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DESIGN OF HIGH-RISE BUILDINGS

LECTURE NOTES

(for students of the second (master's) level of higher education all forms of education speciality 192 – Building and Civil Engineering, of education program "Industrial and civil construction")

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CONTENT

INTRODUCTION

Modern urban planning is impossible without the development of high-rise construction - a complex multidisciplinary industry that combines innovative architectural approaches, advanced engineering technologies and complex constructive solutions. The emergence and evolution of high-rise buildings are due to the intensive growth of megacities, limited free are and the need for rational use of space.

The purpose of this academic discipline is to form in higher education students a systematic understanding of the basics of designing high-rise buildings, taking into account architectural, functional-planning, constructive, technological, engineering and operational requirements. The study of the discipline is based on modern regulatory documents, engineering practices and trends in sustainable design.

The lecture notes cover a wide range of topics, starting from the requirements for site selection and building form, ending with innovative approaches to seismic protection, energy saving and greening of skyscrapers. Special attention is paid to constructive systems of the type "soil base - foundation - structure", the use of software complexes in design, monitoring of the technical condition of objects and fire safety.

The content of the discipline is built on an interdisciplinary approach that integrates knowledge of architecture, structural mechanics, geotechnics, resistance of materials, engineering networks, safety and ecology. This makes it possible to train highly qualified specialists who are able to solve complex design problems and participate in creating the infrastructure of the future.

The material is presented in a logical sequence in order to facilitate assimilation, the formation of professional thinking and practical skills in the field of design of high-rise buildings.

Topic 1 VOLUME-PLANNING AND CONSTRUCTIVE SOLUTIONS OF HIGH-RISE BUILDINGS

1.1 Requirements for the selection and planning of a construction site

The selection of a site for the construction of a high-rise building is carried out based on the conclusions of the urban planning justification, taking into account the results of additional research and development, including:

– visual and landscape analysis of the location of a high-rise building, taking into account its volumetric and spatial perception in a given development area and in the surrounding areas;

– analysis of the possibility of geological risk and the results of developing a predictive assessment of changes in the geological environment, engineering-geological and hydrogeological conditions under the influence of expected loads from a high-rise building on the construction site and the adjacent territory;

– analysis of the impact of new construction on the technical condition of structures and engineering networks of existing buildings and structures and landscaping facilities located in the surrounding area;

– calculations of the carrying capacity of urban transport and engineering infrastructure capacities, taking into account additional loads from a high-rise building;

– light-climatic calculations of the level of insolation and natural lighting for a high-rise building and surrounding buildings for compliance with current standards;

– calculations of expected sound levels and sound pressures;

– calculations of the provision of the population with green areas for public use and public facilities within the designed functional and planning formations in the territories adjacent to the construction site of a high-rise building;

– assessment of microclimatic indicators of the ambient air environment, concentration of pollutants, level of external noise and vibration;

– the impact of aerodynamic indicators in the high-rise construction zone (speed and direction of wind flows, turbulence zones, wind shear, rarefied zones, etc.) on the functioning of ventilation systems;

– heating of existing buildings, exhaust of gas combustion products, especially in buildings equipped with autonomous instantaneous water heaters (columns, boilers, etc.);

– analysis of the interaction between a high-rise building and aircraft in the areas where airports and airfields are located, ground infrastructure facilities, radio-technical means of air traffic control, radio navigation, landing, communications and meteorological services are located, including taking into account the interaction of aviation noise on a high-rise building and the high-rise building on aircraft maneuvering in the airfield areas.

It is not recommended to locate high-rise buildings in airfield areas and in aircraft maneuvering zones on the territory of Ukraine, as well as in residential development restriction zones around the airfield due to the negative impact of aviation noise, electromagnetic radiation and other impacts and risks. The location of high-rise buildings in residential development restriction zones is decided on the basis of the conclusions of the sanitary and epidemiological examination.

Planning of the construction site for a high-rise building must be carried out taking into account the requirements for organizing an obstacle-free environment for groups of the population with limited mobility in accordance with DBN V.2.2-17.

To ensure access and circular movement of fire equipment near a high-rise building, it is necessary to provide roundabouts.

If it is impossible to fulfill this point, it is necessary to develop a diagram of the entrance and location of fire equipment in the area adjacent to the high-rise building as part of the design documentation, with its approval by the state fire supervision authorities.

The planning of the construction site should ensure the possibility of separate operation of different functional parts of the high-rise building. The territory that belongs to the residential part of the high-rise building, including the entrances and entrances, is recommended to be designed separately, which is determined in the design task.

It is recommended that the construction site of a high-rise building be designed with a high level of landscaping quality and provide for the arrangement of green spaces and recreation areas. In this case, the development of these areas should be carried out by placing them on the stylobate surfaces, arranging internal recreational spaces, winter gardens, sports halls, etc.

It is recommended that entrances and exits from the construction site be provided on the local street network or local thoroughfares of main streets of urban significance.

When planning a construction site, it is necessary to provide passageways for the effective evacuation of people from a high-rise building.

The composition of the construction site zones and the requirements for them are determined in the design task. It is necessary to take into account the arrangement of parking areas for residents, employees, and visitors.

It is not recommended to locate high-rise buildings at a distance of less than 100 m from sources of vibration and noise (metropolitan, railway or other high-speed modes of transport, etc.). When locating high-rise buildings at a distance of less than 100 m from the specified sources of vibration and noise, it is necessary to carry out a special justification in accordance with DBN V.2.3-7, taking into account sanitary standards in accordance with DSP 173, other factors ("effect", vibration creep of the soil base, etc.) and coordination with the city metro service with the participation of the design organization-developer of the project of tunnel structures and buildings in the zone of influence of the construction of a high-rise building.

Spatial planning solutions. The general structure, list and areas of functional elements, number of storeys, height of the building and number of underground floors are determined in the design task taking into account the requirements of DBN V.2.2-9, DBN V.2.2-15, DBN V.1.1-7 and the provisions of this document. Typological requirements that do not contradict fire-fighting, sanitary-epidemiological, environmental and other regulatory requirements for high-rise buildings should be adopted in accordance with DBN V.2.2-15, DBN V.2.2-9.

The height of fire compartments should not exceed 50 m. The height of technical floors is determined taking into account structural, technical, fire-fighting and sanitary-epidemiological requirements.

The type and number of stairwells is determined by calculation in the design documentation in agreement with the state fire supervision authorities. In high-rise buildings, service premises should be provided for the central high-rise building control point (CHCP) or dispatcher, building security, fire station, operation and monitoring service for the condition of the main load-bearing structures and engineering systems, etc. The list, area and requirements for the placement of service premises, the possibility of their blocking or joint location are determined in the design task and design documentation, taking into account technological, sanitary-epidemiological and fire-prevention requirements, as well as technical conditions for the design of the relevant services operating engineering networks.

It is recommended to locate office premises closer to the main entrance of a high-rise building on the first or basement floors with access to the lobby or directly to the street and ensuring their protection from unauthorized access.

Office premises with long-term (24-hour) occupancy must have natural lighting and an individual sanitary facility.

The list of built-in and built-in-attached premises, premises located in underground and basement floors, as well as premises without natural lighting in residential and public buildings is determined in accordance with DBN V.2.2-15, DBN V.2.2-9 in the design task and project documentation, taking into account sanitary and epidemiological and fire safety standards. The location of public premises in a residential building is carried out in accordance with DBN V.2.2-15 and additional fire and sanitary and epidemiological requirements, including requirements for noise protection of residential premises

It is recommended that bathroom facilities in apartments be located adjacent to common corridors.

Public premises may be located on the first, second, third and basement floors of residential buildings in accordance with DBN.V.2.2-15. The placement of public premises on other floors is determined in the design task, subject to compliance with all sanitary and epidemiological and fire safety standards and is agreed with the state sanitary and epidemiological and fire supervision authorities.

The location on the technical floors of premises for other purposes is permitted with special justification and is carried out taking into account planning measures to prevent the impact of noise, vibration and electromagnetic fields of engineering equipment on the nearest residential premises with permanent occupancy and in agreement with the state fire and sanitary and epidemiological supervision authorities. The slope and width of staircases and ramps, the height of steps, the width of the tread and stairwell are determined in accordance with DBN V.2.2-9, DBN V.2.2-15 taking into account the functional purpose of the high-rise building. In this case, the width of the staircases must be at least 1.2 m. The distance between the staircases must be at least 0.12 m (in the light). Recreational and summer rooms of a high-rise building located above 73.5 m must be glazed and have appropriate fences for safety and reducing psychological discomfort - people afraid of heights

When glazing facades completely, it is recommended to provide constructive measures (fences) on the inside up to a level of 1.2 m from the floor in order to ensure the safety of people and reduce psychological discomfort - people who are afraid of heights.

Windows in high-rise buildings located above 73.5 m must, for safety reasons, be equipped with latches that allow the opening angle of the sash elements to be adjusted, and with reinforced curtains designed for high-speed wind pressure. All window sashes must open inwards.

Requirements for landscaping and improvement of the surfaces of high-rise residential and public buildings, for the arrangement of internal recreation areas and winter gardens are determined taking into account the requirements of current regulations and sanitary and epidemiological requirements, including those for soil.

When designing facade systems, it is necessary to take into account the requirements for the installation of air conditioners, advertising and the organization of lighting at night. In a high-rise building, it is necessary to provide means for repairing and cleaning facades and their glazing elements.

1.2 Form formation as an architectural and artistic image of high-rise buildings

The formation of the volumetric and spatial structure of high-rise buildings is carried out taking into account various aspects - natural and climatic conditions, urban planning significance, functional purpose, the possibility of applying modern constructive and technological solutions, engineering systems, etc.

The influencing factor in creating the form of a high-rise building is natural and climatic conditions. When creating the form of an object, it is necessary to take into account temperature differences, changes in atmospheric pressure, perception of wind and aerodynamic influences, solar radiation and insolation. Thus, simple forms are transformed into forms that ensure rational perception of wind loads, improve aerodynamic properties, create comfortable conditions for natural lighting, ventilation, insolation of premises, etc. An example of the formation of the form of an object from the conditions of perception of wind loads and solar radiation, see Fig. 1.1, aerodynamic influences and solar insolation, see Fig. 1.2.

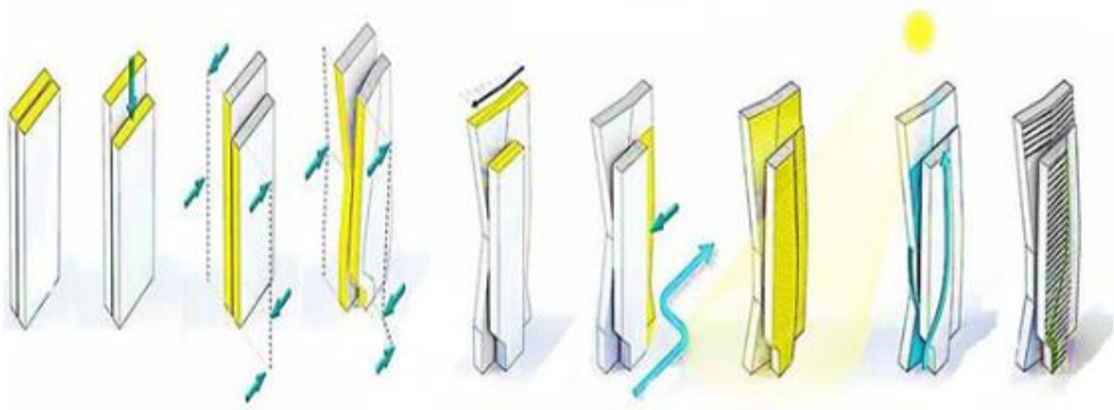


Figure 1.1 – Formation of a high-altitude object under conditions of wind and solar radiation

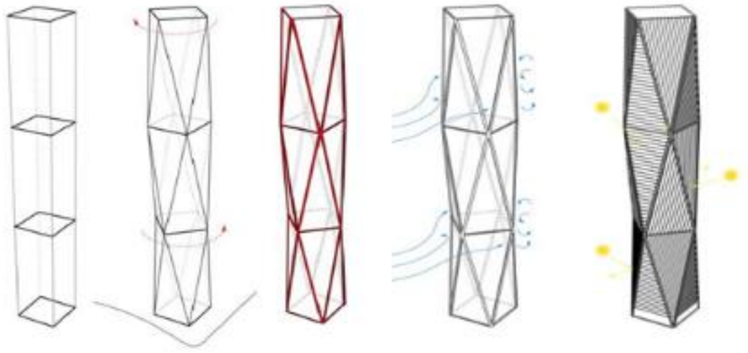


Figure 1.2 – Formation of a high-altitude object under the conditions of aerodynamic influences and solar insolation

In the world practice of high-rise form formation, the following methods of identifying the shape of high-rise volumes can be distinguished. Since the shapes of most high-rise buildings are based on simple geometric shapes, it is possible to distinguish a separate group of high-rise buildings that are created by a holistic volumetric form.

A solidly formed three-dimensional shape - has a clearly pronounced silhouette contour. Using techniques for identifying the composition of simple volumes, an architectural and artistic image of the object is achieved (Fig. 1.3)

a) b) c)

Figure 1.3 – Integrally formed composite volumes of the form:

a – “Aon Center” in Chicago; b – “Swiss Re Headquarters” in London;

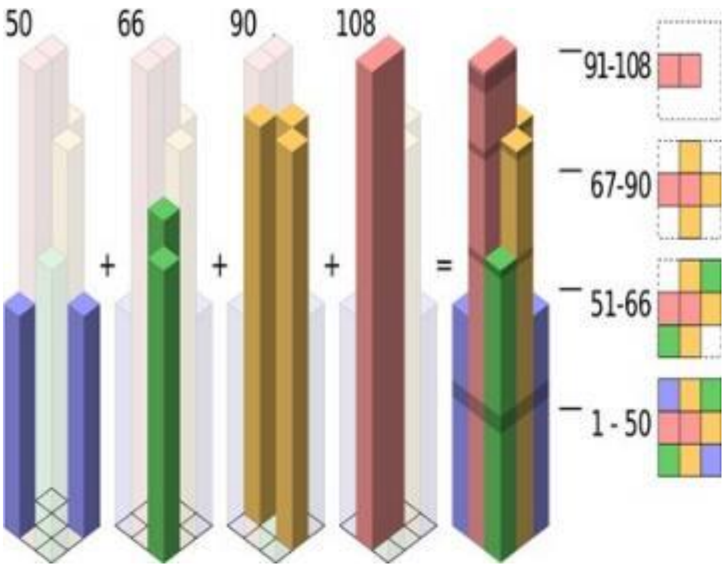
c – “World Trade Center Tower 1” in New York

Creating a stepped compositional structure – the construction of a form is based on the combination of vertical volumetric elements in height, which gradually reduce its volume and size. An example is a building “Willis Tower” (Fig. 1.4), where the floor blocks proportionally reduce their size and narrow the composition of the form. The basis of the plan composition is four modular squares, which increase in volume in height, forming a stepped composition.

An equally striking example is the “Bankof China Tower” (Fig. 1.5). The building is designed in the High-Tech style and represents a harmonious combination of modern architecture and traditional Chinese design. It is called a “geometric work of art”.

The spectacular appearance, consisting of simple geometric shapes (Fig. 1.5, a, b) form a three-dimensional structure (Fig. 1.5, b), and innovative design solutions (Fig. 1.5, c) create an unusual crystal-like shape. The source of inspiration for the author of the project, an American of Chinese origin, Bei Yumin, was the bamboo plant, which symbolizes strength and vitality in Eastern culture.

a)



b)

Figure 1.4 – Creating a stepped compositional structure “Willis Tower” (formerly known as “Sears Tower”) in Chicago: a – combination of vertical volumetric elements in height; b – general view of a high-rise building “Willis Tower”

The entire structure rests on five steel columns located at the corners of the building. The columns are surrounded by triangular frames (Fig. 1.5, c). The walls are made of giant glass triangles that frame metal frames. Due to the unusual shape and reflection of sunlight, the building resemble a huge crystal. Such an unusual multifaceted shape of the building was created not only for beauty. Thanks to this shape, the building can withstand powerful typhoons.

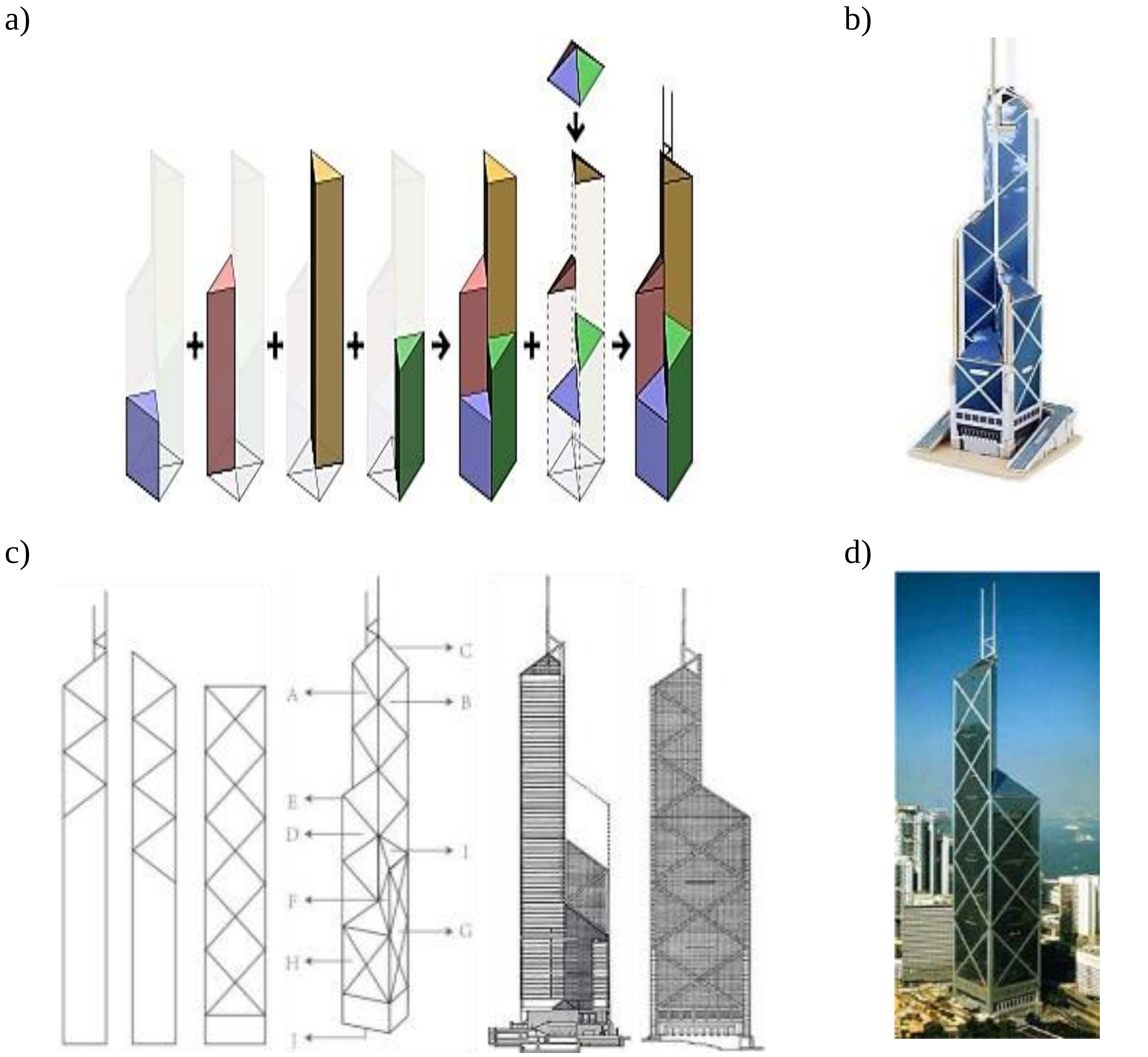


Figure 1.5 – Creating a three-dimensional form of the “Bankof China Tower” building in Hong Kong: a – simple geometric shapes; b – volumetric and spatial structure; c – constructive solution; d – general view of the building

Transformation - formation of form due to twisting of elements relative to the compositional center. Twisting or shifting of volume-planning elements (sections, floors, blocks) gives the form dynamism, spatial activity of all facades (Fig. 1.6).

a)

b)

c)

d)

Figure 1.6 – Transformation of the volumetric shape of high-rise buildings: a – twisting of the external facade relative to the center of the building; b – twisting relative to the compositional centers; c – twisting of volume-planning elements; d – shift of volume-planning elements

For example, the volumetric and spatial composition of the building “Turning Torso” (translated from English as a human torso that unfolds). In the form of this building, architect Santiago Calatrava recreated the natural movements of a person. The prototype of the unusual form was his sculpture – “Turning Torso”. The building consists of 9 pentagonal prisms, which are transformed relative to the central axis at a certain angle, as if a person is unfolding (Fig. 1.7).

a) b) c)

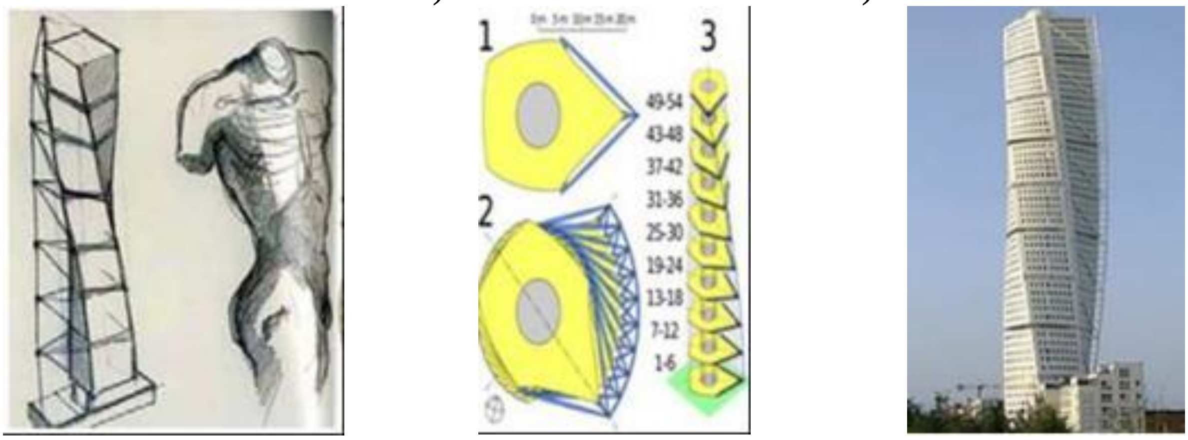


Figure 1.7 – Volumetric and spatial composition of the “Turning Torso” building: a – twisting of the external facade relative to the center of the building; b – the torso of a person unfolding; c – the general view of the building

The record holder among spiral structures is “Infinity Tower” (Kaiyang Tower (known as “Infinity Tower” during construction) is a residential skyscraper inUAE, Dubai. Building has an unusual spiral shape that dynamically stretches into infinity.

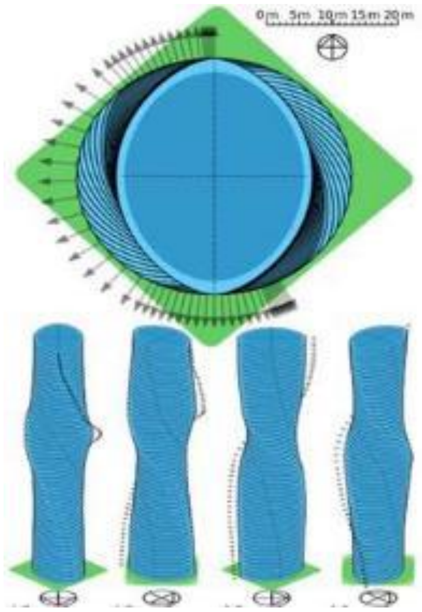
a) b) c)

Figure 1.8 –Spiral three-dimensional composition “Infinity Tower”:
a, b – twisting of the floors of the building - the torso of a person unfolding;
c – general view of the building

Some experts call it the “Dancing Building” because each floor is offset from the previous one at an angle, with the first floor at a ninety-degree angle to the eightieth. Many experts compare this skyscraper to a true work of art, a pearl of Dubai architecture, and consider it unique (Fig. 1.8).

