper and lower bars further prevent the bars from breaking off.

When the ceiling spans more than 4.0 m, it is assumed that the destruction of the central support causes the ceiling joint above this support to be broken and the ceiling collapses. However, the ceiling joint on an undamaged support should be sufficiently strong and elastic to prevent the ceiling from falling off said support. Loop connections work very well in such conditions (Fig. 3c).

A ceiling falling off a support imparts a dynamic load on the ceiling below. When the connection between adjacent ceilings or between a ceiling and a load-bearing wall is made on a support, the load-bearing capacity of the ceiling is usually sufficient to take over this load. If such a connection is absent, the shock caused by the fall of the upper slabs may cause the ceiling slab to slide off the support and result in the rapid successive collapse of all the ceilings down to the ground floor (Fig. 3a). Such disasters occurred several times during the installation of staircases when stair flights and platform slabs were not connected with appropriate anchors.

In the event that parts of the load bearing wall adjacent to the edge are destroyed the upper storeys of the wall form a multi storey or single storey cantilever structure (Fig. 4).

If the ceiling can freely move horizontally over the wall, then the wall works as a flat support in which the tensile force is transmitted only by the reinforcement of the tie beams. Meanwhile, when the joint between a ceiling and a wall can transmit a certain tangential force, a spatial structure is created in which the ceiling takes over a portion of the horizontal force caused by the bending moment.

The curtain (external) wall usually constitutes the load of the cantilever only. However, when this wall is sufficiently rigid and properly connected to the structure, it can work in conjunction with the load bearing wall as an additional component of the spatial cantilever.

In a building whose ceiling rests on its perimeter, the situation is more favourable due to the work of the ceiling without a support, the upper floor wall, as well as the beam broken in the plane.

WORKING CONDITIONS OF THE LOAD BEARING CANTILEVER STRUCTURE

Local destruction of a load bearing wall in the zone adjacent to the outer edge is extremely dangerous due to the possibility of a progressive collapse, which

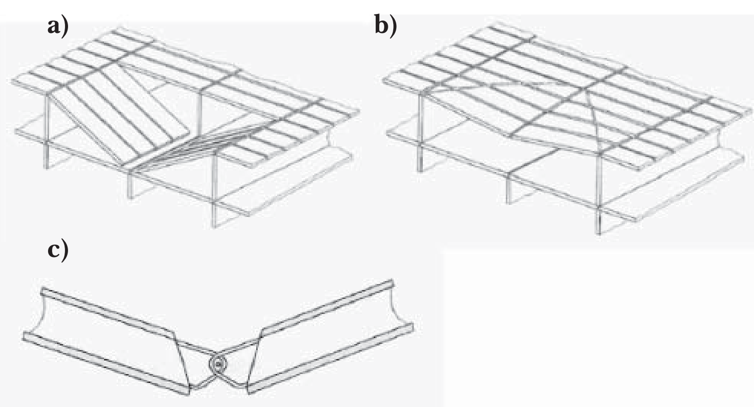

Figure 3 - Consequences of damage to an internal load-bearing wall

a) parts of the ceiling in the destruction zone fall off

b) forming a secondary load-bearing structure of tie rod type

c) loop connection in the collapse zone

a) **b)**

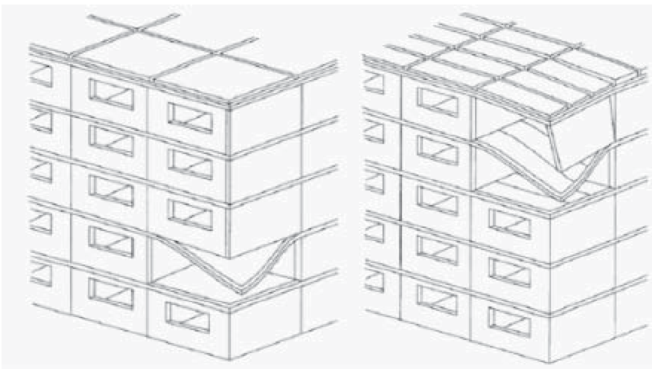

Figure 4 - A secondary cantilever structure after the destruction of external walls in the corner of the building
a) a multi-storey structure
b) a single-storey structure

is why particular attention is paid to this scenario when designing a building. The resulting structural protections in horizontal tie beams, appropriate vertical joints and lintel reinforcements applied throughout the building also provide adequate protection in other cases of local damage to a load bearing wall.

