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A. V. Naboka

ENGINEERING STRUCTURES

LECTURE NOTES

(for students of the second (master's) level of higher education all forms of education speciality G19 – Building and Civil Engineering, education program "Industrial and civil construction")

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CONTENT

INTRODUCTION

In addition to building for production, housing, administrative, and cultural and public purposes, engineering structures are also located on the territories of industrial and civil construction projects. They are designed to ensure transport, loading and unloading, warehousing and production operations (railway and road overpasses at warehouses for raw materials, semi-finished products, finished goods, wagon dumpers; indoor and outdoor transport galleries, bridges, overpasses, aqueducts); movement of human flows (underground and over ground crossings); supply of facilities with electricity (transformer substations), water (reservoirs, water towers, pumping stations, cooling towers), compressed air (compressor units), heat (heating units), gas (gas holders and gas blowers); collection and treatment of wastewater (sedimentation tanks, filters, pump aeration tanks); landscaping (retaining walls); storage of bulk materials (silos, bunkers, warehouses, elevators).

The most complex engineering structures are usually included in special complexes of transport, energy, hydraulic engineering, urban construction (subways, dams, locks, pipes, canals, tunnels, radio and television towers, sports facilities, etc.).

It should be noted that no civil structure or industrial building can function without associated engineering structures. Therefore, information about these structures, their design solutions, and calculations are of a great importance part in the training of general civil engineers.

Less complex engineering structures are included in industrial construction projects. The most important of these are water tanks (round and rectangular in plan) and similar structures. They include capacitive treatment facilities for sewerage and water supply systems, water towers; silos and elevators (large containers for storing bulk materials); bunkers (small containers for receiving and storing bulk materials, devices for loading and unloading operations by gravity);

retaining walls (in raw material warehouses, for landscaping); underground structures at production sites (canals, tunnels, pipes for utility networks, shallow structures), subways, radio and television towers.

These lecture notes examine reinforced concrete structures of engineering structures of this type; the structures of particularly complex engineering structures in transportation, energy, hydraulic engineering, urban development, and sports construction are studied in separate, specialized courses.

The purpose of the material presented in this manual is to familiarize readers with the specifics of the layout, calculation and design of the most common engineering structures, skillset is necessary to determine the design schemes of these structures, to be guided by the functional purpose of the complexes under consideration with the prospect of their further use in design activities or construction and installation practice.

TOPIC 1 RESERVOIRS

1.1 General information and unification of parameters

The term “reservoirs” encompasses a broad category of engineering structures designed for storing or processing various liquid, gaseous and powdery materials.

Storage tanks are used in various industries, including transportation, agriculture, and other fields, but are particularly important in the water and sewer systems of cities and industrial plants, as well as in the oil, oil refining, and food industries. A number of sanitary structures, such as digesters, settling tanks, and others, as well as cooling ponds or swimming pools, differ from conventional storage tanks only in their intended purpose and technological equipment. Structurally, these structures have much in common. This tutorial focuses on reinforced concrete storage tanks.

Their main advantages include: high durability, corrosion resistance, the ability to be installed underground, including in high groundwater conditions; a smaller development area, as well as shorter utility lines; comparatively low operating costs; and the ability to use industrial construction methods.

Based on their characteristics and areas of application, tanks are classified into different types. The most commonly used classification, used in design, construction, and operation, can be divided into the following types:

1) by the content of the internal substance and purpose;

2) by plan shape and section shape;

3) by vertical reference of the tank bottom to the ground surface level;

4) by design features;

5) by construction method and operating conditions.

Based on the content of the internal substance, tanks are classified as those for storing cold or hot water, salt solutions, household and sewage waste, oil and petroleum products, fuel oil and oils; in the food industry, they are used for storing processed vegetable products, grapes, fruits, alcohol components, etc. Depending on their intended purpose, tanks are subject to special requirements regarding their operational qualities. These requirements are usually formulated in regulatory documents or guidelines, such as the "Guidelines for the Design of Reinforced Concrete Tanks for Oil and Petroleum Products" (SN 326-65) or in water storage series 3.900.1-10 (issues 0-3, 3-1) and other documents.

In terms of plan shape, tanks are generally circular or rectangular. The choice of shape is determined primarily by economic calculations, and in some cases by local or special conditions-terrain, site plan, and building shape. When developing measures to ensure the impermeability of the walls, it should be taken into account that rectangular tanks, with the same volume as cylindrical tanks, have a larger wetted surface. Machine tensioning of prestressed external ring reinforcement can be used in cylindrical tanks. Tanks can also have cross-sectional shapes such as spheres, cones, droplets, ellipsoids, and others. However, creating complex cross-sectional shapes in reinforced concrete is comparatively difficult and labor-intensive, so they are not widely used. Standard tank designs have been developed in series 90-1-4-9 – 90-1-4-20; 3.900.1-10, and others. Tables 1.1 and 1.2 provide the basic dimensional diagrams of cylindrical and rectangular tanks used in water design and construction. For other materials (oil products, industrial wastewater, brine), these dimensions may vary.

Table 1.1 – Overall dimensions of cylindrical water tanks

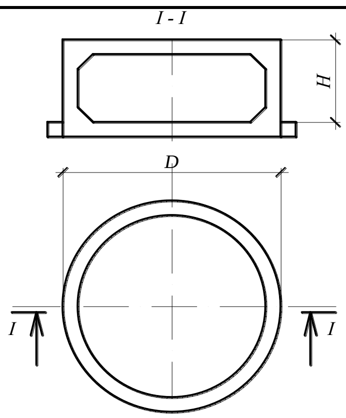
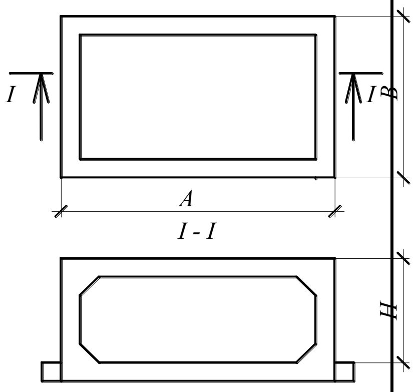
Construction diagram	Nominal capacity, m3	Nominal diameter D, m	Nominal height H, m
 	5	2,4	1,8
	15	3,6	1,8
	25	4,2	1,8
	50	4,2	3,6
	100	6,0	3,6
	250	10,2	3,6
	500	12,0	5,4
	1 000	18,0	5,4
	2 000	24,0	5,4

Table 1.2 – Overall dimensions of rectangular water tanks

Construction diagram	Capacity, m^3	Dimensions		Nominal height H, m
		IN	A	
	50	3,0	6,0	3,6
	100	6,0	6,0	3,6
	250	6,0	12,0	3,6
	500	12,0	12,0	3.6
	1 000	18,0	12,0	4,8
	2 000	24,0	18,0	4,8
	3 000	30,0	24,0	4,8
	6 000	36,0	36,0	4,8
	10 000	48,0	48,0	4,8
	20 000	66,0	66,0	4,8
	30,000	84,0	78,0	4,8
	40,000	90,0	96,0	4,8

1.2 Cylindrical reservoirs

This type of tank shape was used in ancient times and is widely used today. The main advantage of these tanks is the uniform stress state of the tank walls (tensile stress) and the compact footprint of these containers. Furthermore, the cylindrical surface prevents material concentrations in corner areas, which facilitates cleaning and maintenance. These tanks can be filled with liquids (similar to water), gels (similar to fuel oil or bitumen), and even saline solutions.

In tanks designed to store clean water, the required crack resistance and water tightness of the tank components are achieved by using dense concrete (W6, W8) and prestressing the external enclosing ring reinforcement. Clean water has no harmful effect on the concrete structure. Crude oil and dark petroleum products (fuel oil, tar) also have virtually no chemical effect on concrete.

In Baku, in the Eilat fields,1912. reinforced concrete cylindrical tanks with a capacity of up to were built100 m3for storing oil, which are still in operation today.

Light petroleum products are highly permeable through concrete, requiring special measures to seal it. Vegetable oils and acids are destructive to concrete, so when storing and processing such products in reinforced concrete tanks, direct contact should be avoided. Internal hydro chemical protection is required for such materials.

A significant number of these reservoirs are designed for groundwater backwater conditions. Water reservoirs with a capacity of over 10,000 m^3 are currently used in limited numbers, primarily in the water systems of large cities or large industrial facilities. One of the largest water storage reservoirs in the world is a circular reservoir with a capacity of 275,000 m^3, built in 1956 in South Africa. Its diameter is186.5 m, wall height – 10 m, the total area occupied by the tank is 2,8 hectares. The tank is made entirely of monolithic reinforced concrete.

In the USA, predominantly round cylindrical prestressed tanks are built with a capacity of 300 to 250,000 m^3 or more.

For tanks intended for storing flammable and combustible liquids, their location relative to ground level is determined by the choice of certain fire extinguishing means, as well as measures to ensure fire safety [8].

Oil storage tanks are primarily constructed at main pipeline pumping stations, refinery commodity depots, and transshipment terminals. The most common tanks have a capacity of 5,000 and 10,000 cubic meters, but under modern conditions, these capacities must reach 20,000–30,000 cubic meters. This increase in capacity allows for a 1,5-fold reduction in tank farm space, a 2,7-fold reduction in the length of process pipelines, and a 1,5-fold reduction in the length of fire protection pipelines.

Water and fuel oil storage tanks are insulated with soil or special thermal insulation materials to maintain positive temperatures in winter and prevent heat loss during fuel oil heating. The soil backfill thickness for fuel oil tanks is typically 250–300 mm, while the backfill thickness for water tanks depends on the construction site and is typically 500, 700, 1 000 or more mm in standard designs.

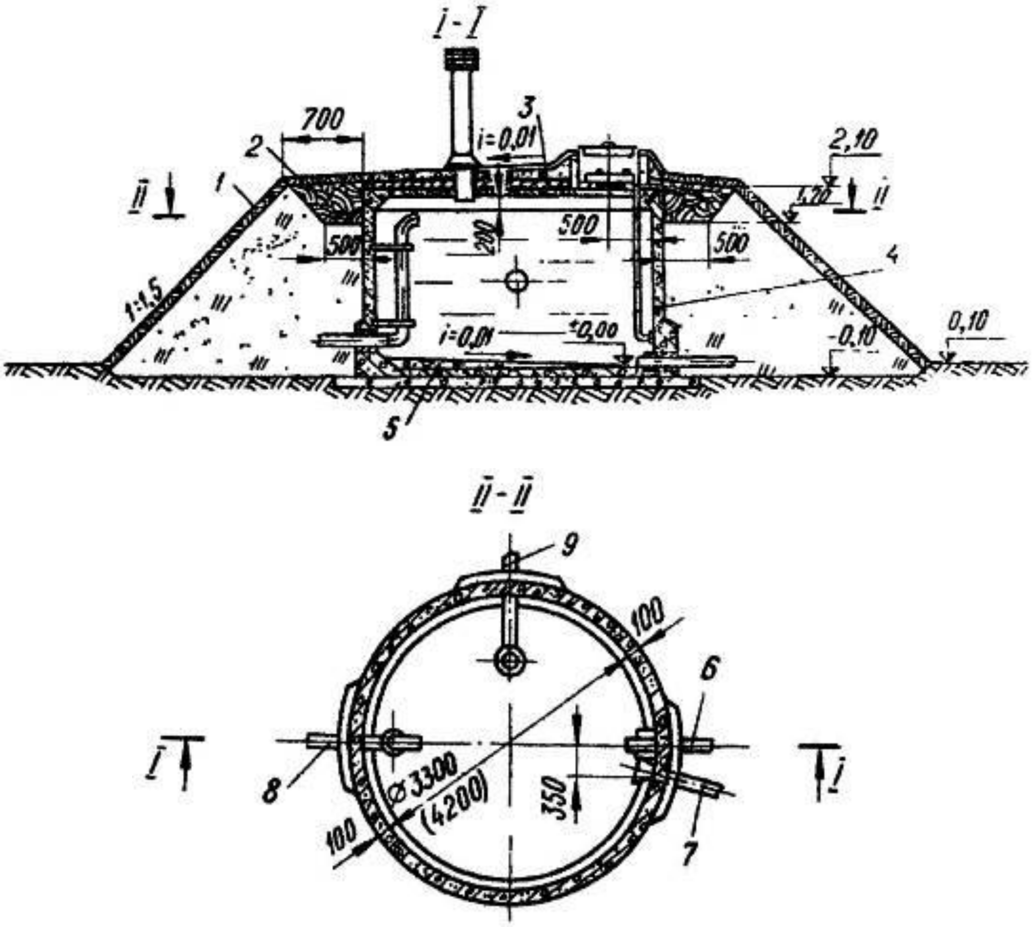


Figure 1.1 – Diagram of monolithic tanks with a capacity of 15–25 m^3 (standard project No 901-4-20): 1 – grass seeding; 2 – clay castle; 3 – covering (asphalt layer – 25 mm, cinder concrete grade 50, one layer of roofing felt, semi-rigid mineral wool slabs on a urea binder, two layers of roofing felt, reinforced concrete floor slab); 4 – wall (shotcrete – 25 mm, reinforced concrete wall)100 mm); 5 – bottom (sub-concrete class C8/10, reinforced concrete monolithic slab, bitumen coating, concrete preparation class C8/10); 6 – mud pipe with a diameter of 100 mm; 7 – outlet pipe with a diameter of 100 mm; 8 – feed pipe diameter of 100 mm; 9 – overflow pipe with a diameter of 150 mm

These tanks are located primarily above ground level, so groundwater does not affect the tank structure. The insulation above the roof in these tanks is made of semi-rigid mineral wool slabs, topped with layers of roofing felt, cinder concrete, and asphalt. The walls are insulated with earthen earth embankments.

In the tanks developed by, the insulation for the roof and walls is provided by a backfill of local soil. The thickness of the backfill for the roof is assumed to be equal to500 mm for areas with estimated temperatures up to -20 C and 1 000 mm – for areas with lower temperatures. Such reservoirs can be used in areas with seismic activity of up to magnitude 7.

One of the options is prefabricated cylindrical tanks with a capacity of 5 to 500 m^3 a tank according to standard series No. 901-4-19 can be provided (Fig. 1.2).

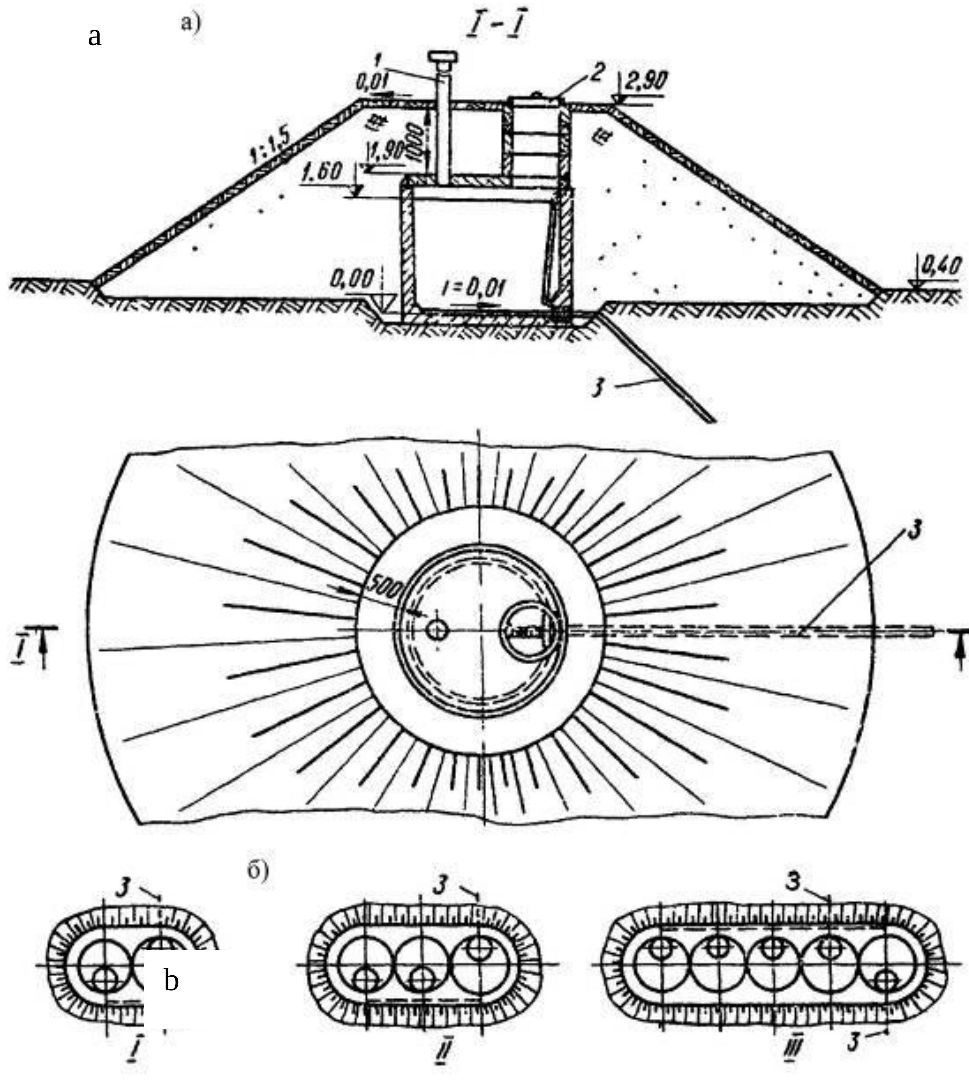


Figure 1.2 – Collection tanks with a capacity of 5 – 25 m^3

(standard design No. 901-4-19): a – design solution; b – diagram of the set of tanks:

I –10 m^3; II –15 m^3; III –25 m3; 1 – ventilation pipe; 2 – hatch;

3 – feed and flow pipe, ∅ 50 mm

The standard design allows for the possibility of combining blocks of two, three, or five with a common bund. This creates tanks with a capacity of 10, 15, 25 or more cubic meters.

Tanks of this type primarily intended for use in field and pasture water supply systems, but in some cases they can also be used on industrial sites. These tanks are designed for use in areas with design outdoor temperatures down to -40 °C and a characteristic snow load of 1,5–2,0 kN/m² with a soil backfill layer thickness.1000 mm.

The walls of these tanks are conical and are cast entirely at a precast concrete plant using C16/20 – C20/25 grade concrete on fine aggregate. The interior surfaces are also ironed there if necessary. The conical blocks are reinforced with welded mesh and individual rods. A cross-sectional view of such a tank is shown in Figure 1.2, a.

The design solution for these tanks is as follows: the bottom is monolithic, reinforced concrete, 120–160 mm thick, the walls are made of vertically installed precast reinforced concrete panels with a thickness of 120 mm. The walls are joined to the bottom by means of a slotted joint, and the covering is precast and monolithic. After installation and grouting of the joints, the walls are compressed by winding high-strength grade Bp1 300 wire with a diameter of 6–8 mm using a reinforcement winding machine, followed by the application of a 25–30 mm layer of shotcrete plaster. If groundwater is present, the bottom and walls are coated with 5–7 mm of bitumen (Fig. 1.3).

The roof panels are individually ribbed, resting on wall panels with calculated monolithic joints.

In tanks with a capacity of 250 and 500 m^3. The roof panels are also custom-made and radially cut. They rest on the wall panels along their outer diameter, and in the center, on a massive ring beam mounted on a precast column. This column, in turn, is installed in the precast foundation cup, which rests on a thickening in the monolithic foundation slab.

Wall panels for this type of tanks are made individually with a nominal width 1,57 m ($\pi/2$) according to series 3.900-2, issue 3.

General view of the tank with a capacity of 500 m^3 is shown in detail in Figure 1.3. This figure shows the left part, which is carried out in the absence of high groundwater, and the right part in the presence of such groundwater.

Storage tank equipment typically consists of four water pipes: supply, discharge, overflow, and sludge. The sludge pipe typically exits from a special sump built into the bottom.

The tank roof is equipped with ventilation pipes, the number and cross-section of which depend on the tank's capacity, as well as skylights and a separate hatch with a ladder leading down to the tank. All modern tanks are also equipped with a chamber for installing water level alarms.

In rectangular water-filled tanks, the walls are subject to tensile bending, and the material consumption (especially reinforcement) for the walls increases compared to round tanks. However, this drawback is offset by a significant simplification of the roof structure, which is made from a limited number of standardized prefabricated elements. Moreover, prefabricated elements can, in some cases, be used in the same way as industrial building frame elements. The most efficient volume for round tanks is considered to be between 50 and 2 000 m^3, and over 2 000 m^3. A more effective solution would be to use rectangular tanks.

Figure 1.3 shows a prestressed, buried storage tank for petroleum products with a capacity of 30 000 m^3. The tank bottom is constructed of monolithic reinforced concrete, while the wall and roof are made of precast reinforced concrete. The wall is assembled from vertically prestressed 2 100 mm × 9420 mm panels. The thickness of the panel changes from 157 to 260 mm, its weight is equal to 100 kN. The wall is rigidly connected to the reinforced concrete ring foundation by welding embedded parts. During installation, the panels are joined together by welding rebar extensions with a diameter of 10 mm, installed along the long sides of the panels with a step1 m. For ease of welding and embedding, a gap of 150 mm. Prestressing is applied not only to the tank wall, but also to the annular reinforced concrete foundation, the bottom, the annular side element laid along the cantilevered cornice of the wall's top, and the roof. To achieve this, several layers of high-strength prestressed wire are wound in the upper and lower zones of the wall using an ANM-5 machine. To ensure more uniform and intensive compression of the wall, bottom, and foundation ring, the bottom is placed on a layer of sand with a thickness of 50 mm. The gas impermeability of the coating is ensured by a water screen.

Figure 1.3 shows a cylindrical water tank with a capacity of 45,000 m^3, built in Oakland (USA). The diameter of the tank is 62,2 m, height – 12,2 m, wall thickness from – 0,6 m at the base before 0,3 m at the top. To reduce the height of the wall, the bottom is given the shape of a truncated cone. The pre-stressed wall of the tank is reinforced with horizontal and vertical reinforcement; the joint between the wall and the bottom is free (sliding).

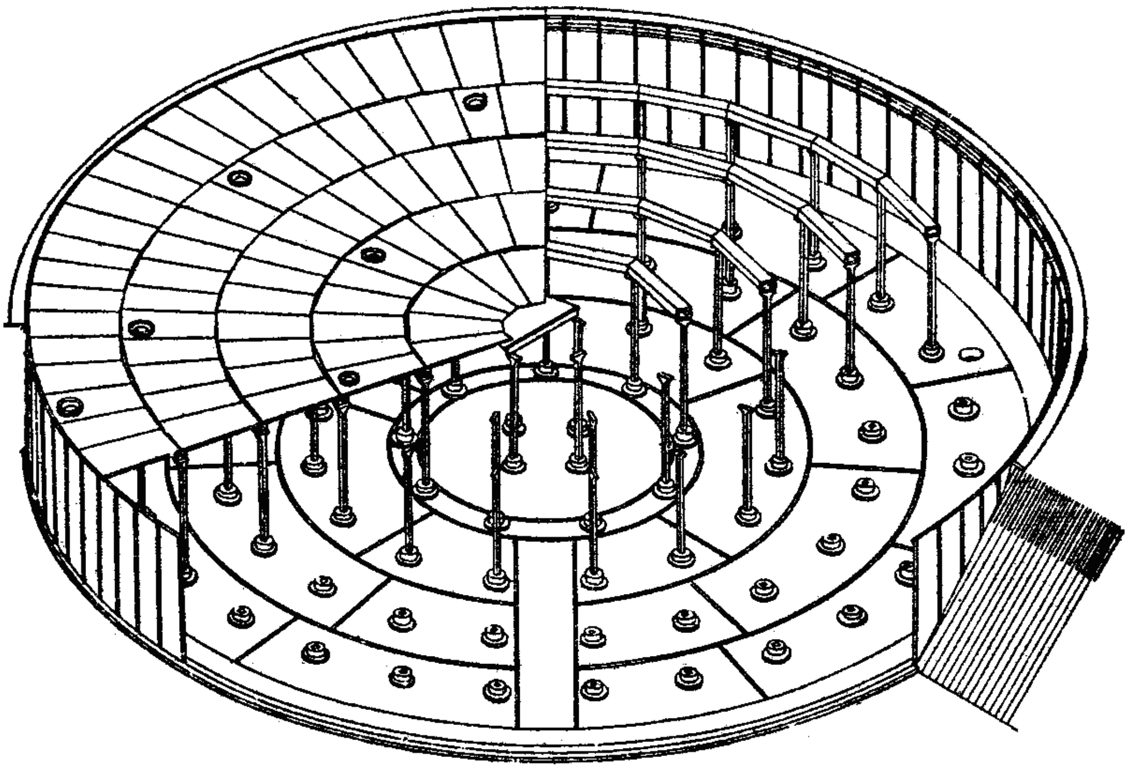


Figure 1.3 – Recessed reinforced concrete tank for petroleum products with a capacity of 30,000 m^3

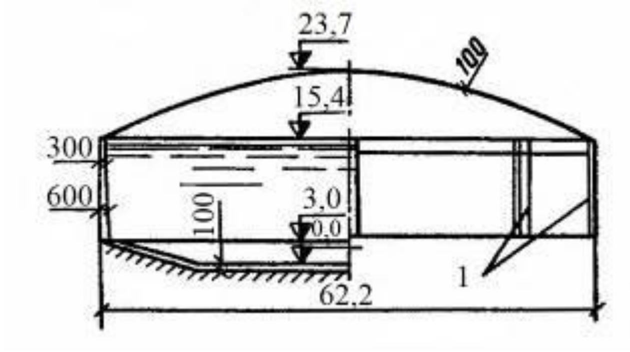


Figure 1.4 – Prestressed tank with a capacity of 45,000 m^3:
1 – pilasters for anchoring prestressed reinforcement

A common design for joining a pre-stressed (prestressed) tank wall to the bottom, foundation ring, and supporting structure of the roof is sliding supports. For this purpose, continuous rubber, neoprene, or choretene (rubber substitute) support pads are placed at the joints (Fig. 1.5, a, b). By allowing for free radial deformation of the wall at the joints, this design allows for tight compression of the wall along its entire height, including areas adjacent to the supporting elements. With rigid or hinged joints without displacement of the supporting sections of the wall, prestressing it does not ensure tight compression or watertight joints between panels in the areas of the supporting nodes. Furthermore, moments arise in the annular sections of the wall in these areas due to both the winding of the annular reinforcement and the loads on the wall, which may lead to cracks.

a b

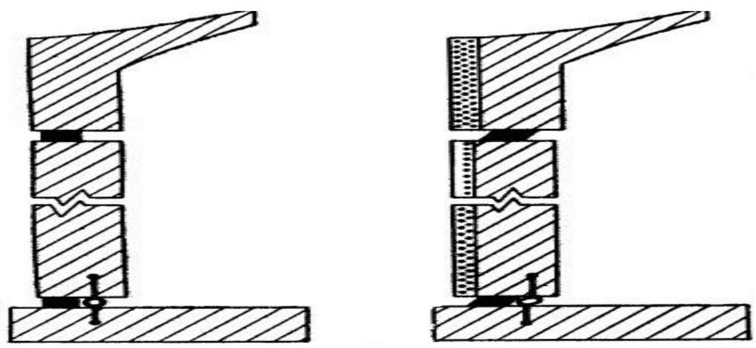


Figure 1.5 – Connection of the monolithic support ring of the coating with the cylindrical shell of the tank and the shell with the bottom support ring using elastic gaskets: a – position of the support gaskets before winding the prestressed reinforcement; b – the same after winding

1.3 Rectangular reservoirs

Along with cylindrical and conical tanks, rectangular tanks are very common. The capacity of rectangular tanks varies widely and can range from 50 to 20 000 m^3 or more. In design and use practice, it is recommended that 2 000 m^3 round (cylindrical) tanks are considered more rational, and within the range from 2 000 m^3. For capacities up to 20 000 m^3 and larger, rectangular tanks are most appropriate. However, global experience shows that cylindrical tanks can also reach volumes of 50 000 m^3 and even 100 000 m^3.

The walls and roofs are prefabricated, while the bottom is monolithic. The bottom plate is flat, with a thickness of 120–150 mm (There are localized thickenings at the column locations.) Considering possible uneven foundation settlements, standard rectangular tank designs include double reinforcement in the bottom slab.

The walls of rectangular tanks are made of prefabricated flat panels, the dimensions of which are determined by the height of the tanks and the distance between vertical joints. The nominal width of the wall panels is 3 m (In some cases, the width is reduced to reduce the dead weight to 1,5 m). The height of the wall panels is a multiple of 600 mm. The panels are reinforced with welded mesh. In rectangular tanks, the wall panels are embedded in a groove in the bottom. The panels are joined by welding horizontal reinforcement, followed by filling the joint with shrinkage-resistant concrete. In fuel oil tanks, temperature effects are taken into account when designing the wall panel joints.

The roofing is constructed using standard slabs and beams from the II-20 series for industrial buildings, which significantly simplifies the construction of the tanks. Precast slabs measuring 1,5 m × 6 m series II-24-2. The slabs are installed on the shelves of reinforced concrete beams or on top of rectangular beams. The slabs are welded to the beams and wall panels (Fig. 1.12) using steel embedded parts, and the gaps between them are filled with concrete. The beams are installed on precast rectangular columns and secured by welding the embedded parts. After welding the upper reinforcement and grouting the joints, the beams operate as continuous beams. A cement screed is laid on top of the slabs, followed by waterproofing.

Figure 1.6 shows one of the options for installing precast wall panels in a monolithic reinforced concrete bottom, which has a thickened section in the area where the wall panel is installed. In this place, a longitudinal glass section is formed, which, after the installation of the panels, is monolithed with concrete of class C12/15 – C16/20. The outer step of the glass is calculated taking into account the magnitude of the soil pressure arising from the dead weight of the tank and the weight of the internal filling. This figure also shows the basic reinforcement of the nodal junction of the walls and the foundation-bottom. Reinforcement of classes A400C and A240 with diameters from 8 mm to 12 mm.