High-rise complex “Absolute World Towers” in Canada, Mississauga. Two twin residential towers have an elegant “twisted” volumetric and spatial composition. A feature of the towers is that they do not have two identical floors and balconies. Such a form formation created not only unique buildings, but also provided an organic combination with the existing urban landscape (Fig. 1.9).

a)

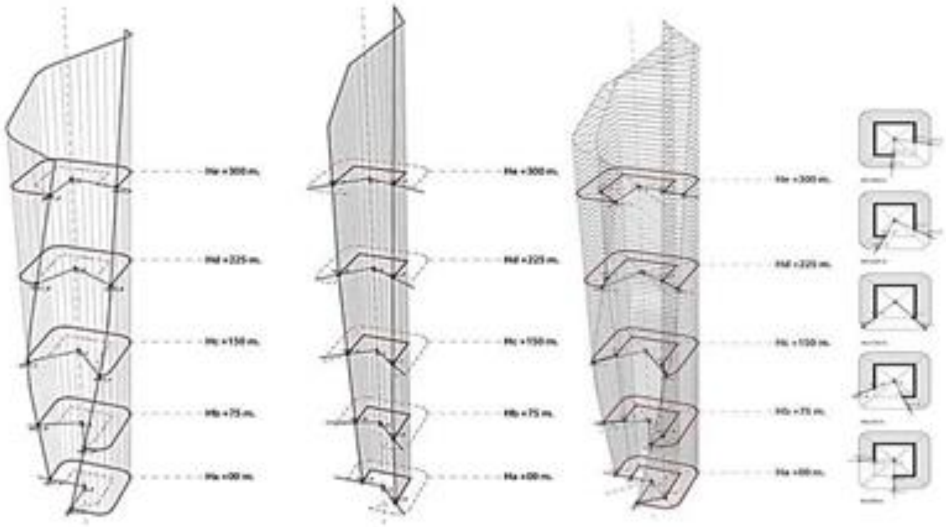


b)

Figure 1.9 – Twisted volumetric and spatial composition of the high-rise complex “Absolute World Towers”: a – twisting of the building's floors; b – general view of the building

The most complex example of form formation is the skyscraper “Al Hamra Tower” in Kuwait City. The tower Al-Hamra is unusual in its asymmetrical shape, resembling a thrown cloak. This form of the building contains not only aesthetic, but also practical goals, creating certain thermal insulation functions to protect against overheating. The skyscraper has an impressive futuristic appearance (Fig. 1.10).

a)



b)

Figure 1.10 – “Al Hamra Tower”:

a – formation of a three-dimensional composition; b – general view of the building from different angles

Form articulation - expressiveness of the composition is achieved through the use of frontal accent elements that protrude (recess) from the plane of the main volume. In this case, the frontal elements can be different in geometry, texture, color. When identifying the form of a high-rise building, functional elements are used - intermediate floors (winter garden, terraces, observation deck, open areas, etc.), technical floors, stairs, etc.

An example of the division of form into volumes is the skyscraper “New York by Gehry” (formerly known as Beekman-Towser), built in the deconstructivism style in New York City. Thanks to the undulating facade of the building, the design of which has the effect of drapery fabric. The facade is made of more than 10,500 steel panels. Some of them are flat, some are convex. Many of the windows are not arranged in a row, their size varies, and due to the undulating facade, each window is configured differently from each other. The south side of the building is completely flat, although made of the same structural panels, but it serves to strengthen the structural composition of the building (Fig. 1.11).

a) b)

Figure 1.11 – Volumetric and spatial composition of the skyscraper “Al Hamra Tower”: a – frontal accent elements of the facade; b – flat, convex and recessed steel panels of the facade

Comparison of volumetric elements of the composition - the use of volumes that are different in geometry, texture, color. The expressiveness of the composition is achieved through the proportional comparison of volumetric and planning elements of different geometry and texture of surfaces, which creates their different scale.

An example of a proportional juxtaposition of spatial and planning elements is the “Burj Khalifa” tower in the UAE, Dubai (Fig. 1.12).

a) b) c)

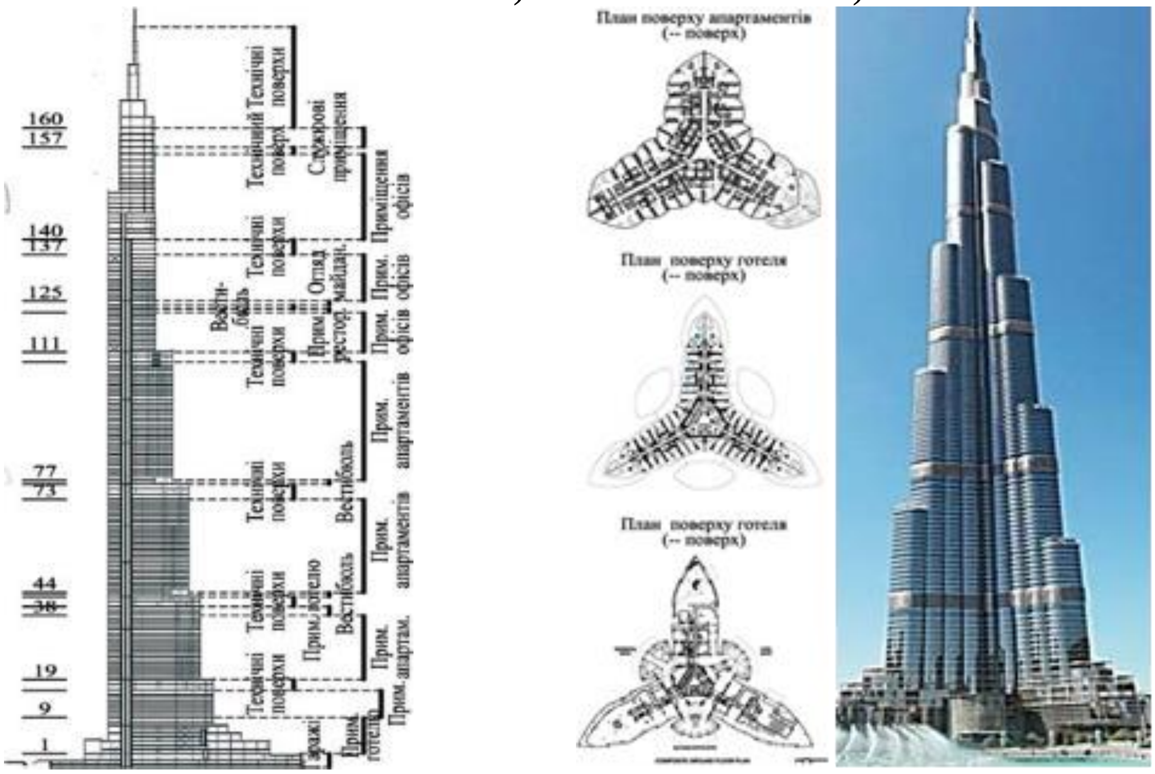


Figure 1.12 – Volumetric and spatial composition of the “Burj Khalifa” skyscraper: a – proportional comparison of spatial planning elements by height; b – layout by number of floors; c – general view of the building

The combination of form-forming techniques in organizing the architectural and artistic expressiveness of high-rise objects allows you to obtain a wide variety of options for harmonious, holistic and dynamic high-rise structures. Thus, it can be noted that the form-forming and architectural and artistic solution of a high-rise building cannot depend only on its purpose. The coherence of architecture in this matter is a mandatory condition for high-rise housing construction. The construction of a volumetric and spatial composition, the artistic solution of facade details should reveal the purpose of the building, creating the appropriate emotional mood of a person. All this consciously determines the uniqueness of the building and implies its status in the urban planning system, the comfort of conditions and the corresponding consumer qualities.

1.3 Functional and planning solutions for high-rise buildings

The functional planning solution of any building depends on the number of functionally forming elements. Depending on the number of functionally forming elements, high-rise buildings are divided into monofunctional, specialized, and multifunctional.

Monofunctional – high-rise buildings that contain one functional-forming element. Such buildings are designed for residential or public purposes (hotels, offices, educational institutions, etc.).

a) b) c)

Figure 1.13 – "Turning Torso" residential complex, Sweden, Malmo:
a – typical plan of a residential floor as a single functional-forming element; b – clockwise rotation of each floor and segments; c – general view of the building

An example of a monofunctional high-rise building is the 54-story residential complex "Turning Torso". The building is based on one functional-forming element - a typical residential floor plan. Each floor has a pentagonal shape with a vertical core supported by an external steel frame. The building structure consists of 9 segments of 5 floors each. Each of the segments is turned clockwise relative to the first segment, and the last segment is turned 90°. The lower two segments are located in offices. From the third to the ninth segment there are 147 residential apartments (Fig. 1.13).

Specialized - high-rise buildings containing two or more functional-forming elements, one of which is the main one and occupies more than 85% of the useful area of the object. Most often, this type of functional interaction takes the form of subordinate secondary functions as part of a monofunctional high-rise building. Thus, the high-rise building of the financial center “Guangzhou CTF Finance Centre” has three main and two auxiliary functional-forming elements. In a skyscraper with 111 floors, 80 floors are occupied by office premises, the other floors are occupied by 414 apartments, 27 hotel rooms, parking for 1705 cars is located on the first and underground floors (Fig. 1.14)

a) b) c)

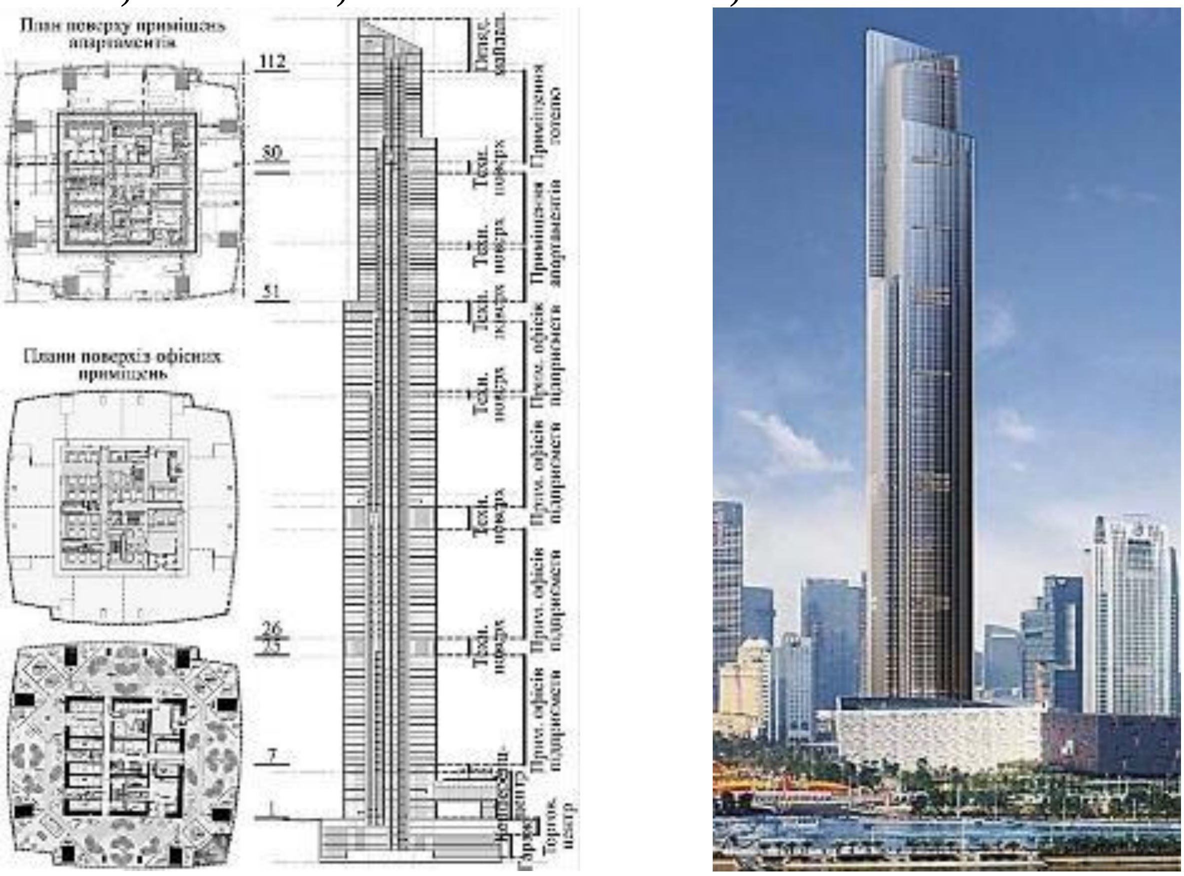


Figure 1.14 – Specialized skyscraper “CTF Finance Centre”, China, Guangzhou: a – functional floor plans as functional-forming elements of the object; b – vertical planning of functional-forming elements; c – general view of the building

Polyfunctional – high-rise buildings, containing many main and secondary functional-forming elements, each of which occupies at least 15% of the useful area of the object. The high-rise building “The Northeast Asia Trade Tower” has four main and three secondary functional-forming elements (Fig. 1.15).

a) b) c)

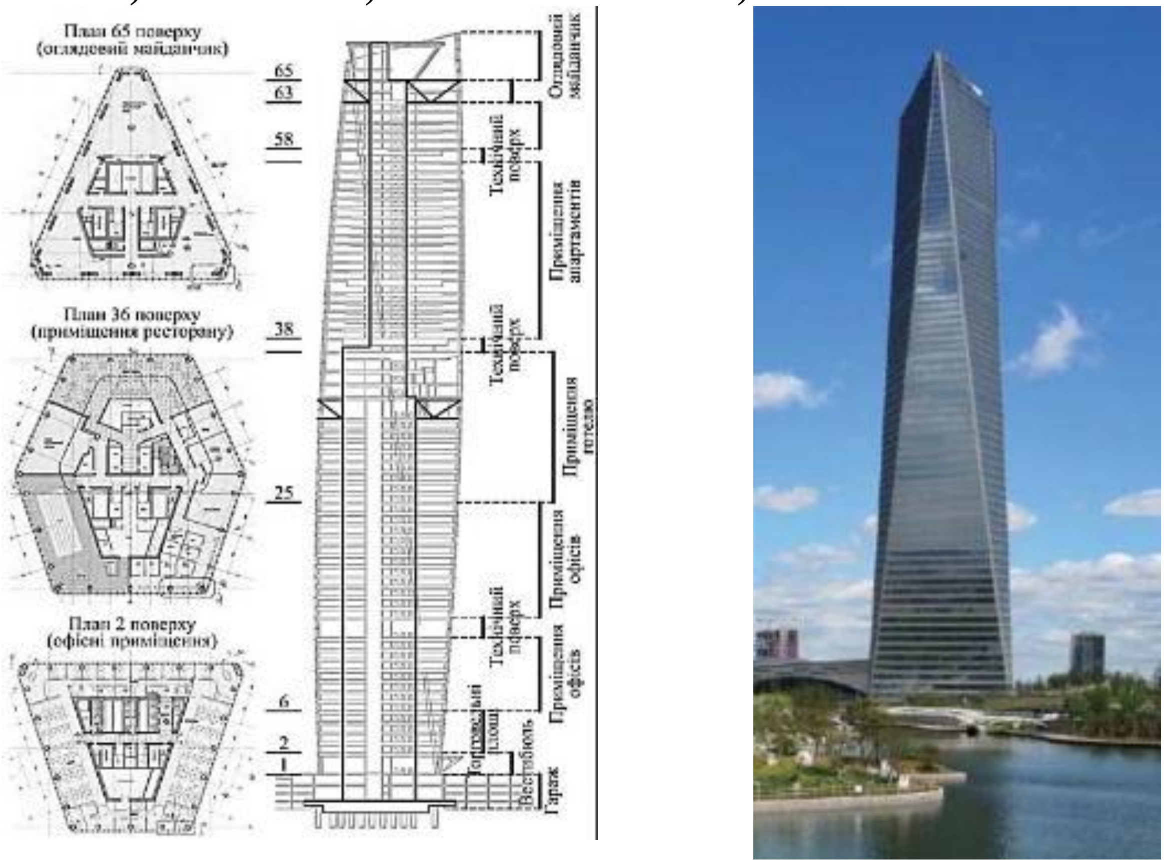


Figure 1.15 – Multifunctional skyscraper “The Northeast Asia Trade Tower”, Incheon district, South Korea:

a – floor plans as functional-forming elements of the object; b – vertical planning of functional-forming elements; c – general view of the building

Due to the costs of ensuring structural rigidity and optimal functioning of engineering systems, the most rational in functional and planning solutions are buildings with a height of 70 floors. As the height of buildings increases, the number of their functional components increases. According to the Council on Tall Buildings and Urban Habitat, the sector of optimal operating conditions for multifunctional high-rise buildings is limited by the parameter of their appropriate height – 250 m, and is determined by the following ratio: 47 % – monofunctional, 25 % – office, 20 % – residential, 8 % – hotels.

1.4 Planning solutions for high-rise buildings

Plan forms as a form-forming element of a high-rise building have different configurations. There are many variations of combining different forms. As a rule, the configuration of a planning solution consists in arranging various elementary forms around a single center (Fig. 1.16). The single center can be any simple figure: a square or rectangle (Fig. 1.16, a, b, c, d), a circle (Fig. 1.16, d, f, h), a triangle (Fig. 1.16, f, g.) and so on.

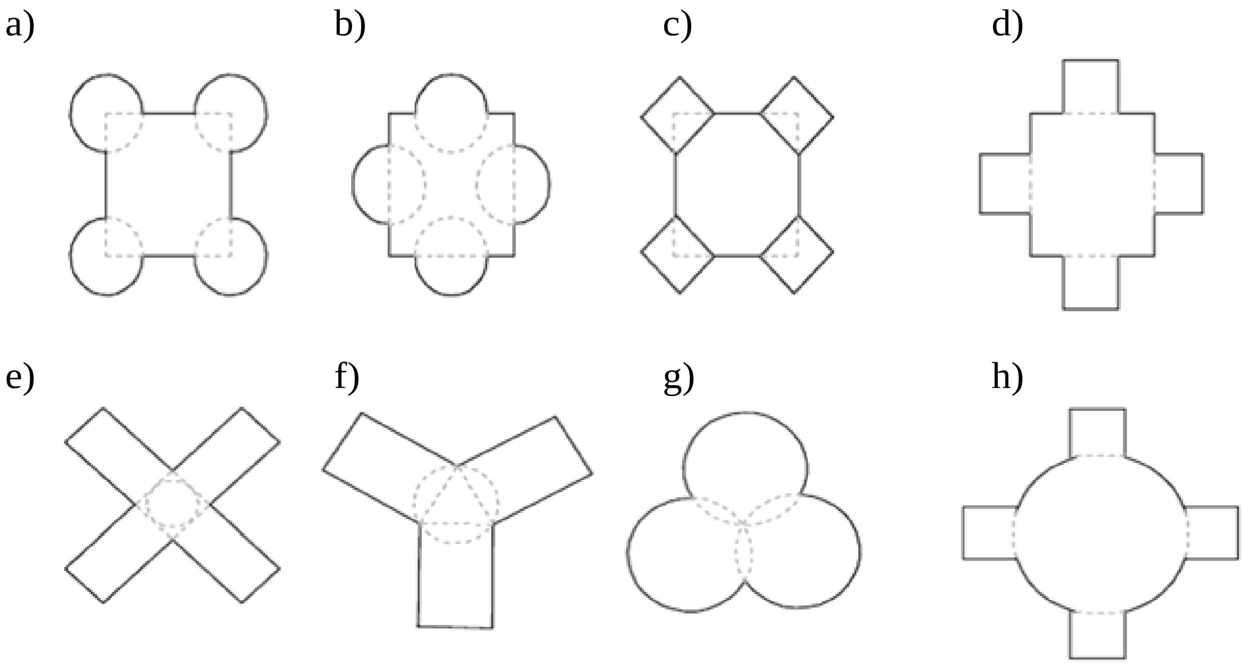


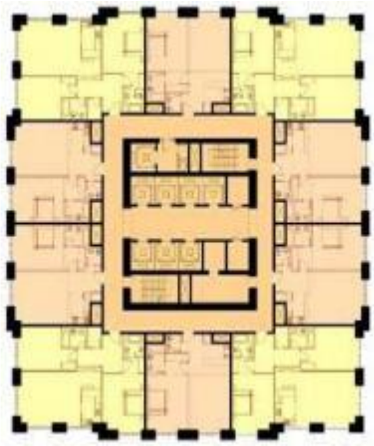
Figure 1.16 – Examples of planning solution configurations

Of all the variety of configuration forms, they are divided into compact plans and plans of complex configuration.

Compact (also called traditional) plans include configuration options that are simple geometric shapes such as a square, rectangle, circle, or ellipse. Such shapes are familiar to others, so they are not unusual. But when comparing several such shapes or transforming them, it is possible to obtain new planning solutions in contrast to simple shapes; in addition, such changes can carry additional functions or increase the functional load.

Plans of complex configuration include cruciform, paired-block, radial planning solutions, or generally plans of arbitrary shape. Planning elements that are located around a single center and, like rays, emanate from it, are called "radial" planning schemes.

a)



b)

Figure 1.17 – Residential skyscraper “Marina 101”: a – rectangular compact functional and planning solution; b – volumetric and spatial composition of the skyscraper

Compact planning solutions. Such solutions are usually sectional. The configuration of the sections is square or rectangular and involves the central placement of elevators and stairwells. In design practice, rectangular planning schemes are most commonly used. In them, the aspect ratio is within 1:1.5-1:2.5. Zoning of the internal space of the plan must meet functional requirements and provide sanitary and hygienic comfort, following the example of a skyscraper project.

“Marina 101”. This is a 101-story residential complex in Dubai, UAE. The project was developed by the architectural firm “National Engineering Bureau” and the Turkish company TAV Construction. The building includes 24 hotel rooms and 516 apartments for permanent residence (Fig. 1.17).

The total area of the building is 153.3 thousand sq. m. The area of apartments on a typical floor varies from 800 to 1700 m^2, and the total area of apartments in the building is 60.0 thousand m^2.

A striking example of the circular configuration of the planning solution of high-rise buildings is the Agbar Tower (cat. Torre Agbar), or Aiguas da Barcelona Tower (cat. Torre Aigés de Barce/ona) in Barcelona, Spain. The height of the 33-story building is 144.4 m. The building has a circular shape that tapers towards the mountain (Fig. 1.18).

a) b) c)

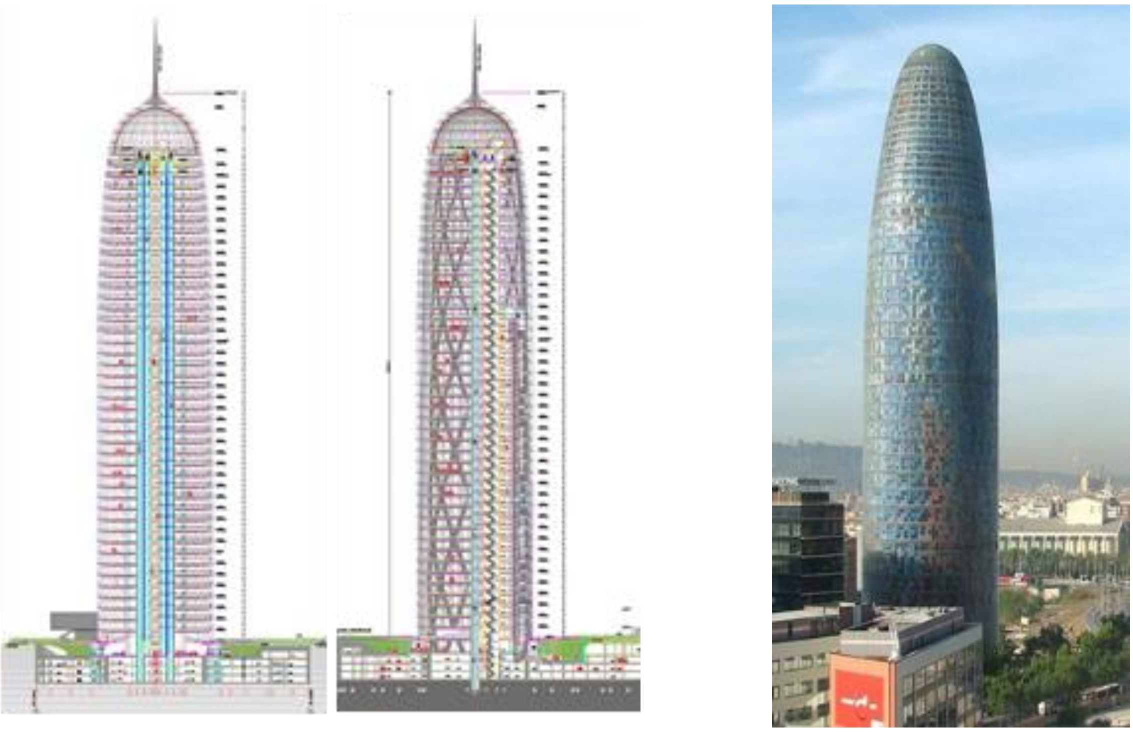


Figure 1.18 – High-rise building “Torre Aigues de Barcelona”:
a – compact functional and planning solution; b – volumetric and spatial composition of the skyscraper; c – general view of the building

The skyscraper was designed by French architect Jean Nouvel in the High-Tech style, and although the design combines a number of different architectural concepts, according to Jean Nouvel, he was inspired by the Montserrat mountain near Barcelona. Due to its unusual shape, the building has several nicknames: “el supositori” (the candle), “l'obús” (the projectile). The skyscraper is built of reinforced concrete, the facade is covered with glass.

The building's defining feature is its multi-colored metal panels, which house around 4,500 LED lighting fixtures. They create complex color combinations (up to

16 million colors), creating a kind of "pixelated" color effect - from a distance, the pixels merge, and the tower appears to shimmer with all the colors of the rainbow.

a)

b)

Figure 1.19 – High-rise buildings with an oval solution in their floor plans: a – skyscraper “Guangzhou International Finance Center”; b – high-rise building “Parus”

The “Guangzhou International Finance Centre”, a high-rise building in Guangzhou, China, is designed as an oval. The height of the 103-story skyscraper is 437.5 m. The building is designed in such a way that from the base, the tower expands slightly to the middle, then narrows again to the roof, which gives the structure a special elegance. The core of the tower is made of reinforced concrete, the shape is given by a tubular frame (Fig. 1.19, a).