The secondary load bearing cantilever (multi-storey) structure is characterized by structural continuity in its vertical joint. In this case, the shear forces occur on the lowest storey, in the immediate vicinity of the destroyed part. The strength of these forces and their distribution along the joint depends on the height of the cantilever and the size of the displacements of the joint's edges δ, which accompany shear forces T′ (Fig. 5).

Separating the lower part of a multi-storey cantilever from the rest does not yet equate its destruction. If the part is properly connected to the rest of the storey at the ceiling level (with a tie beam or a tie beam and a ceiling), a new single-storey cantilever structure may be created there. In addition to individual components working together, this structure is accompanied by significant displacements and cracks, and the shear force in the vertical joint is no longer opposed by its cohesion, but by the frictional force in the horizontal force pressure zone caused by the bending moment. From a utility centred perspective, the part of the building in which such a secondary structure was formed is destroyed and requires thorough repair, as opposed to the preservation condition of the multi-storey cantilever, which needs practically no repairs. However, due to the fact that there is no danger to human life, it can be assumed that the limit state of the structure's load bearing capacity has not been exceeded, which means that in a post-emergency situation, the structure will meet safety conditions. The conditions in which the partially damaged structure is used are very complex and related to the fact that the formation of a secondary load bearing structure is a dynamic phenomenon with unspecified parameters. Therefore, the following findings are used in the calculations, depending on the maximum shear force $Q_{max} = T_{p,max} \cdot h$ (where $T_{p,maxca}$ – maximum unit force shear in the vertical joint, h- height of the storey) in the vertical joint section equal to the storey height of the multi-storey cantilever, calculated on the assumption that the rigidity of the joint $C= C_{Uk}$ (rigidity of the joint under stress):

- when the analysis of the operation of the multi-storey cantilever shows that $Q_{max} \leq U_{Tk}$ - the structure can be considered sufficiently safe,
- when $U_{Tk}<Q_{max}\leq 2U_{Tk}$ – the possibility of creating a single-storey cantilever should be ensured for the construction, whereby at $Q_{max} < 1.5\ _{Tk}$ a dynamic coefficient of 1.1 is adopted for calculations, and when $Q_{max} > 1.5\ U_{Tk}$ -1.2,
- with $Q_{max}>2U_{Tk}$ the situation requires structural intervention; the simplest procedure is to

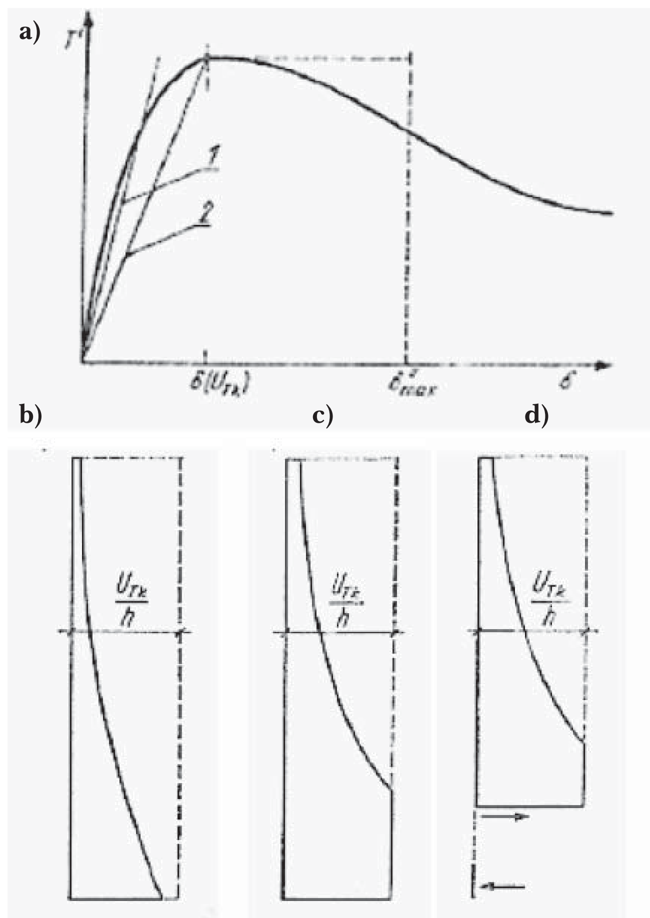

Figure 5 - Working conditions of a vertical joint in a multi-storey cantilever
a) T′(δ) dependence
b) distribution of stresses T′ in the case when $\delta< \delta(U_{Tk})$
c) distribution of stresses T′ in the case when $\delta(U_{Tk}) < \delta< \delta max^{II}$
d) distribution of stresses T′ in the case when $\delta> \delta max^{II}$

increase the load bearing capacity of the joint so that $Q_{max} < 2.0\ U_{Tk}$ or look for other solutions.

