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Translation

HANDBOOK FOR DESIGNING BUILDING COMPONENTS FOR CIVIL DEFENSE SHELTERS

By

V.F. Baranov, et al.

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HANDBOOK FOR DESIGNING BUILDING COMPONENTS FOR CIVIL DEFENSE SHELTERS

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CONTENTS

ANNOTATION

[Text] Compiled to develop the "Instructions for Designing Civil Defense Shelters" (SN [Construction Standard] 405-70). Contains basic provisions and materials for location, planning, design and calculation of supporting and protective structures of built-in and freestanding[1] shelters combined with areas used in peacetime for needs of the national economy.

Intended for designers drawing up standard and individual shelter designs, workers of civil defense staffs, and specialists engaged in appraisal and acceptance of these designs.

1. [Here and elsewhere, used in translating Russian expression "shelter standing apart"]

FOREWORD

The Handbook was drawn up by TsNIIpromzdaniy [Central Scientific Research and Experimental Design Institute of Industrial Buildings and Structures] of USSR Gosstroy and the USSR Ministry of Defense with the participation of Workshop No 18 of Mosproyekt-1 and certain other design organizations.

The Handbook includes three chapters of SN 405-70, the text of which is denoted by a vertical line in the left-hand margin. Critical remarks and suggestions which arose in the process of designing shelters, and USSR Gosstroy decrees on amending individual paragraphs of SN 405-70, were considered in compiling the Handbook.

Explanations are given for each paragraph of the Instructions which give a justification and recommendations on the procedure for using these instructions, as are additional data on space planning and design decisions and on calculating structures for a dynamic load from a shock wave. Handbook materials are illustrated with examples of solutions and calculations with corresponding drawings, diagrams and charts. In some instances the numbering of figures, tables and formulas is dual--numbers corresponding to the SN 405-70 Instructions are given in parentheses.

Participating in compilation of the Handbook were: architect V. F. Baranov, engineers S. A. Lokhov and L. M. Korshak†, Doctor of Technical Sciences M. P. Tsivilev, candidates of technical sciences V. I. Morozov, S. B. Rastorguyev, P. I. Yartsev, V. I. Ganushkin, M. D. Bodanskiy, A. I. Kostin and V. P. Krysin. and Engineer D. V. Myl'nikova.

You are requested to send critical remarks and suggestions on the Handbook to the address: 127238, Moscow, Dmitrovskoye shosse 60b, TsNIIpromzdaniy.

1. BASIC PROVISIONS

1.1. These Instructions extend to the designing of spaces located in basement levels of production and auxiliary buildings of industrial enterprises, residences and public buildings as well as in buried, freestanding structures intended in peacetime for needs of the national economy and adaptable in wartime for the protection of workers and employees (or working shifts) against the injurious effects of nuclear weapons, toxic chemical agents and biological warfare agents.

Notes: 1. For the sake of brevity, the text of the Instructions subsequently will designate as "shelters" spaces located in basement levels of buildings and buried, freestanding structures intended in peacetime for needs of the national economy and adaptable in wartime for protection of people sheltered therein against the injurious effects of nuclear weapons, toxic chemical agents and biological warfare agents.

2. In addition to the recommendations and requirements set forth in these Instructions, requirements of appropriate chapters of the SNiP [Construction Standards and Specifications] and other standardizing documents approved and coordinated by USSR Gosstroy should be followed in designing spaces adaptable as shelters.

For Paragraph 1.1. In those instances where buildings used for needs of the national economy lack basement levels, a portion of the spaces should be designed to be buried and adapted as shelters with observance of all requirements of SNiP chapters and other standardizing documents, and in conformity with requirements of the SN 405-70 Instructions.

1.2. In designing spaces adaptable as shelters, provisions should be made for the most progressive space-planning and design decisions allowing a reduction in structural weight, expenditure of materials and construction costs and an improvement in the technical and economic indicators of the projects as a whole. The dimensions of the spaces should be set at the minimum ensuring fulfillment of requirements for use of those spaces in peacetime for the needs of the national economy and as shelters in wartime. In choosing components and finishing materials, a cost effectiveness analysis of their use should be performed for each construction project, with consideration of the availability of appropriate production facilities and material resources.

For Paragraph 1.2. Particular attention must be given to reducing the area of spaces for sheltered persons and for internal engineering and technical equipment to the limit of permissible standards with the use of the simplest arrangements and small-size units, and to the need for using this equipment for production needs in peacetime without disassembly.

In drawing up plans for adapting spaces as civil defense shelters, an attempt also should be made to simplify engineering decisions in order to create conditions for a reduction in time periods and in construction costs.

1.3. As a rule, spaces adaptable as shelters should be designed to be built into basement levels of buildings and structures with a degree of fire resistance of I or II. The design of such spaces as freestanding buried structures is permitted when there is no opportunity to install built-in shelters.

Designing of freestanding buried or semiburied shelters also is permitted under difficult hydrogeological conditions when their estimated cost will be less than the cost of built-in shelters, and when the construction of these shelters will not entail a complication in the accommodation of other projects of the enterprise master plan.

Shelters are divided into classes according to SN 405-70 based on their degree of protection.

For Paragraph 1.3. A built-in shelter is a structure intended for protection of people and accommodated in the basement or semibasement level of an administrative services, production or auxiliary building, as well as of a residence or public building located next to the enterprise grounds.

Built-in shelters can be designed under the entire building or under a certain part (Fig. 1). Entrances, emergency exits, air intakes, and exhaust ducts may project beyond the limits of the building.

A freestanding shelter is a structure intended for protection of people, erected on a sector of an industrial enterprise which has no buildings, buried fully or partially in the ground and covered on top and on the sides with soil (Fig. 2).

The statement on the preferability of using built-in shelters is explained as follows:

As a rule, the cost of a built-in shelter is less than that of a freestanding shelter;

Connection of the shelter with production spaces is most convenient, providing conditions and the opportunity for the workers to fill it rapidly;

Built-in shelters do not take up industrial grounds, which ensures their most rational use in peacetime and does not detract from the technical and economic indicators of the master plan.

1.4. The basis for a design of spaces adaptable as shelters must be the elaboration of space-planning decisions of spaces intended for the needs of the national economy, in conformity with requirements of chapters of SNiP, Part II,

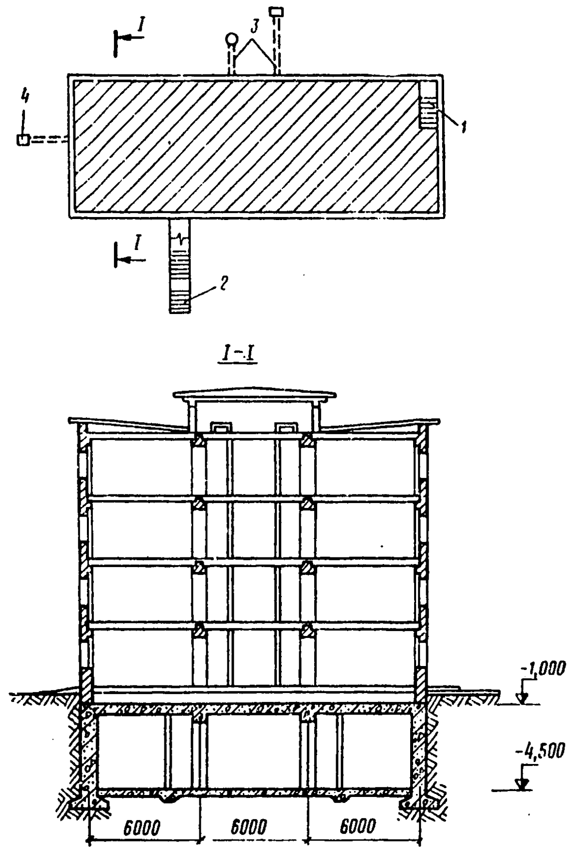


Fig. 1. Built-in shelter in an administrative services building:

1. Entrance No 1
2. Entrance No 2
3. Air intakes
4. Exhaust blower cap

"Construction Design Standards," supplemented by necessary design, space-planning and other decisions, for the purpose of adapting spaces as shelters in conformity with the requirements of SN 405-70.

Shelter design should begin by determining the make-up of spaces which must be accommodated in the protected part of a building basement or in a buried or semi-buried freestanding shelter and intended for normal operation in peacetime. The area of these spaces must not exceed the area needed for a shelter under standards envisaged by these Instructions.

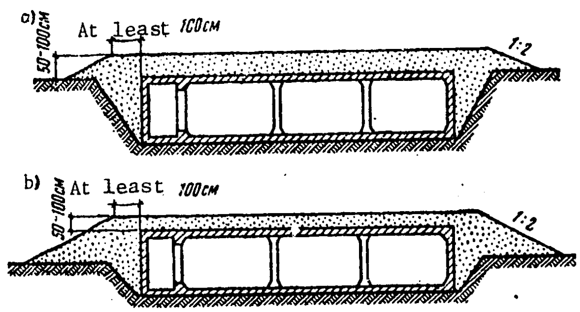


Fig. 2. Freestanding Shelter:

a. Fully buried
b. Semiburied

Space-planning decisions for these spaces subsequently must be supplemented with decisions of bearing components and engineering-technical equipment made in accordance with requirements placed on shelters.

For Paragraph 1.4. The supplementary decisions not covered by the SNiP requirements or by other valid normative documents determining the make-up and design decisions of spaces to be used in peacetime include the following:

Installation and reinforcement of supporting and protective components providing protection against injurious effects of a nuclear burst and fires for sheltered persons;

Installation of protected entrances, fore airlocks, airlocks, airlock-sluices, and emergency exits;

Installation of bunks or benches for accommodating sheltered persons;

Pressurization of shelters and the supply of clean air to persons therein under all air-supply conditions;

Installation of supplementary toilets, containers for storing emergency supplies of potable and recycled water, containers for emergency collection of drainage water, and protected stand-by sources of electrical power;

Installation of protected caps;

Structural arrangements at entrances to engineering lines.

Standard, industrially manufactured components as well as small standardized equipment must be used to the maximum possible extent in designing these installations.

The design of spaces and equipment not to be used in peacetime must be in conformity with SN 405-70 or other valid normative documents which take account of the features of periodic and short-term use of shelter spaces and equipment.

1.5. The following spaces, accommodated in basement levels and buried structures, should be used as shelters:

Personal services rooms (cloakrooms for personal and working clothing with showers and washrooms, smoking rooms, storerooms);

Cultural and personal services rooms (reading rooms, lounges, technical training rooms);

Production spaces in which manufacturing processes are carried out which are not accompanied by the release of harmful liquids, vapors and gases dangerous to people, which require no natural illumination, and which accommodate production fall into to categories Г and Д in fire danger;

Pedestrian and transportation tunnels, rooms for on-duty fitters, electricians and repair teams;

Passenger vehicle garages (only in the form of freestanding structures);

Warehouse spaces for storage of noncombustible materials;

Trade and public nourishment spaces (stores, dining rooms, canteens, coffeehouses, milk distribution points);

Sports facilities (shooting galleries and rooms for athletic activities not requiring natural illumination);

Combines for everyday services to the populace, ZhEK [housing and housing-maintenance] offices and workshops, receiving stations for renting housewares, shoe and clothing repair shops and so on.

Notes: 1. In addition to the list provided in Paragraph 1.5, USSR ministries and departments can establish sector lists of spaces adaptable as shelters in coordination with the USSR Ministry of Health, the USSR MVD [Ministry of Internal Affairs] GUPO [Main Administration for Fire Protection], the USSR Civil Defense Staff and the USSR Gosgortekhnadzor [exact expansion unknown].

2. It is permissible to use warehouse spaces for storage of noncombustible materials as shelters in those instances where there is no opportunity to use other spaces for these purposes.

For Paragraph 1.5. Design practice indicates that cultural and personal services spaces, personal services rooms, and pedestrian tunnels intended for the passage of a small number of people are most suitable for outfitting shelters therein. The advantages of these spaces is that they lack means of transportation, they are accessible at any time and can be designed to be of low height with use of a small bay size without detriment to their use in peacetime.

When using warehouse spaces as shelters, consideration must be given to the fact that these spaces are permitted to store only noncombustible (nonflammable and nonexplosive) materials, there must be physical liability for the safekeeping of stored material, entrances as a rule must be from the enterprise grounds, and their size has to permit the use of appropriate means of mechanization for loading and unloading the warehouse.

Dining rooms, warehouse spaces and technical spaces in public nourishment facilities (dining rooms, canteens, coffeehouses) can be used for accommodating people to be sheltered.

Trade and personal services spaces, offices, workshops, ZhEK and so on, located in a city building in the immediate vicinity of an enterprise, should be adapted as shelters in those instances where it is impossible to accommodate protective structures on the enterprise grounds.

The need may arise to preserve production activities in some of the enumerated spaces in enterprises functioning in wartime. Such spaces may include personal services rooms, certain production and warehouse spaces, rooms for on-duty fitters, electricians, repair teams and so on. Space-planning decisions for shelters in these spaces should be worked out with consideration of the accommodation of sheltered persons without dismantling of equipment and without removal of the minimum materials necessary for production activities.

The production work of such spaces as trade and sports facilities, housewares rental points, shoe and clothing repair shops and others ceases in wartime and the equipment not being used when these spaces function as shelters can be removed when necessary.

The space-planning decisions for a cloakroom with shower, a tool warehouse and a store adaptable as shelters are given as an example in figures 3-8.

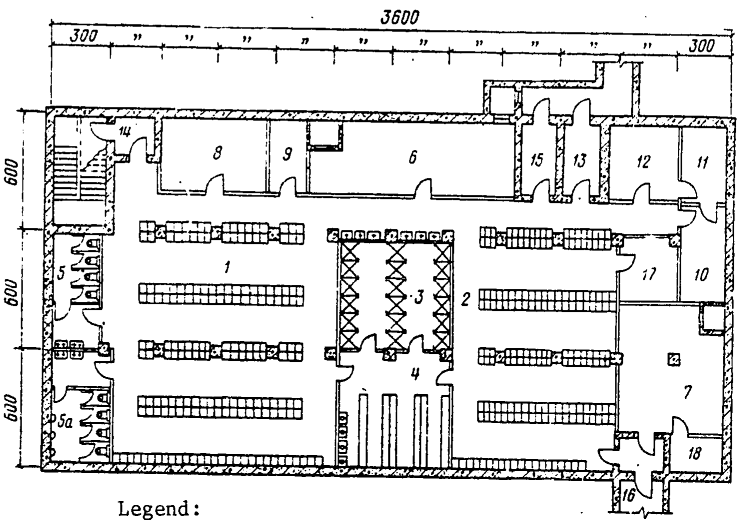
A cloakroom with shower room (Fig. 3 and 4) is designed in the basement of a four-story administrative services building to be serviced by a working shift of 180 persons.

The cloakroom is equipped with double lockers for street and house clothing, shower units, and single lockers for work clothing. With the facility's simultaneous use for services and as a shelter, the lockers for storage of street, house and work clothing are replaced with benches. This decision permits the use of the cloakroom with shower as a shelter without halting the production process.

A similar decision was made for the tool storeroom (Fig. 5 and 6), also located in the basement of an administrative services building. The design and placement of shelves in the storeroom and outfitting of the storage area is calculated for the accommodation of sheltered persons without dismantling the equipment. This permits combining the purpose of the facility for the needs of the national economy and for sheltering the work shifts.

A store with eight work stations (Fig. 7 and 8) is designed in the buried portion of a freestanding building of a trade and public dining enterprise. A combination coffeehouse and dining room is located in the above-ground portion.

The design envisages cessation of the store's production activities in a special period and dismantling of a portion of the equipment. Compressors in the machine room, evaporator batteries in the refrigeration room, and the refrigerated and low-temperature counters are not dismantled.



Legend:

Lockers for street, house and work clothing

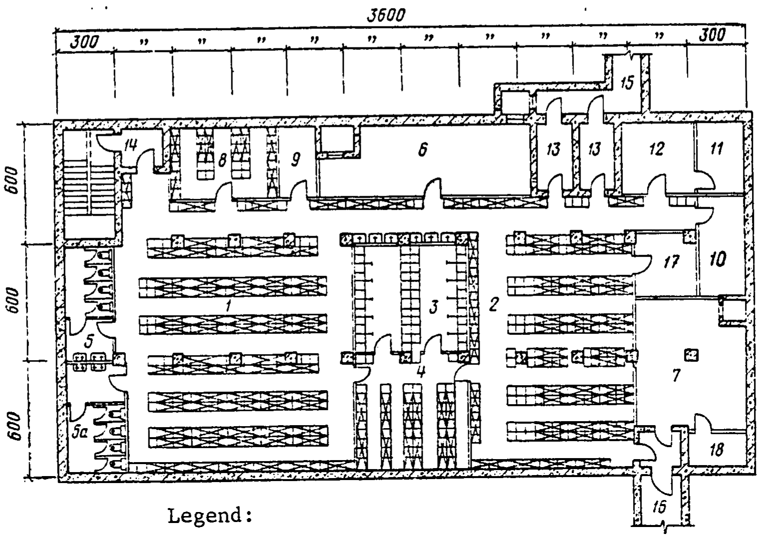
Footbaths

Fig. 3. A design decision for cloakroom with shower room in building basement:

1. Cloakroom for street and house clothing with 180 double lockers
2. Cloakroom for work clothing with 180 single lockers
3. Shower room
4. Dressing room in front of shower room
5. Women's toilet

5a. Men's toilet

6. FVK [filter-ventilation chamber]
7. Diesel generator room
8. Hair drying and small repairs room
9. Clean clothing closets
10. Dirty clothing closets
11. Container room
12. Staff room
13. Stock room
14. Entrance No 1
15. Entrance No 2
16. Entrance No 3 (emergency exit)
17. Panel room
18. GSM [fuels and lubricants] storeroom



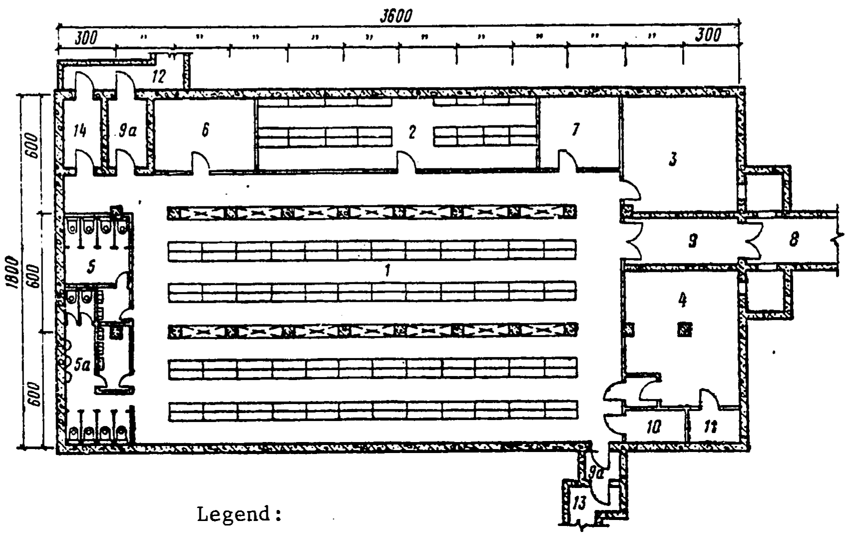
Legend:

– Places for sitting

– Places for lying down

Fig. 4. A design decision for 850-person shelter combined with cloakroom and shower room

1-4; 8. Spaces for accommodation of sheltered persons
5. Women's toilet
5a. Men's toilet
6. FVK
7. Diesel generator room
9. Clean clothing closet
10. Dirty clothing closet
11, 12. Control post
13. Sluice chambers [shlyuzovyye kamery]
14. Entrance No 1
15. Entrance No 2
16. Entrance No 3 (emergency exit)
17. Panel room
18. GSM storeroom



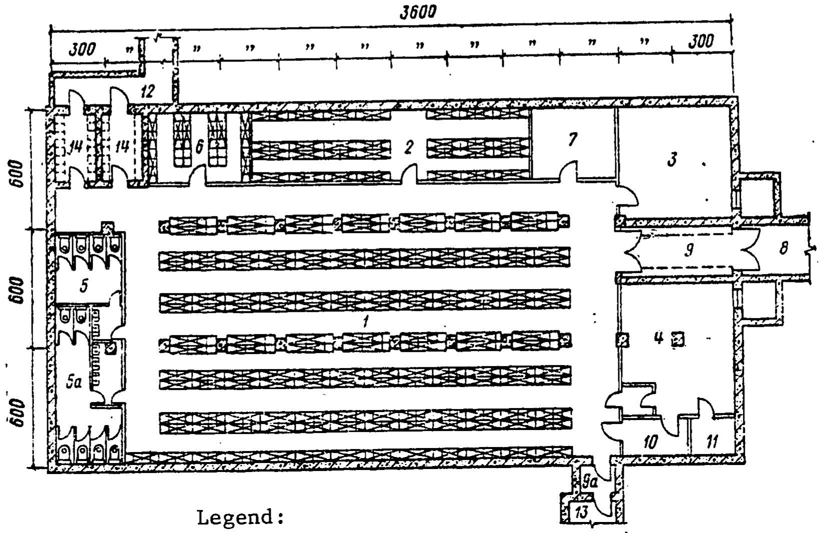
Legend:

- Shelves

- Storage places

Fig. 5. A design decision for central tool storage in building basement

1. Storage space
2. Tool storage
3. FVK
4. Diesel generator room
5. Women's toilet
5a. Men's toilet
6. Closet
7. Storage office
8. Inclined ramp
9. Airlock-sluice [tambur-shlyuz]
9a. Airlock [tambur]
10. Panel room
11. GSM storeroom
12. Entrance No 1
13. Entrance No 2 (emergency exit)
14. Rag storage



Legend:

□ – Place for sitting

⌧ – Place for lying down

Fig. 6. A design decision for 900-person shelter combined with tool storage

1, 2, 6. Spaces for accommodating sheltered persons
3. FVK
4. Diesel generator room
5. Women's toilet
5a. Men's toilet
7. Control post
8. Inclined ramp
9. Inclined ramp sluice
9a. Airlock
10. Panel room
11. GSM storeroom
12. Entrance No 1
13. Entrance No 2 (emergency exit)
14. Sluice chambers

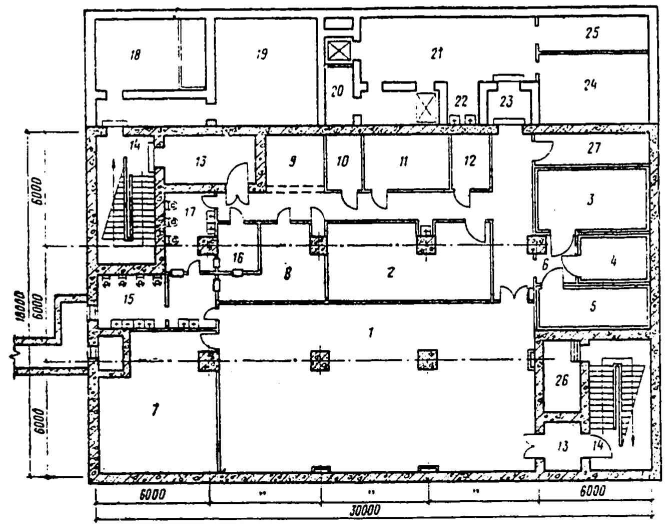
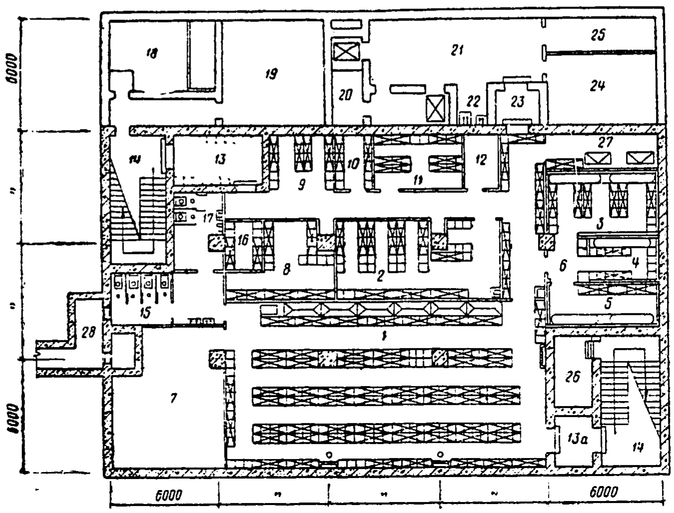


Fig. 7. A design decision for store in buried portion of building

1. Sales room
2. Preparation of goods for sale
3. Meat refrigerator
4. Vegetable room
5. Fish refrigerator
6. Cold chamber airlock
7. Ventilation plant
8. Women's cloakroom
9. Cloakroom for outer clothing
10. Administrative storeroom .
11. Fish storeroom
12. Electric panel room
13. Airlock
14. Stairs
15. Cloakroom for special sanitary work clothing
16. Linen room
17. Men's cloakroom
18. Pump room
19. Heating station
20. Engine room
21. Reception room
22. Washroom
23. Vestibule
24. Vegetable storage
25. Packaging room
26. Heat curtain room
27. Engine room for refrigeration chambers



Legend:

- Place for sitting
- Refrigerated and low-temperature counters
- Place for lying down
- Evaporation battery
- Compressor

Fig. 8. A design decision for 600-person shelter combined with store

1-6, 8-11, 16, 27. Spaces for accommodating sheltered persons
7. FVK
12. Panel room
13. Airlock-sluice
14. Shelter entrance
15. Women's toilet
17. Men's toilet
26. Heat curtain room
28. Emergency exit
18-25. See Fig. 7.

The activity of the coffehouse-dining room does not have to be halted.

1.6. In designing spaces adaptable as shelters, it is authorized, based on conditions of their peacetime use, to install in protective structures the necessary industrial openings fitted with appropriate protective devices.

The conversion of spaces used in peacetime to a shelter regime should be planned for a short period of time (in conformity with SN 405-70).

For Paragraph 1.6. The size of industrial openings depends on the purpose of the facility. Fixed ramps and openings can be made in walls of garages and storage facilities for the entrance and exit of vehicles, forklifts, power trucks and battery-operated trucks. Openings for loading stored materials using conveyors and worm feeders also can be planned in storage facilities.

Openings for the loading and unloading of freight using elevators or cranes can be planned in stores, dining rooms and warehouses.

Industrial openings can be protected by using standard airtight-blast or airtight doors, shutters, gates or seals, and by installing a prop wall on the outer side out of previously prepared reinforced concrete or metal elements. It is preferable to employ doors, shutters, gates or seals in choosing protective devices for industrial openings, inasmuch as this provides an opportunity to make the shelter combat-ready faster. The product list and design of protective devices for industrial openings are given in Appendix 1 (3) and in Fig. 49.

1.7. In designing facilities which will be occupied by fixed equipment, dismantling of the latter is not envisaged in converting to the shelter regime. As a rule, the area occupied by this equipment should not exceed 40 percent of the facility's total area. In case the fixed equipment occupies more than 40 percent of the facility area, its use as a shelter is authorized only with the appropriate feasibility study.

For Paragraph 1.7. In each specific instance, for the purpose of rational use of the total protected area of spaces adaptable as shelters, it is necessary to decide the question of the possibility of using or adapting certain kinds of nondismantled equipment for accommodating the sheltered persons. Only equipment which cannot be used for accommodating sheltered persons should be included with the fixed nondismantled equipment which should not occupy over 40 percent of the total protected area of a facility.

1.8. The capacity of facilities adaptable as shelters is determined by the sum of places for sitting (in the 1st tier) and lying down (in the 2d and 3d tiers) and as a rule is accepted at no less than 150 persons. The designing of shelters holding 50-100 persons is permitted with the appropriate feasibility study [tekhniko-ekonomicheskoye obosnovaniye].

Note. The designing of shelters holding 20-40 persons is permitted in exceptional cases by authority of USSR ministries and departments and is accomplished in conformity with specifications approved under established procedures.

For Paragraph 1.8. In designing facilities adaptable as shelters, it should be borne in mind that the proportionate cost of large-capacity structures is considerably less than for small-capacity structures, and that they can be used more effectively for national economic needs.

The power supply can be provided more reliably in large shelters inasmuch as it becomes economically permissible to install protected diesel-electric power plants (DES) in them.

Shelters holding fewer than 100 persons are especially unprofitable economically, inasmuch as the cost of such structural elements as entrances, emergency exits and airlocks provides for a substantial cost increase of the shelter as a whole. In addition, in a majority of instances the spaces of small-capacity shelters cannot be used effectively for national economic needs.

1.9. The task of designing spaces adaptable as shelters is a component part of the task of designing new enterprises, buildings and structures and reconstructing existing ones.

The class of shelter should be indicated in the design task in conformity with SN 405-70.

The stages in designing built-in facilities adaptable as shelters should be established in conformity with the "Provisional Instruction on Drawing up Plans and Estimates for Industrial Construction" (SN 202-69) and the "Provisional Instruction on Drawing up Plans and Estimates for Civilian Housing Construction" (SN 401-69).

The drafting of standard plans for freestanding facilities adaptable as shelters as well as standard design decisions must be done in two stages. The designing of structures using standard plans must be done in one stage.

Materials of the engineering plans are part of the plans of the aforementioned enterprises, buildings and structures and are made up in the form of an independent section.

Blueprints are published according to established procedures.

For Paragraph 1.9. The portion of the design task concerning shelters indicates:

The instructions and directions serving as the basis of the requirements for construction of shelters;

The maximum possible working shift of the enterprise and its allocation to shops, buildings and structures;

The description of the buildings and structures having basement levels which are expedient to use for adaptation as shelters based on the nature of the production process;

The peacetime purpose of facilities adaptable as shelters, their equipment, the industrial process planned for them, and requirements for adaptation of equipment in the primary spaces for accommodation of sheltered persons;

Class of shelters;

Ratio of men and women to be sheltered;

Primary requirements on space-planning and design decisions of shelters;

Sources of water, heat and electrical power for the shelters;

Location for leading off drainage water;

Hydrogeological conditions of the construction site;

Primary technical and economic indicators which must be attained in designing shelters in basement levels or freestanding buildings (structures).

The designing of built-in or freestanding structures adaptable as shelters can be accomplished:

In two stages--the engineering plan and blueprints;

In one stage--engineering-detail plan (engineering plan combined with blueprints).

An independent section of the engineering plan should include: accommodation of places for sheltered persons and an explanatory note with a substantiation of the decision adopted in the plan as well as an extract from the enterprise master plan indicating the accommodated shelters, assembly radii and movement routes of sheltered persons from work areas to shelter entrances.

1.10. The estimated cost of built-in facilities adaptable as shelters should be determined from the estimate according to Appendix 5 to the Provisional Instruction for Drafting Plans and Estimates for Industrial Construction (SN 202-69) and the expenses for construction of these facilities should be included in the project estimates of buildings and structures.

An explanatory note to the engineering (engineering-detail) plan for built-in and freestanding facilities adaptable as shelters should provide the technical and economic data on supplementary expenses for adapting facilities as shelters.

For Paragraph 1.10. Supplementary expenditures for stages of the engineering (engineering-detail) plan should be determined as the difference between the estimated cost of facilities adaptable as shelters and the average cost of facilities used in peacetime.

The estimated cost of facilities adaptable as shelters can be determined based on consolidated indicators or as the difference of the estimated cost of the project with or without a basement (shelter).

The estimated cost of freestanding shelters should include expenses for hooking up to utility mains to the extent established by normative documents for buildings and structures if the consolidated estimate provided for their development; otherwise the estimated cost includes the utility mains to the full extent, but supplementary expenses include only the hook-up thereto. The estimated cost does not include cost of grading and other area work for the construction and outfitting of control posts and protected DES and artesian wells with their use as an emergency (reserve) source of water and electrical power supply to the enterprise.

The cost of facilities used in peacetime is taken based on the estimated cost in the enterprise of one square meter of area or one cubic meter of volume of similar facilities under ordinary above-ground conditions.

Supplementary shelter costs should be determined based on one sheltered person from the formula

$$C_y = C_M - KC_\Pi, \qquad (1)$$

where C_y represents supplementary costs per person sheltered;
C_M is the estimated cost of facilities adaptable as shelters per person sheltered;
C_Π is the average cost of facilities used in peacetime;
K is the area per sheltered person used in peacetime for enterprise needs, in square meters.

Example. The estimated cost of a built-in shelter in personal services facilities per person sheltered is 140 rubles. The average cost of one square meter of personal services facilities in the enterprise is 120 rubles. The area of facilities used in peacetime per person sheltered is 0.52 square meters. Supplementary costs per person sheltered are

$$C_y = 140 - 0{,}52 \cdot 120 = 77 \text{ rubles 60 kopecks.}$$

Supplementary costs also can be determined from the formula

$$C_y = \frac{C'_M - C'_\Pi}{M}, \qquad (2)$$

where C'_M is the estimated cost of facilities adaptable as shelters;
C'_Π is the estimated cost of similar facilities being used in peacetime;
M is the number of persons sheltered.

Example. A buried freestanding auxiliary production building is adapted as a shelter for 1,000 persons. The estimated cost of this adapted facility is 270,000 rubles. The cost of one cubic meter of a similar project is 80 rubles, with 2,000 cubic meters being used in peacetime. The supplementary costs per person sheltered are:

$$C_y = \frac{270\,000 - 80 \cdot 2000}{1000} = 110 \text{ rubles.}$$

The cost indicators per person sheltered must be shown as a fraction, with the numerator being the estimated cost and the denominator being supplementary costs per person sheltered. In the examples given they are 140/78 and 270/110.

Placement of Shelters

1.11. Spaces adaptable as shelters should be located in places with the greatest concentration of people to be sheltered. The assembly radius of sheltered persons should be taken in accordance with SN 405-70. In instances where there are groups of at least 100 persons outside the limits of the assembly radius, cover should be provided for them in the nearest shelter having an airlock.

For Paragraph 1.11. The assembly radius for sheltered persons is understood to be their maximum permissible distance from shelter entrances. The distance of a shelter entrance from the farthest exit from a production building in which people to be sheltered are located can be determined from the chart in Fig. 9 depending on the building width B, density of workers P and shelter capacity.

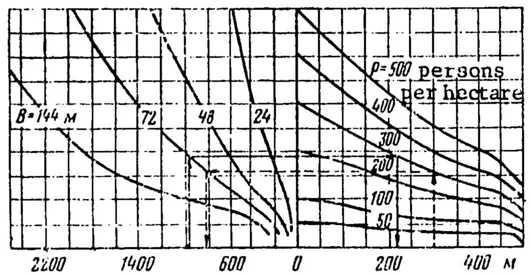


Fig. 9. Chart for determining distance of freestanding shelters from production buildings

B--Building width, meters; P--worker density, persons/hectares

Calculation example. 1. There are 1,000 persons in a shop to be sheltered. The one-story building has a width B = 72 meters and worker density P = 300 persons per hectare. Determine permissible shelter distance on the condition that building exits are according to SNiP. The answer is no more than 220 meters. 2. A shelter is being built 300 m from the shop. How many people can be given refuge in this shelter (B = 72 meters, P = 300 persons per hectare)? The answer is 850 persons.

The time needed for people to descend from upper floors should be taken into account in determining assembly radii at enterprises with a multistory building. The values of the permissible distance taken from SN 405-70 or the chart in Fig. 9 must be reduced for this purpose by three times the height of the corresponding floors. For example, persons to be sheltered descend by stairways from the sixth floor of a production building located at a height of 27 meters. In this instance the assembly radius for a built-in and freestanding shelter should be decreased by 81 meters.

Emergency exits of freestanding shelters must be at a distance from neighboring buildings and structures of at least half their height plus three meters.

As a rule, freestanding shelters and facilities adaptable as built-in shelters should be located in sectors of fire safety or sectors with a III category of fire danger.

Provisions should be made for the possibility of a convenient access to shelters and evacuation of sheltered persons from them to grounds safest from obstructions (areas without buildings, wide passages, road sites for access routes and so on). In choosing a sector for erecting a shelter, avoid waterlogged and structurally unstable soils, heavily pitched beds of sedimentary rock, areas inundated by storm and flood waters, and areas subject to flooding with the possible destruction of hydraulic works. Bear in mind that the area of the site for constructing a shelter must be at least 10 square meters per person sheltered with consideration of the accommodation there of soil removed from the pit, stockpiling of materials and prefabricated structures, work areas for installation machinery and so on. A convenient access must be provided to the building sites.

1.12. Spaces adaptable as shelters must be designed as buried facilities. The bottom of the overhead cover as a rule should be located no higher than the level of the grade. With a high water table it is permissible to locate the overhead cover above the level of the grade with observance of requirements of paragraphs 2.20, 2.21 and 3.8 of SN 405-70.

The floor level of shelters located above the water table should be at least 0.5 meters above the water table. With ground water occurring at a high level, it is permitted to have the shelter floor level below the water table.

The minimum buried depth of shelters (floor level) should be at least 1.5 meters from the surface of the ground in all instances, including freestanding shelters being built where there is a high water table.

For freestanding buried facilities adaptable as shelters, there should be soil filled above the overhead cover with slopes no steeper than 1:2, with the shoulder of the slopes at least 1 meter wide around the perimeter of the structure's outer wall, and with a layer of soil above the overhead cover of no more than 1 meter and no less than 0.5 meters.

For Paragraph 1.12. Shelters buried in the ground provide the most reliable protection against all injurious effects.

When the shelter's overhead cover is located above the earth's surface, there is an increase in the load from the effect of the shock wave on walls protruding above the ground and a reduction in their protective features against penetrating radiation and the thermal effect. Steps must be taken to increase their protective features by providing a cushioning layer of soil or arranging a thermal insulation layer.

When industrial grounds are terraced, it is recommended that shelters be located at sites where the ground level drops.

1.13. In designing facilities adaptable as shelters under conditions of waterlogged soil, provisions should be made for adding a sealer or other waterproofing as well as a catchment basin (sump) within the structure with a sump pump. The emergency exit is usually located above the water table.

Note. With terrain relief permitting ground water to be drained by gravity, it is permitted to plan drainage when an appropriate feasibility study has been accomplished.

For Paragraph 1.13. For technical and economic considerations, it may prove more expedient in some instances to plan shelters with partial burial instead of buried shelters with costly and complicated waterproofing, especially in the presence of ground water under great pressure (see Fig. 2b).

1.14. It is prohibited to lay "through" utility lines--heating, water supply, sewer, compressed air, gas and steam lines, power cables, communications lines and others--through shelters.

It is permitted to lay water supply and sewer lines connected with the building's overall system in built-in facilities adaptable as shelters if cut-off devices are installed which eliminate the possiblity of the shelter's protective features being violated. Drainage standpipes must be placed in steel pipes securely fixed in the overhead cover and floor of the shelter.

For Paragraph 1.14. Drainage standpipes also may be placed in reinforced concrete ducts to prevent their destruction under the effects of the shock wave of a nuclear burst, since otherwise the shelter seal may be broken.

Steel check valves, flanged valves and gate valves can be used as cut-off devices installed at the inlets of plumbing within the shelter.

Shelters can be located near water supply lines up to 150 mm in diameter and near the installation's sewage and power supply systems, but no nearer than 25 meters from large water and sewage mains, destruction of which might lead to shelter flooding. A closer location is permitted if these mains are fitted with cut-off devices.

1.15. A fill of a layer of soil of at least 0.5 meters and the possibility of laying power supply cables and water lines where necessary (see Fig. 10) should be provided above the overhead cover of built-in facilities adapatable as shelters, with observance of the requirements of Paragraph 1.14.

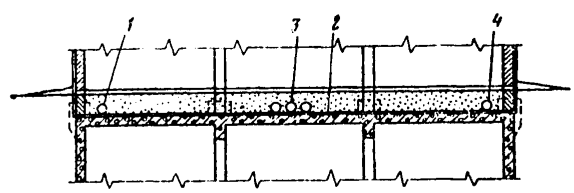


Fig. 10. Overhead cover of built-in shelter

1. Gas line
2. Overhead cover (floor of first story, earth fill of 50-100 cm, waterproofing, reinforced concrete slabs)
3. Ducts for laying communications, signalling and power supply cables
4. Water line

It is permitted to ignore the filling of a layer of soil over the overhead cover of built-in shelters in residences and public buildings in providing for requisite protection against the blast wave and penetrating radiation of a nuclear burst as well as against high temperatures in fires.

For Paragraph 1.15. When it is impossible to lay the lines indicated in para 1.15 above the overhead cover, it is recommended that they be led beyond the limits of the shelter in special ducts or tunnels laid along the outside of the shelter walls. It is also best to lead standpipes not connected with the shelter's water supply and sewage system outside the limits of the shelter.

1.16. Spaces adaptable as shelters should be located, with respect to tanks and industrial units with dangerously explosive products at distances determined based on the amount of products being stored and the degree of shelter protection, but no less than the fire safety separation standardized in the SNiP and other normative documents approved under established procedures.

For Paragraph 1.16. When shelters are located at enterprises which have stored petroleum products, the following requirements must be observed:

Locations of the shelters must be chosen, as a rule, beyond the zone of possible overflow of the petroleum products and their flooding of structures. In individual instances it is permitted to locate shelters in sectors where the overflow of petroleum products is possible. Such shelters and entrances therein must be banked with the bank at least 0.7 meters above the ground level;

Shelters should be located on the windward side of stored petroleum and gas products.

Shelters must be located at a safe distance from containers of liquefied hydrocarbon gases (acetylene, methane, ethane, propane, butane, ethylene, propylene, butylene). This safe distance may be determined by the methodology given in Appendix 4 depending on the amount of the product and degree of shelter protection.

In determining shelter locations in the master plan, consideration also has to be given to their mutual positioning with respect to warehouses and industrial buildings representing a fire danger (lumber yards, lumber dryers and so on). Distances between the shelters and these yards (enterprises) have to be determined according to SNiP Chapter II-M. 1-71 "Master Plans of Industrial Enterprises: Design Standards."

1.17. Spaces adaptable as shelters must be protected against possible inundation by storm or ground waters or by other liquids with the destruction of tanks situated on the surface of the ground or higher stories of buildings and structures.

For Paragraph 1.17. The flooding of shelters by storm waters and other liquids may occur primarily through entrances, exits and air ducts. For this reason, to protect facilities adaptable as shelters against flooding, it is recommended:

Locate them on higher sectors of the terrain;

Install ducts and drainage leading liquid away from shelter entrances;

Provide for an elevated area in front of the entrance preventing liquid from leaking into the shelter;

Accommodate exhausts and air intakes at an elevation safe against liquid leaking in.

2. SPACE-PLANNING AND DESIGN DECISIONS

2.1. Spaces adaptable as shelters are subdivided into primary and auxiliary spaces. The former includes filter-ventilation chambers (FVK), toilets and protected diesel-electric power plants (DES). Protected entrances and exits are envisaged in addition.

For Paragraph 2.1. Space-planning decisions for shelters should ensure:

Simple and precise planning with a minimum diversity of spans and elevations and with the least perimeter of outer walls;

Most economic use of internal volume and areas;

Normal conditions for use of facilities for national economic needs and as shelters;

Convenience of filling it and accommodating sheltered persons;

Creation of conditions needed for a lengthy stay of sheltered persons;

Rational accommodation of internal engineering-technical equipment and convenience of its installation and operation;

Possibility of independent exit of sheltered persons from structures after the effects of estimated weapons.

The design layout of spaces adaptable as shelters must be drawn up so as to provide for the most effective performance of supporting and protective structures under the effects of a load created by the blast wave of a nuclear burst.

The most rational design configuration of a shelter must be chosen on the basis of a technical and economic comparison of decision variants. Design practice indicates that it is best to use a bay size of 6 x 6 meters and 4.5 x 6 meters for most rational use of a structure's area for its national economic purpose and as shelter. A smaller bay size hinders use of the premises in peacetime and forces an increase in the area used as shelter, which leads to overall higher cost of the structure. Use of a 3 x 6 meter base size must be justified by a feasibility study.

In designing shelters there should be an attempt at a maximum possible use, for accommodation of sheltered persons, of the protected area both of primary and other spaces having a subsidiary purpose in peacetime use of the structure (closets, staff rooms, storerooms, ontainer rooms and so on). Only spaces where the presence of people is not recommended for safety reasons can be an exception.

Primary Spaces

2.2. The facility floor area standard for one person should be taken as 0.5 square meters with double-tier bunks and 0.4 square meters with a triple-tier arrangement of bunks for sheltered persons, and the internal volume of the facility should be no less than 1.5 cubic meters per person. The triple-tier arrangement of bunks for sheltered persons should be provided in shelters accommodated in buildings built in areas with outside air parameters as indicated in paragraphs 1, 2 and 3 of Table 15 of these Instructions.

Note. In determining volume per person, consider the volume of all spaces both for primary and auxiliary purposes, with the exception of DES.

For Paragraph 2.2. The area standard of 0.4 and 0.5 square meters and a volume of 1.5 cubic meters per person is the minimum. With an estimated outside air temperature of more than 25°C, an increase in the floor area standard of primary facilities of up to 0.75 square meters per person is authorized to combat excesses of heat. Any increase in the area standard above 0.4 and 0.5 square meters, however, can be authorized only with a feasibility study.

2.3. The height of a facility should be made to conform with requirements of its peacetime use, with at least 2.2 meters from the floor level to the bottom of protruding components of the overhead cover.

For Paragraph 2.3. The floor level at the toilets can be raised above the floor level of the shelter's primary spaces when installing emergency containers for collecting fecal matter in toilets, providing a height of at least 1.7 meters to the ceiling.

2.4. The seating for persons sheltered in a facility should be 0.45 x 0.45 m per person, and the place for lying down in the upper tiers should be 0.55 x 1.8 meters. The height of benches for seating should be 0.45 meters, the height of places for lying down in the second tier should be 1.45 meters, and in the third tier it should be 2.15 meters from the floor. The distance from the top of the tier to the overhead cover or protruding components should be at least 0.75 meters.

The number of places for lying down should be equal to 20 percent and 30 percent of the shelter capacity with a double-tier and triple-tier arrangement respectively.

The width of passages at the level of the benches for seating (first tier) should be:

0.7 meters between transverse rows;

0.75 meters between a lengthwise row and the ends of transverse rows;

0.85 meters between lengthwise rows;

0.9-1 meter for through passages in the shelter (the larger dimension is for passages between lengthwise rows).

For Paragraph 2.4. Fig. 11 provides possible variants for accommodating places for sheltered persons in shelters with various bay sizes, as well as dimensions for lengthwise and transverse passages ensuring normal conditions for filling the structures and the movement of people during a prolonged stay.

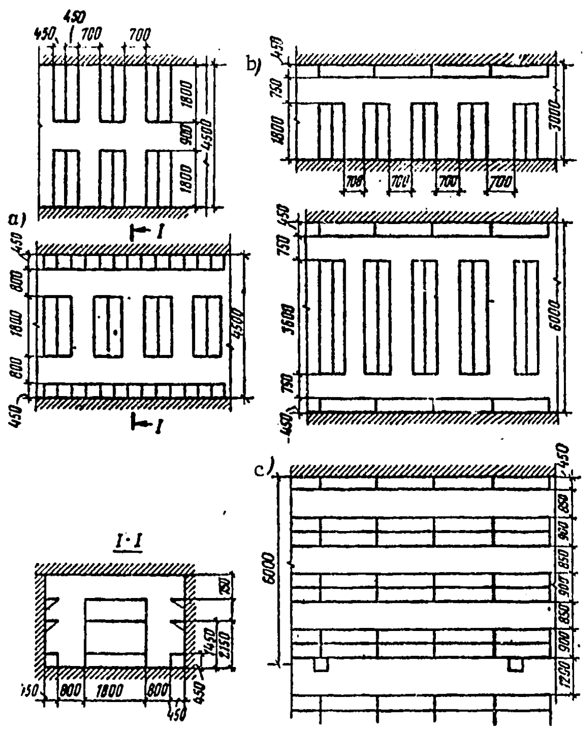


Fig. 11. Variants for accommodating places for sheltered persons with the distance between lengthwise walls (column rows): a. 4.5 meters; b. 3 meters; c. 6 meters

In planning spaces adaptable as shelters special attention must be given to the placement of equipment in primary spaces and its use for accommodating sheltered persons. When the purpose of spaces is combined for national economic needs and for sheltering the working shift, the plan must provide for adapting certain kinds of equipment for accommodating sheltered persons, and the distance between individual types of equipment should be based on the specification of the placement of benches and bunks between the equipment.

Places for sheltered persons to sit and lie down may be permanently installed when the shelter is constructed. If their installation hinders use of the facilities for national economic needs, they should be installed when the facility is converted for use as a shelter based on prepared planning documentation.

In some instances it is best to design benches and bunks to be collapsible.

When production activities in the facilities cease during a special period and the plan envisages the dismantling of certain kinds of equipment from the primary spaces, it is recommended that furniture and equipment (store counters, tables and so on) be designed as sectional so it can be used to accommodate sheltered persons.

2.5. The control post should be accommodated in one of the shelters having, as a rule, a protected source of electrical power.

The control post spaces--a work room and communications room--should be located near one of the entrances and separated from the space for sheltered persons by partitions.

The total number of persons working in the control post should be up to 10, with an area standard per worker of 2 square meters.

Notes: 1. The control post is not set up at enterprises with the number of workers in the largest shift being up to 600 persons. In this instance a telephone and radio broadcast point should be set up in one of the shelters for communications with the local Civil Defense headquarters.

2. The total number of persons working at control posts of certain enterprises can be increased to 25 persons by authorization of USSR ministries and departments.

For Paragraph 2.5. The control post (PU) is intended for accommodating the installation CD staff.

The control post is outfitted with communications facilities providing:

Control of installation CD warning equipment;

Telephone communications of the management and operational personnel with installation CD subunits and with heads of the higher CD staff and with public and production establishments of a city, rayon or oblast;

Telephone communications with enterprise shelters, with shops which do not cease production at the alarm signal, and with the enterprise dispersion area;

Radio communications with the local CD staff and with the dispersion area.

If the control post is designed for five persons or fewer, it is possible to accommodate it in one room of up to 10 square meters in area.

Two work rooms and a communications room are assigned when up to 25 persons are working in the PU. The entrance to the communications room must be through the work room.

It is best to use offices, service rooms, staff rooms and other spaces as control posts.

Fig. 12 shows an example of the layout and location of a PU.

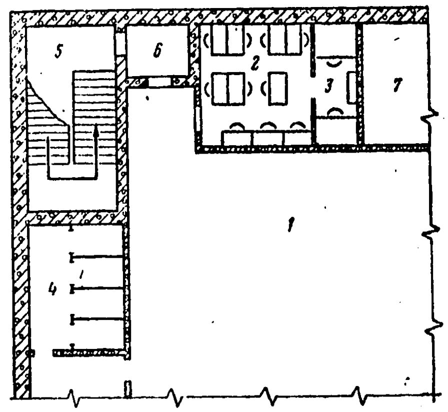


Fig. 12. Example of layout and location of control post:

1. Space for sheltered persons
2. Control post work room
3. Communications room
4. Toilet
5. Entrance
6. Airlock
7. FVK

2.6. An airlock-sluice should be provided at one of the exits of shelters holding 300 persons or more. A single-chamber sluice is provided for shelters holding from 300 to 600 persons, and a double-chamber sluice for shelters of greater capacity.

With an entrance 0.8 meters wide, the area of each airlock-sluice should be 8 square meters, or 10 square meters with an entrance width of 1.2 meters.

There must be provisions for installing swinging or sliding airtight-blast doors in the outer and inner walls of the airlock-sluice according to the shelter protection class. Swinging doors must open outward in the direction of the people's evacuation.

For Paragraph 2.6. The airlock-sluice is for preventing the danger of injury to persons in the shelter when people who did not arrive at the designated time enter it.