An example of an oval planning scheme is one of the tallest buildings in Kyiv at 2 Mechnikova Street - “The Sail”. The total height of the office center is 6 floors, which is 156 m. The project of the high-rise building was developed by the Komarovsky “Architectural Bureau” (Fig. 1.19, b). A feature of the building is that in plan the building resembles an oval lentil grain.

As practice shows, in conditions of compact planning of high-rise buildings, the communication core occupies 25-30% of the floor plan area. Increasing the number of floors requires an increase in the number of elevators and, accordingly, an increase in the area of the communication core, which increases the cost of communications. But the compactness of communications, due to their location in the core, reduces their length, which allows for the rational use of computerized and automated engineering systems.

The compactness of the plans of high-rise buildings allows for the rational use of small-sized plots located in existing buildings for development, creating urban planning accents in cities.

Complex configuration of planning solutions. Such planning solutions include: T-shaped, cross-shaped, radial, pair-block and arbitrary planning forms.

A classic example of a three-beam configuration of a planning solution is the high-rise residential skyscraper Lake Point Tower in Chicago, USA. The height of the 70-story skyscraper is 197 m. The project was developed by the Shipporight-Heinrich Association, initially with architect Gram Anderson. The central planning element of the building is a triangle, from which three rectangular beams emanate (Fig. 1.20).

a) b) c)

Figure 1.20 – High-rise residential building "Lake Point Tower":
a – three-beam configuration of the planning solution; b – volumetric-spatial composition of the skyscraper; c – general view of the building

In any beam planning schemes, the length of the beams is unlimited and can reach 60 m.

An example of an arbitrary planning form is the high-rise multifunctional complex Marina Bay Sands in Singapore. The complex includes three 55-story towers with a height of 194 meters.

On the roofs of the three towers there is a large terrace in the form of a gondola, which houses the Sands SkyPark pool and garden with an area of 12.4 thousand sq. m. meters. The open platform for visitors has an area of 1.2 hectares. The pool is in the style of "Infinity Pool", imitates the absence of sides and has a length of 146 m. The pool consists of three interconnected bowls and contains 1424 m^3 of water. The pool is unique in the world in terms of its height and size (Fig. 1.21). The project of the complex was developed by the American architect, professor of Harvard University Moshe Safdie, who, according to him, borrowed the architectural idea from a deck of cards. To absorb the vibrations of the towers under the influence of wind, three special seams with gaps of up to 500 mm are installed between the pools. Separate supports under each of the three basins of the pool are to balance the individual draft of the three towers and ensure that the pool remains horizontal.

Figure 1.21 – General view of the "Marina Bay Sands" mixed-use complex

When designing the configuration of planning solutions, it is necessary to remember that high-rise buildings shape the aerodynamic situation in the development area and their shape is one of the determining factors in the formation of air flows, natural lighting, insolation and aeration of the surrounding environment.

In addition, high-rise buildings have a high degree of potential fire hazard, so they belong to a special degree of fire resistance and are divided into class "A" buildings with a nominal height of 73.5 to 100 m, and class "B" with a nominal height of 100 to 150 m. The internal structure of skyscrapers is fundamentally different from buildings of ordinary storeys. The main emphasis is on fire safety. After all, evacuating people from a high-rise building in the event of an emergency is extremely problematic. Therefore, the internal space of skyscrapers is divided by special fire barriers. At the same time, one backup elevator in the building always remains connected to the uninterrupted power supply. The newest skyscrapers are planned in such a way that in emergency situations people can hide on technical floors, which are usually not used under normal operating conditions. At the same time, all entrances to the premises are most often equipped with double doors to prevent drafts that provide oxygen to the flame during outbreaks.

Thus, functional planning solutions must prevent the development of a fire and ensure the evacuation of people in the event of its occurrence.

In addition, it should be noted that a person's stay at a considerable height violates the usual scales for a person. There will be a feeling of detachment from the real environment, which can negatively affect the emotional state and psyche of a person when staying at a height. Living at a height primarily affects the health of children and the elderly. These categories of residents most often suffer from the so-called "bird" disease, when the psycho-emotional state undergoes a process of pressure drop. After all, at altitude, atmospheric pressure decreases and arterial pressure increases.

Thus, when designing high-rise buildings, it is necessary to take into account the features of the functional solution of the entire building: the layout of apartments and premises; ensuring unhindered access for low-mobility groups of the population;

access for firefighters to any apartment or premises; placement of elevators; waste disposal and other engineering and technical systems, communications and devices. All this is closely related to constructive solutions, requirements for the decoration of the building and ensuring the possibility of repairing and cleaning facades during operation, etc.

The concepts of any planning solutions for high-rise buildings must be considered in accordance with all urban planning norms and requirements, taking into account the surrounding development in order to rationally use the area of the land plot. The shape, orientation and size of the buildings, as well as obtaining insolation requirements and regulatory distances to existing development, will ensure maximum use of natural lighting and protection from excessive sunlight.

Analysis of domestic and foreign experience in the design and construction of high-rise buildings allows us to identify the main trends in the development of architectural and planning solutions and functional organization:

- increasing the number of floors (height) of objects;
- an increase in the number of their functional components;
- integration of "ecological spaces" in the structure of buildings;
- the emergence and development of the "Vertical City" concepts.

Topic 2 CALCULATIONS OF A HIGH-RISE BUILDING AS A CONSTRUCTIVE SYSTEM “SOIL BASE-FOUNDATION-STRUCTURE”

Calculation and design of buildings under seismic influences should be carried out in accordance with DBN V.1.1-12. Fire resistance of structures and fire safety of buildings must comply with the requirements of DBN V.1.1-7.

When designing reinforced concrete structures, their reliability must be established by calculation according to the limit states of the first and second groups by using the calculated values of loads and material characteristics, which are determined using the corresponding reliability coefficients, taking into account the degree of responsibility of buildings.

The values of characteristic loads, load combination coefficients and structural reliability coefficients, as well as the distribution of loads into permanent and temporary (long-term and short-term) should be taken in accordance with DBN V.1.2-2.

The procedure for applying constant and long-term loads should be determined by the work schedule or on an ad hoc basis.

The design scheme of the house includes data on the load and the physical model. The physical model of the house is a three-dimensional system of columns, walls, slabs, beams and their connections, as well as data on the physical and mechanical properties of materials and loads. The distribution of forces in spatial systems is largely determined by the stiffness characteristics of the elements and their connections, which depend both on the material and its stress state, and on the quality of manufacturing and installation, the presence of defects, the history of the load, the type of structure, the humidity of the material, the degree of damage (wear), temperature and other factors.

Calculations of the stress-strain state of reinforced concrete rod, plate and volumetric elements and their connections are developed in existing regulatory documents only for normal sections under simple influences.

For arbitrary cross-sections of rod and plate elements under complex influences (cases practically characteristic for all elements of the spatial scheme of a high-rise building) it is recommended to use tested computer programs that implement algorithms based on the fundamental provisions of the deformation theory of reinforced concrete (the law of plane sections, nonlinear dependence between stresses and deformations, restrictions on the values of linear deformations, etc.) and the general requirements of the relevant standards.

Complex spatial geometric schemes are simplified by replacing the real structure with a conventional scheme. Columns and beams are approximated by rods brought to the axis, and slabs and walls are approximated by plates brought to the median plane.

The use of continuous, discrete-continuous and discrete computational models is allowed. It is recommended to use computational models based on mathematical and geometric discretization of spatial structures using the finite element method (FEM).

If necessary, computer modeling of individual nodes or fragments of structures is allowed based on the use of physically nonlinear three-dimensional finite elements, special elements that model friction, slippage, pre-tension, etc. In especially critical cases, it is recommended to confirm the results of computer modeling with full-scale experiments.

It is recommended to use software complexes that implement procedures for calculating structures taking into account physical and geometric nonlinearity, as well as processes related to the life cycle of the structure. In necessary cases, it is recommended to use these software complexes to perform calculations taking into account the physical nonlinearity of reinforced concrete and soil (pile) foundations and to comply with DBN V.2.2-24:2009. Computer modeling of the processes of erecting a high-rise building frame, taking into account changes in the physical and mechanical properties of concrete during the construction process, as well as processes related to dynamic influences (seismic, wind), and processes related to force majeure situations (progressive collapse).

The calculation of load-bearing structural systems includes:

– determination of forces in the elements of the structural system (columns, floor and roof slabs, foundation slabs, walls, soil or pile foundations);

– determination of displacements of the structural system as a whole and its individual elements, as well as accelerations of oscillations of the upper floors;

– calculation of the stability of the structural system (stability of shape and position);

– assessment of the resistance of the structural system to progressive collapse;

– assessment of the bearing capacity and deformation of the foundation.

The calculation of the load-bearing structural system, including above-ground and underground structures and the foundation, should be carried out for all successive stages of construction (in the event of a significant change in the design situation) and for the operation stage, adopting calculation schemes corresponding to the stages under consideration. In this case, the following should be taken into account:

– the procedure for applying and changing the vertical load and stiffness of elements during installation and operation;

– the formation of cracks from temperature-shrinkage deformations of concrete during the hardening process and the presence of technological joints when concreting with grippers;

– the value of the strength and stiffness of concrete at the moment the structure is released from the formwork and the load is transferred from the floors above.

The calculation of the load-bearing structural system should generally be carried out in a spatial setting, taking into account the joint operation of above-ground and underground structures, the foundation and the substructure below it.

The calculation of load-bearing structural systems should be carried out using linear and nonlinear stiffnesses of reinforced concrete elements.

The linear stiffnesses of reinforced concrete elements are determined as for a solid elastic body.

Nonlinear stiffnesses of reinforced concrete elements are determined by the cross section, taking into account the possible formation of cracks, as well as the development of inelastic deformations in concrete and reinforcement, corresponding to short-term and long-term loading actions.

The value of nonlinear stiffnesses of reinforced concrete elements should be determined depending on the stage of calculation, the requirements for the calculation, and the nature of the stress-strain state of the element.

At the first stage of calculating the structural system, when the reinforcement of reinforced concrete elements is unknown, it is recommended to take into account the nonlinear operation of the elements by reducing their stiffness using conditional generalized coefficients.

At the subsequent stages of the calculation of the structural system, when the reinforcement of reinforced concrete elements is known, the calculation should include the specified values of the stiffness of the elements, which are determined taking into account the reinforcement, the formation of cracks and the development of inelastic deformations in concrete and reinforcement in accordance with the instructions of the current regulatory documents on the design of reinforced concrete structures.

As a result of the calculation of the load-bearing structural system, the following must be established: in columns – the values of longitudinal and transverse forces, bending moments, and, if necessary, torques; in flat slabs of floors, coatings and foundations – the values of bending and torsion moments, transverse and longitudinal forces.

The forces in the elements of the structural system should be determined from the action of longitudinal design permanent, long-term, short-term and episodic loads, as well as their main and emergency combinations.

At the first stage of the calculation, to estimate the forces in the elements of the structural system, it is allowed to take approximate values of the stiffness of the elements, bearing in mind that the distribution of forces in the elements of structural systems depends not on the magnitude, but mainly on the ratio of the stiffnesses of these elements.

For a more accurate assessment of the distribution of forces in the elements of the structural system, it is recommended to take refined stiffness values with reduction factors. In this case, it is necessary to take into account a significant reduction in stiffness in plate elements that bend (as a result of possible cracking) in comparison with eccentrically compressed elements. In the first approximation, it is recommended to take the modulus of elasticity of the material, which is equal to E_v with reduction factors: 0.6 - for vertical compressed elements; 0.3 - for floor slabs (covers) and beams, taking into account the duration of the load.

As a result of the calculation of the load-bearing structural system, the values of vertical displacements (deflections) of floors and coverings, horizontal displacements of the structural system, and for buildings with a high number of floors - the acceleration of vibrations of the floors of the upper floors must be established. The values of the specified displacements and acceleration of vibrations must not exceed the permissible values established by the relevant regulatory documents.

Horizontal displacements of the structural system should be determined from the action of operational design (for the limit states of the second group) values of constant, long-term, short-term horizontal and vertical loads. Vertical displacements (deflections) of floors and coatings are determined from the action of operational values of constant and long-term vertical loads. At the same time, at the first stage of calculation, it is recommended to take reduced values of stiffness of the elements of the structural system, in particular floor slabs, since vertical displacements (deflections) directly depend on the deformation properties of the slabs. In the first approximation, the values of the reduction coefficients relative to the initial modulus of elasticity of concrete, taking into account the duration of the load, are recommended to be taken: for vertical load-bearing elements - 0.6, and for floor slabs (coatings) - 0.3.

At the following stages of the calculation, with known reinforcement, it is necessary to accept the specified stiffnesses of the slabs, taking into account the reinforcement, the presence of cracks and inelastic deformations in the concrete and reinforcement, which are determined by current regulatory documents.

The acceleration of vibrations of the ceilings of the upper floors of the building should be determined under the action of the pulsating component of the wind load.

When calculating the stability of a structural system, the stability of the shape of the structural system, as well as the stability of the position of the structural system against overturning and shearing, should be checked.

The calculation of the stability of the structural system should be carried out for the action of longitudinal design constant, long-term, short-term and episodic vertical and horizontal loads.

When calculating the stability of the shape of a structural system, it is recommended to take reduced stiffnesses of the elements of the structural system (taking into account the nonlinear behavior of the material), since the stability of the structural system is related to the deformability of the system and individual elements. In this case, the values of the reduction coefficients in the first approximation are recommended to be taken taking into account the fact that the stability of the structural system depends on the resistance of mainly eccentrically compressed vertical elements under prolonged load action and in the stage approaching the limit. The margin of stability must be at least twice

When calculating the overall stability of structures, it is allowed to check for the load combination to which the largest values of longitudinal forces correspond and the found coefficients of the design length, and extend them to other load combinations.

When calculating the stability of the structural system, it should be considered as a rigid undeformed body. When calculating for overturning, the holding moment from the vertical load must exceed the overturning moment from the horizontal load by a factor of at least 1.5.

When calculating for shear, the horizontal holding force should exceed the effective shear force by a factor of 1.2. In this case, the most unfavorable values of the reliability coefficients for the load should be taken into account. The bearing capacity and deformations of the foundation should be estimated in accordance with the relevant regulatory documents based on the forces acting on the foundation found when calculating the structural system of the house. The calculation of distortions from uneven vertical deformations of adjacent load-bearing structures (walls and columns) should be carried out taking into account the actual order of construction of the house, as well as the time and duration of the application of loads to account for nonlinear deformations in reinforced concrete structures

Calculation methods. A spatial structural system is a statically indeterminate system. For the calculation of load-bearing structural systems, it is recommended to use discrete calculation models calculated using the finite element method.

Discretization of structural systems is carried out using shell, rod and volume (if necessary) finite elements, which are used in the adopted calculation program.

When creating a spatial model of a structural system, it is necessary to take into account the nature of the joint work of rod, shell, and volumetric finite elements, associated with a different number of degrees of freedom for each of the specified elements.

Deformational properties of the foundation should be taken into account by using the generally accepted Winkler or Pasternak foundation calculation models.

The bed coefficients should be determined according to the settlement of the foundation, which can be calculated according to the schemes of a linearly deformed half-space or a linearly deformed layer. When calculating dynamic effects, it is allowed to introduce a coefficient of increasing the stiffness of the foundation. When using pile or pile-slab foundations, piles should be modeled as reinforced concrete structures or take into account their joint operation with the foundation in a generalized manner, as a single foundation using the reduced bed coefficient of the foundation.

It is recommended to consider the pile foundation model in a physically nonlinear formulation to take into account the equalization of initially uneven forces in the piles, due to the greater stiffness of the piles at the periphery of the pile field.

Piles are modeled by vertical elements, the stiffness of which is determined by regulatory documents, or as a result of full-scale tests of trial pile bushes. Piles can also be modeled by a rod element (reinforced concrete column) in the soil massif.

The perception of horizontal forces by the soil or pile foundation is allowed to be modeled by introducing horizontal ties of finite stiffness into nodes that lie at the level of contact of the base and the foundation slab.

If it is necessary to take into account various factors of propagation of forced vibrations in the soil (the influence of existing inclusions, modeling of the operation of piles in the soil, etc.), it is recommended to model the soil base massif with flat or three-dimensional finite elements of the soil, which take into account the physical nonlinearity of its operation.

When constructing a finite element calculation model, the dimensions and configuration of finite elements should be set based on the capabilities of the specific calculation programs used, and taken such that the necessary accuracy of determining the forces along the length of columns and the area of floor slabs, foundations and walls is ensured, taking into account the total number of finite elements in the calculation scheme, which affects the duration of the calculation.

It is recommended to apply the principles of fragmentation when calculating the general scheme of the house and individual elements. When calculating the general scheme of the house for horizontal and vertical loads, a model with a fairly sparse finite element mesh should be used. The dimensions of the finite elements in this case may not exceed ½ ÷ ¼ of the floor height. Calculations of individual elements of the house (floor slabs, walls, foundation slabs) in this case are carried out as calculations of individual structural schemes with a denser finite element mesh for local loading and movement of nodes common to the nodes of the general scheme.

When organizing such a calculation, it is recommended to use software complexes (PC "MONOMAKH"), in which the principles of fragmentation are automatically implemented.

After determining the reinforcement in floor and roof slabs, an additional calculation of the structural system should be made to specify the deflections of these structures, taking into account the specified values of the bending stiffnesses of the finite elements of the slabs, taking into account the reinforcement in two directions in accordance with current regulatory documents.

A similar additional calculation should be performed for a more accurate assessment of bending moments in floor elements, coatings and foundation slabs, as well as longitudinal forces in walls and columns, taking into account the nonlinear behavior of reinforcement and concrete up to the limit values.

It is recommended to calculate the floor slabs for the vertical load and movement of vertical load-bearing elements obtained from the general calculation of the house, taking into account the sequence of construction.

The finite element mesh should be thickened in places where the slab rests on columns, pylons and walls. In places of thickening, it is desirable that the dimensions of the finite elements of the slab do not exceed the smallest size of the vertical load-bearing element. For a more realistic representation of the operation of the joints of the slab rests on columns, it is recommended to use methods that simulate the kinematic hypothesis of a plane section in the column dimension (using a body of infinite rigidity), or a static hypothesis, which is based on the use of a linear law of distribution of forces transmitted from the column to the slab.

If necessary, it is recommended to perform calculations of individual nodes and elements based on a three-dimensional model in a physically nonlinear formulation. For example, modeling the node of support of a slab on a column in the case when the grade of concrete of the column exceeds the grade of concrete of the slab by two or more points, or modeling the operation of a barrette in a soil massif, etc.

The calculation of load-bearing reinforced concrete elements of the structural system (columns, walls, floor slabs, coatings and foundations) should be carried out according to the limit states of two groups: by bearing capacity (by strength and stability) and by serviceability (by crack resistance and deformations). In this case, the calculation of the stability of individual compressed elements (columns and walls) is recommended to be carried out within the framework of the calculation of the strength of these elements taking into account the influence of longitudinal bending or within the framework of the calculation of a statically indeterminate structural system.

It is recommended to calculate the strength of a reinforced concrete element cross-section based on a nonlinear deformation model using the limit force method, i.e. under conditions where the forces from the design impacts do not exceed the limit forces that the cross-section being calculated can withstand.

The distribution of relative deformations of concrete and reinforcement over the height of the element's cross-section is assumed to be linear (the hypothesis of plane sections).

As a generalized characteristic of the mechanical properties of materials (concrete and non-stressed reinforcement) in a uniaxial stressed state, state (deformation) diagrams of materials should be taken.

The diagrams establish the relationship between stresses and longitudinal relative deformations under short-term action of a single applied load up to the established limit values of material collapse under uniaxial stress.

Diagrams, limit values of relative deformations and other design characteristics of materials may be determined in accordance with current regulatory documents.

It is recommended to make the transition from the stress diagram in concrete and reinforcement to the generalized internal forces by numerical integration over the normal section. The resistance of the concrete in the tension zone is neglected.

The position of the neutral axis and maximum deformations are determined from the condition of equilibrium of external and internal forces.

When a torque acts on a rod element, the strength calculation is performed for a spatial section formed by a spiral crack located at an angle to the element axis. The calculation of a column operating under torsion should be performed based on the spatial truss model from the condition of equilibrium of the moments of all external and internal forces in a plane normal to the line limiting the compressed zone of the spatial section relative to the axis perpendicular to this plane and passing through the point of application of the resultant forces in the compressed zone. A closed box section with a nominal wall thickness not exceeding the actual thickness is taken as the design one.

The open cross-section should be divided into separate parts, each of which is considered as a rectangular section.

Torsional strength and stiffness should be determined by summing, respectively, the strengths or stiffnesses of the individual rectangular parts.

It is recommended to take into account the combined action of bending and torsional moments, as well as torque and shear forces for simple sections, in accordance with current regulatory documents.

The calculation of the strength and crack resistance of flat slabs of floors, coatings, foundation slabs and walls should be carried out for the combined action of bending and torsional moments, longitudinal and shear forces applied on the sides of the flat selected element in the direction of mutually perpendicular axes, as well as for the action of transverse forces.

When using volumetric finite elements in calculations (for example, in thick foundation slabs), tensile forces must be perceived by longitudinal, transverse or fiber reinforcement, and compressive forces by concrete. Transverse reinforcement of slabs in places of support of vertical elements (releases) is determined by the maximum forces in the lower sections of columns, pylons and walls.

Load-bearing reinforced concrete structures. The main load-bearing elements of the structural system are columns, walls, floor and covering slabs, various foundations, including pile grillages, etc.

The main design parameters of columns are their height, cross-sectional dimensions, concrete compressive strength class, and the content of longitudinal and transverse reinforcement, determined by the spatial calculation of the frame.

When designing, it is recommended to take the optimal structural parameters of columns, which are established on the basis of a feasibility study. In this case, it is recommended to take the minimum cross-sectional size of columns of at least 30 cm, for columns with an elongated cross-section - at least 20 cm, the concrete class, as a rule, is not less than B25 and not more than B60, the percentage of reinforcement in any cross-section (including areas with a lap joint of reinforcement) - not more than 10.

In cases where the feasibility study of the structural parameters of columns shows that the required reinforcement exceeds the maximum values, it is recommended to use steel-reinforced concrete, including tubular concrete, as well as steel-fiber concrete columns.

In cases where the technical and economic analysis of the structural parameters of columns shows that the required concrete class exceeds B60, it is recommended to use steel-reinforced concrete solutions for columns. The use of high-strength concrete B70–B90 and higher is possible on the basis of special experimental studies.

The main design parameters of the walls are the dimensions (wall thickness), the concrete class for compressive strength, and the content of vertical reinforcement (percentage of reinforcement), determined depending on the height of the building, the load on the floors, and the pitch of the walls.

When designing, it is recommended to take the optimal structural parameters of the walls, which are established on the basis of a feasibility study. In this case, it is recommended to take the cross-sectional dimensions (thickness) of the walls at least 20 cm (upper floors at least 18 cm), the concrete class - at least B 25, the percentage of reinforcement in any section of the wall (including areas with a lap joint of reinforcement) - no more than 10.

For spans up to 6-8 m, it is recommended to make the floors flat, for larger spans - flat with capitals or inter-column beams and walls, and for spans up to 12 m - with inter-column beams or walls and ribbed and hollow core slabs.

For hall spaces with a span of 12-15 m, caisson, ribbed or hollow slabs are recommended when supported on four sides by beams and walls.

The main design parameters of flat floor slabs are the cross-sectional dimensions (slab thickness), concrete compressive strength class and longitudinal reinforcement content, which are determined depending on the load on the floor and the length of the spans.

When designing, it is recommended to take the optimal structural parameters of the floors, established on the basis of a technical and economic analysis. In this case, the thickness of flat slabs of solid cross-section is recommended to be taken not less than 1/30 of the length of the largest span and not more than 25 cm, concrete class - not less than B20. The height of hollow, ribbed and caisson slabs is taken not less than 25 cm and not more than DBN B.2.2-24:2009 – 50 cm, concrete class – not less than B25.

In flat floor slabs in densely reinforced areas around columns, where maximum transverse bending forces and torques act, it is recommended to lay fiber-reinforced concrete with a tensile strength class of at least B25 to prevent punching, simplify reinforcement, and facilitate concreting.

The main design parameters of flat foundation slabs are dimensions (slab thickness), concrete compressive strength class, and longitudinal reinforcement content, which are determined depending on the reactive soil pressure of the base and the spacing of columns and walls.

When designing, it is recommended to take the optimal design parameters of the foundation slabs, established on the basis of a technical and economic analysis. In this case, the thickness of the foundation slabs is recommended to be at least 60 cm (the thickness of one of the slabs for box and caisson foundations) and not more than 200 cm, the concrete class is at least B20, the reinforcement is at least 0.3%, and the waterproofness grade is at least W6.

Ribbed and box foundations consist of slab and wall elements and are used to increase the rigidity of the building, and at a height of more than 2 m, also to use the underground space as technical floors.

Pile foundations consist of monolithic grillages in the form of common foundation slabs, strip foundation slabs under walls, separate foundation slabs under columns, and driven, bored, bored injection, and other piles.

The type and location of piles along the foundation slab field should be selected depending on the structural system of the building, the loads on the piles, and the engineering and geological conditions of the foundation.