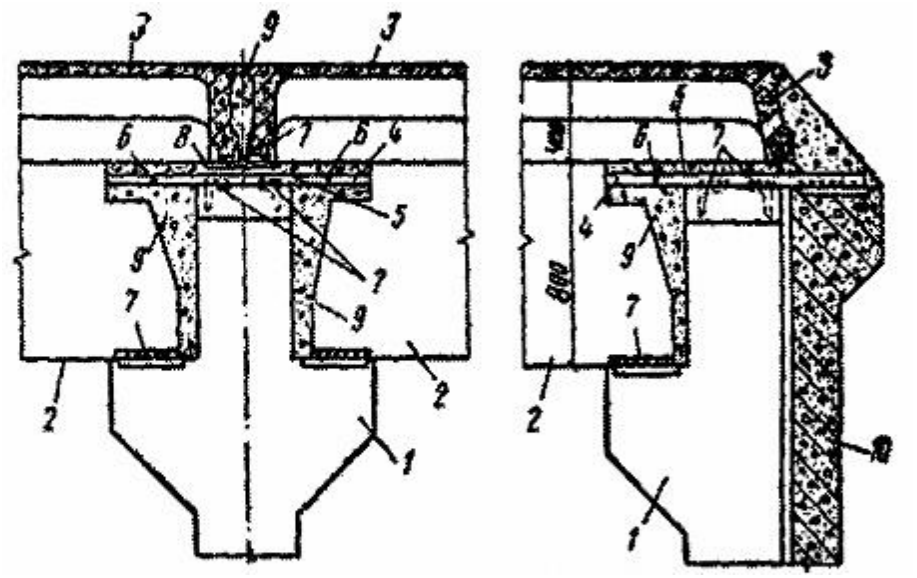


Figure 1.6 – Junction unit of prefabricated roof elements – option for supporting slabs on top of beams:

1 – middle and wall columns; 2 – rectangular cross-section beam; 3 – ribbed slab of size 1,5 m × 6 m; 4 – reinforcement releases from the beams; 5 – short piece made of reinforcing steel (welded to the column and wall panel); 6 – welding bath of reinforcement; 7 – welded seams; 8 – embedded part for fastening slabs; 9 – filling the joints between precast elements with concrete on fine crushed stone; 10 – wall panel

Precast reinforced concrete rectangular tanks of large capacity are assembled using internal columns manufactured according to the II-22-3 series, installed with a 6 cell × 6 m. Beams (II-23-3 series) are laid along these columns in one direction, supported by double column cantilevers. Columns with single cantilevers are provided at the outer (end) walls of the tank to support the outer beams.

Such tanks can be used throughout Ukraine, as well as in areas with outside temperatures not lower than -40 C and a design seismicity of no more than 7 points. The characteristic snow load should not exceed 1,6 kN/m2. The construction of rectangular tanks with a capacity of less than 20,000 m^3 is possible on sites with both dry soils and in the presence of groundwater (with a backwater height of up to 2,0 m). All large tanks are lined with earth to improve the thermal insulation of the enclosing structures. They must be constructed in dry soil.

For all types of tanks, the foundation soils must be homogeneous, non-subsiding and capable of withstanding a characteristic pressure of at least 1,5 kg/cm2 (0,15 MPa).

The soil backfill on the surface is arranged in three sizes – 500, 700, 1 000 mmand depends on the specific climatic conditions of construction.

The waterproofing of the coating, which prevents rainwater from penetrating into the tank, consists of a bitumen coating, hydroisol, laid on cement screed, $\delta = 30$ mm. Wall and bottom waterproofing, as well as interior surface finishing, are the same as for cylindrical tanks. The technical and economic indicators for typical rectangular tanks are presented in Table 1.3.

1.4 Protection of reservoirs from permeability

When designing reinforced concrete tanks, special attention is paid to their impermeability, protecting the tank's material from the harmful effects of the product stored within, and maintaining the product's physical and chemical properties over a long period. The methods and means of tank protection are quite diverse and specific.

One of the most effective means of protecting containers from permeability, as well as from the harmful effects of the stored product, is the use of linings or coatings that must possess the following qualities: strength, durability, and sufficient deformability, resistance to temperature exposure within the required limits, as well as durability and harmlessness with respect to the stored product. All of these qualities must be maintained over a long period of time.

In the US and France, the most common protective coatings are those based on thiokol latex (thiokol latex alkyl polysulfide, a vulcanizable plastic). The coatings are applied either by gluing thiokol sheets (cotton or glass fabric impregnated with thiokol latex) or by painting the surface with thiokol latex-based compounds.

In Ukraine and other countries, coatings made of 0,4–0,6 mm thick film vinyl plastic, glued to concrete with a cement-latex mixture, coatings made from epoxy resins, and fiberglass insulation have been developed and are being used.

For lining the internal surfaces of tanks, sheet steel with a thickness of 1– is also used. 4 mm. Tank lining with steel sheets can be done either after or before concrete pouring, using the pre-assembled steel lining as internal formwork. The first method fails to achieve a tight bond between the lining and the concrete, which can lead to the formation of "craters" and, consequently, corrosion of the lining from the concrete side. The second method requires that the steel lining sheets be of considerable thickness, as they are subject to the load of freshly poured concrete during concreting.

It should be noted that steel lining can in some cases only be used to prevent tank permeability and that the lining itself must be protected from the harmful effects of the stored product. Furthermore, the use of such lining results in significant consumption of sheet steel. Therefore, tank designs that achieve impermeability without the need for special protective coatings are of interest. Research is underway to develop special gasoline-impermeable concretes and to provide hydraulic insulation by saturating the concrete with water.

Currently, gasoline-impermeable concretes have been developed based on Portland cements with the addition of iron oxide hydrate or calcium and sodium chloride salts, as well as concrete in which epoxy resins act as a binder (plastic concrete). Reinforced plastic concrete (armoplastic concrete) can be used independently as load-bearing structures with high corrosion resistance, or as an insulating layer with conventional reinforced concrete (two-layer structures).

An effective method for constructing monolithic reinforced concrete reservoir structures is the use of Penetron additives. These additives offer high penetrating and waterproofing properties. The components of the mixture penetrate the concrete, forming tiny, indestructible crystals that prevent even the slightest water penetration. Penetron also resists the effects of aggressive environments. Penetrating waterproofing can penetrate concrete to a depth of 30–40 cm and more, which is quite sufficient for the walls and bottom of the tank.

Tanks with closed synthetic shells placed inside them, which protect the container from leakage and product evaporation, are very promising. However, sufficient operational experience with such tanks is currently lacking.

Combating product losses due to evaporation and increasing the gas impermeability of tanks is mainly carried out by:

1) installation of floating coverings (roofs) in the form of pontoons made of reinforced cement structures or synthetic materials;

2) by installing a water screen on the coating, i.e. the coating is designed in the form of a roof-bath with a layer of water 100–150 mm. Water, filling the capillaries in concrete, significantly improves its water tightness. This principle underlies the design of tanks, all elements of which (wall, bottom, and roof) are sealed using waterproofing;

3) preliminary stressing of all structural elements of the tank (bottom, walls and coating).

One of the means of combating losses from product evaporation from tanks is the creation of a constant temperature regime in them; however, this measure gives a positive result only during long-term storage of the product and requires the installation of special equipment that controls the temperature regime of the stored product.

1.5 General provisions for reservoirs calculations

When calculating storage tanks (as with any structure), the first step is to determine the design scheme for a specific element. Storage tanks are complex, intricate structures that incorporate a bottom, walls, and roof. Tank calculations can be performed in two ways:

1) each structural element is calculated independently of the others;

2) the tank is calculated as a single structure, taking into account the articulation of the constituent elements and the total load on these elements.

When choosing a calculation scheme (calculation model) for a tank, its shape in plan (round, rectangular, conical), the connection of the bottom with the walls (rigid or hinged), and the connection of the covering with the upper contour element of the walls (hinged, elastic-compliant or rigid) are of significant importance.

The strength of most structural elements of industrial and civil buildings and structures is calculated using the limit state method. In the first group of limit states, plastic deformations develop in concrete and its continuity is disrupted, i.e., cracks appear and open. In the reinforcement, stresses reach a limit state close to the yield strength or ultimate tensile strength of the steel. However, internal forces M, N And V For most statically indeterminate reinforced concrete structures, the cross-sectional parameters used for calculation are determined using elastic structural mechanics methods. Since storage tanks are structures in which cracking during operation is unacceptable, their static analysis is usually performed in the elastic state of stress.

In tanks, the calculations involve coatings, walls, and bottoms, which, as a whole, represent spatial systems. The calculations for cylindrical and circulars will be discussed separately.

Regardless of the adopted design solution and plan shape, the calculation of tanks must be carried out taking into account the following loading cases:

1) the tank is filled with water or another liquid component, but not covered with soil (during testing);

2) the tank is empty, but covered with soil;

3) the tank is partially or completely filled with internal product and covered with soil; in addition, the structures are subject to uneven heating or cooling.

Tanks should be designed for all specified loading cases, taking load combinations in accordance with DBN B.1.2-2006 “Loading and loading. Design standards”.

When calculating reinforced concrete tanks, characteristic loads, safety factors, tension accuracy of prestressed reinforcement, overheating or overcooling should be taken in accordance with the “Guidelines for the design of reinforced concrete tanks for oil and oil products” (SN 326-65).

In the first stage of calculating cylindrical tanks, we will consider a design scheme for a wall in the form of a vertical shell, hinged to the bottom. In this case, the tank wall is subject to a fluid load distributed linearly and depending primarily on the immersion depth of the section under consideration. The deeper the wall section, the greater the pressure transmitted to it from the fluid filling the tank. It should be remembered that, according to Pascal's law, fluid pressure is transmitted equally in all directions. That is, vertical and horizontal pressures have the same magnitude and depend primarily on the height of the liquid column. It is a basic design scheme for a cylindrical tank.

The characteristic value of the liquid pressure on the tank wall at a depth $l-x$ from the upper level of the liquid we designate p_x. Then the calculated value p_x can be found from the similarity of triangles with a known value $p_l = \gamma_f \rho l$:

$$\frac{p_x}{p_l} = \frac{l-x}{l}, \qquad (1.1)$$

from here:

$$p_x = p_l\left(1-\frac{x}{l}\right) = \gamma_f \rho l\left(1-\frac{x}{e}\right) = \gamma_f \rho (l-x), \qquad (1.2)$$

where ρ – average density of liquid (for water) $\rho = 1$, for petroleum product $\rho = (0{,}85-0{,}9)$;

γ_f – reliability coefficient $\gamma_f = 1{,}1-1{,}15$.

The hydrostatic pressure of the liquid causes hoop tensile forces in the wall N_x^0 Their value is determined on the basis of the equilibrium of a semi-ring with a belt height equal to one (this can be 0,5 m; 0,6 m; 1 m; 1,2 m). The entire height of the tank wall is divided into these belts, and the value is determined for each level p_{xi}. Then the tensile force is calculated.

$$N_x^0 = p_x \cdot R\,, \tag{1.3}$$

where R – radius of the ring.

The diagram of the hoop forces in the wall has a linear outline with a maximum at the bottom and a minimum at the surface of the liquid. Under the influence of the hoop forces, the perimeter of the wall increases and the wall itself moves in the radial direction. The diagram of these displacements (W) repeats the outline of the diagram. The area of reinforcement for the ring direction is selected using the formula

$$A_s = \frac{N_x}{\gamma_s f_{yd}}, \tag{1.4}$$

where γ_s – coefficient of operating conditions of reinforcement;

f_{yd} – design resistance of reinforcement.

In case of rigid connection of the walls with the bottom in monolithic tanks or prefabricated tanks with a support unit design. the radial movements at the bottom level are practically equal to zero due to the negligible low deformability of the bottom in its plane. Due to this, the vertical generatrix of the wall is curved, and bending moments arise in it. (M_x), acting along the generatrix, and the corresponding transverse forces (V_x).

The wall is an axisymmetric cylindrical shell. In it, as in other thin-walled spatial systems, bending is local. In the zone of local bending, the following equation holds:

$$\left(\frac{S^4}{4}\right)\frac{\partial^4 W}{\partial x^4} + W = \left(\frac{R^2}{E_{cm} \cdot h}\right) \cdot q\,, \tag{1.5}$$

where $S = 0{,}76\sqrt{R_y t}$; R_y, t – radius and thickness of the shell;

E_{cm} – modulus of elasticity of concrete;

h – shell cross-section parameter: $t \cong h$; $q = p_x$.

The solution to equation (1.5) can be represented in the following form:

$$M_x = C_1 e^{-\varphi}\cos\varphi + C_2 e^{-\varphi}\sin\varphi. \tag{1.6}$$

At the same time $M_x = -D\dfrac{d^2W}{d\,x^2}$; $D = EJ$.

When rigidly fixing the wall to the bottom (taking into account the local moment M_1 and transverse force V_1) final expressions for determining the hoop forces N_x and bending moments M_x in the wall at a level located at a distance x from the bottom, look like this:

$$N_x = N_x^0 - p_l R\left[e^{-\varphi}\cos\varphi + e^{-\varphi}\sin\varphi\cdot\left(1-\frac{S}{l}\right)\right]; \tag{1.7}$$

$$M_x = 0{,}5 p_l S^2\left[\left(1-\frac{S}{l}\right)e^{-\varphi}\cos\varphi - e^{-\varphi}\sin\varphi\right], \tag{1.8}$$

where $\varphi = \dfrac{x}{l}$ – dimensionless coordinate, S (see (1.5));

p_l – liquid pressure at the bottom of the tank.

At the bottom level $x = 0$, meaning $\varphi = \dfrac{x}{S} = 0$, $e^{-\varphi} = 1$, $\sin\varphi = 0$, $\cos\varphi = 1$, then from equation (1.8) the maximum bending moment can be determined:

$$M_{\max} = 0{,}5 p_l S^2\left(1-\frac{S}{l}\right). \tag{1.9}$$

Characteristic diagrams for N_x And M_x are shown.

When the prefabricated cylindrical wall is movably connected to the bottom at its end, a frictional force is generated due to the radial movement of the wall $Q_f = N\cdot\mu$; N – normal pressure from the weight of the wall and the covering on it together with the backfill; $\mu = 0{,}5$ (coefficient of friction).

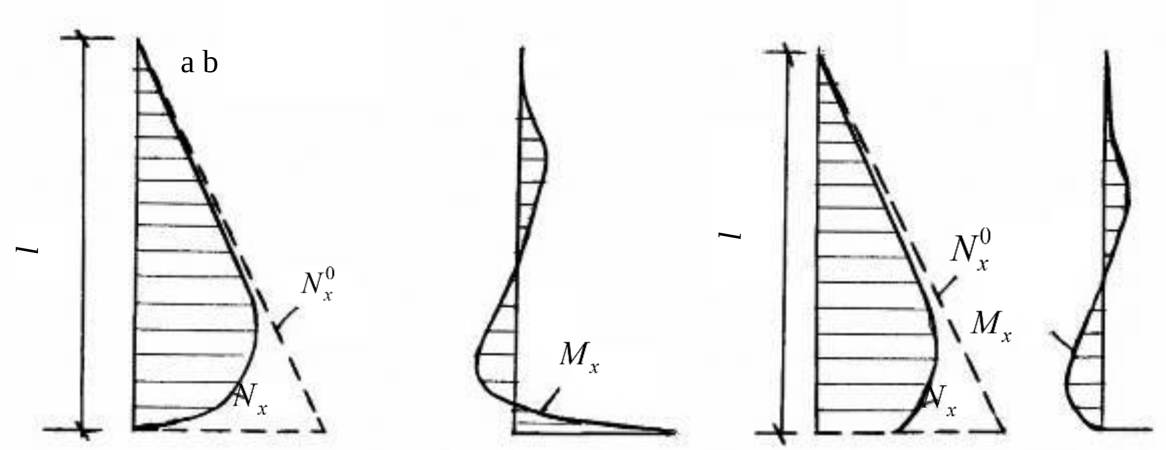


Figure 1.7 – Diagrams of hoop forces and bending moments:
a – the connection between the wall and the bottom is rigid; b – the same is movable

The hoop tensile force in the wall at the level x from the bottom is determined by the formula

$$N_x = N_x^0 - 2\left(\frac{R}{s}\right) \cdot Q_f \cdot e^{-\varphi} \cos\varphi . \qquad (1.10)$$

In this case, the maximum moment

$$M_{x,\max} = Q_f \cdot S \cdot e^{-\varphi} \sin\varphi \qquad (1.11)$$

it is located at a distance x_1 from the bottom

$$x_1 = 0{,}6\sqrt{R \cdot h} . \qquad (1.12)$$

Distribution of efforts N_x and M_x in the case of a movable displacement of the wall, shown in Figure 1.17, b.

The cross-sectional area of the vertical reinforcement of the walls is determined as in a bending slab, separately from the effect of hydrostatic pressure and external backfill. The calculated amount of reinforcement is determined based on the moment values shown in Figure 1.7. The reinforcement is placed in the lower third of the wall with a protective layer.15 mm; above, structural reinforcement with a diameter of 10–12 mm class A400C or B500.

There are no special requirements for the calculation and design of the roofs and columns of cylindrical tanks. They are designed as bending and compressed elements. Under normal conditions (in the absence of groundwater backwater), the weight of the bottom and the liquid above it is balanced by the soil pressure, causing no bending of the bottom. Only in areas adjacent to the wall and column foundations do local bending moments arise in the bottom. Special reinforcement is provided in these areas; in the rest of the area, the reinforcement is installed structurally (diameter 12–16 mm class A400C). The bottoms are usually made monolithic.

For a more accurate calculation of the walls of cylindrical tanks, we can consider the equilibrium of an elementary volume of wall cut out of the tank and compose two equilibrium equations (Fig. 1.8): the sum of the moments about the axis $x-x$ and the sum of the projections of all forces on the axis z.

$$\sum M_{x-x} = 0;\ M_1 - \left(M_1 + dM_1\right) + N\frac{dx}{2} + \left(V + dV\right)\cdot\frac{dx}{2} = 0,$$

where dM_1 – increment of bending moment;

dV – increase in shear force;

$$dM_1 + V\frac{dx}{2} + \left(V + dV\right)\frac{dx}{2} = 0.$$

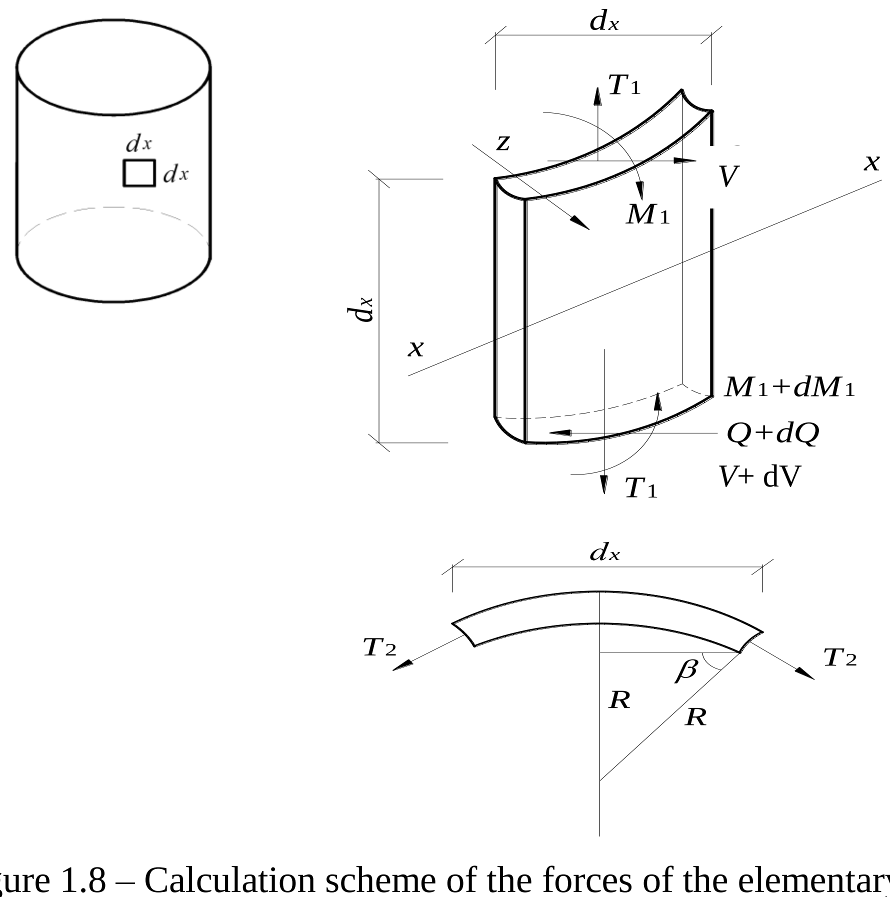


Figure 1.8 – Calculation scheme of the forces of the elementary volume of the tank wall

Discarding the terms of the second degree of smallness, we obtain

$$dM_1 + Vdx = 0;$$

$\frac{dM_1}{dx} - V = 0$ – first differential equation. (1.13)

Let's compose the second equation:

$$\sum Nz = 0; V - (V + dV) + 2T_2 \cos\beta + zdx = 0;$$

$$\cos\beta = \frac{dx}{2R}; \quad -dV + 2T_2 \times \frac{dx}{2R} + zdx = 0; \quad -dV + \frac{T_2 dx}{R} + zdx = 0;$$

$\frac{dV}{dx} - \frac{T_2}{R} - z = 0$ – second differential equation. (1.14)

Then the system of differential equations for determining the stress state of the wall of a round tank will have the form:

$$\left.\begin{aligned} &\frac{dM_1}{dx} - V = 0 \\ &\frac{dV}{dx} + \frac{T_2}{R} - z = 0, \end{aligned}\right\} \tag{1.15}$$

where z– external load from liquid pressure.

By solving two equations (1.15) together and changing the value of the internal pressure z Based on the height of the tank, the value of the tensile force can be determined $T_2(N_x)$ and bending moment $M_1(M_x)$.

When calculating rectangular tanks, two possible design schemes should be considered: the first is a solid, rectilinear tank wall, rigidly fixed to the bottom and pivotally supported in the cover zone; the second involves the installation of vertical ribs along the length of the wall (monolithic or prefabricated).

The walls of rectangular tanks are designed to withstand unilateral hydrostatic pressure in the absence of backfill, as well as unilateral lateral soil pressure when the tank is empty. Soil pressure is determined using the formula:

$$q_{гр} = \gamma_f \cdot \rho H \, tg^2\left(45^0 - \frac{\varphi}{2}\right), \tag{1.16}$$

where $q_{гр}$ – soil pressure on the tank wall in kN/m^2;

ρ – average soil density in kN/m^3;

γ_f – reliability coefficient (1.1) – (1.2);

H – distance from the planned ground level to the section under consideration in m;

φ – angle of internal friction of soil (300–500).

The pressure of the stored liquid or water when testing the tank p_x in kN/m^2 is determined by formulas (1.1) and (1.2).

A monolithic wall without ribs, as well as a prefabricated wall with vertical joints of a tongue-and-groove shape are calculated using a beam scheme, taking the span l_1 equal to the distance from the upper edge of the bottom groove to the coating.

When calculating, a vertical strip of width is allocated 1 m together with the loads located on it. shows a diagram of bending moments acting in the vertical direction; the values of the moments in the clamping and the maximum value in the wall, determined by the formulas of structural mechanics, are indicated.

In a monolithic or precast wall reinforced with pilasters, ribs, or wall columns, each wall section between them is calculated as a slab supported along the contour. Along the edges of the pilasters and the bottom, the slab is considered rigidly clamped, while at the roof level it is hingedly supported. In this case, the aspect ratio should be $l_2 / l_1 \leq 2$, If $l_2 > l_1$ In the case of a precast roof, the hinged support is due to the moment-free connections between the precast roof slabs and the wall panels, while in the case of a monolithic roof, it is due to support on a slab with low bending rigidity.

The walls of rectangular tanks are typically reinforced with double mesh to support the loads from internal liquid pressure and soil pressure when the tank is empty. The lower third of the walls is reinforced with a stronger welded mesh with rods 12–16 mm in diameter, while the upper third is reinforced with smaller diameters of 8–12 mm. In tanks with slabs supported along the perimeter, the main rods are vertical and horizontal rods; in tank walls designed using a beam design, the main rods are vertical rods.

TOPIC 2 BUNKERS

2.1 Purpose and types of reinforced concrete bunkers

Many production processes and process lines in industry, agriculture, processing plants, the chemical industry, and other sectors of the national economy require the permanent or temporary storage, movement, or sorting of various materials. For these purposes, specialized engineering structures are used, such as bunkers, silos, elevators, pile drivers, galleries, and the like. The dimensions of these structures can vary greatly, from small (up to 1 m^2 (in plan) up to several tens and hundreds of square meters; the materials used to construct these structures can also vary: reinforced concrete, metal, brick, wood, plastic, glass, ceramics. This section discusses large-scale reinforced concrete bunkers.

Bunkers are self-unloading containers designed for short-term storage of dry, bulk, coarse and fine materials (sand, crushed stone, lime, coal, ore, clinker, crushed stone, etc.). Structurally, reinforced concrete bunkers are spatial systems of a box-shaped or circular type, typically formed from flat elements. Less common are bunkers consisting of cylindrical and conical shells (walls and bottom, respectively).

Hoppers are loaded from the top and unloaded from the bottom, so their discharge bottoms are designed with sloping walls in the form of funnels. The slope of the bottom-funnel planes should be 5–10° greater than the natural angle of repose of the bulk material and is typically 35–60°. The smallest size of the discharge opening should be equal to six times the largest unit size of the stored material, but not less than 600 mm. If the storage facility has a flat bottom, the hopper is installed on the side of the wall, with a sloping layer of lean concrete or reinforced concrete laid at an angle of 15–30° across the bottom. Steel hoppers suspended from reinforced concrete supporting structures are often used in bunkers.

Wall height in bunkers $H < 1{,}5D_{\min}$, Where $D_{\min}$ – the smallest container size in plan (Fig. 2.1). With this ratio of height and width, the friction forces of the bulk material against the bin walls can be conditionally ignored.

The most common use of reinforced concrete bunkers is in the mining, coal, chemical and building materials industries.

Bin shapes can vary widely. They are determined by production technology, the properties of the bulk material, and economic considerations. Bins are most commonly square or rectangular. They can be single (Fig. 2.1) or located adjacent to one another, forming multi-cell bins (Fig. 2.2). Bins with an elongated rectangular plan are often constructed. The end and intermediate walls are vertical, and the bottom is shaped like a trough. Such bins are called trough or folded bins (see Fig. 2.3). To improve the flow of bulk material, special slopes are installed in the trough, usually made of lean concrete.

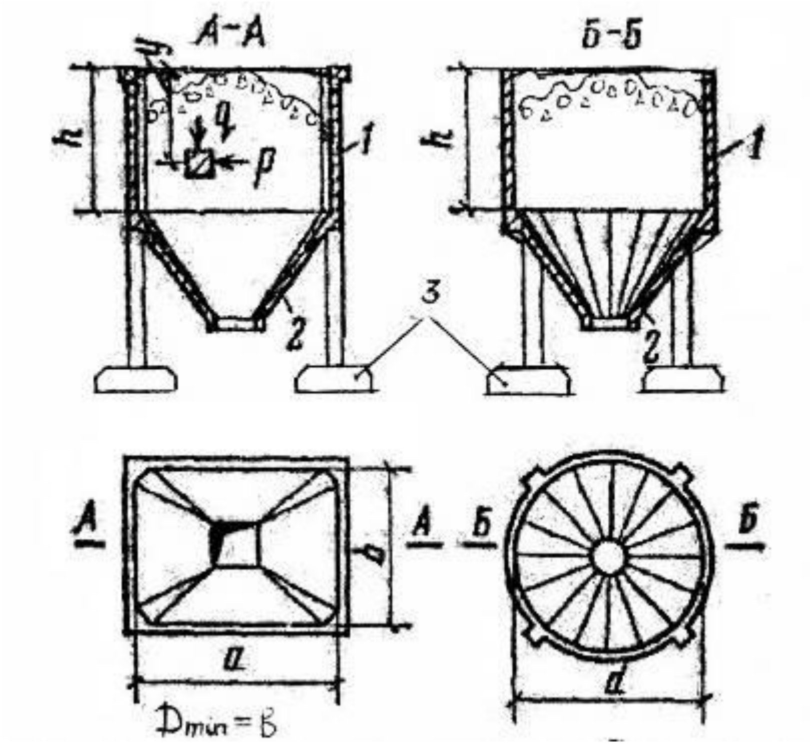


Figure 2.1 – Single bunkers (monolithic):

a – prismatic shape; b – cylindrical shape; 1 – wall; 2 – funnel

To protect the bin walls from abrasion, they are lined with steel sheets, cast iron, or diabase tiles. If the stored material has a harmful chemical effect on concrete, the bin's interior surfaces are lined with protective linings.