The airlock-sluice gives a cyclic passage of sheltered persons. The design decision of entrances with sluices and a diagram of the passage of sheltered persons through the airlock-sluice are shown in Fig. 13.

The outer and inner doors of sluice chambers can be sliding or swinging with an interlock precluding an instance of their simultaneous opening.

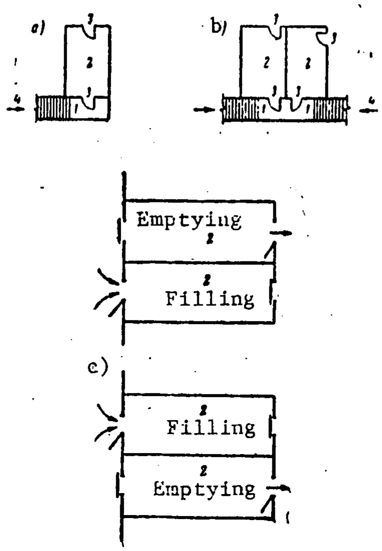


Fig. 13. Design decision of entrances with sluices

a. Single chamber sluice
b. Double chamber sluice
c. Diagram of entrance operation with sluice in operation
1. Fore airlock
2. Sluice chamber
3. Airtight blast door
4. Down stairway

2.7. Finishing of the spaces should be envisaged in conformity with SNiP requirements depending on the peacetime purpose of the spaces, but no higher than improved finishing.

Plastering of the ceilings is not authorized. The joints in ceilings of precast reinforced concrete slabs and gaps between them must be thoroughly covered with mortar or concrete.

Note. It is prohibited to use combustible materials in finishing the interior of spaces, or make bunks and other equipment (lockers) out of combustible synthetic materials.

For Paragraph 2.7. Finishing of the face surface of precast elements of protective components under plant conditions is recommended in erecting shelters made of precast or precast-monolithic components. Only float work or pointing of joints between elements with cement mortar is required in assembling structures out of such elements.

Float work of face surfaces of protective and bearing components is permitted in monolithic reinforced concrete structures.

The walls and ceilings in filter-ventilation chambers are coated with polyvinyl-acetate paint.

It is recommended that the interior finish of walls and partitions in spaces with high humidity be of covering plates (vinyl plastic) or finishing films.

Metal doors and shutters should be coated with synthetic paint (glyptal, alkyd-styrene and so on).

In all cases, the interior surface of shelter spaces (walls, ceilings, partitions) should be painted or faced primarily in light tones.

Auxiliary Spaces

2.8. As a rule, the filter-ventilation chamber (FVK) should adjoin the outer wall of the shelter and be near an entrance or emergency exit.

The size of the FVK space is determined by the dimensions of equipment and the area needed for its surviving, in conformity with Table 14 of SN 405-70.

Note. The filter-ventilation equipment can be accommodated directly in the spaces for sheltered persons in shelters holding 150 persons or less.

Table 1 (14) - Distances (Clearance) between Elements of Engineering-Technical Equipment and Components

Standardized Values	Dimensions, m
Distance between machines and guards or control panels	2
Distance between two manual-electric ventilators (between handle axes)	1.7
Service passages between foundations or housings of machines and between housings and components	1
Service passages between cabinets and walls and between power switchboards	0.8
Service passages between elements of sanitary engineering equipment	0.7
Distance between machine and walls or between housings of machines installed in parallel when there is a passage on the other side	0.3
Distance between units of ventilation equipment and wall with passage on other side of units	0.2

Table 2 - Approximate Area Norms of Auxiliary Spaces

Description of Shelter's Internal Engineering Equipment	Area (m^2) per Person with Shelter Capacity					
	150	300	450	600	900	1200 or more
Without self-contained (protected) power and water supply systems and without air regeneration	0.15	0.14	0.14	0.14	0.14	0.14
With a DES, but without self-contained source of water	--	--	--	0.13	0.12	0.10
With self-contained power and water supply systems and with air conditioning:						
Source of cold--well water (tanks, wells)	--	--	--	0.14	0.13	0.11
Source of cold--freon units	--	--	--	0.18	0.17	0.15

For Paragraph 2.8. In designing auxiliary spaces there must be no excess and at the same time always bear in mind that under special conditions auxiliary spaces will be very loaded down and slight deviations from requirements toward a reduction may cause serious hindrances in their operation.

For most rational use of spaces outfitted as CD shelters, it is necessary for the cumulative area of all auxiliary spaces to be minimum.

The area of auxiliary spaces can be determined according to the data given in Table 2 depending on the nature of the internal engineering equipment and shelter capacity.

As a rule, the FVK should be separated from other shelter spaces by partitions with ordinary doors. The FVK equipment can be separated by a metal grid partition in small-capacity shelters (up to 150 persons).

Air intake, exhaust air, and exhaust gas ducts are component parts of the filter-ventilation equipment.

Separate air intake ducts--for pure ventilation and for filter-ventilation--are provided for ventilation.

It is advisable to supply clean air through the emergency exit. In this case the accommodation of the emergency exit and air intake aperture in the FVK wall should be designed with a displacement of 1.5-2 meters for the axes.

Air intake for filter-ventilation should be accomplished from the fore airlock. It is permissible to accommodate it on grounds that may be obstructed and it can be made of metal pipes with a diameter based on calculations. The pipes' egress at the surface of the ground must be protected from mechanical damage in peacetime. The distance between air intakes must be at least 10 meters.

Exhaust ports and ducts are provided for air removal. The distance between the exhaust and air intake should be at least 15 meters.

Exhaust ducts are installed in shelters with DES and they must be a distance of at least 20 meters from shelter air intakes and can be on grounds that may be obstructed.

To protect elements of the air supply system from the shock wave, air intake and exhaust ducts are fitted with UZS [exact expansion unknown] or MZS [exact expansion unknown] antiblast devices and with expansion chambers or duct areas of equivalent volume.

Expansion chambers are made behind the antiblast devices in front of the filter-ventilation equipment. Minimum volumes of expansion chambers and dimensions of ducts are given in Table 17 of SN 405-70.

2.9. Toilets should be designed separately for men and women. The number of floor bowls (or toilet bowls) and urinals is determined based on the number of persons using this lavatory, figuring 75 women per floor bowl (or toilet bowl) and 150 men per floor bowl (or toilet bowl) and per urinal (or 0.6 meters of a urinal trough). Wash basins in lavatories are planned based on one basin per 200 persons, but at least one per lavatory. The width of the passage between two rows of toilet stalls or between a row of stalls and urinals located opposite them should be 1.5 meters, and the distance between the end of a row of toilet stalls and the wall or partition should be 1.1 meters.

For Paragraph 2.9. Toilet spaces should adjoin the outer walls of shelters and be located as closely as possible to spaces for sheltered persons and at the greatest distance possible from self-contained water supply sources and buried containers with a supply of potable water.

Entrances to toilets should be arranged through vestibules (washrooms) with self-closing doors.

Floor bowls and toilet bowls should be located in separate stalls with doors.

In designing toilets the stall dimensions in axes are taken to be 1.2 x 0.9 meters if doors open outward and 1.5 x 0.9 meters if doors open inward.

In Paragraph 2.9 the width of passages in toilets is taken for stall doors opening outward. If doors open inward the passage width can be reduced and taken as 1.2 meters between stall rows or between a row of stalls and opposite urinals, and 0.8 meters between the end row of toilet stalls and a wall or partition.

The distance between axes of a group of wash basins should be taken as 0.6 meters.

A schematic diagram of a toilet design is given in Fig. 14.

If toilets are required for a small number of workers in peacetime (when the spaces of warehouses, stores, repair shops and so on are to be used as shelters), it is advisable for designs to envisage their use as storage rooms, warehouses and other auxiliary spaces.

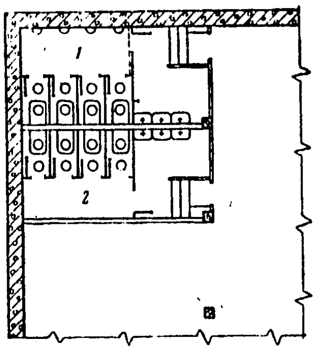


Fig. 14. Example of toilet design in 900-person shelter:

1. Men
2. Women

It is permitted to disconnect the installed equipment (toilet bowls, drainage tanks) from the sewer system and mothball it. In such cases there can be provisions for installing stalls and partitions separating the toilet from the vestibule when the spaces are converted to a shelter regime, using prefabricated sectional elements.

In a number of instances it will be expedient to reject the installation on the first floor of toilets needed for requirements of peacetime operation of spaces adaptable as shelters and use shelter toilets instead. These toilets should be located near the staircase (Fig. 15).

2.10. Spaces for a protected power supply source (DES) should be accommodated near the external wall of a building, separated from other spaces by a fireproof wall with a refractory limit of one hour.

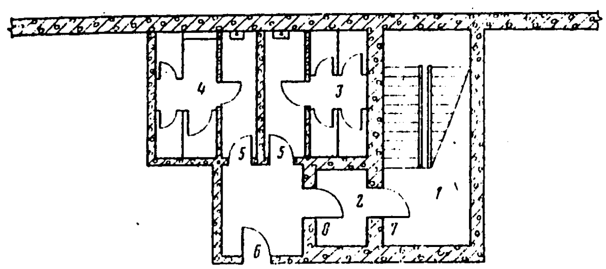


Fig. 15. Schematic decision of toilets in basement off of main floor

1. Staircase
2. Vestibule
3. Women's toilet
4. Men's toilet
5. Entrance to toilets
6. Entrance to space for sheltered persons
7. Airtight blast door
8. Airtight door

The entrance to the DES from the shelter should have an airlock with two airtight doors opening toward the shelter. The exit door in the DES must be fire resistant with a refractory limit of 0.75 hours.

For Paragraph 2.10. A diesel-electric power plant should be planned in the shelter only if, because of ventilation conditions, there will be a requirement for more than ten blowers with manual-electric drive (ERV [manual-electric ventilator]) or air cooling units and conditioners. In all other instances shelters should receive electrical power prior to the employment of estimated means of destruction from unprotected external sources of power, with the manual-electric ventilator (ERV) used for ventilation after means of destruction have been employed.

As a rule, protected sources of electrical power should be planned for a group of shelters located within a radius of up to 500 meters from each other, with the possibility of using the DES as a reserve source of electrical power in peacetime.

The entrance to the DES can be designed from the airlock of the shelter entrance (Fig. 16) or from the space for sheltered persons. An airlock with two airtight doors opening toward the space for sheltered persons is arranged at the entrance to the DES from the space for sheltered persons.

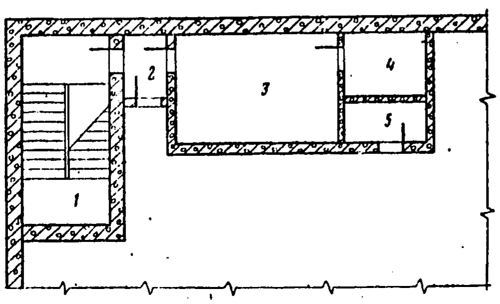


Fig. 16. Example of decision for entrance to DES from shelter airlock

1. Staircase
2. Airlock
3. DES
4. Fuel storage
5. Panel room

Dimensions of passages between equipment elements in the power plant space and between the equipment and structural components (clearance) should be taken in accordance with data given in Table 1 (14).

In shelters with a DES the design should provide for a storage room for a fuel and oil reserve. As a rule, the storage room is located next to the DES and must have fire resistant, airtight doors opening into the DES space and containers for fuel and oil storage. The storage room is separated from the remaining sheltered spaces by blank, airtight and fireproof walls with a refractory limit of at least one hour. With a volume of up to 1.5 cubic meters of fuels and lubricants, they can be accommocated in the DES machine room.

An emergency water supply is provided for shelters in case the external water line is disabled.

Special containers (tanks) with an emergency store of water, open pools, artesian wells and dug wells can be used as an emergency water supply source.

In shelters which do not have industrial units, a water reserve is provided only for drinking needs and extinguishing fires (in shelters with a capacity of more than 600 persons). It is not recommended that the floor area of protected spaces be used to accommodate containers for a water reserve.

It is best to make these containers in the form of cylindrical tanks and install them on brackets under the ceiling of sheltered spaces. The elements fastening the tanks to the overhead cover must be designed for the inertial forces arising from the effect of a shock wave from a nuclear burst (see Appendix 5).

Open pools are set up in the immediate vicinity of a shelter and used for supplying water to industrial units and for putting out fires.

It is authorized to set up artesian wells in the presence of an appropriate feasibility study. It is advisable to design wells for a group of shelters, using them as a peacetime source of water for the enterprise.

Protected Entrances and Exits

2.11. The width of openings and passages into spaces adaptable as shelters must satisfy requirements of SNiP and other regulatory documents applicable to spaces based on their peacetime purpose.

In multistory buildings the entrances to spaces adaptable as shelters with a DES usually should be designed to be isolated from stairwells. It is authorized to use an entrance to such shelters from a common stairwell by installing separate exits for these spaces to the outside, separated from the remaining portion of the stairwell by blank fireproof components with a refractory limit of at least one hour.

It is also permissible to install entrances with a fireproof door into the shelter from the first floor of production and other buildings through an independent stairwell when personal service rooms and other spaces are used as shelters.

Storage spaces must have a separate entrance.

For Paragraph 2.11. Entrances must satisfy the following basic requirements:

Have the necessary throughout capacity;

Provide protection for sheltered persons against injury by the shock wave, penetrating radiation, thermal radiation, toxic chemical agents, bacterial agents and combustion products from conflagration, through entrances.

Entrances consist of a down stairway or fixed ramp, fore airlock, airlock or airlock-sluice, and entrance openings with doors. Entrance elements are shown in Fig. 17.

Depending on conditions for accommodation of built-in and freestanding shelters and their peacetime use, shelter entrances may be of the following types:

Blind;

Through with a covered sector.

Fig. 18 shows space-planning decisions and basic elements of these types of entrances. Bear in mind in choosing the type of entrance that in a blind entrance the loads on walls and blast doors will be approximately twice those in a through passage. For this reason blind entrances should be installed only where no other entrance decision is possible based on conditions of the structure's peacetime use or use under other conditions.

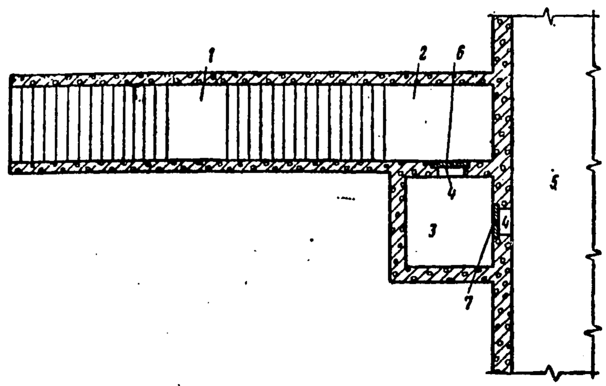


Fig. 17. Entrance Elements

1. Down stairway or fixed ramp
2. Fore airlock
3. Airlock or airlock-sluice
4. Entrance openings with doors
5. Shelter
6. Airtight blast door
7. Airtight door in airlock or airtight blast door in airlock-sluice

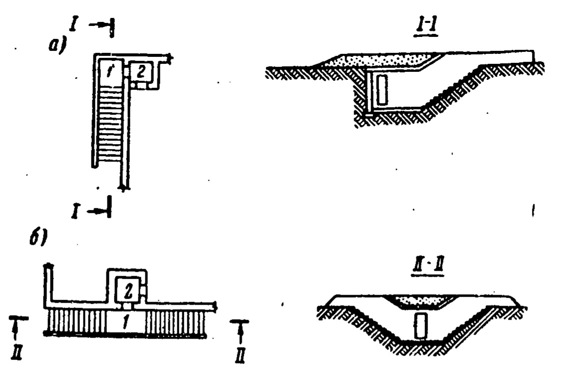


Fig. 18. Space planning decisions for entrances without sluices: a. Blind passages; b. Through passages with cover over fore airlock

1. Fore airlock
2. Airlock

Entrances in built-in shelters can be designed:

From staircases of multistory buildings;

Along independent stairwells from the building's first floor;

From a basement unprotected from the blast wave.

The load on walls and doors of these entrances will be considerably less than in blind or through entrances, while protective components of the aboveground part of the building and stairwell reduce the effect of radiation on entrances.

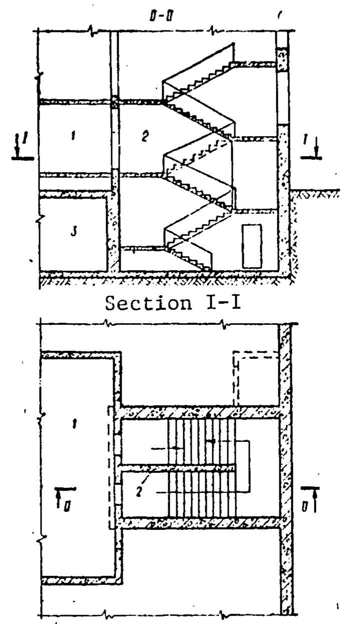


Fig. 19. Entrance to shelter with DES from stairwell of multistory building

1. Vestibule
2. Wall separating flights of stairs leading to second and following floors and to basement
3. Shelter

Isolation of a shelter with DES from the general stairwell in multistory buildings can be achieved by separating the flights of stairs leading to the second floor and basement with a fireproof partition (Fig. 19 and 20).

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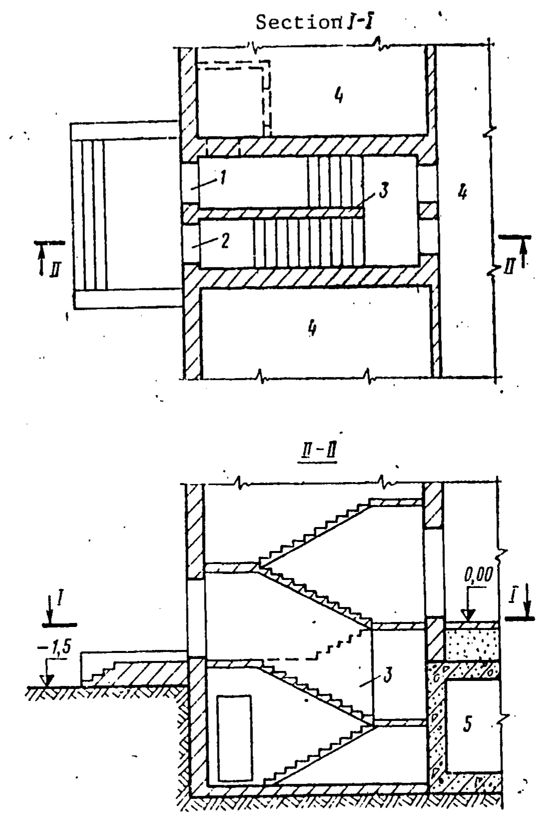

Fig. 20. Entrance to shelter with DES from stairwell of multistory building

1. Entrance to first and higher floors of building
2. Entrance to basement (shelter)
3. Wall separating flights of stairs leading to first and following floors and to basement
4. First floor spaces
5. Shelter

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Basement Layout

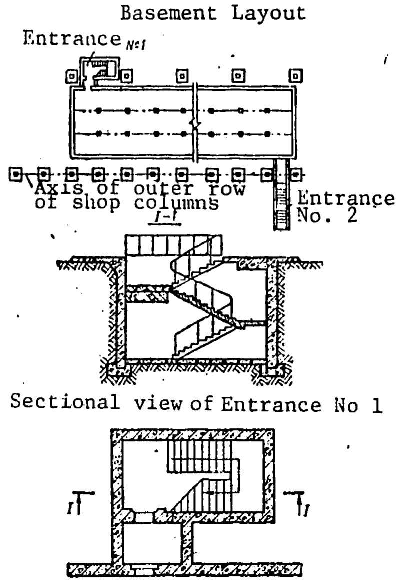


Fig. 21. Entrance to shelter from first floor of production space

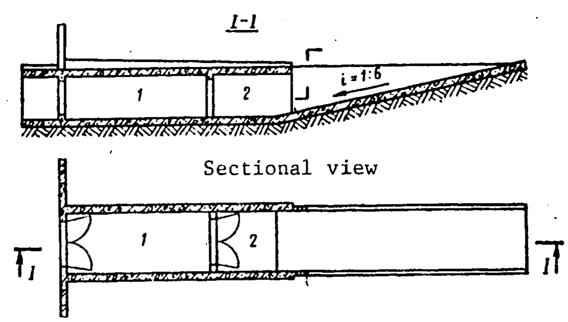


Fig. 22. Schematic of entrance to warehouse space

1. Airlock
2. Fore airlock

Schematics of entrances from the first floor of a production space and to a warehouse space are given in Fig. 21 and 22.

2.12. The number of entrances should be made to depend on the shelter capacity and number of persons arriving at one entrance, but it should be at least two entrances. It is authorized to install one entrance for a shelter holding up to 300 persons, with the emergency exit envisaged as an evacuation exit with a door height of 1.8 meters and gallery cross section of 1.2 x 2 meters.

The number of exits from production buildings for filling shelters located beyond the limits of these buildings is determined in a manner similar to determination of entrances to shelters. The overall width of exits from the building should be no less than the cumulative width of entrances to shelters. It is permissible to consider overhead-swivelled gates for transportation equipped for automatic or manual openings as exits from buildings in addition to ordinary exits.

For Paragraph 2.12. Depending on the width of the door opening, assembly radius, or distance of the shelter from the exit of the building in which the main mass of persons to be sheltered is located, the number of sheltered persons per entrance should be in accordance with data given in SN 405-70.

To reduce the total number of entrances for shelters with large capacity, it is recommended that the throughput of each entrance be increased by installing wide down stairways and door openings or by combining several door openings in a single entrance.

In a number of cases the nature of the people's location within an assembly area may differ substantially from the averaged conditions for which the data of SN 405-70 are taken. In such cases it is advisable to perform supplementary calculations to determine the necessary number of entrances.

The required number of entrances to a shelter depends on the entrance throughput per unit of time and the rate at which people arrive at the shelter. Fig. 23a is a chart showing the rate of arrival of persons at a shelter I_t and their passage into structure Q_t. It is apparent from the chart that the throughput capacity of entrances is not used fully when the shelter begins to be filled at a moment in time from t_α to t_β, and that after t_β the entrances function at full throughput capacity. The throughput capacity of an entrance depends on the density of the flow of people. Fig. 23b shows the relationship of a change in throughput capacity of 1 meter of entrance width Q_{BX} horizontal sectors and stairways with downward movement to the density of the flow of people.

The throughput capacity of a specific entrance may be obtained by multiplying Q_{BX}, determined from the chart, by the width of the door opening.

An excessive consolidation of the flow in the entrance leads to formation of "bottlenecks." Studies of the shelter filling process have shown that bottlenecks do not form if the entrance throughput capacity is more than or equal to 80 percent of the maximum rate of people's arrival $Q_{\text{вх}} \geqslant 0,8 I_{(t)\max}$. (3)

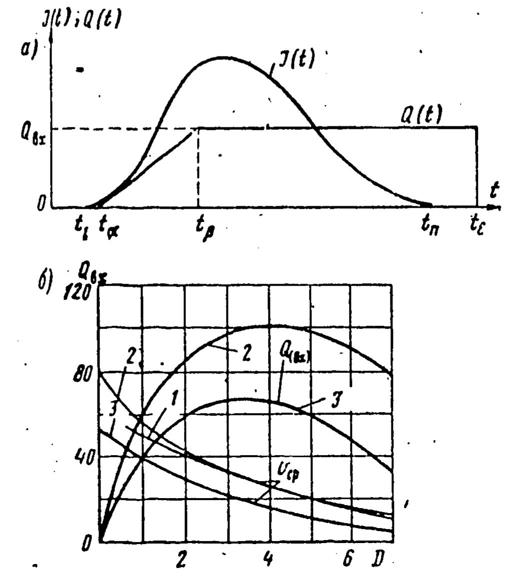


Fig. 23. Charts for determining entrance throughput capacity

a. Rate of arrival and entrance of persons into the shelter

b. Relationship of entrance throughput capacity to flow density

I_t Rate of people's approach to shelter

Q_t Rate of people's passage into shelter

Q_{BX} Throughput capaciaty of 1 meter of entrance width, persons per minute

D Density of flow of people, persons per square meter

T_1 - T_n Moments of arrival at shelter of first and last person respectively

t_α and t_E Moments of entrance into shelter of first and last person respectively

t_β Intermediate moment of entrance into shelter

v_{cp} Average speed of people's movement, km/hr:

1. In a horizontal sector
2. In a door opening
3. On a stairway with downward movement

On this condition the number of entrances n_{BX} can be determined from the formula

$$n_{\text{вх}} = \frac{0{,}8 I_{(t)\max}}{Q_{\text{вх}}}. \qquad (4)$$

The maximum radius of people's arrival at the shelter depends on the assembly radius, the people's movement speed and their density in the assembly area or within the limits of a certain part of this area.

With an even distribution of persons to be sheltered throughout the assembly area, the maximum rate of people's arrival at a shelter, located conditionally in the center of the area, can be calculated from the formula

$$I_{(t)\max} = \frac{2\pi R \rho v_{\text{cp}}}{k}, \qquad (5)$$

With an uneven distribution of persons to be sheltered the assembly area is divided into a number of rings 20-25 meters wide and the rate of people's arrival from each ring is calculated from the formula

$$I_{(ti)} = \frac{N_i v_{\text{cp}}}{\Delta r k}, \qquad (6)$$

where N_i is the number of people within the i-th ring;
v_{cp} is the average movement speed of people within the assembly area;
Δr is the width of a ring;
R is the assembly area radius;
ρ is the density of persons in the assembly area,
k is the coefficient of nonstraight-line movement of people from building exits to the shelter.

The maximum from among the values obtained for individual rings is taken as the computed value $I_{(ti)}$.

Depending on flow density, the speed of people's movement can be determined from the chart in Fig. 23b. Let us examine the calculation procedures in examples.

1. There are 800 persons to be sheltered, distributed evenly in an installation with a density of ρ = 10 persons per hectare. The mean assembly radius is 500 meters. The average movement speed along horizontal sectors is 90 meters per minute.

$$I_{(t)\max} = \frac{2\cdot 3{,}14\cdot 500\cdot 10\cdot 10^{-4}\cdot 90}{1{,}3} = 200 \text{ persons/minute.}$$

The number of shelter entrances with a width of 0.8 meters and Q_{BX} = 80 persons per minute will equal:

$$n_{\text{вх}} = \frac{0{,}8 I_{(t)\max}}{Q_{\text{вх}}} = \frac{200}{0{,}8\cdot 80} = 3.$$

2. The total number of sheltered persons is 800. Their distribution in the assembly area is uneven. The shelter is located on open grounds and persons to be sheltered are in different buildings. There are no persons to be sheltered at a distance of 300 meters from the shelter, there are 200 at a distance from 300 to 400 meters, and there are 600 persons at a distance of 400-500 meters from the shelter. Let us break down the assembly area into 20 rings 25 meters across (Δr = 25 meters). The maximum rate of arrival is $I_{(ti)} = \frac{150\cdot 90}{25\cdot 1{,}3} = 400$ persons/minute.

The required number of entrances 0.8 meters wide with Q_{BX} = 80 persons per minute with free passage of people (without bottlenecks) will be $n_{\text{вх}} = \frac{0{,}8 \cdot 400}{0{,}8 \cdot 80} = 5$.

In this case we should settle on five entrances despite the fact that this variant is less economical than that provided in SN 405-70 (four entrances). Measures can be taken to reduce the rate of arrival by bringing shelters nearer to the main body of persons to be sheltered or by redistributing people to shelters for the purpose of reducing the number of entrances.

2.13. Entrances must be located on opposite sides of shelters with consideration of the direction of movement of the main flows of people with provision for entrances from enterprise grounds in the form of a through-type down stairway or from unprotected basement spaces and stairwells.

All entrances to shelters except for one entrance with an airlock-sluice must be equipped with airlocks.

Doors in the airlocks should be as follows: airtight blast door corresponding to the shelter protection class in the outer wall; airtight door in the inner wall; and doors must open in the direction of people's evacuation.

It is recommended that sliding airtight blast doors be provided on the external side of the airlock and sliding airtight doors within the airlock in the "open" position when entrances are used in peacetime in entrances of spaces adaptable as shelters and visited regularly in peacetime by a large number of people (300 or more).

In addition to airtight blast doors and airtight doors, the entrance openings being used in peacetime must be covered with ordinary doors in accordance with requirements of SNiP chapters on designing buildings and structures and requirements of fire safety standards.

For Paragraph 2.13. In addition to doors, the airlock should have a removable wooden panel flush with the threshold and fixed ramp panels on the outer and inner sides of the airlock. The wooden panels and standard hinged doors must be removed when spaces are converted to a shelter regime.

2.14. The width of the stairways down to the entrance should be 1.5 times greater than the width of the door opening, and that of fixed ramps should be 1.1 times greater than the width of the door opening. The slope of stairway flights should be no more than 1:1.5, and that of fixed ramps should be no more than 1:6. A fore airlock in the form of a recess or reinforced head cover (platform) above the entrance is built at the entrance door into the airlock (airlock-sluice).

The width and length of the airlock and fore airlock must be 0.6 meters greater than the width of the door panel, and the width of the airlock-sluice must be at least 2.2 meters.

Entrances must be protected from atmospheric precipitation. Pavilions (caps) for protecting entrances (exits) against atmospheric precipitation must be made of noncombustible materials.

For Paragraph 2.14. The throughput capacity of stairway descents and fixed ramps is less than that of the door openings, and so a varying width of descents and door openings is taken in order to equalize the rate of arrival of persons to be sheltered.

The number of steps in a flight (ascent) must be at least three and no more than 18 on routes for evacuation and for filling.

The height of a riser in stairway descents must be no more than 16 cm and the width of a tread must be at least 25 cm.

For safety in descending, the surface of fixed ramps must be finished to prevent slipping.

The vertical distance from the surface of a tread and from horizontal surfaces of entrance landings to the lower part of the overhead cover must be at least 1.90 meters.

Handrails must be installed on the sides of stairways and fixed ramps.

It is recommended that intermediate handrails be installed in wide stairway descents so that the distance between these rails and the handrail on the wall is 1-1.5 meters.

To protect entrances from atmospheric precipitation, it is permitted to install light enclosures of noncombustible materials,and which can be demolished by the blast wave,over their openings.

The dimensions of airlocks and fore airlocks in a plan depend on requirements for operation of the spaces in peacetime and on the width of doors and must provide for the free passage of people through door openings with doors being opened and closed at different times.

It is recommended that the width and length of the airlock and fore airlock be made 0.6 meters greater than the width of the door panel. The distance from the axis of the door opening to the wall to which a door opens must be 0.4 meters greater than half the width of the door panel.

The mutual accommodation of doors in entrances is determined by their convenience of operation in peacetime and by the capability of transporting equipment through the entrances. Positioning of doors in a layout at a right angle to each other is the most expedient based on degree of protection against radiation.

Doors may be swinging and sliding. A description of standard airtight blast doors and airtight doors and shutters used in different classes of shelters is given in Appendix 1.

Industrial door openings in shelters should be used, as a rule, for filling the shelters with persons. The openings must be covered with airtight blast doors, gates or shutters having a closing time of no more than 1.5 minutes.

2.15. Spaces adaptable as shelters must have an emergency exit beyond the limits of zones of possible debris from the collapse of buildings and structures.

One of the exits in shelters holding 600 persons or more (up to 3,000 persons) must be fitted as an emergency and evacuation exit in the form of a tunnel with inside dimensions of 1.2 x 2 meters. The exit from the shelter into the tunnel must be fitted with airtight blast and airtight doors 0.8 x 1.8 meters in size, installed in the airlock.

In freestanding shelters one of the entrances, located outside the zone of possible debris, can be designed as an emergency exit.

For Paragraph 2.15. An airlock must be designed when an emergency exit is used as an entrance to a shelter and in those instances where persons from outside the limits of the assembly radius will enter through the emergency exit, an airlock-sluice should be designed. A schematic of an entrance combined with an emergency exit is given in Fig. 24.

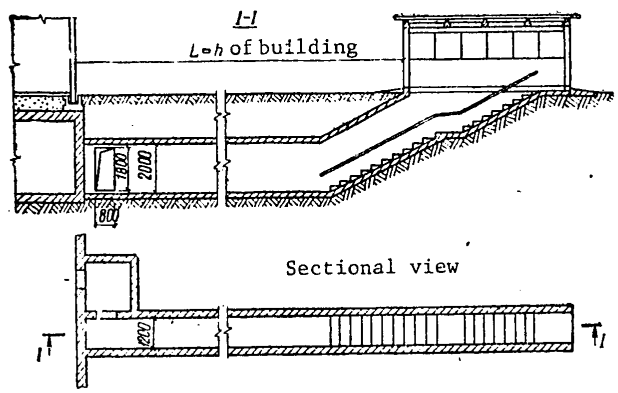


Fig. 24. Emergency exit combined with entrance

L -- Distance from building to entrance pavilion

2.16. An emergency exit in the form of a vertical shaft with a protected cap is authorized in shelters holding up to 600 persons.

The emergency exit must connect with the shelter by a tunnel. Inside dimensions of the tunnel and shaft must be 0.9 x 1.3 meters.

The shelter exit into the tunnel must be closed with airtight-blast and airtight shutters installed on the outer and inner side of the wall respectively. Installation of an airlock is permitted for this exit with appropriate substantiation.

For Paragraph 2.16. A schematic of an emergency exit from a built-in shelter is given in figures 25 and 26.

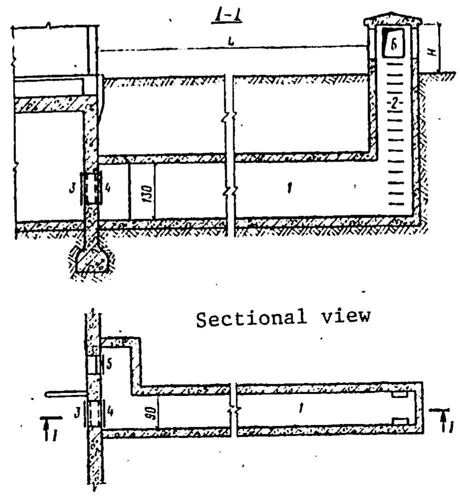


Fig. 25. Emergency exit from built-in shelter

1. Gallery
2. Shaft with protected cap
3. Airtight shutter
4. Airtight blast shutter
5. UZS
6. 60 x 80 cm aperture with louvered grid

H -- Cap height
L -- Distance from cap to building (H = 1.2 meters with L equal to half the building height plus 3 meters and H = 0.5 meters with L equal to the building height)

In freestanding shelters emergency exits in the form of a shaft with cap can be designed adjoining the outer walls of a shelter. A schematic of such an entrance is given in Fig. 27.

2.17. Emergency exit shafts must be fitted with protected caps, with their height above the surface of the ground to be:

1.2 meters with the distance between the cap and building equal to half the building height plus 3 meters;

0.5 meters with the distance between the cap and building equal to the building height.

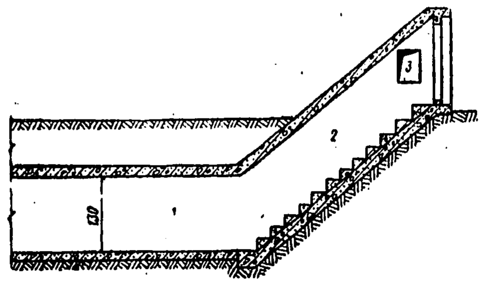


Fig. 26. Schematic decision for emergency exit with down stairway

1. Gallery
2. Down stairway
3. Aperture with louvered grid

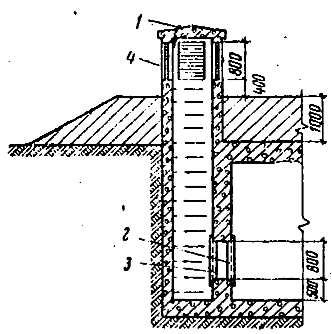


Fig. 27. Schematic decision for emergency shaft exit adjoining freestanding shelter:

1. Cap
2. Airtight shutter
3. Airtight blast shutter
4. Louvered grid

Openings 0.6 x 0.8 meters in size fitted with louvered grids opening inward must be provided in each wall of a cap 1.2 meters high. A hatch 0.6 x 0.6 meters in size must be provided in the overhead cover with a cap height less than 1.2 meters.

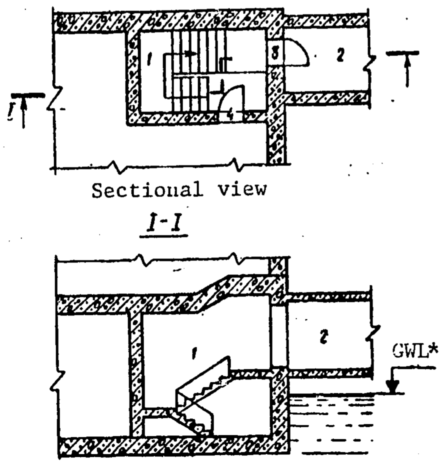
Note: 1. With the emergency exit at a distance equal to the building height, it is permissible to install a down stairway at ground level in place of a protected cap.

2. With the emergency exit at a distance less than half the building height (H), the cap height should be interpolated between the values of 1.2 meters and debris height at the building $h_3 = 0.1H + 0.7$ meters.

For Paragraph 2.17. Cap height should be taken to mean the distance from ground level to the underside of the cap cover.

With shelters located in waterlogged soil, all elements of emergency exits should be located above the level of ground water (Fig. 28).

With a very high ground water level (0.5 meters from the earth's surface) and the impossibility of building cushioned galleries, emergency exits can be made in the form of protected shafts rigidly connected with the overhead cover (Fig. 29). In this case the distance from the earth's surface to the bottom of the aperture with airtight blast shutter must be 0.2-0.3 meters greater than the debris height at the building.



*Ground Water Level

Fig. 28. Schematic layout of entrance combined with emergency exit in waterlogged soil

1. Airlock with stairway descent
2. Gallery
3. Airtight blast door
4. Airtight door

Design Decisions

2.18. The structures of spaces adaptable as shelters must provide protection for persons against effects of the blast wave of a nuclear burst, radioactive emissions, thermal radiation and heat effects from conflagrations.

Spaces adaptable as shelters must be airtight.

For Paragraph 2.18. The design elements of a shelter are as follows:

Bearing and protective components of the main structure: Overhead covers, external walls, internal walls and columns, continuous base plate or individual pillar (strip) foundations.

Design elements of entrances: Walls of airlocks, airlock-sluices, fore airlocks, stairway descents and fixed ramps, covers for them, entrance openings with protective devices (doors, shutters, gates), protected or unprotected caps over shelter entrances;

Design elements of emergency exits: Walls, overhead covers and foundations of galleries and protected caps, openings with protective devices (doors, shutters, standardized protective sections).

2.19. Bearing components must be calculated for the effect of the blast wave of a nuclear burst and possess requisite strength in conformity with the shelter class.

It is recommended that girderless overhead cover be used for shelters, or overhead covers with a girder arrangement and resting on columns according to the column-girder (collar beam)-slab system. In some instances with appropriate substantiation it is permissible to use inner lengthwise and cross bearing walls.

2.20. The weight of protective components for protection against radioactive radiation should conform to SN 405-70.

Note: 1. The weight of overhead cover includes fixed equipment (no more than 200 kg-force per 1 square meter of area occupied), conditionally taken as evenly distributed over the overhead cover area, as well as the layer of soil on the overhead cover.

2. To the weight of walls separating spaces adaptable as shelters from adjoining basements should be added the weight of the portion of overhead cover of these basements equal in width to the height of the wall.

For paragraphs 2.19 and 2.20. The weight of industrial equipment (machine tools, conveyors, racks and so on) and the weight of light partitions installed on the overhead cover which are not dismantled or removed in converting the spaces to a shelter operating regime must be considered in determining fixed equipment weight.

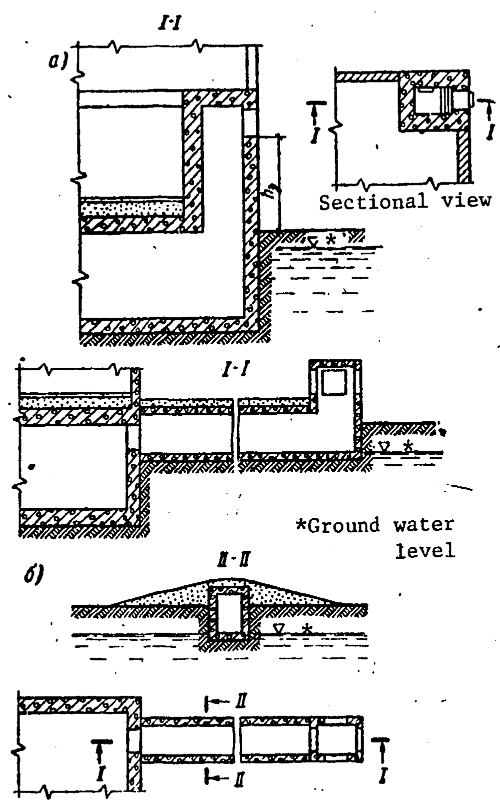


Fig. 29. Schematic layout of emergency exit with ground water level 0.5-1 meter below overhead cover

a. Exit shaft without access gallery
b. Exit shaft with semiburied access gallery

h_3 -- Distance from ground level to opening (h_3 = 0.1H + 0.9 meters, where H is the building height)

2.21. Reinforced concrete walls and overhead cover not covered with soil and with a thickness under 0.6 meters as determined by the structural design must have a thermal insulation layer according to Table 3(2).

Table 3(2) - Thickness of Thermal Insulation Layer, cm

Thermal Insulation Layer	Size of Thermal Insulation Layer with Thickness of Reinforced Concrete Walls and Overhead Cover, cm			
	40	30	20	10
Boiler or blast-furnace slag	10	15	20	30
Slag concrete	12	20	25	35
Heavy concrete	20	30	40	50
Vegetable soil	25	35	45	55

For Paragraph 2.21. The thickness of walls and overhead cover for hollow elements is taken as equal to the total thickness of the component and for ribbed elements it is taken as equal to the thickness of the flange.

The data given in Paragraph 2.21 are valid for calculating the overhead cover of shelters located in a zone subject to obstruction and under thermal effect of conflagration in the rubble.

Protective components of built-in shelters which will not be obstructed as well as components of freestanding shelters located in a zone of conflagration in rubble, but in sectors which will not be obstructed, do not have to be designed for heating in view of the slight thermal effect.

Screens which can be installed on the inner side of the shelter can be used to protect persons from radiant heat coming from the heated surfaces of protective components. In this instance a design temperature of 40°C is permissible on the inner surface of protective components. Asbestos cement and fiberboard slabs, thermal insulation mats and so on may be used as material for the screens. The greatest effect is achieved with double screens installed 10-15 mm from the inner surface of the protection and 10-15 mm from each other.

An approximate computation of the requisite thickness of the overhead cover component for heating with an allowable temperature of 30°C on the inner surface can be performed from the formulas:

In the absence of screens $h_{из} = (2,4 - 4h)\, a_{из} \cdot 10^2$; (7)

In the presence of screens $h_{из}^{эк} = (1,85 - 4h)\, a_{из} \cdot 10^2$. (8)

where h is the thickness of the reinforced concrete bearing component in meters; and $a_{из}$ is the coefficient of temperature conductivity of the thermal insulating material in square meters per hour.

2.22. The structural schematic of built-in spaces adaptable as shelters should be chosen with consideration of the design of the building (structure) in which the shelter is being built and on the basis of the technical-economic estimate of space-planning decisions for adapting spaces for needs of the national economy in peacetime.

The structural decision for connecting frame elements of the above-ground portion of buildings with components of built-in shelters must provide, as a rule, for the building's above-ground components resting freely on the overhead cover of the built-in shelter. To ensure spatial rigidity of the frame of an above-ground building under the effects of operating loads, it is permissible to install "rigid joints" calculated for destruction of above-ground components under a special combination of loads and for assurance of the strength and airtightness of the shelter's overhead cover.

For Paragraph 2.22. The structural schematic of the building's underground portion must correspond to the optimum extent to requirements for assuring strength and stability under the effects of operational loads and a special combination of loads, as well as to economic expediency.

As a rule, the center lines of external and internal bearing walls and individual tiers (columns) of the building's above-ground frame and its basement portion should coincide. The distance between longitudinal and cross center lines of freestanding shelters should be taken as a multiple of 15M (M is the basic modulus taken as equal to 100 mm).

It is permissible to introduce additional tiers in basement spaces reducing the effective span of components of shelter overhead cover, within the accepted distance between bearing components of the building's above-ground portion.

The following structural schematics can be used in erecting shelters:

Frame-panel with full frame (Fig. 30a);

Frame-panel with partial frame (Fig. 30b);

Frameless (Fig. 30c).

The frame-panel schematic with full frame is a system consisting of posts (columns) and collar beams filled in with slabs (panels) solidly connected with frame elements.

In the frame-panel schematic with partial frame, continuous walls are installed in place of the end columns and the fillings.

The longitudinal and cross positioning of collar beams is permitted in frame-panel structures with full frames. The longitudinal positioning of collar beams is recommended in structures with partial frame, inasmuch as this provides an opportunity to reduce the number of complex joints in connections between collar beams and walls and improve the work of longitudinal walls against the joint effects of vertical and horizontal loads.

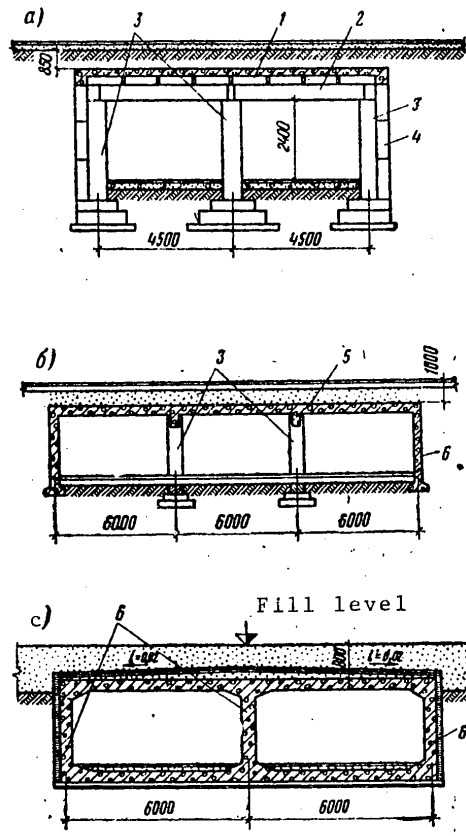


Fig. 30. Structural schematics of shelters

a. Frame-panel with full frame
b. Frame-panel with partial frame
c. Frameless
1. Precast reinforced concrete collar beam
2. Precast monolithic overhead cover
3. Reinforced concrete column
4. Wall slabs
5. Monolithic reinforced concrete slab of overhead cover
6. Monolithic reinforced concrete walls

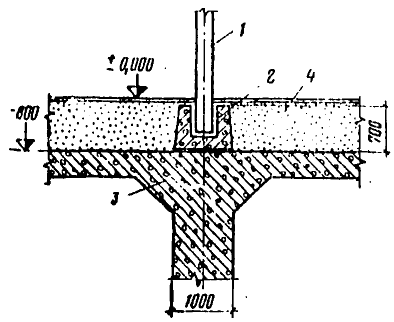


Fig. 31. Schematic of above-ground frame resting on shelter's overhead cover through a sleeve foundation

1. Precast reinforced concrete column of above-ground frame
2. Precast reinforced concrete sleeve foundation
3. Monolithic girderless overhead cover
4. Fill

In designing built-in shelters it is not recommended that frame components (columns) of the above-ground portion of the building be rigidly connected with shelter components. Foundations under the columns rest freely on the shelter's overhead cover. An example of a structural decision for connection of these elements is given in Fig. 31.

The most rational space-planning decisions for spaces adaptable as shelters must be chosen on the basis of a comparative technical-economic analysis of different variants. Appendix 2 provides a methodology for determining cost indicators for various structural-design decisions for shelters which allows selection of the most optimum variants depending on a change of particular parameters affecting the cost of individual structural elements and the structure as a whole. These parameters include the span and height of spaces, type and strength of material used, features of the structural (design) schematic, cost of materials and articles and so on. A determination of cost expenditures for individual shelter elements is performed according to corresponding nomograms in Appendix 2.

The results obtained also can serve as a standard for determining the degree of rationality of the structural-design decisions being worked out.

2.23. Standard reinforced concrete shelter components must be used in drawing up plans for spaces adaptable as shelters. With a ground water level more than 2 meters higher than the level of the shelter floor, the walls and foundation plate of these spaces should be designed as monolithic reinforced concrete, envisaging industrial methods for their construction.

It is permissible to use standard reinforced concrete components of industrial and civilian housing construction for class IV-V shelters, strengthening their elements in necessary cases.

For Paragraph 2.23. The use of unified components of basement spaces under series U-01-02 and U-01-01 and precast reinforced concrete elements with increased supporting power is recommended above all in constructing shelters. For example, this includes elements of pedestrian and production tunnels, collectors, slabs of overhead cover of industrial buildings for heavy loads and so on.

The reinforcement can be increased by using steel with increased strength characteristics and by increasing the sectional area of the effective longitudinal and lateral reinforcement.

2.24. Requirements of SNiP Chapter I-A.3-62 "Application of Single Modular System in Designating Sizes of Precast Components and Articles" should be followed in designing special precast reinforced concrete components.

For Paragraph 2.24. It is preferable to designate the nominal width of base plates of overhead cover, wall slabs and foundation plates from the sizes 1,000, 1,500 and 3,000 mm.

2.25. Overhead covers should be designed from precast and precast-monolithic reinforced concrete components with a refractory limit of one hour. Monolithic reinforced components can be used with consideration of the need for their construction in short time periods.

For Paragraph 2.25. As a rule, elements of overhead cover made of precast reinforced concrete components should be designed as sectional, covering joints with mortar (concrete) and installing a beam of monolithic concrete around the circumference of the structure, connecting the beam to the outer walls with anchors (Fig. 32). It is advisable to design the precast-monolithic components as continuous, with installation of above-pier reinforcement in the layer of monolithic concrete (Fig. 33). A portion of the effective reinforcement (longitudinal and lateral) can be installed between precast elements (Fig. 34).

In designing shelters of monolithic reinforced concrete, it is recommended that the most rational structural decisions be made in which optimum use is made of the strength characteristics of concrete (protective components of curvilinear shape, girderless types of overhead cover and so on). Proressive types of formwork as well as the formless method of work should be used in building shelters.

2.26. Overhead covers should be securely connected with walls made of precast reinforced concrete elements by welding inserts or reinforcement protrusions, and with walls made of masonry (concrete) materials by installing anchors of at least 2 square centimeters cross-section per one meter of wall. Anchor connections should be to a depth of at least 30 diameters of the reinforcement.

For Paragraph 2.26. At the bearing points of precast elements of the overhead cover on internal walls, in addition to the anchors embedded in the wall based on a figure of 2 square centimeters per one meter of wall, reinforcing rods are installed additionally in joints between elements for providing a connection. The total area of reinforcement placed on one side of the overhead cover with consideration of anchors protruding from the wall must be at least 2 square centimeters per one meter of overhead cover.

Placement of anchors at the junctions of precast, precast-monolithic and monolithic overhead cover with walls of masonry materials is necessary for ensuring the interconnection of the structure's elements. This is considered a hinged joint and the installed reinforcement is not considered in the design estimate.

In installing walls and overhead cover of monolithic reinforced concrete, it is recommended that junction points be designed as rigid (frame) joints, with installation of requisite reinforcement in them based on calculation.