Design of the main load-bearing structures of monolithic houses

Columns are reinforced with longitudinal, usually symmetrical reinforcement, located along the contour of the cross-section and, if necessary, inside the cross-section, and with transverse reinforcement along the height of the column, covering all longitudinal bars and located along the contour and inside the cross-section.

The design of transverse reinforcement within the cross-section and the maximum distances between clamps and ties along the height of the column should be taken such as to prevent bulging of compressed longitudinal bars and ensure uniform perception of transverse forces along the height of the column.

It is recommended to reinforce walls, as a rule, with vertical and horizontal reinforcement located symmetrically near the sides of the wall, and with transverse ties connecting the vertical and horizontal reinforcement located on opposite sides of the wall.

The maximum distance between vertical and horizontal bars, as well as the maximum distance between transverse ties, should be taken to prevent bulging of vertical compressed reinforcement and ensure uniform perception of forces acting in the wall.

On the end sections of the wall, transverse reinforcement in the form of U-shaped or closed clamps should be installed along its height, which create the necessary anchoring of the end sections of the horizontal rods and protect the end compressed vertical reinforcement of the walls from bulging.

Wall junction zones at intersections should be reinforced along their entire height with cross-shaped U-shaped or bent clamps, which ensure the perception of concentrated horizontal forces in wall joints, and also protect vertical compressed rods in joints from bulging, which ensures anchoring of the end sections of horizontal rods.

Reinforcement of pylons, which occupy an intermediate position between walls and columns in terms of their geometric characteristics, is performed as for columns or as for walls, depending on the ratio of the length and width of the pylon cross-section. When the ratio of the smaller side to the larger side is more than 1:4, the type of reinforcement approaches the type of column reinforcement. When the ratio of the smaller side to the larger side is less than 1:4, the type of reinforcement approaches the type of wall reinforcement (the main reinforcement is located along the long sides).

The amount of vertical and horizontal reinforcement in the wall should be set according to the forces acting in the wall. It is recommended to provide uniform reinforcement over the wall area with increased reinforcement near the ends of the wall and near openings.

Reinforcement of flat slabs should be carried out with longitudinal reinforcement in two directions, located near the lower and upper edges of the slab, and if necessary (according to the calculation) with transverse reinforcement, located near columns, walls and across the slab area.

At the end sections of flat slabs, transverse reinforcement should be installed in the form of U-shaped clamps located along the edge of the slab, which ensure the perception of torques near the edge of the slab and the necessary anchoring of the end sections of longitudinal reinforcement.

The number of upper and lower longitudinal reinforcement in the floor slab (covering) should be set according to the acting forces. In this case, it is recommended for irregular structural systems to install the lower main reinforcement mesh (Ø10 or Ø12 with a step of 200) in order to simplify reinforcement, and in the spans to install additional reinforcement that corresponds to the span moments. The main upper reinforcement should be taken as the same as the lower one, and additional upper reinforcement should be installed near the columns and walls, which, together with the main one, should perceive the supporting forces in the slab. For regular structural systems, it is recommended to install longitudinal reinforcement along the over-column and inter-column strips in two mutually perpendicular directions according to the forces acting in these strips.

To reduce the cost of reinforcement, it is also recommended to install the lower and upper reinforcement over the entire surface of the slab, corresponding to the minimum percentage of reinforcement, and in areas where forces are acting that exceed the forces that can be perceived by this reinforcement, install additional reinforcement, which, in combination with the above reinforcement, perceives the forces acting in these areas. This approach leads to more complex reinforcement of floors, which requires more careful control of reinforcement work. Reinforcement of foundation slabs should be carried out in a similar way.

In thick foundation slabs, in addition to longitudinal reinforcement installed in the upper and lower faces of the slab, longitudinal reinforcement located in the middle zone along the thickness of the slab should be provided.

To reduce steel consumption and facilitate concreting in columns, beams and foundation slabs, instead of joining bar reinforcement with a diameter of 20 mm and more by bypassing, it is recommended to join it at the end using bath welding or compression couplings. Concreting joints in flat slabs are recommended to be placed in the span at a distance of 1/3÷1/4 of its length from the supports. These joints should not be placed in the extreme spans of the slab or on consoles.

Fire resistance of structures. If the thickness of the protective layer is more than 50 mm for heavy concrete, it must be reinforced with a mesh with a diameter of 1–2 mm and a pitch of no more than 70 mm×70 mm.

To increase the fire resistance limits of statically indeterminate structures, it is possible to increase the area of reinforcement in the upper zone compared to that required based on strength.

Topic 3 SOFTWARE COMPLEXES THAT PERFORM CALCULATION AND DESIGN OF HIGH-RISE BUILDINGS

The entire history of structural mechanics in the pre-computer period was associated with the development of techniques focused on the numerical implementation of one or another calculation scheme.

The methods of moment and angular focal ratios, the Cross method, the method of redistribution of initial angular deformations, the elastic center method and other techniques – this is a small list of a huge number of techniques that existed at that time and methods for calculating structures, which, in fact, came down to the desire to avoid a large number of calculations. And when a specialist was drawing up a calculation scheme, he first of all thought about the possibility of solving the problem in such a formulation. The situation changed radically with the advent of computers and the finite element method. The finite element method (FEM) is a numerical method for solving problems in applied physics. The key idea of the method in analyzing the behavior of structures is the following: a continuous environment (the structure as a whole) is modeled by dividing it into regions (finite elements), in each of which the behavior of the environment is described using a separate set of selected functions representing stresses and displacements in the specified region. These sets of functions are often specified in such a form as to satisfy the conditions of continuity of the characteristics they describe throughout the environment. An example of a finite element model of a connection node of metal structure elements and the distribution of normal stresses in it under loading, obtained using the LIRA-SAPR software package.

The LIRA-SAPR software package is a multifunctional software package for the calculation, research and design of structures for various purposes. The LIRA-SAPR software package is based on the use of the finite element method (FEM) in the form of displacements, recognized worldwide as the main tool for numerical analysis of the strength and stability of building structures. The LIRA-SAPR software package is successfully used in the calculations of construction, mechanical engineering, bridge construction, nuclear power engineering, and oil production facilities.

LIRA software complexes have more than 50 years of history of creation, development and use in scientific research and practice of structural design. They are continuously improved and adapted to new operating systems and graphic environments. In addition to the general calculation of the object model for all possible types of static loads, temperature, deformation and dynamic actions (wind with pulsation, seismic actions, etc.), the LIRA-SAPR PC automates a number of design processes: determination of design combinations of loads and forces, selection and verification of sections of steel and reinforced concrete structures with the formation of sketches of working drawings of columns and beams. The LIRA-SAPR PC allows you to study the general stability of the calculated model, check the strength of sections of elements according to various theories of destruction, provides the opportunity to carry out calculations of objects taking into account physical and geometric nonlinearities, and simulate the process of construction of a structure taking into account the installation and dismantling of elements.

The LIRA-SAPR PC consists of several interconnected information systems:

1. VISOR-SAPR is a system that organizes a single graphical user environment with numerous possibilities for synthesizing and analyzing the task being solved.

2. PROCESSOR – consists of a set of specialized subprocessors that solve problems in linear and nonlinear formulation, perform calculations for stability and dynamic actions, implement a super-element approach, and allow modeling the construction process.

3. LARM-SAPR is a system for designing reinforced concrete elements.

4. STK-SAPR is a system for designing steel elements.

5. CS-CAD is a cross-section designer.

6. RS-SAPR – editor of the rolling stock database.

7. KM-CAD is a system integrated into the AutoCAD environment that allows you to create working drawings based on the calculation results.

8. DOCUMENTATOR – a system for preparing design documentation.

Solving only linear problems - that is, those in which the relationship between the forces acting on the structure and the displacements of its points is assumed to be linear, and the basis of physical equations, as a rule, is the generalized Hooke's law. In reality, this often does not correspond to reality. For example, concrete, wood, plastic and some other building materials are characterized by a nonlinear dependence of stresses on deformations even under small loads. Such nonlinearity is called physical. It manifests itself when plastic deformations occur, with a curvilinear "stress-strain" diagram, when the properties of materials change due to external influences, etc. With significant displacements commensurate with the dimensions of the structure, it is necessary to take into account geometric nonlinearity, for example, in the case of longitudinal and longitudinal-transverse bending of rods, a change in the coordinates of the points of the structure due to relatively large displacements. A special place is occupied by constructive nonlinearity associated with a change in the design scheme of the structure during loading (taking into account one-sided connections): during contact interaction of deformable bodies (one-sided supports, cracks), when calculating cable-type structures, with puffs, etc.

The calculation of nonlinear systems is a more complex task compared to the solution of linear problems, since here it is necessary to take into account the deformed state of the considered area, abandon the principle of independence of the action of forces, and apply special methods of searching for and analyzing the solution. Under these conditions, it is usually not possible to obtain an analytical solution to the problem, therefore the calculation is performed using numerical methods, most often MFE, using the procedures of successive approximations. The methods considered below are often used to solve nonlinear problems. All of them are based on the linearization of nonlinear equations, that is, the search for a solution to nonlinear equations is carried out by solving a recurrent sequence of linear equations. Some of them are relatively little known (the method of successive stiffnesses), although in some cases they are quite effective.

The considered methods can be divided into two classes:

1) iterative, for which the accuracy of the solution is known, but the number of calculations (iterations) that will be required to achieve the given accuracy is unknown:

– elastic solution method – the total stiffness matrix is represented as the sum of linear and nonlinear components;

– the method of variable stiffnesses (in mathematics it is known as the method of secants, - the general stiffness matrix and its iteration are determined based on the displacements of the iteration;

– Newton-Raphson method – at each iteration, a correction is determined based on the deviation of nodal forces;

– Newton-Kantorovich method – modified Newton-Raphson method without recalculating the stiffness matrix at each iteration.

2) straight, for which the number of calculations is known in advance, but it is not known what accuracy will be achieved. These include stepwise methods:

– method of successive stiffnesses;

– method of successive loads taking into account residuals.

Other modifications of stepwise methods are also known. Most modifications of stepwise methods involve refining the solution at each or subsequent steps.

The sequence of modeling the construction process **of the life cycle of structures in the LIRA PC**: 1) a structural diagram of the entire object is given, i.e. all elements are described, including both the main load-bearing elements of the object (columns, beams, slabs, diaphragms) and temporary elements (formwork elements, scaffolding posts, etc.); 2) for each stage of construction, all structural elements that are erected at the time of this stage, temporary supports that are present at this stage, as well as supports that are dismantled at this stage are described; 3) for each stage, the loads (self-weight, installation loads) that act at this stage are given, as well as, if necessary, the reducing coefficient of concrete strength. The initial data for the last stage correspond to the operational stage of the object, i.e. the stage when the object is fully erected, temporary supports are removed, the operational strength of the concrete is reached, and operational loads (self-weight, wind, snow, payloads) are applied. At each stage of the object's construction, the corresponding structural scheme is calculated for its own weight and installation loads, taking into account the existing, removed or added temporary supports. Further calculations for the calculated combinations of operational loads (snow, wind, roof self-weight) are performed not from the "zero" NDS of the assembled structure, but from the NDS corresponding to the last stage of installation.

The LIRA PC has tools that allow you to display all the results on the screen in a user-friendly form, clearly and quickly analyze the received data, and select the necessary ones for compiling a report. The LIRA PC includes the DOCUMENTATOR documentation system, which allows you to automatically create in interactive mode:

– tables with numerical calculation results;
– letters with graphic, tabular and textual information;
– text documents describing the problem, methods for solving it, and results.

Standard tables are generated for all tasks depending on the type of calculation, however, the following are mandatory for all calculations:

1. Node displacements. The table header indicates the node numbers of the scheme in ascending order. Under the header, the load numbers and vibration shapes, if the loads are dynamic, are indicated in separate lines. Under the load numbers, the displacement identifiers are indicated, and in the columns under the node numbers, their values.

2. Forces and stresses. The table header indicates the type of finite element, its serial number in the diagram, and the section number for a rod or node for a 120 plane element.

Under the header, in a separate line, the load numbers and vibration forms are indicated, if the loading is dynamic. Under the load numbers, the force or stress identifiers corresponding to the element type are indicated. Interactive tables allow the user to freely change their content as needed. For interactive tables, a set of standard table forms is provided, which the user can edit or supplement at his own discretion. Creating a table form is the selection of the table content and the corresponding formation of its structure.

The following standard table formats are provided:

- nodes;
- elements;
- nodal loads;
- stiffness;
- local loads;
- effort (rods);
- effort (plate);
- effort (special elements);
- RLZ (rods);
- RSZ (plates);
- oscillation frequencies;
- forms of oscillations and weight of masses;
- inertial loads.

The results are displayed in the units of measurement that were set by the user in the software configuration options.

Topic 4 FIRE SAFETY OF HIGH-RISE BUILDINGS

The design of high-rise buildings must be carried out in accordance with fire safety requirements in accordance with DBN V.1.1-7, other current regulatory documents on fire safety, and the provisions of these Standards.

The list of necessary calculations (time for evacuating people, etc.) for each building is determined during the development of project documentation and is agreed with the state fire supervision authorities.

Fire distances between high-rise buildings and other buildings should be taken in accordance with the requirements of DBN 360 as for buildings of the 1st degree of fire resistance.

The distance from a high-rise building to the nearest fire station should be no more than 2 km when traveling o the city's public transport network.

When choosing a land plot for a high-rise building, the project must include a fire station equipped with special equipment at the specified distance, and if there is an existing depot, it must be equipped with special fire equipment in agreement with the central (territorial) body of state fire supervision.

When designing a high-rise building, access routes for fire engines, as well as platforms for fire engines and helicopters, should be provided in accordance with the requirements of Appendix M.

The need to equip high-rise buildings with collective-use rescue devices, their type, quantity, and locations are determined by the design organization in agreement with the state fire supervision authorities.

Fire safety requirements for the arrangement of parking lots and garages in high-rise buildings must be met in accordance with DBN V.2.3-15.

In high-rise buildings, it is prohibited to install production premises of any category in terms of explosion and fire hazard, as well as warehouses of categories A and B.

The operational documentation includes the following: general instructions on fire safety of a high-rise building; instructions on the actions of the fire safety service and service personnel in the event of a fire; instructions on the operation and maintenance of automatic fire alarm systems, fire extinguishing systems, smoke protection systems, fire water supply systems, fire alarm systems and evacuation management systems, as well as dispatching fire protection systems (FPS).

Spatial planning solutions

On the first floor of high-rise buildings, premises for a fire station (central fire control room, control room) should be provided. The premises for a fire station (central fire control room, control room) should be designed close to the external wall with natural lighting and direct access to the outside.

Premises for different purposes (residential, public) should be separated from each other by solid fire-resistant walls. RECOMMENDATIONS FOR CHOOSING CONSTRUCTIVE SOLUTIONS TO PRESERVE THE BEARING CAPACITY OF REINFORCED CONCRETE STRUCTURES DURING A FIRE:

1. The thickness of the protective layer of concrete in the structure must be sufficient to ensure that the protective layer of concrete warms up to a temperature not higher than 300 °C, and the fire does not affect the further operation of the structure. In a standard fire lasting 180 minutes, the thickness of the protective layer of concrete must be at least 60 mm. In this case, the protective layer of concrete must have reinforcement in the form of an anti-splinter mesh of rods with a diameter of 2-3 mm with mesh sizes not exceeding 50 mm, which will help prevent explosive destruction of concrete.

2. The heating temperature of prestressed reinforcement during a fire should not exceed 100 °C to prevent loss of prestress.

3. In columns with longitudinal reinforcement in the amount of more than four bars in the cross-section, it is advisable to install some of the bars near the core of the column cross-section, if the forces allow, to remove the reinforcement as far as possible from the heated surface.

4. Columns of large cross-section with a lower percentage of reinforcement resist fire better than columns of smaller cross-section with a higher percentage of reinforcement.

5. Beams and columns with rigid reinforcement inside the cross-section have an advantage over beams reinforced with bar reinforcement located near the heated surface.

6. In beams, if there is reinforcement of different diameters and different levels, the reinforcement of larger diameter should be placed as far as possible from the surface that heats up during a fire.

7. It is better to use wide and low beams than narrow and high ones. It is recommended to use more than two rods as the main reinforcement, and to place part of the main reinforcement in the second row, as far away from the heated surface as possible.

8. In slabs, to prevent the longitudinal reinforcement from bulging when heated during a fire, it is necessary to provide structural reinforcement with clamps and transverse rods.

9. Prestressed non-stressed beam and slab structures are preferable to prestressed ones.

10. At the supports between adjacent beams and between the beam and the wall, there must be a gap that allows the beam to extend freely during fire exposure. The width of the gap must be at least 5% of the beam span.

11. Temperature joints must be filled with non-combustible fibrous materials. The width of the temperature joint must be at least 0.15% of the distance between the temperature joints.

Topic 5 ENERGY PERFORMANCE CERTIFICATE AND THERMAL INSULATION OF HIGH-RISE BUILDINGS

Thermal insulation of high-rise buildings must be designed in accordance with DBN V.2.6-31.

The design task establishes the energy efficiency class of a high-rise building A or B in accordance with the classification of DBN V.2.6-31.

The given heat transfer resistance of opaque enclosing structures or opaque parts of enclosing structures (external walls, coverings, floors, etc.) must comply with the requirements of Table 1 of DBN V.2.6-31 in terms of the minimum permissible value of heat transfer resistance $R_{q\ min}$.

The resulting heat transfer resistance of transparent enclosing structures must meet the requirements of clause 6a of table 1 of DBN V.2.6-31 in terms of the minimum allowable value of heat transfer resistance $R_{q\ min}$.

The minimum permissible value of heat transfer resistance of internal inter-apartment structures (walls, ceilings) that delimit rooms with calculated air temperatures that differ by more than 3 °C and rooms with apartment-specific heat consumption regulation is determined in accordance with 2.5 DBN V.2.6-31, while the calculated internal air temperatures of the rooms are taken in accordance with the requirements of Section 7 of this regulatory document.

When calculating the heat transfer resistance of enclosing structures, the calculated value of thermal conductivity is taken under calculated operating conditions B in accordance with Table L.1 of Appendix L of DBN V.2.6-31.

When designing the thermal insulation shell of a building based on specific heat losses for heating, it is mandatory to comply with the requirements of 3.1 of DBN V.2.6-31. The normative maximum heat losses of high-rise buildings are accepted according to Table 4 of DBN V.2.6-31, depending on the purpose of the building and the temperature zone of operation, taking into account the requirements of 6.2 of these Standards.

It is allowed to use individual structural elements of the thermal insulation shell with reduced heat transfer resistance values up to 80% of $R_{q\ min}$. for opaque parts of external walls and up to 85% of $R_{q\ min}$ for other enclosing structures in accordance with formula (1) DBN B.2.6-31, provided that the conditions for these elements of the thermal insulation shell according to formulas (2) and (3) DBN B.2.6-31 are met.

The temperature difference between the temperature of the internal air and the reduced temperature of the internal surface of the enclosing structures Δt_{pr} must comply with the requirements of Table C of DBN B.2.6-31.

The temperature difference Δt_{pr} when checking the fulfillment of the condition according to formula (2) DBN V.2.6-31 for enclosing structures is calculated depending on their glazing coefficient in accordance with Appendix M DBN V.2.6-31. When calculating the temperature difference for enclosing structures with a glazing coefficient of more than 0.18, the reduced temperature of the inner surface of the opaque and translucent parts of the structure is determined, which relates, respectively, to the area of the opaque and translucent parts.

The minimum permissible temperature of the inner surface of opaque enclosing structures in the areas of heat-conducting inclusions, in the corners and slopes of window and door openings, as well as sashes, boxes, imposts, frames, risers and transoms of translucent enclosing structures must be no less than the dew point temperature tp at the calculated environmental parameters.

The temperature difference on the inner surface of translucent enclosing structures, including the zones of distance frames and the temperature of the internal air, must be at least 4 °C.

When determining the minimum temperature of the inner surface of the enclosing structures in accordance with 5.10 and 5.11, the calculated value of the outdoor air temperature is taken depending on the temperature zone of operation of the building in accordance with Appendix Zh DBN V.2.6-31 for enclosing structures up to a height of 73.5 m. For enclosing structures located above the conditional height of 73.5 m, the calculated value of the outdoor air temperature is taken 1 °C lower for every 50 m of the building height relative to the calculated values of Appendix Zh DBN V.2.6-31.

The determination of the heat resistance indicators of enclosing structures is established according to Section 4 of DBN V.2.6-31.

The determination of the air permeability resistance of enclosing structures is carried out in accordance with the requirements of Section 5 of DBN V.2.6-31, taking into account the height of the building and the speed of movement of external air according to Table T1 of Appendix T of DBN V.2.6-31.

The humidity regime of the enclosing structures must comply with the condition of no moisture condensation in accordance with Section 6 of DBN V.2.6-31.

Based on the results of calculations of the thermal performance of the thermal insulation shell, an energy passport of the building is drawn up in accordance with Section 7 of DBN V.2.6-31 in the form in accordance with Appendix F of DBN V.2.6-31. The energy passport is an integral part of the design documentation. The design of the thermal insulation shell of high-rise buildings must be carried out using thermal insulation materials with a service life of at least 40 years; for replaceable seals – with a service life of at least 20 years with maintenance elements thermal insulation shell. The design (in particular, architectural drawings) and operational documentation should provide data on the specified service life of the thermal insulation materials used, and also provide for checking the thermal insulation properties of the enclosing structures after the end of their service life. In this case, it is necessary to ventilate the layer between the wall and the outer facing layer.

The natural flow of outside air into the premises should be carried out through adjustable ventilation valves and openings located in external walls and/or windows, while ensuring permissible noise levels in the premises.

For external enclosing structures of heated buildings and structures and internal structures separating rooms, the air temperature in which differs by 3 °C or more, the following conditions must be met:

$$R_{\Sigma np} \geq R_{q\min},$$

$$\Delta t_{пр} \leq \Delta t_{сг},$$

$$\tau_{в\,min} \rangle t_{min},$$

where $R_{\Sigma пр}$ – reduced heat transfer resistance of an opaque enclosing structure or an opaque part of an enclosing structure (for thermally homogeneous enclosing structures, the heat transfer resistance is determined), reduced heat transfer resistance of a translucent enclosing structure, m^2 K/W;

$R_{q\ min}$ – minimum permissible value of heat transfer resistance of an opaque enclosing structure or an opaque part of an enclosing structure, minimum value of heat transfer resistance of a translucent enclosing structure, m^2 K/W;

$\Delta t_{пр}$ – temperature difference between the temperature of the internal air and the reduced temperature of the internal surface of the enclosing structure, °C;

$\Delta t_{cг}$ – the difference between the temperature of the internal air and the reduced temperature of the internal surface of the enclosing structure, °C, permissible according to sanitary and hygienic requirements;

$\tau_{in\ min}$ – the minimum value of the temperature of the inner surface in the zones of heat-conducting inclusions in the enclosing structure, °C;

t_{min}– minimum permissible value of the internal surface temperature at the calculated values of the internal and external air temperatures, °C.

The minimum permissible value $R_{q\ min}$ of the heat transfer resistance of opaque enclosing structures, translucent enclosing structures and doors of residential and public buildings is set in accordance with Table 5.1, depending on the temperature zone of operation of the building.

Table 5.1 – Minimum permissible value of heat transfer resistance of the enclosing structure of residential and public buildings ($R_{q\ min}$)

№ of the company	Type of enclosing structure	$R_{q\ min}$ value, m^2 K/W, for temperature zone	
		I	II
1	2	3	4
1	Exterior walls	3.3	2.8

Continuation of table 5.1

1	2	3	4
2	Combined coatings	5.35	4.9
3	Attic coverings and ceilings of unheated attics	4.95	4.5
4	Ceilings over driveways and unheated basements	3.75	3.3
5	Translucent fencing structures	0.75	0.6
6	Entrance doors to apartment buildings and public buildings	0.5	0.45
7	Entrance doors to low-rise buildings and apartments located on the first floors of multi-storey buildings	0.65	0.6

Topic 6 MONITORING AND OBSERVATION OF THE TECHNICAL CONDITION OF HIGH-RISE BUILDINGS

Monitoring and surveillance of the technical condition of high-rise buildings and residential and public buildings is a mandatory component of ensuring safety conditions during their construction and operation.

The purpose of monitoring is to assess the impact of natural, man-made, anthropogenic and other factors on the construction site and the environment during construction and operation, develop a forecast of changes in the state of the site, timely identify defects, prevent and eliminate negative processes, clarify the results of the forecast and adjust design solutions.

The task of monitoring is to develop measures to ensure the reliability of buildings and structures during their construction and operation, as well as to prevent negative changes in the environment, prevent and eliminate structural defects, and monitor the implementation of developed and adopted measures.

Monitoring composition. By functional purpose, monitoring is divided into geological-hydrological, object-based, ecological-biological, and scientific analysis of the results obtained.

Geological and hydrological monitoring includes systems of regular observations of changes in the state of soils, levels and composition of groundwater, and the development of destructive processes: erosion, landslides, karst and sufous phenomena, subsidence of the earth's surface, etc.

Site monitoring includes all types of observations of the condition of the foundations, foundations, structures of the underground and above-ground parts of a new construction site and buildings, underground structures, and infrastructure facilities surrounding it.

Ecological and biological monitoring includes systems for observing changes in the natural environment, radiation situation, etc.