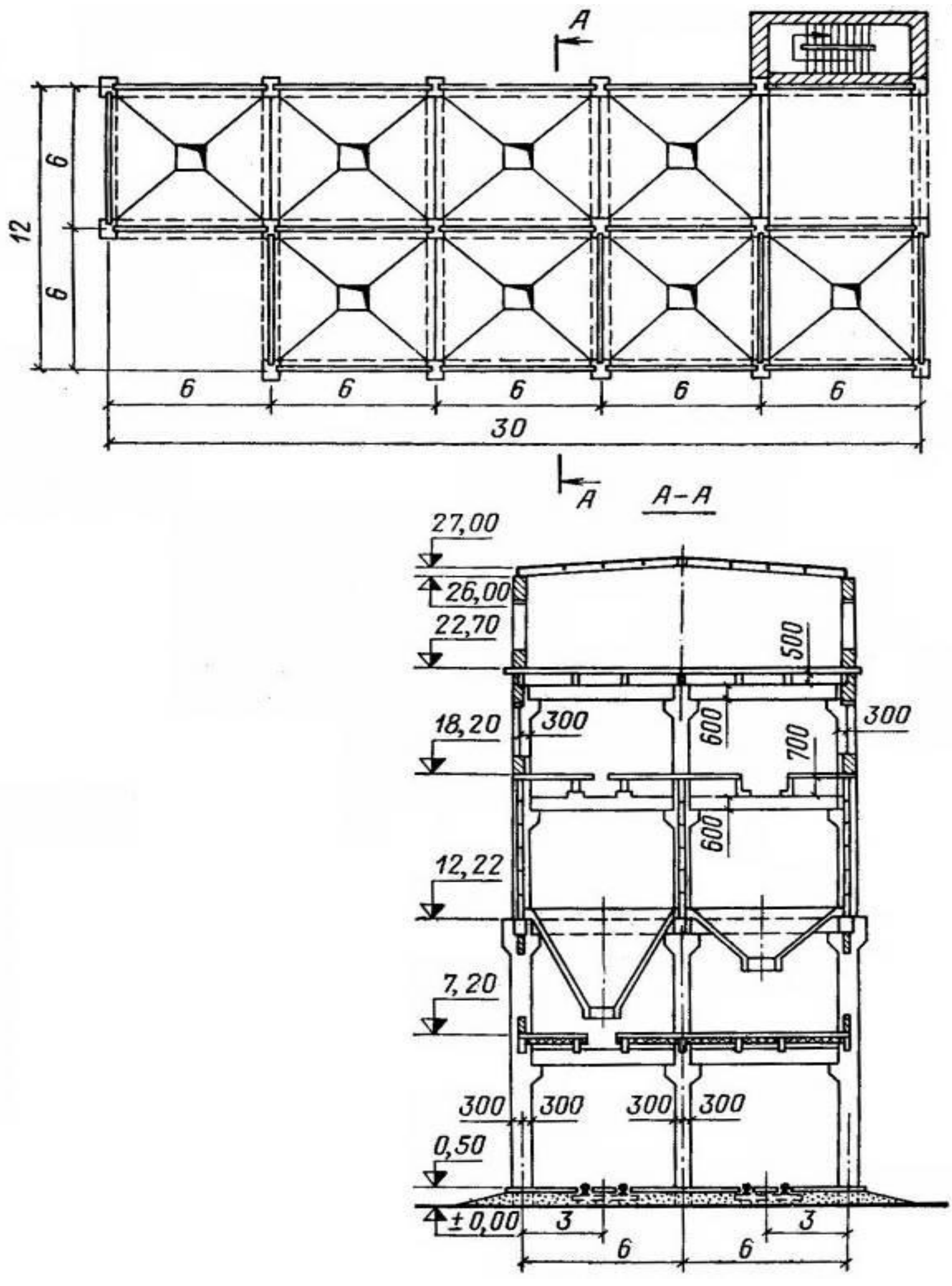


Figure 2.2 – Multi-cell bunkers

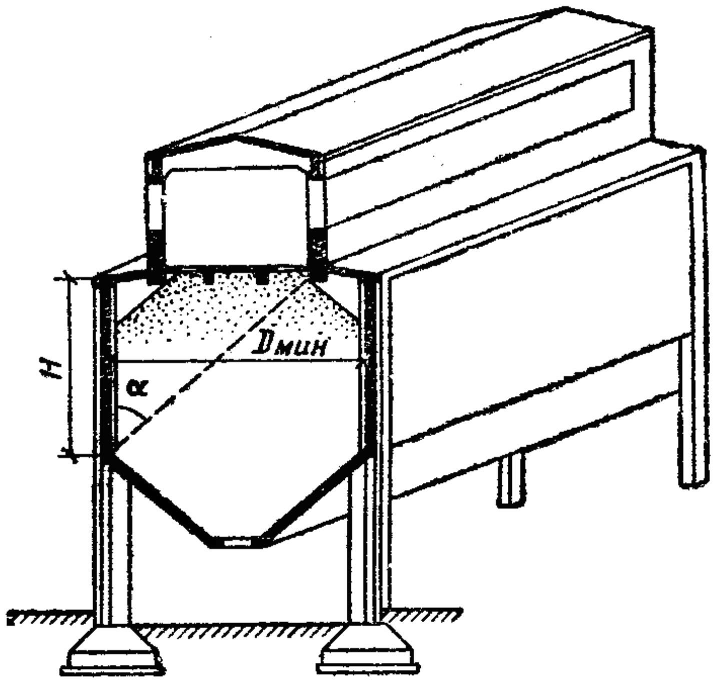


Figure 2.3 – Structural diagram of a chute hopper

Bunker structures consist of bunkers themselves, loading and unloading devices, flow inducers for poorly flowing materials, valves, feeders, automation and control devices.

According to the area of application, bunkers are classified as receiving, intermediate or dispensing.

Receiving bins are designed to receive bulk cargo from vehicles, compensating for uneven supply to a warehouse or production line, so the size of the bin must ensure sufficient unloading space.

Intermediate bins are used for operations related to the storage and dispensing of goods within production and processing lines. These include containers that provide long-term storage of food or non-food materials, including drying, dosing, mixing, and other operations.

Discharge bins are used for accumulation, short-term storage and unloading of bulk materials into containers and vehicles.

All of the above-mentioned types of hopper tanks are equipped with devices and mechanisms that ensure their stable operation. These include loading devices, valves, feeders, and active and passive flow stimulants.

The dimensions of the bunker cells in plan are usually within 4,5–6 or 8 m, wall height from 4 m to 12 m. However, oversized bins are also common in global practice. For example, a Canadian cement plant has built a conical bin for storing clinker (calcined alumina pellets used in cement production). This bin is a combination of two conical (top and bottom) ribbed shells and a single cylindrical ribbed shell in the middle between the conical shells (Fig. 2.4, a).

The internal diameter of the cylindrical part of the structure is 65,2 m, height above the ground – 35,55 m, below ground level the bunker is buried 22,2 m deep.

The bin is loaded through an upper loading chamber, where the material is fed by a conveyor gallery. Unloading occurs through an unloading chamber, connected to an underground conveyor gallery.

Conical covering height 20,9 m consists of 64 prestressed reinforced concrete trapezoidal elements of T-shaped cross-section. The length of the element35,36 m, weight 34 tons.

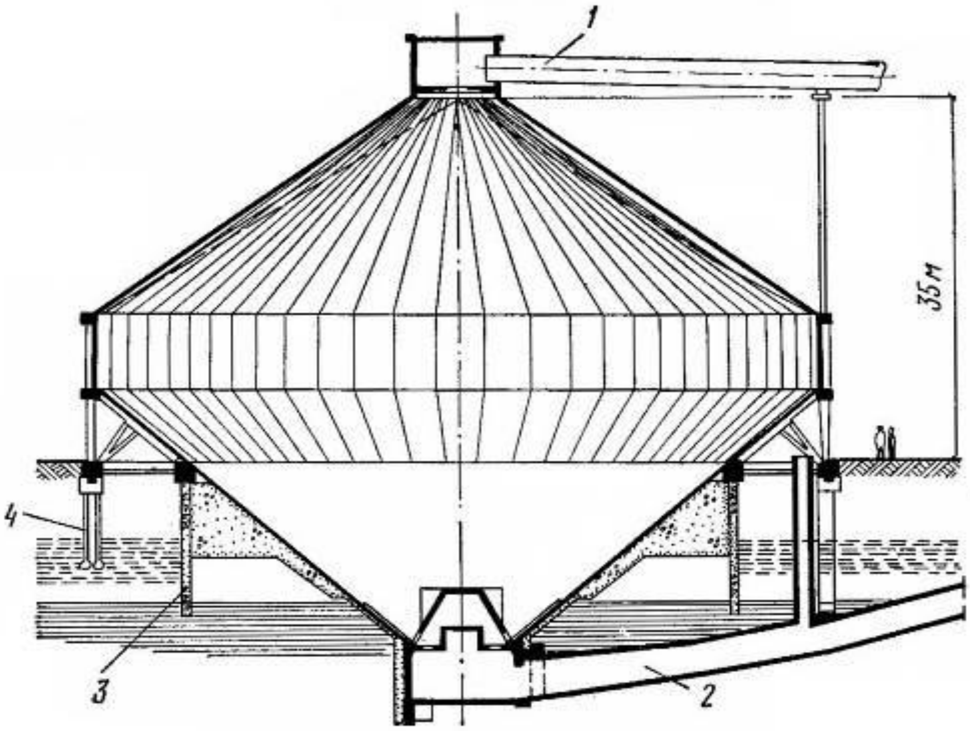
Closed and open-topped bins are used. Open bins (Fig. 2.5, a, b) are less expensive than closed ones, but they are used only for cargo that is not exposed to atmospheric precipitation and that does not emit dust harmful to the health of service personnel. Closed bins with conical roofs do not have empty zones when filling (Fig. 2.5, c). Bins with flat ceilings always have empty zones, especially with a side loading opening (Fig. 2.5, e).

Rectangular bins with vertical walls are shown in Figure 2.6. Based on the bottom shape, such bins are divided into double-pitched (a), triple-pitched (b), quadrilateral (c, d), and multi-pitched (d). A distinction is also made between pyramidal and obelisk-shaped bottoms (Fig. 2.7, a, b). In a pyramidal bottom, all the edges intersect at a single point (0 in Fig. 2.7, a), while in an obelisk-shaped bottom, they intersect in pairs at four points (01–02 and 03–04).

In pyramidal open bins, the upper loading and lower discharge openings are geometrically similar; in obelisk bins, this similarity is not present.

$$\frac{ctg\alpha_1 + ctg\alpha_3}{ctg\alpha_2 + ctg\alpha_4} = \frac{A_b}{B_b}.$$

a



b

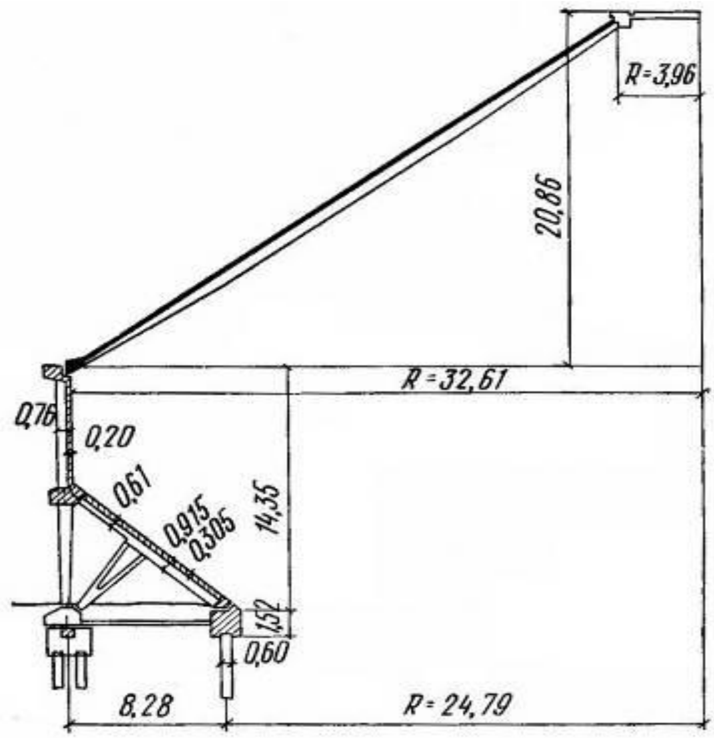


Figure 2.4 – Clinker storage bin:

a – section of the bunker; b – fragment of the support of the conical bottom wall on the pile foundation;

1 – loading transport gallery; 2 – underground unloading conveyor gallery;

3 – filled wall; 4 – piles

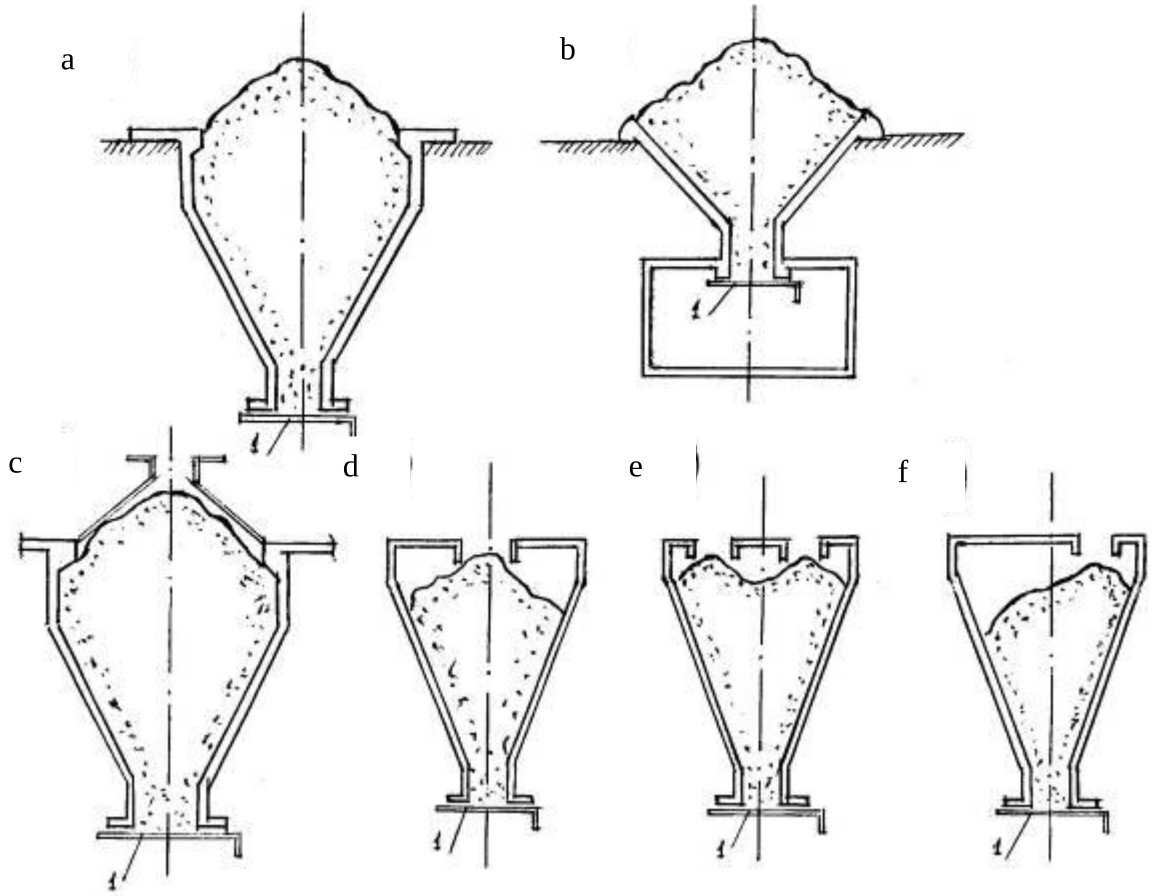


Figure 2.5 – Possible shapes of upper loading platforms:

a, b – open; c – closed pyramidal; d, e – closed flat; f – damper (gate)

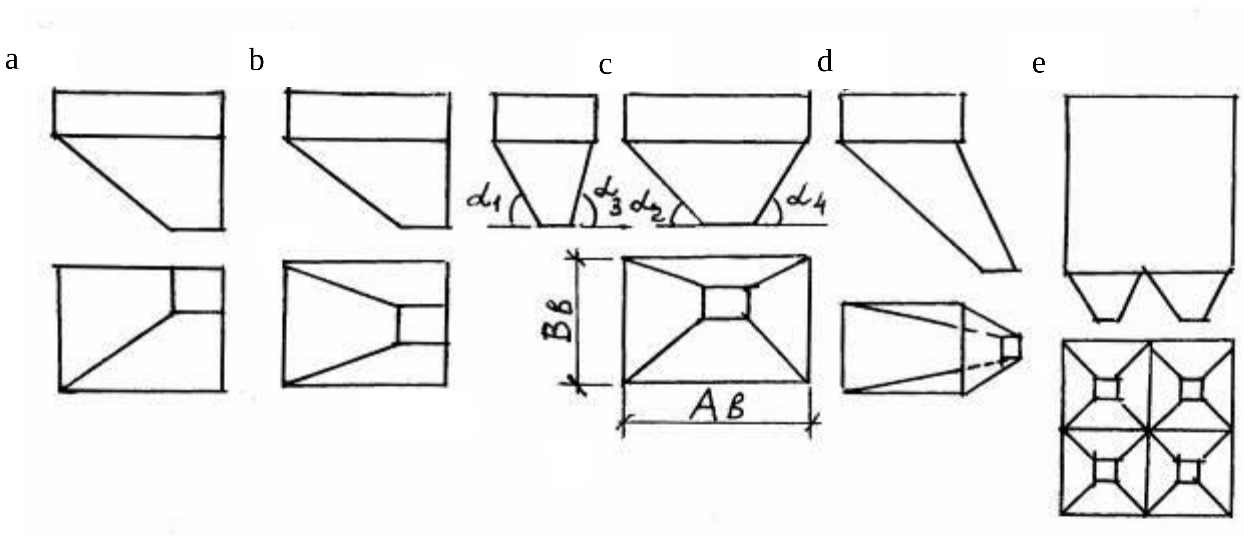


Figure 2.6 – Various shapes of rectangular bins:

a – double-pitched; b – triple-pitched; c – quadri-pitched; d – quadri-pitched with offset outlet; d – single-pitched

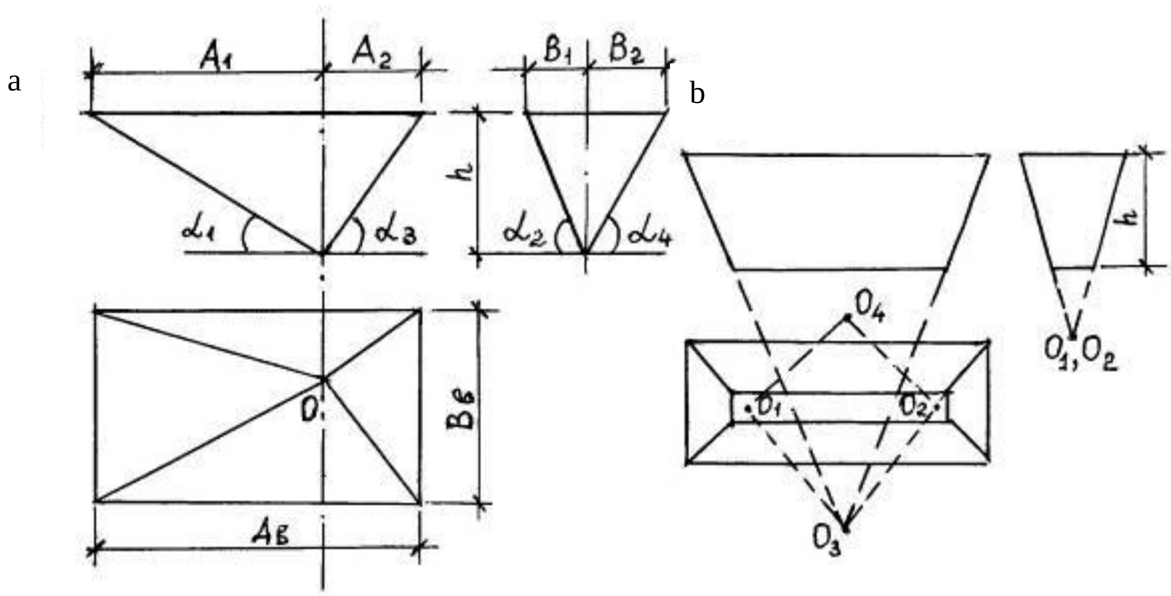


Figure 2.7 – Bottom geometry: a – pyramidal; b – obelisk

2.2 Basic provisions for calculating of bunkers

External influences on various types of bunkers depend on a large number of objective parameters that influence both the choice of the bunker design scheme and the methodology for its calculation.

The order of magnitude and distribution of design loads depend primarily on the bin structure, the parameters of the stored bulk material, and the flow profiles formed during unloading. Inevitable differences in the properties of bulk material stored in different ways and simplifications in load models lead to deviations between the actual loads in the bin and the loads received (design loads). For example, the distribution of unloading pressures along the bin wall changes over time. It is extremely difficult to accurately predict the underlying average pressure, its dispersion, and its variability over time with current knowledge.

Loads on the vertical walls of a bin, both when filled and during unloading with minor eccentricities during filling and unloading, should be determined by an asymmetric load component and an asymmetric partial surface load. For larger eccentricities, the loads should be described by an asymmetric pressure distribution curve.

Symmetrical loads on the walls of the bunker are established due to the horizontal component P_n on the inner surface of the perpendicular walls of the bunker due to the loads acting perpendicularly on the inclined walls $P_n \cos\alpha$, due to friction loads P_w and P_τ,acting in the tangential direction relative to the wall, and due to the vertical component of the load in the stored bulk material, while the loads from friction forces are insignificant and, with a small height of the bin, may not be taken into account in the calculation ($H \leq 1{,}5D$).

Asymmetric loads on the vertical walls of the bin with slight eccentricities in filling and unloading must be accounted for by adding a partial surface load. These loads consist of locally acting horizontal pressures P_p on the inner surface of the bin wall.

Asymmetric loads on the vertical walls of the bunker with large eccentricities of filling and unloading should be additionally recorded due to the asymmetric distribution of horizontal pressures P_p and loads due to friction.

To register unplanned, unaccounted load effects, load reliability factors C should be used: C = 1,1–1,4.

Load increase factors C for bin elements with a given requirement class shall be used solely to record the additional load effects that are taken into account and which arise due to the flow of bulk material during bin unloading.

Load magnification factors C for bin elements with requirements class 1 shall be used to record both additional loading effects during unloading due to bulk material movement and effects due to dispersion of bulk material parameters.

For bunkers of requirement class 2, asymmetrical partial surface loads can alternatively be taken into account by alternately increasing symmetrical loads, the magnitude of which is consistent with the effect of the partial surface load.

A very important parameter when determining the loads on the bin walls and hopper is the bulk material flow profile, commonly referred to as the flow profile. Three main flow profile categories are most commonly considered:

– mass flow of outflow or movement of bulk material;

– a flow of outflow resembling movement in a pipe;

– mixed flow.

These types of flow profiles are shown in Figure 2.16.

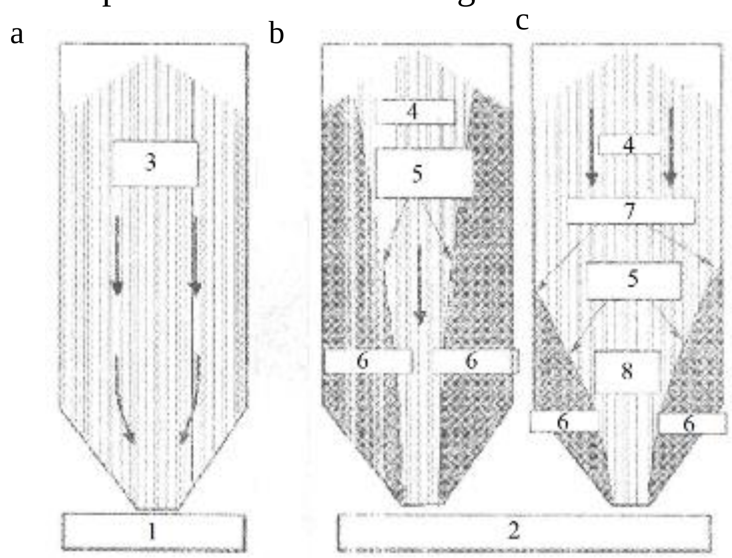


Figure 2.8 – Basic flow profiles:
1 – mass flow; 2 – central flow; a – mass flow; b – central flow (flow in a pipe); c – central flow (mixed flow); 3 – all bulk material in motion; 4 – flowing bulk material; 5 – boundaries of the flow channel; 6 – bulk material at rest; 7 – effective transition; 8 – effective funnel

For bins with pneumatically fed bulk material, two design situations must be considered at maximum filling: 1. The filled bulk material can form a filling cone, as is the case with other types of bulk material. 2. It must be taken into account that the bulk material surface, under certain circumstances, also forms a flat surface, regardless of the inclination angle and eccentricity during filling (see Fig. 2.9). In this case, the eccentricities must be set to zero, and the load is assumed to be symmetrical and uniformly distributed.

In bins for storing powdery bulk materials with a continuous air supply in the bottom area to assist unloading (see Fig. 2.9, b), the entire bulk material zone near the bottom can be fluidized, which can cause effective mass flow even in a low bin. Such bins, regardless of the actual flexibility h_c/d_c, must be designed according to the flexible bin method.

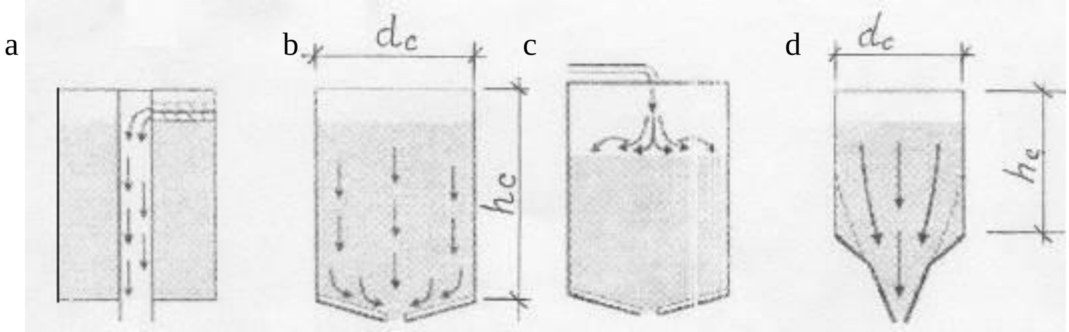


Figure 2.9 – Special filling and unloading arrangements: a – mechanically supported unloading with concentric loads; b – air supply and ventilation slots, creating a mass flow; c – pneumatic filling of powdered bulk material, often resulting in a smooth surface of the bulk material; d – “expanded flow”: the funnel causes a mass flow only in the lower part

In bins for powdery bulk materials with a continuous air supply in the bottom area to assist unloading (see Fig. 2.9, b), only a partial zone of the bulk material near the bottom may be fluidized. This can cause eccentric flow in the pipe, which must be taken into account in the calculation. The eccentricity of the resulting flow channel and the resulting value of the set eccentricity e0 must be determined taking into account the fluidized zone, and not just the position of the discharge outlet.

The vertical walls of a bin with a discharge hopper, which results in "expanded flow" (see Fig. 2.17, d), may be subject to mixed flow conditions of bulk material, which can lead to asymmetric loads during discharge. For this type of bin, the ratio h0/dc should be used instead of h_c/d_c for flexibility.

A bin with a flexibility h_c/d_c of less than 0,4 and a drain funnel should be classified as a low bin. If the bottom is horizontal, such a bin should be classified as a bin with a protective wall.

For a hopper with a non-conical, non-pyramidal, or non-wedge-shaped hopper, an appropriate method must be used to calculate the hopper loads. For a hopper with internal built-in elements, both the hopper loads and the loads on these built-in elements must be determined using an appropriate method.

In a hopper with a wedge-shaped funnel under a round cylinder (drill-shaped funnel), an appropriate calculation method should be used to determine the funnel loads.

For bins with multiple drain holes, taking into account the maximum possible filled state, it is necessary to take into account that during operation either a single drain hole or a combination of simultaneously open drain holes may be in operation.

In bunkers with multiple drain holes, combinations of active drain holes intended for operation should be considered as a normal design situation. Other, non-exclusive situations of opening funnels that are not intended for planned operation should be considered as emergency design situations in accordance with DBN B.1.2-14:2009.

If, in an eccentrically filled, highly flexible bin, mixing effects in different zones result in either different packing densities or cohesion of the bulk material, the asymmetric layering of bulk material particles can cause an asymmetrical central flow. This leads to the creation of zones in the bin where the bulk material flows along the bin wall, causing asymmetrical loads. For such cases, special load distribution techniques are required.

The calculation of bunkers involves determining the pressure of bulk material on individual planes of the bunker structure and the bending moments and forces acting in the plane of the bunker faces.

The pressure of bulk material on the walls of the bin is determined using the theory of bulk pressure in an unlimited mass, according to which at any point on the wall there are two components of bulk pressure:

– vertical pressure $q^{н}$, which depends on the average density of the material γ and from the distance h the point in question from the top of the backfill:

$$q^{н} = q^{н} \gamma h; \tag{2.1}$$

– horizontal pressure $p^{н}$, which is proportional to the vertical:

$$p^{н} = q^{н} \cdot tg^2\left(45^0 - \frac{\varphi}{2}\right), \tag{2.2}$$

where φ – angle of natural repose of bulk material.

Knowing the values of pressure on two mutually perpendicular surfaces, one can determine the pressure on any inclined plane passing through the point in question at an angle α to the horizontal. Based on Figure 2.9, normal $p_n^{\text{н}}$ and tangent $p_t^{\text{н}}$ the pressures at this point are equal to:

$$p_n^{\text{н}} = q^{\text{н}} \cos^2 \alpha + p^{\text{н}} \sin^2 \alpha = q^{\text{н}} \left(\cos^2 \alpha + k \sin^2 \alpha\right); \tag{2.3}$$

$$p_t^{\text{н}} = q^{\text{н}} \sin \alpha \cos\alpha - p^{\text{н}} \sin \alpha \cos\alpha = \left(q^{\text{н}} - p^{\text{н}}\right) \cdot \sin \alpha \cos\alpha \, . \tag{2.4}$$

Since pressure varies linearly, to plot a pressure diagram on an inclined wall, it is sufficient to determine the pressure at any two points. To plot a pressure diagram on a vertical wall, it is sufficient to determine the pressure at only one point.

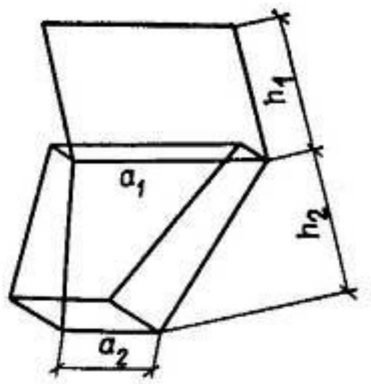


Figure 2.10 – Calculation of trapezoidal slabs

To use existing tables for calculating slabs supported along the contour, the variable load applied to the bin walls is converted to an average uniformly distributed load. The vertical walls are then calculated in the same way as for coffered floor slabs. The average pressure is determined by dividing the total load acting on the slab by its area.