Structural decisions for junction points of overhead cover with walls are shown in figures 35-37.

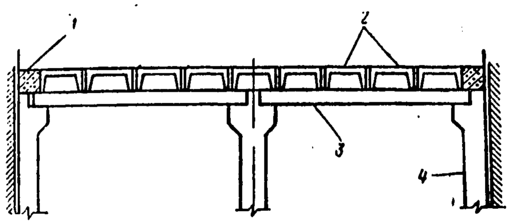


Fig. 32. Arrangement of monolithic collar beam in precast overhead cover

1. Monolithic collar beam
2. Slab
3. Precast collar beam
4. Column

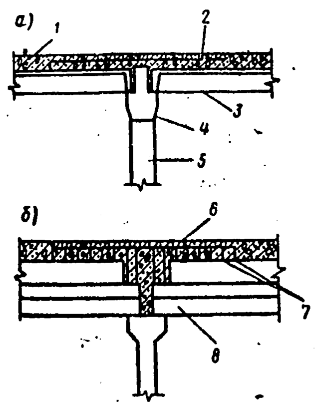


Fig. 33. Installation of above-pier reinforcement in shelter's precast-monolithic overhead cover:

a. In continuous slabs
b. In continuous collar beams
1. Layer of monolithic concrete
2. Above-pier mesh reinforcement in slabs
3. Slab
4. Collar beam
5. Column
6. Above-pier reinforcement of collar beam
7. Protrusions of la‑eral reinforcement from collar beam
8. Collar beam (slabs conditionally not shown)

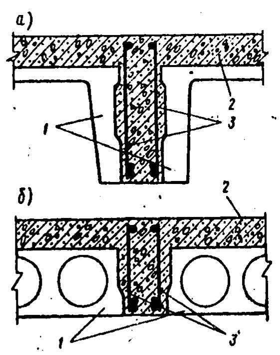


Fig. 34. Schematic of positioning of reinforcing cages between precast elements of precast-monolithic overhead cover of shelter

a. Using channel slabs
b. Using multicavity slabs
1. Precast elements
2. Monolithic concrete
3. Supplementary reinforcing cages

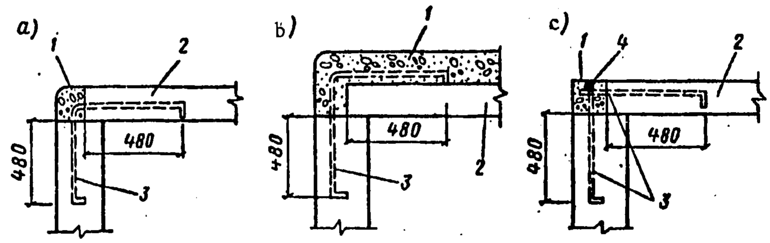


Fig. 35. Anchoring of overhead cover elements with outer walls

a. With a precast reinforced concrete overhead cover
b. With a precast-monolithic reinforced concrete overhead cover
c. With wall construction out of long components.
1. Layer of monolithic concrete
2. Overhead cover element
3. Anchor ϕ 16 mm embedded every 1 meter in layer of monolithic concrete
4. Rod ϕ 20 mm, welded to anchors

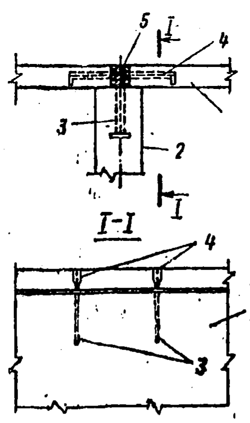


Fig. 36. Installation of anchors between overhead cover and inner walls

1. Precast elements of overhead cover
2. Inner wall of masonry materials
3. Anchors protruding from masonry into joints between overhead cover elements
4. Reinforcing rods laid in joints between precast elements
5. Monolithic concrete

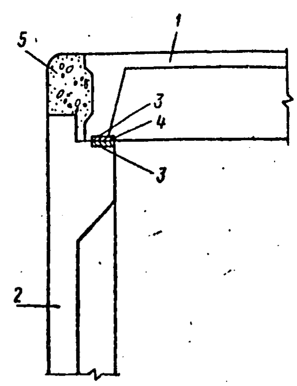


Fig. 37. Welded connection of precast reinforced concrete elements of walls and overhead cover

1. Reinforced concrete element of overhead cover
2. Reinforced concrete element of outer wall
3. Embedded parts in precast elements
4. Weld figuring at least 5 cm per 1 m of wall (height of joint weld is equal to 0.6d, with d being the thickness of an 8 mm embedded part)
5. Monolithic concrete

2.27. Walls should be designed of precast reinforced concrete slabs, concrete blocks, monolithic reinforced concrete and other construction materials satisfying strength requirements as well as other requirements placed on underground portions of buildings and structures.

The joints of walls (corners, abutting and intersecting walls) made of masonry materials and concrete blocks should be strengthened with Class A-I reinforcement in the form of individual rods or mesh with an overall cross section of at least 4 square centimeters per 1 meter of wall with a 50 centimeter projection of the reinforcement from the side of the wall.

It is permissible to design the walls of shelters located in waterlogged soil where the ground water level is up to 2 meters higher than the floor level out of precast components, providing for the filling of vertical joints between wall slabs with impervious concrete (mortar) using nonshrinking or expanding cement or portland cement with sealing additives, and embedding the slabs in a channel of the foundation plate.

For Paragraph 2.27. Reinforced concrete slabs for the outer walls of shelters can be nonbearing or bearing. Nonbearing slabs receive only a lateral (horizontal) load (Fig. 38a and b), while bearing slabs in addition receive an added load from elements of the shelter's overhead cover (Fig. 38c).

The design of horizontal and vertical joints at places where slabs come together must provide a simple and reliable filling of the joints with mortar (concrete). Slabs are fastened to columns by welding of inserted parts (Fig. 39).

As a rule, outer and inner walls of concrete blocks are placed with a bonding of vertical joints. In some cases, when necessary to increase the bearing power of outer walls against the effect of a horizontal load, continuous vertical channels are made in them and filled with concrete and reinforcing cages (Fig. 40).

At places where outer shelter walls come together with components of entrances and emergency exits, floating joints are made in the absence of ground water. When the ground water level is higher than the shelter floor level, components of outer walls and entrances are rigidly interconnected, i.e., they are designed to be continuous. In this instance it is recommended that the grade of the emergency exit floor be located above the ground water level.

Structural decisions for reinforcing joints of masonry shelter walls are given in figures 41-44.

2.28. Columns and foundations should be designed of precast or monolithic concrete. Use of a solid foundation plate is permitted under difficult hydrogeological conditions. Joints of walls and columns with overhead cover and foundations must provide a spatial rigidity to components under installation and design loads.

For Paragraph 2.28. To reduce the cross-section of columns with considerable loads it is recommended that diagonal reinforcement of columns with horizontal mesh be used. If the ground water level is below shelter floor components, the wall footings should be designed as continuous, with pier footings under columns. The grade

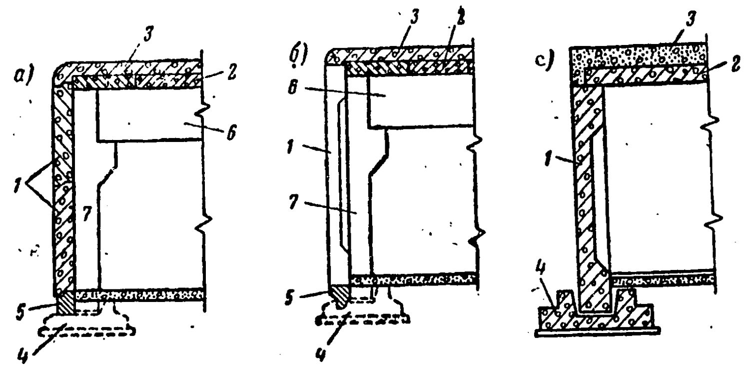


Fig. 38. Variants of decision for outer walls

a. Nonbearing slabs positioned horizontally
b. Nonbearing slabs positioned vertically
c. Bearing slabs
1. Wall slabs
2. Precast portion of overhead cover
3. Monolithic portion of overhead cover
4. Foundation
5. Marginal beam
6. Collar beam
7. Column

of the top of continuous footings for walls should be set at the level of the bottom of the subfloor (see Fig. 40).

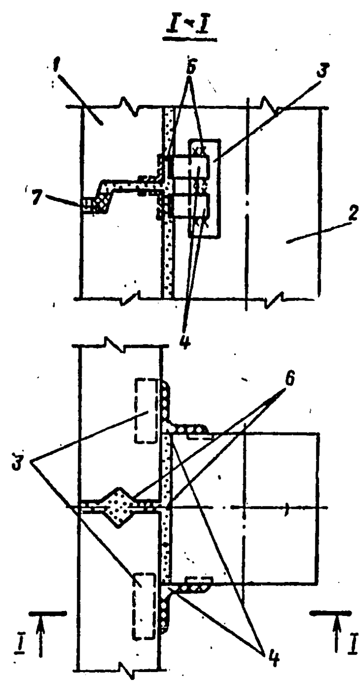
With the ground water level near or higher than the shelter floor level, a solid foundation plate is laid under the structure. Variants of structural decisions for the foundation plate are given in Fig. 45.

The joining of columns with pier foundations and the solid foundation plate should be rigid.

It is recommended that the joints of precast reinforced concrete columns and collar beams be hinged, with welding, using special inserts (Fig. 46).

2.29. Partitions should be designed of precast reinforced concrete and other fireproof materials and fastened to walls and columns. When longer than 3 meters they also should be fastened to overhead cover by anchors with a cross section of 1 square centimeter per 1 meter of the perimeter, with provisions for freedom of deformation of the overhead cover.

For Paragraph 2.29. It is best to use sufficiently strong materials having a small volume weight for building partitions. The use of reinforced brick partitions is permitted. Partition thickness should be determined in accordance with requirements placed on its strength and with consideration of the horizontal acceleration received, sound insulating power and airtightness (in necessary instances). The



Sectional view

Fig. 39. Attachment point of outer wall slab to column

1. Reinforced concrete slabs
2. Reinforced concrete column
3. Inserts of slabs and columns
4. Connecting elements
5. Weld
6. Filling with cement mortar
7. Packing seal

computation for equivalent static loads from the effect of inertial forces is given in Appendix 5. Schematic decisions for junction points of inner walls with surrounding components are shown in Fig. 47.

2.30. Entrances, exits and also industrial openings should be closed with standard airtight blast and airtight doors and shutters, basic characteristics of which are given in Appendix 3.

For Paragraph 2.30. It is recommended that industrial installation openings of considerable size be protected with gates moving on special rails parallel to the surface of the wall in which the opening is located. The openings also can be covered with specially made precast elements ensuring requisite airtightness around

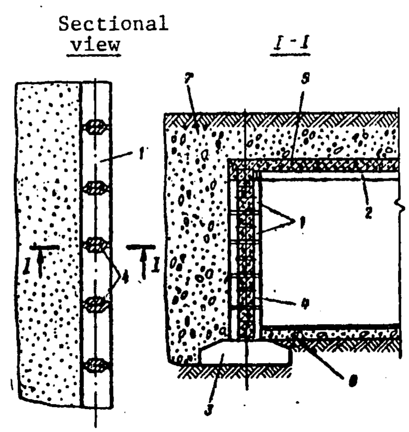


Fig. 40. Structural decision for outer concrete-block walls

1. Concrete blocks
2. Precast overhead cover slabs
3. Continuous footing
4. Reinforcing cages
5. Monolithic concrete
6. Concrete subfloor
7. Earth fill

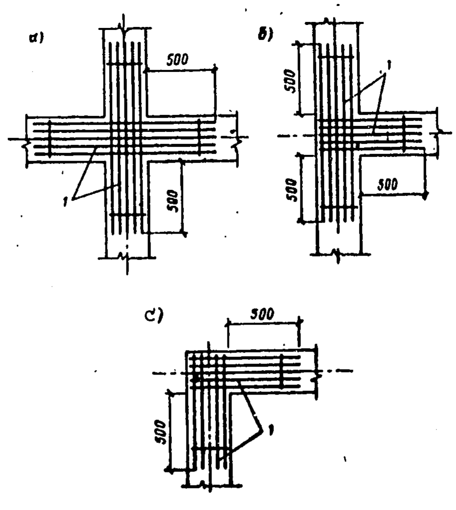


Fig. 41. Structural reinforcement of masonry walls with 6-8 mm reinforcing mesh

a. Intersection
b. Adjoining
c. Corners

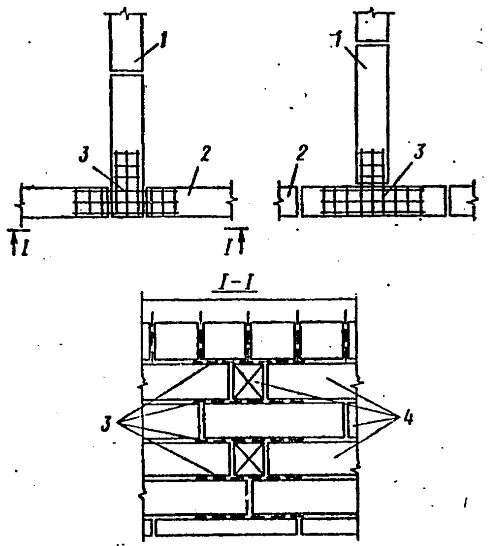


Fig. 42. Reinforcement of joints of precast concrete block walls

1. Inner wall of precast concrete blocks
2. Outer wall of precast concrete blocks
3. Reinforcing mesh
4. Concrete blocks of outer wall

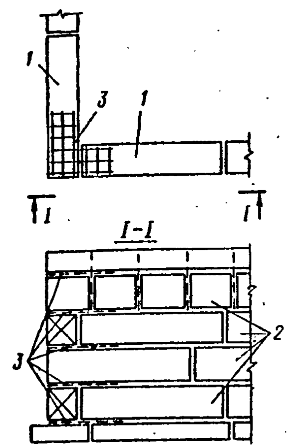


Fig. 43. Reinforcement of corner joints of precast concrete block walls

1. Outer wall of precast concrete blocks
2. Concrete blocks of walls
3. Reinforcing mesh

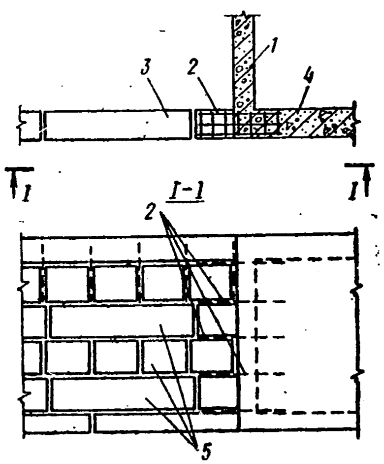


Fig. 44. Joints of walls of precast elements and monolithic reinforced concrete

1. Inner monolithic wall
2. Reinforcing mesh
3. Outer wall of precast concrete blocks
4. Outer monolithic wall
5. Concrete blocks of walls

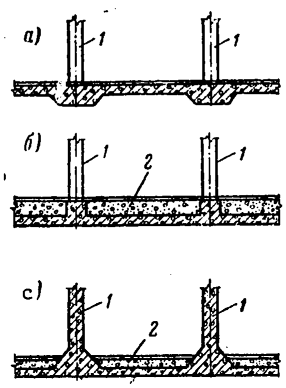


Fig. 45. Structural decisions for solid foundation plate

a. With beams below in the direction of the lesser column spacing
b. With beams above in two mutually perpendicular directions
c. With capitals under the columns
1. Column
2. Fill

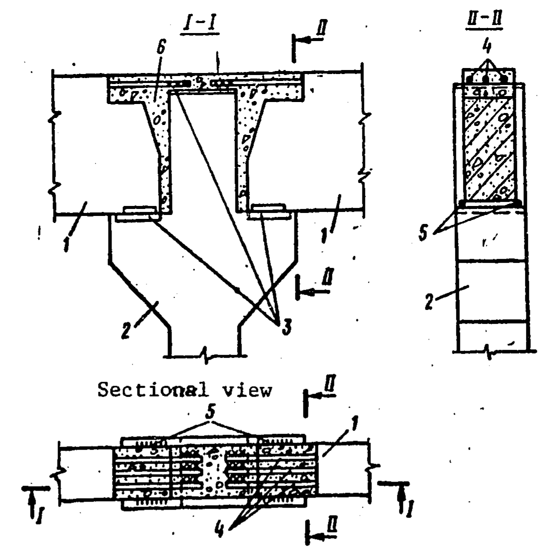


Fig. 46. Hinged attachment point of collar beam on column

1. Reinforced concrete collar beam
2. Reinforced concrete column
3. Inserts in column and collar beam
4. Reinforcement protrusions from collar beam
5. Weld
6. Monolithic concrete

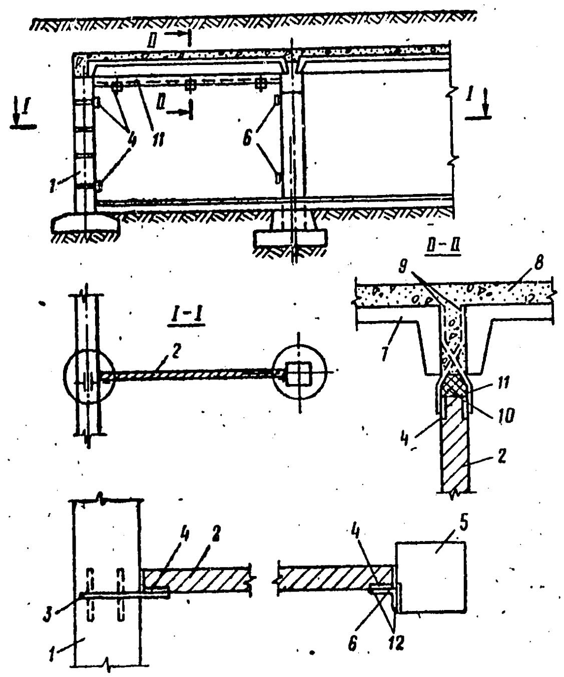


Fig. 47. Structural decision for attachment of precast reinforced concrete inner wall

1. Wall of precast concrete blocks
2. Reinforced concrete inner wall
3. Round bar anchors inserted in outer wall joints
4. Inner wall inserts
5. Column
6. Angle plates
7. Precast elements of overhead cover
8. Monolithic concrete
9. Anchors based on 1 cm^2 of anchor cross-section per 1 linear meter of wall length
10. Clearance between wall and overhead cover slab equal to 1/50 of wall height and filled with pliable material (felt, rags and so on)
11. Component of wood or dry plaster filling clearance between overhead cover and inner wall
12. Weld joints

the perimeter of the opening and between elements and the corresponding component attachment (Fig. 48). Protective components are installed on the outer and inner sides of a wall or partition. It is permissible to install protective components on one side if a ground or sand fill at least 0.5 meters thick is placed above them.

Sectional view

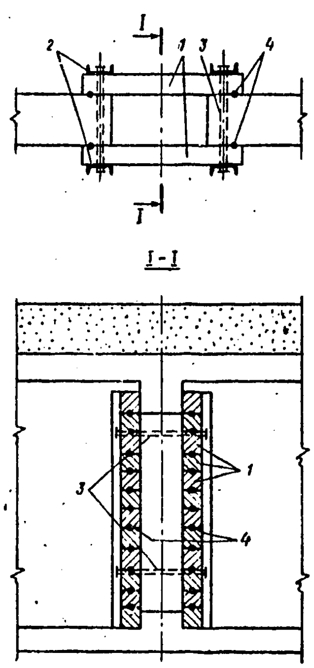


Fig. 48. Closure of opening with precast reinforced concrete elements

1. Precast reinforced concrete beam elements
2. Channels for attaching beams
3. Anchors
4. Airtight liners (packings)

Locations of passage through protective components by industrial devices which are not dismantled and which operate continuously in peacetime (lift trucks, transporters, screw conveyors and so on) can be protected by surrounding them with walls calculated for working loads and installing airtight blast devices in the openings. These components may be located either above or below the overhead cover of the protective structure. Fig. 49 shows a possible structural decision for protecting the spot where a belt conveyor passes through the overhead cover.

A frame of angles (coaming) is built for attachment and transmission of the load from airtight blast and airtight doors to the bearing component of entrances and for ensuring reliable airtightness along the perimeter of the opening. The coaming attachment is made using anchors (Fig. 50).

In designing entrances special attention must be given to designing the joining of airtight blast doors with the shelter's protective components.

If walls of the protective structure are to be made of masonry or precast reinforced concrete, it is best to design special reinforced concrete blocks with a door opening. Fig. 51a depicts a general view of reinforced concrete block with door opening. Reinforced concrete blocks with a door opening can be supported in the vertical plane both along the entire perimeter and on two sides. Provisions must be made for attaching such blocks to the walls of an airlock or sluice chamber to preclude the possibility of its separation under the effect of rarefaction pressure. This attachment can be made using anchors embedded in the walls of an airlock or sluice chamber and welded to inserts in the reinforced concrete block. The number of anchors is determined according to the load created on the blocks by the pressure of rarefaction in the blast wave. The load from rarefaction pressure should be determined according to Paragraph 3.4. For an approximate computation of a reinforced concrete block with door opening in the case of its being supported on two vertical sides, it is suggested that this block be conditionally divided into four parts as shown in Fig. 51b. It is assumed that elements 1 and 2 function as beams on two swing supports and elements 3 and 4 as

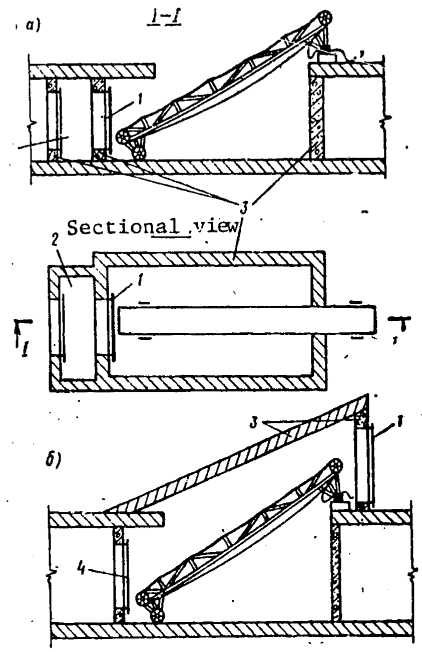


Fig. 49. Protection of industrial openings in shelters

a. Protective components located below overhead cover
b. Protective components above overhead cover
1. Airtight blast door, gate or shutter
2. Airlock
3. Specially built protective components designed for operating loads P
4. Airtight devices

slabs bearing on three sides, one of which has a swing support (the vertical side) and the two others are fixed. The dynamic, evenly distributed load acting on the block is assumed to be equal to the load on walls of the corresponding type of entrance. It is considered that the load is transmitted from the door panel equally to elements 3 and 4, and the load on elements 1 and 2 from elements 3 and 4 is distributed according to the law of triangles (Fig. 51c).

The load transmitted from elements 3 and 4 to elements 1 and 2, distributed according to the law of triangles, leads to an equivalent static force P′ (Fig. 51d). This force in turn can be substituted for a force equal in value acting in the plane of the longitudinal axis of elements 1 and 2 and for the couple of forces in the plane perpendicular to the axis of this element (Fig. 51e). The force acting in the plane of the longitudinal axis of element 1 or 2 will cause a straight bend, while the couple of forces will cause a twist. Elements 1 and 2 thus will experience a complex stress--a bend with a twist. Computation values of the bending moment M_{max}, the cross force Q_{max} and the twisting moment M_{kp} in elements 1 and 2, in case of equal size of elements 3 and 4, can be determined from the formulas:

$$M_{max} = 0,25q_{экв}[0,67HC(B+2C)+0,5b(B+2C)^2]; \quad (9)$$

$$Q_{max} = 0,5q_{экв}(B+2C)(0,5H+b); \quad (10)$$

$$M_{кр} = 0,125q_{экв}bH(B+2C), \quad (11)$$

where $q_{экв}$ is the equivalent static load;

H and B are the height and width of the door opening;

$$C = b^{*} - 0,5d; \quad (12)$$

b is the width of the horizontal element 1 or 2;

b* is the width of elements 3 and 4; and

d is the width of the bearing area of elements 3 and 4.

If the reinforced concrete block is supported above and below or along the entire perimeter, the nature of the elements' work becomes different. Fig. 51f shows the recommended division of a block for approximate computation when there is bearing above and below. In this case elements 3 and 4 can be viewed as beams lying on two swing supports and elements 1 and 2 as plates supported on three sides, one of which has a swing support and the other two are fixed. Division of the block for computation in case there is bearing along the entire perimeter is shown in Fig. 51g. Block elements obtained from this division are best viewed as plates supported on three sides.

2.31. At the inlets to utility lines providing external connections of the given space adapted as a shelter with other spaces, as well as ensuring the functioning of internal equipment systems after the effects of a nuclear burst, provisions should be made for compensatory devices which preclude the possibility of damage to inlets should the shelter settle.

For Paragraph 2.31. Basic decisions for locations where utilities pass through protective components of shelters are shown in Fig. 52. The distance between expansion-contraction joints for shelters made of reinforced concrete or concrete are established in conformity with requirements of SNiP II-B.1-62*, or of SNiP II-B.2-71 for masonry shelters. Structures requiring installation of an expansion-contraction joint must not be buried below the ground water level.

The structural decision for an expansion-contraction joint in protective components is shown in Fig. 53.

Utility inlets into structures must be designed with consideration of the structure's shift with respect to the ground, and utility passages from one block of a structure into another must be designed with consideration of the different displacements of these blocks with respect to the ground. The relative settling of a structure is determined from formula (141) of Appendix 3.

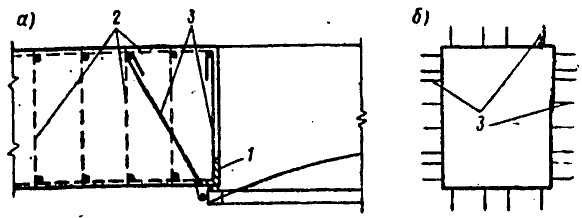


Fig. 50. Attachment of coaming in door opening

a. Coaming anchoring
b. Schematic of anchor positioning around perimeter of door opening
1. Coaming
2. Effective reinforcement based on wall sector calculation
3. Anchors

Waterproofing and Sealing

2.32. Shelter waterproofing must be planned in conformity with requirements of the Instructions for Planning the Waterproofing of Underground Portions of Buildings and Structures. The degree of permissible moistening of shelter protective components should be determined depending on the purpose of spaces being used in peacetime, but not below Category II.

For Paragraph 2.32. The choice and structural decisions for waterproofing must be accomplished based on the requirements for preserving the continuity and imperviousness of the waterproof coating after the effect of design loads.

Materials should be chosen for waterproof coating which possess high adhesion, considerable resistance to rupture, impermeability to water and steam and the greatest relative elongation.

Of the waterproofing material existing at the present time, these requirements are best satisfied by polyvinylchloride plastic compounds Grade 57-40 with a thickness of 2 mm (STU 30-14264-64), high pressure (VD) sheet polyethylene 1.5-2 mm thick, rolled "izol," "brizol," and asphalt-nairit, cement-latex and epoxy-tar compounds.

A waterproof coating of polyvinylchloride plastic compounds and sheet polyethylene is used against ground water with or without pressure and against atmospheric precipitation penetrating into the soil. It is made by gluing these materials to the surface to be waterproofed using asphalt-rubber-solar cement (BKS), the composition of which is as follows (weight in grams): BN-III asphalt--100; SKS-30 latex (recalculated for dry substance)--4; solar oil--16 (or this may be done without gluing).

Individual sheets are joined by lap welding with a 8-10 cm overlap of edges with gluing and 3-4 cm without gluing. Non-glued waterproofing on the substructure is

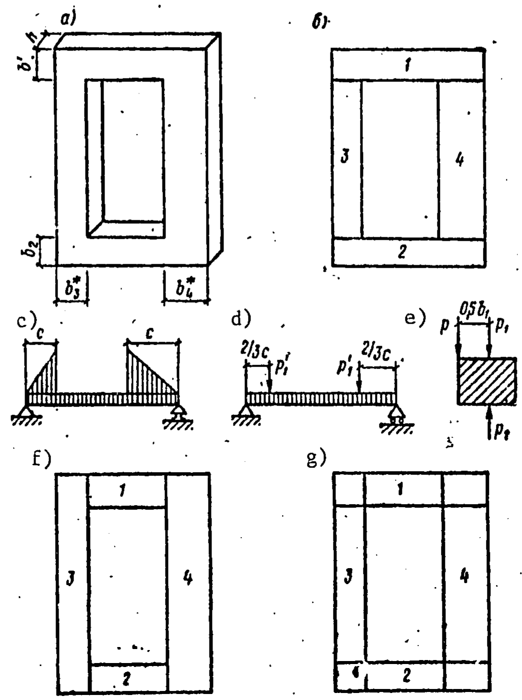


Fig. 51. Computation schematics of reinforced concrete block with door opening

a. General view
b. Division of block into computation elements when supported on the two vertical sides
c. Load diagram
d. Schematic of application of static force P′ equivalent to load distributed from base c according to the law of triangles
e. Schematic of application of couple of forces acting in a plane perpendicular to the axis of element 1 or 2
f. Division of block into computation elements when supported above and below
g. Division of block into computation elements when supported along entire perimeter
1. Upper span piece of block of height b and thickness h
2. Lower span piece of block of height b and thickness h
3. Left post of block of width b_3 and thickness h
4. Right post of block of width b_4 and thickness h

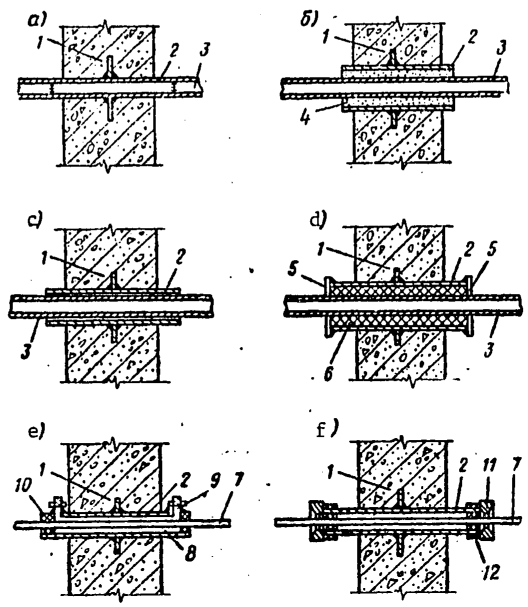


Fig. 52. Inserts of seals at inlets of pipelines and cables

a. For air lines and cold pipes with butt welding (on internal pressurization lines)
b and c. For cold pipes with passage within a sleeve insert
d. Universal inlet for all pipelines
e and f. The same for cables and electric wires
1. Annular steel rib
2. Sleeve insert
3. Pipeline
4. Filling with cement mortar
5. Welded steel flange
6. Asbestos packing
7. Cable or electrical wire
8. Cable cement
9. Nozzle for pouring cable cement
10. Tarred rope packing
11. Nut for stuffing box of SKT [expansion unknown]
12. Cushion

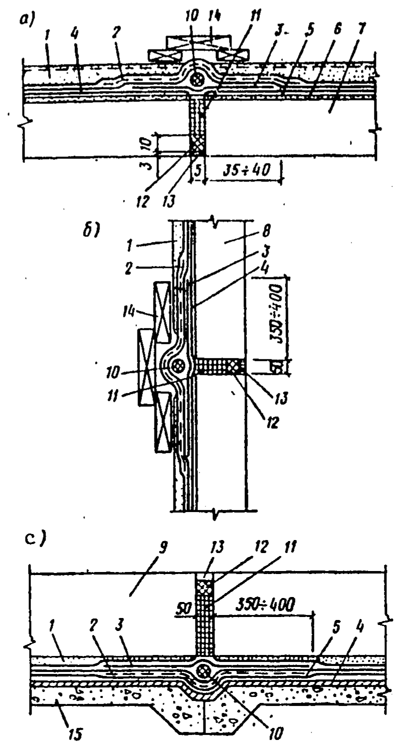


Fig. 53. Structural decision for deformation joint in components of shelters of monolithic reinforced concrete

a. In overhead cover
b. In walls
c. In foundation plates in absence of hydrostatic head
1. Protective layer of cement-sand mortar
2. Glass fabric between two layers of glued waterproofing
3. Additional layer of glued waterproofing
4. Cold bituminous primer
5. Equalizer layer of cement-sand mortar
6. Poured prism with design taper
8. Overhead cover slab
9. Wall
10. Foundation plate
11. ∅ 5 cm asphalt-impregnated braid
12. Filling with "izol" cement or hot asphalt
13. Caulking with tarred braid
14. Cement-sand mortar
15. Clay brick
16. Concrete subfloor

permitted in the absence of a permanent hydrostatic head with a sheet thickness of 1.5-2 mm. To protect the waterproofing against mechanical damage when working on horizontal surfaces, a cement-sand protective bond 3-4 cm thick should be placed on metal gauze. One layer of pergamyn, ruberoid or cardboard is first laid on the waterproof mat of the foundation plate.

Waterproofing of polyvinylchloride plastic compound and sheet polyethylene can be placed on vertical surfaces without installing protective components (bonds, partitions). The fill around structures with waterproofing of the above materials is accomplished with homogenous coarse or medium sand.

Waterproofing of rolled "izol" and "brizol"[1] is used to protect the structure against ground water which is or is not under pressure as well as against atmospheric water. It is installed by gluing these materials to the surface to be waterproofed using the BKS cement. The number of layers of the material is based on calculation, but it is at least three. At least four layers of "izol" or "brizol" are used in the presence of a hydrostatic head. These materials are glued overlapping 10 cm along longitudinal junctions and 15 cm at cross junctions. The panel seams should be staggered both along the length of pieces of material and depending on the number of layers to be glued. To prevent damage of waterproofing during the work a cement-sand protective layer 3-5 cm thick should be placed on horizontal surfaces and cement plaster 3-5 cm thick on metal gauze, or a protective pressure wall of bricks, concrete blocks or slabs with the space between it and the waterproof coating filled with sand-cement mortar with a 1:3 composition should be used on vertical surfaces.

With a wall height more than 2 meters, its stability should be strengthened by installing pilasters, buttresses and so on.

Asphalt-niarit, cement-latex and epoxy-tar waterproofing is used for protecting structures against ground water not under pressure and against atmospheric water. It is installed by mechanized or manual application of the mixture: The first of at least 5-7 layers for a total thickness of 3-4 mm, the second of at least 5 layers 2 mm thick, and the third of at least 3 layers 2-3 mm thick. A cement-sand bond 3-4 cm thick is installed to prevent damage when working on horizontal surfaces. In addition, a protective-pressure wall 1-1.5 meters high out of brick, concrete blocks or slabs is installed on vertical surfaces for cement-latex waterproofing. Asphalt-niarit and epoxy-tar waterproofing requires no installation of protective-pressure walls.

Coarse or medium sand with compacting of layers is used as fill for structures with waterproofing of these materials.

In designing the waterproofing of protective structures, it is necessary to determine zones of possible cracks in protective components and their expansion width under the most unfavorable computed instances of the effects of dynamic loads and, in conformity with this, to make a structural decision for waterproofing of particular material which can deform without rupturing and without losing its waterproofing properties.

1. In connection with "brizol's" insufficient insulation strength in contrast with hot cement, it should be used, as a rule, for waterproofing horizontal components or solvents should be introduced in the cement to lower its working temperature.

Table 4 gives values of crack expansion at which waterproof coatings of different materials deform without rupturing given a time of load increase of 0.15 seconds, a pressure of 5 kg/cm^2 on the waterproofing and relative deformation of 0.2 for sheet polyethylene, 1 for polyvinylchloride plastic compounds, and 0.08 for "izol" and "brizol."

Table 4 – Width of Crack Expansion at which Waterproof Coating Deforms Without Rupturing

Waterproof Coating	No of Layers	Crack Width, mm
Polyvinylchloride plastic compound glued with BKS cement	1	13.8
The same as above, but not glued	1	20
VD sheet polyethylene glued with BKS cement	1	4.9
The same as above, but not glued	1	12
"Izol," glued with BKS cement	3	3
"Brizol," glued with BKS cement	3	4
Asphalt-niarit compound 3-4 mm thick	5-7	1
Cement-latex or epoxy-tar compound	3-5	0.3

If the time values for a build-up in pressure differ from those in Table 4 or with a different number of layers of rolled waterproofing material, a determination of the maximum crack width in concrete at which the material deforms without rupturing can be performed from the following formulas:

a. For coating of a single or multilayer rolled material glued with BKS cement:

$$a_т = k \frac{2E\varepsilon_т^2 \delta}{R_м + qf_1}; \quad (13)$$

b. For coatings of rolled material without adhesion to concrete:

$$a_т = \frac{1,3\varepsilon_т^2 \delta}{q(f_1 + f_2)}, \quad (14)$$

where E is the computed conditional modulus of material deformation in kilograms per square centimeter determined from Table 5;

ε_t is the computed permissible relative deformation of material, taken as 0.2 for sheet polyethylene, 1 for polyvinylchloride plastic compounds and 0.08 for "izol" and "brizol";

δ is the cumulative thickness of all coating layers, centimeters;

R_m is the computed resistance of cement to shear in kilograms per square centimeter determined from Table 5;

q is the standard pressure on the waterproof coating, kilograms per square centimeter;
f_1 is the dynamic coefficient of soil friction on the coating, determined from Table 6;
f_2 is the same for the coating on the concrete determined from Table 6; and
k is the coefficient considering work conditions of the coating in the sector of deformation determined from Table 7.

It can be seen from Table 5 that a number of waterproof coatings deform without rupturing with a rather significant width of crack expansion. But there is a great danger that the waterproofing will shear or be pressed into the cracks and rupture, and so in all instances the width of crack expansion is permitted to be no more than 5 mm.

The relative displacement of elements of precast-monolithic structures at junctions and joints also should be no more than 5 mm for this same reason.

If the crack expansion width by computation is less than 5 mm but greater than the power of the planned waterproofing material to cover them, structural steps should be taken to reduce the crack width or to use waterproofing with greater deformative capacity.

In a number of cases it may be sufficient to reinforce waterproofing in crack expansion zones with additional layers of waterproofing or with a layer of glass fabric.

The following conditions must be observed for glued coatings of synthetic materials for preventing damage to a waterproof coating in a vertical displacement of the structure or with settling of the ground: $qf_1 \leqslant R_M$. (15)

In preparing a structure for waterproofing, its surface should be thoroughly levelled by rubbing over with cement-sand mortar. All the structure's corners must be rounded with at least a 10 cm radius. A cement-sand bond on the coating of freestanding structures must be made with a taper of i = 0.02–0.03 from the longitudinal axis to outer walls.

Examples of structural decisions on installing waterproofing are given in figures 54–56.

2.33. The utility line inlets must be accessible for inspection and repair from the inside of the shelter. Shut-off fittings should be installed within the shelter on water and heat supply inlets.

Inserts for cable, water line and heat supply inlets and for sewer outlets should be installed in the form of steel pipes with annular ribs welded onto their middle part. As a rule, inserts should be installed in protective components before concrete work is done.

For Paragraph 2.33. Measures should be provided for on utility inlets which preclude the possibility of their being severed during mutual displacement of the structure and ground. Structural decisions for inlets of various engineering lines are given in figures 57 and 58.

Table 5 - Computed Resistances and Conditional Deformation Moduli for Rolled Waterproofing Materials and Computed Shearing Strengths of BKS Cement with Varying Load Build-up

Waterproofing Material	Computed Resistances R_p, kg/cm^2 (Numerator) and Conditional Moduli of Elasticity E, kg/cm^2 (Denominator), of Rolled Materials and Computed Resistances of BKS Cement R_m, kg/cm^2, with a Load Build-up Time of t_H, milliseconds, equal to					
	up to 6	8	10	20	40	60	100
Polyvinylchloride plastic compound with ε_t = 0.2	240/1400	230/1200	220/1140	180/920	150/720	140/700	128/650
The same, with ε_t = 1	300/300	285/295	255/290	238/270	240/220	230/215	220/210
Sheet polyethylene with ε_t = 0.2	155/790	143/740	137/710	122/630	115/595	112/560	108/550
"Izol" in 3 layers with ε_t = 0.10	54/560	50/520	47/500	40/430	33/340	31/320	29/300
"Izol" in 4 layers with ε_t = 0.08	72/880	67/820	62/780	54/680	46/550	42/550	39/490
"Izol" in 5 layers with ε_t = 0.08	880/1120	83/1040	79/980	70/830	60/780	54/650	48/560
"Brizol" in 3 layers with ε_t = 0.10	61/630	56/580	53/560	45/480	37/380	35/360	33/340
"Brizol" in 4 layers with ε_t = 0.08	81/990	75/920	70/880	61/765	52/620	47/575	44/550
"Brizol" in 5 layers with ε_t = 0.08	99/1260	93/1170	89/1100	79/935	67/880	61/730	54/650
BKS cement	17.5	17.5	17.5	13	9.8	8	6.2

Note. With other values of t_H the quantities are interpolated.

2.34. Inserts for fastening airtight blast doors or shutters and the inlets and outlets of utility lines should be designed with consideration of loads from the effect of a shock wave.

2.35. After power supply and communications cables have been laid, provision should be made for filling the free space in inserts (tubular portions) with cable cement. The free space within inserts in other inlets and outlets should be filled with sealing gaskets.

Table 6 - Dynamic Coefficients of Friction of Sand on Waterproof Coating f_1 and of Coatings on the Levelled Concrete Surface f_2

Coating Material	f_1 of Sand with Humidity, %				f_2 with a Pressure on Surface up to 5 kg/cm^2
	Medium		Coarse		
	0	5	0	5	
Polyvinylchloride plastic compounds	0.50	0.40	0.55	0.43	0.65
Sheet polyethylene	0.42	0.36	0.45	0.38	0.56
"Izol" and "Brizol"	0.52	0.40	0.60	.045	--

Table 7 - Coefficient Considering Work Conditions of Waterproof Coating in Sector of Its Deformation

$\frac{\delta R_p}{R_M}$	k
1—2	1
>2	1,4

Examples of structural decisions are shown in Fig. 52.

2.36. The degree of airtightness of spaces adapted as shelters must provide an inside air pressure with a single exchange of air per hour:

10 kg/m^2 in conventional shelters;
30 kg/m^2 or more in shelters with higher airtightness.

An operating air pressure of at least 5 kg/m^2 must be provided in spaces adaptable as shelters.

For paragraphs 2.34-2.36. The air pressure in shelters helps prevent the penetration of toxic chemical agents and bacteriological aerosols within the spaces, as well as combustion products when there are outside conflagrations.

An operating pressure is maintained by the supply of air, the amount of which depends on airtightness of protective structures and shelter entrances. The required airtightness of a shelter is achieved:

By high quality of construction work with observance of effective standards for production and acceptance of construction work;

By a reduction in the number of apertures and in the perimeter of input and anti-blast devices, inlets and other inserts;

By conducting activities to prepare shelters for filling.

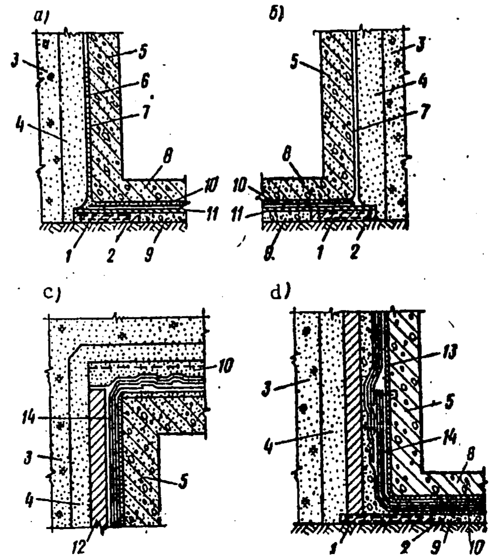


Fig. 54. Example of structural decision for waterproofing

a. Union of polyvinylchloride waterproofing of wall and foundation plate with gluing
b. The same, without gluing
c. Union of "izol" waterproofing of wall and overhead cover
d. Union of "izol" waterproofing of wall and foundation plate
1. Reinforced portion of concrete subfloor
2. Compacted soil
3. Fill
4. Drainage layer
5. Reinforced concrete wall
6. Cement
7. Polyvinylchloride plastic compound
8. Reinforced concrete foundation plate
9. Concrete subfloor
10. Protective bond
11. Pergamyn layer
12. Protective wall
13. Equalizer layer
14. "Izol" ("brizol") waterproofing on cement

In designing civil defense shelters, a reduction in the number of apertures or the perimeter of input and antiblast devices, inlets and other inserts installed

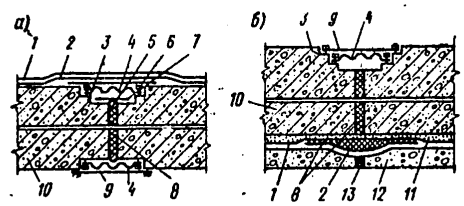


Fig. 55. Waterproofing of deformation joints

a. In coatings and walls
b. In foundation plate
1. Waterproof coating
2. Additional waterproof layer
3. Metal joint cover
4. Rubber compensator
5. Asphalt-impregnated braid
6 and 7. Metal diaphragm and cover
8. UMS-50 cement
9. Metal strip
10. Base component
11. Protective bond
12. Concrete subfloor
13. Tarred board

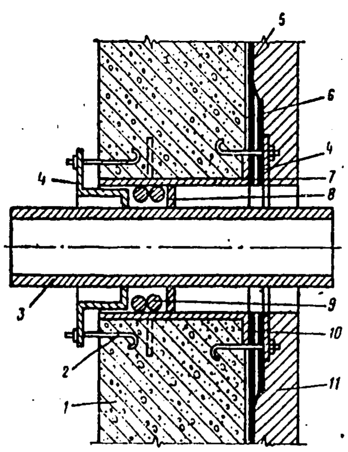


Fig. 56. Waterproofing of pipe passages

1. Main component
2. Anchor bracing bolt
3. Pipe
4. Pressure flange on bolts
5. Waterproof cover
6. Additional waterproof layer
7. Insert (pipe section)
8. Packing stop
9. Seal packing
10. Insert flange
11. Protective wall

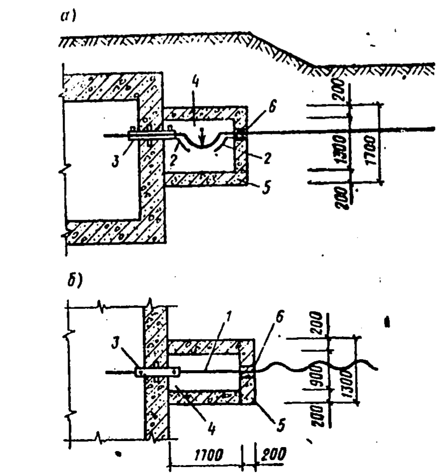


Fig. 57. Compensation device for cable lines

a. Cross section
b. Top view
1. Cable
2. Guides
3. Seal
4. Chamber
5. End wall
6. Seal packing

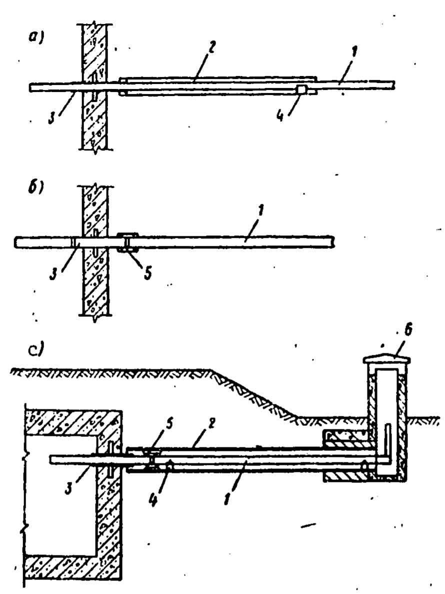


Fig. 58. Compensation device

a. For filled lines
b. For air lines
c. For exhaust gas lines
1. Pipe
2. Pipe casing
3. Seal
4. Braces
5. Union
6. Exhaust shaft

in outer enclosures can be achieved by using wider entrance devices, by replacing several UZS-8's for UZS-25's, and by associating the inlets of utility mains into one or two units.

In addition, avoid embedding various fastening and supporting parts (brackets and so on) in the overhead cover and outer walls which are not covered with soil.

The volume of airlocks should be made no less than 5 m^3 to reduce the danger of contaminating the inner environment of a shelter with harmful substances coming through leaks in entrances.

After the concrete has set, in order to fill possible hollow spots where inserts have been installed, these places should be plugged up by injecting cement mortar prepared from expanding cement. To inject the mortar behind the door casing (coaming), one-inch connections should be installed in the casing figuring one connection per 0.5 m of the casing perimeter. The mortar is injected until the appearance of milky cement where metal joins with the concrete of the enclosure.

3. CALCULATION OF COMPONENTS

Loads and Their Combinations

3.1. Components should be computed for a special combination of loads consisting of permanent and temporary constant loads and a static load equivalent to the effect of a dynamic load from the blast wave (equivalent static load).

In addition, components must be checked by a calculation for the basic combination of loads and effects when sheltered spaces are operated in peacetime, and for the stresses arising and preservation of the airtightness of the shelter's overhead cover with a possible settling by individual supporting piers (columns) of the shelter from the operating load of the above-ground building or structure.

Note. The equivalent static load on a component element is the static load which causes the very same deformations in the element as a given dynamic load.

For Paragraph 3.1. In calculating shelter components for a special combination of loads determined by a given paragraph a load from passing transport is not taken into account. This load is considered in the calculation for the main combination of loads.

A calculation of components for seismic effects during earthquakes is not performed in freestanding shelters erected in seismic areas. Requirements of the SNiP Chapter II-A.12-69, "Construction in Seismic Areas: Design Standards" must be considered in the calculation and design of shelters built into buildings and structures erected in seismic areas.

The main combinations of loads and effects arising while operating shelter spaces in peacetime are compiled in conformity with SNiP Chapter II-A.11-62 from permanent and temporary constant loads and one of the possible momentary loads (which has the most substantial effect on the stressed state of the cross section, element or the entire component being examined).

Components of shelters located on the grounds of oil and gas refineries and chemical production plants are checked by the calculation for the effect of equivalent static loads from the blast wave of a possible explosion of gas-air mixtures formed with the destruction of containers holding dangerously explosive products. The parameters of the shock wave from an explosion of mixtures of hydrocarbon gases of the C_mH_n type are determined from the nomogram and formulas of Appendix 4.

The inner components of shelters (walls, inner walls, columns, intermediate floors) as well as various kinds of devices for fastening internal equipment must be calculated with consideration of inertial forces generated by a displacement of the structure under the effect of loads from the shock wave of a nuclear burst. Recommendations for the calculation and formulas for determining equivalent static loads from the effect of inertial forces are given in Appendix 5.

3.2. The permanent and temporary constant loads must be determined in conformity with requirements of SNiP Chapter II-A.11-62 "Loads and Effects: Design Standards" and in accordance with design standards for construction components of buildings and structures made of different materials.

The permanent load on walls from components of higher floors in the calculation for the special combination of loads should be taken in conformity with SN 405-70.

For Paragraph 3.2. Special combinations are formed from the following loads:

a. Permanent--soil weight and pressure, weight of bearing and protective components of shelters and above-ground building components (only for Class V built-in shelters);

b. Temporary--weight of fixed equipment and of liquids and solids filling the equipment in the process of its operation, and weight of stored supplies and articles;

c. Equivalent static load--from the shock wave of a nuclear burst or the explosion of a gas-air mixture.