In principle, the possibility of local damage to the wall should be verified for all storeys of the building. However, usually, the consideration of the consequences of damage to the wall only on the lowest storey and on the third storey from the top is sufficient. In the first case, the largest Q_{max} will occur due to the greatest height of the cantilever, and in the second case, a single-storey cantilever is the only possible solution. The structures on the remaining storeys should be shaped according to the obtained results.

The multi-storey cantilever is connected to the rest of the building structure by ceilings, therefore, in the analysis of this static diagram, it can be assumed that the cantilever may deform only in the vertical direction by a value of δ equal to the displacements of the vertical joint.

The single-storey cantilever in the building can be loaded with one or two ceilings, depending on whether the ceiling above the lower storey has detached from the wall of the upper storey or is still joined.

BASIC DESIGN RECOMMENDATIONS

Due to the protection of large-panel buildings against the effects of extraordinary loads, the construction requirements complement the general construction requirements in terms of joining prefabricated elements and maintaining spatial rigidity of buildings specified, among others, in the industry standard [3].

A distinction should be made between the buildings with ceilings anchored on the cantilever (supporting reinforcement connected with the reinforcement of adjacent spans or anchored in a tie beam) and the buildings with simply supported ceilings. In the first case, in the conditions of the secondary load bearing structure, the cooperation of the ceiling with the load bearing wall may be taken into account, and in the second case, this cooperation cannot be taken into account. In both cases, on the other hand, an appropriate connection of the tie beam with the load-bearing wall is required (except when the entire reinforcement of the tie beam is located in the wall slab, the so-called concealed tie beam). It is also very important to connect the rods of the tie beam in a way that ensures full load bearing capacity of the reinforcement.

The safety of the building under extraordinary load conditions may not be checked analytically if the cross-section of the tie beam reinforcement (made of class AIII rebars) is not less than:

- 2.3 cm^2 – in the case of anchored ceilings, with a span of up to 6 m (in all walls or in the case of simply supported ceilings with a span of up to 4.8 m (in internal walls),
- 3.4 cm^2 – in the case of simply supported ceilings with a span of up to 4.8 m (in the end wall) or in the case of simply supported ceilings with a span of 4.8 to 6.0 m (in the internal walls),
- 4.6 cm^2 - in the case of simply supported ceilings with a span of 4.8 to 6.0 m (in the end wall)

The cooperation of the ceiling with the load bearing walls reduces the longitudinal reinforcement in the tie beam, but does not reduce the shear forces resulting from the connection of the tie beam with the load bearing wall

Figure 6 - Effects of a gas explosion in a large-panel building (Łódź, Retkinia housing estate)

CONCLUSIONS

When creating systematic solutions for large panel buildings, rules were developed to protect these constructions against a progressive collapse, which may occur in extraordinary situations, e.g. as a result of gas explosion [1]÷[4].

The recommended method of verifying the safety status of large panel buildings with a wall load bearing structure is method II (point 3.2) which assumes the possibility of creating secondary load bearing structures in the main structure after the local destruction of the fragments of support elements.

The methodology of protecting large panel buildings against the effects of a progressive collapse may be the basis for assessing the technical condition of an object after the occurrence of damage related to warfare (Ukraine).

In Poland, extraordinary events occurred occasionally, a spectacular example of which was the gas explosion in December 1983 in the Retkinia housing estate in Łódź (Fig. 6). As a result, the outer (load bearing) end wall at the height of the two lower storeys was damaged, but the building remained stable (after the creation of a multi-storey cantilever secondary structure. As a result of the explosion, 8 people were killed. In the aftermath of the incident, it was decided to place valves shutting off the gas supply outside each building in Poland and mark them accordingly as well as to reduce the pressure in the gas pipelines directly supplying inhabited facilities. After the removal of the debris, the missing part of the building was rebuilt in June 1984.

In the case of war damage to large-panel buildings occurring in Ukraine, due to the extent of destruction, it is recommended that consistent principles be developed for assessing the reliability of facilities (with partial use of the methods described in the article) and, in the case of the possibility of reconstruction that a catalogue of reinforcement solutions dedicated to each large panel system be created.

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