Scientific analysis of the obtained results includes evaluating the results of observations, performing calculated forecasts, comparing the predicted values of parameters with the results of measurements, taking measures to prevent or eliminate the negative consequences of harmful impacts and preventing an increase in the intensity of these impacts.

Hydrogeological monitoring involves monitoring the state of the environment, namely:

– groundwater level;
– piezometric pressure of water in the soil massif;
– water consumption associated with filtration;
– determination of the filtration coefficient;
– soil temperatures in the massif;
– chemical composition, temperature and turbidity of filtered water in drains and collectors;
– efficiency of drainage water lowering and anti-filtration systems.

Site monitoring involves monitoring the condition of the foundations, foundations, structures of the underground and above-ground parts of a new construction site and the buildings, underground structures and infrastructure facilities surrounding it.

Monitoring of foundations, foundations and underground structures involves:

– geodetic measurements of movements and monitoring of the technical condition of buildings, underground structures and infrastructure facilities surrounding the construction site;
– geodetic measurements of movements of the foundations of a high-rise building or structure, as well as of base benchmarks located within the possible impact of construction: vertical movements (settlement, heave), horizontal movements (landslides), rolls and uneven vertical movements;
– post operativegeodetic control of the accuracy of installation of structural elements;
– measurement oscillation underground structures by the presence of dynamic influences;

– control of the deformation and stress state of the foundations, foundations and supporting structures of the underground part;

– recording and monitoring the appearance and opening of cracks;

– monitoring the condition of underground enclosing structures, their humidity and the condition of waterproofing;

– measurement of layer-by-layer deformations of the base soils and subsidence of the earth's surface within the possible impact of construction; recording changes in the physical and mechanical characteristics of soil properties.

Monitoring of the above-ground part of the building structures involves:

– geodetic measurements of movements of a high-rise building or structure: vertical movements (settlement, heave), horizontal movements (landslides), rolls and uneven vertical movements;

– post-operational geodetic control of the accuracy of installation of structural elements;

– control of the deformation and stress state of load-bearing structures (columns, pylons, crossbars and the reinforced concrete frame as a whole);

– monitoring the condition of enclosing structures;

– measurement of vibrations under the influence of dynamic loads (wind, seismic, ambient temperature, dynamic loads of a technogenic nature);

– recording and monitoring the appearance and opening of cracks;

– control of geometric dimensions and wall sections;

– wall deformation control;

– control of load-bearing elements of the floor and their connections (loads, geometric dimensions and deformations);

– monitoring the condition of balconies, bay windows, loggias, stairs, rafters and other structural elements.

Ecological and biological monitoring involves monitoring possible changes in the natural environment. It is necessary to monitor the following natural and man-made factors that contribute to the deterioration of the ecological situation:

– change in groundwater level;

– soil and groundwater pollution;
– gas emission;
– radiation emission;
– man-made physical fields;
– vibration and shock effects.

Assessment of possible oscillations and vibrations must be carried out not only in terms of their impact on the building, but also on people.

Organization and design of monitoring work.

Monitoring is an integral part of the scientific and technical support of new construction in accordance with DBN V.1.2-5. It must be carried out by specialized organizations dealing with scientific and technical research, development of design solutions and technology for performing work.

The project and monitoring program are developed according to the technical specifications, which are drawn up by the monitoring organization, agreed upon by the general designer, and approved by the customer.

The scope of monitored parameters is established by the designer when developing the technical specifications for monitoring.

The monitoring program details the general provisions set out in the technical specifications, provides for the composition of observations, the scope of work, methods of performing observations depending on the specific construction conditions, determines the principles of building the monitoring system, the selection of its main elements and blocks, and specifies the requirements for the technical characteristics of the system.

At the design stage, the following should be determined:

– basic operational requirements for facilities;
– forecast of calculated values of deformations and loads;
– requirements for the technical characteristics of the monitoring system have been clarified based on the forecast of the calculated values of the monitored parameters;
– composition and design of the monitoring system;

– layout of the monitoring system at the facility;
– design of mounting nodes for monitoring system elements;
– technology for performing monitoring work;
– methodology for processing and analyzing the obtained data.

When developing the layout of the monitoring system at the facility and the design of the mounting nodes of the monitoring system elements, it is necessary to ensure arrangement of points for installing monitoring system elements on building structures or in other places, depending on the functional purpose of the relevant elements. If it is necessary to create basic measuring points, they should be organized outside the influence of construction on the monitored parameters.

The arrangement of the monitoring system should ensure protection of its elements from damage during the construction and operation of the building. The installation locations of the system elements should be located in monolithic reinforced concrete or brick niches that prevent unauthorized access, or in lockable metal containers.

The installation locations of the system elements must be provided with power supply and a communication system with the information collection point.

The project should provide for the location of an information collection point, which can be combined with the control room.

If the monitored parameters exceed the tolerance by a value greater than the specified value, an emergency alert system must be activated.

During the construction stage, observation systems are installed, observations are conducted, and scientific analysis of the results is performed.

Monitoring, as a rule, must be carried out using complex automated systems.

Instruments and equipment used for observations must be certified, calibrated or attested in accordance with the requirements of regulatory documents.

At the operation stage, monitoring of the foundations, foundations, underground structures and structures of the above-ground part of the building is carried out, as well as geodetic measurements of movements and control over the technical condition of buildings, underground structures and infrastructure facilities surrounding the construction site during the stabilization period, unless otherwise provided for by the project.

Stationary automatic monitoring systems provided for by the project should, if possible, function throughout the operation of the facility.

Reporting form for monitoring work

The organization conducting building monitoring reports to the customer and the general designer.

Reporting form – scientific and technical report, which includes:

– monitoring results, which can be presented in the form of defect information, graphs of the development of settlements and slopes of the building, deformations of structures, inspection reports of the condition of above-ground and underground structures of the building, materials reflecting the quality control of the work, and other materials in accordance with the monitoring program;

– conclusions about the condition of houses located near the new construction; materials of scientific analysis of the results obtained, including an assessment of the results of observations;

– performing calculated forecasts, comparing predicted parameter values with measurement results, proposing measures to prevent or eliminate the negative consequences of harmful impacts and preventing an increase in the intensity of these impacts.

In the event of deformations and other phenomena that differ from those predicted and pose a danger to the surrounding development or new construction, this must be brought to the attention of the customer, general contractor, and design organization for joint emergency measures.

Terms of Reference for monitoring the construction and operation of high-rise buildings

The terms of reference should cover the following main issues:

– name of the construction site;

– location, type of construction, dimensions and design of the object, location in relation to existing development;

– names and brief descriptions of existing buildings and structures located in the area of influence of the new construction;

– geological site structure, presence of dangerous geological processes that may be initiated by new construction;

– justification for monitoring;

– design stage;

– goal, objectives and composition of the work;

– summary of reporting materials and customer responsibilities;

– name of the customer and project organization

Topic 7 LOADS ON STRUCTURAL ELEMENTS OF A HIGH-RISE BUILDING

To determine stiffnesses when taking into account load combinations

When forming a table of calculated force combinations (CFC) for nonlinear system calculation (physical, geometric, structural and other types of nonlinearity), a multi-criteria problem arises. In this regard, when assigning the current value of the corresponding stiffness (flexural, tensile-compressive, shear, etc.), depending on a specific load, it is permissible to take

$$\mathrm{B}_m = \frac{\sum_{L=1}^{\mathrm{N}} q_L \mathrm{B}_L}{N}, \qquad (7.1)$$

where B_L is the stiffness of the element, which is determined for the L-th load;

q_L – weight coefficient $0.0 \le q_L \le 1.0$;

N – the number of loads entered into the calculation;

B_m – averaged stiffness value.

With a significant spread of B_L values, corresponding to equally safe, but significantly different loads:

$$B_m = \frac{1}{3}\left(B_m + B_{mg} + B_{np}\right), \qquad (7.2)$$

where
$$\mathrm{B}_{mp} = \left(\prod_{L=1}^{N} q_L B_L\right)^{\frac{1}{N}}, \qquad (7.3)$$

$$\mathrm{B}_{mg} = \frac{N}{\sum_{L=1}^{\mathrm{N}} q_L \mathrm{B}_L}. \qquad (7.4)$$

It should be borne in mind that the geometric nonlinearity of the problem appears when nonlinear dependencies between deformations and displacements are introduced into it (the square of deviations, their product, etc.) or when exact expressions are adopted for the geometric parameters of the object of study (for example, curvatures). In turn, structural nonlinearity usually arises when considering bielements such as, for example, “hanging wall - edge beam”, “foundation - base”, “slab - beam cage”, without shear bonds, etc. The nonlinearity of deformation, in these cases, is due to crack formation, the transition from one type of NDS to another, the violation of contact between parts of the bielement, etc.

Building layout. Design solutions must ensure a service life of a high-rise building of at least 100 years, taking into account proper operational maintenance and possible restoration of the resource through major repairs.

The structural system must ensure the strength and stability of the load-bearing structures and elements of a high-rise building under the action of design loads and impacts, as well as resistance to progressive collapse in the event of emergency situations.

High-rise buildings use structural systems consisting of vertical (columns, walls, cores, diaphragms, and other stiffening elements) and horizontal (floors, coverings, beams, diagonal chords, and other stiffening elements) load-bearing structures that ensure their load-bearing capacity and spatial rigidity.

To ensure spatial rigidity of the structural system of a high-rise building, it is recommended to use:

– developed in plan and symmetrically arranged cores and stiffness diaphragms;

– structural systems with external walls around the entire contour of the house (shell type);

– structural systems with symmetrical and uniform arrangement of load-bearing structures in plan and along the height of the building and, accordingly, with uniform distribution of vertical loads; monolithic floor discs that combine vertical load-bearing structures and perform the functions of horizontal stiffness diaphragms under the action of wind and seismic loads;

– horizontal beam or diagonal stiffening belts at the level of the grillage floors, which ensure the joint bending of all vertical structures of the house, as well as rigid nodal connections between the supporting structures.

– The supporting structures of high-rise buildings can be made of monolithic reinforced concrete, steel-reinforced concrete, and metal.

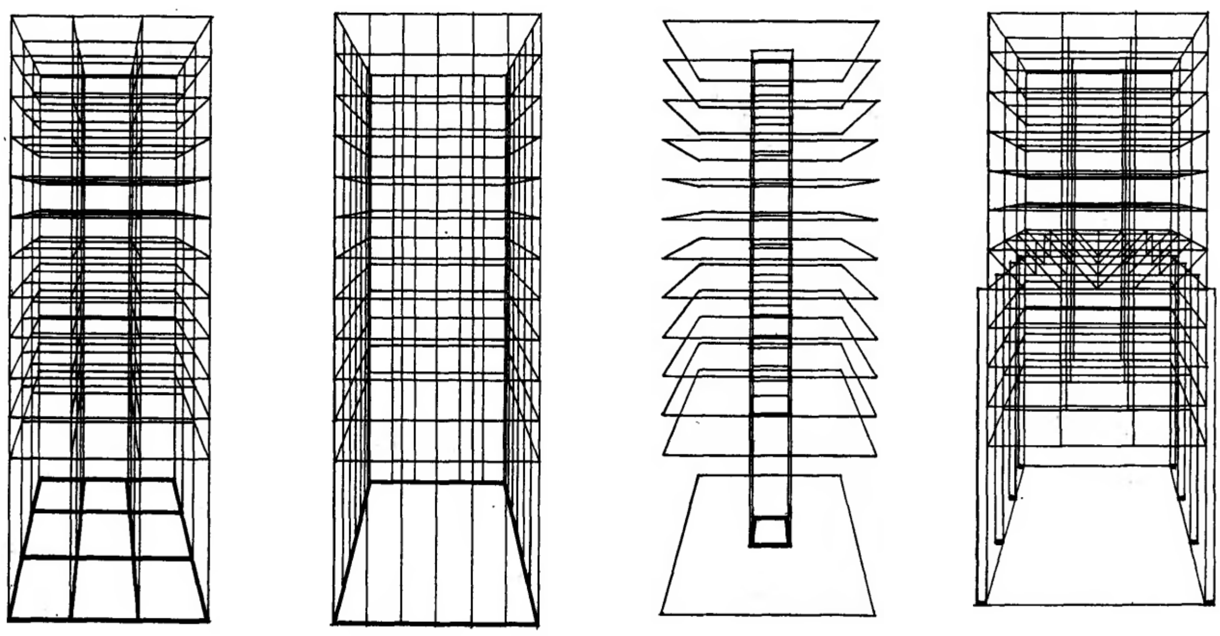
It is recommended to make load-bearing structures made of monolithic reinforced concrete from concrete of class no lower than C25/30.

The load-bearing structures of high-rise buildings require the integrity of the elements that transmit the load to the foundation, and thus the consistency of the load transfer for each floor. Therefore, the distribution of load transfer points should be determined not only by statics considerations, but also by the principle of rational use of space.

The load-bearing structures of high-rise buildings may differ in different systems of floor load transfer. Here it is appropriate to introduce the following classification of systems: raster, shell, trunk, bridge.

Integrally, these systems are designated as height-active systems.

In a raster system, the load transfer points are distributed evenly over the entire plan; in a shell system, they are located on the periphery; in a trunk system, the load concentration zone is in the center; in bridge systems, the load concentration points are perceived by the structure located above (Fig. 7.1).

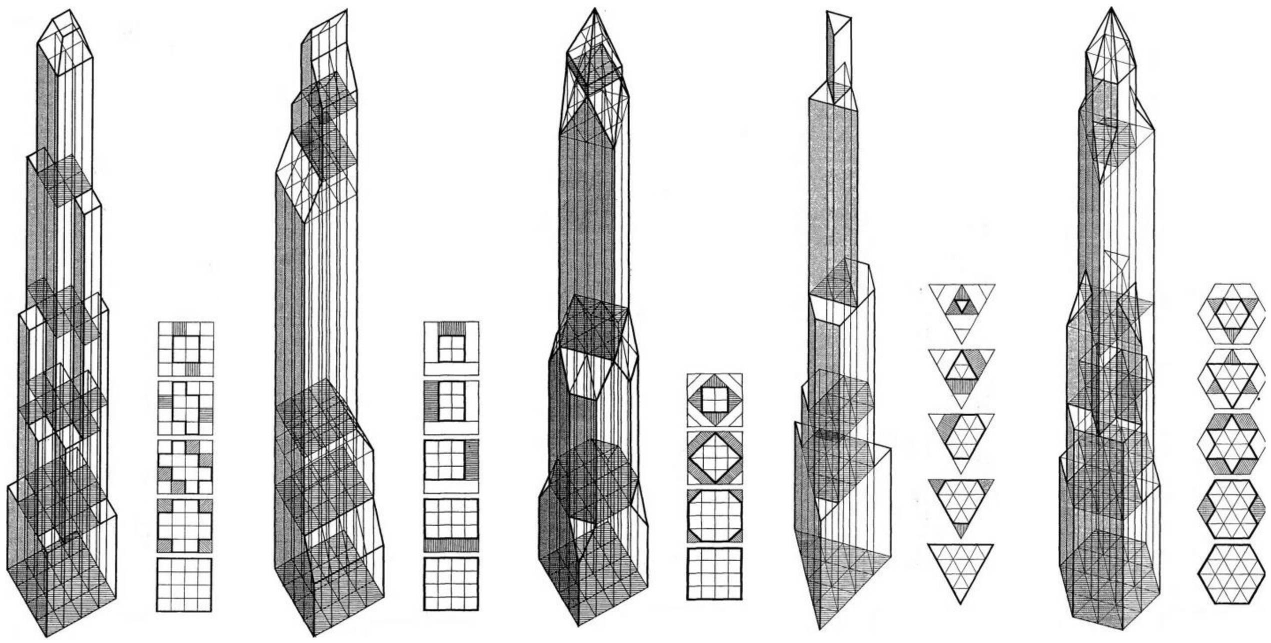


Raster high-rise buildings	Shell high-rise buildings	High-rise buildings with trunks	Bridge high-rise structures

Figure 7.1 – Classification of active load-bearing systems by height (prototypes)

In high-rise buildings, load transfer systems are closely related to the shape and structure of the plan. This relationship is determined by the number of pylons and their location in the plan.

To create the necessary prerequisites for flexible floor planning and opportunities for further redevelopment of the premises on each floor, the design of load-bearing systems active in height aims to reduce the number of load-bearing elements as much as possible.

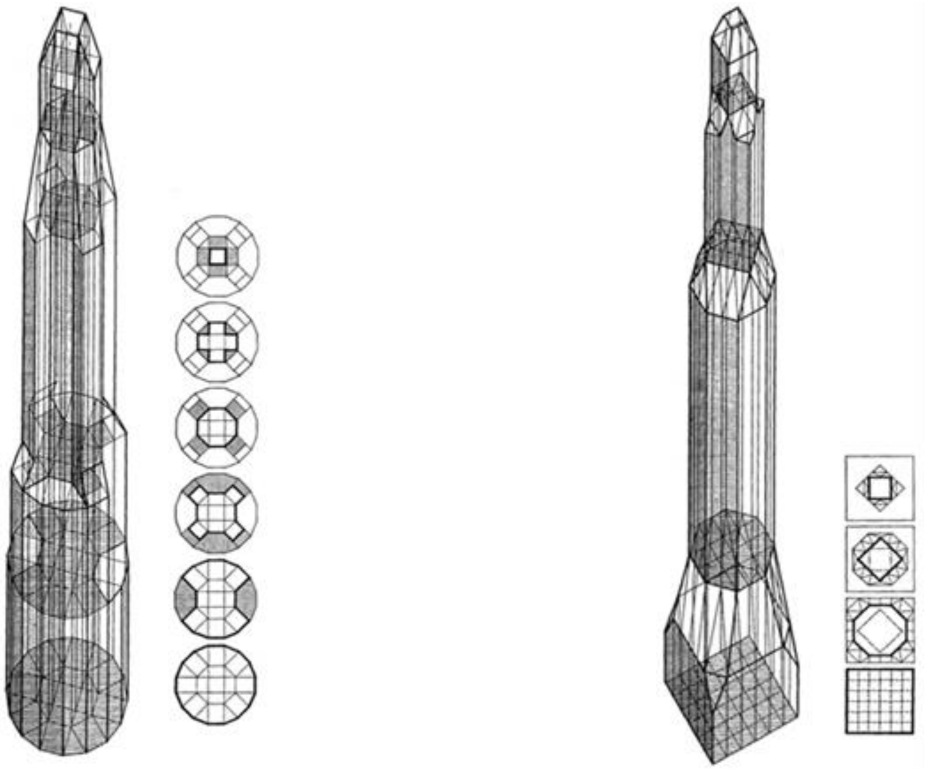


Symmetric axial sequence of floor plans | Horizontal projection transitions beveled on one side | Symmetrically beveled floor plan transitions | Sequence of plans floors above the triangle as the main plan form | Sequence floor plans over a hexagon as the basic plan form

Figure 7.2 – Examples of tower shapes in a raster system

High-rise structures require large cross-sectional areas of supports to transmit vertical loads, which limits the usable floor area. By suspending instead of supporting the floor, a significant reduction in the cross-sectional area of the load-bearing elements can be achieved; however, this indirect load transfer requires an overhead supporting system to ultimately transfer the load to the foundation.

Due to the need to limit the number of load-bearing elements to a minimum for optimal use of space, all spatial elements required for a high-rise building must be



Sequence over the circle as the basic form of the plan

Gradual change of plan over several floors

Figure 7.3 – Examples of tower shapes in a shell system

When designing high-rise (H>73.5m) and medium-height buildings (40.0m≤H≤73.5m), preference should be given to the minimum number of pylons with developed dimensions in plan (Fig. 7.4).

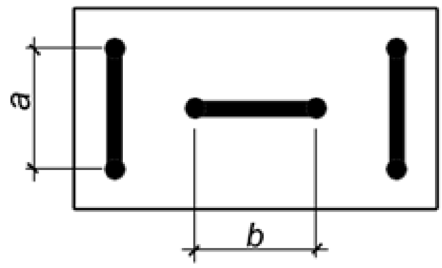


Figure 7.4 – Pylon layout

The minimum necessary and sufficient to ensure the geometric immutability of the building is a system of pylons, which includes at least three flat diaphragms, the planes of which do not intersect on the same straight line and are not parallel (Fig. 7.4)

In buildings with an extended plan, the distance between parallel pylons should be no more than 30.0 m, and the distance from the extreme pylon to the end of the building (cantilevered section of the floor) should be no more than 12.0 m. If necessary, changes to the above restrictions should be confirmed by calculation.

The minimum required number of stiffening elements in a building (pylons, cores, etc.) should be determined from the condition of ensuring its overall stability:

$$\frac{G_{cr}}{G^n} \geq 1{,}5 \tag{7.5}$$

where G^n is the standard weight of the building, including permanent and temporary standard loads. In this case, the value of G^n should be taken equal to the weight of the above-ground part of the building, increased by 10%;

G_{cr} – critical weight of the building (see stability calculation).

When determining the dimensions of a building, its external dimensions should be determined from the condition of limiting the acceleration of the upper points ($y_{max} \leq 0{,}08\ m/sec^2$).

When designing high-rise buildings, the ratio of height to the minimum cross-sectional size of the building should not exceed $h/d = 7$ (where h is the height of the building, d is the minimum cross-sectional size located at a level of $^2/_3\ h$).

If the above ratio $h/d > 7$, it is necessary:

– perform a verification calculation for vortex excitation (wind resonance);

– take into account the possibility of aerodynamically unstable oscillations such as galloping.

When calculating high-rise buildings with an asymmetric cross-sectional shape of typical floors, as well as in cases where the center of mass of typical floors does not coincide with their center of rigidity, it is necessary to take into account the possibility of aerodynamically unstable oscillations of the divergence type.

Precast-monolithic structures can be used for the construction of floors and walls using prefabricated elements as remaining formwork or as part of the load-bearing structure. Calculations and construction of concrete and reinforced concrete structures must be carried out in accordance with current regulatory documents.

Reinforced concrete load-bearing structures, which are made of concrete and hot-rolled and welded steel elements (I-beams, channels, pipes, angle elements), are recommended for use mainly for columns with limited cross-sectional area and high loads, when their bearing capacity when using flexible load-bearing reinforcement is insufficient.

In some cases, steel-reinforced concrete load-bearing structures can be used for the construction of walls, stiffening cores and floor slabs. The displacements (deflections) of the building should be estimated in accordance with Table 7.1. The horizontal limit displacements of frame buildings, limited based on design requirements (ensuring the integrity of the frame filling with walls, partitions, window and door elements), are also given in Table 7.1.

Horizontal movements of the frame should be determined in the plane of the walls and partitions (Fig. 7.5), the integrity of which must be ensured. For connecting frames of multi-storey buildings with a height of more than 40 m, the skew of the floor cells adjacent to the stiffness diaphragms, equal to (f_1/h_s + f_2/l), should not exceed 1/300: for pos. 2, 1/500 for pos. 2, a and 1/700 for pos. 2, b of table 7.1.

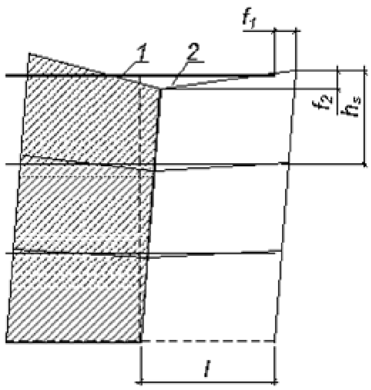


Figure 7.5 – Scheme of skew of floor cells (2) adjacent to stiffness diaphragms (1) in buildings with a connected frame (the dotted line shows the frame diagram before the application of the load)

Table 7.1 – Horizontal limit movements of frame buildings, limited based on design requirements (ensuring the integrity of the frame filling with walls, partitions, window and door elements)

Buildings, walls and partitions	Fastening walls and partitions to building frame	Limit displacement f_u
1. Multi-storey buildings	Any	h /500
2. One floor of multi-storey buildings: a) walls and partitions made of brick, plasterboard, reinforced concrete panels; b) walls lined with natural stone, ceramic blocks, glass (stained glass windows).	Flexible Hard Hard	h_s /300 h_s /500 h_s /700
3. One-story buildings (with self-supporting walls) with floor height hs, m: $h_s \le 6$ $h_s = 15$ $h_s \ge 30$	Flexible	 h_s /150 h_s /200 h_s /300

The designations adopted in Table 3.7:
h– the height of multi-storey buildings, equal to the distance from the top of the foundation to the axis of the roof beam;
h_s – the height of the floor in one-story buildings, which is equal to the distance from the top of the foundation to the bottom of the rafter structures; in multi-story buildings: for the bottom floor – equal to the distance from the top of the foundation to the axis of the floor beam; for other floors – equal to the distance between the axes of adjacent beams.

Note 1. For intermediate values of h_s (item 3), the horizontal limit displacements should be determined by linear interpolation.

Note 2. For the upper floors of multi-storey buildings designed using single-storey building roof elements, the horizontal limit displacements should be taken as for single-storey buildings. In this case, the height of the upper floor h_s is taken from the axis of the inter-storey floor beam to the bottom of the rafter structures.

Note 3. Flexible fasteners include fasteners of walls or partitions to the frame that do not prevent the frame from shifting (without transferring forces to the walls or partitions that could cause damage to structural elements); rigid fasteners include fasteners that prevent mutual displacement of the frame, walls or partitions.