In computing the special combination of loads from above-ground building components on walls and columns of Class V built-in shelters, consideration should be given, in addition to the permanent load, to the temporary constant load in the intermediate floors within limits of the computed height of the surviving portion of the building as determined in conformity with SN 405-70.

3.3. In the calculation for the special load combination, the ratios of the combination of loads and overloads to the equivalent static, permanent and temporary constant loads should be assumed equal to unity.

For Paragraph 3.3. In the calculation of shelter components for basic load combinations, the values of the overload factors, load reduction factors and so on are taken in conformity with SNiP II-A.11-62 and with design standards for construction components of buildings and structures of different materials.

Dynamic Loads from the Shock Wave

3.4. The dynamic load on component elements is determined by conditions of the shock wave's effect on the shelter depending on burial in the ground and the ground water level (Fig. 59(?)).

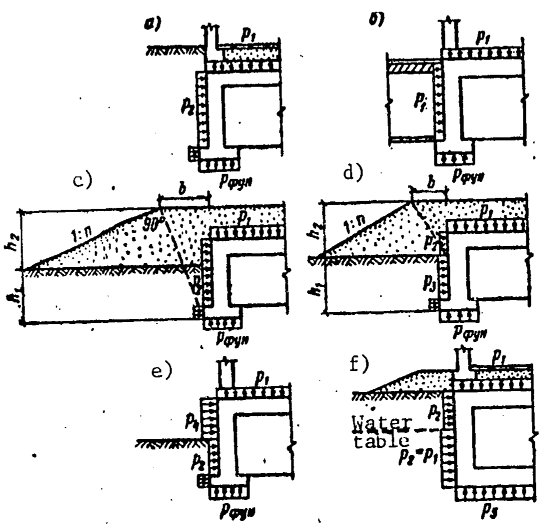


Fig. 59(2). Schematic of application of dynamic loads on components

a and b -- With full burial of built-in shelter and with it adjoining basement spaces not protected against shock wave

c and d -- With partial burial of freestanding shelters banked with soil, with the shoulder slope carried out to a distance b greater than ("c") and less than ("d") the relationship $\frac{h_1 + h_2}{n}$ respectively

e -- With partial burial of built-in shelter and exposed wall sectors

f -- With full burial of built-in shelter and with ground water level above shelter floor

The dynamic load p_n in kilograms per square centimeter is assumed to be evenly distributed in area and applied perpendicular to the surface of components.

For Paragraph 3.4. The dynamic loads on shelter components from the shock wave arise during a nuclear burst or the explosion of conventional explosives such as TNT and mixtures of hydrocarbon gases with air. A load from the collapse of building components above acts on the overhead cover of built-in shelters in addition to the load from a shock wave flowing into first floor spaces through wall openings. But the load from a collapse is not determining, since it is several times less than the dynamic load from the shock wave of a nuclear burst and begins to act at a moment in time when the load from the shock wave has reduced considerably and the load on components has been relieved.

The dynamic load from a shock wave is characterized by direction (vertical, horizontal), by its method of application on components, by the law of a change of its value in time, and by parameters of the shock wave (frontal pressure and time of action).

A shock wave propagates in the atmosphere with supersonic velocity in the form of a compression region (phase) and a rarefaction phase which immediately follows. At the moment the shock wave arrives, the pressure on the earth's surface in the vicinity of the shelter leaps from atmospheric pressure p_0 to the value Δp_Φ at the wave front and then gradually reduces (Fig. 60).

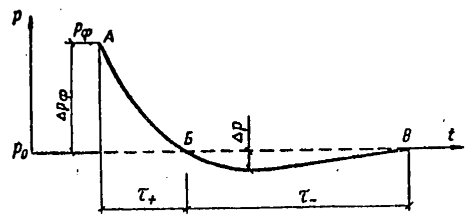


Fig. 60. Change in pressure at fixed point on the ground during passage of shock wave

Δp_Φ -- Overpressure in front
Δp_- -- Maximum rarefaction
τ_+ and τ_- -- Duration of compression and rarefaction phase
p_0 -- Atmospheric pressure
A, B, C -- Points on the curve

After the passage of time τ_+ from the moment of the shock front's arrival, the compression phase passes into a rarefaction phase. Air movement in the direction of wave propagation arises simultaneously with pressure in the compression phase. The air moves in the opposite direction in the rarefaction phase. A change in air velocity u and density ρ in time is analogous to the change in overpressure in the shock wave. In the majority of cases calculated loads on shelter components depend on the overpressure Δp_Φ and dynamic pressure $q = \frac{1}{2}\rho u^2$ in the compression phase.

The rarefaction phase is of substantial importance in determining reverse (suction) loads on blast doors of shelter entrances and on hatches (shutters) of emergency exits. The dynamic effect of the rarefaction phase is small in view of the smooth increase in negative pressure, and so in this phase the equivalent static suction load on doors and hatches is assumed to be equal to the maximum absolute value of the negative overpressure. In the rarefaction phase it can be assumed equal to 0.25 kg/cm^2 for Class II and III shelters and 0.15 kg/cm^2 for Class IV and V shelters.

The basic characteristics of the compression phase are the overpressure Δp_{Φ} at the shock front and time of action τ_+. The magnitude of pressure at the front of a nuclear burst shock wave is taken in conformity with the given degree of shelter protection from SN 405-70. The time of action τ_+ of the compression phase with a given yield of a nuclear burst is determined from corresponding reference data depending on the shelter class.

In calculating components, the effective chart (see Fig. 60) of the change in overpressure in the positive phase is replaced by a triangular chart equivalent in impulse with a linear dependence of pressure on time

$$\Delta p_{(t)} = \Delta p_{\Phi}\left(1 - \frac{t}{\theta}\right), \tag{16}$$

where θ is the effective time of action of the shock wave, which is calculated in the formula

$$\theta = \left.\begin{matrix}(0,72 - 0,08\Delta p_{\Phi})\,\tau_+ \text{ при } 1 < \Delta p_{\Phi} \leqslant 3; \\ (0,85 - 0,2\Delta p_{\Phi})\,\tau_+ \text{ при } \Delta p_{\Phi} \leqslant 1.\end{matrix}\right\} \tag{17}$$

In case data on the yield of a nuclear burst are absent in the assignment for designing a shelter, the effective time of action of a shock wave should be taken from reference data.

The maximum value of dynamic load on a structural element of the structure and the law on its change in time depend on the medium (soil, air) through which the load from the shock wave is transferred, on the conditions for interaction of the shock wave with the component, and on its dimensions. The simultaneous loading of all components of a structure is assumed. The dynamic load is considered to be applied perpendicular to the surface of the calculated component on all spans simultaneously, evenly distributed in area, and changing in time according to linear laws (Fig. 61).

3.5. The dynamic vertical load p_1 on shelter overhead cover (Fig. 59(2)a-f) and overhead cover of the galleries of emergency exits, and the horizontal load p_1 on walls separating the shelter from adjoining basement spaces unprotected from a shock wave (Fig. 59(2)b) should be assumed equal to the pressure in the front of a shock wave Δp.

3.6. The dynamic horizontal load p_2 transmitted through the soil to elements of outer walls (Fig. 59(2)a, c, d, e, f) should be taken from the formula

$$p_2 = k_б \Delta p, \tag{18(1)}$$

where $k_б$ is the coefficient of lateral pressure taken from Table 3; and

Δp is the pressure in the shock wave front in kilograms per square centimeter.

For paragraphs 3.5 and 3.6. For overhead cover of freestanding shelters and of galleries of emergency exits with ground fills no higher than 1 m (or without fill), and for overhead cover of shelters built into structures, the first floor of which has a glass area greater than 50 percent or an easily demolished wall space both with or without openings (for example, nonbearing attached panels of industrial buildings), the law of change in vertical dynamic load in time is shown in the chart in Fig. 61a and maximum pressure is taken equal to the pressure in the shock front.

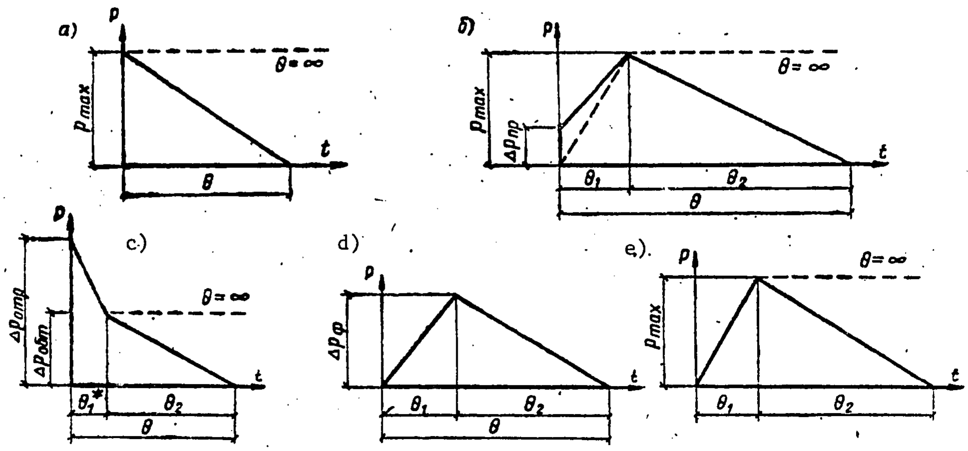


Fig. 61. Laws on the change of dynamic load in time

a. On overhead cover of freestanding shelters and galleries of emergency exits without earth fill and with a fill no higher than one meter
b. On overhead cover of built-in shelters and walls separating the shelter from adjoining basement spaces
c. On frontal element of component rising above ground level
d. On rear element of component rising above ground level
e. On outer wall banked with soil; on overhead cover with a fill greater than one meter; and on foundations on soft, nonrocky soil

For overhead cover of shelters built into buildings, the first floor of which has a glass area of no more than 50 percent, and for walls separating a shelter from adjoining basement spaces unprotected from a shock wave, the law for change of vertical (horizontal) dynamic load is shown in the chart in Fig. 61b, in which:

$\Delta p_{\text{пр}}$ is the overpressure in the front of a shock wave which has passed through openings; for overhead cover the value $\Delta p_{\text{пр}}$ is determined from the chart in Fig. 62 depending on the coefficient α_0 (ratio of the area of openings in first-floor walls to total wall area) and pressure in the shock front in the vicinity of the shelter; for walls separating the shelter from adjoining basement spaces, the value $\Delta p_{\text{пр}} = 0$;

Δp_{max} is the maximum pressure equal to pressure in the shock wave front in the vicinity of the shelter;

θ_1 is the time of load build-up to maximum value, which tentatively is taken equal to 0.15 seconds for Class V shelters, 0.09 seconds for Class IV, 0.06 seconds for Class III and 0.04 seconds for Class II.

Table 8(3) - Coefficient of Lateral Pressure $k_б$

Soil Description in Accordance with SNiP Chapter II-B.1-62*	Coefficient $k_б$
Sandy with a moisture content $G < 0.8$*; sandy loams with a consistency $B<1$; loams and clays with a consistency $B<0.75$*	0.5
Waterlogged soils (below the ground water level); sands with a moisture content $G>0.8$; sandy loams, loams and clays with a consistency $B>1$	1

*In the presence of surveys, it is permissible to take: $k_б = 0.4$ for sands with a moisture content $G \leq 0.5$ and $k_б = 0.6$ for clay with a consistency $0.75<B<1$.

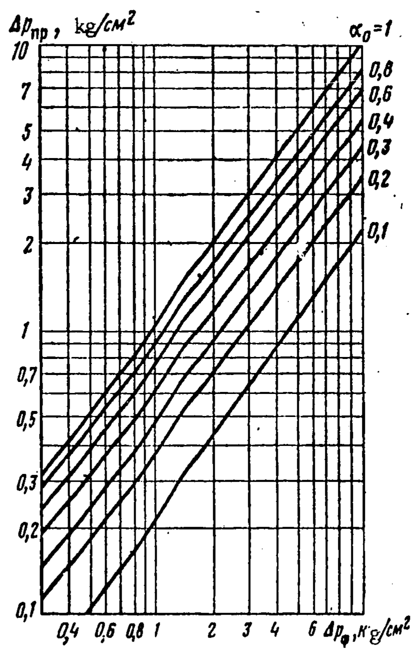


Fig. 62. Chart for determining pressure $\Delta p_{пр}$ at front of shock wave which has passed through openings in walls, depending on the coefficient α_0 and the protective structure class

In the presence of an earth fill more than 1 m thick on the overhead cover of a shelter or gallery of an emergency exit, the overhead cover experiences the effect of the compression wave generated by the shock wave propagating along the earth's surface. Typical of the compression wave is the gradual build-up in stress to a maximum value (see Fig. 61e) in time θ_1, which is determined from the dependence

$$\theta_1 = \frac{x}{a_1}\left(1 - \frac{a_1}{a_0}\right), \tag{19}$$

in which x is the distance from the ground level to the cross section in question; and a_0 and a_1 are the velocities of propagation of elastic and plastic compression waves respectively in the soil.

Velocity a_1 is determined from results of tests on soil samples in accordance with the "Instructions for Calculating Residual Strains of Soils under Dynamic Loads" (Moscow, Stroyizdat, 1967). Tentative values of a_0 and a_1 are given in Table 9.

Table 9 - Compression Wave Propagation Velocities in Soil

Type of Soil	Soil Density, $kg \cdot sec^2/m^4$	Compression Wave Propagation Velocity, m/sec	
		Elastic a_0	Plastic a_1
Loose sand	160	200	100
Loam and sandy loam fills	160	250	150
Clay fills	180	300	150
Undisturbed sand	170	500	250
Undisturbed loam and sandy loam	170	700	350
Compact clay	200	1500	500
Sand and waterlogged sandy loam with an entrapped air content of 0.8-1.5 percent in volume	190	600	450

If pressure Δp_Φ acting on the earth's surface is greater than stress σ_s corresponding to the dynamic limit of soil elasticity under compression, then with an increase in depth the maximum vertical stress (pressure) $\sigma_m(x)$ in the compression wave reduces and becomes equal to σ_s at a depth $x \geqslant x_s$ (where x_s is the distance from the earth's surface to the lower boundary of the zone of formation of plastic deformations of uniform soil). The depth of the zone of formation of plastic deformations of uniform soil is calculated from the formula

$$x_s = \frac{2a_1\theta}{1-(a_1/a_0)^2}\left(1-\frac{\sigma_s}{\Delta p_\Phi}\right), \quad \Delta p_\Phi > \sigma_s. \qquad (20)$$

The dynamic limit of elasticity (σ_s) of soft soils comprises 1-1.5 kg/cm^2 and is determined from results of tests of soil samples.

The distribution of maximum stresses $\sigma_m(x)$ in the compression wave in soil depth is determined by the dependence

$$\left.\begin{aligned} \sigma_m(x) &= \Delta p_\Phi \left[1 - \left(1 - \frac{a_1^2}{a_0^2}\right)\frac{x}{2a_1\theta}\right], \quad 0 \leqslant x < x_s; \\ \sigma_m(x) &= \sigma_s, \quad x \geqslant x_s. \end{aligned}\right\} \qquad (21)$$

In calculations it is convenient to use the attenuation factor, representing the ratio of the maximum pressure in a compression wave at depth x to pressure at the earth's surface.

The attenuation factor $k_{зат}$ is determined from the formula

$$\left.\begin{aligned} k_{зат} &= 1 - \left(1 - \frac{a_1^2}{a_0^2}\right)\frac{x}{2a_1\theta}, \\ & 0 \leqslant x < x_s; \\ k_{зат} &= \frac{\sigma_s}{\Delta p_\Phi}, \quad x \geqslant x_s. \end{aligned}\right\} \qquad (22)$$

As a result of an obstruction to the vertical movement of soil particles offered by a structure's overhead cover, there occurs a process of reflection of the compression wave leading to an increase in pressure on the overhead cover. Reflection of the compression wave is taken into account by the reflection factor $k^*_{отр}$, determined from the chart in Fig. 63 depending on the relationship $\sigma_m(x)/\sigma_s$ and a_0/a_1.

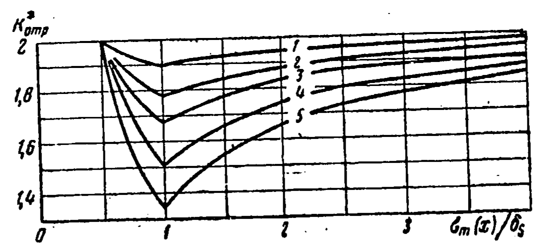


Fig. 63. Chart for determining factor for reflection of compression wave from fixed obstacle

The curves and values a_0/a_1 are: 1 -- 1.1; 2 -- 1.3; 3 -- 1.5; 4 -- 2; 5 -- 3

The maximum dynamic load (see Fig. 61e) on the overhead cover of a structure with dirt fill x = H (H>1 m) will equal:

$$\left.\begin{aligned} p_{max} &= \Delta p_\Phi k_{зат} k^*_{отр} \text{ при } \Delta p_\Phi > \sigma_s; \\ p_{max} &= \Delta p_\Phi k^*_{отр} \text{ при } \Delta p_\Phi \leqslant \sigma_s, \end{aligned}\right\} \qquad (23)$$

where $k_{зат}$ is determined from formula (22) with x = H;

$k^*_{отр}$ are determined from the chart in Fig. 63, and when $\Delta p_\Phi \leqslant \sigma_s$ the relationship $\Delta p_\Phi/\sigma_s$ is used instead of $\sigma_m(x)/\sigma_s$.

The time for build-up in load to p_{max} is calculated from formula (19).

Since soil particles in the compression wave are displaced primarily in a direction perpendicular to the earth's surface, then there is no reflection of the compression wave in the soil from walls of fully buried shelters. A lateral pressure acts against the walls with a gradual build-up to maximum value (see Fig. 61e), comprising a portion of the maximum vertical pressure in the compression wave:

$$\left.\begin{aligned} p_{бок} &= k_б \Delta p_ф k_{зат} \text{ при } \Delta p_ф > \sigma_s; \\ p_{бок} &= k_б \Delta p_ф \text{ при } \Delta p_ф \leqslant \sigma_s. \end{aligned}\right\} \quad (24)$$

Here the attenuation factor from formula (22) usually is determined for cross section x at the level of the middle height of the wall.

The magnitude of the lateral pressure coefficient $k_б$ depends on the properties and granulometric composition of the soil, degree of water saturation, the amount of stresses in the soil and rate of deformation of a soil element. Its value is limited on the one hand by unity for waterlogged soils and, on the other hand, by approximately 0.4-0.5 for soils with natural moisture content. As a rule, the local value of $k_б$ usually is unknown. The value $k_б = 1$ for waterlogged soils ensures a certain reserve in determining maximum pressure on walls.

Example. It is necessary to determine the dynamic loads on the overhead cover and walls of a structure fully buried in the soil, located in various zones of the effect from an explosion of a gas-air mixture (see Appendix 4). The distance from the earth's surface to the structure's overhead cover is H = 2 m, wall height (from top of cover to floor level) is h_{CT} = 4 m, and the soil is loam with natural moisture content. The pressure on the earth's surface and effective time of action of the shock wave in different zones from the explosion of GVS [gas-air mixture] were determined in the example of Appendix 4 and comprise: (1) $\Delta p_ф$ = 17 kg/cm^2 in the action zone of the detonation wave; θ = 0.113 seconds; (2) $\Delta r_ф$ = 3.15 kg/cm^2 in the zone of action of blast products, θ = 0.158 seconds; (3) $\Delta p_ф$ = 1.35 kg/cm^2 in the zone of action of the air blast wave; θ = 0.258 sec.

From Table 9 the propagation velocity of compression waves in fill loam equals: a_0 = 250 m/sec, a_1 = 150 m/sec. The lateral pressure coefficient, in conformity with Table 3 of SN 405-70, is $k_б$ = 0.5.

We will take the dynamic limit of elasticity to be σ_s = 1.5 kg/cm^2. The build-up time θ_1 in the compression wave is computed from formula (19):

For a cross section at the level of the top of a cover (x = 2 m)

$$\theta_1 = \frac{2}{150}\left(1 - \frac{150}{250}\right) = 0{,}006 \text{ seconds}.$$

For a cross section at the level of the middle of the wall

$$\left(x = H + \frac{h_{ст}}{2} = 2 + \frac{4}{2} = 4\,м\right) \qquad \theta_1 = 0{,}012 \text{ seconds}.$$

For a structure located in the action zone of a detonation wave the depth of the zone of plastic deformation formation from formula (20) will equal:

$$x_s = \frac{2 \cdot 150 \cdot 0{,}113}{1 - \left(\frac{150}{250}\right)^2}\left(1 - \frac{1{,}5}{17}\right) = 48\,м,$$

i.e., greater than the depth of cross sections examined in the example. To determine maximum stresses in the compression wave we can use formula (21), where computations provide the following stress values:

At the level of the cover (x = 2 m)

$$\sigma_m = 17\left[1-\left(1-\frac{150^2}{250^2}\right)\frac{2}{2\cdot150\cdot0,113}\right] \approx 17\cdot0,96 = 16,3\ \text{kg/cm}^2;$$

At the level of the middle of the wall (x = 4 m)

$$\sigma_m = 17\left[1-\left(1-\frac{150^2}{250^2}\right)\frac{4}{2\cdot150\cdot0,113}\right] \approx 17\cdot0,92 = 15,6\ \text{kg/cm}^2.$$

Attenuation factors at these levels will equal 0.96 and 0.92 respectively.

The coefficient of reflection of the compression wave from the cover with a value $\sigma_m/\sigma_s = 16,3/1,5 = 11$ will be assumed equal to $k^*_{\text{отр}} = 2$.

The maximum dynamic load on the cover from formula (23) equals:

$$p_{max} = 17\cdot0,96\cdot2 = 32,6\ \text{kg/cm}^2.$$

From formula (24) the horizontal dynamic load on the walls equals: $p_{max} = 0,5\,(17\cdot0,92) = 7,8\ \text{kg/cm}^2$. A chart of the dynamic load on the cover and walls is depicted in Fig. 61a.

When a structure is located in a zone of dispersion of blast products the depth of the zone of plastic deformation formation from formula (20) equals:

$$x_s = \frac{2\cdot150\cdot0,158}{0,64}\left(1-\frac{1,5}{3,15}\right) = 39\ \text{м}.$$

We will perform further computations based on the very same formulas as in the first instance. Stresses in the compression wave and attenuation factors:

At the level of the cover $\sigma_m = 3,15\left[1-0,64\,\frac{2}{2\cdot150\cdot0,158}\right] = 3,15\cdot0,97 = 3,1\ \text{kg/cm}^2.$

$$k_{\text{зат}} = 0,95;$$

At the level of the middle of the wall $\sigma_m = 3,15\left[1-0,64\,\frac{4}{2\cdot150\cdot0,158}\right] = 3,15\cdot0,95 = 3\ \text{kg/cm}^2;$

$$k_{\text{зат}} = 0,95.$$

The coefficient of reflection of the compression wave from the cover according to the chart on Fig. 63 with the value $\sigma_m/\sigma_s = 3,1/1,5 = 2$ and $a_0/a_1 = 1.7$ will be $k^*_{\text{отр}} = 1.8$ and the maximum dynamic load will be $p_{max} = (3,15\cdot0,97)\,1,8 = 5,5\ \text{kg/cm}^2$.

The load p_{max} on the walls equals: $p_{max} = 0,5\,(3,15\cdot0,95) = 1,5\ \text{kg/cm}^2$.

The chart showing the load on walls and cover is depicted in Fig. 61e.

In case a structure is located in the action zone of the air shock wave with a presure $\Delta p_\Phi = 1.35 < \sigma_s = 1.5\ \text{kg/cm}^2$, there will be no attenuation of the compression wave for the levels examined and the maximum dynamic load on the walls will equal: $p_{max} = k_6\,\Delta p_\Phi = 0,5\cdot1,35 = 0,68\ \text{kg/cm}^2$. The coefficient of reflection of the compression wave from the cover based on the chart in Fig. 63 with the value

$$\Delta p_\phi/\sigma_s = 1{,}35/1{,}5 = 0{,}9 \quad \text{and}$$

$$a_0/a_1 = 1{,}7 \quad \text{will be} \quad k^*_{отр} = 1{,}65 \text{ и } p_{max} = \Delta p_\phi k^*_{отр} = = 1{,}35 \cdot 1{,}65 = 2{,}2 \text{ кg/см}^2.$$

The chart of the change in dynamic load in time will remain the same (see Fig. 61e).

The example given graphically shows the low attenuation of the compression wave at depths down to 5-10 m with long action times of the shock wave on the earth's surface.

3.7. The dynamic horizontal load p_3 on elements of outer shelter walls (Fig. 59(2)d) should be determined from the formula

$$p_з = K_б k_{отр} \Delta p, \qquad [25(2)]$$

where $k_{отр}$ is the coefficient which considers reflection of the shock wave taken from Table 4.

Table 10(4)

Slope of banked shoulders	1:5	1:4	1:3	1:2
Coefficient $k_{отр}$	1	1.1	1.3	1.5

For Paragraph 3.7. When the air shock wave runs onto the front slope of an embankment, maximum pressure on the slope in comparison with pressure at the front of the passing shock wave increases sharply as a result of the wave's reflection from the inclined obstacle. But this surge of pressure is very transient and the maximum stress which it generates in the compression wave is intensively attenuated at a shallow depth from the surface of the slope. For this reason the effect of this surge of pressure on the compression wave is not considered and it is assumed that a state of stress is created in the soil by the passing shock wave and that a plane longitudinal wave propagates from the surface of the slope, with the displacement of soil particles in the wave being perpendicular to the slope surface. The frontal wall offers resistance to displacement of soil particles, as a result of which the compression wave reflects at an angle. The pressure of reflection acts against the walls with its value determined by multiplying the maximum stress σ_m in the compression wave from the slope by the reflection factor computed in the formula

$$k_{отр\,\varphi} = k^*_{отр} \cos^2\varphi + k_б \sin^2\varphi, \qquad (26)$$

where ϕ is angle of incidence of the compression wave (the angle between perpendiculars to the surface of the embankment slope and to the surface of the obstacle (Fig. 64a);

$k_{отр}$ is the reflection factor with a perpendicular incidence of the compression wave on the obstacle (determined from the chart in Fig. 63).

Formula (26) is derived by substituting the state of stress in the soil from effective pressure σ_m by the equivalent state of stress created by pressure σ_1 and σ_2 (see Fig. 64a), which are found by using known formulas for the conversion from certain coordinate areas to others from the elasticity theory.

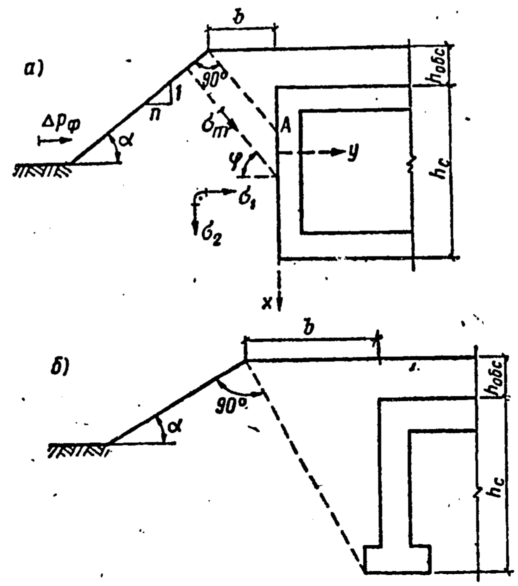


Fig. 64. Schematics of shelter embankment

a. With slope shoulder extended $b < \frac{h_c + h_{обс}}{n}$;
b. The same, with $b > \frac{h_c + h_{обс}}{n}$.

Pressure in compression wave σ_m is determined for a cross section at the level of the middle height of a wall with consideration of attenuation in depth. It is permitted to take σ_m equal to pressure at the front of an air shock wave in view of the slight attenuation of the compression wave from a nuclear burst at practically applicable depths for placement of shelters. A change in the load in time on a frontal wall considering the effect of a slope is taken from the chart (see Fig. 61e), i.e., analogous to a change in load on walls of fully buried shelters. The maximum load (see Fig. 61e) on a wall below point A (see Fig. 64a) equals:

$$p_{max} = \sigma_m(x)\left(k^{*}_{отр}\cos^2\varphi + k_б \sin^2\varphi\right). \quad (27)$$

Above point A we taken the load $p_{max} = \sigma_m(x)\,k_б$. From the drawing (see Fig. 64a) it follows that the compression wave's angle of incidence on the wall equals $\phi = 90° - \alpha$, where α is the dip angle of the slope to the horizontal (tg α = 1/n).

Formula (27) can be reduced to the form (with $\sigma_m(x) = \Delta p_\Phi$)

$$p_{max} = k_б\left(\frac{k^{*}_{отр}}{k_б}\sin^2\alpha + \cos^2\alpha\right)\Delta p_\Phi.$$

From a comparison with formula [25(2)] of SN 405-70 it is apparent that

$$k_{\text{отр}} = \frac{k^{*}_{\text{отр}}}{k_{б}} \sin^2 \alpha + \cos^2 \alpha. \qquad (28)$$

Table 4 of SN 405-70 provides averaged values of $k_{\text{отр}}$. To reduce the load on the front wall, it is recommended that extension b of the slope shoulder be arranged so that the projection of the slope does not fall on the wall (see Fig. 64b). The amount of extension is determined from the geometrical relation

$$b \geqslant (h_{\text{обо}} + h_0) \operatorname{tg} \alpha = \frac{h_{\text{обо}} + h_0}{n}. \qquad (29)$$

With such values of b the maximum load on the wall is determined by formula [18(1)] of SN 405-70. Since the shelter's orientation with respect to the center of a nuclear burst is practically never known, then all walls of a partially buried shelter are calculated for the loads examined above.

3.8. The dynamic horizontal load p_4 for outer walls rising above ground level and directly receiving the load from the shock wave (Fig. 59(2)e) is determined from the formula

$$p_4 = \Delta p + \frac{2,5 \Delta p^2}{\Delta p + 7}. \qquad [30(3)]$$

For Paragraph 3.8. In the process of the shock wave's interaction with shelter components rising above ground level (outer walls, caps of emergency exits), two phases are distinguished: diffraction and flow-past. The diffraction phase is the initial one -- acting on the front component is the reflected pressure $\Delta p_{\text{отр}}$, determined from the chart in Fig. 65 or from the formula

$$\Delta p_{\text{отр}} = 2\Delta p_{ф} + \frac{6\Delta p_{ф}^2}{\Delta p_{ф} + 7,2}. \qquad (31)$$

A rarefaction wave arises at the edges of the projecting portion of a structure flowed around by the shock wave because of a difference in pressures in the incident and reflected waves. Propagation of the rarefaction wave leads to a drop in pressure on the component from the value $\Delta p_{\text{отр}}$ to the value of the flow-past pressure $\Delta p_{\text{обт}}$ (see Fig. 61c). The time from the beginning of reflection to the beginning of establishment of the flow-past regime θ_1^* is tentatively taken as equal to the least of two values calculated from the formula

$$\theta_1^* = \left. \begin{array}{l} 3h/D_{ф}; \\ 3b/2D_{ф}, \end{array} \right\} \qquad (32)$$

where h and b are characteristic dimensions determined in conformity with Fig. 66; and $D_{ф}$ is the velocity of propagation of the shock wave front.

In case an easily collapsed superstructure is built above a shelter, dimension h should be taken equal to the height of the portion of the shelter wall rising above ground level.

The propagation velocity of the shock front is determined from the formula

$$D_{ф} = 340 \sqrt{1 + 0,83 \Delta p_{ф}}. \qquad (33)$$

Table 11 gives values of D_{φ} depending on Δp_{φ}.

In the flow-past phase loads on shelter components rising above ground level form from static pressure (overpressure) in the wave and dynamic pressure

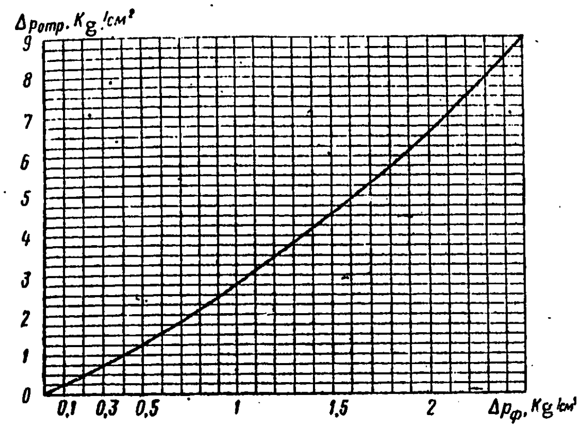


Fig. 65. Chart for determining pressure of reflection of air shock wave

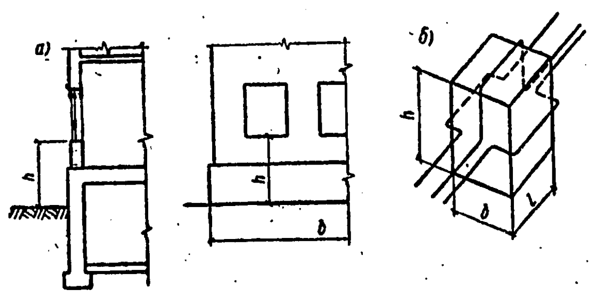


Fig. 66. Schematics for determining dimensions affecting flow-past time

a -- Built-in shelter
b -- Shock wave flow-past of cap
h -- Distance from earth's surface to window opening (height of cap)
b -- Length of building side turned to blast
l -- Length of building side (cap) situated in direction of shock wave movement

(from the velocity head) arising as a result of retardation of the flow. The maximum amount of flow-past pressure is approximately half the reflective pressure:

$$\Delta p_{обт} = 0,5 \Delta p_{отр}. \quad (34)$$

Table 11 - Propagation Velocities of Air Shock Wave Front

Δp_Φ, kg/cm²	0,05	0,1	0,3	0,5	1	1,5	2	2,5	3
D_Φ, m/sec	347	354	380	404	460	510	555	596	635

With a flow-past time $\theta_1^* < \frac{0,2\pi}{\omega}$ it is permissible (for the purpose of simplifying calculations) to ignore the momentary reflected pressure impulse in the diffraction phase and take the maximum load on the component as equal to flow-past pressure from the formula 30(3) of SN 405-70, in which the second term on the right side determines the load from the velocity head.

Example. The outer walls of a built-in shelter with dimensions in a plan view of 12 x 36 m rise 1 m above ground level. The distance from ground level to the bottom of window openings in building walls is h = 2 m. The shelter is in the action zone of an air shock wave from the explosion of GDS (see Appendix 4). Pressure in the front and the effective action time of the passing shock wave equal:

$$\Delta p_\Phi = 1,35 \text{ kg/cm}^2;$$

$$\theta = 0,258 \text{ sec}.$$

The requirement is to determine parameters of the dynamic load on the outer walls of a shelter rising above ground level.

According to formula (31), the maximum load from reflected pressure equals:

$$\Delta p_{\text{отр}} = 2\cdot 1,35 + \frac{6\cdot 1,35^2}{1,35+7,2} = 3,98 \text{ kg/cm}^2$$

with the velocity of the shock wave front determined by the formula

$$D_\Phi = 340\sqrt{1+0,83\cdot 1,35} = 495 \text{ m/sec}$$

The least value of flow-past time from formula (32) will be (with the characteristic dimension h = 2 m)

$$\theta_1^* = \frac{3\cdot 2}{495} = 0,012 \text{ sec}.$$

A change in the load in time is represented in the chart in Fig. 61c.

3.9. The dynamic load p_5 on the continuous foundation plate (Fig. 59(2)f) arising as a result of soil resistance should be taken equal to the pressure at the front of the shock wave Δp.

For Paragraph 3.9. As a result of the load's effect on the structure's cover, it begins to shift in the soil and stresses arise in the foundation hindering a shift of the structure. The maximum dynamic load on the base of the foundation arising from soil resistance can be determined from the formulas derived from an examination of a unidimensional soil movement with the structure located therein as a rigid body:

For freestanding shelters $p_{\max} = \frac{A_\Phi}{A_{\text{п}} + A_\Phi k_\Phi} k^*_{\text{отр}} \sigma_m \Phi_1;$ (35)

For built-in shelters
$$p_{max}=\frac{\Delta p_{max}}{k_\Phi}\Phi_2, \qquad (36)$$

where A_Π and A_Φ are the acoustic resistances of the cushioning layer of soil above the cover (index "П") and beneath the foundation (index "Ф"), expressed by the formula $A = a_1\rho$ (in which a_1 is the propagation velocity of the elastico-plastic compression wave in the soil and ρ is soil density);

$k^{\bullet}_{отр}$ is the coefficient of compression wave reflection from the cover, determined from the chart in Fig. 63;

σ_m is the maximum pressure in the compression wave at the level of the top of the cover;

Δp_{max} is the maximum pressure of the air shock wave on the cover of a built-in shelter;

$k_\Phi=F_\Phi/F_\Pi$ is the ratio of area F_Φ of the base of foundation to the area F_Φ of the structure cover;

Φ_1 and Φ_2 are functions with values determined from the formula

$$\Phi_1=1-\frac{t_m-\theta_1}{\theta-\theta_1}+\frac{m_c}{(A_\Pi+k_\Phi A_\Phi)\,\theta_1}\times$$
$$\times\left[\left(e^{\frac{A_\Pi+k_\Phi A_\Phi}{m_c}\theta_1}-\frac{\theta_1}{\theta-\theta_1}\right)e^{-\frac{A_\Pi+k_\Phi A_\Phi}{m_c}(t_m-\theta_1)}+\right.$$
$$\left.+\frac{\theta_1}{\theta-\theta_1}\right]. \qquad (37)$$

The value of function Φ_2 is calculated from this same formula with $A_\Pi = 0$. Formula (37) has the following notations:

θ_1 is the time of load build-up on the cover to a maximum value (see paragraphs 3.5 and 3.6), in seconds;

θ is the load action time (effective time) in seconds;

m_c is the structure mass per square meter of foundation area (dimension $[m_c]$ - kg·sec^2/m^3 with [A] - kg·sec/m^3);

t_m is the time of load build-up on the base of foundation, determined from the formula

$$t_m=\theta_1+\frac{m_c}{A_\Pi+k_\Phi A_\Phi}\cdot ln\left[\frac{\theta}{\theta_1}-\frac{\theta-\theta_1}{\theta_1}\left(e^{-\frac{A_\Pi+k_\Phi A_\Phi}{m_c}\theta_1}\right)\right], \qquad (38)$$

which has the very same notations as in formulas (35)-(37). The computation of t_m for a built-in shelter is performed according to formula (38) with substitution therein of $A_\Pi = 0$.

Calculation of shelter components usually must be performed in several attempts. For calculations in the first approximation, the values of functions Φ_1 and Φ_2 can be determined from the simplified formula

$$\Phi_1\approx\Phi_2\approx 1-\frac{t_m-\theta_1}{\theta-\theta_1}. \qquad (39)$$

The chart of a change in the load on the foundation base in time is depicted in Fig. 61e, where the value θ_1 is numerically equal to t_m in formula (38). The load fall-off time θ_2 is taken as equal to the effective action time of the shock wave.

Example 1. An air shock wave from the explosion of a gas-air mixture with parameters Δp_Φ = 1.35 kg/cm^2 and θ = 0.258 seconds acts on a built-in Class III shelter. The shelter mass per square meter of foundation area is m_c = 350 kg·sec^2/m^3. The soil under the solid foundation plate is loam with an undisturbed structure. It is necessary to determine the load on the foundation plate.

We take the coefficient $k_\Phi = F_\Phi/F_\Pi = 1$. In accordance with Table 9, a_1 = 350 m/sec, ρ = 170 kg·sec^2/m^4. The load build-up time on the cover is θ_1 = 0.06 sec (see paragraphs 3.5 and 3.6). The acoustic resistance of soil under the foundation plate is A_Φ = 170·350≈6·10^4 kg·sec/m^3. Let us calculate

$$\frac{m_c}{k_\Phi A_\Phi} = \frac{350}{1{\cdot}6{\cdot}10^4} = 5{,}9{\cdot}10^{-3}\ \text{sec}.$$

$$\frac{k_\Phi A_\Phi}{m_c} = 170\ \text{sec}^{-1}$$

According to formula (38), the time for maximum build-up of the load on the plate equals:

$$t_m = 0{,}06 + 0{,}0059\ln\left(\frac{0{,}258}{0{,}06} - \frac{0{,}258-0{,}06}{0{,}06}e^{-170\cdot 0{,}06}\right) \approx 0{,}082\ \text{sec}.$$

Function Φ_2 with $A_\Pi = 0$ from formula (37) equals

$$\Phi_2 = 1 - \frac{0{,}82-0{,}06}{0{,}258-0{,}06} + \frac{0{,}0059}{0{,}06}\left[\left(e^{-170\cdot 0{,}06} - \frac{0{,}258}{0{,}258-0{,}06}\right)\times\right.$$

$$\left.\times\, e^{-170(0{,}082-0{,}06)} + \frac{0{,}06}{0{,}258-0{,}06}\right] \approx 1 - 0{,}111 +$$

$$+\,0{,}1\,[(0-1{,}3)\,0{,}024 + 0{,}3] \approx 0{,}92.$$

From formula (39), $\Phi_2 \approx 0.89$, i.e., the difference from the preceding value is less than 4 percent. The maximum load on the foundation plate from formula (36) equals:

$$p_{max} = p_s = \frac{1{,}35}{1}\,0{,}92 = 1{,}24\ \text{kg/cm}^2.$$

Example 2. The load from a shock wave with Δp_Φ = 1.35 kg/cm^2 and an effective action time θ = 0.258 sec acts on a freestanding shelter with a layer of loam 1 m high above the overhead cover. The shelter mass from Example 1 is m_c = 350 kg·sec^2/m^4. There is undisturbed loam beneath the shelter foundation. It is necessary to determine the load on the solid foundation plate.

From Table 9, for filled loam: a_0 = 250 m/sec, a_1 = 150 m/sec, ρ = 160 kg·sec^2/m^4; for undisturbed loam: a_1 = 350 m/sec, ρ = 170 kg·sec^2/m^4. Maximum pressure in the compression wave at the level of the cover is $\sigma_m = \Delta p_\Phi$ = 1.35 kg/cm^2. The time of load build-up on the cover to the maximum from formula (19) equals:

$$\theta_1 = \frac{1}{150}\left(1 - \frac{150}{250}\right) = 0{,}003\ \text{sec}.$$

We compute the acoustic resistances: $A_\Pi = 160{\cdot}150 = 2{,}4{\cdot}10^4$ kg·sec/m^3;

$A_\Phi = 170{\cdot}350 = 6{\cdot}10^4$ kg·sec/m^3

and the values

$$\frac{m_c}{A_\Pi + A_\Phi} = \frac{350}{(2{,}4+6)\,10^4} = 0{,}0042\ \text{sec};\quad \frac{A_\Pi + A_\Phi}{m_c} = 240\ \text{sec}^{-1}$$

From formula (38) time t_m equals: $t_m = 0{,}003 + 0{,}0042 \ln\left(\frac{0{,}258}{0{,}003} - \frac{0{,}258 - 0{,}003}{0{,}003} e^{-240\cdot 0{,}003}\right) = 0{,}019$ sec.

Function Φ_1 from formula (37) and (39) equals 0.94 and 0.937 respectively, i.e., it has practically identical values.

The reflection factor from the chart in Fig. 63 with values

$$\sigma_m/\sigma_s = 1{,}35/1{,}5 = 0{,}9$$

$$a_0/a_1 = 250/150 = 1{,}7$$

will be $k^*_{\text{отр}} = 1.65$. Then the maximum load on the foundation plate from formula (35) equals:

$$p_{max} = p_s = \frac{6\cdot 10^4}{(2{,}4+6)10^4}\, 1{,}65\cdot 1{,}35\cdot 0{,}94 = 1{,}5 \text{ kg/cm}^2.$$

The time for build-up of this load is $t_m = 0.019$ sec.

3.10. The dynamic horizontal load on walls at the location of entrances should be determined depending on the entrance type and be taken as equal to pressure at the shock front multiplied by the coefficient k_B in accordance with Table 5.

Table 12(5) - Coefficient k_B, Considering Effect of Entrance Type

Entrance Type	Coefficient k_B
From basements unprotected against shock wave	1
Through entrance with covered sector	1.2
Blind and other types	2.3

The amount of dynamic load on inner walls of airlock-sluices should be taken as 20 percent less than the dynamic load on entrance walls.

For Paragraph 3.10. Loads on entrance elements (walls, airtight blast doors and so on) basically depend on parameters (Δp_Φ and θ) of the passing shock wave, entrance type and its orientation with respect to the blast center.

Values of coefficient k_B equal to the ratio of maximum dynamic load at the entrance (determined with consideration of previous remarks) to pressure at the front of the passing shock wave are given in Table 13. This coefficient determines the maximum horizontal dynamic load on sectors of external shelter walls at entrances and on the first (outer) blast or airtight blast doors installed below, in the fore airlocks.

The chart on the change in the load on outer walls and doors at entrances from first floor spaces, basements and stairwells based on time is taken from Fig. 61b with $\Delta p_{\text{пр}} = 0$ and a build-up time of θ_1, determined based on shelter class in conformity with recommendations for paragraphs 3.5 and 3.6. The load chart is taken from Fig. 61a for the remaining entrance types indicated in Table 13.

Table 13 - Coefficient k_B Values based on Entrance Type

Entrance Type	k_B with Shelter Class			
	V	IV	III	II
From first floor spaces and basements unprotected against shock wave	1	1	1	1
From building stairwells (from the street)	1.75	2	2.2	2.5
Through entrance with covered sector opposite entrance opening	1.4	1.25	1.12	1
Through entrance without covered sector	1.7	2	2	1.5
Inclined blind entrance without cap or with light (destructible) cap	1.9	2.2	2.5	2.66

Dynamic load p_t on inner walls of airlocks and airtight doors arises as a result of the shock wave flowing through possible leaks in outer parts of an entrance and around the perimeter where the outer door contacts the door frame. Such leaks are the results of concealed defects of construction and installation work during the installation of inserts and outer door elements. The load on inner airlock walls rises smoothly over a relatively long time until its maximum value p_t and so is taken to be acting statically. The maximum value of p_t is determined from the curves in Fig. 67, each of which correspond to a specific ratio V/l. Values of the product of coefficient k_B (see Table 13) and the amount of pressure at the shock front are laid off along the abscissa axis.

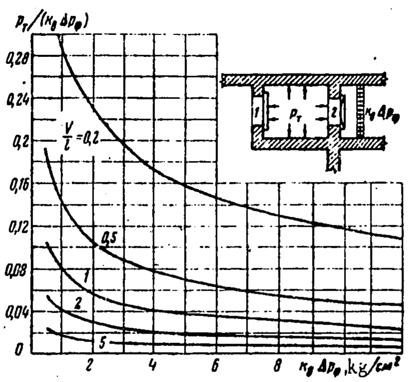


Fig. 67. Chart for determining load p_t on inner walls of airlocks and airtight doors (shutters):

V -- Airlock volume
1 -- Perimeter of airtight blast door opening
1 -- Airtight door
2 -- Airtight blast door

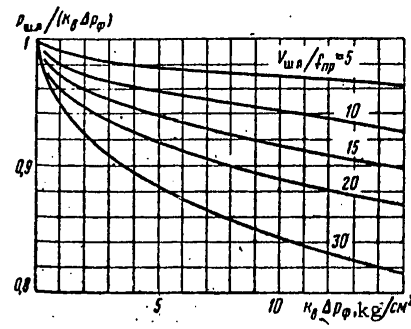


Fig. 68. Chart for determining load $p_{ШЛ}$ on inner walls of airlock-sluice and on second airtight blast door

V -- Airlock-sluice volume, m^3
f -- Opening area for first airtight blast door, m^2

Dynamic load $p_{ШЛ}$ on inner walls of an airlock-sluice and on the second airtight blast door of the sluice is determined from the condition of a possible flow of the shock wave through the open first airtight blast door at the moment the airlock-sluice is being filled. The maximum amount of this load $p_{ШЛ}$ is found from the curves in Fig. 68, each of which correspond to a specific ratio $V_{шл}/f_{пр}$, where $V_{ШЛ}$ is the volume of the airlock-sluice in m^3 and $f_{ПР}$ is the area of the opening in m^2 for the first airtight blast door. The load chart is assumed to have a linear increase until the maximum value of $p_{ШЛ}$ in the time

$$\theta_1 = \theta\left(1 - \frac{p_{шл}}{k_в\,\Delta p_ф}\right) \qquad (40)$$

with a subsequent drop to zero in time $\theta_2 = \theta - \theta_1$ (see Fig. 61e).

Dynamic load on components covering entire cross section of a shaft or gallery of an emergency exit is determined by multiplying the pressure at the shock front by the coefficient given in Table 14.

Table 14 - Coefficient for Determining Loads on Components Located within Emergency Exits

Component Location	Coefficient with Shelter Class:			
	V	IV	III	II
In the shaft	1.8	1.8	1.65	1.63
Within a gallery	1.6	1.6	1.55	1.5

Note. Coefficient values were determined with the amount of clear opening area of louvered grids no less than the area of the shaft cross section.

Equivalent Static Loads

3.11. In determining the equivalent static load on shelter component elements, consideration is given to the magnitude and character of the dynamic load, plastic or elastic properties of materials, and conditions of the components' work. Equivalent static load is determined from the formula $q_{экв} = k_д p_п$, [41(4)] where $k_Д$ is the dynamic-response factor which takes account of a change in dynamic load in time and its interaction with the component. $k_Д$ must be determined from a computation in drawing up standard components; $p_П$ is the maximum dynamic load determined in conformity with paragraphs 3.5-3.10.

For Paragraph 3.11. The effect of a shock wave on shelter component elements is replaced in caculations by the effect of equivalent static loads which generate in elements the very same deformations as the dynamic loads from a shock wave.

Equivalent static load is assumed to be evenly distributed and applied perpendicular to the component surface. Its magnitude per unit surface area equals:

a. In determining bending moments $q^{(м)}_{экв} = p_{max}\,k_м$; (42)

b. In determining cross forces

$$q_{экв}^{(Q)} = p_{max} k_Q; \qquad (43)$$

c. In determining displacement (angles of rotation, deflections)

$$q_{экв}^{п} = p_{max} k_п; \qquad (44)$$

d. In determining longitudinal forces

$$q_{экв}^{N} = p_{max} k_N, \qquad (45)$$

where p_{max} is the maximum dynamic load; k_m, k_Q, k_N, $k_П$ are dynamic-response factors for corresponding stresses and displacements which take account of the dynamic response of the load in conformity with the calculated limiting state.

The basic effect of the rate of deformation on strength properties of steels is taken into account by the reinforcement factor, determined in conformity with paragraphs 3.22-3.24.

The dynamic-response factor for longitudinal force K_N is taken to be equal to unity for both cases of the limiting state (1a and 1b) in calculating bending components.

3.12. The magnitude of equivalent static load on bending elements and eccentrically compressed elements with great eccentricity in reinforced concrete components of overhead cover in computing them for bend and cross force should be taken as equal to the dynamic load determined from Paragraph 3.5, multiplied by the dynamic-response factor $k_Д$, which should be taken from Table 6 in the calculation of supporting power for the bending moment for overhead cover elements depending on the component type and from that same Table 6 with a 10 percent increase for freestanding shelters, but no more than $k_Д = 2$, in calculating for cross force.