In general, for a trapezoidal plate (Fig. 2.10):

$$p_n^c = \frac{1}{3} \gamma h^2 m \cdot \left(1 + \frac{a_2}{a_1 + a_2} + 3 \cdot \frac{h_1}{h_2}\right). \tag{2.5}$$

at $a_2 = 0$, i.e. for a triangular plate,

$$p_n^c = \frac{1}{3} \gamma \, m \cdot \left(h_2 + 3h_1\right), \tag{2.6}$$

at $a_2 = a_1 = 0$ (rectangular slab)

$$p_n^c = \gamma \, m \cdot \left(\frac{h_2}{2} + h_1\right), \tag{2.7}$$

where $m = \cos^2 \alpha + k \sin^2 \alpha$;

$$k = tg^2\left(45^0 - \frac{\varphi}{2}\right).$$

Tensile horizontal forces in vertical walls are determined by the formulas

$$N_a = \frac{p_n^c b}{2}; \qquad N_b = \frac{p_n^c a}{2}, \tag{2.8}$$

where p_n^c – average normal pressure on a vertical wall;

a, b – dimensions of the bunker cell in plan.

The hopper walls are trapezoidal or triangular in shape. In a symmetrical square hopper, they can be accurately calculated for out-of-plane bending as rigidly clamped slabs along the contour (ribs). For use in tables, their load can also be converted to uniform distribution, and the slab shape can be transformed to rectangular. The calculated dimensions of the given rectangular slab are determined by the formulas:

– for a triangular plate with a base a and a height h:

$$a_{расч} = \frac{2}{3}a; \quad h_{расч} = h - \frac{a}{6}; \tag{2.9}$$

– for a trapezoid with bases a And b ($a > b$) and height h:

$$a_{расч} = \frac{2}{3}\frac{(2b+a)\cdot a}{a+b}; \; h_{расч} = h - \frac{(a-b)\cdot a}{6\cdot(a+b)}. \tag{2.10}$$

In some cases there is no need to replace the trapezoidal or triangular load with a uniform one, since corresponding tables have been compiled for these load cases.

The calculation of vertical wall slabs and the funnel of an asymmetrical rectangular bin can be performed in the same manner. However, in this case, the clamping moments of the slabs mating at the same edge will be different in magnitude. For calculations, a balanced moment should be used, assumed to be equal to half the sum of the clamping moments of the adjacent slabs mating at the same edge.

The calculation of the bunker funnel walls for bending can also be performed using tables for calculating triangular or trapezoidal plates rigidly clamped along the contour.

In addition to bending, the funnel walls experience tension in their planes in two directions. In the horizontal direction, the tensile forces per unit wall length, measured along the slope, are determined by the formulas

$$N_a = \frac{p_n^c b}{2}\sin\alpha; \qquad N_b = \frac{p_n^c a}{2}\sin\alpha, \tag{2.11}$$

where a, b – the dimensions of the funnel in plan at the level of the strip in question are poured in.

In an asymmetrical bunker hopper, the horizontal tensile forces at the ends of the slab strip will differ in magnitude, as the slope angles of the opposite walls of the hopper are different. In this case, the calculated tensile force at the midspan of the slab should be taken as half the sum of the end tensile forces.

Tensile forces tending to separate the funnel from the vertical walls are generally distributed unevenly around the perimeter of the joint in the case of an asymmetrical funnel, since the center of gravity of the funnel volume does not coincide with the center of gravity of the perimeter of the joint (Fig. 2.11). These forces per unit length of the perimeter can be determined using the formulas for eccentric tension.

$$N = \sigma\delta = \frac{G}{2\cdot(a+b)}\cdot\left[1 \pm \frac{12e_y\cdot(a+b)\cdot y}{a^3+3a^2b} \pm \frac{12e_x\cdot(a+b)\cdot x}{b^3+3ab^2}\right], \tag{2.12}$$

where σ – tensile stress at an arbitrary point of the mating contour;

δ – wall thickness (constant around the entire perimeter);

G – weight of the contents of the bin and funnel.

Plus signs in brackets are accepted when the coordinate signs x, y the points of the contour under consideration coincide with the signs e_x And e_y.

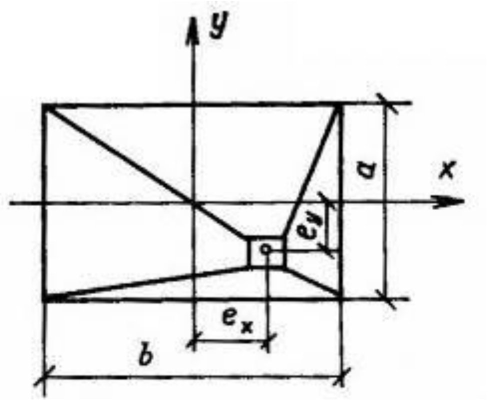


Figure 2.11 – To determine the forces in an asymmetric funnel

In order to determine the forces along the slope of the funnel wall, the vertical forces obtained using the formula given above should be divided by the sine of the corresponding angles of inclination of the funnel walls to the horizon.

The forces at any level of the funnel are also determined by this formula, followed by dividing the obtained values by the sines of the wall inclination angles. It should only be replaced a, b, e_x And e_y take the corresponding dimensions of the funnel in the section under consideration, and G shall be taken as equal to the sum of the weight of the column of material above the section in question (within the plan of this section), the weight of the part of the funnel and the bulk material located below the section in question.

When an asymmetrical bunker is supported by four columns located at the corners, the load on the columns is determined by the formula

$$N = \frac{G}{4} \cdot \left(1 \pm \frac{2e_x}{b} \pm \frac{2e_x}{a} \right). \tag{2.13}$$

The rule for signs in this formula is the same as for the previous formula.

Limit state analysis of structures is known to be based on the consideration of limit state forms, or failure modes. Some of the possible failure modes for stand-alone bunkers are shown in Figure 2.11. The diagram in Figure 2.11, a, illustrates the failure of each individual bunker slab "along the envelope," i. e., the reinforcement calculation in this case is performed as for slabs supported along the contour, under the effect of a uniformly distributed load. Other modes (Figures 2.11, b, c, d, e) correspond to bunker rupture along a vertical crack strip, separation of the funnel from the main bunker shell, and the possibility of the entire bunker collapsing along normal and inclined sections.

The mutual influence of adjacent walls on each other is taken into account approximately, assuming that along the line of their contact they are in a clamped state.

In addition to bending, the walls of the bunker are subject to tension: vertical walls – in the horizontal direction, inclined walls in the funnel – in both directions.

Trapezoidal slabs are approximately designed for an average uniformly distributed load with an intensity of

$$p_n^c = \frac{1}{3} \cdot \gamma \cdot h_2 \cdot \left[\cos^2 \alpha + \sin^2 \alpha \cdot tg^2 \left(45^0 - \frac{\varphi}{2} \right) \right] \cdot \left(1 + \frac{a_2}{a_1 + a_2} + 3 \cdot \frac{h_1}{h_2} \right), \qquad (2.14)$$

replacing the trapezoidal outline with a rectangular one with calculated side dimensions (Fig. 2.19).

$$l_{расч} = \frac{2}{3} \cdot (a_1 + 2a_2) \cdot \frac{a_2}{a_1 + a_2}, \qquad (2.15)$$

where α – the angle of inclination of the funnel wall to the horizontal.

2.3 Design of bunker components

Reinforced concrete bunkers offer two fundamentally different design options: monolithic and precast. A precast-monolithic solution is also possible. The bunker's design is significantly influenced by its intended use (mining, metallurgical plants, thermal power plants, chemical production, light industry, etc.), as well as the composition of the bulk material filling the bunker (sand, crushed stone, ore, coal, lime, soda, clinker, cement, etc.).

When choosing a design solution for a bin of a cellular, trough or multi-cell and combined type, the first thing to decide is whether the bin is supported by supporting structures, then factors such as the bin loading method, unloading method, loading and unloading equipment used, bin operating conditions (temperature, humidity, operating mode, presence of vibration, seismicity of the construction area), the influence of an aggressive environment and other circumstances.

The design of bunkers begins with the development of the foundation and an assessment of specific soil conditions. Most often, freestanding monolithic columnar foundations are used, supported by monolithic or precast reinforced concrete columns. The column spacing can range from 2,5 to 6 mand more. Reinforced concrete beams are laid on the columns, directly supporting the vertical walls of the bin. In addition to columns, longitudinal and transverse reinforced concrete and brick walls can also serve as supporting structures for the bins. In this case, the foundations for the walls are monolithic reinforced concrete strip foundations.

Figures shows an example of the design solution for a monolithic reinforced concrete cell-type bunker for a thermal power plant bunker rack for finely crushed coal. The cell sizes reach 6–8 min plan, height – up to 12–18 m. Trough bins are designed for even larger sizes and handle significant volumes of bulk materials. The example bin shown is a four-level design with a loading gallery, an unloading platform, and a sub-bin floor. The bin wall thickness is 120 mm, the thickness of the funnel is 100 mm. Concrete of class C20/25 was used, with a water resistance grade of W6, i.e. a fairly high density.

TOPIC 3 SILOS

3.1 General information

Reinforced concrete silos are used for bulk, typically long-term, storage of various fine-grained bulk raw materials, semi-finished and finished products, such as cement, lime, coal, crushed ore, sand, clinker, grain, and others. This type of enclosed storage facility is widely used due to its compact layout, operational reliability, and the high level of possible mechanization of loading and unloading operations. Silos essentially expand the dimensions of bunkers, significantly increasing their volume and height.

In everyday practice, silos are commonly used to refer to storage facilities with a wall height $H > 1{,}5D_{мин}$, $D_{мин}$ – a smaller silo cross-section. This ratio requires taking into account the frictional forces of the bulk material against the silo walls. Silos are typically designed as round silos, but polygonal, rectangular, fluted, and even elliptical cross-sections are also common.

The walls of circular silos operate in almost pure tension in the circumferential direction, allowing for smaller wall cross-sections and material savings. Figure 3.1 shows the main cross-sectional shapes of silos used in construction.

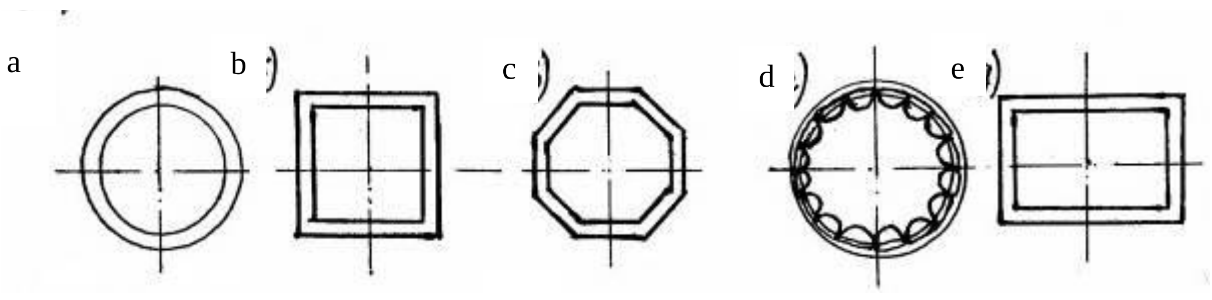


Figure 3.1 – Possible cross-sectional shapes of silos:
a – round; b – square; c – polygonal; d – fluted; e – rectangular

Reinforced concrete silo buildings typically consist of several silo bins. Freestanding silo bins are rare. The bins can be arranged in rows, with direct contact between them, or staggered. A row-by-row arrangement is recommended as the simplest and easiest to manufacture. The distance between silos in this case is called a “star”. A staggered arrangement is only permitted in certain cases, such as when expanding or renovating existing silo buildings.

In some cases, silo buildings can be manufactured with separate banks, located at a certain distance from each other – 0,8–1,2 m. This solution allows for pre-stressing of the cans using winding machines. The layout of the silo buildings is shown in Figure 3.2, a, b, c.

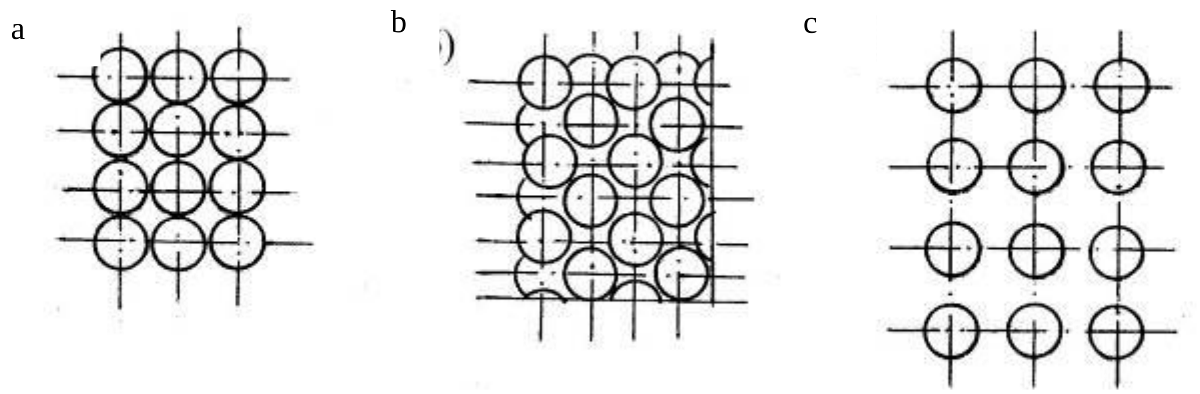


Figure 3.2 – Layout diagrams of silo body cans:
a – ordinary; b – checkerboard; c – free-standing

Based on their bottom design, silos can be either with or without sub-silo floors. The choice of one type or another depends primarily on the properties of the bulk material and the type of unloading equipment. Silos of types I–IV (see Fig. 3.3) do not have sub-silo floors and are therefore more economical than types V–VII. Silos of the first three types are used to store sand, gravel, lime, soda ash, and cement. Unloading of type I silos is accomplished pneumatically, i.e., by supplying compressed air to the upper layers of the bulk material.

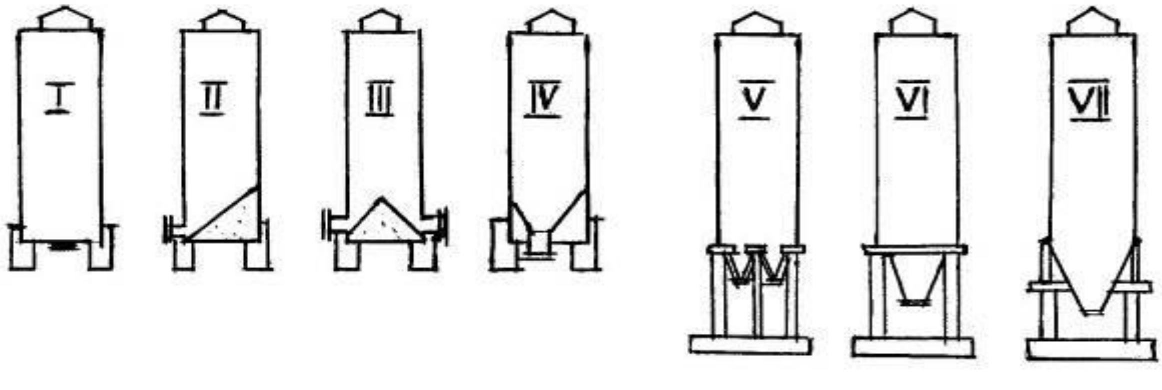


Figure 3.3 – Types of silos with different organization of unloading devices

For types II and III, unloading is carried out by gravity, through side outlets directly into railcars or trucks. Material stored in such silos must not exhibit caking or arching properties. Silos of types V–VII are used for storing any bulk solids (clinker, ore, expanded clay, etc.). They are the most convenient to operate, but are also the most expensive.

The practice of using round silos has shown that the most rational and economical parameters for the outer diameter are the following dimensions (in m):

– for cement – 6, 12, 18;

– for coal – 12;

– for soda (calcined) – 18;

– for heavy grain (wheat) – 6;

– for light grain (sunflower) – 12, 18.

For silos that are square in plan, the optimal side size is 3–4 m. Currently, standard series have been developed for the construction of both silo buildings with round banks and silo buildings with rectangular and square banks, which are called elevators or grain receiving (grain) elevators.

There are so-called “granaries” – storages for standardized raw material stocks for enterprises and construction sites. In most cases, these are rectangular containers, interconnected into a single structure. Their height can be 3.6, 4.8, and 6.0 m. These containers are open and can be buried or located on the ground surface. Bins can be considered a special case of silos.

Silos, either separate or combined into buildings, are part of expanded production facilities: industrial (silos for cement, coal, soda, etc.) or agricultural (elevators for grain, seeds, flour, pellets, etc.).

At the top of the silo building, as a rule, a gallery is provided for loading equipment, and at the bottom – a sub-silo room for unloading the contents into transport mechanisms.

Silos are constructed as monolithic (Fig. 3.4) or precast (Figs. 3.5 and 3.6). Each of these options has its advantages and disadvantages. Monolithic silos require no additional transportation or factory costs, nor do they require the use of heavy cranes. The location of such silos can be freely chosen, independent of the availability of a precast concrete production facility; this ensures increased reliability and operational safety. However, the technological process of silo bin manufacturing, the installation and movement of formwork, the maintenance of concrete curing times, and the arrangement of a storage area for building materials and process equipment are all more complex.

Prefabricated silos significantly reduce construction time and improve the quality of the silo bins. Construction is not dependent on the time of year (summer or winter), and the reduced cross-section of the silo walls reduces overall labor costs and construction material consumption. However, the prefabricated option requires a manufacturing facility, such as a precast concrete plant. This reduces the durability and reliability of the structures due to the presence of butt joints and a large number of nodal connections. Additional transportation services (delivery of prefabricated structures to the site) and lifting equipment, such as heavy-duty assembly cranes, are required.

The total estimated cost of prefabricated silos is higher than the cost of monolithic silos of similar parameters.

Silos belong to a category of engineering structures with a variable payload of tens of thousands of tons, and taking into account the structure's own weight, the total load on the ground can reach hundreds of thousands of tons. This poses a number of complex engineering challenges that must be addressed, taking into account the actual ground conditions, the uneven loading of individual sections of the silo structure, the method of unloading each bin, the manufacturing technology, and other factors.

Current practice in the design, construction, and operation of silo buildings shows that at various stages of the life cycle of these engineering structures, a number of unforeseen circumstances arise, leading to either localized failure of individual structures or the collapse of the entire silo building. Since 1932, more than 25 serious accidents have been recorded worldwide, resulting in the complete failure of silo buildings.

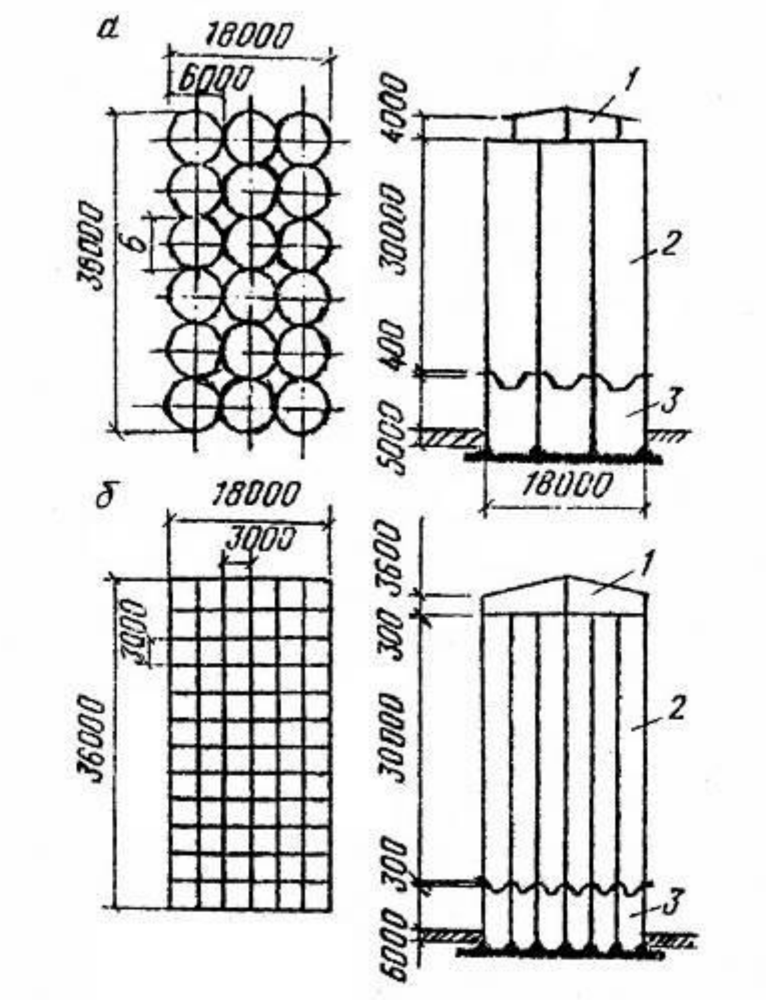


Figure 3.4 – Monolithic silo buildings:

a – with cylindrical silos; b – with square silos;

1 – gallery above the silo; 2 – silo; 3 – sub-silo floor

The height of the silo buildings ranges from 30–36 m, and in some cases can reach 42 m and more.

It should be noted that the regulatory documents for the design of silos allow for the settlement of these engineering structures 40 cm. Currently, the main regulatory documents for the design of silos and elevators are the standards developed back in 1984–1985. At the current stage of silo design and construction, they are already outdated, but the basic conceptual provisions of these standards remain relevant today.

3.2 Design solutions for standardized silos

Given the high demand for silo-type storage facilities across various industries and agriculture, the most efficient and suitable standardized design schemes for these engineering structures, suitable for various construction conditions, have been developed over many years. One of the first standard designs for cement storage was a silo designed in 1948 by Giprocement for six cans with a diameter of 6 m each one made of monolithic reinforced concrete.

The most popular series for the construction of silos in the 60-70s of the last century was the IS-01-09 series (later replaced by 3.012-3), which considered the designs of reinforced concrete silos with a diameter of 6 and12 mfor storing bulk materials.

For example, in the designs of typical silos developed in the IS-01-09 (3.012-3) series, the following components are considered: foundation, sub-silo floor, bottom with funnels, silo can walls, ceiling above the silo and above-silo gallery.

These series standardize only the main parameters of silos: the dimensions of silo cans in cross-section, height, and relative position; the heights of sub-silo floors, column sections, and their axial connections; the thickness of monolithic walls erected in sliding formwork; and various designs of over-silo floors (over-silo slabs).

The outer (nominal) diameter of round silos is taken to be 3, 6 and 12 m. The distance between the silo centers is set equal to the standard value of their diameter. The height of the silo walls and sub-silo floors is assigned as a multiple of 0,6 or 1,2 m. To simplify the design of the equipment and reduce the number of its standard sizes, the monolithic walls of the outer and middle silos, erected in sliding formwork, are adopted to be of the same thickness.

Additional loads arising from silo unloading, pneumatic system operation, and bulk material collapse within the silo are taken into account separately in the design (when selecting element cross-sections). The temporary design load on the silo floor is assumed to be 500 kg/m² (5 kN/m²).

The volumetric layout solutions for elevator silo buildings are also standardized in most cases. Designs have been developed for single and group silos, single-row and double-row silos, with diameters6 m, with a full body height of 15–25,8 m, with a capacity of 250–3 000 m^3, as well as silos with a diameter12 m, height 24,6–42,6 m, with a capacity of 1 700–12 000 m^3.

For grain elevators, silos of four types with plan dimensions of 36 are recommended for use × 24 m, 30 m × 18 m, 36 m × 18 m and 24 m × 18 m. The length of the body may be longer, but it should not exceed48 m for round and 42 m for square silos. This limitation is dictated by the need to install temperature and shrinkage joints. The typical silo height is 30 m, on high-strength soils (rocky) it can be increased to 42 m, and in some cases even more.

In housings consisting of cylindrical or octagonal silos, the space between the cylinders or octagons ("stars") is also used for storing bulk materials.

Among the most commonly used, it is worth noting the standard design 702-1-3 – a three-row silo body of the SKM-6-18 type with a capacity of 11,700 tons with monolithic silos with a diameter6 m. This building includes three blocks with plan dimensions of 18 m × 24 m, body height – 30 m.

In addition to monolithic versions of standard series for elevators, a number of design solutions have been developed using precast reinforced concrete structures. For example, series 702-1-16-90 and 702-1-17-90 utilize precast cement blocks of the SKS-3x144 type with structural protection of the external walls. The capacity of these silos ranges from 18 000 to 27 000 tons of grain. Structural reinforced concrete components of the component parts, including foundations, columns, silo walls, and floor slabs, are developed in series 3.702.1-4. It should be noted that the wall thickness of precast block walls can be as thin as 60–90 mm.

The most ambitious design is standard project 702-17 for a grain elevator with a total capacity of 140 000 tons of grain. This design is based on the use of precast reinforced concrete block cells measuring 3 mm for the cans. × 3 m and includes six independent block sections with dimensions of 18×30 m. The grain storage capacity for this elevator can reach 75 000 tons (i.e., grain preparation, sorting, drying, etc.).

This elevator includes auxiliary buildings and structures, architecturally and structurally designed for the planned volume of received grain.

The walls of round monolithic silos typically extend to the foundation slab. In the sub-silo area, the walls are reinforced with pilasters, which support the hoppers from above (Fig. 3.5, a). Flat bottoms are also installed on their own columns with backfill along the top (Fig. 3.5, b). Prefabricated round silos, together with the hoppers (which can also be prefabricated), are supported in the sub-silo area on U-shaped frames (Fig. 3.5, c). Support for square silos is similar.

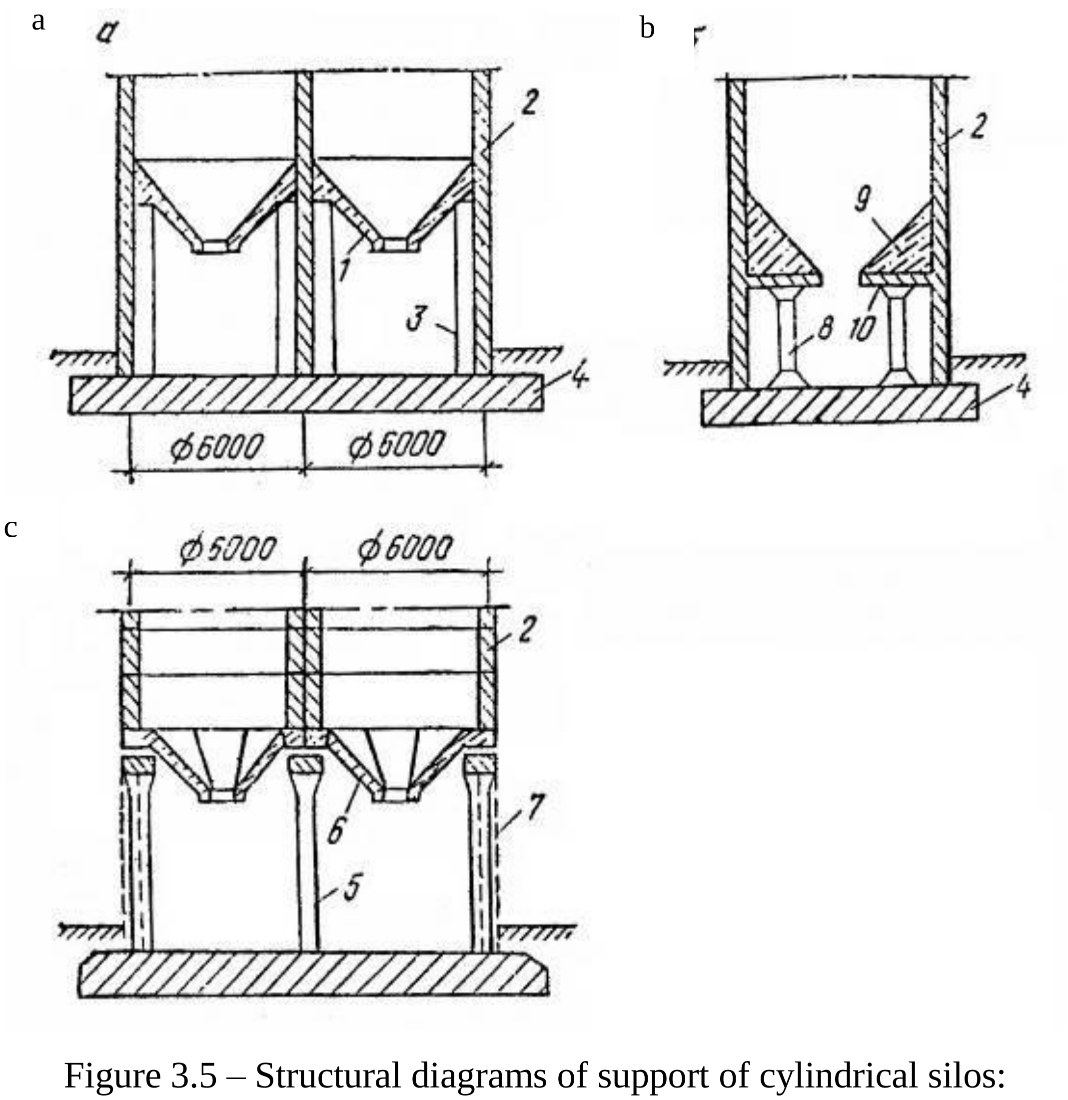


Figure 3.5 – Structural diagrams of support of cylindrical silos:
a – monolithic with monolithic funnels; b – monolithic with a flat bottom;
c – prefabricated with prefabricated funnels;
1 – monolithic funnel; 2 – silo walls; 3 – pilasters; 4 – foundation slab;
5 – U-shaped frames; 6 – collection funnel; 7 – wall enclosure;
8 – columns; 9 – backfill; 10 – flat bottom

The walls of monolithic silo buildings are usually erected in sliding formwork held on jack frames, therefore the walls are reinforced in the horizontal direction with individual rods of relatively short length with a rod pitch of 100–200 mm; the thickness of the protective layer must be at least20 mm. The joints of the rods are made staggered with the ends overlapping by $60d$ + 100 mm. In small diameter silos, prestressing of the wall reinforcement is not performed; class A400C reinforcement (periodic section) is used for reinforcement.