Note 4. For single-story buildings with curtain walls (as well as in the absence of a hard disk covering) and multi-story racks, the limit displacements may be increased by 30% (but not more than h_s /150).

Note 5. The ratio of the maximum horizontal displacements of the upper part of high-rise buildings f to their height *h* should not exceed the ratio of 1/1000 at a building height of 73.5 m to 100 m inclusive (*f* – horizontal displacements of the upper part of the building at the level of the upper floor; *h* – the height of the building, determined by the difference in marks from the surface of the roadway around the building and the upper floor of the building).

The thickness of the walls of the stiffening cores, as well as of the load-bearing piers and stiffening diaphragms, can be taken as variable in height. The flexibility of columns and walls (ratio l_o /, where l_o is the design length and i is the radius of inertia of the cross-section) should be taken as no more than 60.

It is recommended to increase the bearing capacity of vertical structures, taking into account the gradual increase in load from upper to lower floors, by:
a) increasing the coefficient of longitudinal reinforcement;
b) increasing the strength of concrete;
c) increasing the size of load-bearing elements, taking into account planning restrictions;
d) using "hard" reinforcement. As "hard" reinforcement it is recommended
use rolled steel profiles (I-beams, including wide-shelf ones, angle elements, channels, sheet steel and pipes).

The horizontal limit deflections of columns (posts) of frame buildings from temperature, climatic and shrinkage effects should be taken as equal to:

– $h_s/150$ – for walls and partitions made of brick, gypsum concrete, reinforced concrete and hinged panels;

– $h_s/200$ – for walls lined with natural stone, ceramic blocks, glass (stained glass);

– where h_s is the floor height, and for single-story buildings with overhead cranes, the height from the top of the foundation to the bottom of the crane track beams.

In this case, temperature effects should be taken without taking into account daily fluctuations in outside air temperature and temperature drops from solar radiation.

When determining horizontal deflections from temperature, climatic and shrinkage effects, their values should not be summed with deflections from wind loads and from foundation tilt.

With different layouts of stiffening elements (pylons, cores), it is permissible to use Table 7.2 and Figure 7.6.

Table 7.2 – Stiffener locations

Position of frame elements	Cell type	Angles of skew of structures
In the plane of wind load	The cell is adjacent to the "free" columns	$\beta_1 = \varphi$
	The cell is adjacent to the pylon on one side.	$\beta_2 = \varphi \cdot \left(1 + \frac{c_2}{d_2}\right)$
	The cell adjoins the pylons on both sides.	$\beta_3 = \varphi \cdot \left(1 + \frac{c_2 + c_3}{d_3}\right)$
In the plane perpendicular to the wind load	The cell adjoins on both sides the pylons, offset on the module.	$\beta_3 = \varphi \cdot \frac{c_4 + c_5}{d_4}$
	The cell is adjacent to the pylon on one side.	$\beta_5 = \varphi \cdot \frac{c_5}{d_4}$

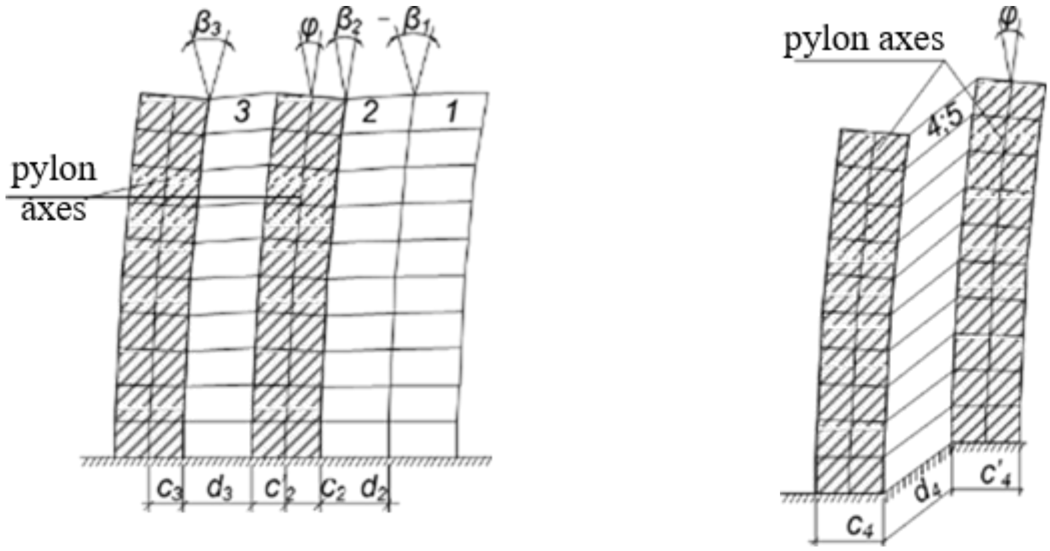


Figure 7.6 – Before determining structural distortions

For buildings with a height exceeding 73.5 m, storm wind loads should be taken into account. In this case, the horizontal displacements of the top of the building, with an overload factor of 5.4, taking into account the static and pulsation components, should not exceed 1/500 of the height of the building above the ground.

The relative difference in settlement of foundations (structures) should not exceed the standard value of 0.002.

Special cases. When designing and calculating buildings in which a certain number of pylons do not reach the foundations, the following tasks should be solved separately:

– assessment of the sufficient load-bearing capacity of the remaining elements (usually columns) reaching the foundations;

– assessment of the shear force of the corresponding column cross-section resource in the place of pylon collapse;

– assessment of the strength of the overlap disk at the location of the pylon break;

– assessment of the bearing capacity (in terms of transverse force) of other additionally loaded pylons reaching the foundations.

When designing and calculating buildings in which some of the pylons do not reach the top, the following should be considered:

– assess the strength of the overlap disk located at the point of the pylon break;

– assess the bearing capacity of the remaining pylons and columns for shear at the location of the pylon failure.

Inaccuracy The construction of vertical structures should take into account:

– when checking the strength of columns by introducing a random eccentricity;

– when the strength of the floor disks is checked, taking into account the calculated fracture angle of vertical structures ε=0.0075.

For the numerous implementation of the listed tasks, appropriate local calculation models should be formed. At the same time, final conclusions about the reliability of acceptable structural solutions should be made on the basis of a joint consideration of the calculation results obtained according to the general scheme of the model "the building itself - foundation structure - base" and the local models mentioned.

The bearing capacity of the vertical section of the pylon at shear T is taken equal to the sum of the shear resistances of all connecting elements included in the section under consideration. If the vertical section consists of several sections of height hj, differing in shear resistance per unit height tj, then

$$T = \sum t_j h_j \ . \tag{7.6}$$

The bearing capacity t_j of the vertical section at the considered displacement is taken as:

– for solid sections of walls – equal to their bearing capacity for transverse force without taking into account reinforcement, equal to the multiplication of the calculated resistance of the concrete of the pylons in tension f_{ctk} by their thickness;

– for lintels over openings – equal to the lesser of their bearing capacities under shear force or bending moment divided by the height of the floor. The strength of normal sections of the stiffener cores is checked as for pylons of an open profile.

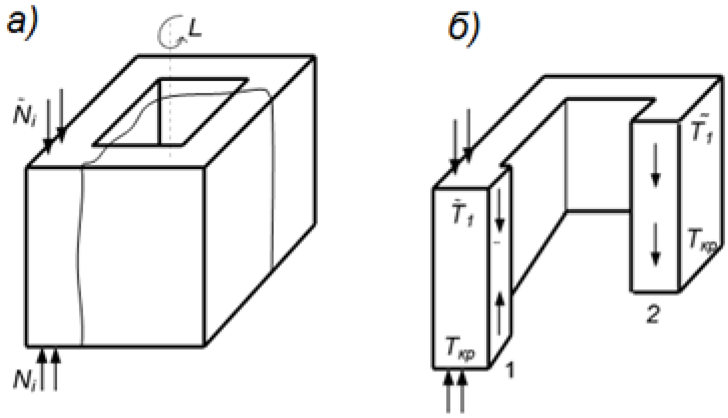


Figure 7.7 – Before checking the shear strength of the stiffener cores

The shear strength test of the stiffness cores differs from the shear strength test of open-profile pylons: any surface with a vertical generatrix intersects a closed-profile pylon simultaneously in two vertical sections (Fig. 7.7, a). In addition, the torques, which can be quite large in closed-profile pylons, cause shear forces T_{cr} (Fig. 7.7, b). These forces consist of shear forces from vertical loads and forces from transverse bending of the pylons.

The lintels above the openings in the pylons should be designed so that they are of equal strength in terms of transverse force and bending moment.

In this case, the bending moment in the jumper is determined by:

– with the width of the smaller of the pylon branches adjacent to the opening being greater than or equal to the height of the lintel above the opening

$$I'_{ï} = 0{,}5V_{ï}b_{ï}; \tag{7.7}$$

– with the width of the smaller branch less than the height of the lintel above the opening

$$I'_{ï} = 0{,}5V_{ï}b_{ï}\left(1 - 0{,}5\frac{b_c}{h_{ï}}\right). \tag{7.8}$$

Here all the notations are according to Figure 7.8.

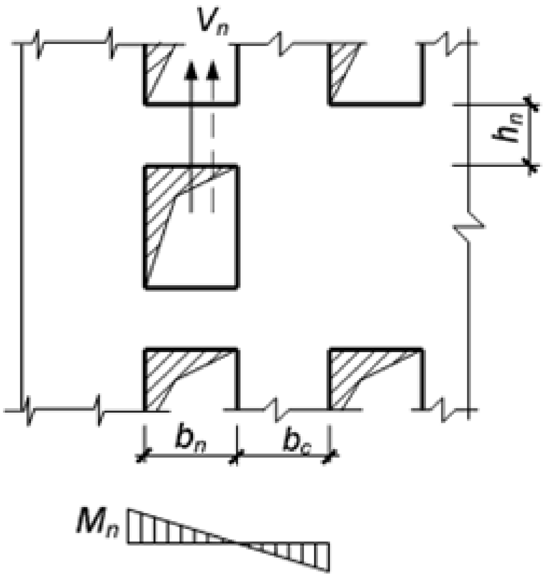


Figure 7.8 – Before calculating the lintel above the opening

The transverse force in the lintels V_n is taken on the basis of static calculation. The force from the local load should be added to the forces V_n and M_n.

To roughly estimate the level of weakening of the pylon in its plane by openings and to simplify the calculation of the building, it is permissible to use the reduction coefficient of O. R. Rzhanitsyn:

$$\beta = \sqrt{\frac{3J_i H^2}{2a^3 h}\left(\frac{F_1 + F_2}{F_1 F_2} + \frac{v^2}{J_1 + J_2}\right)}. \tag{7.9}$$

where J_n – moment of inertia of the lintel; F_1, F_2 – cross-sectional areas of the pylon branches; J_1 and J_2 – moments of inertia of the pylon branches; H – diaphragm height; h – floor height; a – half the width of the opening; v – distance between the centers of gravity of the pylon branches.

At $\beta \geq 15$, the compliance of the jumpers is small and the diaphragm works with a single cross-section and is calculated according to the beam-cantilever scheme. At $\beta \leq 0.5$, the compliance of the jumpers is large and their influence on the operation of the pylons is insignificant; the branches of the pylons (diaphragms) work separately and are calculated as separate independent beam consoles. At intermediate values $15 > \beta > 0.5$, the nature of the operation of the diaphragm with openings resembles the operation of the frame. In this case, the forces in the diaphragm elements can be determined as in frame systems.

Progressive collapse. When designing buildings and structures belonging to responsibility class CC3, responsibility category A, complexity categories IV–V, it is necessary to take into account the possibility of force majeure situations, the occurrence of which is likely to occur during the life cycle of the system (structure).

In this case, during design, it is necessary to perform a calculated assessment of the safety of the supporting structures of a high-rise building from collapse in the event of an emergency (fire or explosion), which may lead to local destruction in the volumes for high-rise buildings with a reinforced concrete frame:

– destruction (removal) of two intersecting walls in sections from the point of their intersection (for example, from the corner of the house) to the nearest opening in each wall or to the next vertical intersection with a wall of the other direction over a total length of no more than 10 m, which corresponds to the destruction of structures in a circle with an area of 80 m^2 (area of local destruction);

– destruction (removal) of a separate column (pylon) or column (pylon) with adjacent sections of walls located on the same floor in the area of local destruction;

– collapse of a section of the ceiling of one floor in the area of local destruction.

In all cases, the cross-sectional area of all removed vertical elements located on an area of 80 m^2 must not exceed 0.9 m^2 for reinforced concrete elements, 0.7 m^2 for fiber-reinforced concrete elements, and 15% for rigid reinforcement;

The main structural measures recommended to ensure the stability of a high-rise building with a reinforced concrete frame against progressive collapse include:

– creating continuity of floors and ensuring continuity of structural reinforcement in plan and height;

– use of double continuous reinforcement of floors in the upper and lower zones with a total area in both directions of not less than 0.25% of the concrete cross-sectional area;

– reinforcement (if necessary) of adjacent vertical load-bearing structures to which the load from the destroyed element may be transferred;

– reliable joining and anchoring of reinforcement at the joints of walls, columns and floors.

For a high-rise building with a metal load-bearing frame, to prevent progressive collapse, it is recommended:

– apply structural schemes of frames with rigid nodes of connections of crossbars with columns to redistribute forces during destruction;

– give preference to a reduced pitch of peripheral facade columns;

– provide for appropriate outrigger systems to redistribute effort;

– use combined steel-reinforced concrete load-bearing systems with the arrangement of reinforced concrete frames in the most dangerous zones of the lower floors for progressive destruction;

– give preference to structural solutions in the form of solid reinforced concrete monolithic slabs with dimensions for the entire floor.

When calculating building structures for resistance to progressive collapse, one should be guided by current regulatory documents.

The destruction of buildings whose load-bearing structures are designed with a metal frame must be considered according to special scenarios, in which the destruction (removal) of individual elements should be assigned to the most dangerous places, depending on the adopted structural scheme in accordance with assessments of possible risks.

It is recommended to calculate building structures as a spatial system "base - foundation - structure" using software packages that allow taking into account physical and geometric nonlinearities, as well as ensuring the highest accuracy of calculation results and reducing additional material costs.

It is recommended to calculate according to the following scheme:

– perform the calculation of the entire circuit in a physically nonlinear formulation for permanent and temporary loads included in the emergency connection;

– the obtained stressed-deformed state is taken as the starting one for calculating the load from the removed elements;

– The calculation of the additional load arising from the removal of elements is performed in a physically and geometrically nonlinear formulation. The load from the removal of elements corresponds to the forces obtained in them at the previous stage of the calculation and increased by a dynamic coefficient of 1.2. The verification of the bearing capacity of the remaining elements is performed without taking into account longitudinal bending.

Thus, the calculation of the bearing capacity and stability is carried out for an emergency combination of loads and effects, which includes permanent and long-term temporary loads, as well as the effect on the structure of the building of local hypothetical collapses. A local collapse can be located anywhere in the building.

The load is accepted in accordance with current regulatory documents.

The calculation of the building in the case of local collapse of the supporting structures is performed only according to the limit states of the first group. The displacement of structures and the opening of cracks in them in the considered emergency situation are not limited.

The model may take into account elements that are non-load-bearing under normal operating conditions (for example, hinged external wall panels, reinforced concrete balcony railings, etc.), and in the presence of local influences, take an active part in the redistribution of forces in the elements of the structural system. The design model of the house must take into account the possibility of removal (collapse) of individual vertical structural elements and be analyzed separately, taking into account each (one) of the local collapses.

In some cases, it is advisable to consider the operation of floors above a removed column (pylon, wall) with large deflections as a hanging reinforced concrete shell, taking into account membrane effects, which are due to the physical and geometric nonlinearity of its operation.

Each floor of a high-rise building must be designed to withstand the weight of the upper floor floor section (constant and long-term load with a dynamic coefficient $k_f=1.5$) on an area of 80 m^2.

An effective method of analyzing a structural system when calculating progressive collapse is to build its energy portrait. The algorithm of actions for this consists of the following procedures:

– construction, taking into account physical and/or geometric nonlinearity, of the limiting strain energy density (LSDED) field in the system elements;

– based on the analysis of the constructed field of the GShED, establishing elements with its maximum values;

– accepting established elements as “definitive”;

– performing calculations according to local calculation schemes that take into account the absence of "determining" elements;

– strength analysis of the results (calculation according to the first group of limit states) with an increase (if necessary) in the cross-sections of the reinforcement and the class of concrete.

The resistance of a high-rise building against progressive collapse should be ensured by the most economical means:

– rational structural and planning solution of the building, taking into account the possibility of the emergency situation in question;

– constructive measures that ensure the integrity of structures;

– using materials and design solutions that ensure the development of plastic deformations in structural elements and their connections;

– designing technical floors in the form of a spatial system - a box-section slab capable of absorbing loads caused by the removal of vertical elements located between technical floors [3.12].

The effective operation of ties that prevent progressive collapse is determined by ensuring their plasticity in the limit state so that they are not excluded from operation and allow the development of necessary deformations without collapse. To meet this requirement, ties must be designed from plastic sheet or reinforcing steel and the anchoring strength of the reinforcement must be greater than the force that causes its destruction.

Connections of prefabricated elements with monolithic structures that prevent the progressive collapse of buildings should be designed with unequal strength, with the element whose limit state provides the greatest plastic deformations of the connection should be the least strong.

To meet this condition, it is recommended to design the connection for a force that is 1.5 times greater than the load-bearing capacity of the elements being connected. It is necessary to pay special attention to the actual accurate implementation of the design solutions of the plastic elements.

To increase the effectiveness of resistance to progressive collapse of a building, it is recommended to:

– Over-opening lintels, which act as shear ties, should be designed so that they fail from bending, not from the action of transverse force;

– keyed joints in prefabricated monolithic structures should be designed so that the shear strength of individual keys is 1.5 times greater than their crushing strength;

– ensure sufficient anchoring length of reinforcement when it works as shear reinforcement;

– The supporting cross-sections of beams and cross-members, as well as the nodes of their connections with columns (walls, pylons), must have a transverse force strength 1.5 times higher than their bending load-bearing capacity in the span, taking into account plastic properties.

The minimum cross-sectional area of both longitudinal and transverse reinforcement in reinforced concrete floors and coverings is determined by calculation and must be at least 0.25% of the cross-sectional area of the concrete. In this case, the specified reinforcement must be continuous and joined in accordance with the requirements of current regulatory documents for the design of reinforced concrete structures.

Topic 8 ENSURING THE STABILITY OF HIGH-RISE BUILDINGS

The primary position in the design of monolithic buildings should be the procedure for selecting and assigning the load-bearing structure scheme: frame, mixed, frame-tie, tie. The choice of the scheme should be preceded by a technical and economic comparison of options, which is based on the analysis of the height of the structure, engineering conditions of the site (geology, hydrogeology, seismic responsibility), the level and composition of loads, the configuration of the building, technological features of the construction, etc. In this regard, it should be taken into account that the frame system is a system whose functioning, reliability and durability are ensured by the rigidity of the frame nodes. The influence of the floor disks here is secondary. A mixed system is a system whose functioning, reliability and durability are ensured in one direction by the rigidity of the frame nodes, and in the other by the tie system. A frame-tie system is a system whose functioning, reliability and durability are ensured by the frame nodes and ties. Communication system – the functioning, reliability and durability of which is ensured by a system of pylons (cores) and slab disks. Rigid (frame) node – connection of elements with full (100%) moment perception (Fig. 8.1).

A conditionally rigid node is a connection of elements that ensures the perception of a partial (specified) fraction of the moment (Fig. 8.2).

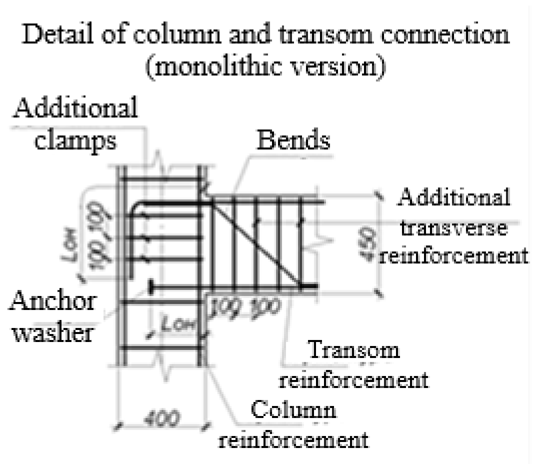


Figure 8.1. – Rigid (frame) node

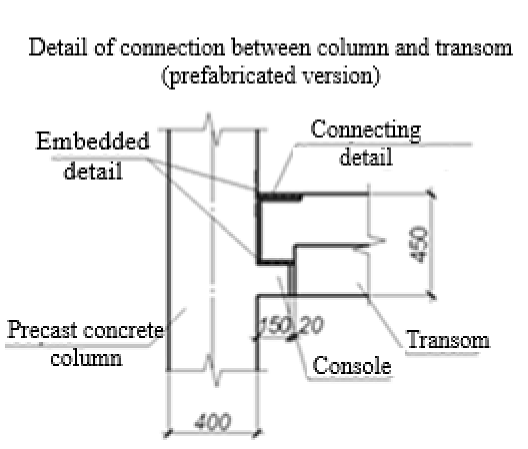


Figure 8.2. – Conditionally rigid node

External columns should be included in the work of the connection system to absorb horizontal loads. The inclusion is carried out through grillages (grillage outrigger floors) (Fig. 8.3).

It should be noted that the most rational and practical method of structural solutions that can reduce the probability of collapse of a structural system with the condition of local destruction of a separate element (when implementing a special case) is the addition of outrigger floors to the entire structure. The peculiarity of this structure is that it increases the overall rigidity of the building. This is achieved by placing spatial trusses on a certain floor that connect the stiffness core with the outer perimeter of the columns. For each building, the location of the "rigid" floor is determined separately depending on its architectural and geometric properties, but the most expedient is its location at 2/5–3/5 of the height of the entire building.

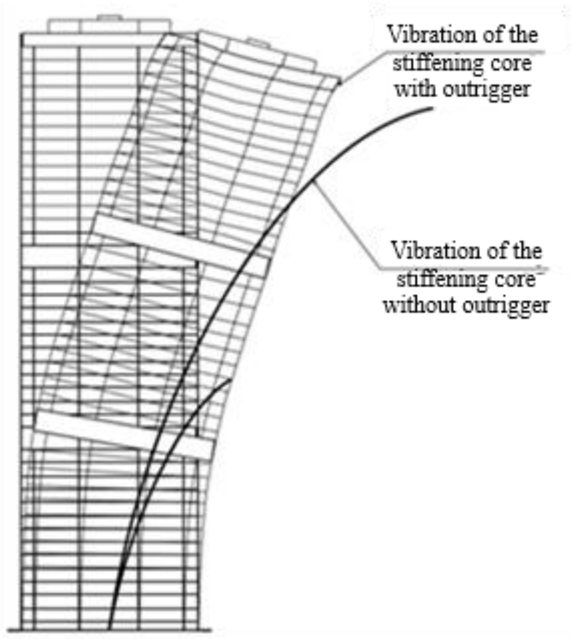


Figure 8.3 – Comparative diagram of the overall stiffness of a building with and without an outrigger floor

When comparing models of a multi-story building with and without a rigid floor, the overall stiffness of the building increases, which positively affects the ability of this building to contain a progressive collapse.

In this case, the columns receive only additional (vertical) loads. In addition, the inclusion of external columns in the operation of the general connection system for horizontal loads significantly facilitates the system and at the same time increases the overall rigidity of the building. It is advisable to use the lattice floors as technical and for the location of seismic and vibration damping systems within them.

Experience in operating high-rise buildings shows that seismic vibration damping is most expediently implemented by using dynamic vibration dampers (DVDs). As is known, the theoretical basis of their design is an interesting result obtained by integrating the equations of oscillations of systems with two degrees of freedom.

Let, for example, be a two-mass system (Fig. 8.4) experiencing the action of a disturbing force *Psinωt* (*P* is the amplitude of the force, *ω* is the frequency of oscillations).

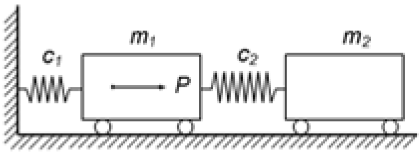


Figure 8.4 – Two-mass system

The equation of motion for this system can be written as follows:

$$\begin{cases} m_1\ddot{x}_1 + c_1 x_1 - c_2(x_2 - x_1) = P\sin\omega t, \\ m_2\ddot{x}_2 + c_2(x_2 - x_1) = 0, \end{cases} . \tag{8.1}$$

where m_1 and m_2 are the main and connecting masses;

c_1 and c_2 are the spring stiffness coefficients;

$\ddot{x}_1$ and $\ddot{x}_2$ –acceleration.