Table 15(6) - Dynamic-Response Factor $k_Д$ for Overhead Cover Elements

Calculation State	Class of Reinforced Steel	$k_Д$ Factor for Spaces Adaptable as Shelters: Built-in	Free-standing
For supporting power (Case 1a)	A-I, A-II, A-III A-IIv, A-IIv, A-IV	1 1.2	1.2 1.4
For supporing power (Case 1b)	A-I, A-II, A-III A-IIv, A-IIIv, A-IV	1.2 1.4	1.8 2

In determining the magnitude of longitudinal force for eccentrically compressed elements of overhead cover, the equivalent static load should be taken as equal to the dynamic load determined from paragraphs 3.6, 3.7 and 3.8 of this section with a dynamic-response factor $k_Д = 1$.

For Paragraph 3.12. For calculating a component it is necessary to establish the limiting state, the design schematic of the component, maximum dynamic load, law for its change in time, and preliminary dimensions of element cross sections.

Preliminary dimensions of element cross sections are assigned based on design practice or are established by an approximate computation for equivalent static load equal to the maximum dynamic load multiplied by the dynamic-response factor taken from corresponding paragraphs of SN 405-70.

The methods for determining dynamic-response factors and performing the calculation for bend (strength check) of overhead cover elements for various arrangements are set forth in Chapter 4.

A check of the resistance of overhead cover elements to a cross force in calculating for cases 1a and 1b is performed in conformity with Paragraph 3.37. The cross force at the face of a support from a special load combination is determined from the formula

$$Q_1 = Q_{\text{экв}} + k_t k_y Q_{\text{ст}}, \qquad (46)$$

where $Q_{\text{ЭКВ}}$ and Q_{CT} are the cross forces from the equivalent static load and static load respectively;

k_t is the factor which takes account of an increase in the concrete's strength in time; and

k_y is the hardening factor of concrete.

The calculated value of the cross force is found according to Paragraph 3.38.

3.13. In calculating the centrally and eccentrically compressed supports of frames, columns and inner walls, the evenly distributed vertical equivalent static load should be taken as equal to the dynamic load on the overhead covers (according to Paragraph 3.5), multiplied by the dynamic-response factor $k_{\text{Д}}$ which, in conformity with Table 6, is equal to:

1-1.2 for built-in shelters;

1.2-1.4 for freestanding shelters.

The $k_{\text{Д}}$ factor should be taken equal to 1.8 when freestanding shelters have a floor grade below the ground water level or when they are on a rock foundation.

For Paragraph 3.13. In determining the longitudinal force acting on centrally and eccentrically compressed supports of frames, columns and inner walls, it is recommended that the ratio of dynamic response k_N to the vertical dynamic load on the area from which the longitudinal force is accumulated be made equal to:

For built-in shelters $k_N \approx 1$; (47)

For freestanding shelters with foundations located on several types of soil and with the base grade above the ground water level, according to the chart in Fig. 69 depending on parameters $\bar{q}_1$ and r, computed from the formulas

$$\bar{q}_1 = \frac{a_1 \rho F_{\Phi}}{km}; \qquad (48)$$

$$r = \sqrt{\frac{a_1}{\bar{q}_1 D} - 1}. \qquad (49)$$

where a_1 is the propagation velocity of the elasticoplastic compression wave in foundation soil, taken from Table 9;

where ρ is the foundation soil density;

F_ϕ is the area of the foundation base under a column (wall); for a wall F_ϕ = bD, where b is the distance between axes of beams (plates) resting on walls;

D is the long side of the column foundation base or the width of the base of a wall's continuous footing;

k is a factor equal to k = 2 for columns and k = 1 for walls;

$$m = m_\Phi + m_\kappa + m_n,$$

m_Π is the mass of that part of the overhead cover from which the load on the column (wall) is accumulated;

m_k, m_ϕ is the mass of the column (wall) and foundation beneath the column (wall) respectively.

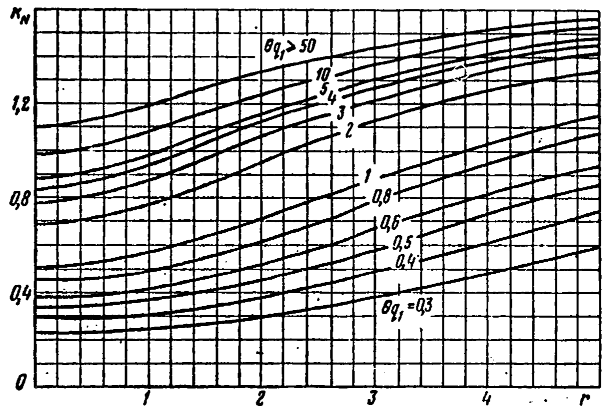


Fig. 69. Chart for determining dynamic response factors for longitudinal force in a column (wall) and beneath the foundation base

The strength calculation is performed by static methods according to appropriate chapters of SNiP from the formula

$$N_{\text{экв}} + 1{,}2N_{\text{ст}} \leqslant N_{\text{в}}, \qquad (50)$$

where $N_{\text{ЭКВ}}$ and N_{CT} are the longitudinal forces from the equivalent static load and static load;

1.2 is the hardening factor; and

N_B is the limiting value of longitudinal force determined with consideration of buckling.

3.14. The vertical equivalent static load on outer walls from the effects of a shock wave on the overhead cover should be determined as pressure on supports from the overhead cover with the action thereon of an equivalent static load equal to $0.8p_1$ and applied within span limits in the clear. In addition, consideration is made for a load directly on the wall section equal to p_1, determined according to Paragraph 3.5, with $k_Д$ = 1.

In determining longitudinal force in masonry walls from the effects of a shock wave on the overhead cover with a span of 2.5 m or less, a corrected span is introduced in conformity with Table 7.

Table 16(7) - Corrected Spans of Overhead Cover

Actual, m	1	1.5	2	2.5
Corrected, m	1.5	1.8	2.2	2.5

The calculation for outer masonry walls (for Case 1a) which adjoin and do not support overhead cover is performed for the longitudinal force from the load directly on the wall's horizontal cross section and from the load from the adjoining cover 1 m wide applied at a distance of 4 cm from the wall's inner surface.

Note. In calculating outer walls consideration should be given to the fact that longitudinal forces act simultaneously with the horizontal equivalent static load.

For Paragraph 3.14. The longitudinal force in outer walls reduces the tensile forces arising from the wall's bending under a horizontal load. The magnitude of the longitudinal force depends on inertial and compressive forces transmitted to the wall from the overhead cover and foundation. The maximums of the bending moment and compressive forces do not coincide in time and precise determination of longitudinal force is rather difficult, since charts of the change in stress in time must be constructed. The dynamic-response factor k_N for longitudinal force in outer walls loaded by the overhead cover is taken to be approximately equal to 0.8 on the basis of calculations.

The vertical force arising from friction of the wall's upper end against supporting components (overhead cover) is considered in the calculation of longitudinal force in outer walls which adjoin but do not support overhead covers.

3.15. The horizontal equivalent static load in calculating reinforced concrete bending elements of outer walls and eccentrically compressed elements of outer walls with great eccentricity should be taken as equal to the dynamic load determined in conformity with the schematic in Fig. 2 and in conformity with paragraphs 3.5-3.8, multiplied by the dynamic-response factor $k_{Д}$ in the calculation for the bending moment in conformity with Table 8, and with an increase of 10 percent but no more than $k_{Д} = 2$, in the calculation for cross force in conformity with the same Table 8.

For Paragraph 3.15. A maximum dynamic load changing in time (see Fig. 61e) and determined from formulas (24) and (27) acts on buried and embanked walls (see Fig. 59(2)a, c, d). There is also a change in the load on walls adjoining basement spaces unprotected against the shock wave [see Fig. 59(2)]. The build-up time θ_1 of the load on buried and embanked walls is calculated from formula (19), in which the value of x is taken equal to the distance from the earth's surface

Table 17(8) - Dynamic-Response Factor $k_Д$ for Elements of Outer Reinforced Concrete Walls

Calculation State	Reinforced Steel Class	$k_Д$ Factor for Walls: Buried and Embanked [see Fig. 59 (2), a, b, c, d]	$k_Д$ Factor for Walls: Not Embanked [see Fig. 52 (2) d]
For bearing power (Case 1a)	A-I, A-II, A-III A-IIv, A-IIIv, A-IV	1 1.2	1.3 1.5
For bearing power (Case 1b)	A-I, A-II, A-III A-IIv, A-IIIv, A-IV	1.2* 1.4*	1.8 2

*For walls in waterlogged soils (with h of ground water higher than 1 m from the shelter floor)

to the middle of the wall, and for walls adjoining basement spaces it is determined in explanations to paragraphs 3.5 and 3.6. Such walls are calculated from formulas applicable for calculating overhead cover components for a load analogous in its character of changes in time, using the charts in Chapter 4 for determining k_M.

A load changing in time according to the chart of Fig. 61c acts on unembanked walls rising above the ground level [see Fig. 59(2)e]. Its maximum value is equal to the reflected pressure determined from formula (31) or from the chart in Fig. 65. The flow-past time is calculated from formula (32).

Table 8 of SN 405-70 provides averaged values of the ratio of dynamic response to the maximum load equal to the flow-past pressure ($\Delta p_{обт}$) for the case where flow-past time is small $\theta_1^* < \frac{0,2\pi}{\omega}$, the reflected pressure impulse is not considered, and a change of the load in time can be taken as analogous to the chart of Fig. 61a, but with maximum pressure $\Delta p_{обт}$.

If the flow-past time is $\theta_1^* > \frac{0,2\pi}{\omega}$, then the reflected pressure impulse in the diffraction phase cannot be ignored and dynamic-response factors have to be determined from the maximum dynamic load equal to the reflected pressure. In the calculation for Case 1b the dynamic-response factor is determined from the chart in Fig. 70 depending on the ratio $\frac{\omega\theta_1^*}{2\pi}$. The calculation for Case 1a is performed with the help of the chart in figures 71-74 and the formulas used for calculating overhead cover components (see Chapter 4).

When $\theta_1^* < \frac{0,2\pi}{\omega}$ the calculation can be performed for a linearly reducing load from a shock with maximum pressure equal to flow-past pressure to zero. The chart and formulas of Chapter 4, applicable for calculating beams with appropriate end fastenings, are used here.

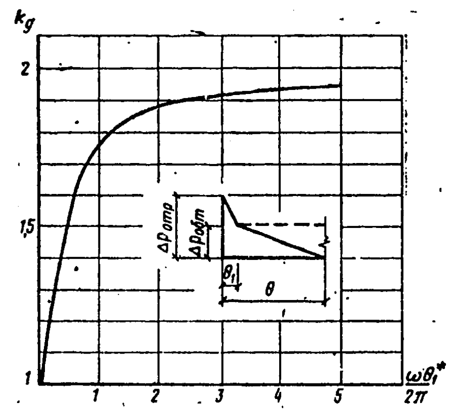


Fig. 70. Chart for determining dynamic-response factors in calculating unembanked walls rising above ground level for Case 1b for a load with diffraction

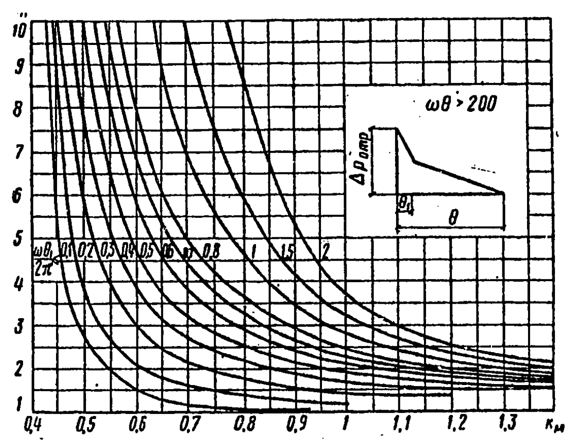


Fig. 71. Chart of the ratio of k_{Π} to k_M in calculating unembanked walls for Case 1a with consideration of the effect of the rate of deformation on strength properties of reinforced steels

3.16. The horizontal equivalent static load on eccentrically compressed reinforced concrete walls with load eccentricity and on masonry walls should be taken:

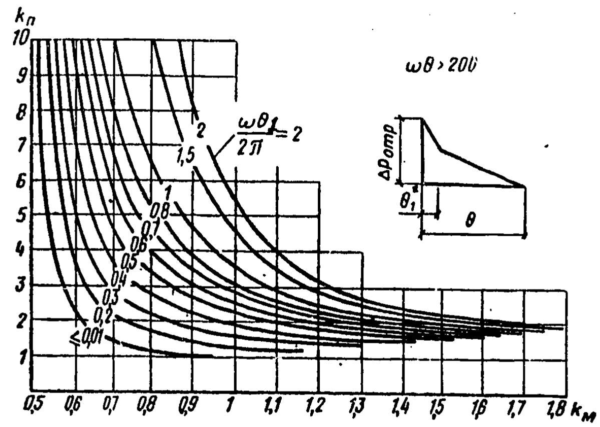


Fig. 72. Chart of the ratio of $k_{П}$ to k_{M} in calculating unembanked walls for Case 1a without consideration of the effect of the rate of deformation on strength properties of reinforced steels

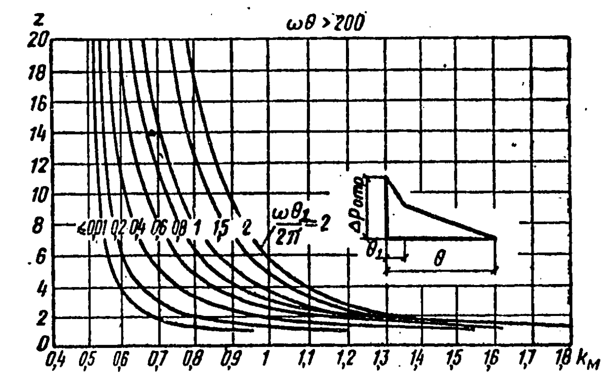


Fig. 73. Chart of dynamic-response factor values for bending moment (without consideration of the hardening of reinforced steel) for the limiting value of dynamic load on unembanked walls

For embanked walls and walls with adjoining basement spaces unprotected against the shock wave--equal to the dynamic load in conformity with the schematic of Fig. 59(2)a, b, c, d, determined from paragraphs 3.5-3.7 with $k_{Д} = 1$;

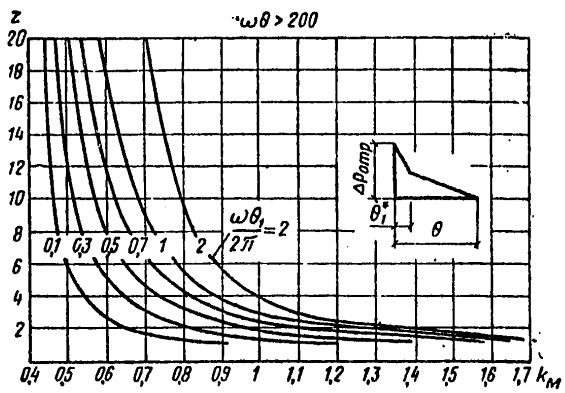


Fig. 74. Chart of dynamic-response factor values for bending moment (with consideration of hardening of reinforced steel) for the limiting value of dynamic loads on unembanked walls

For unembanked walls and walls located below ground water level--equal to the dynamic load (Fig. 59(2)f), determined from paragraphs 3.6 and 3.8 of this section, multiplied by the dynamic-response factor $k_Д = 1.8$, and $k_Д = 2$ for masonry walls without longitudinal reinforcement.

For Paragraph 3.16. The dynamic-response factors k_M and k_θ for embanked walls [see Fig. 59(2)a, c, d] and walls adjoining basement spaces unprotected against a shock wave [see Fig. 59(2)b] can be determined from Paragraph 4.3 depending on $\omega\theta_1$ and the ratio θ_2/θ_1; for walls in waterlogged soil use Paragraph 4.2; for unembanked walls use the chart in Fig. 70 with

$$\theta_1^* \geqslant \frac{0,2\pi}{\omega}(p_{max} = \Delta p_{отр}) \text{ and from Paragraph 4.2;}$$

$$\text{with } \theta_1^* < \frac{0,2\pi}{\omega}(p_{max} = \Delta p_{обт}).$$

3.17. In calculating continuous footings and freestanding foundations, the vertical equivalent static load should be taken to be the same as in determining longitudinal forces in corresponding walls, columns and frame pillars.

In calculating solid foundation plates, the vertical equivalent static load should be taken equal to the dynamic load according to Paragraph 3.9, multiplied by the dynamic-response factor $k_Д$, equal to:

1, in the calculation for Case 1a;

1.2, in the calculation for Case 1b.

In calculating pile foundations the vertical equivalent static load from a shock wave should be taken equal to the dynamic load Δp acting on the overhead cover of built-in shelters, with a dynamic-response factor $k_Д = 1$, and $k_Д = 1.2$ for freestanding shelters.

For Paragraph 3.17. In calculating continuous footings and freestanding foundations for vertical load for freestanding shelters, the dynamic-response factor is determined from the chart of Fig. 69 depending on the parameters $\overline{q}_1$ and r, computed from formulas (48) and (49).

3.18. Emergency exit caps rising above ground level should be calculated for the horizontal equivalent static load equal to the pressure at the shock front multiplied by the dynamic-response factor $k_Д = 2$.

3.19. The horizontal equivalent static load on outer walls at entrance locations should be taken: as equal to the dynamic load determined in conformity with Paragraph 3.10 multiplied by the dynamic-response factor $k_Д = 1.2$, at entrances from spaces unprotected against a shock wave; and equal to the dynamic load determined in conformity with Paragraph 3.10 multiplied by the dynamic-response factor $k_Д = 1.8$ for remaining entrances.

The horizontal equivalent static load on walls within a sluice should be taken as equal to the dynamic load determined in conformity with Paragraph 3.10 of this section, multiplied by the dynamic-response factor $k_Д = 1.2$.

For paragraphs 3.18 and 3.19. In determining horizontal equivalent static load on outer walls and doors, dynamic-response factors can be determined:

At entrances from spaces unprotected against a shock wave and from stairwells--according to Paragraph 4.3, in the charts of which θ_1 is defined in the recommendations for paragraphs 3.5 and 3.6;

For remaining entrance types -- according to Paragraph 4.2.

The ratio of dynamic response to the load on inner walls of airlocks and airtight doors determined according to Fig. 67 is taken as equal to unity.

The ratio of dynamic response to a load on inner walls of a sluice and on the second airtight blast door of a sluice determined from Fig. 68 is found from the charts of Pharagraph 4.3 with θ_1 calculated from formula (40). The fastenings, hinges and anchors of airtight blast doors and shutters are calculated for the equivalent static load from the rarefaction phase, taken as equal to 0.25 kg/cm^2 for class II and III shelters, and 0.15 kg/cm^2 for class IV and V shelters.

3.20. Walls of stairway descents and horizontal exposed sectors of shelter entrances located above the highest ground water level are not calculated for the effect of loads from a shock wave, but with shelters located in waterlogged soil they are subject to calculation for bearing power for strength (Case 1a) for loads in conformity with paragraphs 3.15 and 3.16 of these Instructions.

Protective visors built above a through-type entrance in front of the first airtight blast door should be calculated for an equivalent force from a load applied from below equal to the pressure at the shock front multiplied by a factor of 0.2. In addition, protective covers should be checked by a calculation for the load from collapse of components above them, equal to 3 tons-force/m^2.

Covered sectors of entrances and entrance ramps as well as emergency exit galleries should be calculated both for the effect of the equivalent static load from a shock wave in conformity with requirements of paragraphs 3.11-3.16 of these Instructions, and for the joint effects of an equivalent static load from the shock wave flowing past, equal to 1.6 Δp_{Φ}, with a dynamic-response factor of $k_{Д} = 1.3$.

For Paragraph 3.20. In all instances, the walls of stairway descents and horizontal exposed sectors of shelter entrances are checked by a calculation for the joint effects of the load from the rarefaction phase, the value of which is taken in conformity with recommendations for Paragraph 3.4, and the load from the soil's own weight.

The equivalent static load on components (doors, shutters, antiblast devices) located within emergency exit galleries is taken as equal to the dynamic load (see explanation to Paragraph 3.10) with a dynamic-response factor determined from the chart of Paragraph 4.2.

Sectors of emergency exit galleries extending from the shelter to airtight blast doors, shutters or antiblast devices which prevent masses of air from the shock wave from flowing within the galleries, are calculated only for external loads.

Sectors of emergency exit galleries extending from the caps to the airtight blast doors, shutters or antiblast devices are designed based on a calculation for two types of loading:

a. *Only from without.* The magnitude of loads on structural elements of a gallery and the dynamic-response factors are determined by the methods examined above in corresponding paragraphs;

b. *Resultant loads from without and within.* The load from the passing air shock wave acts on gallery elements from within. It is determined by multiplying the amount of pressure at the shock front by the factor in conformity with Table 14. The dynamic-response factor for the resultant load is taken as equal to unity. The load examined above acts from without.

The external equivalent static load on galleries in the form of cylindrical and air intake pipes of large cross sections is determined by multiplying the external pressure transmitted through the soil by the dynamic-response factor, which is equal to unity. The direction of pressure at any point on the pipe is taken along a radius (Fig. 75). The law of the change in pressure on the circumference is taken as symmetrical with respect to the vertical and horizontal diameters of the pipes

$$p_{(\alpha)} = \sigma_{max}\,(k^{*}_{отр}\cos^2\alpha + k_{б}\sin^2\alpha), \qquad (51)$$

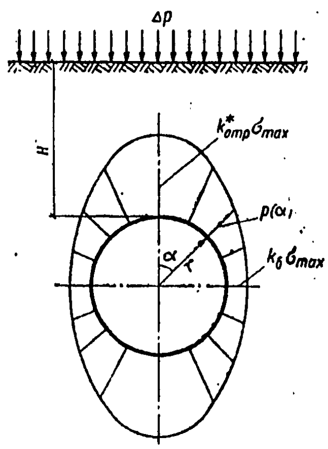


Fig. 75. Schematic of external loading of cylindrical pipe

where α is the central angle read from the vertical diameter;

σ_{max} is the maximum pressure in the compression wave at the level of the top of the pipe, determined from formula (21);

$k^*_{отр}$ is the reflection factor determined from the chart of Fig. 63 in conformity with recommendations for paragraphs 3.5 and 3.6; and

k_σ is the coefficient of lateral pressure.

Materials and Their Estimated Performance

3.21. Concrete for precast and monolithic reinforced concrete components must conform to a design grade for compressive strength of at least 200, and concrete for columns and collar beams--at least 300.

Concrete blocks for walls should be specified with a grade for compressive strength of at least 200, and mortar for filling in joints of precast reinforced concrete components and for concrete block masonry--a grade of at least 100.

Materials with design grades for compressive strength no lower than 100 for bricks, 150 for quarrystone and 50 for masonry mortar should be used in masonry and reinforced masonry components.

For Paragraph 3.21. Grade 100 concrete can be used for secondary components (subfloors, sublayer for external flights of stairs, fixed ramps and so on). The use of light concretes with a grade no lower than 100 is authorized for internal shelter components (inner walls or walls bearing a small vertical load).

3.22. The calculated dynamic resistances of concrete and masonry work in components should be taken as equal to the calculated resistances in accordance with SNiP chapters II-B.1-62* and II-B.2-71, multiplied by the dynamic hardening factor $k_y = 1.2$.

In calculating the cross sections of reinforced concrete and concrete shelter components, consideration must be given to an increase in the strength of concrete depending on hardening periods. The factor k_t should be taken as equal to 1.25 for ordinary cements.

The calculated dynamic shear strength of concretes R^y_{cp} should be taken as equal to the prism strength of concrete $R_{пр}$ multiplied by a factor of 0.25.

Calculated resistances for heavy concrete and values of its initial modulus of elasticity and of calculated dynamic shear strength are given in Table 18(9).

For Paragraph 3.22. The coefficient of dynamic hardening of concrete and masonry materials ($k_y = 1.2$), which takes account of an increase in strength characteristics of materials with high strain rates, is introduced in calculating components

Table 18(9) - Calculated Resistances of Concrete in Reinforced Concrete Components in Structural Design

Stressed State	Symbol	Calculated Resistance of Concrete, kg/cm^2, in Design Grade of Concrete for Compressive Strength				
		200	300	400	500	600
Axial compression (prism strength)	$R_{пр}$	80	130	170	200	230
Bending compression	$R_{и}$	100	160	210	250	280
Axial tension	$R_{р}$	7.2	10.5	12.5	14	15
Shear (with consideration of dynamic hardening) $0.25R_{пр}$	$R^{y}_{ср}$	20	30	40	50	60
Initial modulus of elasticity of compression	$E_{б}$	$2.65 \cdot 10^5$	$3.1 \cdot 10^5$	$3.5 \cdot 10^5$	$3.8 \cdot 10^5$	$4 \cdot 10^5$

for a special load combination and for the effect of inertial forces. Increased strength characteristics of materials are provided for in all types of stressed states of components. The calculated dynamic shear strength of concrete ($R^{y}_{ср}$), determined from Table 18(9), is used in calculating foundations for puncturing, in determining the dimensions of capitals of girderless overhead cover, and in checking for shear stresses at places where monolithic reinforced concrete is connected with precast concrete in the precast-monolithic components of overhead cover.

An increase in strength characteristics of concrete by time through the introduction of the coefficient $k_t = 1.25$ must be considered in calculating shelter components made of heavy concrete and reinforced concrete for the special combination of loads. An increase in the strength of concrete by time is not considered in the protective components of shelters (walls, foundation plates) located below ground water level.

3.23. The "Instructions for Use of Reinforcing Rods in Reinforced Concrete Components" (SN 390-69) should be used as a guide in designing reinforced concrete components.

Reinforcing steels having higher plastic properties, of classes A-II and A-III, should be used as working reinforcements of unstressed reinforced concrete components.

The work condition factor $m_a = 1.1$ should be used in the bending calculation for the steels indicated. Other classes of steel may be used with observance of requirements of Paragraph 3.45.

Reinforced steels of classes A-IV and A-V, steels strengthened by elongation in classes A-IIv and A-IIIv, and heat strengthened steels of classes At-IV and At-V should be used in prestressed components with appropriate substantiation and observance of requirements of Paragraph 3.45.

Class A-I reinforced steel should be used as structural and installation reinforcement.

For Paragraph 3.23. It is permissible to use reinforced steels of classes A-IV, A-V, A-IIv, A-IIIv, At-IV and At-V as working reinforcement for unstressed reinforced concrete shelter components calculated for Case 1b. The work condition factor m_a for these steels is taken to be equal to unity.

In designing components for limiting states, the conditions precluding their onset are considered by introducing three types of design factors: degree of uniformity of material, work conditions and the load factor.

To guarantee fulfillment of the condition $R>\sigma$ in a certain requisite number of cases, where R is the ultimate strength of the material and σ is the actual stress, the calculations use the lesser values of material resistances and greater values of loads. With a load factor equal to unity (see Paragraph 3.3), simultaneous use of the degree of uniformity and work conditions factor (the values of which are less than unity) in determining design resistance leads to a situation where the probability of the onset of the limiting state becomes extremely low. A limiting state does not mean failure of a component, but merely the assumption of certain amounts of strain. Shelter protective components are designed for a one-time or two-time effect of the load from a shock wave having a clear-cut random character, and so the design of these components based on design resistances, the values of which are determined in SNiP by the method presented above, will lead to an excessive safety factor. It is advisable to design shelter protective components based on resistances of materials approaching standard resistances, the values of which are defined in SNiP with a lesser safety characteristic (number of standards) than design resistances. Therefore as a first step in calculating steels of classes A-II and A-III for bending, a supplementary work conditions factor $m_a = 1.1$ is introduced, by which the design resistance of steel is multiplied.

3.24. In the calculation for a special combination of loads, the calculated dynamic resistances of reinforcement (R_y) of the components should be designated with consideration of the hardening of steel at high strain rates and be taken as equal to: for elongated longitudinal reinforcement, lateral reinforcement and recurved reinforcement, in the bending calculation for diagonal cross sections, equal to design resistances of reinforcement (R_a) in conformity with Table 11, multiplied by the dynamic hardening factor (k_y), depending on the class of steel and frequency of the components' natural oscillation in conformity with Table 10;

Elongated lateral and recurved reinforcement, in calculating for lateral force--equal to design resistances of reinforcement (R_{ax}) in accordance with Table 11, multiplied by the hardening factor in conformity with Table 10;

For compressed reinforcement--equal to design resistances of reinforcement (R_{ac}) in conformity with Table 11 with $k_y = 1$.

The design resistances of reinforcement in the structural design, the modulus of elasticity and relative elongation at fracture are given in Table 11.

Table 19(10) - Dynamic Hardening Factor of Reinforced Steel k_y

Type and Class of Reinforced Steel	k_y Factor for Shelters					
	Built-In Regardless of Frequency of Natural Oscillation	Freestanding with Frequency of Natural Oscillation of Components ω, 1/second				
		25	50	100	300	1000
Hot-rolled steel rod, classes A-I, A-II, A-III	1.35	1.2	1.25	1.3	1.4	1.5

Notes: 1. For intermediate values of ω, the value k_y is determined by linear interpolation.

2. For other types and classes of reinforced steel: k_y = 1 for hot-rolled steel of class A-IV and A-V, and for stretch-strengthened and heat-strengthened steels

For Paragraph 3.24. The calculated dynamic resistances of reinforcement R_y of shelter components are determined from the formula

$$R_y = m_a k_y R, \tag{52}$$

where m_a is the work conditions factor;

k_y is the hardening factor considering increased reinforcement resistance with dynamic loading (high-speed deformation);

R represents design resistances of reinforcement with static loading taken in conformity with SNiP Chapter II-B.1-62* and Table 20(11).

The dynamic hardening factor for stretched reinforcement from class A-I, A-II and A-III steels in bending elements is determined based on the lowest frequency ω, rad/sec, of natural oscillation of the component according to the formula

$$k_y = \omega^{1/17}. \tag{53}$$

The k_y factor also may be determined according to the chart in Fig. 76.

For compressed reinforcement of all classes of steels, k_y = 1.

High-carbon steels, high-tensile alloyed steels and heat-hardened steel have low sensitivity to strain rate and so k_y = 1 for reinforced steels of classes A-IIv, A-IIIv, A-IV, A-V, At-IV and At-V.

In designing centrally and eccentrically compressed reinforced concrete shelter components, Class A-II steel is recommended for use as reinforcement installed in the compression zone.

3.25. Welded connections of reinforcement should be made in accordance with requirements of "Instructions for Welding Reinforcement Connections and Inserts of Reinforced Concrete Components" (SN 393-69).

Table 20(11) - Design Resistances of Reinforcement in the Structural Design

Designation and Class of Reinforced Steels	Design Resistance of Reinforcement, kg/cm^2 — Stretched: Longitudinal, Lateral and Recurved with Bending Calculation for Inclined Cross Section R_a	Stretched: Lateral and Recurved with Calculation for Lateral Force R_{ax}	Compressed: R_{ac}	Modulus of Elasticity E_a, kg/cm^2	Relative Elongation at Rupture δ_5, %
Hot-Rolled Reinforced Steel					
Class A-I Smooth	2100	1700	2100	2.1×10^6	25
Periodic Section Class:					
A-II	2700	2150	2700	2.1×10^6	25-19
A-III	3400	2700	3400	$2 \cdot 10^6$	14
A-IV	5100	4100	3500	$2 \cdot 10^6$	6
A-V	6400	5100	3600	1.9×10^6	7
Stretch-Hardened Reinforced Steel					
Periodic Section Class:					
A-IIv	3250	2600	2700	2.1×10^6	8
A-IIIv	4000	3200	3400	$2 \cdot 10^6$	6
Heat-Strengthened Reinforced Steel					
Periodic Section Class:					
At-IV	5100	4100	3600	1.9×10^6	8
At-V	6400	5100	3600	1.9×10^6	7

In the elongated zone of elements it is recommended that welded joints be staggered, but no nearer than 50 cm from each other and not in places of greatest stresses.

The following types of welding should be used for butt joints of working reinforcing rods up to 32 mm in diameter of classes A-I, A-II and A-III: pressure

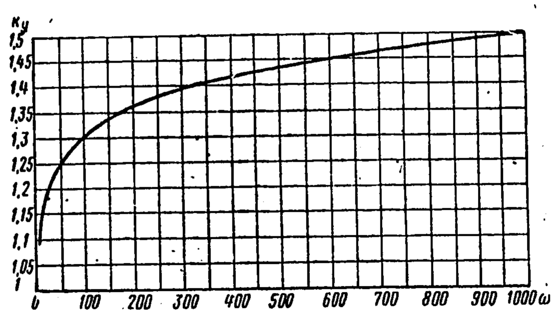


Fig. 76. Chart for determining dynamic hardening factor of reinforced steels, classes A-I, A-II and A-III

contact butt welding, multilayer arc welding on steel liners, multi-electrode vat arc welding on compound steel linings, and vat welding in stock copper form.

When welding working reinforcing rods by other methods or under other conditions of placement, the following work conditions factors should be introduced:

m = 0.95 when using butt joints by arc welding with round straps made of class A-I, A-II and A-III reinforcement;

m = 0.9 with the placement of reinforcement joints in cross sections where the bending moment exceeds 90 percent of the maximum design value.

In case arc welding is used for connecting the intersecting rods of reinforcement cages, the design value of relative strain of working reinforcements made of class A-I, A-II and A-III steels must be taken as $\varepsilon_a = 0{,}2\,\varepsilon_{пр}$ in conformity with Paragraph 3.45.

3.26. The calculated dynamic resistance for rolled sheet and sectional steel in components should be taken as equal to the design resistances in conformity with SNiP Chapter II-B.3-72 entitled "Steel Components: Design Standards," multiplied by the following factors: dynamic hardening k_y = 1.4 and work conditions m = 1.1.

For paragraphs 3.25 and 3.26. Material to be used for steel components should be carbon open-hearth steels of ordinary quality, grades VMSt3sp and VMSt3ps, supplied according to mechanical properties and with supplementary requirements for chemical composition for group V GOST 380-60.

In calculating bending metal components for a special combination of loads, the limiting moment of internal forces in sectional beams of constant cross section can be determined from the formula

$$M_{пред} = R_a k_y m W^n, \qquad (54)$$

where R_a is the design resistance of steel established on the condition of the metal having reached the flow limit (in conformity with SNiP II-B.3-72);

k_y is the dynamic hardening factor equal to 1.4;

m is the work conditions factor equal to 1.1; and

W^n is the plastic moment of resistance equal to twice the static moment of half the sectional area of the relative axis passing through the section's center of gravity. For rolled H-beams and channel steel, $W^n = 1.12\ W$, with bending in the wall plane and $W^n = 1.2\ W$ with bending parallel to flanges.

3.27. In the calculation for a special load combination, standard pressures on nonrocky soils of a base should be taken as equal to the standard pressures on soils in conformity with SNiP Chapter II-B.1-62*, multiplied by the dynamic hardening coefficient $k_y = 5$, but no greater than 15 kg/cm^2.

3.28. Design resistances of bases of rocky soils should be taken as equal to the ultimate resistances of samples of rocky soil for axial compression in a waterlogged state, multiplied by the dynamic hardening factor $k_y = 1.3$.

Basic Design Provisions

3.29. Calculation of components for a special load combination should be performed in conformity with SNiP Chapter II-A.10-71 "Construction Components and Foundations: Basic Design Provisions," for the first limiting state and for bearing power for strength with consideration of the additional requirements set forth in these Instructions.

3.30. Calculation of components made of ordinary reinforced concrete for the first limiting state for strength should be performed with consideration of the plastic properties of materials and the appearance of cracks in the concrete's elongated zone.

The calculation of the strength of elements of reinforced concrete components for sections perpendicular to the element's axis should be performed for the first case, where failure may begin at the most stressed elongated side of the section; the calculation is performed based on the following:

Resistance of elongated concrete is not considered and all tensile forces are transmitted to the reinforcement with stresses therein equal to the design resistance of the reinforcement to stretching, multiplied by the reinforcement's dynamic hardening factor, considering an increase in mechanical characteristics of reinforced steel at high rates of strain;

The diagram of normal stresses in the compression zone of concrete is taken to be right-angled, and the magnitude of stresses is taken for the corresponding standards for designing concrete and reinforced concrete components with a dynamic hardening factor of concrete at high rates of strain.

For paragraphs 3.29 and 3.30. At the present time the design of components of civilian and industrial structures for the effects of static and dynamic loads is performed for limiting states. Momentary loads generated by the effects of a blast wave are one of the varieties of dynamic loads and so all provisions of the method of limiting states are applicable to the calculation for shelters.

Special operating requirements may be placed on protective structures. One of them is that a structure's components must withstand the one-time effect of a load without failure. Great residual strains and displacements may arise which, in reinforced concrete and masonry components are accompanied by strongly developed cracks. Stresses in component material at the most dangerous sections reach limiting values near fracture values. Full use of the strength properties of materials permits obtaining the most economic design decisions.

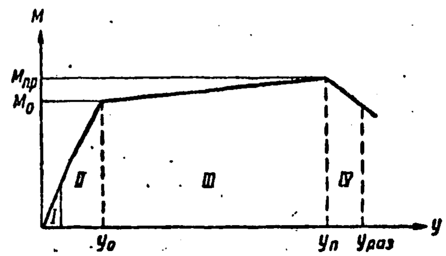


Fig. 77. Bending moment diagram for reinforced concrete beam

$M_{пр}$ -- Limiting bending moment
M_o -- Moment of internal stresses
y -- Sag values

Fig. 77 depicts a typical moment diagram--the sag of bending reinforced concrete components reinforced by low-carbon steel with a yield area. The boundary between stages I and II of the stressed state of the section corresponds to the appearance of cracks in the concrete's elongated zone. The boundary between stages II and III (sag y_o) corresponds to the beginning of yield in the stretched reinforcement, and the boundary between stages III and IV (sag $y_п$) corresponds to the beginning of fracture of the concrete in the compression zone. The strain diagram characterizes in stage IV the process of the component's fracture (a decrease in bearing power).

Theoretical studies indicate that under the effects of a momentary dynamic load, a structure may function without collapsing even in the fracture stage (stage IV). At the present time, however, there are no reliable component strain diagrams in this stage of its performance. In connection with this, in calculating components for the effects of momentary dynamic loads, the attainment of a limiting state before full bearing power is characterized as the beginning of fracture of the material's compression zone, i.e., for a reinforced concrete beam, by the attainment of sag $y_п$, corresponding to the end of stage III. Under this condition, this limiting state coincides with the first limiting state for bearing power (Case 1a), established in SNiP Chapter II-A.10-71.

The methods used for designing structures for the first limiting state for the effect of momentary loads take account of the performance of materials in the plastic range. The fact that strength properties of a majority of materials depend on strain rate is an important feature having a substantial effect on a component's performance. The development of plastic deformations and the increase in strength of materials as a result of the effect of strain rate leads to a situation where, in a number of cases, a component withstands a dynamic load which exceeds the critical static load.

The essence of the second operational requirement on components of protective structures consists of increased strength requirements: A component must take the repeated effects of momentary loads; no substantial residual strains should appear in a component with the one-time effect of a load and all cracks should close after the load's effect ends. Under a load's effect, cracks may develop in a component (such as in the stretched zone of concrete of a reinforced concrete beam), with the possible formation, after the cracks close, of insignificant residual strains which need not be taken into account.

In accordance with these requirements, the attainment of a limiting state is characterized by the appearance of residual strains in the component material. Inasmuch as slight residual strains nevertheless are permissible, this may be termed the limiting state for the absence of large residual strains.

The attainment of this limiting state signifies that the component material is reaching a stage of development of large plastic deformations (sag y_o). Therefore this limiting state also can be attributed to the first limiting state in accordance with SNiP II-A.10-71. SN 405-70 calls it the first limiting state for bearing power (Case 1b).

Necessary guarantee (reliability) of the appearance of limiting states is achieved by introducing design resistances of materials.

3.31. The first limiting state of reinforced concrete bending elements and of eccentrically compressed elements of components with great eccentricities is characterized by the following calculated cases:

Case 1a -- Plastic deformation of stretched reinforcement is allowed;

$$\sigma_а = R_а^{н.у};\ \sigma_б \leqslant R_а^{у};$$

Cast 1b -- Performance of reinforcement in elastic stage considered;

$$\sigma_а \leqslant R_а^{н.у};\ \sigma_б \leqslant R_у^{а}.$$

3.32. The limiting states for bearing power of rectilinear hinged-bearing bending and eccentrically-compressed elements (first case) are characterized by the quantity k--the ratio of full deflection of components in the accepted limiting state ($y_{пр}$) to the quantity of elastic deflection of the component (y_o):

$$k = \frac{y_{пр}}{y_0}. \qquad [55(5)]$$

Full deflection of components should be taken as $1/75l$ in the Case 1a calculation; $1/200l$ in the Case 1b calculation (l is the effective span of the element determined in accordance with Paragraph 3.34).

In the calculation (Case 1a) assume k = 3. Elements of main bearing and protective components and emergency exit galleries are calculated for this state.

The soil's passive resistance must be considered in calculating the walls of galleries located in the soil for the joint effect of the equivalent static load and the load from the passing shock wave.

In the calculation (Case 1b) assume k = 1. Elements of components in which the appearance of residual strains (sags) after load removal is not admissible should be calculated for this state. Components erected in waterlogged soils or subjected to the effects of repeated dynamic loads from secondary factors should be included among them.

In addition, prestressed components with reinforcement of classes A-IIv, A-IIIv, A-IV, A-V, At-IV and At-V should be calculated for this state.

Note: The limiting states of nonsectional, arched and other components also can be standardized by the values of relative deformation and angle opening in a hinge of plasticity.

For paragraphs 3.31 and 3.32. The limiting state 1a of bending and eccentrically-compressed reinforced concrete components with great eccentricity is attained as a result of their performance in the plastic state, i.e., when the stretched reinforcement at the most stressed sections perpendicular to the longitudinal axis is in a state of plastic flow. Cracks, which break the entire component up into separate little deformed sectors, open up strongly in these sections and develop along the elevation of a beam. The component appears in the form of a mechanism of rigid elements connected by fixed hinges of plasticity in which concentrated bending moments are applied. Attainment of the first limiting state is characterized by the beginning of fracture of the concrete in the compression zone in sections performing in the plastic stage at the moment the component achieves the greatest displacements. It is assumed that the reinforcement possesses a sufficient margin of plastic deformation and does not break until complete fracture of the compressed concrete, and that the section is not over-reinforced, i.e., the compressed concrete does not fracture until the beginning of the reinforcement's yield.

Fracture along diagonal cracks from lateral force is very dangerous for reinforced concrete components. For this reason, to prevent a large expansion of diagonal cracks, it is advisable for the reinforcement receiving the lateral force to perform only in the elastic stage. This is ensured by a reduction in values of design resistances and an increase in dynamic-response factors for lateral force.

Fracture of the compressed zone sets in at the moment when stresses of the concrete reach ultimate compressive strength in bending. At this moment the deflection of the component should be maximum and its rate of movement equal to zero.

The limiting state for strength (Case 1a) is standardized by the values of deformations which are so chosen that they can be found by a dynamic calculation of the component and at the same time be convenient for experimental determination. The crack opening angle in the hinge of plasticity introduced by A. A. Gvozdev is the most convenient standardizing value for bending reinforced concrete elements. The strength condition of a component in which n hinges of plasticity form has the appearance

$$\psi_i \leqslant \psi_{n\,i}, \quad i = 1, 2, 3, \ldots, n, \tag{56}$$

where ψ_i is the opening angle in the i-th hinge of plasticity obtained from the dynamic calculation;

$\psi_{\Pi i}$ is the limiting opening angle in the i-th hinge of plasticity.

The value of the limiting opening angle ψ_Π depends on the relative height α_p of the concrete's compression zone in the section with a crack during fracture equal to, for a rectangular cross section,

$$\alpha_p = \frac{R_a}{R_u}\mu, \tag{57}$$

where R_a and $R_И$ are design resistances of reinforcement and concrete given in SNiP II-B.1-62*;

μ is the reinforcement factor of relatively stretched reinforcement.

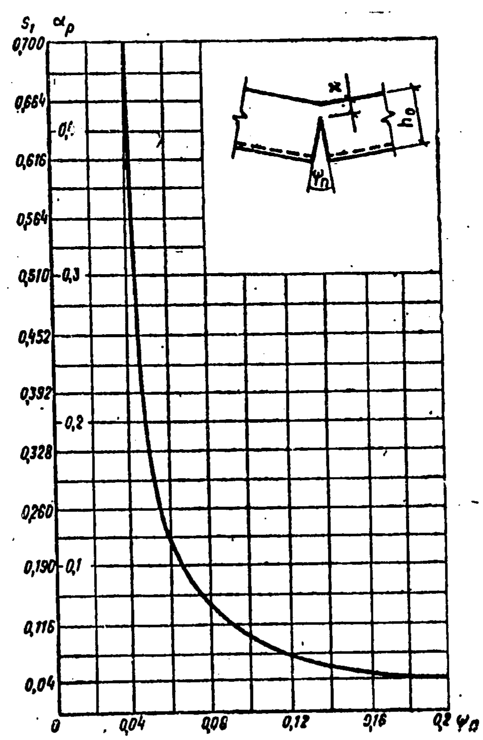


Fig. 78. Chart for determining limiting opening angle in hinge of plasticity ψ_Π

$$S_1 = \frac{S_\delta}{S_0}, \quad \alpha_0 = \frac{R_0}{R_u} \quad \mu = \frac{x}{h_0},$$

where x is height of concrete's compression zone;

h_0 is the effective height of section;

μ is the percent of reinforcement

Fig. 78 shows a chart of ψ_{Π} depending on α_p and the section characteristic $\frac{s_б}{s_0}$, constructed from empirical data. The values α_p and $s_б$ should be determined from design resistances of reinforcement and concrete given in SNiP II-B.1-62*, and without consideration of compressed reinforcement F'_a, since its effect on the value ψ_{Π} has not been studied. The chart in Fig. 78 is approximated by the relationship

$$\left.\begin{aligned} \psi_п &= 0{,}035 + \frac{0{,}003}{\alpha_p} \quad \text{при } \alpha_p \geqslant 0{,}02; \\ \psi_п &= 0{,}2 \quad \text{при } \alpha_p < 0{,}02. \end{aligned}\right\} \tag{58}$$

For rectilinear hinged-bearing beams with standardization of the limiting state 1a by the value k -- the ratio of sag from formula [55(5)] of SN 405-70, the strength condition (56) is written in the form

$$\psi \leqslant \psi_{max}, \tag{59}$$

where ψ is the opening angle in the hinge of plasticity in the middle of the beam span obtained from dynamic calculation;

ψ_{max} is the maximum permissible opening angle in the hinge of plasticity, equal to

$$\psi_{max} = \frac{5M_0 lk}{12B}. \tag{60}$$

The value ψ_{max} must not exceed ψ_{Π} in formula (58).

In formula (60) M_0 is the moment of stress in stretched reinforcement of a medium-section beam with respect to the center of gravity of the concrete's compression zone determined from the calculated dynamic resistance of the reinforcement

$$M_0 = m_a R_a^y F_a l h_0 \left(1 - 0{,}5\alpha_p\right). \tag{61}$$

The strength condition (59) is automatically fulfilled in determining the dynamic-response factors for a hinged-bearing beam using formulas (99) and (100).

The calculation of bending reinforced concrete components for the limiting condition 1b ensures performance of the component without permanent elongations of stretched reinforcement. Attainment of this limiting state is characterized by the beginning of the appearance of plastic deformations in the most stressed sections of the stretched reinforcement at the moment the component reaches the greatest displacements. Here cracks (stage II of the stressed state of the section, see Fig. 77) appear in the concrete's tensile zone, but they close after the load's action ceases. In calculating components without prestressing it is usually possible to ignore stage I of their sections' performance (up to the appearance of cracks in the concrete's tensile zone, i.e., the sections are assumed to be performing only in stage II. Displacements and stresses are determined by conventional methods of the dynamics of elastic structures, and the rigidity of component elements is determined for the most stressed section with consideration for the appearance of cracks in the concrete's tensile zone. Inasmuch as the components' stresses and displacements can be determined by calculations, standardization of the limiting state 1b is performed with the stresses: at the moment maximum displacements are achieved, stresses in the stretched reinforcement of the most stressed sections reach the value of the dynamic flow limits.

3.33. The determination of internal stresses in component elements (bending moments, longitudinal and lateral forces) should be performed by the rules of structural mechanics from equivalent static and other loads in accordancd with demands of Paragraph 3.1 of these Instructions.

In the calculation (Case 1a) of statically indeterminate beam and frame components, it is permissible to consider the redistribution of stresses between supports and a span as a result of plastic deformations and crack expansion. A reduction in the bending moment on a support obtained from a calculation for equivalent static loads must not exceed 50 percent for beams and 30 percent for slabs of the overhead cover and foundation.

3.34. It is recommended that the effective span of a bending element be taken as:

The distance between centers of support areas for hinged-bearing elements;

The internal width for rigidly fixed elements;

The internal width multiplied by 1.05 for continuous components if the thickness of supporting components is greater than 30 cm, or the distance between centers of supporting components if their thickness is equal to or less than 30 cm. The effective length of compressed elements is taken to be equal to 0.7 times their internal length for monolithic, precast-monolithic, and precast components with joints covered and equal to the internal length for precast components without joints covered.

3.35. To prevent the possibility of a fracture of the concrete's compression zone in bending reinforced concrete elements before the component attains a given limiting state for strength, the section characteristic must satisfy the condition

$$S_б^y / S_0 \leqslant \zeta, \qquad [62(6)]$$

where $S_б^y$ S_0 are the static moments respectively for the sectional area of the concrete's compression zone determined with consideration of dynamic design resistances, and of the entire effective section with respect to the axis perpendicular to the plane of action of the bending moment and passing through the point of application of resultant stresses in the stretched reinforcement;

ζ is the coefficient, the maximum value of which is taken from Table 12 for rectangular cross sections of bending and eccentrically compressed elements.

For rectangular cross sections condition [62(6)] can be presented as $x \leqslant x_{np}$, [63(7)]
where x is the height of the compression zone determined from the formulas:

$$x = \alpha h_0; \quad \alpha = \mu \frac{R_a^y}{R_и^y}; \quad x_{np} = \alpha_{np} h_0; \qquad [64(8)]$$

α_{np} is the coefficient taken from Table 12;
h_0 is the effective cross section height;
μ is the reinforcement factor of stretched reinforcement, equal to $\mu = \frac{F_a}{bh_0}$;

R_a^y, $R_и^y$ are the dynamic design resistances of the reinforcement under tension and of the concrete to compression while bending respectively.

Table 21(12) - Values of Coefficients ζ and $\alpha_{\text{пр}}$

Calculated State	Coefficients	Coefficient Values for Design Grade of Concrete for Compressive Strength		
		400 and below	500	600
For bearing power (Case 1a)	ζ $\alpha_{\text{пр}}$	0.6 0.4	0.5 0.3	0.45 0.26
For bearing power (Case 1b)	ζ $\alpha_{\text{пр}}$	0.8 0.5	0.7 0.45	0.65 0.4

3.36. For eccentrically compressed elements with rectangular cross sections, instead of $\alpha=\mu\frac{R_a^y}{R_u^y}$ we should take

$$\alpha_0=\alpha+\frac{N-R_a^y F'_a}{bh_0 R_u^y}, \qquad [65(9)]$$

where N is the longitudinal force;

F'_a is the sectional area of compressed reinforcement;

b is the sectional width of the concrete's compression zone;

h_0 is the effective height of cross section.

3.37. For reinforced concrete elements with rectangular, T, H, and box section, in cases where the crack expansion width is not standardized, the condition $Q\leqslant 0{,}35R_u^y bh_0$, [66(10)] must be satisfied and, in remaining cases, in accordance with Paragraph 7.25 of SNiP Chapter II-B.1-62*

$$Q\leqslant 0{,}25R_u^y bh_0, \qquad [67(11)]$$

where Q is the transverse force from the equivalent static load.