A similar layout is also used in prefabricated silo buildings (Figs. 3.5 and 3.6). They are assembled from elements of three standard sizes of spatial block in the form of a closed frame, L-shaped, and flat. The nominal height of the prefabricated tier is 1,2 m. Prefabricated elements can be smooth in thickness 100 mm or ribbed with wall thickness60 mm. Individual elements are connected into a silo body using galvanized bolts.

Small diameter prefabricated cylindrical silos (3 m) can be assembled from solid rings, but such silos are rarely used. Prefabricated silos with a nominal diameter 6 m assembled as shown in Figure 3.5, a. Each tier consists of four elements connected with bolts (Figure 3.5, b). The prefabricated elements can be smooth (thickness100 mm) and ribbed (with a wall thickness of 60 mm and a rib height of 150 mm).

Prefabricated octagonal silos (Fig. 3.7) are constructed from a spatial block in the form of a closed frame and flat ribbed slabs. Bolted connections are also provided for the prefabricated elements. However, this design is not widely used.

Standard unified designs of prefabricated round silos of large diameter have been developed (12 m) from fluted shell panels (Fig. 3.7) with nominal widths 1.54 m and height 3 m. The panels are equipped with end ribs, the outer grooves of which accommodate prestressed ring reinforcement of the silo. This reinforcement is tensioned during the assembly of individual tiered collets on a special stand, in which the internal thrust is created by compressed air. After tensioning, the reinforcement is protected with a cement mortar applied using gunite.

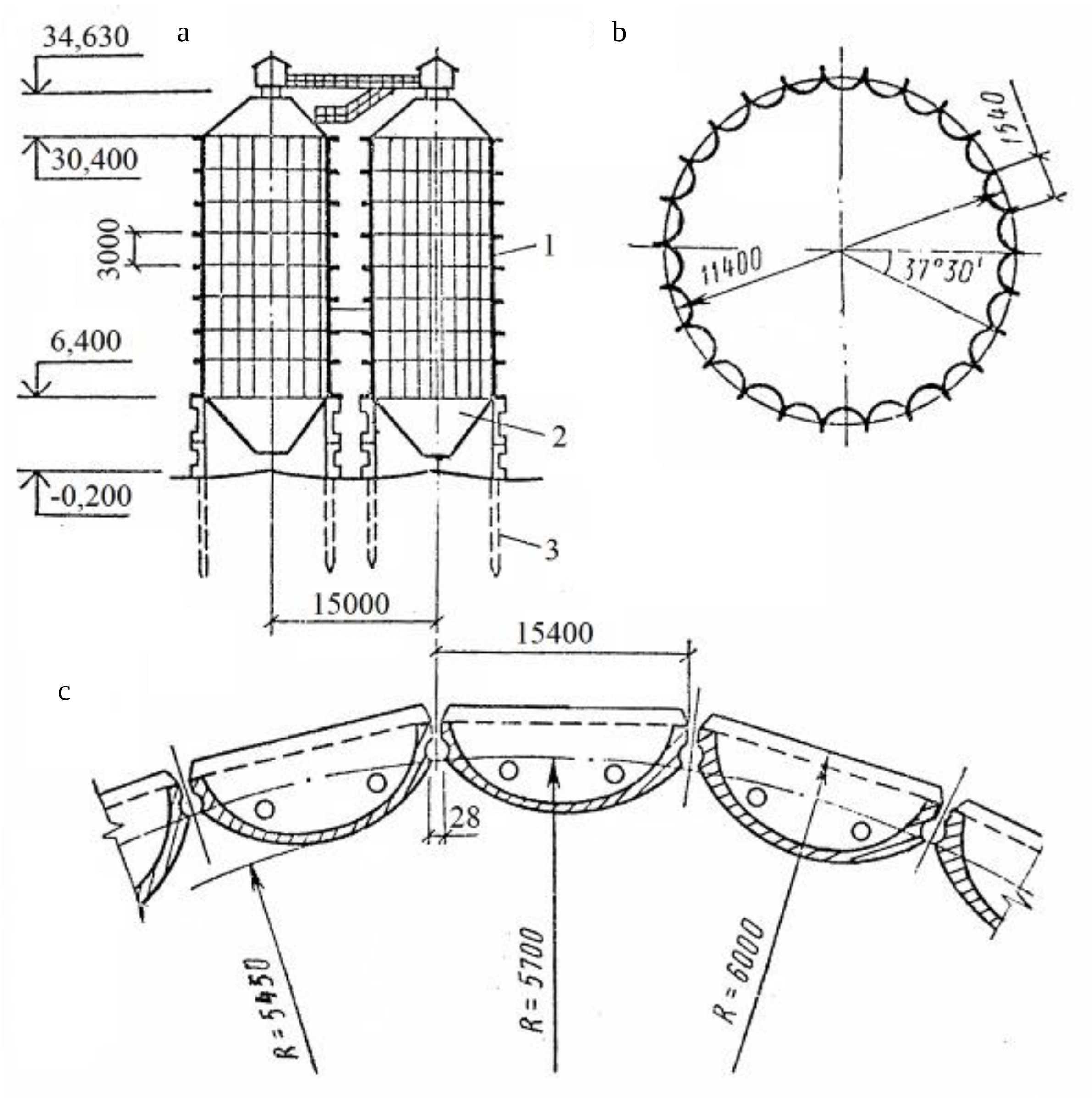


Figure 3.6 – Structural diagrams of a prefabricated round silo with a diameter12 m with fluted panels: a – section; b – plan; c – plan detail; 1 – shell panels; 2 – metal funnels; 3 – reinforced concrete piles

Bottom design for monolithic silos with a diameter of 12 m developed in two versions: for storing materials with an average density of more than 10 kN/m^3 – monolithic, ribbed, with holes for the device of half-funnels (Fig. 3.6); for storing light bulk materials ($\rho < 10$ kN/m^3) in each silo, a precast or monolithic ring beam is installed on columns, supporting the silo walls and steel hopper. Precast ring beams are assembled from four separate elements.

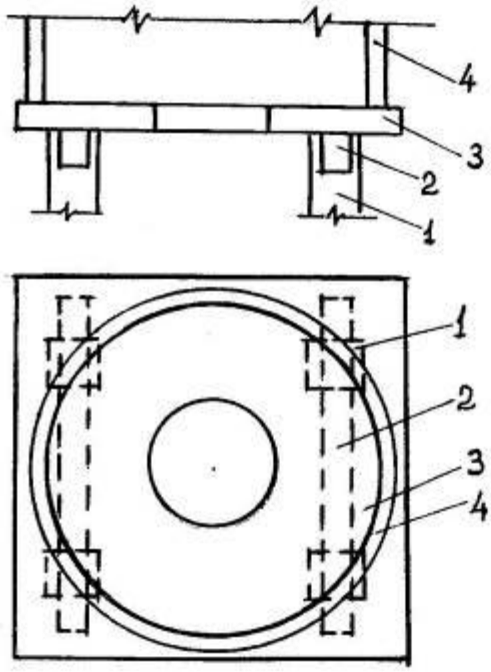


Figure 3.7 – Precast monolithic bottom of silo with a diameter of 6 m: 1 – columns; 2 – precast beams; 3 – monolithic slab; 4 – silo walls

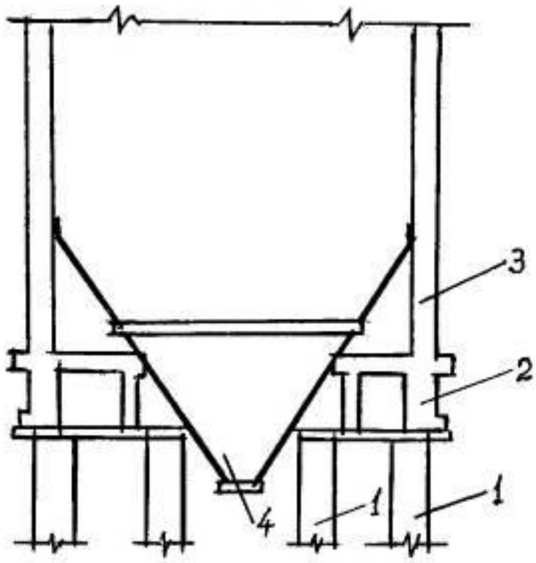


Figure 3.8 – Diagram of the ribbed bottom of silos with a diameter of 12 m: 1 – columns; 2 – monolithic ribbed bottom;3 – silo walls; 4 – funnel

The columns of the sub-silo floors in the unified silo building are designed mainly as prefabricated ones, with a rectangular cross-section of size (mm) 500 × 600; 600 × 800; 600 × 1 000 and others.

The floors (over-silo slab) are designed using flat precast reinforced concrete slabs with a thickness of100 mm, having nominal dimensions in plan 3 m × 3 m with corner extension plates 3 m × 1,5 m. The slabs are mounted on precast reinforced concrete or metal beams located at intervals3 min two mutually perpendicular directions. This beam arrangement evenly distributes the loads on the silo walls, making it easy to construct monolithic sections where necessary and to secure the frame supports of the over-silo galleries. A reinforced concrete layer (thickness) is laid on the precast slabs. 30 mm for silos with a diameter of 3 m and 40 mm for 6 and 12 m silos), which, together with the prefabricated slabs, forms a horizontal diaphragm, increasing the overall rigidity of the silo body.

Solid reinforced concrete slabs, slabs with protruding cross beams at the intersection of which columns are installed, as well as slabs with the installation of high- or low-type glasses for the installation of precast reinforced concrete columns under the walls of silo banks can be used as foundations for silo buildings.

3.3 Basic calculation provisions

When designing silos, loads from the dead weight of the structure, the weight of bulk materials, horizontal and vertical pressure of bulk materials on the walls of the silos, pressure of bulk materials on the bottom of the silos, the weight of process equipment, snow and wind, shrinkage, creep of concrete, temperature effects and reactive pressure of the soil base are taken into account.

Loads from the dead weight of structures and the weight of bulk materials are accepted in accordance with the instructions of DBN and SN 302-65. The safety factor for bulk materials is taken equal to 1.3. When calculating the compression of the lower zone of silo walls, sub-silo floor columns and foundation slabs, the calculated load from the weight of bulk materials is multiplied by a factor of 0.9.

The snow load is accepted in accordance with the construction area according to DBN B.1.2-2:2006, the wind velocity pressure should be not less than 0.70 kN/m^2. The safety factor for the snow load is 1.14, and for the wind load – 1.3; the aerodynamic coefficient for single silos is 1.0, and for blocked silos – 1.4. Calculation of temperature effects, shrinkage and creep of concrete is carried out according to a special method, their influence in simplified engineering calculations is taken into account by the coefficients of working conditions.

A key feature of silo calculations is the consideration of frictional forces between bulk material and the silo walls. These frictional forces increase the vertical load on the silo walls and, in turn, alter the magnitude of horizontal pressure. p .

In order to determine the magnitude of the horizontal pressure p in its characteristic value, we will use the method for determining the horizontal pressure p proposed by Jansen and Kenen at the end of the 19th century; the final formula, in honor of the authors, is called the Jansen-Kenen formula.

Let us consider the equilibrium of an elementary layer of loose material with a height dy at some depth in a round silo. The following loads act on this layer: a uniformly distributed load q from the vertical pressure of a column of bulk material of height y on the upper surface of an elementary layer; the layer's own weight $\rho A dy$, Where ρ – average density of materials in kN/m^3, A – layer area in m^2, dy – layer height; from below, a reactive force acts on this layer in the opposite direction $q+dq$ the lower column of bulk material; a reactive force arises along the perimeter of the layer due to frictional forces $R = p \cdot fUdy$, where p is the horizontal pressure (which is unknown), f – coefficient of friction of bulk material against a reinforced concrete wall, U – the perimeter of the ring of the bulk layer and dy its height.

Let us formulate the equilibrium condition for the elementary layer under consideration dy, projecting all vertical forces onto the y-axis:

$$qA-(q+dq)A+\rho Ady-pfUdy=0. \tag{3.1}$$

Expanding the brackets and reducing by A, we get:

$$-dq+\rho dy-pf\frac{U}{A}dy=0. \tag{3.2}$$

Attitude $\frac{A}{U}=r$ is called the hydraulic radius for a circular cross-section. Between the vertical pressure q and horizontal pressure p in bulk materials there is a certain relationship, the simplest of which is $p=qtg^2\left(45^0-\frac{\varphi}{2}\right)$, so instead of p you can substitute the value $p=k\cdot q$, then equation (3.2) can also be represented as:

$$dq=\left(p-q\frac{kf}{r}\right)dy \tag{3.3}$$

or

$$dy=\frac{dq}{p-cq}, \tag{3.4}$$

where $c=\frac{kf}{r}$ – factors of constant values.

Solving the differential equation (3.4), we obtain:

$$\int dy = \int \frac{dq}{p - cq}; \quad y = -\frac{1}{c}\ln(p - c\cdot q) + C_1 .$$

WITH$_1$– an arbitrary constant that is determined from the boundary conditions: when $y = 0$ magnitude $q = 0$; then $C_1 = \frac{1}{c}\ln p$ and the value of y can be determined as follows:

$$y = \frac{1}{c}\ln\frac{p}{p - cq}. \tag{3.5}$$

From the definition of the concept of logarithm let`s write:

$$e^{cy} = \frac{p}{p - cq} \quad \text{or} \quad e^{-cy} = \frac{p - cq}{p}.$$

After performing simple algebraic transformations, we obtain:

$$1 - e^{-cy} = \frac{c}{p}\cdot q$$

or

$$q = \frac{p}{c}\left(1 - e^{-cy}\right). \tag{3.6}$$

Substituting the value *With* into formula (3.6), we obtain the well-known Jansen-Kenen formula for the values of vertical and horizontal pressure:

$$q = \frac{pr}{kf}\left(1 - e^{-\frac{kf}{r}\cdot y}\right); \tag{3.7}$$

$$p = \frac{pr}{f}\left(1 - e^{-\frac{kf}{r}\cdot y}\right). \tag{3.8}$$

When deriving formulas for q and p do not take into account a number of factors of significant importance, namely: the flexibility of the can walls, the shape of its cross-section, the method of loading and unloading, and others. Therefore, correction factors are introduced into formulas (3.7) and (3.8) that partially take these factors into account. In particular, to the formulas for q and the coefficient a is introduced.

The value of the coefficient *A* In formulas (3.7) and (3.8), a = 2 is used to calculate the bottoms and the lower zone of the walls over 2/3 of their height and a = 1 for the upper third of the walls; when calculating the walls of silos for compression, as well as columns and foundations, a = 1. Adjustments are made to this coefficient depending on the type of bulk material. The vertical characteristic pressure transmitted through friction against the walls of silos is determined by the formula

$$q_{f,ser} = f \cdot p_{ser} \,. \tag{3.9}$$

The calculated values of the load from bulk materials are determined by the formulas

$$q = p_1 = p_{k1} \frac{\gamma_f}{\gamma_k} \,; \qquad p = p_2 = p_{k2} \frac{\gamma_f}{\gamma_k} \,, \tag{3.10}$$

where γ_f – load safety factor;

γ_k – coefficient of operating conditions of structures.

Coefficient γ_f for bulk materials it is equal to 1,3; when calculating the compression of the lower zone of silo walls, sub-silo floor columns and foundation slabs, the value of the calculated load from the mass of bulk materials is multiplied by a coefficient of 0,9.

When calculating the strength of the walls of round silos, the hoop tensile force N_2 in kN on 1 m heights are determined by the formula:

$$N_2 = \frac{a \cdot \gamma_f}{\gamma_k} \cdot \frac{p_{k2} D}{2} \,, \tag{3.11}$$

where γ_f and γ_k – the same coefficients as in formula (3.10);

p_{k2} – characteristic value of horizontal pressure determined by the Jansen-Koenen formula.

Cross-sectional area of horizontal reinforcement of cylindrical silos per unit wall height determined by the formula:

$$A_s = \frac{N_2}{\gamma_s f_{yd}} \,, \tag{3.12}$$

where γ_s – coefficient of operating conditions of reinforcement;

f_{yd} – design resistance of reinforcement.

Along the wall height, 10–15 sections are considered, for which their own reinforcement area is determined.

The wall of a silo of any shape in the vertical direction is compressed by a linear load:

$$N_1 = \frac{A(\rho y - p_{k1})}{U} \cdot \frac{\gamma_f}{\gamma_k}, \tag{3.13}$$

where ρ – average material density kN/m^3;

at– the distance from the upper edge of the silo wall to the section in question;

$p_{k1} = q$ – vertical characteristic pressure, determined by formula (3.7);

A and U is the area and perimeter of the silo cross-section.

The pressure in the bulk solid within the silo funnel is determined by the formulas (3.7) and (3.8) taking into account the correction factor a. Moreover, the reduction in the transverse dimensions of the silo within the funnel is not taken into account.

The characteristic normal pressure of bulk material on the inclined surface of the bottom of the funnel can be calculated using the formula:

$$q_{n\alpha} = p \sin^2 \alpha + q \cos^2 \alpha, \tag{3.14}$$

where α – the angle of inclination of the surface under consideration to the horizon.

The cell of a square monolithic silo is calculated at each tier of height as a closed frame, which is subject to internal pressure p_2. The wall is subjected to tension by forces $p_2 l/2$ and bending moments $p_2 l^2/12$ in the corners and $p_2 l^2/24$ -in the span.

In this regard, the walls of square or rectangular silos must be designed for eccentric tension with a tensile force of N and the bending moment M. The greatest influence of the bending moment M is felt in rigid corner joints. In the middle section, this influence is significantly less, but in any case, the walls are reinforced with double mesh to accommodate both the tensile force and the bending moment.

When selecting the sections of typical silo walls, the calculated concrete resistances are taken with the concrete working conditions coefficient γ_b, which is equal to 0,75 for silo walls erected in slipform, and 0,85 for precast elements concreted in a vertical position.

The calculation of monolithic or precast columns of the sub-silo floor is carried out for the maximum forces transmitted from the foundation slab, taking into account the bending moment arising due to the wind load, the tilt of the structure due to uneven settlement and possible displacement of structures during installation. The value of uneven settlement is taken with the tilt of the body, equal to 0,004. Moment in kN·m from the possible deviation of the top of the columns or displacement of the funnel elements during installation is determined by the formula: $M = 0{,}025N$, where N – column load in kN.

In this case, additional forces from the body's tilt are not taken into account. The column's design is based on a post with a bottom anchor at the top of the shoe and a hinged joint at the bottom of the sub-silo beam or silo bottom plate.

Thickness of the solid foundation slab d in cm is determined from the condition of ensuring that the concrete fully absorbs the entire transverse force according to the formula:

$$d = \frac{V}{0{,}75 f_{ctd}} + c, \tag{3.15}$$

where V – calculated transverse force in kN per 1 running meter of slab;

f_{ctd} – calculated resistance of concrete to axial tension, kN/cm^2;

with – the distance from the bottom plane of the slab to the center of gravity of the tensioned reinforcement, cm.

In addition to calculations for the first group of limit states, silos are required to be calculated for the second group of limit states, i.e., for suitability for normal and safe operation. These calculations primarily include, calculation of crack resistance of silo walls. According to DBN B.2.6-98:2009, the operation of silos is permitted with the formation of cracks, but the width of their opening is strictly limited and should not exceed 0,1–0,2 mm.

When calculating the crack opening, the characteristic tensile hoop force N_2 from the horizontal pressure p is determined without taking into account the coefficient 1,3. In this case, the long-term component of the force is taken to be equal to N_2, and the short-term component is equal to the value $N_{кр} = N_2\left(\frac{a}{\gamma_k} - 1\right)$; $\frac{a}{\gamma_k} \leq 1{,}25$.

The walls of circular or rectangular silos are calculated based on crack formation and width in accordance with the guidelines for tension elements. Design and operational experience has shown that for monolithic silo walls made of C16/20 grade concrete and A400C grade reinforcement (without prestressing), the following percentage of reinforcement is required: $0{,}75 \leq \mu \leq 1$ the crack opening does not exceed the permissible size 0,2 mm at characteristic load values.

According to the currently existing classification of the complexity of structures and the degree of consequences during their operation, silos belong to the class of consequences CC2 in accordance with DSTU-N B V.1.2-16:2013.

Particular attention is paid to the design and construction of silo buildings located in seismic zones. For these scenarios, special calculations are performed, taking into account both the total mass of the silo and the potential vibration acceleration, depending on the seismic zone and the height of the structure.

3.4 Methods of reinforcing silo elements

The most common. The reinforcement used for reinforcing walls, columns, floor slabs, foundation slabs and other elements of silos is non-stressed reinforcement of classes A400C, A500C and A240.

The walls of round silos with a diameter of 3–6 m are sufficiently reinforced in the upper zone with single horizontal reinforcement with a diameter of 8–12 mm with a step obtained as a result of the calculation; in normal practice, this step in the lower zone is 100–150 mm, in the upper third of the silo height it is equal to 200–250 m. Single horizontal ring reinforcement is used only at 1/3 of the height from the top of the silo; in the external and internal walls at 2/3 from the bottom of the silo, double ring reinforcement is required (Fig. 3.9, a, b), which is due to the perception of bending moments generated during the staggered filling of the silo body cans with bulk material.

The vertical rods are taken with a diameter of 10–12 mm in increments of 200–300 mm for external walls of silos and 300–340 mm – for internal ones. The total cross-section of vertical bars is assigned to be no less than 0,4 % of the concrete cross-section per 1 linear meter of the silo wall arc. Thus, for a silo wall thickness of 180 mm 7,2 cm^2 of vertical reinforcement is required, which corresponds to 10 rods with a diameter10 mm, installed in increments 200 mm on each side of the double ring reinforcement.

Some of the vertical rods are installed in the form of knitted frames (Fig. 3.9, c), located through 1–1,5 m along the length of the ring, which ensures the design position of the horizontal ring reinforcement during concreting. The joints of the vertical bars are staggered with a 35-degree overlap of the ends. d .

The vertical and horizontal rods are tied together with knitting wire at all points of intersection; with double reinforcement (see Fig.3.9, c) both grids are connected with transverse clamps with a diameter of 4–5 mm.

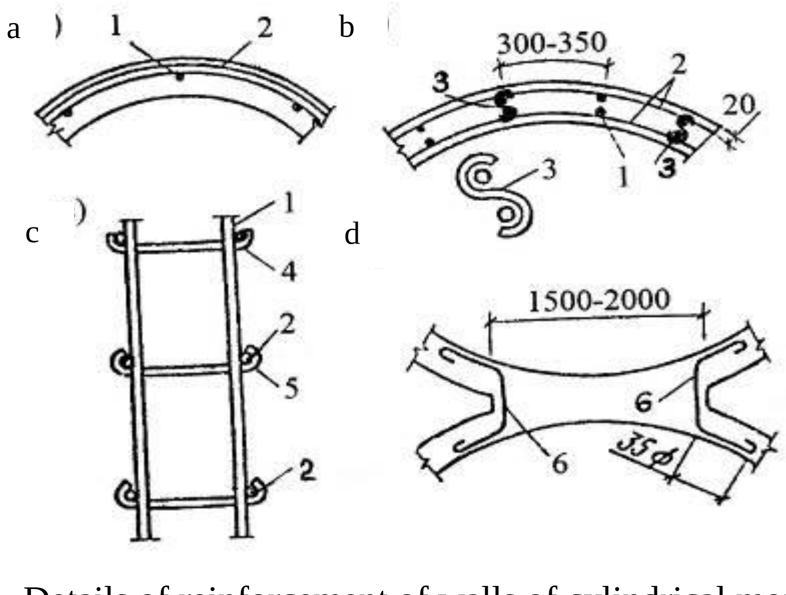


Figure 3.9 – Details of reinforcement of walls of cylindrical monolithic silos:
a – single; b – double; c – vertical knitted frame; d – additional reinforcement at the junction points adjacent silos; 1 – vertical rods; 2 – rods of the ring working reinforcement; 3 – connecting pins; 4, 5 – clamps before and after laying horizontal rods; 6 – additional rods

At the junction points of adjacent silos, the incoming corners are reinforced with additional rods (Fig.3.9, d), their diameter and pitch are the same as those of the main ring reinforcement. The ring reinforcement in these areas should overlap from two adjacent silo cans.

To ensure increased silo strength, external prestressed reinforcement is sometimes used, rolled onto a reinforced concrete can that has attained sufficient concrete strength. The concrete grade in this case must be no lower than C25/30. The reinforcement used is wire of classes B1300-B1400 with a diameter of 4–5 mm, rolled onto a cylindrical surface using winding machines. A special anchor unit is installed at the bottom, to which the starting end of the wire is secured. After the reinforcement is rolled, the entire surface of the silo can is covered with shotcrete with a layer thickness of 30–40 mm. Reinforcement is typically wound to a height equal to 2/3 of the silo's bottom. This method of reinforcing silos is particularly effective during intensive bulk material unloading, which creates additional dynamic loads.

Double reinforcement is always installed in the walls of square monolithic silos (Fig.3.10), taking into account that the pressure on the intermediate walls is possible from one side or the other and that the horizontal reinforcement must withstand moments in the corners twice as large as in the span (see Fig. 3.10). The diameter of the horizontal reinforcement is 8–10 mm class A400C (A500C), the step is determined by calculation and primarily depends on the height of the silo body and the filling material, usually it is taken equal to 100–200 mm. The vertical reinforcement is taken to be not less than10 mm, and its step is 200–300 mm, considering that the dimensions in terms of square or rectangular cans are small (3–4 m). To maintain the design position of the horizontal rods, connecting pins made of soft wire with a diameter of 5 – are installed between the vertical rods. 6 mm. For these purposes, wire of classes B500 or A240 can be used.

In foreign practice of building rectangular silo buildings, pre-stressed wire reinforcement with a diameter of 5 mm, stretched in two mutually perpendicular directions, for which purpose the prefabricated sections are provided with horizontal channels with a step 200 mm by the height of the sections.

In prefabricated silos of round or square cross-section, the reinforcement principles outlined for monolithic silos are retained, with the only additional consideration being the vertical joining of the prefabricated elements. Joints can be made of. Embedded parts connected with galvanized bolts are used. Prefabricated reinforced concrete wall elements are manufactured in factories, allowing them to be reinforced with high-strength wire reinforcement, thereby reducing the overall steel consumption per assembly element.

The design of the silo building's foundation deserves special attention. Of all the reinforced concrete elements of the silos, the foundation is the most complex and labor-intensive. How rational is it? The foundation is designed technically competently and soundly, so the entire silo building will be used reliably and for a long time.

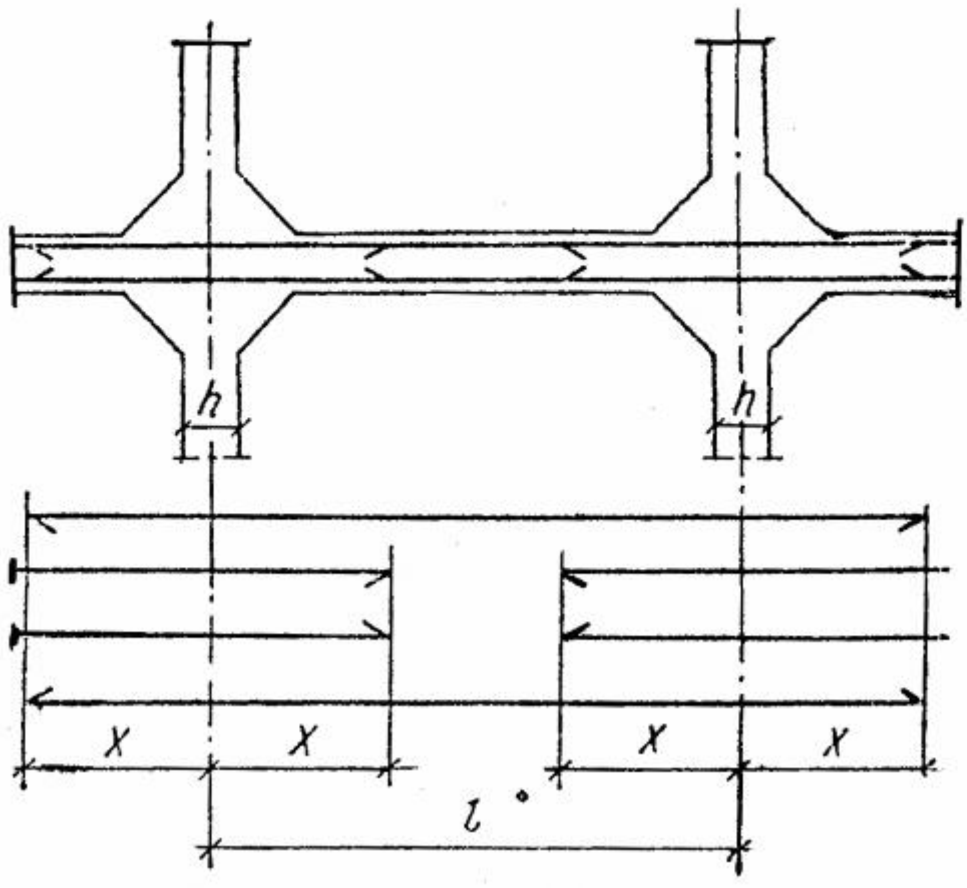


Figure 3.10 – Reinforcement diagram for walls of square monolithic silos

The foundation base depth is determined based on technological requirements and engineering and geological survey data, engineering and geological surveys must take into account the requirements necessary for the design of foundations for silo buildings: the geological and lithological section at the site of the construction of the structure must be investigated to a depth of 25 m. From the foundation base or to practically incompressible soils (if highly compressible soils exist throughout the entire depth, the type of the nearest incompressible soil and the position of its roof should be determined). It is also necessary to determine the bearing capacity of the foundation based on soil stability, taking into account the slope of strata, mine workings, slopes, and other factors that may cause instability of the foundation under the silo building. A pile foundation should be considered the most reliable foundation for silos [2].

Silo foundations are typically designed as a solid monolithic slab with cup-type columns. Foundations can also be designed as slabs and beams for each silo, or as individual footings for each column if the foundation is made of particularly strong, virtually incompressible soils. It is recommended that the slab thickness be chosen to eliminate transverse reinforcement with bent bars and stirrups. Slab thickness should be between 0,8–1,5 mand more. The average physical and mechanical characteristics of soils are as follows:

– for sandy soils: $\varphi = 32^0$; E = 24 MPa; C = 0,00;

– for clay soils: $\varphi = 21^0$; E = 20 MPa; C = 0,03 MPa (30 kN/m^2).