I took a private solution of the system in the form:

$$x_1 = a_1 \sin\omega t; \qquad x_2 = a_2 \sin\omega t; \tag{8.2}$$

and substituting its in (8.1), we obtain two equations with two unknown amplitudes of oscillations a_1 and a_2:

$$\begin{cases} -m_1\omega^2 a_1 + c_1 a_1 - c_2(a_2 - a_1) = P, \\ -m_2\omega^2 a_2 + c_2(a_2 - a_1) = 0, \end{cases} . \tag{8.3}$$

The solution (8.3) gives

$$a_1 = \frac{P\left(c_2 - m_2\omega^2\right)}{\left(c_1 + c_2 - m_1\omega^2\right)\left(c_2 - m_2\omega^2\right) - c_2^2};$$
$$a_1 = \frac{P\left(c_2 - m_2\omega^2\right)}{\left(c_1 + c_2 - m_1\omega^2\right)\left(c_2 - m_2\omega^2\right) - c_2^2}; \tag{8.4}$$

Analysis of (8.4) shows that in the special case when

$$c_2 - m_2\omega^2 = 0,$$
$$a_1 = 0; \qquad a_2 = -\frac{P}{c_2}; \tag{8.5}$$

The first mass remains stationary, although a disturbing force (antiresonance) is applied to it.

Thus, to damp the oscillations of the system under consideration, it is sufficient to attach an additional mass (*m_2*) to the main mass (*m_1*) on an elastic connection, subjecting the parameters of the attached mass to the condition. It should also be noted that in practical cases, damping elements are additionally introduced into the DHC to damp possible resonances that arise when the excitation frequency changes.

The above result justifies the main criteria used when selecting DHA. In particular:

– The most important parameter when choosing a dynamic damper is the optimal value of the damper frequency deviation (optimal tuning value) from the natural frequency of oscillations of the structure. If the damper frequency, which is used to determine the tuning value, is not optimally matched to the structure, small frequency changes can significantly reduce the efficiency of the damper;

– an important criterion is the optimal mass ratio coefficient, which ensures a wide range of effective operation of the damper;

– Damping is a less important criterion compared to the first two, but, nevertheless, allows the use of devices with large deviations from optimal parameters without a significant loss of efficiency.

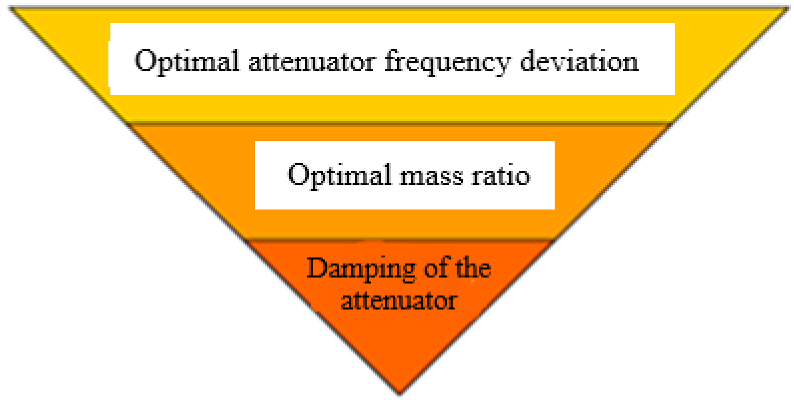


Figure 8.5 – Priorities of the three main criteria for selecting DHCs

For effective operation of dynamic dampers, the following conditions must be met when selecting them:

Choosing the optimal mass.

The ratio of the mass of the dynamic damper to the equivalent mass of the main system (μ) must be sufficient and justified. At small masses of the dynamic damper (at $\mu \leq 0.025$), the efficiency of the damper decreases. In addition, there are problems with the size of the gaps, which must provide large amplitudes of mass oscillations. Such dampers are more expensive, since they require the use of longer springs. At a mass ratio $\mu > 0.025$, the frequency range in which effective oscillation damping is increased.

Choosing the optimal oscillation frequency of the dynamic damper.

For optimal operation of the dynamic vibration damper, the following ratio between the partial frequencies must be observed:

$$k_{opt} = \frac{f_D}{f_H} \tag{8.6}$$

where f_D is the partial frequency of the mass of the dynamic damper;
f_H– the frequency of natural oscillations of the main system.

The coefficient k is called the tuning coefficient. These frequencies must be specially tuned. The value of the optimal ratio between the frequencies is determined by the inequality:

$$k_{opt} = \frac{1}{1+\mu} < 1 \tag{8.7}$$

Selection of optimal damper parameters.

For the effective operation of a dynamic damper, it is necessary to select a damper whose characteristics will correspond to the mass ratio μ and satisfy the following condition:

$$\xi_{D,optt} = \sqrt{\frac{3\mu}{8\cdot(1+\mu)^3}} \tag{8.8}$$

Correctly selected damping of the dynamic damper also allows to ensure the effective operation of the device. However, it should be noted once again that the deviation of the damping value from the optimal one has a lesser effect on the change in the efficiency of the vibration damper compared to the influence of the deviation from the optimal frequency ratio.

For effective practical implementation of vibration damping of high-rise buildings, the following two types of DHC can be used.

The MTMD-P dynamic damper is installed at the point of the structure corresponding to the maximum amplitude during horizontal vibrations of the structure with its natural frequency.

Fastening of the DHK to the beams or brackets of the building is carried out using bolts.

The MTMD-P consists of an oscillating mass attached to the end of a pendulum rod. The rotating force is created by the gravitational force of the pendulum mass when it is displaced from its equilibrium position.

Damping is provided by plates moving relative to each other or by viscous damping devices.

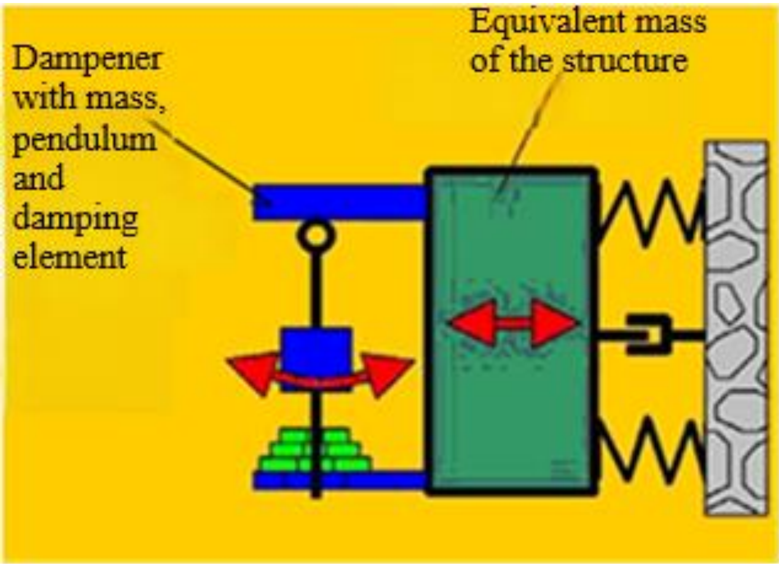


Figure 8.6 – Principle of operation of the MTMD-P damper

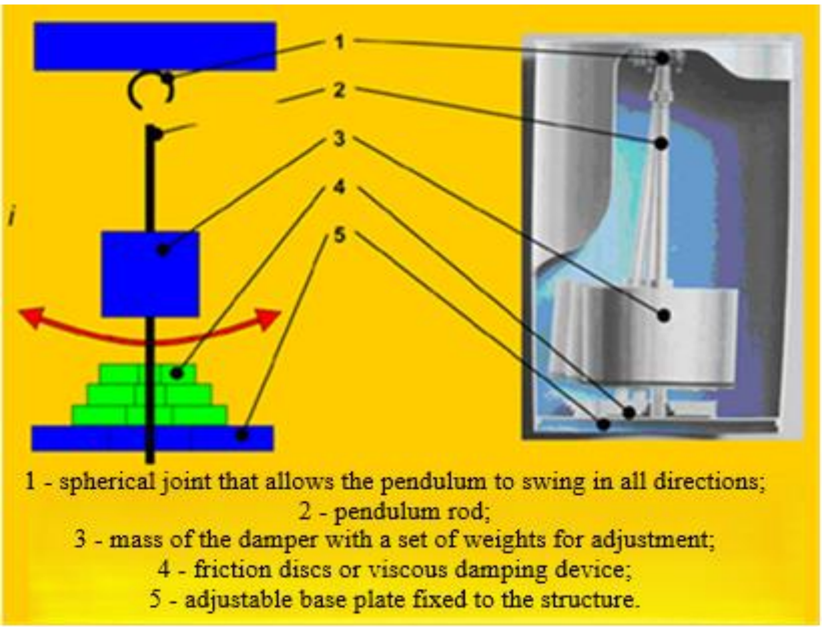


Figure 8.7 – Description of MTMD-P

MTMD-H: Dynamic vibration damper, oscillating horizontally (Fig. 8.8, 8.9).

The MTMD-H dynamic damper is installed at the point of the structure corresponding to the maximum amplitude during horizontal vibrations of the structure with its natural frequency.

Fastening of the DHK to the beams or brackets of the building is carried out using bolts.

The MTMD-H consists of a horizontally guided, moving mass attached to steel springs. A damping element is attached parallel to the springs.

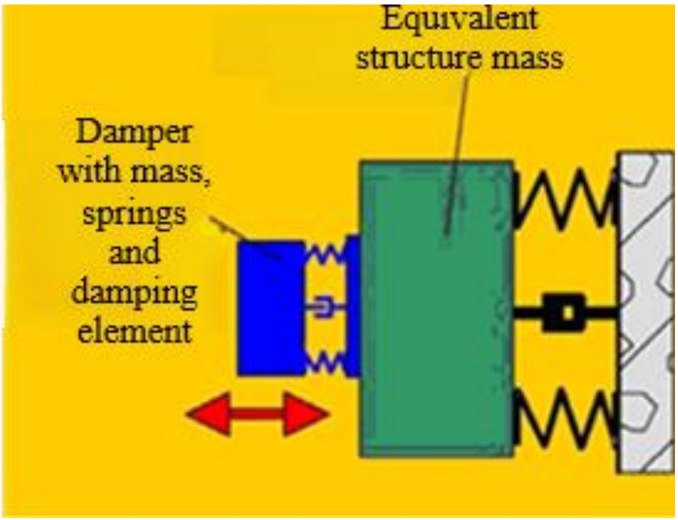


Figure 8.8 – Principle of operation of the MTMD-N damper

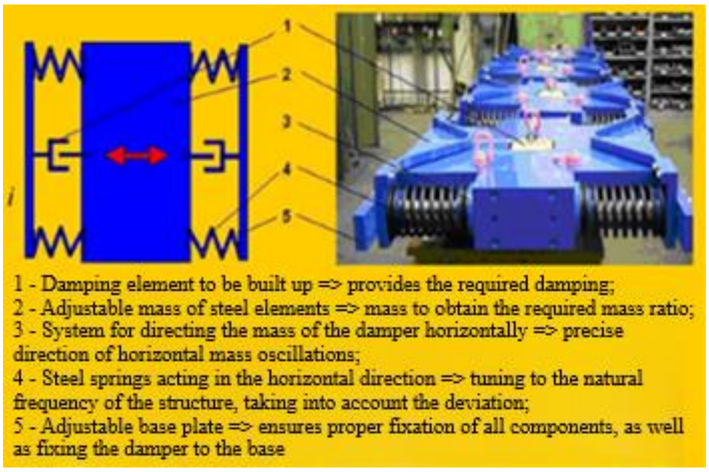


Figure 8.9 – Description of MTMD-N

The schematic arrangement of the mentioned DHCs within the outrigger (grill) floors is shown in Figures 8.10 and 8.11.

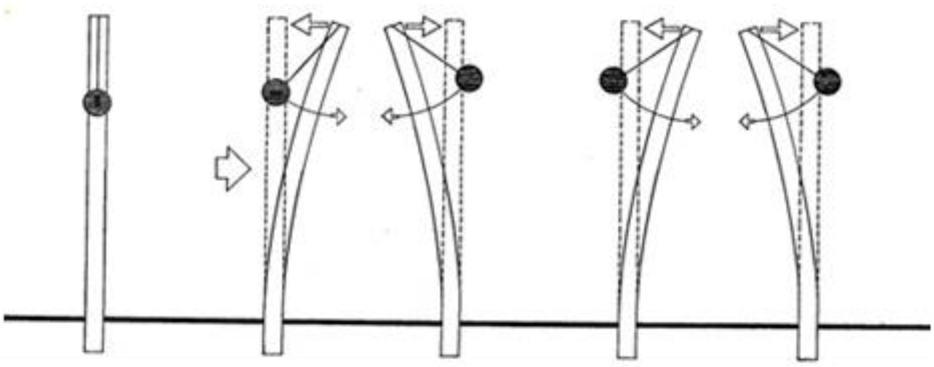


Figure 8.10 – Pendulum vibration damping system, which is advisable to be located in a grid floor

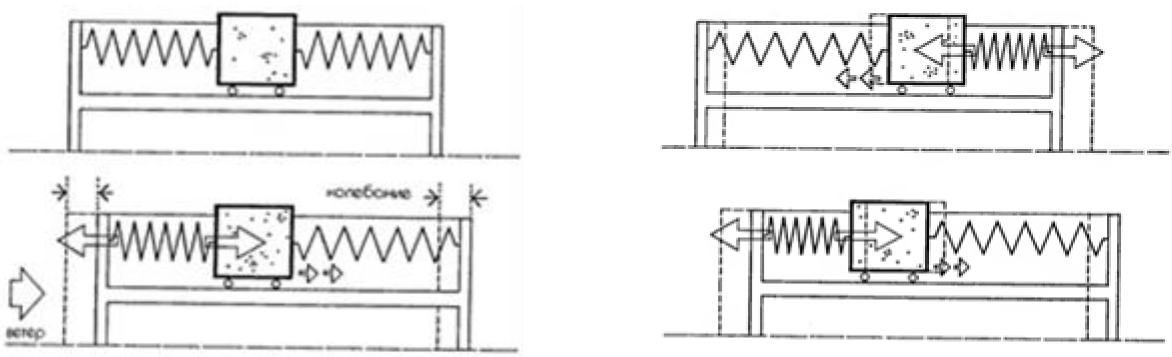


Figure 8.11 – Horizontal vibration damping system, which is advisable to be located in grid floors

Slab disks (especially for communication systems) must be absolutely rigid (non-deformable) in their plane. Their connection points with pylons must ensure full (100%) transmission of all components of the forces arising from the disk to the pylon.

For spans up to 6–8 m, it is recommended to make the floors flat, for larger spans – flat with capitals or inter-column beams and walls, and for spans up to 12 m – with inter-column beams or walls and ribbed and hollow core slabs.

For hall spaces with a span of 12–15 m, caisson, ribbed, or hollow slabs are recommended when supported on four sides by beams and walls.

The main design parameters of flat floor slabs are the cross-sectional dimensions (slab thickness), the concrete compressive strength class, and the amount of longitudinal reinforcement, which are determined depending on the load on the floor and the length of the spans.

When designing, it is recommended to take the optimal structural parameters of the floors, determined on the basis of a technical and economic analysis. In this case, the thickness of flat slabs of solid cross-section is recommended to be taken not less than 1/30 of the length of the largest span and not more than 25 cm, the concrete class is not less than C20/25. The height of hollow, ribbed and caisson slabs is taken not less than 25 cm and not more than 50 cm, the concrete class is not less than C20/25.

In flat floor slabs in densely reinforced areas around columns, where maximum transverse bending forces and torques act, it is recommended to lay fiber-reinforced concrete with a tensile strength class of at least C20/25 to prevent punching, simplify reinforcement, and facilitate concreting.

The main design parameters of flat foundation slabs are dimensions (slab thickness), concrete compressive strength class, and the amount of longitudinal reinforcement, which are determined depending on the reactive pressure of the foundation soil and the spacing of columns and walls.

When designing, it is recommended to take the optimal design parameters of the foundation slabs, determined on the basis of a feasibility study. In this case, the thickness of the foundation slabs is recommended to be at least 60 cm (the thickness of one of the slabs for box and caisson foundations) and not more than 200 cm, the concrete class is at least C16/20, the reinforcement is at least 0.3%, and the waterproofness grade is at least W6.

Ribbed and box foundations consist of slab and wall elements and are used to increase the rigidity of the building, and at a height of more than 2 m, also to use the underground space as technical floors.

Topic 9 ENSURING THE STABILITY OF HIGH-RISE BUILDINGS

The primary position in the design of monolithic buildings should be the procedure for selecting and assigning the scheme of load-bearing structures: frame, mixed, frame-connecting, connecting. The choice of the scheme should be preceded by a technical and economic comparison of options, which is based on the analysis of the height of the structure, engineering conditions of the site (geology, hydrogeology, seismic responsibility), the level and composition of loads, the configuration of the building, technological features of construction, etc. In this regard, it should be taken into account that the frame system is a system whose functioning, reliability and durability are ensured by the rigidity of the frame units.

9.1 The impact of seismic vibrations on building structures

High-rise buildings can be divided into four main groups by the type of building material of the load-bearing structures: reinforced concrete, wooden, masonry, metal. The most important characteristics of materials that determine their resistance to seismic loads are the design resistance and the ratio of strength to specific gravity of the material. The higher the design resistance, the better the material resists seismic effects. The nature of the destruction of buildings and their structural elements made of different materials is also influenced by other factors - chipping strength, adhesion strength, flexibility. According to the specified characteristics, metal buildings are distinguished by the greatest resistance in the event of seismic effects, wooden and reinforced concrete buildings are slightly less resistant, and masonry buildings are the least resistant.

The strength of building materials and structures depends not only on physical properties, but is largely determined by the conditions in which they are under operational loads. In earthquake conditions, the strength characteristics of materials will naturally be determined to a greater extent by the characteristics of the seismic load itself.

One of these features, characteristic of any earthquake, is the short duration of the load action, i.e. the relatively small number of cycles of its repetition. Another factor of great importance for the performance of building structures and materials is the frequency of the load.

Since steel structures are most often used in the form of thin rods (columns, beams, ties), and steel has high tensile and compressive strength with the highest specific gravity of all materials, the destruction of metal structures during earthquakes occurs from the loss of stability. The overall stability of metal structures can be increased by changing the fastening of ties on the supports, or by installing additional ties along the length, reducing the load by reducing the pitch of columns, girders, beams and trusses. The local stability of metal profiles is increased by closed sections (pipe, profile pipe). It is very important to use ductile steels that have a relative elongation at break of more than 20%. In structures made of such steels, most of the energy is spent on the formation of plastic hinges, which preserves the integrity of the building as a whole.

When considering the bearing capacity of structures and materials, it should be borne in mind that a strong earthquake is a relatively rare phenomenon, so ensuring the operational fullness of objects after an earthquake may be economically impractical, since the service life of such buildings may be less than the period of recurrence of strong earthquakes. Therefore, in earthquake-resistant construction, there is no requirement to ensure the full preservation and suitability for further operation of buildings that have been subjected to seismic loads; the main thing is to ensure the safety of people and the preservation of the most valuable property. Such a requirement requires the concept of the limit state of the structure.

9.2 Basic principles of designing high-rise earthquake-resistant buildings

Seismic resistance – the ability of buildings and structureswithstand earthquakes with minimal damage.

The seismic resistance of an object, first of all, depends on its height, its weight as a whole, the structural system that takes on the seismic action, the seismic regions where the object is being built, including microseismic regionalization, since in areas of low seismic activity there may be geological faults that can pose an increased geodynamic hazard to individual objects, especially high-rise buildings.

Traditional methods and means of protecting buildings and structures from seismic effects include a large set of various measures aimed at increasing the bearing capacity of building structures, the design of which is carried out on the basis of norms and rules developed by domestic and foreign construction experience.

The design of buildings and structures in seismically hazardous areas begins with adherence to the general principles of earthquake-resistant construction:

1. Principle of symmetry: the mass and rigidity of the structure must be distributed evenly and symmetrically relative to the plane of symmetry passing through the center of gravity of the structure, which is achieved by the volumetric planning solutions of buildings (Fig. 9.1).

a) b)

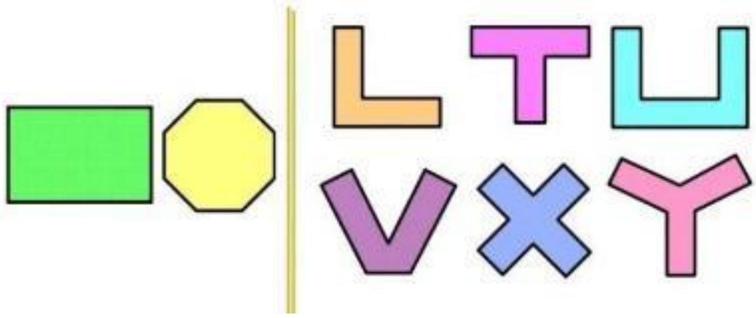


Figure 9.1 – Examples of spatial planning solutions for buildings:
a – simple; b – complex

Structural asymmetry of buildings leads to a decrease in their seismic resistance, which can cause local destruction or complete collapse. With an asymmetric shape, it is necessary to limit the size of protrusions, and also try not to use structural solutions that are elongated in plan.

2. The principle of harmony: necessary to observe proportionality in the dimensions of the building, while its length or height should not be excessively large. The maximum dimensions, number of floors, and height of floors of buildings are adopted in accordance with the requirements.

3. Anti-gravity principle: the mass of the structure has a significant impact on the value of seismic loads. Therefore, when exposed to seismic forces, it is necessary to strive for the maximum possible reduction in the weight of structures and the resulting loads.

4. Principle of elasticity: it is desirable to use materials in the structure that are strong, light, and have elastic properties; structures made of them should have uniform properties.

5. Closed loop principle: the load-bearing elements of the structure must be interconnected, forming closed loops in both the vertical and horizontal directions.

The design of such circuits should ensure the independent operation of each of them during seismic oscillations. This is achieved by means of anti-seismic joints, which, for the purpose of economy, should be combined with temperature and sedimentary joints.

Anti-seismic joints are arranged in buildings if, from the point of view of functional and architectural-planning considerations, it is impossible to avoid a complex and asymmetrical shape of the building in plan with height differences of 5 m or more. Anti-seismic joints in buildings with a wall structural system are arranged in the form of double load-bearing walls, and in frame buildings - by placing double frames.

6. Application of seismic isolation: it is advisable to use devices that reduce the intensity of vibration processes transmitted from the soil to the building.

7. Ensuring the reliability of foundations: for earthquake-resistant structures, foundations must be strong, deep enough laid, preferably on pliable layers or special substructures that replace weak soils, to ensure the homogeneity and strength of the soil base.

9.3 Seismic protection systems for high-rise buildings

In buildings with frame structural schemes, it is possible to distinguish three seismic protection systems: passive, active, and comprehensive.

Passive system provides for:

- selection of spatial planning solutions and building configurations, as a rule, with symmetrical structural schemes;
- design of anti-seismic closed compartments of simple shape;
- uniform distribution of structural stiffness and their weight;
- ensuring the strength of load-bearing structures without weakened nodes;
- installation of stiffening diaphragms at the levels of floors and coverings, for uniform distribution of seismic loads between vertical structures (in multi-story buildings).

Active system provides for:

- reduction of seismic loads by adjusting their dynamic parameters to prevent resonant increase in building vibration amplitudes, reduction of resonant effects;
- change in dynamic stiffness or periods of natural oscillations of buildings during earthquakes. In this case, special structural solutions are used: elms (Fig. 9.2), sliding belts, vibration dampers, kinematic foundations, pile foundations that have dissipative characteristics of self-organization, in frame-linkage systems, the assembly of stiffness diaphragms, rubber-steel cylindrical supports, etc.

a) b) c) d) e) f)

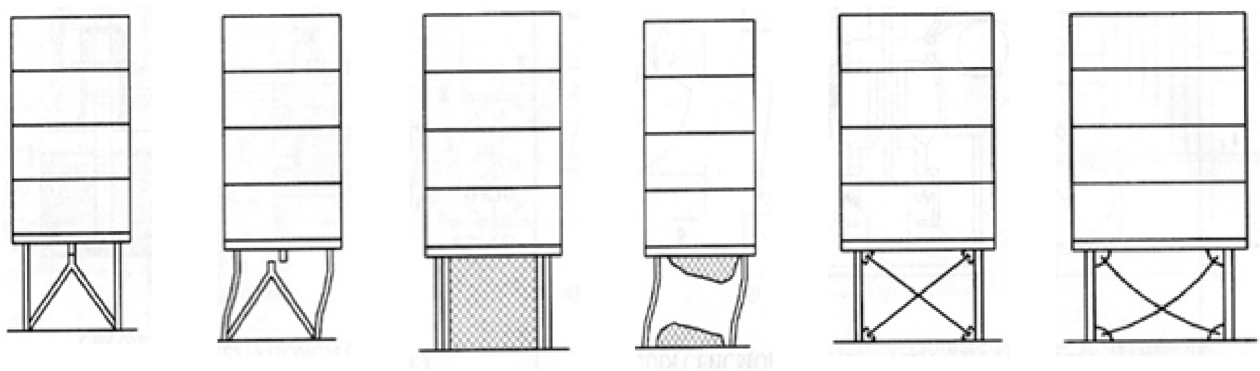


Figure 9.2. – Active seismic protection system using disconnectable ties:
a – special disconnectable elements; b – collapsible connecting panels; c – elastic ties; d – sagging struts; e – stops limiting deformations; f – rigid panels

The main requirement for the effective operation of active seismic protection systems is the divergence of the frequencies of their natural oscillations from the dominant frequencies of seismic mobility o the soil foundations of the building.

Complex system combines the application of passive and active seismic protection of buildings.

9.4 Promising directions of seismic protection of high-rise buildings

Effective seismic protection is provided by damping structures (Fig. 9.3), which can be implemented during repair work.