For Paragraph 3.37. Condition [66(10)] is used for checking bending reinforced concrete shelter elements calculated for Case 1a.

In checking the cross section, the limiting value of transverse force Q is determined at the margin of an element's support only from the effect of the equivalent static load.

If condition [66(10)] is not observed, then there must be an increase in the size of cross section or in the grade of concrete.

3.38. In calculating bending elements of components for diagonal cross section, the calculated value of lateral force from the equivalent static load and the element's dead weight should be determined with consideration of that portion of the load applied to the element within limits of the length of the diagonal section's projection and reducing the value of the lateral force (if this load is not applied within limits of the element's height). The amount of load from the effects of the shock wave within limits of the diagonal section should be taken as 0.8 of the dynamic load (p_n), determined from paragraphs 3.5-3.10 without consideration of the dynamic-response factor, and from dead weight with a factor of 0.5.

For Paragraph 3.38. In reinforced concrete bending components with a solid as well as a T section with the flange in the compression zone, it is permissible to reduce the calculated value of lateral force Q_p from the effects of the equivalent static, permanent, and temporary long-acting load. It can be determined from the following formula:

$$Q_p = Q - 0{,}8p_{max}\,bC_0 - 0{,}5q_{ct}\,bC_0, \qquad (68)$$

where Q is the lateral force on the support, determined in the calculation for a special load combination from formula (46);

p_{max} is the maximum value of dynamic load;

Q_{ct} is the sum of the permanent and temporary long-acting load;

b is the width of load collection on the calculated component;

C_0 is the projection length of the nonoptimum diagonal section on the element's longitudinal axis, determined in conformity with Paragraph 7.32 of SNiP II-B.1-62*.

3.39. Eccentrically compressed reinforced concrete elements are calculated:

For the first case, corresponding to relatively high eccentricities, if condition [62(6(] is satisfied;

For the second case, corresponding to relatively low eccentricities, if [62(6)] is not satisfied.

3.40. The calculation of cross sections for bending and eccentrically compressed (for the first case) elements of precast-monolithic components is performed in the very same manner as for monolithic components. The effective height of cross sections should be taken with inclusion of the height of precast elements.

In precast-monolithic reinforced concrete components, a connection must be provided between precast elements and additionally placed concrete by roughing up the surface of precast elements, installing tongues and by other methods. When precast elements are located in a compression zone, there must be a check for shear, and the concrete's calculated resistance to shear along a joint should be taken:

Equal to 0.6 R^{y}_{cp} with an unprocessed surface of precast elements;

Equal to 0.8 R^{y}_{cp} with a processed surface (for a "shuba" or tongues).

For receiving a transverse force on supports, additional structural decisions must be provided which ensure the joint performance of monolithic and precast reinforced concrete. It is advisable to leave gaps between precast elements to be covered over after placing therein calculated reinforcement for lateral force.

The work condition factor m = 0.9 is introduced in calculating cross sections of precast-monolithic components, both compressed and eccentrically compressed (Case 1a).

For paragraphs 3.39 and 3.40. In precast-monolithic bending components, joint performance of precast elements and of monolithic concrete is ensured in those instances where shearing stresses along the contact surface τ determined from the formula

$$\tau = \frac{Q}{0{,}9bh_0}, \qquad (69)$$

will not exceed limiting values of τ found from the expression

$$[\tau] = 0{,}25 R_{\text{пр}} k_{\text{пов}}, \qquad (70)$$

where Q is the lateral force in the given section of the element determined from the condition of the calculation for a special load combination;

b is the effective width of the element;

h_0 is the effective height of cross section;

$R_{\text{пр}}$ is the prism strength of concrete with axial compression;

$k_{\text{пов}}$ is a factor taking account of the degree of surface roughness of the precast element and equal to 0.45 for a smooth (even) surface; 0.60 for a natural (rough) surface; 0.68 in the presence of local depressions (15 x 15 x 10 mm) with a spacing of 10 x 10 cm; and 0.8 with embedding of crushed rock 20-40 mm in size every 5-7 cm in freshly placed concrete of a precast element.

If $\tau > [\tau]$, then provisions must be made for projections of reinforcement out of the precast element into the monolithic concrete layer perpendicular to the surface and in an amount determined by the calculation for lateral force. The surface of precast elements must be clean and not contaminated by various kinds of oils.

In precast-monolithic bending slabs (panels), lateral reinforcement in the cross section may be installed in the precast element and in intervals especially left between them. The calculation takes account only of those lateral reinforcing rods of precast elements which are intersected by a diagonal crack and have sufficient anchoring in the concrete. In this regard it is advisable to provide for a layer of monolithic concrete of minimum thickness based on the possibility of locating effective reinforcement above the support in it and the convenience of accomplishing the work. The minimum thickness of a layer of monolithic concrete placed on slabs (panels) can be taken as equal to 10 cm.

In all cases, projections of lateral reinforcement into the layer of monolithic concrete are arranged in precast-monolithic bending collar beams. Reinforcement area is determined by the calculation for lateral force.

Components of precast elements in precast-monolithic overhead cover must be checked by the calculation for the effect of the weight of freshly placed concrete and other loads arising in the process of erecting a protective structure.

In precast-monolithic components performing with axial or eccentric compression, provisions must be made for measures to increase the cohesion of precast elements with the monolithic concrete. To this end it is recommended that precast elements with a T-section be used with flanges oriented in the direction of the monolithic concrete. With flat precast elements, projections of lateral reinforcement into the monolithic concrete should be provided based on a figure of at least 5 cm^2/1 m^2 of surface.

3.41. The minimum cross sectional area of longitudinal reinforcement is taken in accordance with requirements of Paragraph 12.13 of SNiP Chapter II-B.1-62*.

The optimum percentage of reinforcement for bending and eccentrically compressed reinforced concrete elements is determined by calculation. For reinforced concrete bending elements and eccentrically compressed elements of beam and frame components, it is recommended that the reinforcement percentage be taken with consideration of the value $\alpha=\mu \frac{R_a^y}{R_u^y}$, set at α = 0.25-0.35 for monolithic components and α = 0.3-0.4 for the precast components.

For Paragraph 3.41. A nomogram (see Appendix 6) may be used to determine the optimum percentage of reinforcement for bending precast and precast-monolithic reinforced concrete components.

3.42. In bending reinforced concrete elements with effective longitudinal reinforcement only in the tensile zone, longitudinal reinforcement is placed in the compression zone in an amount no less than $F'_a=0{,}0015bh_0$.

3.43. The natural oscillation frequencies ω of components are determined from reference data. A frequency is taken corresponding to the form of oscillations which most closely coincide with the elastic line from a static load numerically equal to the dynamic load.

The rigidity of reinforced concrete elements is taken with consideration of crack expansion in the concrete's tensile zone, and the weight of soil fill is considered for overhead cover.

For paragraphs 3.42 and 3.43. The total (own and associated) masses and flexural rigidities of component elements must be calculated and appropriate design arrangements chosen for a determination of natural oscillation frequencies.

The total mass of an element is determined by dividing all static loads (distributed and concentrated) having weight and actually acting on it by the acceleration of gravity. Static loads without weight (reactions of springs, pressure of gases, frictional forces and so on) are not considered in determining masses. Dynamic loads have no effect on natural oscillation frequencies and are not taken into account in determining them. Of the static service weight loads, only the most probable and long-acting are considered (weight of equipment, raw materials, finished products, embankment by a layer up to 1 m and so on). When the cover is banked by a layer of soil of more than 1 m, the mass of soil is not considered. Random and momentary static loads (episodic crowds of people in a production space, repair loads and so on) are not considered.

The distribution of masses over an element of the overhead cover is taken in conformity with the actual diagram of the transfer of static loads to the element.

Cyclic natural oscillation frequencies ω for the most frequently encountered shelter components may be determined from the following formulas:

For single-span and multispan continuous beams with equal spans

$$\omega=\frac{\alpha^2}{l^2}\sqrt{\frac{B}{m}}, \qquad (71)$$

where B is the rigidity at the center of the beam's span;
m is the linear mass of the beam determined from the formula

$$m=\frac{q_{ct}}{g}; \qquad (72)$$

q_{ct} is the sum of linear permanent and temporary long-acting loads;
g is the acceleration of gravity;

α^2 is the square of the frequency factor equal to:

a. For single-span beams:

With two hinged-bearing ends -- 9.87;
With one hinged and a second fixed end -- 15.42;
With two fixed ends -- 22.37;

b. For continuous equal-span beams with end swing supports;

With two spans -- 15.4;
With three spans -- 18.5;
With four spans -- 19.9;
With five or more spans -- 20.7;

For single-span slabs hinged-bearing around the perimeter: $\omega = 9{,}87\left(\frac{1}{l_1^2}+\frac{1}{l_2^2}\right)\sqrt{\frac{D}{m}}$, (73)

where l_1 and l_2 are the sides of a rectangular slab;
D is the cylindrical rigidity of the slab determined from the formula

$$D = \frac{E_б h^3}{12(1-\nu^2)} = \frac{E_б h^3}{11{,}6}, \quad (74)$$

$E_б$ is the initial modulus of elasticity of the concrete;
h is the slab height;
ν is the Poisson's ratio;
m is the mass per unit area of the slab.

For single-span rectangular slabs fixed along the perimeter:

$$\omega = \frac{22{,}37}{l_1^2}\sqrt{1+0{,}605\frac{l_1^2}{l_2^2}+\frac{l_1^4}{l_2^4}}\sqrt{\frac{D}{m}}. \quad (75)$$

l_1 and l_2 are the sides of a rectangular slab, and $l_1 > l_2$.

The natural oscillation frequencies of components not included in the above list may be determined according to available reference data (see "Instructions for Calculating Overhead Cover for Transient Loads," Moscow, Stroyizdat, 1966; "Theoretical Design Reference for the Designer," Moscow, Stroyizdat, 1960).

3.44. The rigidity of bending reinforced concrete elements with consideration of crack expansion in the concrete's tensile zone may be determined from Paragraph 9.7 of SNiP Chapter II-B.1-62* from the formula

$$B = 0{,}8 E_a F_a (h_0 - x)\left(h_0 - \frac{x}{2}\right), \quad [76(12)]$$

where E_a is the reinforcement's modulus of elasticity;
x is the height of the concrete's compression zone determined from formula 64(8).

For Paragraph 3.44. In substituting in the formulas of Paragraph 9.7 of SNiP II-B.1-62* the values corresponding to the momentary effect of a load, the following expression is obtained for determining rigidity of rectangular and T sections with a flange in the compression zone and with $x \leqslant h_п$;

$$B = E_a F_a h_0^2 \frac{1-0{,}5\xi}{1+\frac{1{,}8n\mu}{\xi}}, \quad (77)$$

where E_a is the reinforcement's modulus of elasticity;
F_a is the area of the stretched reinforcement;

h_o is the effective height of the cross section;
μ is the coefficient of reinforcement;
$n = \frac{E_a}{E_б}$ is the ratio of the initial moduli of elasticity of the reinforcement and concrete;
and
ζ is the relative height of the compression zone of concrete in the section with the crack;

$$\xi = \frac{1}{1,8 + \frac{1}{10n\mu}}. \tag{78}$$

The value of the reinforcement factor entering into the formulas for T-sections is equal to:

$$\mu = \frac{F_a}{b_п h_0}, \tag{79}$$

where $b_п$ is the effective width of the T-beam flange.

Table 22 provides the values of coefficients for calculating rigidity from formula (77) with different classes of reinforced steel and grades of concrete depending on reinforcement percentage. The table uses the notation

$$\beta = \frac{1 - 0,5\xi}{1 + \frac{1,8n\mu}{\xi}}.$$

3.45. Figuring bending and eccentrically compressed elements with consideration of plastic deformation of the reinforcement (Case 1a), it is necessary to ensure observance of the condition which precludes the possibility of the reinforcement fracturing in the calculation sections at the moment the component attains the calculated limiting state:

$$\varepsilon_a \leqslant 0,6\varepsilon_{пр}, \tag{80(13)}$$

where ε_a is the relative strain of stretched reinforcement corresponding to the calculated limiting state;
$\varepsilon_{пр}$ is the general relative elongation with fracture of the reinforcement; taken to be equal to δ_5 in accordance with Table 11.

For bending and eccentrically compressed elements, the value ε_a is determined from the formula

$$\varepsilon_a = \varepsilon'_a \frac{k_1}{k'}, \tag{81(14)}$$

where ε'_a and k' are values corresponding to the beginning of fracture of the concrete's compression zone taken from Table 13.

3.46. When using prestressed components, the calculated breaking moment must be greater than the moment of crack formation in the component, but no less than by 25 percent. Prestressed components in which the reinforcement has no cohesion with the concrete are not authorized for use in shelters.

For paragraphs 3.45 and 3.46. In condition 80(13), 0.6 is the assurance factor for a greater guarantee against reinforcement fracture, and it should be used for important civil defense structures. To obtain economic decisions in mass civil defense structures, it is advisable to require observance of the condition

$$\varepsilon'_a < \varepsilon_{пр}, \tag{82}$$

Table 22 - Coefficient Values for Calculating Rigidity of Bending Elements of Continuous Rectangular Sections and a Section with Rectangular Compression Zone

μ, %	Concrete Grade 200		300		400		500		600		μ, %
	1—0,5 ξ	β	1—0,5 ξ	β	1—0,5 ξ	β	1—0,5 ξ	β	1—0,5 ξ	β	
1	2	3	4	5	6	7	8	9	10	11	12
a) Reinforcement of Class A-I steels											
0,5	0,885	0,676	0,896	0,695	0,903	0,707	0,908	0,715	0,914	0,725	0,5
0,6	0,872	0,654	0,883	0,674	0,891	0,687	0,896	0,697	0,903	0,707	0,6
0,7	0,861	0,634	0,873	0,655	0,880	0,669	0,886	0,679	0,893	0,690	0,7
0,8	0,852	0,615	0,864	0,638	0,871	0,652	0,877	0,663	0,884	0,675	0,8
0,9	0,844	0,598	0,855	0,622	0,863	0,637	0,869	0,648	0,876	0,660	0,9
1	0,837	0,538	0,848	0,607	0,856	0,623	0,862	0,635	0,868	0,647	1
1,1	0,831	0,568	0,842	0,593	0,849	0,609	0,855	0,621	0,862	0,635	1,1
1,2	0,824	0,555	0,836	0,580	0,843	0,597	0,849	0,609	0,856	0,623	1,2
1,3	0,820	0,542	0,830	0,568	0,838	0,585	0,844	0,598	0,850	0,611	1,3
1,4	0,815	0,530	0,826	0,556	0,833	0,573	0,839	0,587	0,845	0,601	1,4
1,5			0,821	0,555	0,828	0,563	0,834	0,576	0,840	0,591	1,5
1,6			0,817	0,535	0,824	0,553	0,830	0,566	0,836	0,581	1,6
1,7			0,813	0,525	0,820	0,543	0,826	0,557	0,832	0,572	1,7
1,8			0,810	0,516	0,817	0,534	0,822	0,548	0,828	0,563	1,8
1,9			0,807	0,507	0,813	0,525	0,819	0,539	0,825	0,554	1,9
2			0,804	0,498	0,810	0,516	0,815	0,531	0,821	0,546	2
2,1			0,801	0,490	0,807	0,508	0,812	0,523	0,818	0,538	2,1
2,2			0,798	0,482	0,805	0,500	0,810	0,515	0,815	0,531	2,2
2,3			0,796	0,474	0,802	0,493	0,807	0,508	0,813	0,523	2,3
2,4					0,800	0,486	0,805	0,500	0,810	0,516	2,4
2,5					0,797	0,479	0,802	0,493	0,808	0,510	2,5
2,6							0,800	0,487	0,805	0,503	2,6
2,7							0,798	0,480	0,803	0,497	2,7
2,8							0,796	0,474	0,801	0,490	2,8
2,9							0,794	0,468	0,800	0,484	2,9
3							0,792	0,462	—	—	3
b. Reinforcement of Class A-II steels											
0,5	0,885	0,676	0,896	0,695	0,903	0,707	0,908	0,715	0,914	0,725	0,5
0,6	0,872	0,654	0,883	0,674	0,891	0,687	0,896	0,697	0,903	0,707	0,6
0,7	0,861	0,634	0,873	0,655	0,880	0,668	0,886	0,679	0,893	0,690	0,7
0,8	0,852	0,615	0,864	0,638	0,871	0,652	0,877	0,663	0,884	0,675	0,8
0,9	0,844	0,598	0,855	0,622	0,863	0,637	0,869	0,648	0,876	0,660	0,9
1	0,837	0,583	0,848	0,607	0,856	0,623	0,862	0,635	0,868	0,647	1
1,1	0,831	0,568	0,842	0,593	0,849	0,609	0,855	0,621	0,862	0,635	1,1
1,2			0,836	0,580	0,843	0,597	0,849	0,609	0,856	0,623	1,2
1,3			0,830	0,568	0,838	0,585	0,844	0,598	0,850	0,611	1,3
1,4			0,826	0,556	0,833	0,573	0,839	0,587	0,845	0,601	1,4
1,5			0,821	0,545	0,828	0,563	0,834	0,576	0,840	0,591	1,5
1,6			0,817	0,535	0,824	0,553	0,830	0,566	0,836	0,581	1,6

Table 22 - Coefficient Values for Calculating Rigidity of Bending Elements of Continuous Rectangular Sections and a Section with Rectangular Compression Zone [Continued]

μ, %	Concrete Grade 200		300		400		500		600		μ, %
	1—0,5 ξ	β	1—0,5 ξ	β	1—0,5 ξ	β	1—0,5 ξ	β	1—0,5 ξ	β	
1	2	3	4	5	6	7	8	9	10	11	12
1,7			0,813	0,525	0,820	0,543	0,826	0,557	0,832	0,572	1,7
1,8			0,810	0,516	0,817	0,534	0,822	0,548	0,828	0,563	1,8
1,9					0,813	0,525	0,819	0,539	0,825	0,554	1,9
2							0,816	0,531	0,821	0,546	2
2,1							0,812	0,523	0,818	0,538	2,1
2,2							0,810	0,515	0,815	0,531	2,2
2,3							0,807	0,508	—	—	2,3
c. Reinforcement of Class A-III steels											
0,5	0,888	0,682	0,899	0,702	0,906	0,712	0,910	0,719	0,914	0,725	0,5
0,6	0,876	0,660	0,888	0,681	0,894	0,693	0,899	0,701	0,903	0,707	0,6
0,7	0,865	0,641	0,877	0,663	0,884	0,675	0,889	0,684	0,893	0,690	0,7
0,8	0,856	0,623	0,868	0,646	0,875	0,659	0,880	0,668	0,884	0,675	0,8
0,9			0,860	0,630	0,867	0,644	0,872	0,653	0,876	0,660	0,9
1			0,852	0,616	0,859	0,630	0,864	0,639	0,868	0,647	1
1,1			0,846	0,602	0,853	0,616	0,858	0,627	0,862	0,635	1,1
1,2			0,840	0,589	0,847	0,604	0,852	0,614	0,856	0,623	1,2
1,3			0,834	0,577	0,841	0,592	0,846	0,603	0,850	0,611	1,3
1,4			0,830	0,566	0,836	0,581	0,841	0,592	0,845	0,601	1,4
1,5					0,832	0,571	0,836	0,582	0,840	0,591	1,5
1,6							0,832	0,572	0,836	0,581	1,6
1,7							0,828	0,536	0,832	0,572	1,7
1,8							0,824	0,554	—	—	1,8

where ε_a' is relative deformation in percent of the stretched reinforcement, corresponding to the beginning of fracture of the concrete's compression zone, determined from Table 23(13) depending on the ratio S_0^y/S_0.

Condition (82) can be worded in the following manner: To prevent reinforcement fracture before beginning of fracture of the concrete's compression zone in bending and eccentrically compressed elements of reinforced concrete components with great eccentricities, calculated for Case 1a, reinforced steels should be used with relative elongation ($\varepsilon_{\text{пр}}$, %) under GOST 5781-61 standards (see SNiP I-B.4-62 "Reinforcement for Reinforced Concrete Components") which must be no less than the value ε_a' .determined from Table 23 (13).

3.47. For eccentrically compressed elements of masonry components, the determination of the design case (large or small eccentricity) must be performed in accordance with SNiP Chapter II-B.2-71 "Masonry and Reinforced Masonry Components: Design Standards."

The calculation of masonry walls is performed without a check of the tensile zone for crack expansion. The greatest value of eccentricity e_0 in the calculation for bearing power must satisfy the condition:

Table 23(13) - Factors ε'_a and k'

Ratio of Static Moment of Cross Section S_0^y/S_0	0,50	0,35	0,25	0,20	0,15	0,10
Factor ε'_a	4	4	5	6	7	10
Factor k'	3	4	6	8	10	12

Note: For intermediate values S_0^y/S_0 the values for ε'_a and k' are determined by interpolation.

In the calculation for Case 1a $e_0 \leqslant 0,95y$;

In the calculation for Case 1b $e_0 \leqslant 0,8y$,

where y is the distance from the element section's center of gravity to the edge of the section toward the eccentricity.

3.48. The calculation of foundation spaces is performed in accordance with requirements of the SNiP chapter for designing foundations of buildings and structures with consideration of dynamic hardening of soils in accordance with paragraphs 3.27 and 3.28 of these Instructions.

The calculation of pile foundations is performed in accordance with requirements of the SNiP chapter on designing pile foundations: for deformations -- for the basic combination of standardized loads; for bearing power -- for the main and supplementary or special combination of design loads arising during peacetime operation of buildings and structures which have basements in which there are built-in shelters.

In the calculation for a special load combination (with consideration of the shock wave effects), bearing power P, tc, of pile foundations of floating piles designed in the form of bands or groups of piles or a pile field is determined from the formula

$$P = kmR_y^H\left(F + \mu n\Sigma l_i \sqrt{\frac{1-k_{\delta i}}{2}}\right) \qquad [83(14a)]$$

where R_y^H is the allowable stress of soil (ton-force/m^2) under the foundation mat with consideration for dynamic hardening of soils in conformity with Paragraph 3.27 of Instructions of SN 405-70;

F is the area of the foundation mat/distribution plate in square meters;

u is the perimeter of pile cross section in meters;

n is the number of piles;

l_i is the thickness of the i-th layer of soil coming in contact with the lateral surface of the pile in meters;

$k_{\delta i}$ is the lateral pressure coefficient of the i-th layer of soil taken from Table 3 of these Instructions;

m is the work condition factor, taken to be m = 1;

k is the coefficient of soil uniformity, taken to be k = 0.7.

The bearing power of pile foundations of column piles is determined in accordance with requirements of the SNiP chapter on designing pile foundations, with consideration of dynamic hardening of the base in accordance with paragraphs 3.27 and 3.28 of these Instructions.

For paragraphs 3.47 and 3.48. Pile foundations for shelters can be designed in the form of bands under walls with the arrangement of piles into one, two or more rows; or in the form of groups under columns. Piles are joined above by a pilework slab. A varient is possible in which a solid foundation plate (foundation mat) is laid under the entire structure with piles arranged in places where the vertical load from walls and columns is applied.

Formula [83(14a)] is used to determine bearing power of pile foundations of floating piles in position with a foundation mat on the soil, where the foundation mat can be used to transmit a load directly to the soil. Here the bearing power of the pilework accumulates from resistance of the base soil under the pilework footing which arises as a result of the vertical displacement of the entire structure under the effect of a shock wave load, and as a result of the shearing strength of piles from the tangential stresses over the lateral surface.

The pilework area in formula [83(14a)] is determined from outer dimensions in the plan view, i.e., it also includes the area of the piles' cross sections.

In calculating foundations for the effect of a momentary load, it is usually assumed that stresses in the soil base under the foundation footing are proportionate to the rate of displacement z of the foundation plate

$$\sigma = \dot{z}\rho a_1, \tag{84}$$

where a_1 is the rate of propagation of elasticoplastic strains (waves) in the soil; ρ is the density of the base soil.

When the pilework plate attains a certain rate of displacement, stresses in the soil will reach the maximum permissible limits

$$R_y^{\text{н}} = \dot{z}_n \rho a_1, \tag{85}$$

where z_Π is the limiting rate of displacement; R^H_y is the soil resistance under the pilework footing determined in conformity with Paragraph 3.27.

In examining the joint performance of pilework on soil and floating piles, the reactance under the pilework will be determined as the sum of resistances of the foundation mat and piles

$$P = \rho a_1 \dot{z}_n F + \rho b \dot{z}_n S, \tag{86}$$

where b is the propagation rate of transverse waves determined from the formula

$$b = a_1 \sqrt{\frac{1-2\mu}{2(1-\mu)}} = a_1 \sqrt{\frac{1-k_б}{2}}, \tag{87}$$

μ is Poisson's ratio for soil;
$k_б$ is the coefficient of lateral pressure;
F is the foundation mat area per pile;
S is the area of a pile's lateral surface.

The second term in the right part of expression (86) is the pile's shearing strength from tangential stresses over the lateral surface. For a single pile we will obtain from expression (86), with consideration of (85) and (87):

$$P = R_y^{\text{н}} F \left(1 + \frac{S}{F} \sqrt{\frac{1 - k_6}{2}}\right). \tag{88}$$

Thus, consideration of the presence of a pile is equivalent to having the magnitude of permissible stresses in the base increase $\left(1 + \frac{S}{F} \sqrt{\frac{1-k_6}{2}}\right)$ times.

The coefficient of lateral pressure for waterlogged soils (below ground water level) should be taken equal to k_6 = 0.9 in calculating pile foundations. In Table 3 of SN 405-70 the value k_6 for waterlogged soils is assumed equal to unity in order to create a certain margin in determining the horizontal load on walls.

When it is impossible to use the foundation mat for transmitting a load directly to the soil, the bearing power of a pile arrangement should be determined with consideration of soil resistance beneath the lower ends of floating piles in position from the formula

$$P = kmn \left(R^{\text{н}} F_{\text{св}} + R_y^{\text{н}} u \Sigma l_i \sqrt{\frac{1 - k_{6i}}{2}}\right), \tag{89}$$

where R^H is the allowable stress of soil beneath the lower end of a pile, taken from SNiP II-B.5-67* "Pile Foundations: Design Standards";

F_{CB} is the area of a pile resting on the soil, taken from the area of the pile's cross section. The remaining notations are as in formula 83(14a).

4. METHODS OF DETERMINING DYNAMIC-RESPONSE FACTORS AND CALCULATION OF ELEMENTS OF OVERHEAD COVER

Calculation of Hinged-Bearing Beam for First Limiting State (Case 1b)

4.1. The dynamic-response factors for stresses and displacement in formulas (42)-(44) equal:

When calculating components without consideration of the effect of the rate of deformation on strength properties of steels

$$k_{м} = k_Q = k_{п} = k_{д,1}; \qquad (90)$$

When calculating components with consideration of the effect of rate of deformation

$$k_{м} = k_Q = k_{п} = k_{д,2}. \qquad (91)$$

In formulas (90)-(91) the dynamic-response factors $k_{д,1}$ and $k_{д,2}$ depend on the character of change in the dynamic load in time and on the component's natural oscillation frequency, and they are determined from paragraphs 4.2-4.5.

Fulfillment of the strength condition of bending and eccentrically-compressed elements of overhead cover (Case 1) under the effect of static and dynamic loads is checked from the formula

$$M_{экв} \leqslant \overline{M}_0,$$

$$\overline{M}_0 = M_0 - k_y M_c; \qquad (92)$$

where M_0 is the limiting moment of internal stresses in the most stressed section, determined from the calculated dynamic resistance of component material;

$M_{ЭКВ}$ is the bending moment from an equivalent static load;

M_C is the bending moment (with the same sign as $M_{ЭКВ}$) from the calculated static load;

k_y is the hardening factor determined in conformity with Paragraph 3.24.

4.2. For a dynamic load jumping to a maximum value and then reducing over time (see Fig. 61a), the dynamic response factors $k_{д,1}$ and $k_{д,2}$ are determined from the chart in Fig. 79 or are computed from the formulas:

With consideration of the effect of deformation rate (reinforced steels of classes A-I, A-II and A-III)

$$k_{д,2} = 1.915\left(1 - \frac{\operatorname{arctg}\omega\theta}{\omega\theta}\right), \quad \omega\theta \geqslant 5;$$

Without consideration of the effect of deformation rate (reinforced steels of classes A-IV and A-V, heat- and stretch-hardened)

$$k_{д,1} = 2\left(1 - \frac{\mathrm{arctg}\,\omega\theta}{\omega\theta}\right), \omega\theta \geqslant 2,33. \quad (94)$$

Here ω is the beam's natural oscillation frequency, $seconds^{-1}$, determined from formulas of Paragraph 3.43;

θ is the effective action time of dynamic loads in seconds.

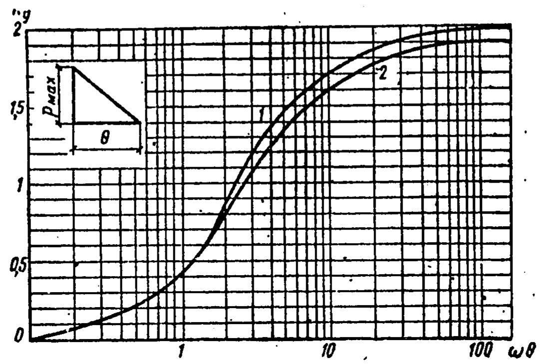


Fig. 79. Chart for determining dynamic-response factor in calculating components for Case 1b

1. Curve for determining dynamic-response factor $k_Д$ without considering the effect of deformation rate on strength properties of reinforced steels
2. The same, with consideration of the effect of deformation rate on strength properties of reinforced steels

4.3. For a dynamic load rising gradually to maximum value over time θ_1 and then reducing to zero over time θ_2 (Fig. 61d and e), the factors $k_{д,1}$ and $k_{д,2}$ are determined from curves of figures 80 and 81 depending on the values $\omega\theta_1$ and θ_2/θ_1.

With values $\omega\theta_1$ near to $2n\pi$ (where n is a positive integer), the curve for the change in dynamic-response factor has minimums shown by the broken line in the chart in Fig. 80.

To avoid understating the dynamic-response factor in connection with the fact that values ω and θ_1 are determined approximately, the curves in the charts of figures 80 and 81 are plotted for maximum values of the dynamic-response factor.

When $\omega\theta_1 = 20$, the values of dynamic-response factors are assumed equal to $k_Д = 1.1$, and by interpolation in the interval $10<\omega\theta_1<20$.

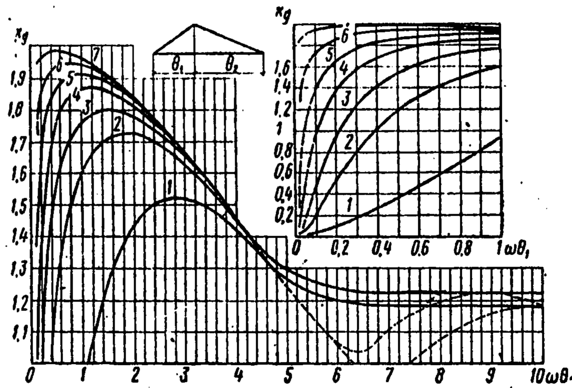


Fig. 80. Chart of dynamic-response factor $k_{д,1}$ values for an increasing load in calculating components for Case 1b without consideration of the effect of deformation rate on strength properties of reinforced steels (the curves and values of θ_2/θ_1 respectively: 1--1; 2--5; 3--10; 4--20; 5--40; 6--100; 7--⩾200)

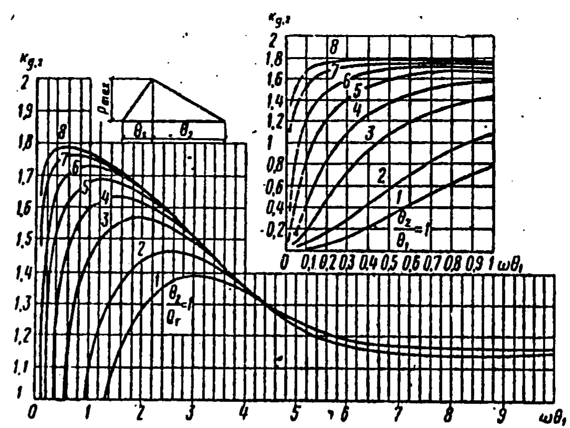


Fig. 81. Chart of dynamic-response factor $k_{д,2}$ values for increasing load when calculating components for Case 1b with consideration of the effect of deformation rate on strength properties of reinforced steels (the curves and values θ_2/θ_1 respectively: 1--1; 2--2; 3--5; 4--10; 5--20; 6--40; 7--100; 8--⩾200)

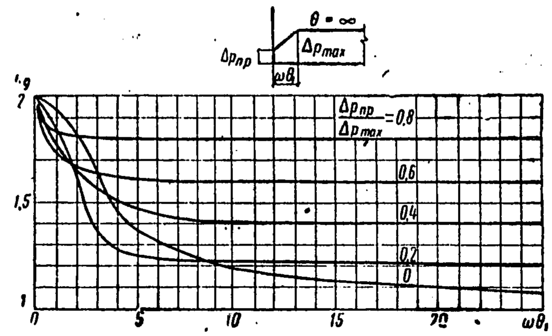


Fig. 82. Chart of dynamic-response factor values ($k_Д = k_{Д,1} = k_{Д,2}$) for a load increasing linearly from a shock $\Delta p_{пр}$ to maximum value, in calculating components for Case 1b

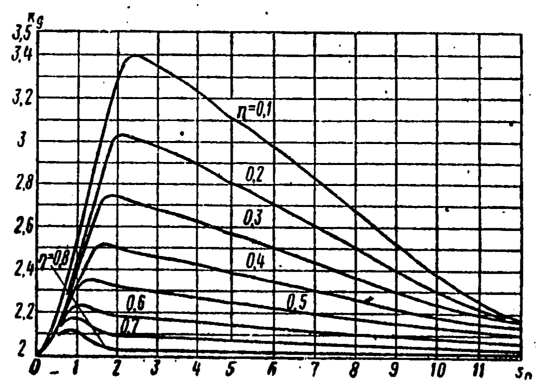


Fig. 83. Chart for determining dynamic-response factor ($k_Д = k_{Д,1} = k_{Д,2}$) in calculating overhead cover with ground embankment greater than 1 m for Case 1b

4.4. For a dynamic load increasing linearly from a shock ($\Delta p_{пр}$) to maximum value over time θ_1 (see Fig. 61b), the dynamic-response factor $k_Д=k_{Д,1}=k_{Д,2}$ is determined from the curves of Fig. 82 plotted for $\theta = \infty$, which ensures a margin in determining $k_Д$.

4.5. The dynamic-response factor $k_Д=k_{Д,1}=k_{Д,2}$ for overhead covers with ground fill greater than 1 m thick is determined from the chart of Fig. 83 depending on the dimensionless parameters

$$S_n = \frac{\omega H}{a_1};\qquad (95)$$

$$\eta = \frac{\rho a_1}{2m_1\omega},\qquad (96)$$

where H is the thickness of the soil layer above the overhead cover;
a_1 is the rate of propagation of elasticoplastic deformations in soil (see Table 9);
ρ is the density of embankment soil;
m_1 is the component mass per unit area of loaded surface;
ω is the component's oscillation frequency determined without consideration of soil mass above the cover.

In this case the dynamic-response factor considers the effect of reflection of the compression wave from the overhead cover and the free surface of the ground in addition to the effect of the load's dynamic action. In constructing the chart of Fig. 83, the reflection factor is assumed equal to two. The equivalent static load on the overhead cover in this case is determined from the formula

$$q_{экв} = \Delta p_\Phi k_{зат} k_д,\qquad (97)$$

where Δp_Φ is the pressure on the surface of soil embankment;
$k_{зат}$ is the attenuation factor from formula (22).

With $S_n > 0{,}25$ the change in the reflection factor depending on the magnitude of pressure (see Fig. 63) can be considered approximately by multiplying the magnitude of pressure Δp_Φ in formula (97) by the ratio $k^*_{отр}/2$, where $k^*_{отр}$ is determined from the chart of Fig. 63.

The attenuation factor $k_{зат}$ from formula (22) is best determined for a short time of the shock wave's effect on the soil's surface (for example, from the explosion of GVS).

4.6. The limiting value of the dynamic load for a beam with known characteristics of cross section is determined from the formula

$$p_{max} = \frac{8(M_0 - k_y M_c)}{b l^2 k_м},\qquad (98)$$

where b is the width of the loaded zone;
M_0 is the limiting moment of internal stresses;
M_c is the bending moment from static load;
k_y is the hardening factor of reinforced steel;
l is the effective span; and
k_m is the dynamic-response factor for the bending moment.

Calculation of Hinged-Bearing Beam for First Limiting State (Case 1a)

4.7. Calculation of a reinforced concrete hinged-bearing beam for bend is performed from the condition

$$\psi = \frac{\bar{p} l^3}{19{,}2\,B} k_n \leqslant \psi_{max},\qquad (99)$$

where ψ is the opening angle in the hinge of plasticity in the middle of the beam's span;
ψ_{max} is the maximum permissible opening angle in the hinge of plasticity, established in conformity with Paragraph 3.32;

l is the effective span of the beam;
B is the beam's rigidity at mid-span (see Paragraph 3.44);
$\bar{p} = bp_{max}$ is the width of the loaded zone;
p_{max} is the maximum value of dynamic load; and
k_{Π} is the dynamic-response factor for displacements determined from the charts of figures 84-87 depending on the cyclic natural oscillation frequency of the component ω, values characterizing the law of a load's change over time (shown in the charts), and the dynamic-response factor for bending moment, the value of which is computed from the formula

$$k_M = \frac{M_0 - k_y M_c}{M_p};\ M_p = \frac{\bar{p}l^2}{8}. \quad (100)$$

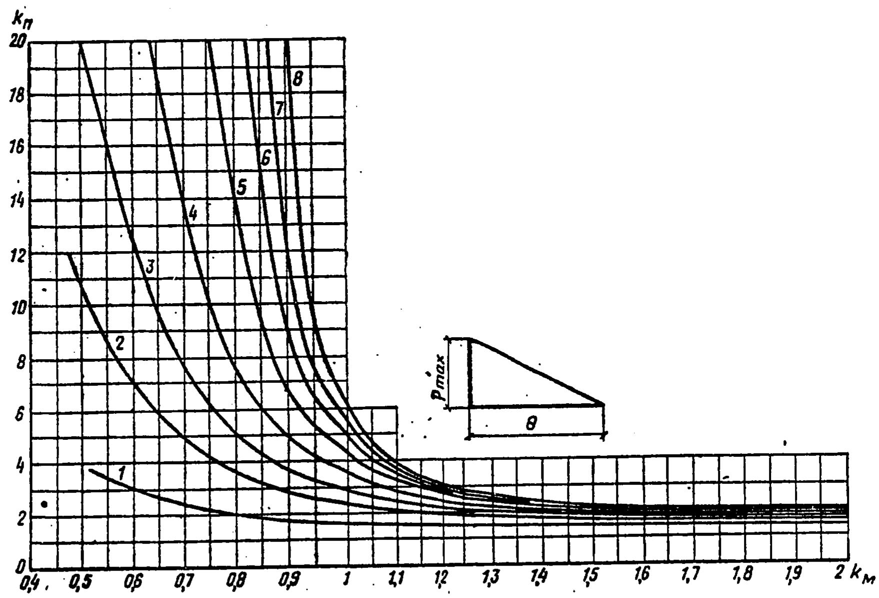


Fig. 84. Chart of the relationship of the dynamic-response factor for displacements k_{Π} to the dynamic-response factor for bending moment k_m in calculating hinged-bearing beams for Case 1a with consideration of the effect of deformation rate on strength properties of reinforced steels (the curves and values ωθ respectively: 1--5; 2--10; 3--15; 4--25; 5--50; 6--100; 7--200; 8--≥300)

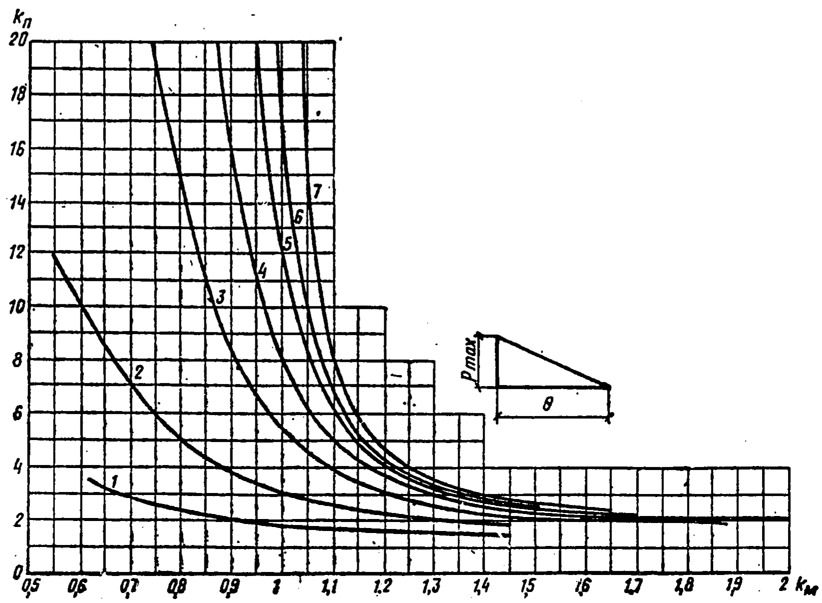


Fig. 85. Chart of the relationship of the dynamic-response factor for displacements k_Π to the dynamic-response factor for bending moment k_m in calculating hinged-bearing beams for Case 1a without consideration of the effect of deformation rate on strength properties of reinforced steels (the curves and values of $\omega\theta$ respectively: 1--5; 2--10; 3--25; 4--50; 5--100; 6--200; 7--$\geqslant$300)

In figures 86 and 87 that part of the curve drawn as a continuous line corresponds to the beam's performance in the plastic stage, and the broken line is its performance in the elastic stage.

With $\omega\theta_1 > 2\pi$ and $\omega\theta_2 \geqslant 100-200$ it is best to calculate components for a load with a build-up only in the elastic stage (Case 1b of the limiting state), since a consideration of plastic deformations produces no economic effect.

The dynamic-response factor for transverse force k_Q is assumed equal:

For a load with a jump (see Fig. 61a and c) $k_Q \approx 0,04 + 1,1\, k_M$;

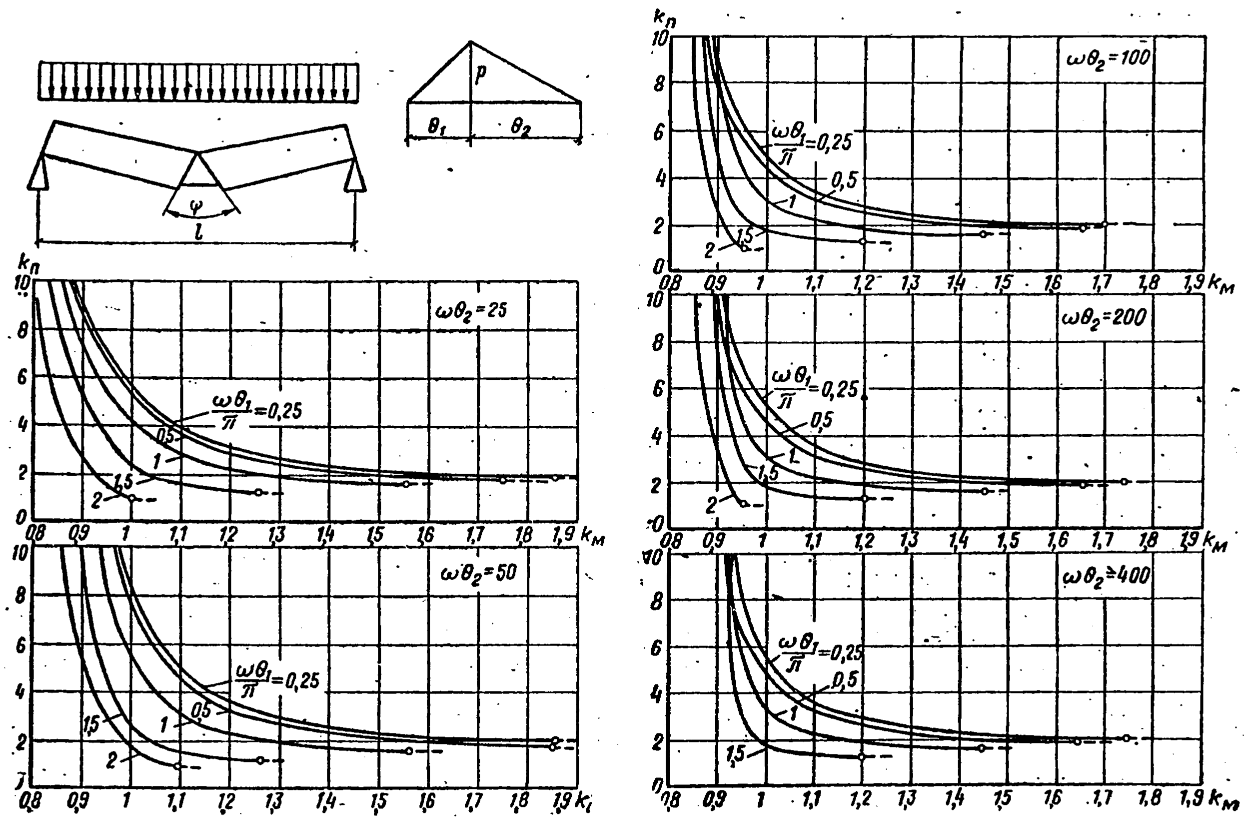


Fig. 86. Charts of the relationship of the dynamic-response factor for displacement k_Π to the dynamic-response factor for bending moment k_m in calculating hinged-bearing beams for Case 1a for a load with a build-up without consideration of the effect of deformation rate on strength properties of reinforced steels.

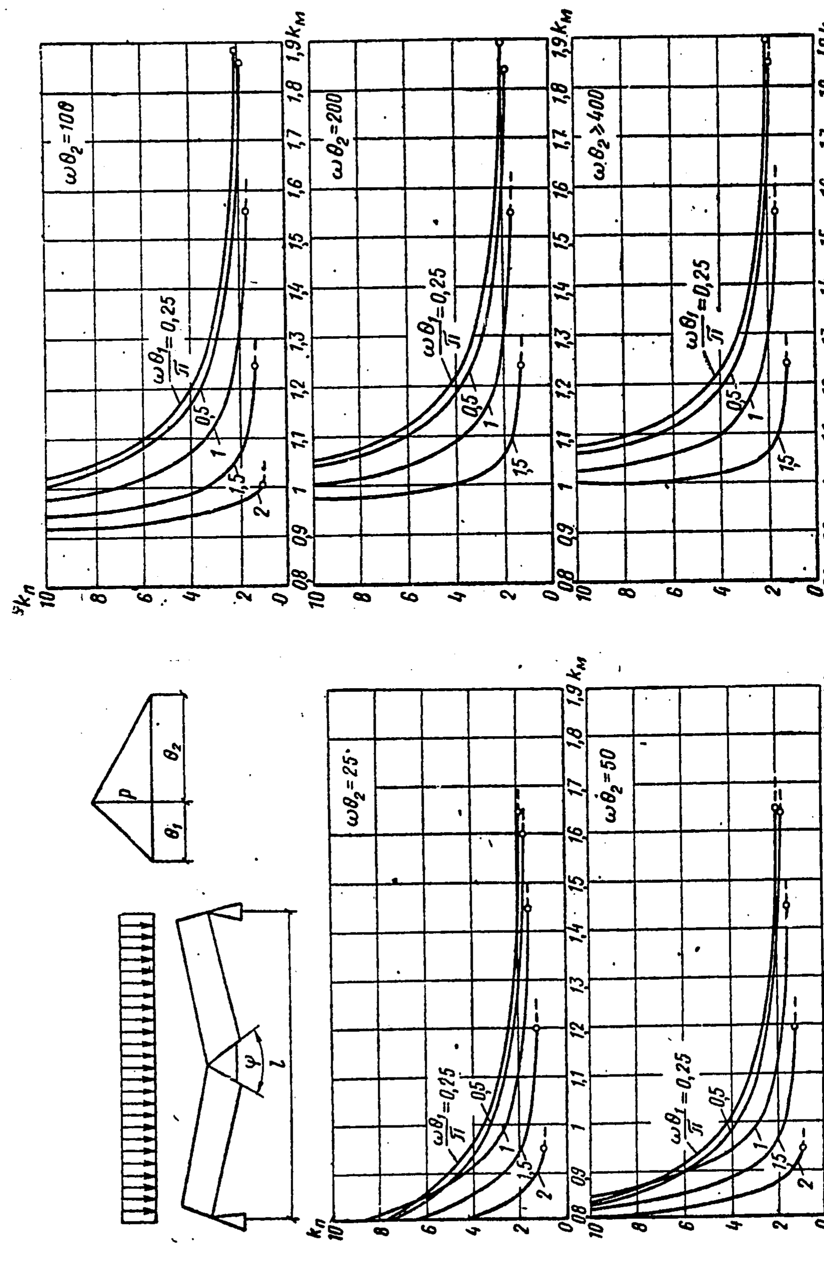


Fig. 87. Charts of the relationship of dynamic-response factor for displacement k_Π to the dynamic-response factor for bending moment k_m in calculating hinged-bearing beams for Case 1a for a load with build-up and with consideration of the effect of deformation rate on strength properties of reinforced steels

For a load with a build-up (see Fig. 61d and e)

$$k_Q \approx 1,1\, k_м.$$

4.8. For overhead cover with a soil embankment greater than 1 m thick, the dynamic-response factor for displacements in formula (99) is determined from the charts of figures 88-90 depending on the dimensionless parameters $S_п$ and η, calculated from formulas (95) and (96), and on the dynamic-response factor for bending moments computed from formula (100). The maximum value of the dynamic load p_{max} is assumed equal to

$$p_{max} = \Delta p_ф\, k_{зат} \frac{k^*_{отр}}{2}. \tag{101}$$

In determining M_c it is necessary to consider the weight of the soil embankment.

The dynamic-response factor for transverse force is assumed equal to

$$k_Q = 1,1\, k_м.$$

4.9. The limiting value of dynamic load for a beam with known characteristics of cross section is calculated from formula (98), in which the dynamic-response factor k_m is determined from the charts of figures 91-93 depending on the cyclic natural oscillation frequency of the component, the values characterizing the law of the load's change over time, and parameter z, found from the formula

$$z = \frac{2,4\, B\psi_{max}}{(M_0 - k_y M_c)\, l}, \tag{102}$$

in which there are the very same notations as in formulas (98) and (99).

Since the action time of the dynamic load and its maximum value are functions of the distance from the center of the blast to the structure and of the yield (energy) of the burst, then the limiting value of the dynamic load in the first approximation should be determined with its action time $\theta = \infty$.

With the given yield of burst and the value of the load found in the first approximation, its action time is determined from known formulas[1] and the magnitude of the critical load is refined.

Calculation under Case 1b of Beam with Both Ends Fixed and of Frame Elements with Ends Fastened in Rigid Joints

4.10. The dynamic-response factors with consideration of expressions (90) and (91) are determined from the charts of figures 79-83 depending on the cyclic natural oscillation frequency of the component, the values characterizing the law of the load's change over time, and the dimensionless parameters $S_п$ and η, computed from the formulas (95) and (96) (for overhead cover with a ground embankment greater than 1 m).

1. See Ganushkin, V. I., V. I. Morozov, B. I. Nikonov and G. I. Orlov, "Prisposobleniye podvalov sushchestvuyushchikh zdaniy pod ubezhishcha" [Adapting Basements of Existing Buildings as Shelters], Moscow, Stroyizdat, 1971.