Reinforcement of silo slab foundations is performed after appropriate static and dynamic calculations, taking into account the combined action of the entire silo building and the slab. Structural requirements for reinforcement are the same as for conventional reinforced concrete slabs and beam elements.

Figure shows a variant of reinforcement of a monolithic slab-beam foundation for a silo building for storing cement with six silo cans with a diameter6 m. Slab height 900 mm, height of cross beams – 1,2 m. A distinctive feature of the slab section reinforcement is the intensive reinforcement of the lower zone, due to the significant bending moments at the column locations. The diameter of the working reinforcement in two mutually perpendicular directions is 28 mm, reinforcement class – A400C, rod pitch – 200 mm. In total, three meshes are laid in the lower zone of the slab. In the upper zone of the slab, that is, between the rib beams, two meshes of reinforcement with a diameter of 25 mm, step by step 200 mm, class A400C. It should be noted that the reinforcement pitch in 200 mm is the most common due to its ability to ensure reliable concreting of the entire slab, a simplified mesh manufacturing scheme, and the ability for workers to move along the mesh surface. In some cases, the reinforcement spacing is thickened to 100–150 mm. However, this can ultimately lead to excessive reinforcement consumption. It is most practical to connect the bars with cross-shaped tying wire; diagonal tying is unreliable and does not provide sufficient strength for the nodal connection. Welding is also possible, but it is energy-intensive and significantly more expensive than using tying wire. To ensure the design position of the upper meshes, support frames or support brackets are used. The pitch of the support frames is 400–600 mm. The support brackets are installed in a staggered pattern with a 600 mm × 600 mm cell spacing.

The longitudinal and transverse beams are reinforced with welded cages located above the mesh level of the lower zone of the slab. The working rods of the beam cages are taken within the range of 20–28 mm, their height arrangement is carried out in such a way as to allow the passage of beam rods perpendicular to each other.

At the intersections of the beams, platforms are installed for monolithic reinforced concrete columns, so-called pedestals. Reinforcement bars with a diameter of 22–32 mm for future columns.

Despite all the advantages of reinforced concrete silos (durability, reliability, fire resistance, and seismic resistance), they also have a number of significant disadvantages, namely, significant construction time, increased labor intensity, and high cost. Therefore, modern silo construction practices utilize metal, plastic, and even wood structures.

The K-150 collapsible silo of this design has a capacity of 1000 m^3 consists of ten cells, the volume of each is 100 m^3. Storage facilities without a sub-silo floor have also been developed with various cross-sectional plan shapes. The most common are rectangular, hexagonal, and dodecagonal cells. Silo blocking and the number of rows in a grain storage facility can also vary. Blocked silos reduce metal consumption by 25–30 % compared to single silos. A two-row arrangement of cells is optimal from a technological standpoint.

The walls of cylindrical or conical metal silos are smooth, while those of rectangular or polygonal types are made of corrugated sheet metal. To increase rigidity, vertical posts are installed in the corners of rectangular silos and connected with horizontal braces. To increase the rigidity of smooth silo sheets, they are stamped with cylindrical indentations spaced vertically at a certain distance from each other.

In foreign construction practice, preference is given to metal silos of various designs. Silos made from rolled sheet steel are widely used. The Butler company (USA) produces and builds metal cylindrical silos (with a diameter of 36,6 m; H = 13,7 m) and square silos (4,25 m × 4,25 m) with a capacity of 6,6–56 m^3. They are made of galvanized steel or aluminum with bolted connections. In England, cylindrical silos are manufactured from stamped corrugated steel panels with a galvanized coating, which are bolted together at the vertical joints with rubber gaskets. Square silos are also manufactured from prefabricated corrugated steel panels assembled with bolts. The capacity of such silos is 100–500 tons.

In recent years, Germany, Japan, and the UK have been using combined flexible grain silos with a metal frame and a wall made of synthetic material (plastic film, butyl rubber, etc.), as well as silos made of fiberglass with stiffening elements made of hollow posts.

In Hungary, large-capacity plastic silos are being built, which are made from individual elements based on polyester resins, 2 m wide and 2 m high. 14 m. The element is curved across its width, has two layers, and is reinforced with steel wire. To seal the vertical joints, strips of fiberglass impregnated with a binder are glued.

Despite all the advantages of easily erected metal and plastic silos, their service life is limited to 20–25 years, meaning these types of storage facilities will need to be rebuilt. Therefore, reinforced concrete silos, with a projected service life of 100–120 years, will remain relevant and in demand today and in the foreseeable future.

TOPIC 4 TOWER-TYPE STRUCTURES

The structures discussed in this section serve various purposes, but structurally they are all tower-type structures. For these structures, the primary load is their own weight and wind pressure. The design wind load, which varies along the height of the structure, is determined taking into account the dynamic effect. Pulsations of dynamic pressure caused by wind gusts. In flexible structures of cylindrical or square shapes, wind action causes vibrations perpendicular to the direction of its flow. Therefore, for such structures, in addition to calculating the dynamic pressure taking into account pulsations, a verification calculation for resonance is also performed. Additionally, tower-type structures are calculated for the vertical bending moment caused by the curvature of the tower axis due to the wind.

Tower-type structures include chimneys, water towers, radio relay and radio television towers, power transmission line supports, airfield control towers, marine lighthouses, cooling towers, and other structures. Each type of these engineering structures has its own specific calculations, design features, and operating conditions, so specific standards and recommendations for their reliable and safe operation are used for these structures.

It should be noted that most tower-type structures exhibit increased deformability, especially in the upper half of the structure, which leads to significant curvature of the vertical axis. The manifestation of deflection of the structures in question, caused by wind loads, temperature effects, and foundation rotation, leads to the emergence of additional moments from overlying vertical loads (the dead weight of the structure, the weight of process equipment, variable payload, etc.). This necessitates the calculation of tower-type structures using a deformed model, which is not typical for other engineering structures discussed previously. Such a calculation is not required for cooling towers and other structures whose height-to-bottom-diameter ratio is relatively small (within N/D = 1 : 2 – 1 : 3).

All other loads acting on the shafts of tower structures are given in the corresponding regulatory document DBN B.1.2-2:2006 Loads and impacts.

This section discusses only two types of tower-type engineering structures: water towers and radio and television towers; other structures can be found in additional specialized literature.

4.1 Radio and television and radio relay towers

Humanity’s desire to construct high-rise structures can be traced throughout the history of science and technology in the construction industry. This quest addresses both purely technological challenges and specific aspects of prestigious technologies. Suffice it to mention the famous Eiffel Tower in Paris, which stands at a height of 305 m (currently its height 324 m), built in 1887. The tower’s functional purpose at the time was very limited, determined by the installation of observation decks at heights of 50, 100, and more meters. Furthermore, the goal was to achieve maximum heights in the construction of specialized structures using metal structures. With the advent of radio communication, the Eiffel Tower’s functionality expanded significantly; it was equipped with communications receiving and transmitting antennas, as well as hydrometeorological sensors. Over time, the Eiffel Tower became one of Paris's landmarks, its symbol. The example of this tower’s construction inspired the replication of similar structures in other cities around the world.

The advent and development of television has contributed to the further improvement of the construction of high-rise tower supports in many major cities around the world.

Tower structures are an integral part of modern cities and individual towns. High-rise buildings dominate the surrounding development and have a significant impact on the architectural appearance of the area. Therefore, the architectural and structural design of radio relay and radio television towers requires strict technological and structural requirements, along with aesthetic elements and sophisticated design.

For a long period of time (until the 1950s), the use of reinforced concrete in the construction of high-rise tower supports for radio and television systems as a pressing issue. The first step in this direction was taken by the German Fritz Leonhardt, whose design for a 1,000-foot-tall radio and television tower was built in Stuttgart in 1956. 216 m. Following the example of this first experimental structure, similar reinforced concrete tower supports for radio and television systems were subsequently built in many cities in Europe, Asia, and America.

In parallel with the reinforced concrete design of radio and television towers, metal structures were also widely used. Thus, the world's tallest radio and television tower is currently the Skytree Tower (Fig. 4.1), built in Japan (Tokyo) in 2012. Its height reaches 634 m. The trunk of this tower is designed with lattice metal structures. Also noteworthy is the original radio and television tower in the form of a hyperboloid of revolution, built in Guangzhou, China, 2010, height 610 m. The supporting frame of this tower is made of metal structures. Towers in the cities of Kyiv are noteworthy (372 m, 1978.), Tashkent (375 m, 1985.), Riga (368 m, 1989.), Baku (310 m, 1997.), Newton (USA,324 m, 1994.), Harbin (China, 336 m, 2000.) and many others.

Among the reinforced concrete radio and television towers, the tallest tower with reinforced concrete structures is in Toronto (Canada) (height 554 m), built in 1976 (Fig. 4.2); tower in Shanghai (China), “Oriental Pearl” (height 468 m), built in 2008. (Fig. 4.3); the Menara TV tower in Kuala Lumpur (Malaysia) (height 421 m), built in1996. (Fig. 4.4); TV tower in Beijing (China) (height 405 m), built in1995.

In the UK, the reinforced concrete radio and television tower (Yorkshire) is worth noting, with a height of 330.5 m, built in1971.; in the USA in the city of Seattle in 1962. the Space Needle tower was built, its height is 184 m. An unusual design solution in the form of a console-lever type is the radio tower in Barcelona (Spain), its height is small – 136 m; built in 1992. (by Calatrava), an unusual original design was used.

Figure 4.1 – Tokyo Skytree Tower. Tokyo, Japan, the tallest tower in the world. Height 634 m. Year of construction – 2012. Construction – metal.

Figures show schematic views of the most famous reinforced concrete radio and television towers erected in major cities across Europe, America, and Asia. It's worth noting the uniformity of their design, which consists of various platforms and structures attached to a reinforced concrete shaft. These platforms serve technological functions, as well as additional entertainment options for visitors (cafes, restaurants, lounges, play areas, observation decks, etc.), and also serve as platforms for hydrometeorological observations.

Unlike radio relay towers, radio and television towers are multi-purpose. They house transmitting and receiving antennas, television and radio broadcasting equipment, and relay equipment. In addition to these functions, they often have additional uses, such as restaurants, observation decks, balcony facilities for meteorological instruments and equipment, and other scientific research

Figure 4.2 – CN Tower, Toronto, Canada, height
554 m, year of construction 1976. Structures – reinforced concrete

Figure 4.3 – Oriental Pearl Tower, Shanghai,
China, height 468 m, year of construction – 1994.
The structures are reinforced concrete

Figure 4.4 – Menara TV Tower in Kuala Lumpur,
Malaysia, height 421 m, year of construction – 1994. Structures – reinforced concrete

Tall tower structures made of reinforced concrete have two main types, designed for use in radio and space communications. These include radio relay and radio television towers.

Radio relay towers are single-purpose engineering structures; typically, they house only receiving and transmitting antennas and directional radio communication equipment. Radio relay towers are relatively small in height, typically 80 – 100 m. The main design parameters of these supports are determined by the technological requirements of the equipment being installed. The shape of a radio relay tower is generally cylindrical. The tower diameter ranges from 1/7 to 1/12 of its height is 5–8,5 m. The wall thickness of the cylindrical shafts at the junctions with the foundations is 150–1 80 mm (with a tower height of 40–50 m), 180–200 mm at a height of 60–70 m and 220–250 mm. At a height of 80–100 m, the wall thickness is assumed to be uniform throughout the entire height. These towers house a staircase or elevator equipment designed for servicing radio equipment. External staircases are also possible.

In many cities around the world, radio and television towers are the city's calling cards, distinguished by the originality and distinctiveness of their architectural forms, emphasizing the country's national characteristics, having a specific design solution, and fulfilling a variety of entertainment functions.

The main advantages of this type of tower support include a small footprint, the ability to be located near residential areas, and ease of operation. Furthermore, reinforced concrete high-rise towers, typically solid-wall structures, provide protection for numerous cables from direct exposure to the external environment, create convenient conditions for their maintenance, and allow for the placement of various utility rooms within the shaft without the need for special fencing.

The disadvantages of reinforced concrete tower supports include their significant dead weight, concentrated in a small area; danger to air traffic (airplanes, helicopters, airships, etc.); and a large area of damage in the event of an accidental fall.

The design parameters of reinforced concrete tower supports for communication and television systems are determined by technological requirements:

– the height of the structure depends on the characteristics of the radio and television equipment installed on the tower, as well as on the terrain;

– the dimensions of the cross-section of the shaft must ensure the placement of appropriate communications and designed rooms inside it;

– the rigidity of the cross-section of the trunk and the foundation support must be such that the largest angles of deviation transmitting antennas from the vertical do not exceed 0,3–3°.

Most of the erected reinforced concrete tower supports have a circular or nearly circular shape in plan. This cross-section corresponds to. The structure's better aerodynamic properties, which is crucial for tower-type structures. Furthermore, the cross-sectional shape of some of the towers erected is different: the tower in Stockholm is square, the one in Minkolz is diamond-shaped, and so on.

It should be noted that the development of satellite radio and telecommunication systems in recent years has led to a significant reduction in the importance of high-rise tower supports; in addition to their direct functional purpose, they now serve only as architectural accents and entertainment facilities.

4.2 Tower shafts

The main structural element of radio relay and radio television towers is the reinforced concrete shaft. Reinforced concrete tower support shafts are constructed either as shells of revolution of cylindrical, conical, combined, or more complex shapes, or as prismatic or pyramidal shells. Three-lobed, four-lobed, or multi-lobed cross-sections can also be used.

Cylindrical shafts with a constant outer diameter are most typical for radio relay towers up to 100 m. Conical shafts and shafts with a complex multi-stage shape are typical for multi-purpose radio and television towers with a height of more than 200 m. Shafts in the form of a conical tube, without bends in the meridional direction, are used in radio and television towers quite rarely. Conical shafts with cylindrical sections in the upper zone and in the areas where high-rise external structures are located are more commonly used. In some cases, with a significant tower height ($H \geq 250 - 300$ m) and on a weak soil foundation, the conical shaft is supported on a transition base in the form of a conical or hyperbolic shell, which is located mostly above the ground surface. By constructing such a transition base in the lower zone of the tower, the foundations can be designed without an elevated section.

Diameters $(D_н)$ trunks at the points of connection with the foundations or in the lower developed parts of the towers are taken to be equal to approximately 1/12 – 1/15 of the distance from the base of the trunk to its top (H_1). In the Toronto Television Centre towers, the ratio of the base diameter of the conical shaft and the three-lobed contour to its height is approximately 1/16 – 1/18. This is explained by the fact that in this case the shafts are pre-stressed, unlike the shafts of other towers.

The angle between the generating conical surface of the trunk and the vertical axis in radio and television towers is taken to be approximately 1°11′ – 1°30′, which corresponds to a change in the barrel diameter by 4–5 m for every 100 meters of height. The walls of conical trunks, as well as trunks of complex shapes, have variable thickness. The thickness of the trunk walls is greatest at its base.

In most constructed tower supports, the wall thickness at the base of conical shafts, as well as those with complex shapes, is 300–400 mm for shaft heights of 100–120 m, and 600–1 000 mm for heights over 150–180 m. In the upper sections of tower supports, the wall thickness is thinner – 200–300 mm. When constructing reinforced concrete shafts using prestressed meridional reinforcement, the wall thickness at the base of the shaft can be reduced to 400–500 mm.

A prismatic shaft shape can only be justified by more optimal use of the usable space inside the tower, while a pyramidal shape is justified by architectural considerations. In terms of aerodynamic performance, tower supports of this type are inferior to structures shaped like shells of revolution. Parameters such as the base plane size and shell wall thickness for prismatic and pyramidal shafts can be selected in a similar way to those for cylindrical and conical shafts, respectively.

The shafts are constructed from concrete grades C25/30 – C32/40. A400C and A500C grade reinforcement is used as non-stressed reinforcement in the shaft structures, while K1400 cables, bundles, or strands of high-strength VR1300 – VR1400 wire serve as prestressed reinforcement. The shaft wall is reinforced with meridional and ring reinforcement.

The meridional reinforcement is installed based on the horizontal cross-sections of the shaft, and the annular reinforcement is installed based on the vertical cross-sections resistance to temperature and humidity. Double reinforcement of the wall is typically provided. It is recommended that approximately two-thirds of the meridional reinforcement be placed near the outer surface of the shaft.

The diameter of the meridional reinforcement should be up to 25 mm, preferably before 18–20 mm, placing it evenly around the perimeter of the section at a distance of 125–250 mm. The spacing of the rods on the outer and inner surfaces may vary.

Minimum diameter of ring reinforcement bars – 10 mm, the maximum pitch of these rods is 250 mm.

We have a solution where all the reinforcement is concentrated at the outer surface of the shaft, and only at the junctions of horizontal shear diaphragms, balconies, platforms, etc., is double reinforcement with meridional and ring reinforcement provided. This reinforcement option can be used if it is justified by calculations for a wall thickness of 180–300 mm. The method proposed by Fritz deserves more attention. Reinforcement of walls up to 300 mm. Single meridional and double ring reinforcement with the inner ring reinforcement arranged in separate groups at intervals of 2–2.5 m along the shaft's height. This reinforcement technique, while ensuring the absorption of possible tensile ring forces at the inner wall surface in sections with cracks, also simplifies the technological processes of shaft wall construction.

Various types of diaphragms, in the form of reinforced concrete ring slabs of solid or box cross-section, can be installed within the shaft. Diaphragms are typically provided at points where the shaft cross-section changes, where the shaft connects to the elevated portion of the foundation, or where the tower base. In some cases, they are provided at the junctions of external platforms with significant overhangs or the main load-bearing reinforced concrete structures of the external structures. Special stiffening diaphragms are rarely installed throughout the entire shaft, spaced at intervals of 30 – 50 m. The number of powerful diaphragms in the shaft should be kept to a minimum, as their installation entails additional manufacturing difficulties and they introduce undesirable disturbances into the stress-strain state of the shaft, particularly due to temperature and humidity effects on the structure.

The tower cap is designed to firmly clamp a metal antenna mast. The detailed solution can be implemented either as a single, fairly massive slab up to 1200 – 1500 mm thick or as two slabs spaced apart.

External platforms, as well as supporting structures of high-rise external structures of round towers, can be made of reinforced concrete in the form of:

1) monolithic cantilever ring slabs, rigidly connected to the shaft wall ;

2) monolithic cantilever ring slabs, pivotally connected to the shaft;

3) monolithic cantilever ring slabs connected to the shaft in such a way that the possibility of radial displacement of the slab relative to the shaft wall is allowed (sliding connection;

4) monolithic or prefabricated beam cantilever structures;

5) conical shallow thin-walled shells with or without prestressing.

The second, third, and fourth design solutions are inferior to the first in terms of material consumption, but they reduce the negative impact of the platform on the shaft wall, particularly due to temperature and humidity effects. Of the listed design solutions for external platforms, the thin-walled conical shell is the most cost-effective in terms of material consumption.

Reinforced concrete structures of external platforms and balconies, as well as high-rise external structures, are designed in accordance with the general design and calculation rules for similar reinforced concrete structures. They are calculated to withstand vertical loads from the dead weight of the structure, process equipment, people, snow, ice, and temperature effects.

4.3 Calculation of tower shafts

The design scheme of the shaft of a tower-type structure is a rod of constant or variable height cross-section, clamped at the lower base, that is, the console. Due to the action of the wind velocity pressure, the flexible trunk is subject to bending and oscillation, therefore, during the period of free oscillation of the structure $T > 0{,}25$ The wind load should be determined taking into account the dynamic effect of the velocity pressure pulsation caused by wind gusts. Furthermore, for a shaft with an outer surface slope of no more than 1,2 %, a resonance verification calculation is performed.

The period of natural oscillations of a structure is determined by the formula:

$$T = \xi H^2 \sqrt{A_1 \rho_m / (g B_1)}, \tag{4.1}$$

where ξ – coefficient depending on the shape of the barrel;

H – trunk height;

A_1 And $B_1 = M / p_1$ – area and bending rigidity of section 1-1 of the trunk, respectively (Fig. 6.22);

ρ_m – average density of the barrel material;

g – acceleration of gravity.

When calculating shaft structures, the following main loads and impacts are taken into account:

a) vertical loads from the shaft`s own weight, the weight of external high-rise buildings and balconies, antenna mast, and process equipment; if necessary, these must be taken into account variable loads from people, snow, ice;

b) horizontal wind load taking into account the dynamic impact of velocity pressure pulsations;

c) temperature effects from one-sided heating of the barrel, caused by solar radiation, as well as the difference in air temperature inside and outside the tower;

d) effects of prestressing;

e) seismic impacts.

The shaft of radio and television type structures is calculated according to. The so-called deformable system. Since the trunk bends and the foundation rotates under the influence of wind, temperature, and other factors, additional bending moments arise in the trunk sections from the existing vertical loads. Under the influence of sunlight, the sun-facing side of the trunk heats up more than the opposite, shaded side. Therefore, the trunk bends additionally toward the shaded side.

Bending moments in normal sections of the trunk console are determined taking into account its deformation under the influence of horizontal loads arising from wind, temperature effects, and foundation tilt. The total moment can be determined using the formula (Fig. 4.5):

$$M = M_w + M_g, \tag{4.2}$$

where M_w – bending moment due to wind load $\left(M_w = \sum W_i \cdot z_i\right)$;

M_g – additional moment arising from vertical loads due to deviation of the trunk axis from the vertical position $\left(M_g = \sum G_i \cdot f_i\right)$.

The total deviation of the barrel axis from the vertical in$i$-at that point is (previously the entire height H is divided into i points)

$$f_i = f_{i_1} + f_{i_2} + f_{i_3}, \qquad (4.3)$$

where f_{i_1} – trunk deflection from horizontal loads:

$$f_{i_1} = \frac{W(H - z_i)^4}{12 E_i J_i k}. \qquad (4.4)$$

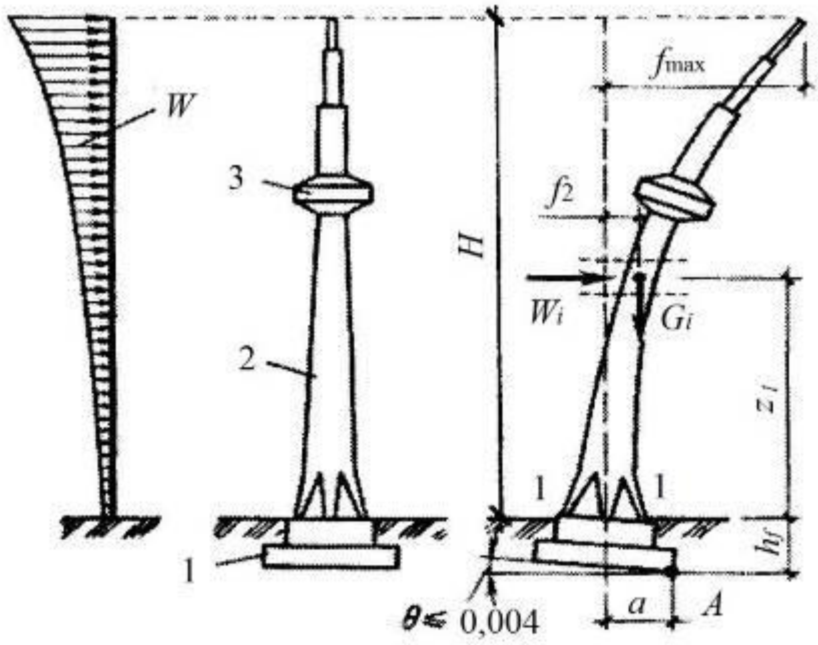


Figure 4.5 – For the calculation of tower structure shafts:

1 – foundation; 2 – shaft; 3 – lining

In formula (4.4) W – wind pressure at the corresponding point i; H – full height of the trunk; z_i – distance from the top of the foundation to the i-th point; $E_i J_i$ – bending rigidity of the i-th section; $k = 0{,}85$.

f_{i_2} – deviation of the trunk from the foundation tilt:

$$f_{i_2} = (z_i + h_ф) tg\theta, \qquad (4.5)$$

where z_i – distance from the top of the foundation to the i-th point;

$h_ф$ – foundation height; $tg\theta = 0{,}004$.

f_{i_3} – deflection caused by one-sided heating of the trunk by solar radiation:

$$f_{i_3} = \int_0^{z_i} \frac{1}{\rho_t(z)} M_{p_i=1}(z)dz\,, \tag{4.6}$$

where $\frac{1}{\rho_t(z)} = \frac{\alpha_t \Delta t_i}{Д_{сеч}}$ – curvature of an element in a section located at the level z;

α_t – coefficient of linear expansion;

Δt_i – temperature difference in the section (20°C, 30°WITH);

$Д_{сеч}$ – the diameter of the section under consideration or the diagonal;

$M_{p_i} = 1$ – bending moment at level z from the action of a single force $P_i = 1$.

To determine the total moment from all vertical loads in a given section $i - i$ It is necessary to find the deflections in the i-th section and in $i+1$ section (k – in that section), then:

$$M_g = \sum_k^n P_i\left(f_i - f_k\right), \tag{4.7}$$

where P_i – all vertical loads located above the section in question.

After this, the total moment in an arbitrary section is determined using formula (4.2).

Additionally, calculations are used to check the stability of radio and television towers against overturning at foundation point A (Fig. 4.5). The stability of the structure is ensured if the following condition is met:

$$\chi = \frac{M_1}{M_1} = \frac{\sum G_i \cdot (a - f_i)}{\sum W_i \cdot (z_i + h_ф)} \geq 1{,}5\,, \tag{4.8}$$

where M_1 – holding moment;

M_2 – overturning moment.

Values f_i And z_i are taken according to the formulas given earlier.

4.4 Transitional support base of trunks

Some design solutions for radio relay and radio television towers may utilize shaft support bases, providing a smooth transition from the main shaft to the recessed foundation, which is developed in plan. These transition bases can take the form of a widened cylinder with conical slopes, a hyperboloid of revolution, individual support legs of a rectangular or trapezoidal shape, a conical shell, and other shapes.

The shaft wall thickness can be 300–600 mm or more, depending on the shaft height and the thickness of its wall at the point of contact with the base. The supporting bases are reinforced more heavily than the shaft walls; they are typically equipped with a double mesh cross-section with densely spaced ring and meridional reinforcement of grades A400C, A500C, and higher. Higher-grade concrete is used: C25/30, C32/40, and in some cases C45/50.

The barrel`s support base shell is designed to withstand the same loads as the barrel itself. The barrel's impact on the shell or other structures is three internal forces: N, M, V These forces can either be distributed uniformly around the perimeter of the shell or proportionally distributed among the supporting legs. An example of the distribution of forces around the perimeter of the shell is shown.

The following loads are simultaneously applied to the upper edge of the support base:

– uniformly distributed load from N_{1-1}:

$$p = \frac{N_{1-1}}{\pi Д}; \tag{4.9}$$

– variable distributed load from bending moment M_{1-1}:

$$p_b \cos\alpha \approx \frac{M_{1-1} \cdot \delta_{1-1}}{W_{1-1}} \cdot \cos\alpha \tag{4.10}$$

– and horizontal shear load from the transverse force V_{1-1}:

$$p_2 \sin\alpha \approx \frac{V_{1-1} \cdot S_{1-1}}{2J_{1-1}} \cdot \sin\alpha . \tag{4.11}$$

In these formulas N_{1-1} – resultant of all vertical loads, including the dead weight of structures and the weight of equipment, as well as variable loads; M_{1-1} – bending moment in section 1–1 of the base of the trunk from horizontal and vertical loads acting on the trunk (taken from the calculation of the trunk according to the deformed scheme); V_{1-1} – transverse force in section 1–1 from horizontal loads acting on the trunk; angle α shows the deviation of the point of the circle at which the load is determined from the horizontal line; S_{1-1}, J_{1-1}, W_{1-1} – respectively, the static moment, the moment of inertia and the moment of resistance of the base section relative to the central axis; δ_{1-1} And $Д$ – wall thickness and average cross-sectional diameter 1–1.

Forces in the barrel base clip N_1 And N_2 from the action of vertical loads, including the load p, with its variable thickness are determined based on the moment theory of shells using the following formulas:

$$N_1 = -\frac{a}{\rho \sin\psi}\left\{\frac{Q_0\rho_B}{a} + \left(g_{об} + g_{изол}\right)\left|\left(\xi - \xi_B\right) + \frac{c_1}{3}\left(\xi^3 - \xi_B^3\right)\right| + \right.$$

$$\left. + \frac{\Delta g_{об}}{\chi} e^{\chi(\xi - \xi_н)}\left[1 + \frac{c_1}{\chi^2}\left(\chi^2\xi^2 - 2\chi\xi + 2\right)\right]\right\}; \quad (4.12)$$

$$N_2 = \frac{A}{\rho^2} N_1 - A \cdot ag\xi\ ; \quad (4.13)$$

$$g = \left\{\gamma_{изол}\delta_{изол} + \gamma_{жб}\left[\delta_{об} + \Delta\delta_{об}\right] \cdot e^{\chi(\xi - \xi_н)}\right\}; \quad (4.14)$$

$$c_1 = \frac{\bar{\rho}_н - 1}{\xi_н^2}.$$

In the given formulas (4.12), (4.13) and (4.14) the following designations are adopted for the shape of the support base in the form of a single-sheet hyperboloid of revolution (Fig. 4.6) with a height H_1:

$\xi = \frac{z}{a}$: z – current altitude coordinate; a – radius of the circle at this level;

$\xi_н = \frac{z_н}{a}$: $z_н$ – distance from the lower ring of the support base to the section under consideration;

$\xi_B = \frac{z_B}{a}$: z_B – distance from the upper ring of the support base to the section under consideration;

r – radius of parallel at a given level;

$\rho_н = \frac{r_н}{a}$ – relative radius of the lower section;

$\rho_н = \frac{r_н}{a}$ – relative radius of the upper section;

$R = \frac{\left(1 + r'^2\right)^{3/2}}{r''}$ – radius of curvature of the meridian.