Currently, in world practice, there is a trend to provide seismic protection of frame buildings on the principle of seismic isolation of the object - damping (damping of vibrations). Damping of buildings has been known for a very long time and is constantly being improved using calculation methods, new design solutions, developments and modifications (Fig. 9.4), the use of special alloys and composite materials with memory. This method of seismic protection, although relatively expensive, is the most reliable and effective.

According to the recommendations of regulatory documents, calculations of structures for emergency combination of loads, taking into account seismic effects, should be performed using:

- spectral method;
- direct dynamic method using instrumental recordings of ground accelerations during earthquakes or a set of synthesized accelerograms;
- nonlinear static calculation, which is used when it is necessary to take into account the nonlinear response of structures as an alternative to nonlinear dynamic calculation.

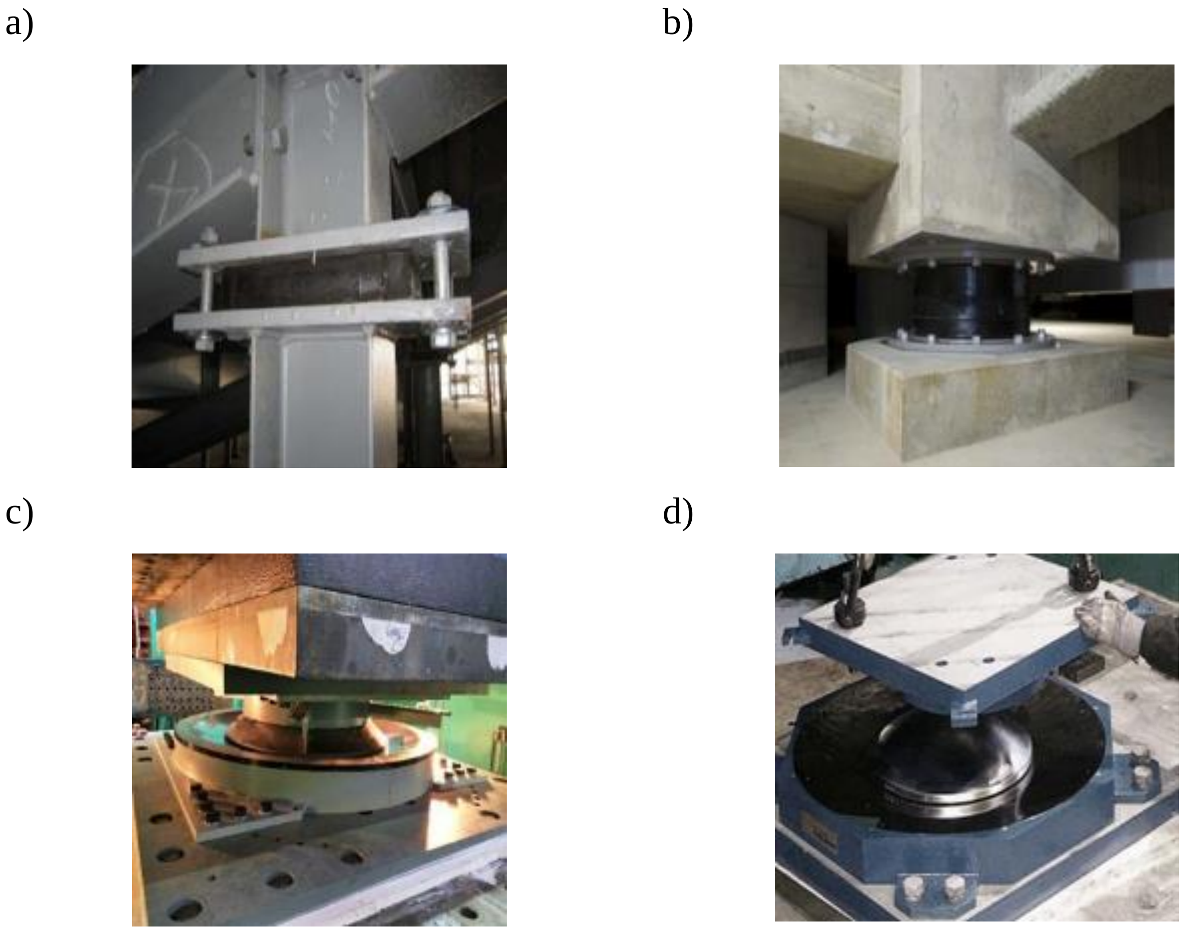


Figure 9.4. – Seismic-resistant supports: a – rubber-metal; b – friction;
c – sliding; d – pendulum type

Nonlinear static calculation is recommended in cases:

- as an alternative to the direct dynamic method using an accelerogram package, given the possible complexity and cumbersomeness of such calculations, as well as due to the significant uncertainty of the initial data;
- when designing buildings and structures using a methodology based on studying the state of structures at different levels of seismic impact (the so-called Performance - Based Seismic Design);
- when assessing and restoring the seismic resistance of buildings in operation, taking into account their actual technical condition (defects, damage, etc.).

Nonlinear static analysis is a tool for assessing the bearing capacity of structures. It involves monotonically loading nonlinear multi-mass calculation models with a set of distributed seismic horizontal forces until certain displacement limits are reached at a selected level. The BRM can be loaded until failure in order to assess its ultimate deformations and bearing capacity.

Nonlinear static calculation is the first stage of a two-stage procedure, as a result of which a curve (spectrum) of the bearing capacity of the BRM is obtained. The bearing capacity spectrum is the ratio of the base displacement under seismic action to the horizontal response (displacement) of the building. The bearing capacity spectrum is constructed in the coordinates "spectral acceleration - spectral displacement" using the dependencies "restoring force - displacement" for each level along the height of the building. The obtained spectrum is used to determine the displacements of the equivalent one-mass system (EOMS) under a certain seismic action by means of a nonlinear dynamic calculation of the EOMS. Thus, the need for a nonlinear dynamic calculation of the original BRM is avoided. In addition, the compliance coefficients, as well as the reduction coefficients (reduction of seismic response) of buildings under seismic actions are determined based on the spectrum.

The methodology using the spectrum is reflected in the regulatory documents of the USA and DSTU-N B EN 1998-1, and is also implemented in common calculation complexes.

Modern methods of performing nonlinear static calculations allow us to take into account the asymmetry of the object in plan and in height, the influence of higher forms of oscillations for high-rise buildings and long structures, and the interaction in the "base - foundation - building frame" system. With the development of computer modeling, it has become possible to analyze the behavior of a building in its overall volumetric and spatial structure.

Topic 10 INNOVATIVE TECHNOLOGIES OF HIGH-RISE HOUSING CONSTRUCTION

10.1 Evolution of architectural and technological solutions for high-rise buildings

Analyzing the development trends of high-rise buildings of the late 20th - early 21st centuries, it is possible to distinguish four main directions of their architectural and stylistic solutions:

1. Structural expressionism - detailing of facades by revealing structural elements of the load-bearing structures of the external frame of buildings, metal diagonal lattices;

2. "Digital" architecture - characterized by the creation of forms of elevational volumes that go beyond the Cartesian coordinate system and do not have an isometric description;

3. Bioecological architecture - ensures the harmonization of the architectural envelope of buildings with the local climatic features of the territory of their location, the use of the energy capabilities of the external climate for energy supply, the use of natural lighting of premises, the introduction of energy-efficient technologies and engineering and technical support;

4. "Kinematic" architecture - involves the constant movement of buildings in space, the development and implementation of environmental and energy-efficient solutions.

Since the mid-20th century, architects and engineers have been developing scientific and methodological foundations for the design and construction of buildings that improve the artificial environment for human activity.

Energy consumption of high-rise buildings is highlighted as the main criterion for assessing the quality of the project, given that "traditional" buildings are endowed with large reserves of increasing thermal efficiency. In high-rise building projects, the main attention is paid to energy consumption and energy conservation measures. The main attention is paid to energy efficiency of high-rise buildings. Energy efficiency is understood as the ability of a building and its engineering networks to provide a given level of energy consumption to maintain optimal parameters of the internal microclimate in the premises of the facility, according to its functional purpose. At the same time, the optimal parameters of the internal microclimate are a combination of indicators that, with prolonged and systematic exposure, ensure the normal psychophysiological state of a person in the premises. The current period of development of high-rise construction is characterized by the emergence of energy-efficient facilities.

Currently, the construction of skyscrapers is focused on increasing the number of floors, renovation or complete energy independence of high-rise buildings and the use of ecological objects, the use of environmentally friendly, safe materials. In these areas, designers resort to the use of composite materials. As a rule, such materials reduce the total weight of buildings by an average of 10%, significantly increase the thermal insulation properties of structures, and due to the use of new technological solutions, accelerate the construction technology.

Today, such advanced technologies are used in Asian countries. An example of this is the alarming systems with increased stability of high-rise structures, which is due to the high probability of natural disaster factors coming into effect.

For example, during the construction of the 88-story Jin Mao Tower skyscraper, 421 m high, located in Shanghai, innovative solutions were used to increase seismic protection. The building is divided into 16 segments, each segment is one-eighth shorter than the sixteen-story base. The central concrete frame has 8 corners, it is surrounded by 8 composite columns and 8 external steel columns. This creates a postmodern form, on the upper floors the building becomes more complicated and slightly narrows, thereby creating a rhythmic pattern characteristic of Chinese architecture. According to experts, it can maintain the integrity of its structures at wind speeds of more than 200 km/h, as well as withstand earthquakes with a power of up to 7 points. This is ensured by the implementation of movable joints inside the supporting steel columns (Fig. 10.1).

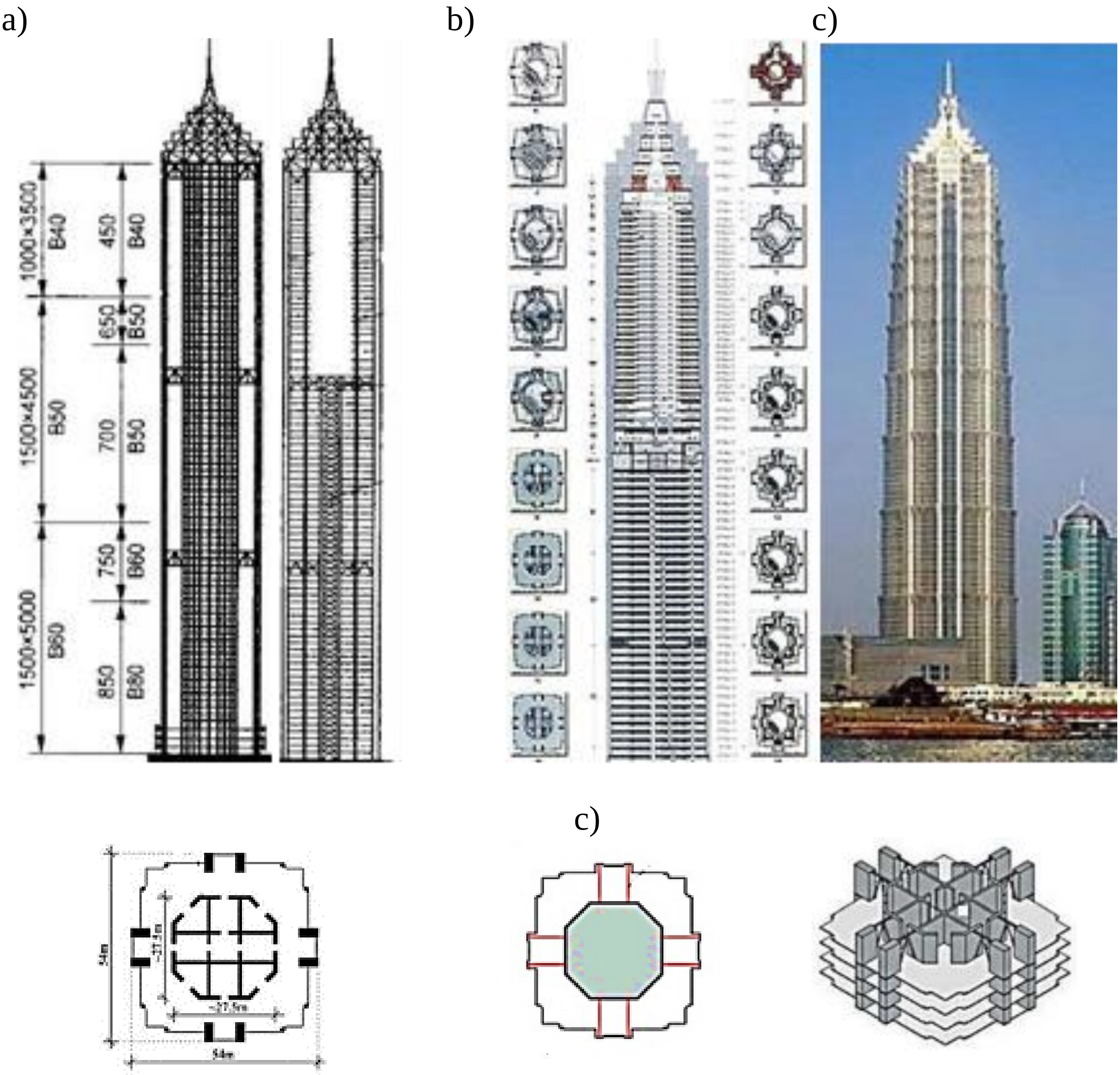


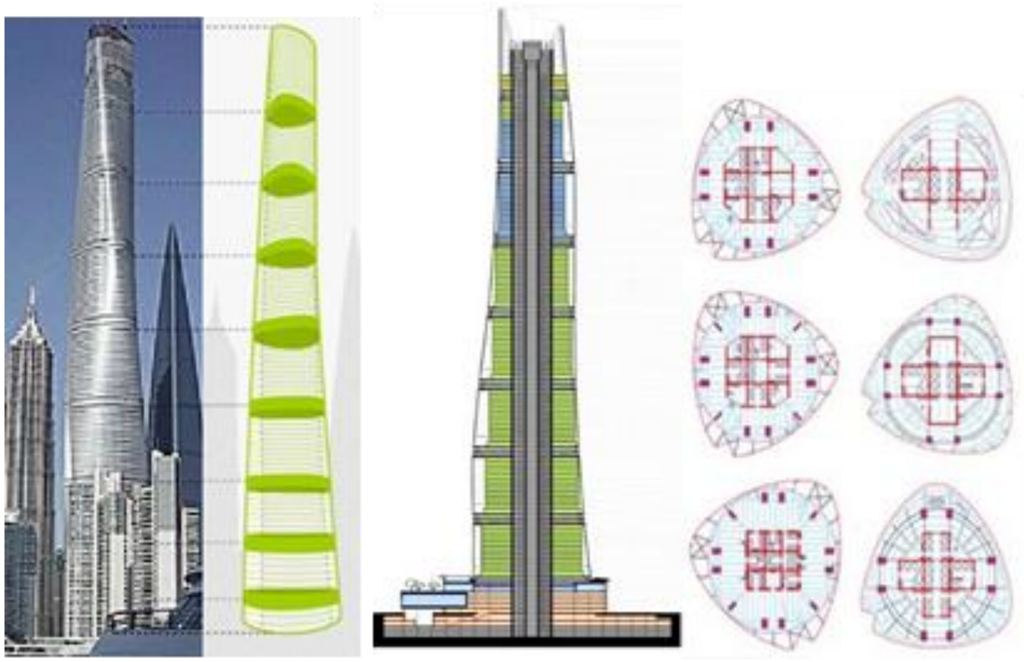
Figure 10.1 – Jin Mao Tower: a – constructive solution;
b – general view of the building; c – design of the moving joints of the skyscraper

An equally striking example of innovative design solutions is the Shanghai Tower in Shanghai, Pudong district. The 128-story, 632 m high super skyscraper was designed by the American architectural firm Gensler (Fig. 10.2).

The skyscraper consists of nine cylindrical buildings (Fig. 10.2, a, b), which are built one on top of the other and have a second double "covering" of triangular shape on the outside, which creates a rotating shape of the inner part of the building

(Fig. 10.2, c).Double glazing improves the thermal insulation properties of the building and ensures the transmission of a lot of natural light, reducing energy costs for lighting and heating the premises.(Fig. 10.2, d)The tapered shape of the tower reduces wind loads on the building by almost 25%.The presence of a swimming pool located on the 57th floor of the skyscraper has a huge impact on ensuring the stability of the structure, which allows the building to balance in space under the influence of wind and seismic loads.

a) b)



c) d)

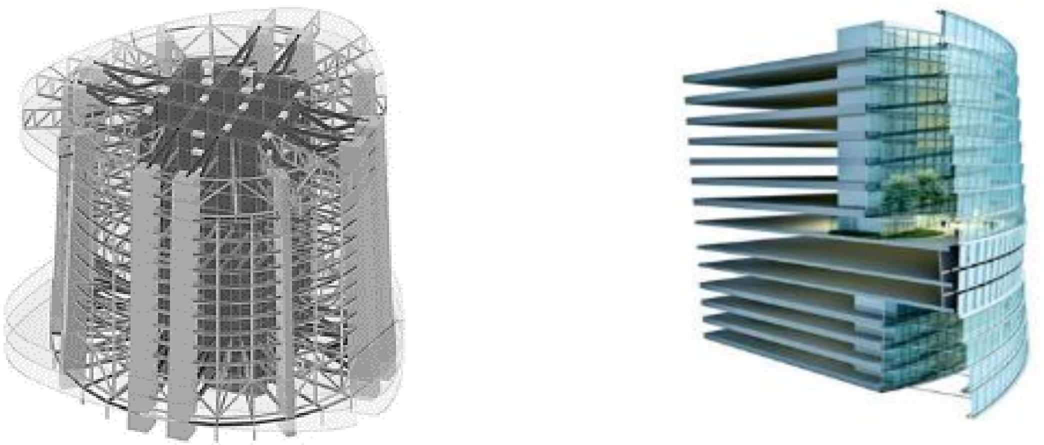


Figure 10.2 – Super skyscraper “Shanghai Tower”:

a – general view of the building; b – constructive solution;

c – a constructive solution that gives the shape of rotation;

d – construction double glazing on the building facade

The building is designed nine green garden areas, and 270 wind turbines installed on the facade of the building.

Not the least of which is the increased concern for the environment when constructing high-rise buildings. Modern skyscrapers are increasingly playing the role of air filters, which remove greenhouse gases and other harmful substances from the air space. A striking example is the building Bank of America (English: Bank of America), located on Manhattan Island in New York. The building project, developed by the firm “Severud Associates”, was created with the aim of creating the greatest environmental friendliness. The 54-story skyscraper with a height of 65 m is provided with rainwater collection and purification system. The wall structure has cleaning systems that are capable of removing polluted air and returning it back into the space in a purified form. The tower has a power plant that can produce 4.6 MW of electricity (Fig. 10.3).

Figure 10.3 – Bank of America skyscraper

10.2 Concepts of ecological skyscrapers

People always perceive high-rise buildings as non-ecological objects of the natural environment. In their construction, artificial materials are used that do not contribute to the naturalness and balance of their coexistence with the natural environment. In addition, a lot of energy resources are spent on supporting the functions of traditional high-rise buildings - skyscrapers, which in turn has a certain environmental impact on the environment. In these conditions, architects are faced with the task of creating such skyscrapers that should concisely fit into the environment and not cause environmental changes. In this direction, architects, designers and engineers around the world are developing various concepts. One of such concepts is copying nature and renewing the energy consumed by the high-rise object itself.

Nothing in nature is wasted. Therefore, in high-rise buildings it is necessary to use renewable materials and consume as little non-renewable energy as possible. This does not mean at all that you can install a wind turbine above a high-rise building and thus solve the issue of environmental friendliness. Fundamental architectural solutions have gone further and provide, as the main condition, the organic integration of the building into the environment.

The World Building Council (WGBC), a global organization promoting green building, reports that 65% of electricity in the United States and 40% in Europe is consumed by high-rise buildings. The global construction market is growing at 3% annually, while green building is increasing by 30% annually. It is also noted that green towers are economical in terms of final energy, water, waste disposal, and more.

A survey of 170 existing eco-homes in the US found that their construction was 1.5% more expensive than traditional ones. But the savings in operation significantly outweighed the initial costs. So, if the decision that the investment should be environmentally friendly is made at the design concept stage, the experience of the contractors will significantly reduce the initial costs and you will not have to pay more than for conventional construction.

The idea of creating ecological “green” skyscrapers capable of providing themselves with electricity is not new, but for the most part there are only projects. The “EDITT Tower” project for Singapore deserves special attention. The author of the project, Ken Ueang, is transforming the cityscape of Singapore into a blooming oasis (Fig. 10.4).

According to the project, half of the surface of this skyscraper will be covered with plants, and the 26-story tower will be transformed into green terraces. Inside the building there are open spaces and greenhouses. But the “environmental friendliness” of the building does not end there. Since Singapore is known for its downpours, the building will collect rainwater that will be used for watering plants and for technical purposes (for example, for flushing toilets). In addition, the building will be equipped with 855 sq. meters of photovoltaic panels, which will provide almost 40% of electricity needs. The plans include the conversion of waste and wastewater into fertilizers and biofuels.

Figure 10.4 – Ecological skyscraper project in Singapore

The project of the Dutch design firm, which designed buildings in the form of green mounds for South Korea, deserves attention - they should fit into the local hills away from the smog of Seoul. The upper floors will be residential, and the lower floors will be schools, offices, shops. The eco-city provides for the residence of 77 thousand people. The buildings without corners are surrounded by terraces with living plants and a waterfall (Fig. 10.5).

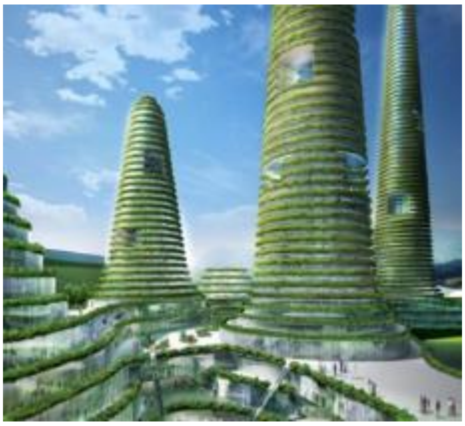



Figure 10.5 – Eco-city Project in Seoul

It is worth noting, from the point of view of aesthetic pleasure, a 12-story building in Australia together with a two-hectare field for growing vegetarian food (Fig. 10.6.). The Danish project provides for complexes with the possibility of moving each room measuring 7.8 by 7.8 meters. This size is optimal for a separate office or apartment - this is exactly what the building was designed for. All this is the result of innovative ideas of architects, designers and engineers of the conceptual development of ecological skyscrapers.


Figure 10.6 – Danish ecological skyscraper project

Construction of the luxurious eco-friendly Royal Atlantis hotel has begun in Dubai, United Arab Emirates. According to its creators, the skyscraper should become a “standard of environmental friendliness” and the hotel a “standard of luxury”. This glass palace will have 800 rooms and 250 apartments. The construction cost is $1.4 billion, opening in 2018 (Fig. 10.7).

Figure 10.7 – General view of the Environmental Project of the Royal Atlantis hotel

10.3 Concepts of ecological artificial skyscraper cities

A new conceptual “high-rise city” project, developed by French architectural firm VCA, is likely to be built in the Chinese city of Shenzhen, one of the fastest growing cities in the world.

The entire complex will consist of six buildings and, together with the surrounding vegetation, will occupy a little more than 19,000 square meters. Green plants will grow not only around, but inside the skyscrapers themselves and on special platforms outside.

Skyscrapers consist of a central part, on which, if separate blocks were strung together – “farmscraper” (Fig. 10.8).

Figure 10.8 – General view of the "High-rise City" project

In general, the buildings resemble pyramids of pebbles, usually laid out on the seashore. Despite the fact that there are no specific accents in the design, the overall complex of six buildings looks holistic and innovative. The glazed greenhouses are pleasing to the eye.

“Stones” will be made of metal structures, this technology will allow to form a complex, natural form. Inside the unusual skyscrapers will be located all the necessary facilities. Moreover, each "stone" will satisfy one need. One block will house offices, another – apartments, a third - shops and so on, underground – huge car parks. Inside each department will be formed a green zone with full-fledged plants, which will contribute to the improvement of the overall ecological situation in the city (Fig. 10.9).

Figure 10.9 – Spatial planning structure of the "High-rise City" project

The unusual concept of the “High-rise City” is energy-independent. Wherever possible, without observing the laws of symmetry, wind turbines are installed on the outside of the “pebble”, which will complement the solar panels and photovoltaic cells. It remains to rejoice for the residents of Shenzhen, and in principle for all humanity, that such living and real skyscrapers will soon appear on our planet.

The project “Lilypad”, by architect Vincent Callebaut from Belgium, “City on Water” falls into the category that can only be dreamed of, and for the courage in the fictional, only applauded. In fact, this is an artificial city of houses that have become underwater as a result of climate change. Such a city can accommodate 50,000 residents and is able to sail around the world like an independent and completely autonomous luxury super ship. On board are artificial lakes, artificial mountain landscapes, real solar panels, wind generators and installations that use wave energy - all for the production of electricity. There are no roads, no cars, no dirty production. “Lilypad” is the cleanest green metropolis of the future slightly flooded world (Fig. 10.10).

Figure 10.10 – The project “City on Water”

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