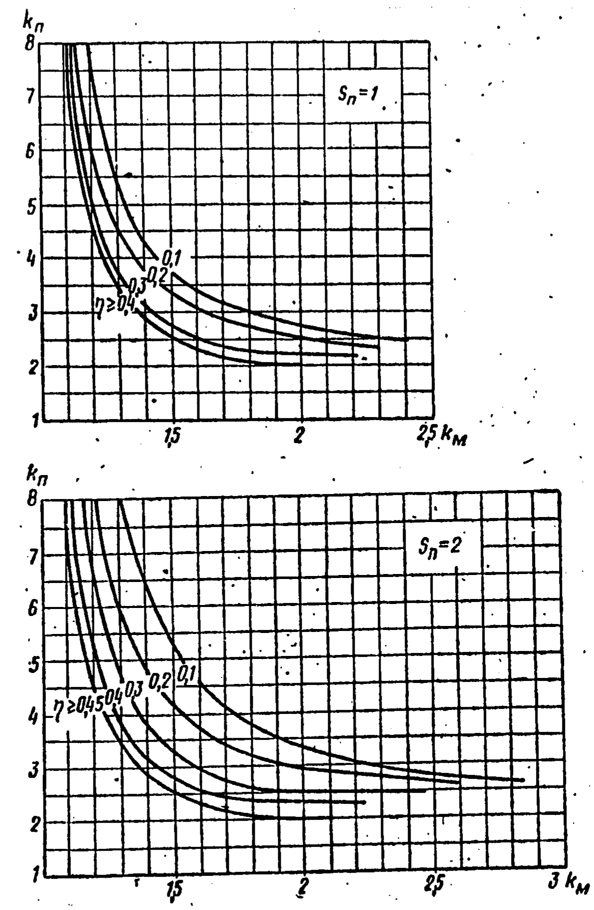


Fig. 88. Charts of the relationship of the dynamic-response factor for displacement $k_П$ to the dynamic-response factor for bending moment k_m in calculating overhead cover with soil embankment greater than 1 m for Case 1a (parameter $S_П$ = 1 and 2)

4.11. Strength conditions are checked for the bearing and span sections from the formulas:

$$\left.\begin{aligned} M^{оп}_{экв} &\leqslant M^{оп}_0 - k_y M^{оп}_c; \\ M^{пр}_{экв} &\leqslant M^{пр}_0 - k_y M^{пр}_c, \end{aligned}\right\} \qquad (103)$$

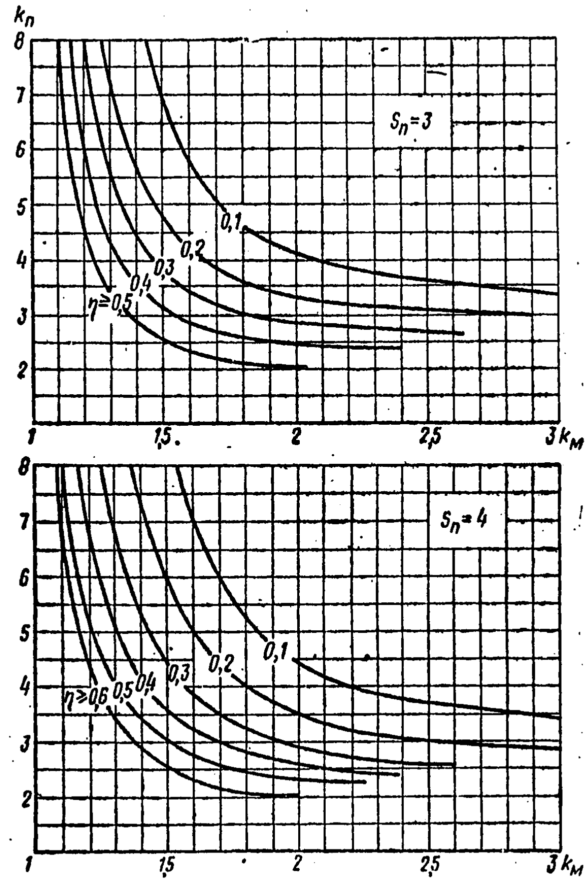


Fig. 89. Charts of the relationship of the dynamic-response factor for displacement $k_П$ to the dynamic-response factor for bending moment k_m in calculating overhead cover with a soil embankment greater than 1 m for Case 1a (parameter $S_П$ = 3 and 4)

where $M^{оп}_{экв}$, $M^{пр}_{экв}$ are the bending moments on the support and in the span from equivalent static load respectively;

$$M^{оп}_{экв} = \frac{\bar{q}^{(м)}_{экв}\, l^2}{12} k_1; \qquad (104)$$

$$M^{пр}_{экв} = \frac{\bar{q}^{(м)}_{экв}\, l^2}{24} (3 - 2k_1). \qquad (105)$$

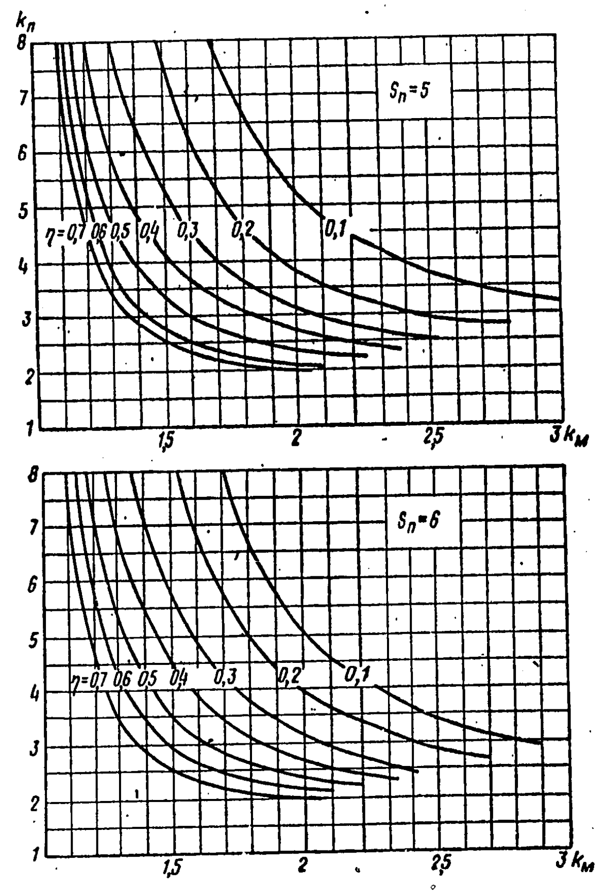


Fig. 90. Charts of the relationship of the dynamic-response factor for displacement k_{Π} to the dynamic-response factor for bending moment k_m in calculating overhead cover with a soil embankment greater than 1 m for Case 1a (parameter S_{Π} = 5 and 6)

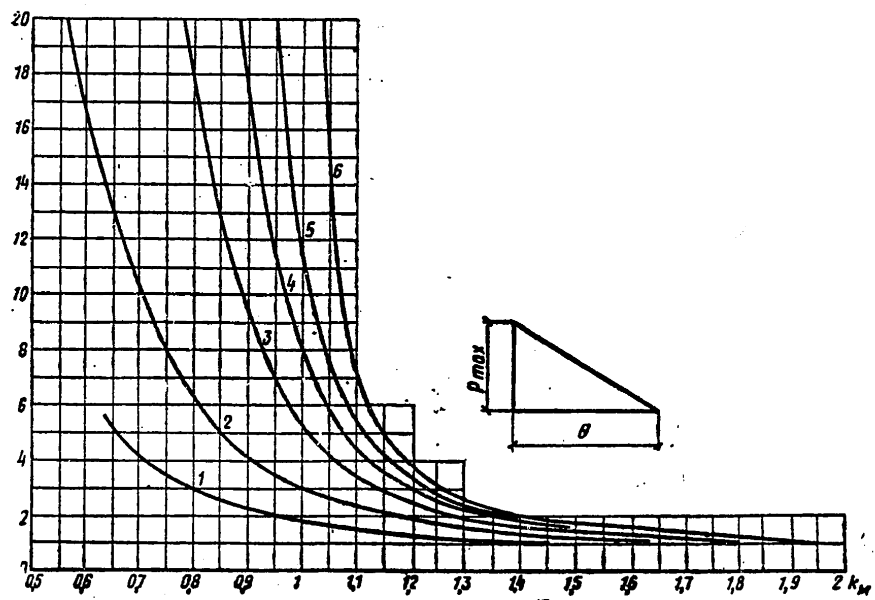


Fig. 91. Chart of the values of the dynamic-response factor for bending moment (without consideration of the hardening of reinforced steel) for the limiting value of a dynamic load increasing by a jump and then reducing linearly over time (the curves and values $\omega\theta$ respectively: 1--5; 2--10; 3--25; 4--50; 5--100; 6--$\geqslant$200)

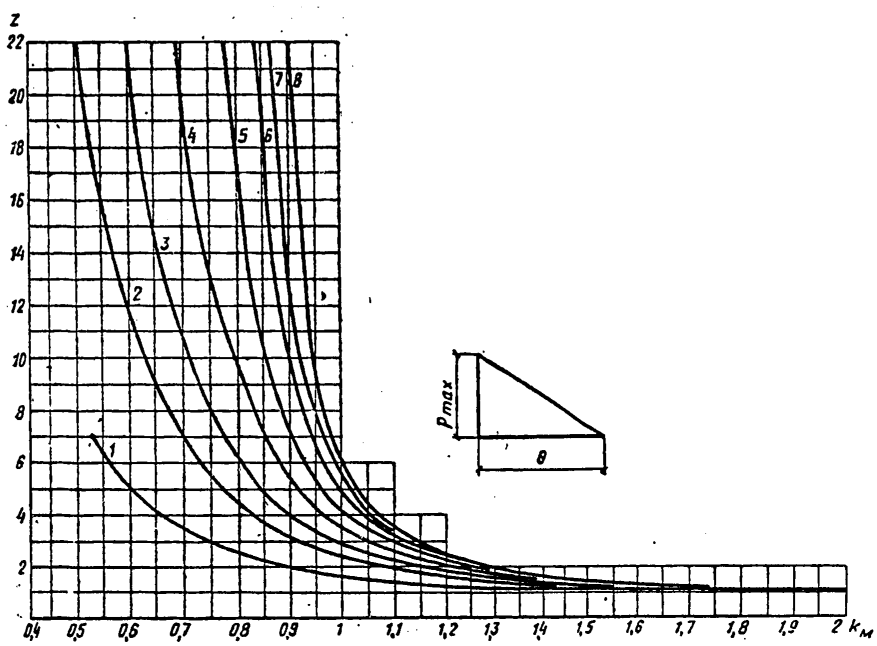


Fig. 92. Chart of values of the dynamic-response factor for bending moment (with consideration of hardening of reinforced steel) for the limiting value of a dynamic load increasing with a jump and then reducing linearly over time (the curves and values ωθ respectively: 1--5; 2--10; 3--15; 4--25; 5--50; 6--100; 7--200; 8--⩾300)

Here $\bar{q}^{(\text{м})}_{\text{экв}} = p_{\max} k_{\text{м}} b$, where b is the effective width of the beam's loading zone.

For overhead cover with the thickness of the soil embankment layer greater than 1 m, the value of p_{max} is determined with $S_{\text{п}} > 0{,}25$ from formula (101), and with $S_{\text{п}} \leqslant 0{,}25$ from formula $p_{max} = \Delta p_{\Phi} k_{\text{отр}}$.

Factor k_1 which considers the redistribution of values of bending moments in the span and on the support as a result of crack expansion in the concrete's tensile zone is determined from the formula

$$k_1 = \frac{0{,}269 + 0{,}731\,\beta_1}{0{,}46 + 0{,}54\,\beta_1}, \qquad (106)$$

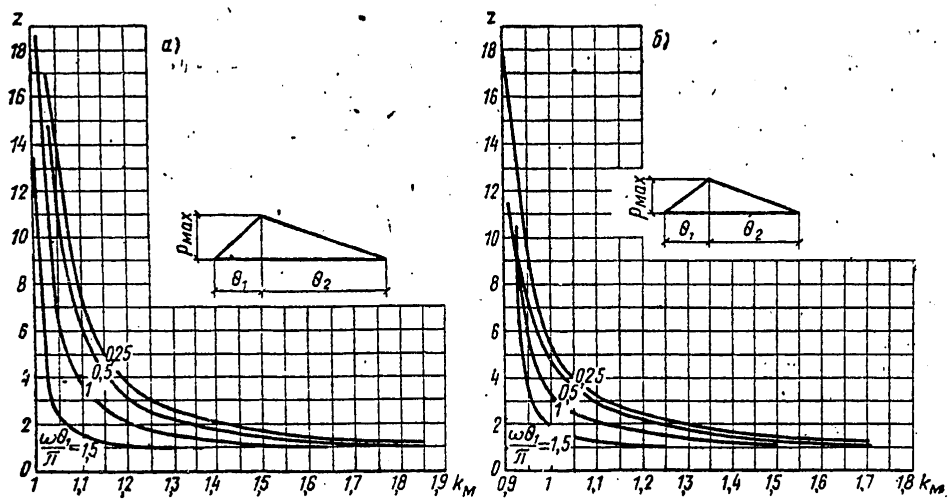


Fig. 93. Chart showing values of dynamic-response factor for bending moment for the limiting value of a dynamic load increasing linearly to maximum value

a. Without consideration of the effect of deformation rate on strength properties of reinforced steel
b. With consideration of the above

in which

$$\beta_1 = \frac{B^{оп}}{B^{пр}}, \tag{107}$$

where $B^{оп}$, $B^{пр}$ respectively are the beam's rigidity on the support and at mid-span, determined with consideration of crack expansion in the concrete's tensile zone (see Paragraph 3.44).

4.12. The limiting value of the dynamic load for a beam with known sectional characteristics equals the least of two values calculated from the formulas:

$$p_{max} = \frac{12\left(M_0^{оп} - k_y M_c^{оп}\right)}{bl^2 k_1 k_м}; \tag{108}$$

$$p_{max} = \frac{24\left(M_0^{пр} - k_y M_c^{пр}\right)}{bl^2 (3 - 2k_1) k_м}, \tag{109}$$

Calculation for Case 1a of a Beam with Both Ends Fixed and Frame Elements with Ends Fastened in Rigid Joints

4.13. Calculation of a reinforced concrete beam with fixed supports for flexure is performed:

For a support

$$\psi^{оп} = \frac{\bar{p} l^3}{192 B^{пр}} k_п^{оп} \leqslant 0,5\, \psi_п^{оп}; \tag{110}$$

For a span

$$\psi^{\text{np}} = \frac{\bar{p} l^3}{96 B^{\text{np}}} k_{\text{n}}^{\text{np}} \leqslant \psi_{\text{n}}^{\text{np}}, \tag{111}$$

where $\psi^{\text{on}}, \psi^{\text{np}}$ are expansion angles in the bearing and span hinges of plasticity respectively;

$\psi_{\text{n}}^{\text{on}}, \psi_{\text{n}}^{\text{np}}$ are limiting expansion angles in the same hinges of plasticity, established in accordance with Paragraph 3.32;

$k_{\text{n}}^{\text{on}}, k_{\text{n}}^{\text{np}}$ are the dynamic-response factors for displacements for the bearing and span respectively, determined from charts of figures 115-122, Appendix 7.

The calculation is performed in the following sequence.

We determine the beam's sectional rigidity at the bearing and in the span (see Paragraph 3.44), the natural oscillation frequency of components ω_1 (Paragraph 3.43) and the product $\omega_1\theta$. From formulas (106) and (107) we find the factors β_1 and k_1. We compute the factor β_2:

$$\beta_2 = \frac{M_0^{\text{on}} - k_y M_c^{\text{on}}}{M_0^{\text{np}} - k_y M_c^{\text{np}}}, \tag{112}$$

where $M_0^{\text{on}}, M_0^{\text{np}}$ are the limiting support and span bending moments of inner stresses [see formula (61)];

k_y is the dynamic hardening factor of reinforced steel;

$M_c^{\text{on}}, M_c^{\text{np}}$ are bending moments at the support and in the span from the static load, determined from formulas (104) and (105) respectively, substituting therein the values $q_c = \bar{q}_c b$ in place of $\bar{q}_{\text{экв}}^{(\text{м})}$.

Compute the dynamic-response factor for bending moment for the support from the formula

$$k_{\text{м}}^{\text{on}} = \frac{M_0^{\text{on}} - k_y M_c^{\text{on}}}{M_p^{\text{on}}}, \tag{113}$$

where M_p^{on} is the bending moment from dynamic load at the support, equal to

$$M_p^{\text{on}} = \frac{\bar{p} l^2}{12} k_1. \tag{114}$$

Based on the computed value $k_{\text{м}}^{\text{on}}$ with the help of charts of figures 115-122, Appendix 7, in relation to ω_1, θ, β_1 and β_2 we determine the dynamic-response factors for displacements $k_{\text{п}}^{\text{on}}$ (solid line on the charts) and $k_{\text{п}}^{\text{np}}$ (broken line on the charts), and the dynamic-response factor for lateral force k_Q from tables 24 and 25.

Then a check is made of the component's strength for conditions (110) and (111) and a calculation is performed for lateral force in conformity with paragraphs 3.37 and 3.38 with consideration of formula (46). If curve $k_{\text{п}}^{\text{np}}$ (broken line) in the charts of figures of 115-122, Appendix 7, is absent with the given value of $k_{\text{м}}^{\text{on}}$, then the beam is performing in the elasticoplastic stage and the flexure calculation is performed only based on condition (110).

4.14. For overhead cover with a soil embankment of more than 1 m in thickness, the dynamic-response factors for displacements in formulas (110) and (111) are taken as equal to the factor k_{Π}, which is determined from the charts of figures 88-90 in relation to the dimensionless parameters S_{Π} and η, calculated from

Table 24 - Dynamic-Response Factors for Lateral Force for Single-Span Fixed Beams and Central Spans of Continuous Beams (Class A-I, A-II and A-III Steels)

β_1	β_2	values k_Q^{on}, k_M^{on} equal to							
		0,6	0,7	0,8	0,9	1	1,1	1,2	1,3
1	2	3	4	5	6	7	8	9	10
0,6—0,9	0,56	0,9	1	1,2	1,3	1,4	1,4	1,5	1,6
	0,64	0,8	0,9	1,1	1,2	1,3	1,4	1,5	1,6
	0,72	0,8	0,9	1	1,1	1,2	1,4	1,5	1,6
	0,8	0,7	0,8	1	1,1	1,2	1,3	1,4	1,5
0,9—1,06	0,7	1,1	1,3	1,3	1,3	1,4	1,4	1,5	1,6
	0,8	1	1,2	1,3	1,3	1,4	1,4	1,5	1,6
	0,9	1	1,1	1,3	1,3	1,4	1,4	1,5	1,6
	1	0,9	1,1	1,2	1,3	1,4	1,4	1,5	1,6
1,06—1,24	0,78	1,1	1,2	1,3	1,3	1,3	1,4	1,5	1,5
	0,89	1	1,2	1,3	1,3	1,3	1,4	1,5	1,5
	1	0,9	1,1	1,3	1,3	1,3	1,4	1,5	1,5
	1,11	0,9	1	1,2	1,3	1,3	1,4	1,5	1,5
1,24—1,54	1,1	0,9	1,1	1,2	1,3	1,3	1,4	1,4	1,5
	1,24	0,9	1	1,1	1,3	1,3	1,4	1,4	1,5
	1,38	0,8	1	1,1	1,2	1,3	1,4	1,4	1,5
	1,52	0,8	0,9	1,1	1,2	1,3	1,4	1,4	1,5
1,54—1,95	1,56	0,8	1	1,1	1,2	1,3	1,3	1,4	1,5
	1,73	0,8	0,9	1,1	1,2	1,3	1,3	1,4	1,5
	1,9	0,8	0,9	1	1,1	1,2	1,3	1,4	1,5
	2,08	0,8	0,9	1	1,1	1,2	1,3	1,4	1,5
1,95—2,5	1,99	0,8	0,9	1	1,2	1,2	1,3	1,4	1,5
	2,21	0,8	0,9	1	1,1	1,2	1,3	1,4	1,5
	2,43	0,7	0,9	1	1,1	1,2	1,3	1,4	1,5
	2,66	0,7	0,8	1	1,1	1,2	1,3	1,4	1,5

Table 25 - Dynamic-Response Factors for Lateral Force for Single-Span Fixed Beams and Central Spans of Continuous Beams (Class A-IIv, A-IIIv, A-IV, A-V, At-IV and At-V Steels)

β_1	β_2	values k_Q^{on}, k_M^{on} equal to							
		0,6	0,7	0,8	0,9	1	1,1	1,2	1,3
0,6—0,9	0,56	0,8	0,9	1	1,2	1,3	1,4	1,4	1,5
	0,64	0,7	0,8	1	1,1	1,2	1,3	1,4	1,5
	0,72	0,7	0,8	0,9	1	1,1	1,2	1,3	1,5
	0,8	0,6	0,7	0,8	1	1,1	1,2	1,3	1,4
0,9—1,06	0,7	1	1,1	1,3	1,3	1,3	1,3	1,4	1,4
	0,8	0,9	1	1,2	1,3	1,3	1,3	1,4	1,4
	0,9	0,8	1	1,1	1,3	1,3	1,3	1,4	1,4
	1	0,8	0,9	1,1	1,2	1,3	1,3	1,4	1,4
1,06—1,24	0,78	0,9	1,1	1,2	1,3	1,3	1,3	1,4	1,4
	0,89	0,9	1	1,2	1,3	1,3	1,3	1,4	1,4
	1	0,8	1	1,1	1,2	1,3	1,3	1,4	1,4
	1,11	0,8	0,9	1	1,2	1,3	1,3	1,4	1,4
1,24—1,54	1,1	0,8	0,9	1,1	1,2	1,3	1,3	1,3	1,4
	1,24	0,8	0,9	1	1,1	1,3	1,3	1,3	1,4
	1,38	0,7	0,9	1	1,1	1,2	1,3	1,3	1,4
	1,52	0,7	0,8	0,9	1,1	1,2	1,3	1,3	1,4
1,54—1,95	1,56	0,7	0,8	1	1,1	1,2	1,2	1,3	1,4
	1,73	0,7	0,8	0,9	1	1,2	1,2	1,3	1,4
	1,9	0,7	0,8	0,9	1	1,1	1,2	1,3	1,4
	2,08	0,7	0,8	0,9	1	1,1	1,2	1,3	1,4
1,95—2,5	1,99	0,7	0,8	0,9	1	1,1	1,2	1,3	1,4
	2,21	0,7	0,8	0,9	1	1,1	1,2	1,3	1,4
	2,43	0,6	0,8	0,9	1	1,1	1,2	1,3	1,4
	2,66	0,6	0,7	0,8	0,9	1	1,2	1,3	1,4

formulas (95) and (96), and in relation to the dynamic-response factor for the bending moment for a support computed from formula (113). The maximum value of dynamic load p_{max} in formulas (110), (111) and (113) is determined from formula (101). Weight of the earth embankment must be considered in determining M_c.

The dynamic-response factor for lateral force is taken to be equal to $k_Q^{оп}=1,1\,k_м^{оп}$.

Calculation for Case 1b of Beam with Fixed End and Hinged-Bearing End

4.15. Dynamic-response factors, with consideration of expressions (90) and (91) are determined from the charts of figures 79-83 by the methods examined above.

4.16. Strength conditions are checked for the bearing and span sections from formulas (103), in which

$$M_{экв}^{оп}=\frac{\bar{q}_{экв}^{(м)}\,l^2}{8}\,k_2; \tag{115}$$

$$M_{экв}^{пр}=\frac{\bar{q}_{экв}^{(м)}\,l^2}{8}\left(1-\frac{k_2}{4}\right)^2. \tag{116}$$

Here $\bar{q}_{экв}^{(м)}=p_{max}k_м b$, where b is the effective width of the beam's loaded zone. For overhead cover with the soil embankment layer thicker than 1 m, the value P_{max} is determined from formula (101) with $S_п>0.25$ and from the formula

$$p_{max}=\Delta p_ф \bar{k}_{зат} \text{ with } S_п\leqslant 0,25.$$

Factor k_2, which takes account of the redistribution of bending moment values in the span and on the support as a result of crack expansion in the concrete's tensile zone, is determined from the formula

$$k_2=\frac{0,26+0,74\,\beta_1}{0,578+0,422\,\beta_1}, \tag{117}$$

in which β_1 is determined from expression (107).

4.17. The limiting value of the dynamic load for a beam with known sectional characteristics is equal to the lesser of two values calculated from the formulas:

$$p_{max}=\frac{8\left(M_0^{оп}-k_y\,M_c^{оп}\right)}{bl^2\,k_2\,k_м}; \tag{118}$$

$$p_{max}=\frac{8\left(M_0^{пр}-k_y\,M_c^{пр}\right)}{bl^2\left(1-\frac{k_2}{4}\right)^2 k_м}. \tag{119}$$

Calculation for Case 1a for a Beam with Fixed End and Hinged-Bearing End

4.18. The flexure calculation of a reinforced concrete beam with a fixed and a hinged end is performed:

For the support

$$\psi^{оп}=\frac{\bar{p}\,l^3}{106,8\,B^{пр}}\,k_п^{оп}\leqslant 0,5\,\psi_п^{оп}; \tag{120}$$

For the span

$$\psi^{пр}=\frac{\bar{p}\,l^3}{44\,B^{пр}}\,k_п^{пр}\leqslant\psi_п^{пр}. \tag{121}$$

The hinge of plasticity in the span is situated at a distance (0.36-0.45)1 from the swing support.

Dynamic-response factors for displacements $k_п^{оп}$ and $k_п^{пр}$ are determined from the charts of figures 123-130, Appendix 7, in relation to the values ω_2, θ_1, β_1 (107), β_2 (112) and the dynamic-response factor for the bending moment for support $k_м^{оп}$, computed from the formula

$$k_м^{оп} = \frac{M_0^{оп} - k_y M_о^{оп}}{M_p^{оп}}, \qquad (122)$$

in which

$$M_p^{оп} = \frac{\bar{p} l^2}{8} k_2. \qquad (123)$$

In computing the factor β_2 from formula (112) the values $M_о^{оп}$ and $M_о^{пр}$ should be substituted therein, as determined from expressions

$$M_c^{оп} = \frac{\bar{q}_c l^2}{8} k_3; \qquad (124)$$

$$M_c^{пр} = \frac{\bar{q}_c l^2}{8} \left(1 - \frac{k_2}{4}\right)^2. \qquad (125)$$

If curve $k_п^{пр}$ (broken line) in charts of figures 123-130, Appendix 7, is absent with the found value $k_м^{оп}$, then the beam is performing in the elasticoplastic stage and the flexure calculation is performed only based on condition (120) for the support.

4.19. Dynamic-response factors for lateral force at the fixed $k_Q^{оп}$ and swing support $k_Q^{ш}$ are taken from tables 24-25 in relation to the values β_1, β_2 and $k_м^{оп}$.

Lateral forces on the supports from an equivalent static load equal:

At the fixed support

$$Q_{экв}^{оп} = \frac{\bar{p} l}{2} k_Q^{оп}; \qquad (126)$$

At the swing support

$$Q_{экв}^{ш} = \frac{\bar{p} l}{2} k_Q^{ш}. \qquad (127)$$

4.20. For overhead cover with an earth embankment thicker than 1 m the dynamic-response factors for displacements in formulas (120) and (121) are taken as equal to the factor $k_П$, which is determined from the charts of figures 83-90 in relation to $S_П$ and η and the dynamic-response factor for bending moments for the support, computed from formula (122). The maximum value of dynamic load is determined from formula (101). In finding M_c it is necessary to consider the weight of the earth embankment. The dynamic-response factors for lateral force are taken as equal to $k_Q^{оп} = k_Q^{ш} \approx 1{,}1\, k_м^{оп}$. Lateral forces from the equivalent static load at the supports are determined from tables for static calculation.

Calculation of Continuous Beams with End Swing Supports

4.21. A continuous beam is presented in the form of a system of single-span beams: with fixed ends (central spans) and with one fixed end and the other a hinged-bearing end (end spans). The number of spans is denoted by n, with spans numbered from left to right (i = 1, 2, ..., n). Strength conditions must be satisfied simultaneously for all spans.

In calculating central spans of a continuous beam we use formulas for a single-span beam fixed at both supports and, in calculating end spans with a swing support, we use formulas for a beam with one fixed support and the other a swing support. In these formulas the values of factors K_1 and K_2, which take account of the redistribution of bending moments at the support and in the span, must be changed, since the relationships between bending moments at the supports and in the spans of continuous beams differ from those in single-span beams.

For central spans (i = 2, 3, ..., n-1) the values of factors $k^{н}_{1,i}$ in relation to the number of the beam's spans will equal:

With n = 3: $k^{н}_{1,2} = 1{,}2k_{1,2}$;

With n = 4:

For the second $k^{н}_{1,2} = 1.07k_{1,2}$;

For the third $k^{н}_{1,3} = 1.07k_{1,3}$;

With n = 5:

For the second $k^{н}_{1,2} = 1.105k_{1,2}$;

For the third $k^{н}_{1,3} = 0.948k_{1,3}$;

For the fourth $k^{н}_{1,4} = 1.105k_{1,4}$.

Here
$$k_{1,i} = \frac{0{,}269 + 0{,}731\,\beta_{1,i}}{0{,}46 + 0{,}54\,\beta_{1,i}}; \qquad (128)$$

$$\beta_{1,i} = \frac{\overline{B}^{оп}_i}{B^{пр}_i}; \qquad (129)$$

$$\overline{B}^{оп}_i = \frac{1}{2}\left(B^{оп}_i + B^{оп}_{i+1}\right); \qquad (130)$$

$B^{оп}_i$ is the rigidity of the i-th support.

With unequal values of moments of inner stresses on supports, it is possible to take their mean values

$$\overline{M}^{оп}_{0,i} = \frac{1}{2}\left(M^{оп}_{0,i} - k_y M^{оп}_{c,i} + M^{оп}_{0,i+1} - k_y M^{оп}_{c,i+1}\right). \qquad (131)$$

where $M^{оп}_{0,i}$ is the limiting moment of internal stresses at the i-th support;
$M^{оп}_{c,i}$ is the bending moment from static load at the i-th support.

For end spans (i = 1, n) the factors $k^{н}_2$ will equal:

Two-span beam (n = 2) $k^{н}_2 = k_2$;

With any number of spans greater than two (n⩾3): $k^{н}_2 = 0{,}8k_2$;
k_2 is calculated from formula (117).

4.22. In the calculation for the first limiting state (Case 1b), the dynamic response factor is determined from the charts of figures 79-83 with consideration of the expression (90) and (91).

4.23. The calculation for the first limiting state (Case 1a) is performed in the following sequence: The dynamic-response factor for bending moment ($k_{м}^{оп}$) for the i-th span is computed from formulas (113) for central spans or (122) for end span. Here k_1 and k_2 are substituted for $k_{1,2}^{н}$ and $K_2^{н}$ respectively. The factors $k_Q^{оп}$ ($k_Q^{ш}$) and $k_{п}^{оп}$, $k_{п}^{пр}$ (see tables 24-31) are found according to the found value $k_{м}^{оп}$ with the help of charts of Appendix 7 in relation to the fixing of the ends of the i-th span and the values $\omega_1^{н}\theta\,(\omega_2^{н}\theta)$, $\beta_{1,i}$, $\beta_{2,i}$.

Strength conditions of the continuous beam would be written in the form:

For supports $\psi_i^{оп,н} \leqslant \psi_{пi}^{оп}, \; i=2, 3, \ldots, n,$ (132)

where $\psi_i^{оп,н} = \psi_{i\,лев}^{оп} + \psi_{i\,прав}^{оп}$.

The values $\psi_{i\,лев}^{оп}$, $\psi_{i\,прав}^{оп}$ are determined from formula (110) for central spans or formula (120) for an end span;

For spans $\psi_i^{пр} \leqslant \psi_{пi}^{пр}, \; i=1, 2, \ldots, n,$ (133)

where $\psi_i^{пр}$ is determined from formula (111) for central spans or formula (121) for an end span.

The hinge of plasticity in an end span is located at a distance (0.36-0.45)1 from the swing support.

For overhead cover with a soil embankment greater than 1 m, it is recommended that the dynamic-response factors be determined for all spans of the continuous beam from the end span with use of charts of figures 88-91 in relation to the dimensionless parameters S_{Π}, η and the factor $k_{м}^{оп}$, computed from formulas (95), (96) and (122) respectively.

Calculation of Rectangular Slabs

4.24. Rectangular slabs for which the ratio of the greater side b to the lesser a satisfies the condition $\frac{b}{a}>2$, are beam slabs and their calculation is performed by the methods set forth above.

If $$1 \leqslant \frac{b}{a} \leqslant 2, \qquad (134)$$

then the calculation of slabs should be performed with consideration of conditions of support on all four sides.

4.25. In the calculation for the first limiting state (Case 1b), dynamic-response factors with consideration of expressions (90) and (91) are determined from the charts of figures 79-83 in relation to the cyclic natural oscillation frequency of the slab, the values characterizing the law of a load's change over time, and the dimensionless parameters S_{Π} and η computed from formulas (95) and (96) (for overhead cover with an earth embankment greater than 1 m).

The moments (bending and twisting) and lateral forces from the equivalent static load are found from slab calculation tables.

4.26. In the calculation for the first limiting state (Case 1a) of rectangular hinged-bearing slabs, the dynamic-response factor for the bending moment is computed from the formula

Table 26 - Dynamic-Response Factors for Lateral Force for Single-Span Beam with a Fixed Support and Swing Support and for a Continuous Two-Span Beam (Class A-I, A-II and A-III Steels)

β_1	β_2	Values k_Q^m (numerator) and k_Q^{on} (denominator) with k_M^{on} equal to							
		0,6	0,7	0,8	0,9	1	1,1	1,2	1,3
0,5—0,77	0,53	1,2/1,5	1,4/1,7	1,4/1,8	1,4/1,9	1,4/1,9	1,4/1,9	1,4/2	1,4/2
	0,82	0,8/1,1	1/1,3	1,1/1,5	1,2/1,6	1,3/1,8	1,4/1,9	1,4/2	1,4/2
	1,12	0,6/0,9	0,6/1	0,7/1,1	0,8/1,3	0,9/1,4	1/1,5	1,1/1,7	1,2/1,8
0,77—1,07	0,7	1,1/1,5	1,3/1,7	1,3/1,8	1,3/1,8	1,3/1,8	1,3/1,9	1,3/2	1,3/2
	0,85	0,9/1,3	1,1/1,5	1,2/1,7	1,3/1,8	1,3/1,8	1,3/1,9	1,3/2	1,3/2
	1	0,8/1,2	1/1,4	1,1/1,5	1,2/1,7	1,3/1,8	1,3/1,9	1,3/2	1,3/2
1,07—1,2	0,79	1/1,4	1,2/1,6	1,3/1,7	1,2/1,8	1,2/1,8	1,2/1,9	1,2/2	1,3/2
	0,96	0,9/1,3	1/1,4	1,2/1,6	1,2/1,8	1,2/1,8	1,2/1,9	1,2/2	1,3/2
	1,13	0,8/1,1	0,9/1,3	1/1,5	1,1/1,7	1,2/1,8	1,2/1,9	1,2/2	1,3/2
1,2—1,36	0,9	1/1,3	1,1/1,6	1,2/1,7	1,2/1,8	1,2/1,8	1,2/1,9	1,2/2	1,2/2
	1,09	0,8/1,2	1/1,4	1,1/1,6	1,2/1,8	1,2/1,8	1,2/1,9	1,2/2	1,2/2
	1,28	0,7/1,1	0,8/1,3	1/1,5	1,1/1,6	1,2/1,8	1,2/1,9	1,2/2	1,2/2
1,36—1,54	1,16	0,8/1,2	0,9/1,4	1,1/1,6	1,2/1,7	1,2/1,8	1,2/1,9	1,2/1,9	1,2/2
	1,38	0,7/1,1	0,8/1,3	0,9/1,4	1/1,6	1,1/1,8	1,2/1,9	1,2/1,9	1,2/2
	1,59	0,6/1	0,7/1,2	0,8/1,3	0,9/1,5	1/1,7	1,1/1,8	1,2/1,9	1,2/2
1,54—1,86	1,67	0,7/1,1	0,8/1,2	0,9/1,4	1/1,6	1/1,7	1,1/1,9	1,1/1,9	1,2/2
	1,95	0,6/1	0,7/1,2	0,8/1,3	0,9/1,5	0,9/1,6	1/1,8	1,1/1,9	1,2/2
	2,23	0,5/0,9	0,6/1,1	0,7/1,2	0,8/1,4	0,9/1,5	0,9/1,7	1/1,8	1,1/2

Table 27 - Dynamic-Response Factors for Lateral Force for Single-Span Beam with a Fixed Support and Swing Support and for a Continuous Two-Span Beam (Class A-IIv, A-IIIv, A-IV, A-V, At-IV and At-V Steels)

β_1	β_2	Values k_Q^m (numerator) and k_Q^{on} (denominator) with k_M^{on} equal to							
		0,6	0,7	0,8	0,9	1	1,1	1,2	1,3
0,5—0,77	0,53	1,1/1,3	1,2/1,5	1,4/1,8	1,4/1,8	1,4/1,8	1,4/1,9	1,4/1,9	1,4/2
	0,82	0,7/1	0,8/1,1	1/1,3	1,1/1,5	1,2/1,6	1,3/1,8	1,4/1,9	1,4/2
	1,12	0,5/0,7	0,6/0,8	0,6/1	0,7/1,1	0,8/1,2	0,9/1,3	1/1,5	1/1,6
0,77—1,07	0,7	1/1,3	1,2/1,5	1,3/1,7	1,3/1,8	1,3/1,8	1,3/1,8	1,3/1,9	1,3/1,9
	0,85	0,8/1,1	1/1,3	1,1/1,5	1,2/1,7	1,3/1,8	1,3/1,8	1,3/1,9	1,3/1,9
	1	0,7/1	0,8/1,2	1/1,4	1,1/1,5	1,2/1,7	1,3/1,8	1,3/1,9	1,3/1,9
1,07—1,2	0,79	0,9/1,2	1,1/1,4	1,2/1,6	1,3/1,7	1,2/1,8	1,2/1,8	1,2/1,8	1,2/1,9
	0,96	0,8/1,1	0,9/1,3	1/1,4	1,2/1,6	1,2/1,8	1,2/1,8	1,2/1,8	1,2/1,9
	1,13	0,7/1	0,8/1,2	0,9/1,3	1/1,5	1,1/1,6	1,2/1,8	1,2/1,8	1,2/1,9
1,2—1,36	0,9	0,9/1,2	1/1,4	1,2/1,6	1,2/1,7	1,2/1,7	1,2/1,8	1,2/1,8	1,2/1,9
	1,09	0,7/1,1	0,9/1,2	1/1,4	1,1/1,6	1,2/1,7	1,2/1,8	1,2/1,8	1,2/1,9
	1,28	0,6/1	0,7/1,1	0,9/1,3	1/1,4	1,1/1,6	1,2/1,8	1,2/1,8	1,2/1,9
1,36—1,54	1,16	0,7/1,1	0,8/1,2	1/1,4	1,1/1,6	1,2/1,7	1,2/1,8	1,2/1,8	1,2/1,9
	1,38	0,6/1	0,7/1,1	0,8/1,3	0,9/1,4	1/1,6	1,1/1,8	1,2/1,8	1,2/1,9
	1,59	0,6/0,9	0,6/1	0,7/1,2	0,8/1,3	0,9/1,5	1/1,6	1,1/1,8	1,2/1,9
1,54—1,86	1,67	0,6/0,9	0,7/1,1	0,8/1,2	0,9/1,4	1/1,6	1/1,7	1,1/1,8	1,1/1,9
	1,95	0,5/0,9	0,6/1	0,7/1,2	0,8/1,3	0,8/1,4	0,9/1,6	1/1,7	1,1/1,9
	2,23	0,5/0,8	0,5/1	0,6/1,1	0,7/1,2	0,8/1,4	0,8/1,5	0,9/1,6	1/1,8

Table 28 - Dynamic-Response Factors for Lateral Force for End Span of Continuous Three-Span Beam (Class A-I, A-II and A-III Steels)

β_1	β_2	Values k_Q^m (numerator) and k_Q^{on} (denominator) with k_M^{on} equal to							
		0,6	0,7	0,8	0,9	1	1,1	1,2	1,3
0,5—0,77	0,53	1/1,2	1,1/1,4	1,3/1,6	1,4/1,8	1,5/1,9	1,5/1,9	1,5/2	1,5/2
	0,82	0,7/0,9	0,8/1	0,9/1,2	1/1,3	1,1/1,5	1,2/1,6	1,3/1,7	1,4/1,9
	1,12	0,6/0,8	0,7/1	0,8/1,1	0,9/1,2	1/1,3	1,1/1,5	1,1/1,6	1,2/1,7
0,77—1,07	0,7	0,9/1,2	1/1,4	1,2/1,5	1,3/1,7	1,4/1,8	1,4/1,9	1,4/1,9	1,4/2
	0,85	0,8/1	0,9/1,2	1/1,4	1,1/1,5	1,2/1,7	1,3/1,8	1,4/1,9	1,4/2
	1	0,7/0,9	0,8/1,1	0,9/1,2	1/1,4	1,1/1,5	1,2/1,7	1,3/1,8	1,4/1,9
1,07—1,2	0,79	0,8/1,1	1/1,3	1,1/1,5	1,2/1,6	1,3/1,8	1,4/1,9	1,4/1,9	1,4/2
	0,96	0,7/1	0,8/1,2	0,9/1,3	1/1,5	1,1/1,6	1,3/1,8	1,3/1,9	1,4/2
	1,13	0,6/0,9	0,7/1,1	0,8/1,2	0,9/1,3	1/1,5	1,1/1,6	1,2/1,8	1,3/1,9
1,2—1,36	0,9	0,8/1,1	0,9/1,3	1/1,4	1,1/1,6	1,3/1,7	1,3/1,9	1,3/1,9	1,4/2
	1,09	0,7/1	0,8/1,1	0,9/1,3	1/1,4	1,1/1,6	1,2/1,7	1,3/1,9	1,4/2
	1,28	0,6/0,9	0,7/1	0,8/1,2	0,9/1,3	1/1,4	1/1,6	1,1/1,7	1,2/1,8
1,36—1,54	1,16	0,7/1	0,8/1,1	0,9/1,3	1/1,4	1,1/1,6	1,2/1,7	1,2/1,9	1,3/2
	1,38	0,6/0,9	0,7/1	0,8/1,2	0,8/1,3	0,9/1,4	1/1,6	1,1/1,7	1,2/1,8
1,54—1,86	1,31	0,6/0,9	0,7/1,1	0,8/1,2	0,9/1,4	1/1,5	1,1/1,7	1,2/1,8	1,3/1,9
	1,56	0,5/0,9	0,6/1	0,7/1,1	0,8/1,3	0,9/1,4	1/1,5	1,1/1,7	1,1/1,8

Table 29 - Dynamic-Response Factors for Lateral Force for End Span of Continuous Three-Span Beam (Class A-IIv, A-IIIv, A-IV, A-V, At-IV and At-V Steels)

β_1	β_2	Values k_Q^m (numerator) and k_Q^{on} (denominator) with k_M^{on} equal to							
		0,6	0,7	0,8	0,9	1	1,1	1,2	1,3
0,5—0,77	0,53	0,9/1,1	1/1,2	1,1/1,4	1,3/1,6	1,4/1,8	1,5/1,9	1,5/1,9	1,5/1,9
	0,82	0,6/0,8	0,7/0,9	0,8/1	0,9/1,2	1/1,3	1,1/1,4	1,2/1,6	1,3/1,7
	1,12	0,5/0,7	0,6/0,8	0,7/0,9	0,8/1,1	0,8/1,2	0,9/1,3	1/1,4	1,1/1,5
0,77—1,07	0,7	0,8/1	0,9/1,2	1,1/1,4	1,2/1,5	1,3/1,7	1,4/1,8	1,4/1,9	1,4/1,9
	0,85	0,7/0,9	0,8/1,1	0,9/1,1	1/1,4	1,1/1,5	1,2/1,7	1,3/1,8	1,4/1,9
	1	0,6/0,8	0,7/1	0,8/1,1	0,9/1,2	1/1,4	1,1/1,5	1,2/1,6	1,3/1,8
1,07—1,2	0,79	0,7/1	0,9/1,2	1/1,3	1,1/1,5	1,2/1,6	1,3/1,8	1,3/1,8	1,4/1,9
	0,96	0,6/0,9	0,7/1	0,8/1,2	0,9/1,3	1/1,5	1,1/1,6	1,2/1,7	1,3/1,9
	1,13	0,5/0,8	0,6/0,9	0,7/1,1	0,8/1,2	0,9/1,3	1/1,5	1,1/1,6	1,2/1,7
1,2—1,36	0,9	0,7/0,9	0,8/1,1	0,9/1,3	1/1,4	1,1/1,6	1,3/1,7	1,3/1,8	1,3/1,9
	1,09	0,6/0,8	0,7/1	0,8/1,1	0,9/1,3	1/1,4	1,1/1,5	1,2/1,7	1,3/1,8
	1,28	0,5/0,8	0,6/0,9	0,7/1	0,8/1,1	0,8/1,3	0,9/1,4	1/1,5	1,1/1,7
1,36—1,54	1,16	0,6/0,8	0,7/1	0,8/1,1	0,9/1,3	1/1,4	1/1,5	1,1/1,7	1,2/1,8
	1,38	0,5/0,8	0,6/0,9	0,7/1	0,7/1,1	0,8/1,3	0,9/1,4	1/1,5	1,1/1,7
1,54—1,86	1,31	0,5/0,8	0,6/0,9	0,7/1,1	0,8/1,2	0,9/1,4	1/1,5	1,1/1,6	1,2/1,8
	1,56	0,5/0,7	0,5/0,9	0,6/1	0,7/1,1	0,8/1,2	0,9/1,4	0,9/1,5	1/1,6

Table 30 - Dynamic-Response Factors for Lateral Force for End Span of Continuous Beam with More than Three Spans (Class A-I, A-II and A-III Steels)

β_1	β_2	Values k_Q^{m} (numerator) and k_Q^{on} (denominator) with k_M^{on} equal to							
		0,6	0,7	0,8	0,9	1	1,1	1,2	1,3
0,5—0,77	0,53	1/1,2	1,1/1,4	1,3/1,6	1,4/1,8	1,5/1,9	1,6/2	1,6/2,1	1,6/2,1
	0,82	0,7/0,9	0,8/1	0,9/1,2	1/1,3	1,1/1,5	1,2/1,6	1,3/1,7	1,4/1,9
	1,12	0,6/0,8	0,7/1	0,8/1,1	0,9/1,2	1/1,3	1,1/1,5	1,1/1,6	1,2/1,7
0,77—1,07	0,7	0,9/1,2	1/1,4	1,2/1,5	1,3/1,7	1,4/1,9	1,5/2	1,5/2	1,5/2,1
	0,85	0,8/1	0,9/1,2	1/1,4	1,1/1,5	1,2/1,7	1,3/1,8	1,4/2	1,5/2,1
	1	0,7/1	0,8/1,1	0,9/1,2	1/1,4	1,1/1,5	1,2/1,7	1,3/1,8	1,4/2
1,07—1,2	0,79	0,8/1,1	1/1,3	1,1/1,5	1,2/1,6	1,3/1,8	1,5/2	1,5/2	1,5/2,1
	0,96	0,7/1	0,8/1,2	0,9/1,3	1/1,5	1,2/1,6	1,3/1,8	1,4/2	1,4/2
	1,13	0,6/0,9	0,7/1	0,8/1,2	0,9/1,3	1/1,5	1,1/1,6	1,2/1,8	1,3/1,9
1,2—1,36	0,9	0,8/1,1	0,9/1,3	1/1,4	1,1/1,6	1,2/1,7	1,4/1,9	1,4/2	1,4/2,1
	1,09	0,7/1	0,8/1,1	0,9/1,3	1/1,4	1,1/1,6	1,2/1,7	1,3/1,9	1,4/2
	1,28	0,6/0,9	0,7/1	0,8/1,2	0,9/1,3	1/1,4	1/1,6	1,1/1,7	1,2/1,9
1,36—1,54	1,16	0,7/1	0,8/1,1	0,9/1,3	1/1,4	1,1/1,6	1,2/1,7	1,3/1,9	1,3/2
	1,38	0,6/0,9	0,7/1	0,8/1,2	0,8/1,3	0,9/1,4	1/1,6	1,1/1,7	1,2/1,8
1,54—1,86	1,31	0,6/0,9	0,7/1,1	0,8/1,2	0,9/1,4	1/1,5	1,1/1,7	1,2/1,8	1,3/1,9
	1,56	0,5/0,9	0,6/1	0,7/1,1	0,8/1,3	0,9/1,4	1/1,5	1,1/1,7	1,1/1,8

Table 31 - Dynamic-Response Factors for Lateral Force for End Span of Continuous Beam with More than Three Spans (Class A-IIv, A-IIIv, A-IV, A-V, At-IV and At-V Steels)

β_1	β_2	Values k_Q^{m} (numerator) and k_Q^{on} (denominator) with k_M^{on} equal to							
		0,6	0,7	0,8	0,9	1	1,1	1,2	1,3
0,5—0,77	0,53	0,9/1,1	1/1,2	1,1/1,4	1,3/1,6	1,4/1,8	1,6/1,9	1,6/2	1,6/2
	0,82	0,6/0,8	0,7/0,9	0,8/1	0,9/1,2	1/1,3	1,1/1,4	1,2/1,6	1,3/1,7
	1,12	0,5/0,7	0,6/0,8	0,7/0,9	0,8/1,1	0,8/1,2	0,9/1,3	1/1,4	1,1/1,5
0,77—1,07	0,7	0,8/1	0,9/1,2	1,1/1,4	1,2/1,5	1,3/1,7	1,4/1,9	1,5/2	1,5/2
	0,85	0,7/0,9	0,8/1,1	0,9/1,2	1/1,4	1,1/1,5	1,2/1,7	1,3/1,8	1,4/2
	1	0,6/0,8	0,7/1	0,8/1,1	0,9/1,2	1/1,4	1,1/1,5	1,2/1,6	1,3/1,8
1,07—1,2	0,79	0,7/1	0,9/1,2	1/1,3	1,1/1,5	1,2/1,6	1,4/1,8	1,5/2	1,5/2
	0,96	0,6/0,9	0,7/1	0,8/1,2	0,9/1,3	1/1,5	1,1/1,6	1,3/1,7	1,4/1,9
	1,13	0,5/0,8	0,6/0,9	0,7/1,1	0,8/1,2	0,9/1,3	1/1,5	1,1/1,6	1,2/1,7
1,2—1,36	0,9	0,7/1	0,8/1,1	0,9/1,3	1/1,4	1,1/1,6	1,3/1,7	1,4/1,9	1,4/2
	1,09	0,6/0,8	0,7/1	0,8/1,1	0,9/1,3	1/1,4	1,1/1,5	1,2/1,7	1,3/1,8
	1,28	0,5/0,8	0,6/0,9	0,7/1	0,8/1,2	0,9/1,3	0,9/1,4	1/1,5	1,1/1,7
1,36—1,54	1,16	0,6/0,8	0,7/1	0,8/1,1	0,9/1,3	1/1,4	1,1/1,5	1,2/1,7	1,2/1,8
	1,38	0,5/0,8	0,6/0,9	0,7/1	0,7/1,2	0,8/1,3	0,9/1,4	1/1,5	1,1/1,7
1,54—1,86	1,31	0,6/0,9	0,7/1,1	0,8/1,2	0,9/1,4	1/1,5	1,1/1,7	1,2/1,8	1,3/1,9
	1,56	0,5/0,9	0,6/1	0,7/1,1	0,8/1,3	0,9/1,4	1/1;5	1,1/1,7	1,1/1,8

$$k_{м} = \frac{M_0^a - k_y M_c^a}{M_p^a}, \tag{135}$$

where M_0^a is the linear moment of internal stresses in the middle of the slab in a cross section parallel to side b;

M_c^a, M_p^a are the linear bending moments in the middle of a slab in a cross section parallel to side b from the static and dynamic load of intensity p_{max} respectively, determined from slab calculation tables;

k_y is the hardening factor of reinforced steel.