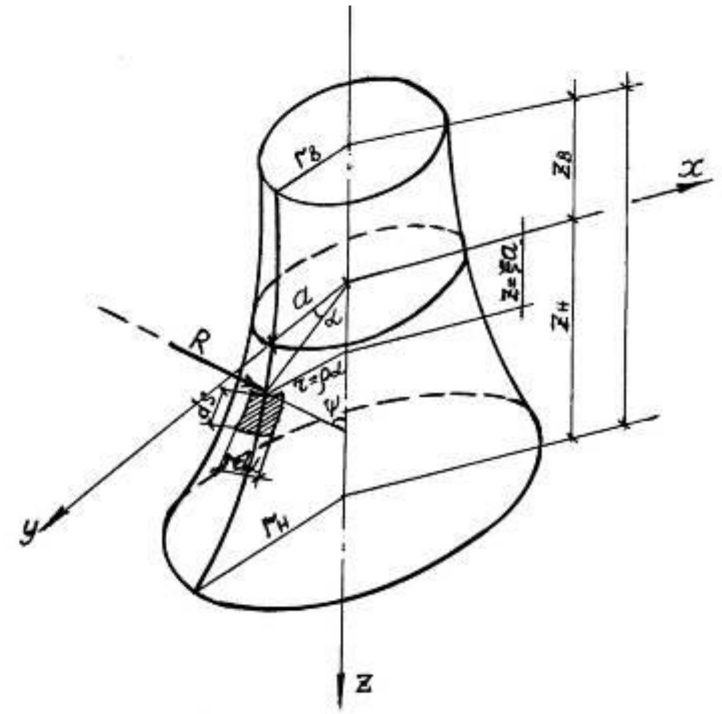


Figure 4.6 – Calculating the support base of a radio and television tower

The remaining parameters in formulas (4.13) and (4.14) are determined as follows:

$$\rho = \frac{r}{a} = \sqrt{1 + A\xi^2}\ ; \quad A = \frac{\left(\rho_н^2 - 1\right)}{\xi_н^2}; \quad \bar{\rho} = \sqrt{1 + Б\xi^2}\ ; \quad Б = A + A^2;$$

$$\sin\psi = \frac{\rho}{\bar{\rho}}; \quad \cos\psi = \frac{A\xi}{\bar{\rho}}; \quad r = x^2 + y^2 .$$

Coefficient χ can be taken with the range from 1,8 to 2.

When all the above parameters are determined, the coordinate z is taken into account with its sign. The external force parameters have the following designations:

Q_0 – vertical load on 1 meter the perimeter of the upper stiffening ring from the weight of the ring, insulation on the ring, snow, etc.;

$g_{из}$ – vertical load from weight1 m2insulation layer ($\delta_{из}$ – thickness of the insulation layer;

$\gamma_{из}$ – average insulation density);

$g_{об}$ – vertical load from weight1 m2shell walls of constant thickness $\delta_{об}$;

$\Delta g_{об} \cdot e^{\chi(\xi - \xi_{н})}$ – increment of vertical load (at the level $\xi = \frac{z}{a}$) from weight1 m2shells due to the smooth thickening of its wall in the lower zone;

$(\delta_{об} + \Delta\delta_{об})$ – the greatest thickness of the shell wall at the point where it connects to the support ring or support colonnade.

If the supporting base of the trunk has a constant thickness throughout its height and is loaded with a symmetrical vertical load, the equation for determining the meridional N_1 and ring N_2 efforts are significantly simplified:

$$N_1 = -\frac{Q_\psi}{2\pi r \sin\psi}; \tag{4.15}$$

$$N_2 = -\left(p_3 - \frac{N_1}{R}\right)\frac{r}{\sin\psi}; \tag{4.16}$$

where Q_ψ – resultant of all vertical loads acting on the shell above the horizontal section under consideration;

p_3 – horizontal load on1 m^2, acting perpendicular to the area under consideration (wind load component).

Efforts under moment-free stress state in the hyperbolic shell of the barrel base under the influence of loads $p_B \cos\alpha$ And $p_2 \sin\alpha$ can be defined for any point on the surface with coordinates α And ψ (Fig. 4.15) according to the following formulas:

$$N_1 = p_B \cos\alpha \frac{a^2}{r^2 \sin\psi}; \tag{4.17}$$

$$N_2 = p_b \cos\alpha \frac{a^6}{b^2 r^4} \sin\psi; \tag{4.18}$$

$$S = -p_B \cos\alpha \frac{a^3}{b r^2} \sqrt{1 - \frac{a^2}{r^2}}; \tag{4.19}$$

from the load $p_2 \sin\alpha$

$$N_1 = p_2 \sin\alpha \frac{b}{r \sin\psi} \sqrt{1 - \frac{a^2}{r^2}}; \tag{4.20}$$

$$N_2 = p_2 \sin\alpha \frac{a^4}{b r^3} \sin\psi \sqrt{1 - \frac{a^2}{r^2}}\,; \tag{4.21}$$

$$S = p_2 \sin\alpha \frac{a^3}{r^3}\,. \tag{4.22}$$

Parameter values a, ψ, α, r given above, width b corresponds to the size of the section in the direction y.

4.5 Foundations of radio and television towers

In radio relay and radio television towers constructed of reinforced concrete and cylindrical or conical shafts, the foundations are predominantly solid, circular, or ring-shaped slabs. In some cases, depending on the design of the tower's support base or the towers themselves, polygonal, square, or even rectangular shapes may also be used.

For relatively shallow foundations, the foundation itself is constructed without a raised section. In this case, the foundation slab directly contacts the tower shaft (Fig. 4.7, a) or the shaft's support base (Fig. 4.7, b). Alternatively, the ring-shaped foundation may be connected to the shaft using trapezoidal legs (Fig. 4.7, c).

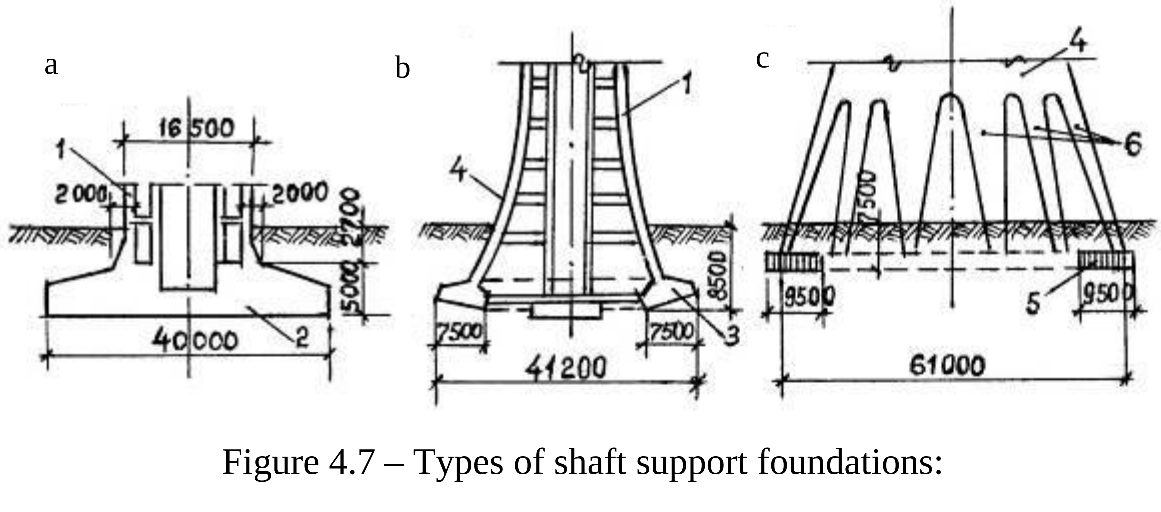


Figure 4.7 – Types of shaft support foundations:
a – solid plate (2) connected to the barrel (1) (Munich); b – ring plate (3) connected to the transition base (4) (Berlin); c – ring plate (5) connected to the base support legs (6)

In addition to the foundation slab (solid or circular), a raised section foundation includes a transition section from the slab to the shaft, most of which is located below the ground surface. The raised section is typically constructed as either a conical shell or a monolithic low-rise reinforced concrete stack (Fig. 4.8).

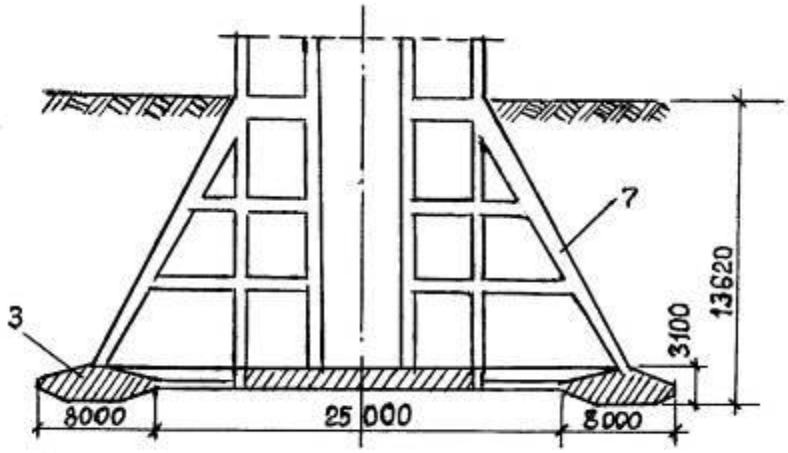


Figure 4.8 – Ring fragment (3) with transition section (7) (Hamburg)

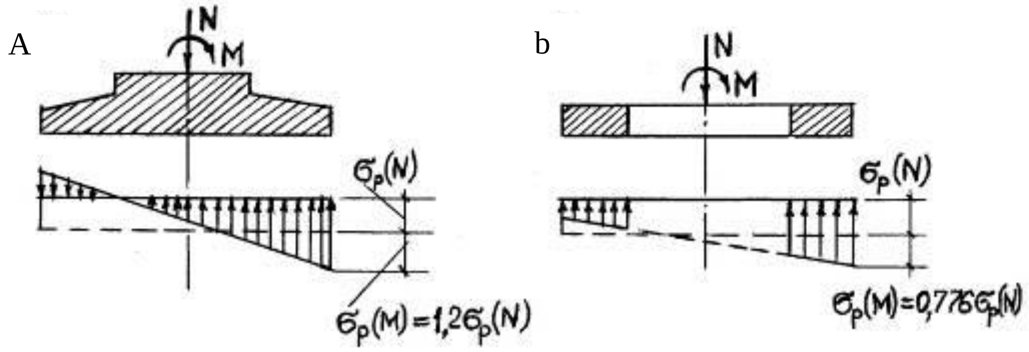


Figure 4.9 – Calculation of different types of foundations:

a – solid plate; b – ring plate

The design of foundation slabs, especially large-diameter ring slabs, must be carried out taking into account the support of spacer structures (conical and hyperbolic shells, inclined legs, etc.), which can cause significant tension in the slab. To prevent the formation of normal and inclined cracks in the slab or to reduce their opening width, the foundation slabs of high tower supports are often prestressed with prestressed reinforcement tensioned on the concrete. The prestressed reinforcement used is either K1400, K1500 cables, or bundles and strands of high-strength Bp1300 wire. After tensioning and anchoring all prestressed reinforcement, it must be reliably protected by a layer of concrete (or shotcrete); high-strength mortar can be used. If prestressing reinforcement on concrete, it is always necessary to keep in mind the possibility of systematic wetting of the foundation structures, which poses a certain risk to the prestressed reinforcement (especially made of high-strength wire BP1300; BP1400), located in a layer of uncompressed concrete or mortar, where the appearance of stress and shrinkage cracks is almost inevitable.

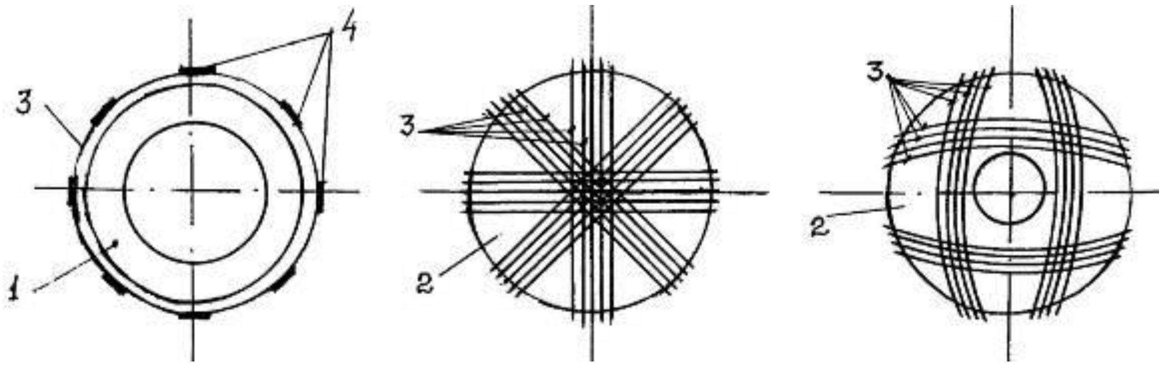


Figure 4.10 – Layout of prestressed reinforcement foundation slabs: 1 – ring foundation slab; 2 – solid foundation slab; 3 – prestressed reinforcement; 4 – anchorage points of ring prestressed reinforcement

Foundations for radio relay and radio television towers, like those for other tower structures, are constructed of high-grade concrete (C25/30; C30/35). Reinforcement meshes for slabs and frames must be knitted to ensure increased reliability and safety during operation. Furthermore, it is recommended to install control and measuring sensors (indicators, benchmarks, and angle gauges) during construction to record foundation behavior under wind, temperature, and process loads. Since most radio relay and radio television towers are classified as “experimental construction” facilities, they must comply with the requirements of DBN B.1.2-5:2007 “Scientific and Technical Support for Construction Projects”. These standards require duplicate calculations and theoretical analyses and monitoring of the facility's behavior at all stages of its life cycle – from design and operation to complete dismantling.

TOPIC 5 CHANNELS AND TUNNELS

Canals and tunnels are a type of engineering structure that facilitate the movement of materials, utilities, vehicles, and people in a given direction, while separating these flows from the external environment. Materials used for such separation include wood, metal, stone, reinforced concrete, plastics, synthetic films, rubber structures, and others. Reinforced concrete canal and tunnel structures are the most widely used, durable, and cost-effective. In this section, “canals” refers to utility canals, not hydraulic ones.

In most cases, such engineering structures are located underground, but in some exceptional situations they can be located above the ground surface, in which case special supports and pylons are installed.

In industrial and civil construction, canals are used for inter-block, inter-shop, and intra-shop installations of various utilities, electrical cables, pipelines for various purposes, wastewater drainage, and the transportation of various products and waste from industrial, chemical, and agricultural production. Most canals are shallow, 500 – 700 mm deep, and small in size.

Unlike canals, tunnels have a more diverse range of applications, larger cross-sections, and a variety of design solutions. Tunnels are typically multifunctional and can serve both as canals and transport corridors.

In this regard, it is advisable to analyze the design solutions, purpose and calculation methods of reinforced concrete channels and tunnels separately in order to emphasize both their differences and the similar elements of these engineering structures.

5.1 Types and design features of channels

There are two main types of channels: impassable and semi-passable. Semi-passable channels, meaning a person can walk through them while slightly bent, must be at least 1500 mm high; impassable channels are lower and are used only for utility lines.

It should be noted that the channels are designed for different purposes. They can be hydraulic or technological. The former are almost always open, while the latter are usually closed.

The history of canal construction is extremely ancient and rich. Even in primitive communities, people attempted to create water pipes using stones and local building materials – clay, sand, and wood. Over time, as technology and human intellect developed, canals began to be built. More durable and functionally diverse. Water-carrying and sewer canals were known in ancient Rome (3^{rd} – 4^{th} centuries BC), as were the underground sewer canal – the Cloaca Maxima, the Appian Aqueduct, and others. In ancient China, canals for irrigation and water supply were built as early as the 3^{rd} and 2^{nd} millennia BC. Over time, they began to be used for shipping, connecting rivers, lakes, and seas. Hydrotechnical canals are studied in specialized engineering disciplines and are not discussed in this textbook.

They only characterize the various types of industrial reinforced concrete channels, which are mostly used as closed channels and designed to carry various utility systems. These channels are primarily constructed using precast reinforced concrete in standard series, and only in some cases are they constructed as monolithic continuous channels.

Based on the unification of standard series TS-04-01 and TS-01-03, dimensional diagrams have been developed for single-, double-, and multi-section channels with two types of structures: trough-type channels and precast concrete channel-by-channel channels consisting of bottom slabs, walls, and roof slabs. Channels are marked with letters and numbers indicating the type of structure, the number of sections, and their geometric dimensions. The letters KL denote channels constructed from trough elements covered by slabs; the letters KLS denote channels constructed solely from trough elements; and the letters KS denote channels constructed from precast slabs. If channels are constructed using a semi-underground method, the index P is added. The number before the letters indicates the number of sections in multi-section channels, and the numbers after the letters indicate the nominal geometric dimensions of the channels in centimeters.

The width dimensions of the channels are taken as multiples of 300–600, 900, 1 200, 1 500 and 2 100 mm; the height dimensions are 300, 450, 600, 900 and 1 200 mm. The actual dimensions of the channels may differ from the accepted overall dimensions on ± 30 mm. The established dimensions allow for a significant reduction in the number of overall layouts, effectively meeting the requirements of all industrial sectors and construction types.

Along the route of the canals, inspection wells (chambers) are necessarily installed, either manufactured, prefabricated or monolithic in the form of rectangles or cylindrical structures.

The length of the prefabricated channel elements along the route is 3 m, with the exception of the bottom plates of two-section channels, the length of which is 1,5 m. The length of the selected tray elements is 0,6; 0,9; 1,2; 1,5 and 2,1 m; floor slabs – 0,9; 1,2; 1,5; 1,8 and 2,1 m, wall slabs – 0,9 and 1,2 m.

The distance between expansion joints is no more than 50 m for underground channels and no more 30 m. For semi-underground systems. Expansion joints are recommended at the junctions of channels with inspection chambers and expansion joints, or at the boundaries of sections with significantly different cross-sections, loads, etc. The bend angles of single- and multi-section channels and expansion joints can be designed in two ways: with a monolithic bottom, brick walls, and precast floor slabs, or with a monolithic bottom, walls, and precast floor slabs.

Protective layer of concrete in trays, wall slabs and bottom slabs with a structure thickness of up to 100 mm is 15–20 mm, and if the thickness is greater 100 mm – 20–25 mm. In floor slabs, the protective layer of concrete is 15 mm.

The standard series provide guidelines for the use of standardized precast reinforced concrete channels on soils with subsidence properties. Two types of soil conditions at construction sites are considered: 1) when the soil subsidence due to its own weight is practically absent and does not exceed 3–5 cm (type I); 2) when the soil subsidence exceeds 5 cm (type II).

According to the degree of possible soil waterlogging in the foundations of the canals, they are also divided into two types-for laying cables, air ducts and other communications that do not contain water or aqueous solutions; channels for laying heating, technological or other pipelines from which water or aqueous solutions may leak.

Canal routes that are not intended to carry pipelines carrying water or aqueous solutions must be laid in a manner that ensures the unimpeded flow of atmospheric surface water. The distance from the canal axis to the nearest non-channel utilities containing water must be at least5 m for soil conditions such as I and 10 m for soil conditions II type. Channels for laying water pipelines, constructed in soil conditions of the type I, must be made of precast reinforced concrete structures made of dense concrete (W6, W8), and the foundation for the channels must be made of compacted soil with a thickness 200 mm or concrete thickness 100 mm. The minimum distance from the canals to the structures should be 2,5 m.

If channels containing pipelines with water or aqueous solutions are constructed in soils of the type II, then the foundation must be made of compacted soil 400 mm thick or concrete 150 mm, and at the joints of the tray elements, linings are provided with the lateral gaps between the linings and the channel elements treated with strands impregnated with bitumen.

When constructing canals in seismic zones, it is recommended to carefully prepare the canal bottoms, backfill the canals with careful layer-by-layer soil compaction, and construct inspection chambers from monolithic concrete or reinforced concrete and precast reinforced concrete floor slabs designed for normal construction conditions. In areas with a seismicity of magnitude 9, the joints of precast elements of the KL, KLP, and KLS grades should be reinforced with tray-type liners, similar to those used when constructing canals on subsidence soils. For KS and KSp canals, in which the bottom and wall slabs are installed with overlapping joints, joint reinforcement is not necessary.

To drain water, the bottoms of the channels in the transverse direction have a slope of at least 0,002.

Reinforcement of the walls and bottom of channels can be done using single or double mesh. The first type of reinforcement is used in small-sized channels 600 × 300 (h) mm, 900 × 300 (h) mm, 900 × 450 (h) mm with wall thickness of 50–60 mm. In channels with large dimensions and wall thickness of 80–100 mm. A double mesh is used both in the walls and at the bottom. The diameter of the working reinforcement in most cases is 6 – 10 mm class A400C, diameter of mounting fittings – 4–5 mm, class B500 (Bp-I) or 6A400C.

Intrashop ducts are widely used for laying various types of pipelines and cables, which can also be used as air ducts. These ducts are described in detail in the IS-01-04 series, issue 7. Intrashop ducts are designed with a floor-level ceiling, but can also be used with a recessed ceiling, subject to appropriate design verification.

Elements of intra-shop channels can be used under normal conditions, on subsiding soils, in seismic zones and areas with high groundwater levels.

The overall diagrams of the intra-shop ducts differ from those given in that they include ducts with a cross-section of 300 mm × 300 mm and 450 mm × 300 mm; excluding channels with a width of 2 100 mm and additionally included are tray channels with a height of 900 and 1 200 mm, which can be produced at open ranges.

For internal plant installations, channels made of tray elements and individual reinforced concrete slabs are used. The floors for these channels are made of flat reinforced concrete slabs, or, if appropriately justified, of corrugated steel. In areas where frequent slab removal is required, slabs no wider than 600 mm. In some cases, the floor slabs of intra-shop channels may have an internal textured layer.

When laying channels in workshops with an earthen floor, the floor slabs laid at the workshop floor level have fixing stops made of corners welded to the embedded elements in the slabs.

The calculation of impassable channels is performed using a simplified calculation scheme, which is consistent with the calculation of a U-shaped frame placed on the ground. The frame supports are connected to the roof slab by hinges.

Only those loads that correspond to the actual operating conditions of the channels are considered.

Semi-pass channels are calculated in almost the same way as tunnels, with all the loads on them and various types of calculation schemes.

Basically, channels are calculated for two groups of limit states, but in some cases it is sufficient to perform the calculation only in relation to the first group of limit states, that is, only the strength, which concerns mainly small-sized channels.

5.2 Types of tunnels and their areas of application

Tunnels were first built in ancient times, primarily for water supply, sewage disposal, and military purposes. The first mountain railway tunnel was 1 190 m was built in 1826–1830 in England. One of the oldest in the world is the Simplon Tunnel, which is 19,78 km, connecting Italy with Switzerland, was built between 1898 and 1906. Railway tunnels in Ukraine began to be built in 1859. By the end of the last century, numerous tunnels had been built on the railways of the Carpathian region. After 1917, major tunnels were built on the Nikolaev-Kherson, and Black Sea Railways, as well as a number of tunnels in eastern Ukraine and the Carpathians.

Railway tunnels were built in various ways in the ground, with the walls lined with massive stones to protect moving trains from rock falls; later, concrete and reinforced concrete began to be used.

Tunnels, in their simplest understanding, are extended underground or underwater structures designed to convey vehicles, pedestrians, water, utility lines, or process lines through a high-rise or contour obstacle.

All transport tunnels typically have two exits to the surface, and in special cases, only one (dead-end tunnels for transport or special purposes). Utility tunnels do not have such exits; they are connected by inspection chambers.

The operation of tunnels is ensured by a complex of coordinated underground and above-ground structures and devices, the composition of which depends on the purpose, length and location of the tunnel.

Railway or road transport tunnels, including subways, in addition to the railway or roadway, must also have drainage, ventilation, fencing and protective structures and devices that ensure the safety of pedestrians and service personnel.

Drainage systems are necessary to remove water from the tunnel, whether it penetrates the walls or flows from the water intake during cleaning operations. They are constructed as longitudinal channels or pipes installed in the middle or side of the tunnel.

Ventilation structures are designed to purify the air from exhaust gases, harmful substances, and dust generated by vehicle movement. The design and composition of these structures depend on the ventilation system and the tunnel length. With artificial ventilation, ventilation shafts, underground chambers, or above-ground fan structures may be constructed.

Enclosing and protective structures include portals, lined and supporting walls along the slopes of pre-portal depressions, catching walls and obstacles with enclosing ramparts and trenches on gentle slopes, galleries in pre-portal semi-cuts on steep slopes where there is a high risk of rockfalls, collapses and avalanches.

Water protection structures include water collection and drainage ditches, surface and underground drainage systems.

Safety devices include electric tunnel lighting, warning and fencing systems, telephone communications, fire-fighting equipment, etc.

The range of applications and types of tunnels is so broad that it is only possible to compile a general classification based on their purpose, location, depth of excavation, and construction method.

We show a schematic diagram of the different types of tunnels, taking into account their purpose, location, depth of excavation and construction method.

Unlike canals, all tunnels are through tunnels, that is, their height exceeds their size 1.8 m, and depending on their purpose, the dimensions of the tunnels can be very large (up to 5–9 m in height and 6–15 m or more in width).

Hydraulic tunnels are constructed within hydroelectric, pumped storage, or nuclear power plant systems to divert and supply water to power units (energy and diversion). Hydraulic tunnels also include drainage tunnels for drying and moistening land, water supply tunnels, and timber floating tunnels.

Utility tunnels in cities and villages are constructed for a variety of utility lines: high- and low-voltage electrical cables, communication cables, heating networks, drainpipes, water pipes, gas pipes, sewer lines, and others. In many cases, collector tunnels are constructed to carry multiple utility lines.

Mining tunnels are built at mining operations, shafts, and pits. They are used to transport ore and rock, ventilate, and drain underground workings.

Special-purpose tunnels include underground parking lots and tunnel-type garages, tunnels for scientific research (e.g., charged particle accelerators, tunnels for aerodynamic testing), gas and oil storage facilities, underground warehouses, and tunnels for military defense purposes.

Transport tunnels are classified by location as urban, mountain, and underwater. Urban vehicle and pedestrian tunnels are used to streamline vehicle and pedestrian traffic on city highways and streets. This distinction should be considered arbitrary, as mountain and underwater tunnels can also be located in urban areas separated by high-rise or water barriers. Mountain Tunnels are constructed primarily in mountainous areas to overcome high-altitude obstacles-mountain ranges, spurs, hills, and heights. Underwater tunnels are located at the intersections of contour obstacles: rivers, lakes, canals, reservoirs, sea bays, and straits.

Depending on the depth of the foundation from the ground surface *N* distinguish between deep tunnels ($H > 2-3B$) and small ($H < 2-3B$) of the tunnel, where B is the largest dimension (span or height) of the tunnel's cross-section. There are also ultra-deep tunnels, whose depth is 30–40 m or more below the ground surface.

According to the construction method, tunnels are divided into those constructed using closed, open or immersion methods, each of which has several varieties.

Closed methods (mountain, shield, and jacking) involve work without disturbing the surface, while open methods (pit and trench) require preliminary surface disturbance. Using caisson methods (caisson shafts, underwater tunnel sections), tunnel structures are constructed on the surface and then lowered to the design elevation.

In the most complex engineering and geological conditions, for preliminary stabilization or drainage of the soil mass, the previously listed methods are used simultaneously with special work methods - dewatering, artificial freezing, plugging or chemical soil stabilization.

The choice of one or another construction method is determined mainly by engineering and geological conditions, the length of the tunnel and the dimensions of its cross-section, as well as taking into account technical, economic and environmental conditions.

Mining and underwater tunnels are often built using mining or shield methods, while shallow urban tunnels are often built using excavation or trench methods.

The mining method is used primarily in rocky soils. In this case, the tunnel workings are opened in one go or partially, securing them with temporary supports, and then a permanent structure – a structure (equipment) – is erected at some distance from the working face (Fig. 5.1, a). In soft and weak soils, the most effective is the shield method, based on the use of a mobile support for closed outlines – a shield, under the cover of which the soil is excavated and the lining is erected (Fig. 5.1, b). Using the pit method, the tunnel structures are erected in a pre-excavated pit (Fig. 5.1, c), and using the trench method, the walls on which the ceiling rests are first lined in trenches, and then the soil between the walls is excavated and the tunnel trench is concreted (Fig. 5.1, d).

A tunnel is a complex and expensive type of artificial structure, widely used in railway and highway construction. In terms of their structural form, size, and construction conditions, tunnels used in transport engineering differ from other similar structures, such as hydraulic engineering, municipal, industrial, mining exploration, and special-purpose tunnels.

Mountain tunnels can be pass tunnels, constructed across high watersheds; hillside tunnels, laid along mountain slopes; loop and spiral tunnels, constructed for the development of road routes in mountainous conditions.

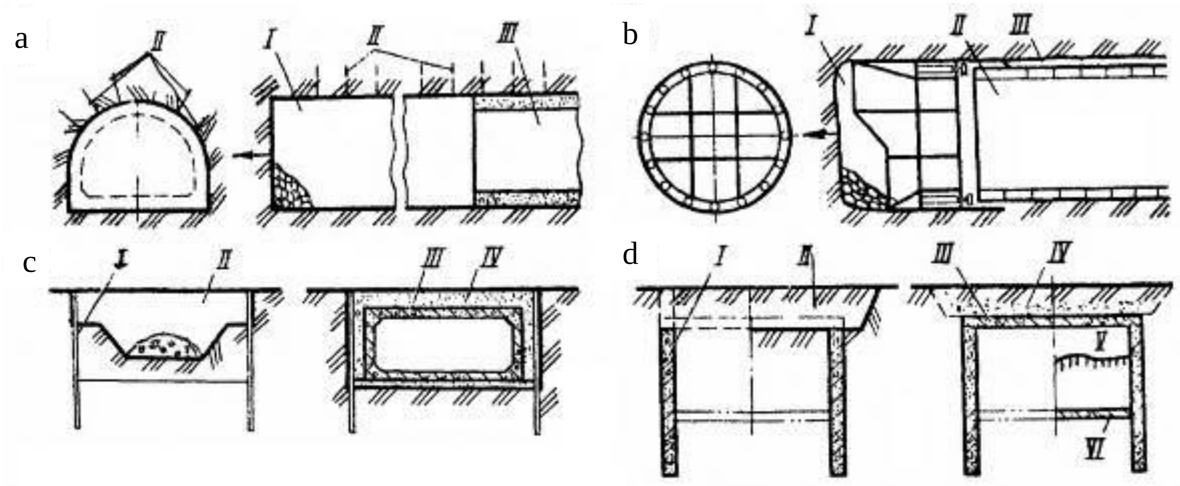


Figure 5.1 – Construction schemes of tunnels I, II, III, IV, V, VI: sequence of construction works and construction of tunnel structures

When a road crosses large water barrier, underwater tunnels are constructed in addition to bridge crossings to ensure continuous transport links between the banks.

To overcome deep but relatively narrow water obstacles, underwater tunnels on artificial dams, separate supports (tunnel-bridges), as well as “floating” tunnels anchored to the bottom with guy ropes or kept afloat by special floating supports are effective.

Urban pedestrian tunnels are constructed in areas of heavy traffic to ensure the movement of public transport and pedestrians at different levels and to improve traffic safety.

5.3 Types of tunnel linings

The tunneling process involves excavating the soil to the shape and dimensions of the excavation, removing it to the surface, and securing the excavated space with temporary or permanent support.

The space formed after the soil has been developed and removed is called a mine working.

Depending on its purpose, a mine working may be called an adit (when driving adits), a tunnel (when driving a tunnel across its entire cross-section), a calotte (when driving the upper part of a tunnel using mining methods), etc. A workplace where soil is developed. This is the working face. The front part of the excavated soil is called the face. During the work, the face moves forward as the working progresses.

The structure installed to secure the tunnel workings during the period of work is called temporary support, and the one used for permanent securing of the workings-finishing or arrangement.

Temporary lining is designed to prevent collapse and soil falls on the roof, sides, and face of a mine. A permanent lining made of concrete, reinforced concrete, or cast iron tubing is erected beneath it. Temporary linings can be made of wood, metal, wood-metal, and, less commonly, reinforced concrete. The need for lining the roof, sides, face, and bottom of the mine, as well as the design of the temporary lining, are determined through a special work execution plan (WEP). In shield mining, temporary lining is used only to support the face, while the roof, sides, and bottom are supported by shield structural elements, under the protection of which the permanent lining is erected.

In general, the lining (or arrangement) of tunnels can have different outlines (Fig. 5.2).

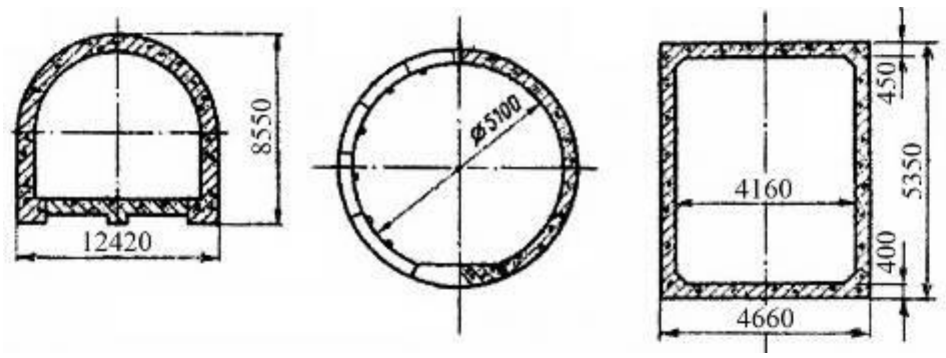


Figure 5.2 – Types of tunnel linings: a – horseshoe-shaped; b – circular; c – rectangular

Horseshoe-shaped linings are typically used in the construction of chambers, bell-shaped tunnels, and transport tunnels. Circular linings are used for subway tunnels constructed using the closed method. Rectangular linings are used for open-cut tunnels.

The following structural elements of the lining are distinguished: the upper part – the vault – in horseshoe-shaped and circular linings, and the ceiling – in rectangular linings. The lateral part of the lining – the walls, and the lower part – the reverse vault (flat tray).

Prefabricated circular linings are assembled from individual elements – tubing or blocks.

The construction of underground tunnels and the resulting overburden pressures require careful consideration. In natural soils, layers of soil are in a state of equilibrium and lie in horizontal or inclined strata. During tunneling operations, this equilibrium is disrupted. Deformations occur around the tunnel, manifesting as loosening, subsidence, and the shedding of individual pieces and entire blocks, which press on the tunnel supports.

In monolithic rock formations, unsupported workings can persist for long periods. In unstable sandy soils, temporary or permanent support must be installed immediately.

The stability of a working's roof is influenced by its shape and size. Vaulted roofs with a small span (width) are more stable. When a flat roof collapses, a natural arch forms within it, called a collapse arch (or pressure arch).

Hydrostatic pressure is essential for tunnel construction and operation. Excavation disrupts the natural water regime in the ground. This is especially true in aquifers. Groundwater from the surrounding rock flows into the excavation, from where it must be pumped to the surface. This leads to a lowering of the groundwater table in the ground. During this period, the lining experiences virtually no groundwater pressure. To prevent water from entering the constructed tunnel, waterproofing work is performed: controlled injection of cement mortar behind the lining, sealing of joints, and external or internal waterproofing. After these works are completed, water stops entering the tunnel, and it begins to fill cracks and voids in the surrounding rock, saturating them with water. The groundwater level gradually rises and, after some time, reaches its original level or even rises slightly higher. A layer of water forms above the tunnel, and the lining begins to experience pressure from the groundwater pressure, which is called hydrostatic pressure.

Tunnel linings are constructed from structures that ensure high strength, durability, longevity, and reliability over the specified service life. The most suitable material for this type of engineering structure is reinforced concrete or, in some cases, cast iron. Ceramic products such as bricks or precast blocks can also be used.

5.4 Tunnel design solutions

In recent years, the most common design solutions for tunnels of various purposes have been tunnels using precast reinforced concrete structures. Standard series 3.006-2 and 3.006-3 provide a variety of tunnel designs, which can be roughly categorized as single-section, dual-section, and multi-section.

Unified prefabricated structures for single-section tunnels are composed of corner wall elements combined with roof and floor slabs. For two-section tunnels, an additional row of intermediate columns is provided, upon which longitudinal purlins are laid, and upon which the roof slabs rest.

For tunnels with corner wall elements, the following overall dimensions are provided for the height of 2,1; 2,4; 3,0 and 3,6 m, the width according to the internal dimensions of the walls for single-section tunnels is 1,5; 1,8, 2,1 m, and for two-section ones – 2,4; 3,0; 3,6 and 4,2 m. The length of the prefabricated elements in the longitudinal direction is 3,0 m. The blocks are connected lengthwise by a quarter join along the walls and bottom, with dowels in the covering.

Tunnel designs also exist in the form of closed, integral elements – volumetric blocks – or as masonry made of artificial (natural) materials Volumetric blocks are the most technologically advanced and convenient to construct, and standard series have been developed for them. Figure 5.3 shows tunnel options made of volumetric blocks and artificial materials. These solutions allow for the construction of single-section, two-section, or multi-section tunnels, depending on their purpose, construction, and operational conditions.

In naturally damp soils, tunnels are coated with a two-layer adhesive waterproofing membrane consisting of Gidroizol or Akvaizol on a bitumen mastic. This is then protected with a 30–40 mm thick layer of cement-sand mortar. The walls are coated twice with hot bitumen.

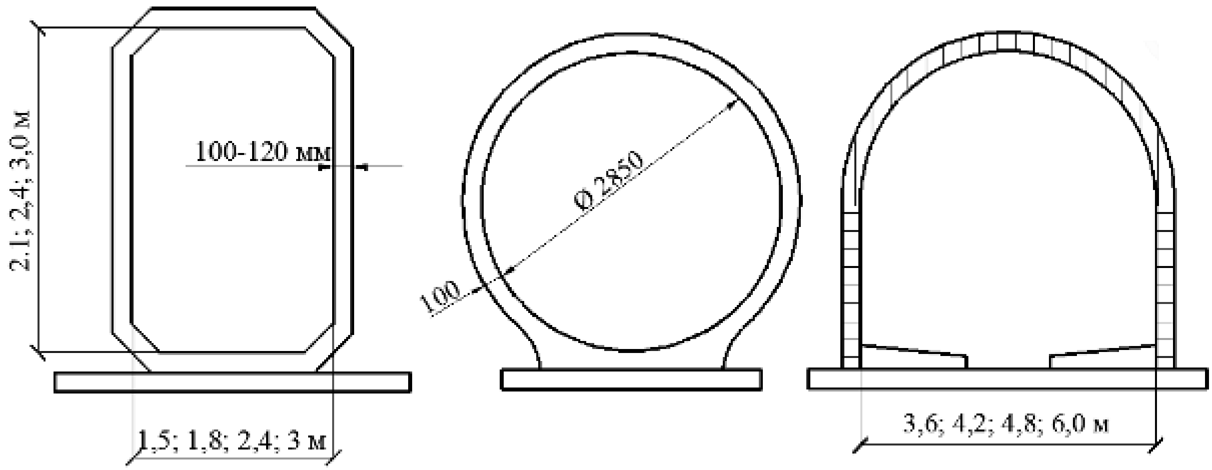


Figure 5.3 – Options for tunnels made of volumetric blocks and masonry:
a – rectangular volumetric blocks; b – ring-shaped volumetric blocks; c – laying tunnels with individual small blocks

The presented types of tunnel design solutions can be used for transportation needs (cars, railways, subways), technological purposes (water pipelines, sewerage, communication systems, heating mains, gas pipelines, etc.), pedestrian needs, military purposes and much more.

The tunnels must be provided with entrance shafts for people and mounting holes for the installation of process equipment. Temperature-deformation joints are installed 60–100 m along the length of the tunnels, ensuring the deformability of reinforced concrete or stone structures under conditions of temperature changes and the manifestation of soil pliability.

Tunnel designs have been developed for their placement under highways to a depth of 0,5–6 m to the upper surface of the tunnel, and under railway tracks to a depth from the bottom of the sleepers to the top of the tunnel of 1–4 m, in the middle of industrial buildings workshops – to a depth of up to 6 m. Regulatory documents recommend placing tunnels at least 0,7 m above the ground surface.

If there is groundwater, adhesive waterproofing must be placed under the bottom, as well as on the walls up to a height 0,5 m. Above the estimated groundwater level. The soil base under the canals and tunnels is compacted, and a sand backfill is applied to it for canals, and a concrete base 90–120 mm thick for tunnels. All assembly joints of precast elements are filled with M10–M15 grade cement mortar.

Precast elements for canals and tunnels are made from concrete grades C20/25 and C25/30, and also include enhanced water resistance grades W6 – W8. The structures are reinforced with welded or knitted meshes and cages made from A400C and A240 grade rebar, as well as standard B500 wire.

5.5 Calculation of shallow channels and tunnels

In most cases, tunnel and canal structures are subject to complex influences from various forces. These include external forces (soil pressure, temporary or permanent loads on the ground surface, hydrostatic pressure from groundwater, uneven subsidence of the soil foundation, seismic effects, and many others), as well as internal influences (the weight of internal utility systems, temperature fluctuations, additional pressure from tunnel filling, various emergency situations, repair work, etc.). Additionally, aggressive influences on the structures must be associated with the transportation of industrial wastewater, domestic sewage and other chemicals.

Developing a design scheme for tunnels is a complex task. Such a scheme must take into account not only the tunnel's cross-section but also its longitudinal section. This means that a tunnel is a complex spatial structure.

It is very difficult to simultaneously consider all these factors, so at the current stage of development of existing calculation methods, certain simplifications and limitations have been adopted in the selection, calculation schemes, and assignment of loads to tunnel structures. Specifically, calculation schemes are based on the general appearance of only the tunnel cross-section, ignoring the longitudinal parameter. Structural connection joints (hinged or rigid) are also somewhat simplified, and the flexibility of the structure is not considered.

Loads acting on the tunnel lining are divided into constant and variable. Constant loads include the structure's own weight; the weight of buildings and other structures on the ground surface; soil pressure; and hydrostatic pressure. Variable loads include: the weight of the process equipment in the tunnel; loads from train traffic in the middle of the tunnel and from ground vehicles; external pressure generated during tunneling operations (from grout injection behind the lining; and, during shield tunneling, pressure from hydraulic cylinders, etc.).

In the practice of tunnel design, the theory of Professor is used to determine the forces of rock pressure in our country. M. M. Protodyakonova, according to whom rock pressure is determined based on the assumption that a pressure dome (collapse dome) forms above the tunnel, within which the rock is loosened and presses on the lining. The weight of the rock located outside this dome exerts no pressure on the tunnel lining.

If tunnels are located at relatively shallow depths (5 – 10 m) in interbedded sandy-clayey soils, no collapse arch will form above the tunnel, and settlement may reach the ground surface. In this case, vertical rock pressure is created by the weight of the rock column (soil) between the tunnel roof and the ground surface.

The water pressure is taken to correspond to the pressure head determined for the soil layer in which the tunnel is located.

For tunnels constructed using the open method in pits, the vertical load consists of the weight of the soil filled above the tunnel, temporarily stored materials and the impact of moving ground vehicles.

Variable loads that may act on tunnel linings during construction or operation are determined based on regulatory documents for these types of loads. Seismic loads, which occur in seismically vulnerable areas, deserve special attention.

In addition to constant and variable loads, when calculating the external linings of tunnels (these can be individual blocks or volumetric structures), engineering-geological and hydrogeological conditions must also be taken into account. In this case, the lining, that is, the cross-section of the tunnel, is calculated for the most unfavorable combination of loads, for which the safety factor is taken from 1,2 to 1,4.

The lining is a permanent structure designed to securely support the internal surface of a mine working and give it a regular outline consistent with the design. The tunnel lining must withstand overburden or earth pressure along the entire working contour, hydrostatic pressure from groundwater, variable loads transmitted from the ground surface (in shallow tunnels), seismic effects, and other loads. The lining is a load-bearing structure; it must possess sufficient strength, stability, and water resistance. Therefore, when selecting the design and protective materials for tunnel linings, the geology and hydrogeology of the construction site, the tunnel depth, the clearance distance from existing buildings, climatic and seismic conditions, operational requirements, construction methods, labor intensity, and cost are all taken into account.

Shallow tunnels, like canals, are most often constructed using open-cut methods. In this case, the calculated load from the weight of the soil for the covering and walls – vertical p_1 and horizontal p_2 – are determined by the formulas:

$$p_1 = \gamma_f \cdot p_{\kappa_1} = \gamma_f \cdot \rho \cdot h; \tag{5.1}$$

$$p_2 = \gamma_f p_{\kappa_2} = p_1 tg^2 (45^0 - 0{,}5\varphi), \tag{5.2}$$

where h – distance from the ground surface to the upper surface of the tunnel, m;

ρ – average soil density depending on the type of soil and its moisture content is 16 – 20 kN/m^3;

γ_f – load safety factor, taken as 1?2;

φ – the standard angle of internal friction, which is within the range of 25–450.

The distribution of load from soil pressure is shown in Figure 5.4, a.

When calculating loads on the structures of canals and tunnels, the temporary load on the ground surface from vehicles must be taken into account. The values of the loads (NK and AK) are given in DBN B.1.2-15:2009 Bridges and pipes. Loading and intrusion [4].

For a tunnel under highways, the load is taken from two columns of three-axle NK-30 vehicles (with maximum axle pressure $P_k = 120$ kN; Fig. 5.5, a), in other cases, that is, when the tunnel is not located under a highway, the load from one column of two-axle NK-10 vehicles (with maximum axle pressure) is taken into account $P_k = 95$ kN; Fig. 5.5, b). In this case, the load safety factor is taken to be 1.4. Thus, the calculated load from the wheels (P) makes up $1{,}4P_k$.

The distance between the wheels of the vehicles in the transverse direction is taken as shown in Figure 5.5, c. The support area of one wheel is taken equal to 0.2 min longitudinal and 0.6 min the transverse direction (Fig. 5.5, g). The uniformly distributed load AK (A8 and A11) is also taken into account if the tunnel passes under a highway.

Vertical pressure on depth from the earth's surface $h < 1{,}2$ m is determined by the formula:

$$p_1 = \frac{P}{a \cdot b}, \tag{5.3}$$

where a And b – dimensions of the pressure area at a variable depth h_{vario} (see Fig. 5.4, b; Fig. 5.5, d).

At a depth $h \geq 1{,}2$ m. The pressure from the vehicles is taken as a vertical load (with a safety factor for the load) $\gamma_f = 1{,}4$) characteristic value $P_{\kappa_1} = 20$ kN/m^2 (Fig. 5.4, d).

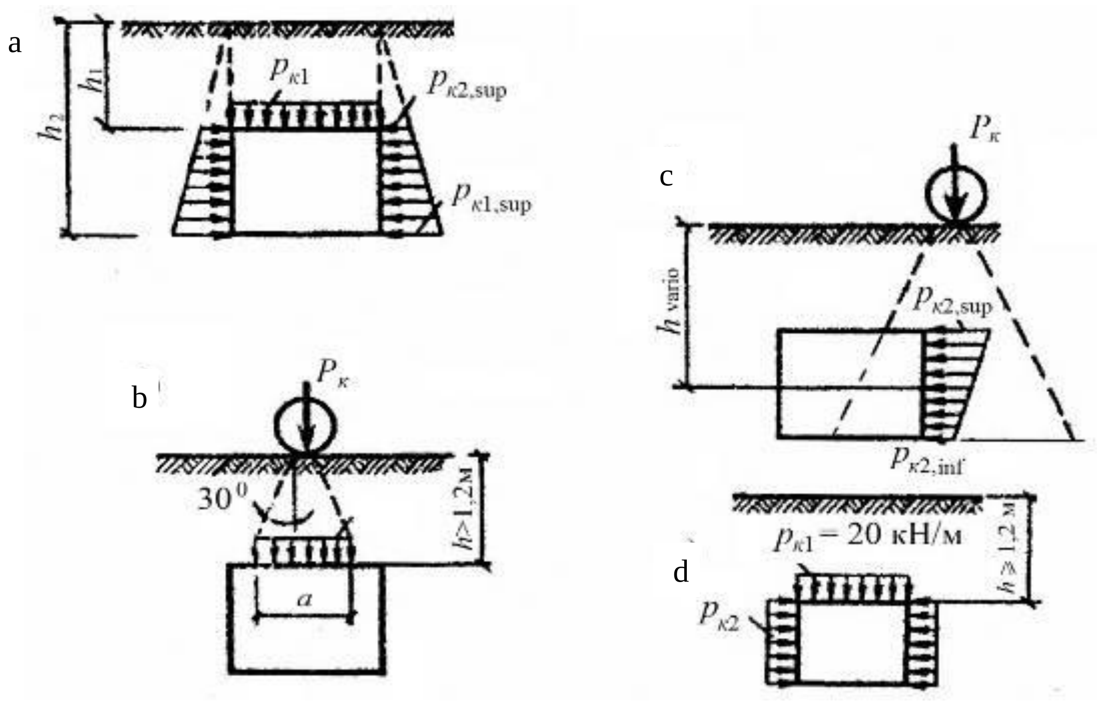


Figure 5.4 – Load diagrams for underground channels and tunnels:

a – vertical and horizontal pressure from the soil; b – vertical pressure from the transport load on the earth's surface at h < 1,2m; c – horizontal pressure from transport *h* < 1,2m; d – vertical and horizontal pressure on the tunnel surface from vehicles at h ≥ 1,2 m

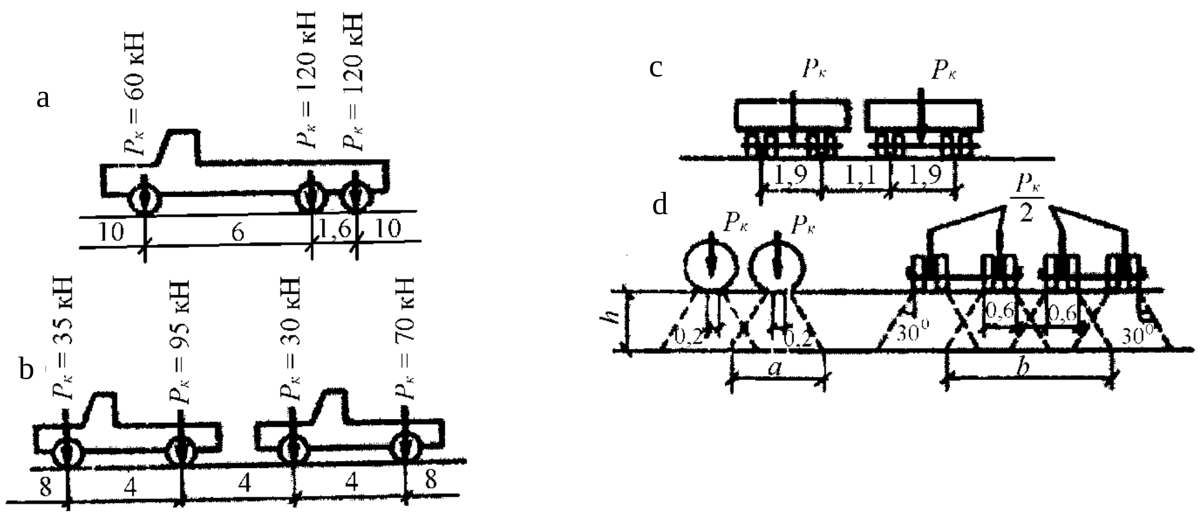


Figure 5.5 – Vehicle load diagrams:

a – location of axles and axle loads for NK 30 vehicles; b – the same for NK 10 load; c – distance between the wheels of the cars in the transverse direction; d – distribution pressure from the bearing area of the car wheel

Horizontal pressure soil and the load on the ground surface in both cases are determined by formula (5.2) with the moment diagrams shown in Figure 5.7, a, b, c.

Three main types of design schemes for channels and tunnels can be considered (Fig. 5.6, a, b, c).

In underground structures, the entire vertical load from the ceiling and walls is balanced by the reactive pressure of the soil, which is considered to be uniformly distributed over the base of the bottom.

The roof slabs of canals and tunnels are calculated using a single-span beam scheme with a hinge resting on supports. Walls rigidly connected to the bottom in single-section channels and tunnels - according to the diagrams shown in Fig. 5.6, a, b, c.

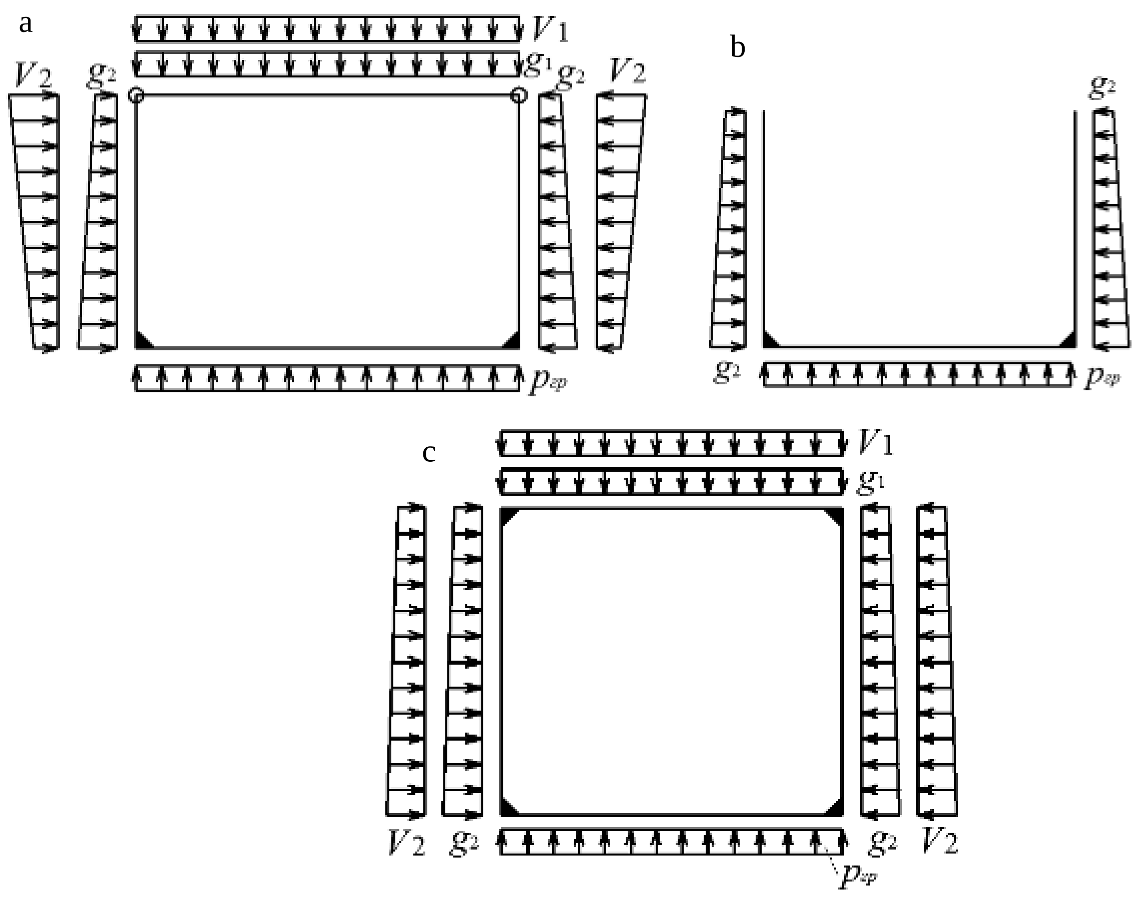


Figure 5.6 – Possible design schemes for tunnels:

a – in the form of an inverted U-shaped frame with a strut and a hinge connection; b – in the form of an inverted U-shaped frame without spacers; c – in the form of a closed rectangular frame

In addition to the efforts M forces also appear in the tunnel elements N And V , which must be taken into account when calculating the load-bearing capacity of the bottom, walls, and roof. The most stressed areas in channels and tunnels are the lower joints between the walls and the bottom, so these areas are typically reinforced only with double mesh for the bottom and walls. Haunches are installed at these joints to reduce stress concentration at the junctions of the walls and bottom. Inclined rods are also installed in these haunches to increase the reliability of the connection between the walls and the bottom. The diameter of the working reinforcement is accepted within the range of 8 – 12 mm. Reinforcement class in most cases is A400C, concrete class is C16/20; C20/25.

If a tunnel is constructed as a vaulted system, the calculation methods are applied as for non-hinged arches. Only the load from the round pressure is applied. For water-driven collectors (tunnels), an additional load from the water pressure is applied depending on the tunnel's fill level. For transport tunnels, the loads on the bottom are the traffic flows (vehicle, rail, pedestrian, etc.).

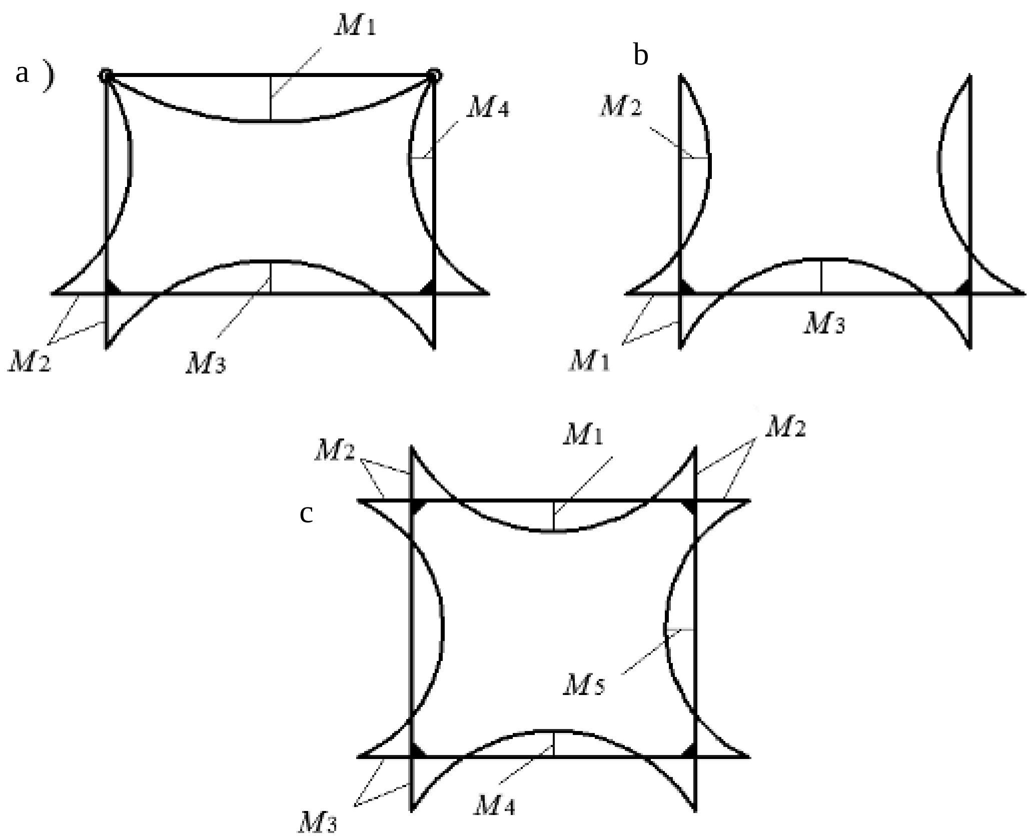


Figure 5.7 – General view of the moment diagrams for calculation schemes shown in Figure 5.6

The efficiency of tunnel construction in global practicing due to further improvement and widespread implementation of advanced designs and technologies, the implementation of comprehensive mechanization of work based on scientific and technological progress, increased labor productivity, and a reduction in the cost and material intensity of tunnel engineering structures.

A range of efficient universal equipment is being developed for use in a variety of tunnel construction conditions; the latest materials (concrete, mortar, and reinforcement) are being used to improve the durability and reliability of engineering structures.

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