The charts in figures 94 and 95 are used to determine $k_П$ in relation to k_m, $\omega\theta$ and $\chi = \frac{a}{lb}$. The dynamic-response factor for displacement $k_П$ for intermediate values of χ can be determined by interpolation.

Fulfillment of the strength condition is checked from the formula

$$\psi = \frac{p_{max}\, a^3 k_п}{15 D (1 + \chi^2)^2} \leqslant \psi_п, \tag{136}$$

where D is the slab cylindrical rigidity;

$\psi_П$ is determined from the chart in Fig. 78.

In the presence of a soil embankment thicker than 1 m above the slab, the factor $k_П$ in formula (136) is determined from the charts of figures 78-80 in relation to the dimensionless parameters $S_П$ and η (96) and the dynamic-response factor for the bending moment computed from formula (135). Dynamic load p_{max} is determined from formula (101).

4.27. Rectangular slabs fixed along the entire perimeter can be calculated approximately by the methods examined above by taking the sum $M_{0\,пр}^{(a)} + M_{0\,оп}^{(a)}$ in place of the value $M_0^{(a)}$, where $M_{0\,пр}^{(a)}$, $M_{0\,оп}^{(a)}$ are linear limiting moments of internal stresses in the slab span and at the support respectively.

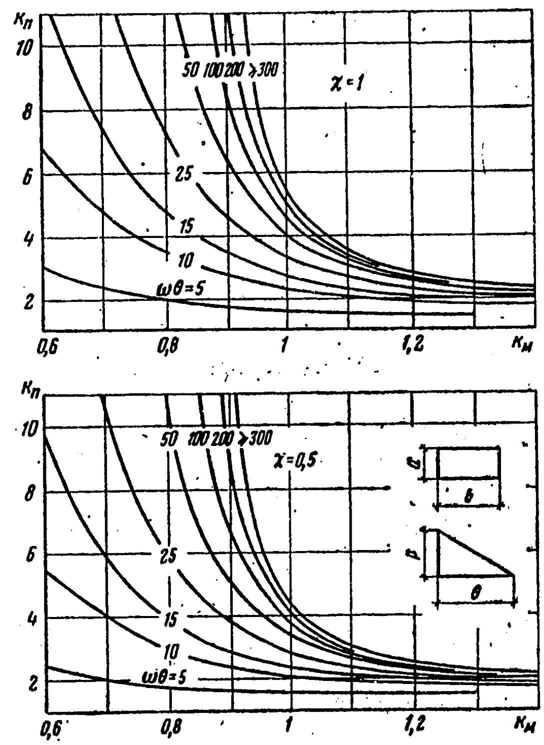


Fig. 94. Charts showing relationship of dynamic-response factors for displacements k_Π to dynamic-response factors for bending moment k_m in calculating hinged-bearing slabs for Case 1a with consideration of the effect of the strain rate on strength properties of reinforced steels with ratios of the lesser slab side a to the greater b (χ = 1 and 0.5)

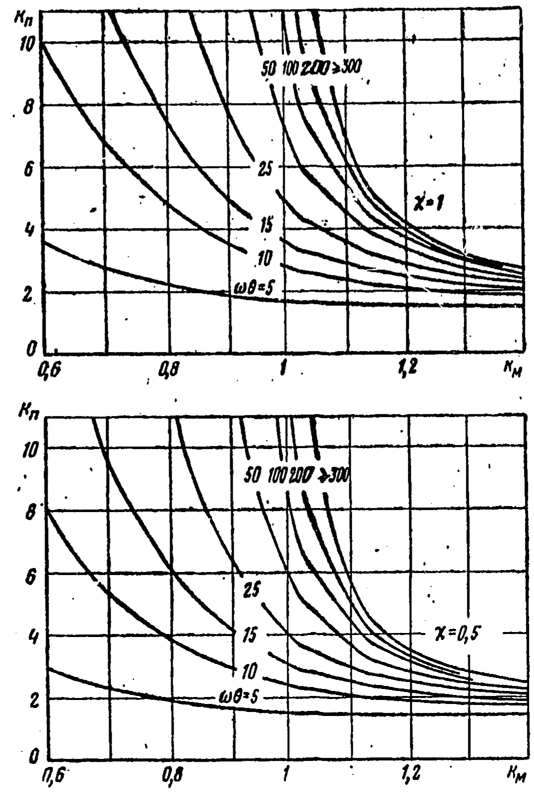


Fig. 95. Charts showing relationship of dynamic-response factors for displacements $k_П$ to dynamic-response factor for bending moment k_m in calculating hinged-bearing slabs for Case 1a without considering the effect of strain rate on strength properties of reinforced steels (χ = 1 and 0.5)

Appendix 1 - Basic Characteristics of Airtight and Airtight-Blast Doors, Gates and Shutters for Protective Civil Defense Structures

Door, Gate, Shutter Code	Old Code	Opening Size, cm	Recommendations for Use of Airtight Blast Doors, Gates and Shutters Based on Shelter Class and Entrance Factor												Detail Plan Album Designation
			Shelter Class II			III			IV			V			
			Entrance Factor												
			1	1,2–1,5	2,3–2,66	1–1,12	1,2	2–2,5	1	1,2–1,25	2–2,3	1	1,2–1,4	1,7–1,4	
Airtight Blast Doors															
DU-I-7 3	—	80×180	—	+	+	—	—	+	—	—	—	—	—	—	TDK-N-I-71
DU-III-6	—	80×180	+	—	—	+	+	—	+	+	+	+	+	+	"
DU-I-1	ZGD-100	120×200	—	+	+	—	—	+	—	—	—	—	—	—	TDK-N-I-67, Part II, Section IV, 1969
DU-I-2	DZGM	120×200	—	+	+	—	—	+	—	—	—	—	—	—	TDK-N-I-68, Part II, Section IV, 1971
DU-III-5	—	120×200	+	—	—	+	+	—	+	+	+	+	+	+	TDK-N-I-71
DU-I-5	—	180×240	•	+	+	•	•	+	—	—	•	—	—	—	TDK-N-I-70, Part II, Sec IV, Album No 5 & 6
DU-IV-4	—	180×240	—	—	—	—	—	—	+	+	—	+	+	+	TDK-N-I-71
DU-I-6	—	300×240	•	+	+	•	•	+	—	—	•	—	—	—	TDK-N-I-70, Part II, Sec IV, Album No 5 & 6
DU-IV-5	—	300×240	—	—	—	—	—	—	+	+	—	+	+	+	TDK-N-I-71
Airtight Doors															
DU-IV-3	DGM	80×180	+	+	+	+	+	+	+	+	+	+	+	+	TDK-N-I-68, Part II Sec IV, 1971
DU-IV-2	DGM	120×200	+	+	+	+	+	+	+	+	+	+	+	+	"
Gates															
VU-I-1	—	220×240	—	—	+	—	—	+	—	—	—	—	—	—	TDK-N-I-69/9 under plans A-II/III-1000-70/8
VU-II-1	—	220×240	+	+	—	+	+	—	+	+	+	—	—	+	"
VU-III-1	—	220×240	—	—	—	—	—	—	—	—	—	+	+	+	"
VU-I-2	—	300×240	—	—	+	—	—	+	—	—	—	—	—	—	TDK-N-I-71/10 under plans A-II/III-2500-71/2
VU-II-2	—	300×240	+	+	—	+	+	—	+	+	+	—	—	—	"
VU-III-2	—	300×240	—	—	—	—	—	—	—	—	—	+	+	+	"
Airtight Blast Shutters															
SU-I-1	—	80×80	—	—	+	—	—	+	—	—	—	—	—	—	TDK-N-I-67, Part I, Sec IV, 1969
SU-II-2	ZS-70	80×80	+	+	—	+	+	—	+	+	+	+	+	+	
Airtight Shutters															
SU-IV-1	GS	80×80	+	+	+	+	+	+	+	+	+	+	+	+	"

Notes: 1. Before articles of new series are put into production it is permitted to use doors DU-I-3 or DZG-80x180-1.35 of the USSR MO [Ministry of Defense] list in place of doors DU-I-7, and doors DU-II-1 or DZG-80x180-0.5 of the USSR MO list instead of doors DU-III-6.

2. Doors DU-IV-4, DU-IV-5 and gates VU-III-1 and VU-III-2 also can be used as airtight units in all types of shelters.

3. Use of the D-IV-1(GD) door is permitted in place of the airtight DU-IV-3 door.

4. Airtight doors are used as entrance doors in DES and as inner doors of shelter airlocks.

5. The DU-III-1(ZGD-15) doors and SU-III-1(ZGS-15) shutters presently being produced can be used as external doors and shutters in Class IV structures with an entrance factor of 1-1.25, and DU-III-2 and DU-III-3 doors can be used as outer doors in Class V structures with entrance factors of 1-1.2.

6. For the purpose of standardization, it is recommended that the inner airtight blast door in the airlock-sluice be made identical with the outer door.

Conventional symbols:
+- Components recommended for use;
*- Components permissible to use until development of new ones;
-- Components not recommended for use.

Appendix 2 - Methodology for Determining Cost Indicators of Basic Structural Elements of CD Shelters

1. The optimum cost of various structural elements of spaces adaptable as shelters can be determined on the basis of factual data of rational design decisions for structures depending on the type of materials used and their strength characteristics, design loads, capacity, technological features of production located in the space in peacetime, and other features.

The degree of optimity for cost indicators of a design decision can be assessed by comparing the planned cost of individual structural elements or of the structure as a whole with proportionate costs computed theoretically for the most rational components and space planning decisions for structures.

Proportionate costs U represent the cost (in rubles) of individual structural elements of a structure per 1 m^2 of protected area. The basic structural elements which determine the project cost for adapting spaces as shelters are: overhead cover, walls, columns, foundations, entrances and emergency exits. Proportionate costs for installing these components, calculated for bearing power, can be determined from the nomograms depicted in figures 96-109. The nomogram calculation sequence is shown in the figures by conditional lines.

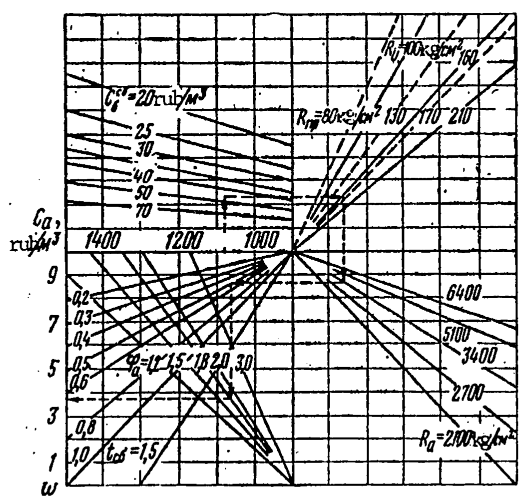


Fig. 96. Nomogram for determining the factor w

Sample calculation. Determine the factor w for an overhead slab with the following data: cost of concrete $C_б^{сб}=45$ rubles/m^3; reinforcement cost C_a = 1,080 rubles/m^3; grade 300 concrete ($R_И$ = 160 kg/cm^2); class A-III reinforcement (R_a = 3,400 kg/cm^2); $t_{Cб}$= 0.5;Φ_a= 1.8. We will obtain for these data in the order indicated by the arrows: w = 3.7 for overhead slabs; w = 2.5 for outer walls of precast reinforced concrete elements ($t_{Cб}$= 1).

2. From the nomogram shown in Fig. 96 we determine the intermediate factor w for the overhead slab and collar beam, which is dependent on the cost of 1 m^3 of precast concrete with consideration of cost for the transportation and installation of the precast elements $C_б^{сб}$; on the cost of 1 m^3 of reinforcement with consideration of cost for placing cages (meshes) in forms C_a; the reinforcement use factor, expressed as a ratio of the entire reinforcement weight per 1 m of the element to the weight of the effective working reinforcement Φ_a; to given grades of concrete and reinforcement and the section coefficient $t_{Cб}$.

The value of the reinforcement use factor Φ_a is taken, dependent on element components, as equal to:

1.4-1.6 for continuous and hollow slabs;

1.6-1.8 for flanged slabs; and

1.8-2 for collar beams.

The design resistances of concrete $R_И$ and reinforcement R_a are taken in accordance with paragraphs 3.21-3.28 of SN 405-70.

The section coefficient is equal to:

$$t_{сб} = 1 \pm \frac{F_{з.п}}{F_{сеч}}, \qquad (137)$$

where $F_{з.п}$ is the area of various kinds of openings, cavities, projections and so on within the effective width of an element's cross section b;

$F_{сеч}$ is the element's sectional area equal to the product of its width b times its height h.

3. In the nomogram depicted in Fig. 97 we determine the values of intermediate factors η_1 and η_2, dependent on ω, η_3 and height of the precast element $A_П$.

The factor η_3 equals the product of the ratio of the cost of 1 m^3 of monolithic concrete "on the job" $C_б^{мон}$ to the cost of 1 m^3 of precast concrete "on the job" $C_б^{сб}$ and the ratio of the monolothic element's section coefficient $t_{МОН}$ to the section coefficient of the precast element $t_{Cб}$. The procedure for determining the factor $t_{МОН}$ is analogous to determining $t_{Cб}$ and is set forth in Paragraph 2.

The precast element height $A_П$ is taken from available standard si :es in catalogues.

4. Proportionate costs for a precast-monolithic cover $U_{у.п}$ are determined from the nomogram in Fig. 98 dependent on previously computed factors w, η_1, η_2, η_3, the protective structure class, the effective span of the structure 1, the coefficient a, which takes account of conditions of fixing at the support, and n_1, which is equal to the ratio of the structure's area on the inner face of outer walls F to the protected area $F_з$; $n_1 = \frac{F}{F_з}$.

The protected area $F_з$ is taken to mean the area of a structure being used for accommodation of sheltered persons and industrial equipment (without considering the area occupied by partitions and inner walls).

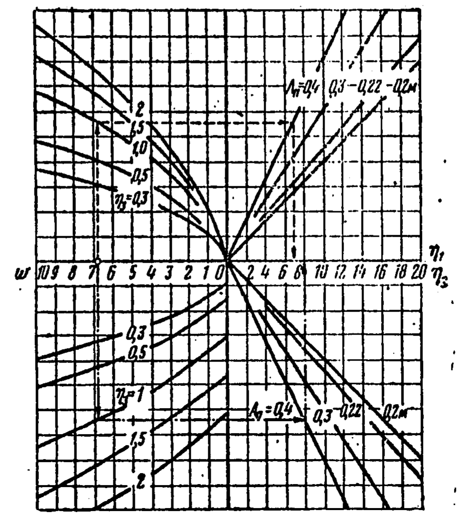


Fig. 97. Nomogram for determining the factor η_1 and η_2

Sample calculation. Determine factors η_1 and η_2 with w = 6.7; η_3 = 1 and A_{Π} = 0.4. Performing the calculations in the order indicated by arrows, we obtain η_1 = 7 and η_2 = 8.

The effective span l of the cover slab is taken in relation to the structure's structural schematic.

The value of the coefficient a, which takes account of conditions of fixing at the supports, can be taken as equal to 8 for sectional components and 11-16 for continuous components.

5. In determining proportionate cost for a precast-monolithic flanged overhead cover $U_{y.\Pi p}$, the proportionate costs for the collar beam must be added to the proportionate costs obtained for the precast-monolithic slab.

The proportionate cost for the collar beam can be determined from the nomogram in Fig. 99 in relation to the factor w, the cost of 1 m^3 of precast concrete on the job C_6^{c6}, the design load per 1 m of collar beam corresponding to the given class of protective structure, the effective span of structure l, the support fixing conditions factor a, grade of concrete and collar beam parameter b. In calculating the cost of a collar beam for a precast-monolithic flanged overhead cover, the value b is determined from the formula $b = \frac{b_p}{\sqrt{b_{\Pi.p}}}$, where b_p is the collar beam width and $b_{\Pi.p}$ is the effective width of the collar beam's flange considering its performance in the span as a T section. The value $b_{\Pi.p}$ is taken as equal to $1/3\, l + b_p$. The value b is taken as equal to b_p in calculating the cost of the collar beam for a precast overhead cover.

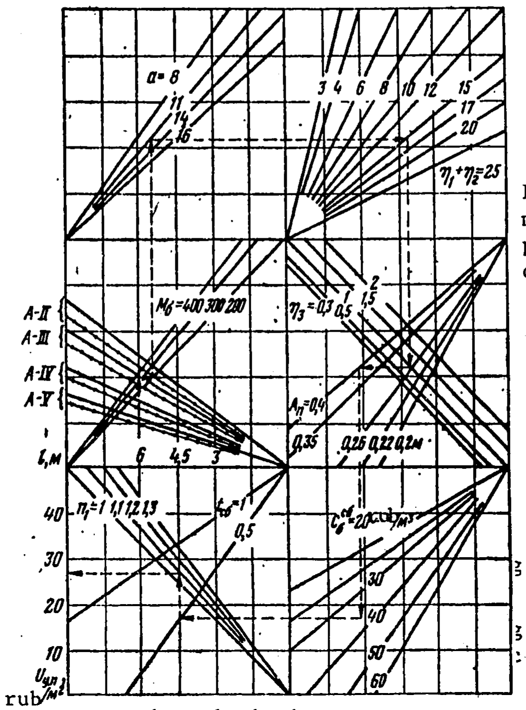


Fig. 98. Nomogram for determining cost of 1 m^2 of a precast-monolithic overhead cover slab $U_{y.\Pi}$

Freestanding
Built-in

Sample calculation. Determine the cost of 1 m^2 of overhead cover slab of a freestanding class A-III shelter with a span of 6 m; concrete grade 200; a = 11; w = 6.7; $\eta_1 + \eta_2 = 15$; $\eta_3 = 1$; $A_\Pi = 0.4$; $C_б^{cб} = 36$ rubles/m^3; $t_{cб} = 0.5$ and $n_1 = 1.1$. With these data in the order indicated by the arrows, we will obtain $U_{y.\Pi} = 27$ rubles/m^2.

Values of the factor w are determined from the nomogram in Fig. 96. The values of other parameters are taken in accordance with instructions presented in paragraphs 2-4.

6. The cost of 1 m^2 of precast reinforced concrete cover of a frame structure is added up from proportionate costs of the slab and collar beam. The cost of 1 m^2 of precast reinforced concrete slab can be determined from the nomogram in Fig. 100 in relation to the design load (structure class), span, concrete grade, structural decision of the slab, and the factors w and $t_{cб}$. The methodology of determining the collar beam's cost is examined in Paragraph 5. The values of factor w are determined from the nomogram in Fig. 96, while the values of other parameters which determine the cost of the collar beam are taken in accordance with the recommendations set forth above.

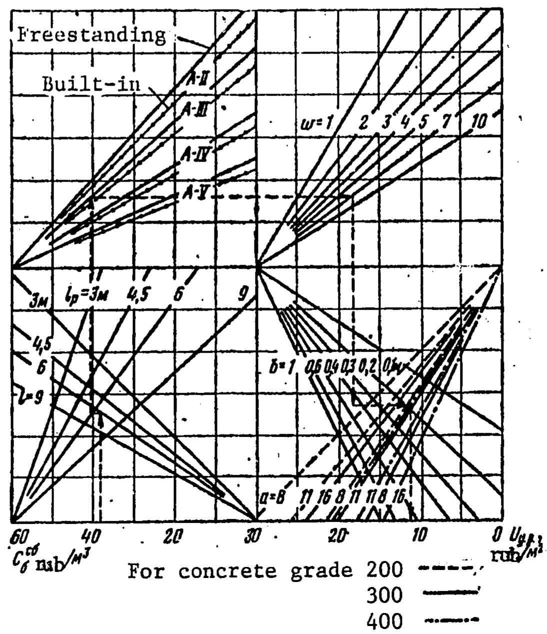


Fig. 99. Nomogram for determining proportionate costs for collar beam $U_{y.p}$

Sample calculation. Determine proportionate costs for collar beam with following data: $C_б^{cб}$ = 36 rubles/m^3; w = 7; $l = l_p$ = 6 m; Class A-III freestanding shelter; $b = \frac{P\ b_p}{\sqrt{b_{п.р}}} = 0{,}45$; concrete grade 300; a = 11. By making calculations in the sequence indicated by the arrows, we will obtain $U_{y.p}$= 11.5 rubles/m^2.

7. The proportionate costs for erecting outer walls of precast reinforced concrete elements $U_{y.c.м}$, with bending strain in a horizontal direction, are determined from the nomogram given in Fig. 101 in relation to the cost of 1 m^3 of precast concrete "on the job" $C_б^{cб}$; the structure perimeter П; lateral pressure coefficient $k_б$, dependent on the type of soil and taken in conformity with Paragraph 3.6; wall span l; concrete grade; and the factor w, the value of which is determined in conformity with Paragraph 2 from the nomogram in Fig. 96. Proportionate costs for outer walls are determined from the condition of their calculation for bearing power.

8. Proportionate costs for inner and outer masonry walls which are fully buried are determined from the nomogram in Fig. 102.

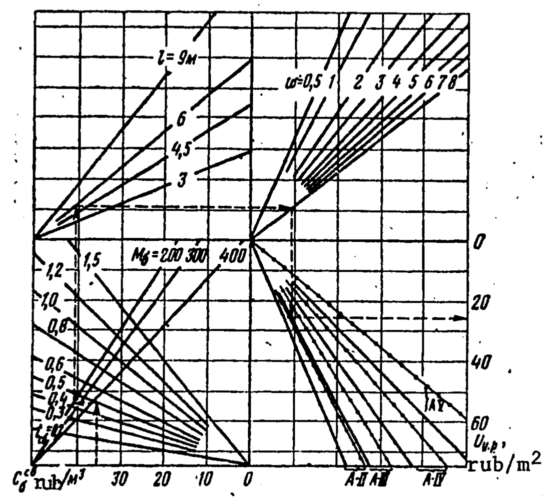


Fig. 100. Nomogram for determining proportionate costs for slab of precast reinforced concrete cover $U_{y.\Pi.c}$

Sample calculation. Determine proportionate costs for slab of precast overhead cover of free-standing class A-III shelter with a span l = 6 m; concrete grade 200; t_{c6} = 0.5; w = 7; C_6 = 36 rubles/m³. Performing the computation in the order indicated by the arrows, we will obtain the proportionate costs, with consideration of the factor n_1 = 1.1, equal to: $U_{y.\Pi.c}$ = 26·1.1 = 28.6 rubles/m².

The thickness of masonry wall d can be determined by a calculation according to existing formulas or from quadrant I of this same nomogram in relation to the structure class, span of overhead cover and design resistance of masonry taken from Paragraph 3.22 of SN 405-70.

The cost of walling also has to include the cost of structural reinforcement based on an average of 10 kg-force of reinforcement per 1 m³ of wall. Quadrant I, II, V, and VI of the nomogram (see Fig. 102) make it possible to determine the cost indicators of internal walls, while quadrants I-V do the same for outer walls. Results for inner walls obtained from the nomogram are multiplied by the factor n_2, determined from the chart in Fig. 103.

9. Proportionate costs for inner walls of precast reinforced concrete panels are determined from the nomogram given in Fig. 104 in relation to the wall thickness, span of overhead cover, height of spaces, and the factors ε and w. Wall thickness for the given class of structure, span of overhead cover and concrete grade can be determined from the chart in Fig. 105. In all instances here the

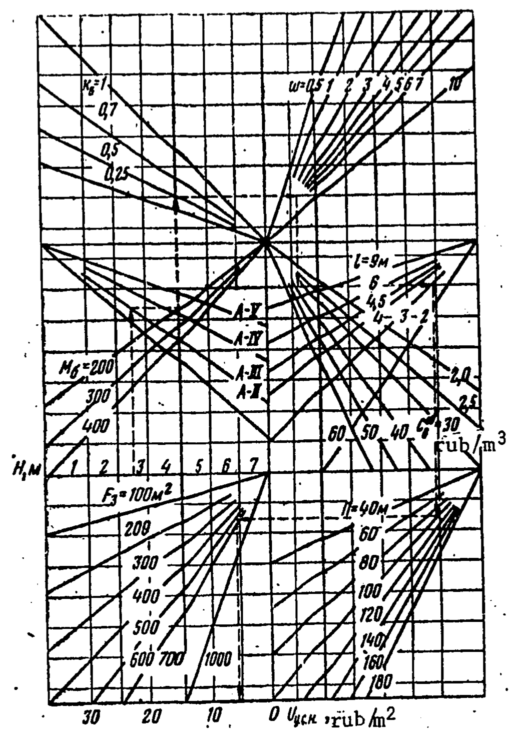


Fig. 101. Nomogram for determining proportionate costs for outer walls of precast reinforced concrete $U_{y.c.H}$

Sample calculation. Determine proportionate costs for outer walls made of precast reinforced concrete elements for Class A-III shelters with effective span l = 3 m; wall height H = 3 m; concrete grade 200; t_{c6} = 1; k_{6} = 0.5; w = 3.5; C_{6}^{c6} = 36 rubles/m³; outer wall perimeter Π = 96 m; protected area F_3 = 540 m². After performing the computation, we will obtain $U_{y.c.H}$ = 6 rubles/m².

minimum thickness of a panel is taken as equal to 10 cm. The factor ε is computed from a well-known formula and takes account of the component's percentage of reinforcement:

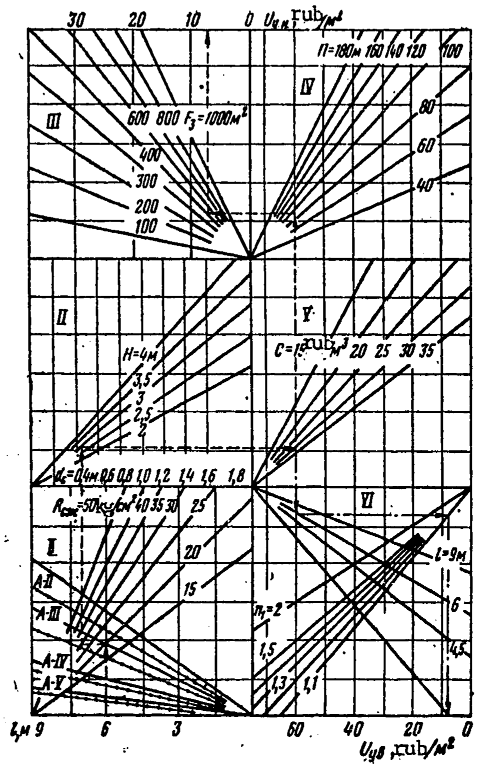


Fig. 102. Nomogram for determining proportionate costs for inner and outer masonry walls

Sample calculation. Determine proportionate costs for erecting outer and inner masonry walls for freestanding Class A-III shelter with a span l = 6 m; $R_{СЖ}$ = 30 kg/cm^2; wall height H = 3 m; walling cost C = 30 rubles/m^3. The I, II, V, and VI quadrants of the nomogram are used for inner walls. With n_1 = 1.1 the proportionate costs for inner walls will equal: $U_{У.В}$ = 8 rubles/m^2. Using quadrants I, II, V, IV and III, for outer walls with structure perimeter П = 100 m and $F_З$ = 540 m^2 we obtain: $U_{У.Н}$= 7 rubles/m^2.

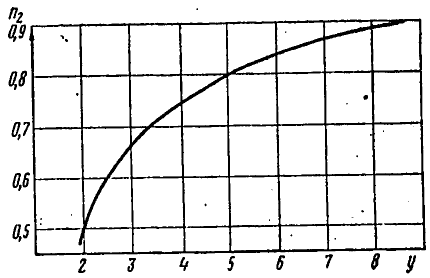


Fig. 103. Chart for determining factor n_2:

y -- Number of spans in length of structure

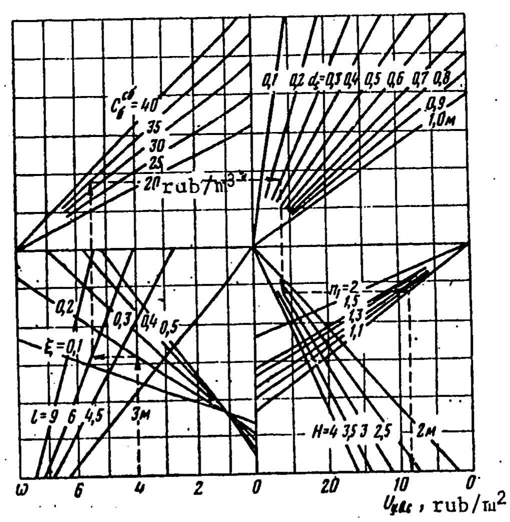


Fig. 104. Nomogram for determining proportionate costs for inner walls of precast reinforced concrete

Sample calculation. Initial data: w = 4; ξ = 0.2; l = 6 m; $C_б^{сб}$= 36 rubles/m^3; d_c = 0.3 m; H = 3 m and n_1 = 1.1. Performing the computations in the order indicated by conditional lines, we obtain $U_{y.B.C}$= 8.8 rubles/m^2.

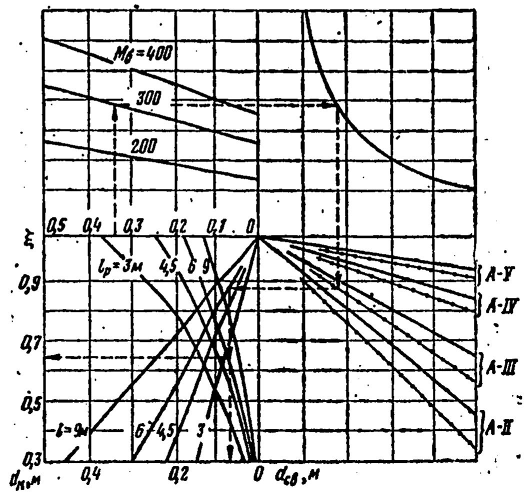


Fig. 105. Nomogram for determining cross section of column and thickness of inner walls of precast reinforced concrete

Sample calculation. Determine thickness of inner walls of precast reinforced concrete and cross-section of columns for a freestanding Class A-III shelter with $\xi = 0.330$; concrete grade 300; span $l = 6$ m. We obtain the answer on the lower scale of quadrant IV (in all cases the minimum wall thickness is taken as equal to 10 cm). For determining the column cross section, we take account of the span in the other direction $l_p = 6$ m and obtain the answer on the vertical scale of quadrant IV $d_K = 0.65$ m.

The values of w are found from the nomogram in Fig. 96 for design dimensions and characteristics of materials obtained and used for inner walls. Results obtained from the nomogram are multiplied by the factor n_2, determined from the chart in Fig. 103.

10. Proportionate costs for inner columns of reinforced concrete in relation to the column cross section and height, span of overhead cover (in both directions), cost of 1 m^3 of concrete "on the job," the factor w and the factor ε, which takes account of percentage of reinforcement, can be determined from the nomogram given in Fig. 106.

The dimensions of a column with square cross section are determined from the nomogram in Fig. 104. The values of factors ε and ω are determined in conformity with

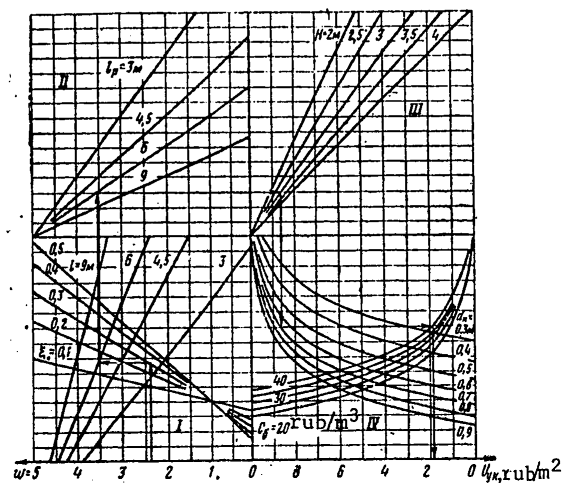


Fig. 106. Nomogram for determining proportionate cost for inner columns of reinforced concrete $U_{y \cdot K}$

Sample calculation. Initial data: w = 2.2; d_K = 0.65 m; ξ = 0.33; 1 = 1_p = 6 m; H = 3 m; C_6 = 38 rubles/m³. With the calculation sequence indicated by arrows, we will obtain for these data $U_{y \cdot K}$= 1.8 rubles/m².

Paragraph 9. The proportion costs obtained for inner columns refer to the primary space occupied, for which these costs must be multiplied by the factors n_1, n_3 or n_4. The values of factors n_3 and n_4 are determined from the chart in Fig. 107, and the factor n_1 is determined in conformity with Paragraph 4.

11. Proportionate costs for erecting columnar foundations are determined from the nomogram in Fig. 108 in relation to the dimensions of column cross sections d_K, the given class of protective structure, grade of concrete, standard pressure on soil σ_{IP}, multiplied by the hardening factor K_y^{IP} , and the cost of reinforced concrete "on the job" based on a reinforcement expenditure of 50 kg-force/m³ of concrete. The costs obtained, just as for the columns, must be multiplied by the factors n_1, n_3 or n_4, determined from Paragraph 10.

Proportionate costs for continuous footings in relation to the given class of protective structure, standard pressure on soil with consideration of hardening, footing height and cost of reinforced concrete "on the job" under inner walls are determined from the nomogram in Fig. 109 (quadrants I-IV) and under outer walls for structures with frameless arrangement and partial frame, from this same nomogram (quadrants I-III, V and VI). The height of continuous footings is taken as 50 cm for a structure of Class A-I with a span of 4.5-6 m and with a span of 6 m

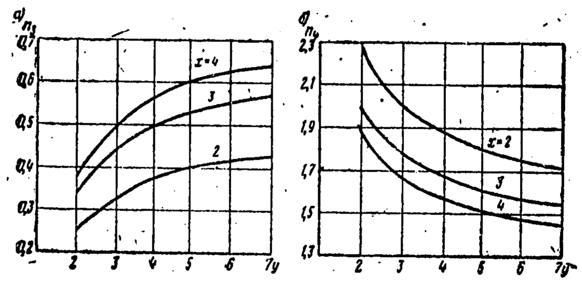


Fig. 107. Chart for determining factors n_3 and n_4

a -- For partial frame structures
b -- For full-frame structures
y -- Number of spans in length of structure
x -- Number of spans in width of structure

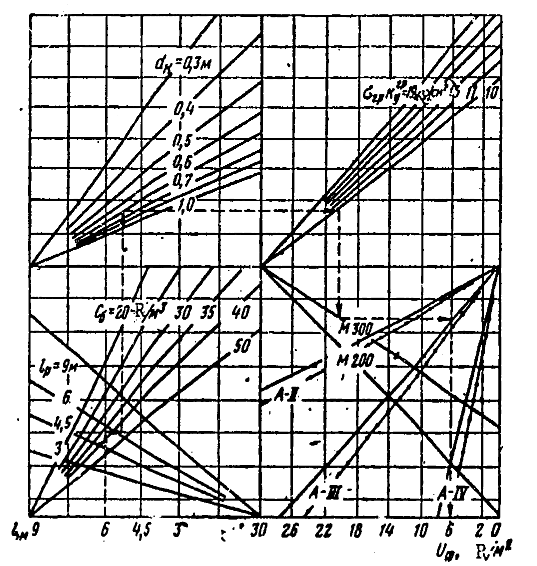


Fig. 108. Nomogram for determining proportionate costs for columnar formations

Sample calculation. Initial data: Freestanding Class A-III shelter; span $l = l_p = 6$ m; cost of concrete $C_6 = 43$ rubles/m^3; $d_K = 0.65$; $\sigma_{rp}k_y^{rp} =$ 10 kg-force/cm^2; concrete grade 300. Performing the computations, we obtain $U_{\Phi.K} = 6$ rubles/m^2.

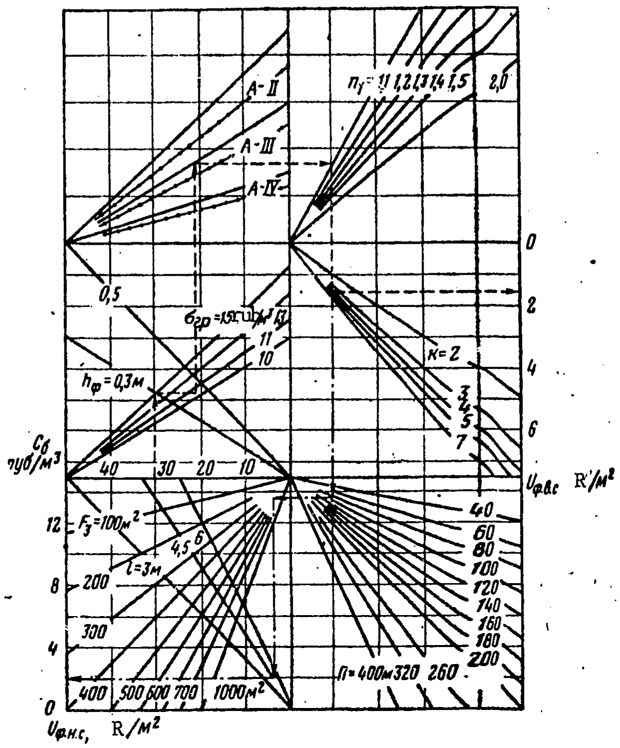


Fig. 109. Nomogram for determining proportionate costs for continuous footings under inner and outer walls U_{Φ}

Sample calculation. Initial data: freestanding Class A-III shelter; footing height h_{Φ} = 0.3 m; $\sigma_{rp}k_y$ = 10 kg-force/cm^2; cost of concrete $C_б$= 30 rubles/m^3; n_1 = 1.1; k = 5; l = 6 m. For footings under inner walls we use quadrants I-IV of the nomogram and obtain the answer from the scale of quadrant IV: $U_{\Phi \cdot B}$= 1.6 rubles/m^2. For footings under outer walls we use quadrants I-III, V and VI with consideration of wall perimeter Π= 100 m and F_3 = 540 m^2. We obtain the answer from the scale of quadrant VI: $U_{\Phi \cdot H}$= 2 rubles/m^2.

for Class A-II. For the remaining classes of structures, the height of continuous footings is taken as 30 cm.

The value k (quadrant IV) considers the number of spans in a structure's length.

The cost of footing under inner walls obtained from the nomogram must be multiplied by the factor n_2, determined from the chart in Fig. 103.

12. Proportionate costs for constructing shelter entrances are determined as the ratio of cost for entrances $U_{B.X}$ to the total protected area of the structure F_3.

$$U_{y.B.X} = \frac{U_{B.X}}{F_3}. \quad (138)$$

Proportionate costs for entrances of various types can be determined from the data of Table 32, which gives the relative change in percentage of total capital costs for constructing one entrance with walls of monolithic reinforced concrete. The capital costs equal to 1100 rubles for a blind entrance with door opening width of 0.8 m and wall height of 2.2 m were taken as 100 percent.

Table 32 - Relative Costs for Various Entrance Types, %

Entrance Types	Door Opening Width, m	
	0.8	1.2
Blind, with stairway along shelter wall	100	130
Blind, with stairway perpendicular to shelter wall	110	140
Through	145	175
Through, with two-chamber sluice	270	360

Note. Entrance evaluation performed with respect to walls of monolithic reinforced concrete 30 cm thick with a cost of 1 m^3 of reinforced concrete "on the job" of 25 rubles.

Costs for one entrance are additional, since they do not include the cost of overhead cover and outer walls with foundations beneath them, which must be considered in Paragraph 3.11.

The cost of airtight blast doors and airtight doors is considered in the calculation with a door cost equal to 321 rubles for 1 ton.

Approximate costs for entrances with walls of other materials can be determined by using the table's data and by introducing corrections proportionate to the change in cost of 1 m^3 of material "on the job." For example, capital costs for entrances with walls of precast concrete blocks, the cost of 1 m^3 of which is taken as 30 rubles "on the job" are increased by 20 percent in comparison with the data given in Table 32.

With entrance wall height of 3 and 4 m, additional capital costs obtained from Table 32 must be increased by 20 percent and 70 percent respectively.

13. Proportionate costs for constructing emergency exits $U_{y.a.B}$ are determined as the ratio of the cumulative costs of exits to the total protected area of structure F_3:

$$U_{y.a.B} = \frac{U_{a.B}}{F_3}. \qquad (139)$$

The tentative costs of an emergency exit from a freestanding buried shelter can be taken as 450-500 rubles.

If an emergency exit is combined with one of the entrances, its cost is taken from the data of Table 32 with the addition of the cost of 1 m of a full-passage gallery with dimensions used in the plan.

14. The space use factor $k_{\Pi Л}$ is an indicator of the economical nature of the overall planning of a protective structure. It can be determined from the following formula:

$$k_{\Pi Л} = \frac{F_3 - F_T}{F_\Pi}, \qquad (140)$$

where F_T is the structure's protected area occupied by industrial equipment which is not dismantled when the shelter is placed in combat readiness;

F_Π is the total protected area required by the norms for shelter capacity provided by the plan.

Theoretically the shelter space use factor is $k_{\Pi Л} = 1$. In actual planning, however, the positioning of individual shelter spaces and its engineering equipment in a certain interrelationship makes it necessary for a certain increase in areas above those covered by the standards.

An analysis of the most economical shelter design decisions indicates that the minimum value of the space use factor $k_{\Pi Л}$ which does not cause substantial degradation in the shelter cost indicators may be 1.05 for structures where the entire protected area F_3 is used as primary and auxiliary.

Sample calculation. The technical-economic estimate of the protected structure and a determination of proportionate costs for individual elements and the structure as a whole is performed using as an example a freestanding Class III shelter of frame-panel design located on nonrocky soil. The number of spans equals five in structure length and three in structure width.

The structure has outer walls of precast reinforced concrete slabs with continuous cross section ($t_{c б} = 1$) with an effective span of 3 m and an inner frame of reinforced concrete columns with bay size of 6 x 6 m.

The height of spaces is 3 m. The overhead cover of flanged components made of precast-monolithic reinforced concrete is continuous. Foundations under the columns are columnar.

Precast concrete of grade 200 ($C_б^{сб}$= 45 rubles/m^3 "on the job") and monolithic concrete of grade 300 ($C_б^{м}$ = 23 rubles/m^3) has been used for slabs of the overhead cover and foundation, $t_{сб}$= 0.5, Φ_a= 1.8. The working reinforcement is Class A-III steel (C_a = 1080 rubles/m^3).

Concrete of grade 300 ($C_б^{сб}$= 45 rubles/ m^3) has been taken for the collar beam, $A_П$= 0.4, $t_{сб}$= 0.37, t_M = 1, Φ_a= 2.2; for the columns Φ_a= 1.5; $t_{сб}$= 1. The working reinforcement for the collar beam is Class A-III steel, and Class A-II for the columns.

From the nomogram in Fig. 96 we find w = 2.5.

The factor $\eta_1 = \frac{C_б^м}{C_б^{сб}} \cdot \frac{t_т}{t_{сб}} = \frac{23 \cdot 1}{45 \cdot 0{,}5} = 1$;

From the found values w and η_3 from the nomogram in Fig. 97 we find the factors η_1 = 3.8 and η_2 = 5.5. The sum of these factors will equal 9.3.

Using the found value of factors with n_1 = 1.1, we find from the nomogram in Fig. 98 the proportionate cost of a slab of a precast-monolithic cover $U_{у.п}$= 21 rubles/m^2.

For determining the proportionate cost of the collar beam, the value w will equal 6 (see Fig. 96). With this value of w and $b = \frac{b_p}{\sqrt{b_{п.р}}} = \frac{0{,}7}{\sqrt{2{,}4}} = 0{,}45$ м, the proportionate costs for the collar beam will be $u_{у.р}$ = 12.7 rubles/m^2.

The cost of outer walls of precast reinforced concrete elements with horizontal bending strain will be determined from the nomogram in Fig. 101. With w = 3.5, $t_{сб}$= 1, the effective slab span l = 3 m, the structure perimeter П= 96 m and a protected area F_3 = 540 m^2, proportionate costs will be $U_{у.с.м}$= 6 rubles/m^2.

Proportionate costs for inner reinforced concrete columns will be found from the nomogram in Fig. 106, $U_{ук}$= 2.6 rubles/m^2, after first determining the dimensions of the column from the nomogram in Fig. 109; d = 0.65 m; w = 2.6 from the nomogram in Fig. 96 and ξ = 0.33 from Paragraph 9. The cost for the structure with full frame which is obtained must be multiplied by the factor 1.6, determined from the chart in Fig. 107.

With consideration of the fact that the effective slab span for outer walls is taken as equal to l = 3 m, columns are additionally installed every 3 m around the perimeter of the structure and so the factor n_4 = 1.6 x 2 = 3.2. Then the proportionate column cost will equal: $U_{у.к} = 2{,}6 \cdot 1{,}1 \cdot 3{,}2 = 9{,}15$ R/$м^2$.

Costs for erecting columnar foundations, from the nomogram in Fig. 108, equal $U_{Ф.С}$= 6 rubles/m^2.

With a doubling of the number of columns around the perimeter for the calculated case, factor n_2 is increased 1.3 times, since the foundation of the additional columns is calculated only for the lateral load. For this reason the proportionate costs for columnar foundations will equal: $U_{Ф.с} = 6 \cdot 1{,}1 \cdot 1{,}6 \cdot 1{,}3 = 13{,}7$ R/$м^2$.

Proportionate costs for constructing entrances will be determined from Paragraph 12:

$$U_{y.в.x} = \frac{U_{в.x}}{F_з}.$$

The structure has a through entrance with double-chamber sluice and one blind entrance. According to Table 32, the cumulative proportionate costs of these entrances equals:

$$U_{y.в.x} = \frac{1100 \cdot 2,7 + 1100}{540} = \frac{4080}{540} = 7,56 = 7,6\ R/м^2.$$

With this planning decision, an emergency exit is combined with a blind entrance, and so is not considered in determining the proportionate cost.

Cumulative proportionate costs for erecting structural elements of a Class III shelter frame with the conditional values of material and components' cost taken in the example are 70 rubles/m^2.

The actual cost must be determined in relation to the actual cost of materials and components "on the job" with consideration of zone factors.

These costs do not consider the cost of floors, supplementary internal walls and partitions, waterproofing, earthwork or finishing work. According to available consolidated indicators of certain projects of a similar purpose, the cost for this work, depending on the shelter capacity, may comprise 25-35 percent of the frame cost for shelters holding 1,000 persons and up to 35-50 percent for shelters holding 900 or fewer persons.

Takeing the costs for this work equal to 50 percent of the frame cost for our example, we obtain cumulative proportionate costs for construction and installation work for erecting the shelter frame to be tentatively 70·1.5 = 105 rubles/ 1 m^2 of protected area.

Considering the overhead expenses and planned savings, the cost of the structure frame will equal 126 rubles/m^2.

Appendix 3 - Determination of Vertical Displacements of Structure with Respect to the Ground

Vertical displacements of the entire structure or a portion (block) with respect to the ground may be determined from the formula

$$Z_{отн}=\frac{10^7 p_1}{a_1\gamma}\left\{\frac{F_п}{F_ф}\left[\frac{3H_c}{a_1}-\frac{G}{a_1\gamma F_ф}\left(1-e^{-\frac{3\gamma F_ф H_c}{G}}\right)\right]-\frac{2,7\,H_c}{a_1}\right\}, \qquad (141)$$

where Z_{OTH} is the vertical displacement of the structure (block) with respect to the ground, in centimeters;

p_1 is the dynamic load, kg/cm^2 (see Paragraph 3.5);

a_1 is the propagation rate of plastic deformations in the soil, m/sec (see Table 9);

γ is the volumetric weight of soil, kg/m^3;

$F_П$ is the area of the structure's (block's) overhead cover, m^2;

F_ϕ is the area of the base of foundation of the structure (block), m^2;

H_c is the structure's height from top of overhead cover to underlying layer, m;

G is the cumulative weight of structure (block) and embankment above it in kg;

$$e = 2.718.$$

In determining the displacement of a portion of the structure (block), the value $F_П$ should be taken as equal to the area of overhead cover from which the load is collected, which is transmitted to the foundation area of the portion of the structure (block) in question.

In a number of cases the result of a determination of the value Z_{OTH} may turn out to be negative. This attests to the fact that the vertical displacement of soil particles is greater than the displacement of the structure or its block.

Example. Let us determine the relative settling of a foundation under a column of a freestanding Class A-III structure with bay size of 6 x 6 m. We will take the following initial data: $p_1 = 2\ kg/cm^2$; $a_1 = 200$ m/sec; $\gamma = 1600\ kg/m^3$; $F_П = 36\ m^2$; $F_\phi = 4.7\ m^2$; $H_c = 4$ m. Volume of concrete per 1 m^2 of overhead cover $V_б = 0.65\ m^3$. Height of soil above overhead cover is 0.8 m. Weight of foundation beneath column is $G\phi$ = 12,000 kg-force.

In this case the weight of the structure block will include:

Weight of overhead cover with an area of 36 m^2;

Soil weight beneath overhead cover, column weight and foundation weight.

The weight of overhead cover together with column weight will be found from the formula

$$G_б=\gamma_б V_б F_п,$$

where $\gamma_б$ is the volumetric weight of concrete.

$$G_б = 2200\cdot 0,65\cdot 36 = 52\,000 \text{ kg}.$$

Weight of soil above overhead cover is $G_{гр}=\gamma h_{гр} F_п = 1600\cdot 0,8\cdot 36 = 46\,000$ kg.

Cumulative weight of the block is $G = G_б + G_{гр} + G_ф = 52\,000 + 46\,000 + 12\,000 = 110\,000$ kg.

Let us substitute all initial data into the formula for determining relative settling:

$$Z_{\text{отн}} = \frac{10^7 \cdot 2}{200 \cdot 1600} \left\{ \frac{36}{4{,}7} \left[\frac{3 \cdot 4}{200} - \frac{11 \cdot 10^4}{200 \cdot 1600 \cdot 4{,}7} \left(1 - e^{-\frac{3 \cdot 1600 \cdot 4{,}7 \cdot 4}{11 \cdot 10^4}} \right) \right] - \frac{2{,}7 \cdot 4}{200} \right\} = 62{,}5 \{ 7{,}65 [0{,}06 - 0{,}073 (1 - e^{-0{,}82})] - 0{,}054 \} = 5{,}7 \text{ см}.$$

Appendix 4 - Determination of Overpressure in Shock Front and Effective Time of Action in Explosion of Gas-Air Mixtures

Three action zones are distinguished in the explosion of a gas-air mixture (GVS) (Fig. 110): Action zone of the detonation wave within the GVS cloud, action zone of GVS explosion products, and action zone of the air shock wave. Parameters of the GVS explosion (pressure in the front and effective action time of the shock wave) depend chiefly on the zone in which the shelter will be located, on the distance from the explosion center and on the composition of the GVS. The formulas given below correspond to averaged physical-mechanical and energy characteristics of a stoichiometric mixture with air of hydrocarbon gases of the type C_mH_n (methane, ethane, propane, butane, ethylene, propylene, pentane, butylene) and an idealized schematic of the explosion (detonation) of a GVS cloud in the form of a hemisphere with an initiated burst in its center.

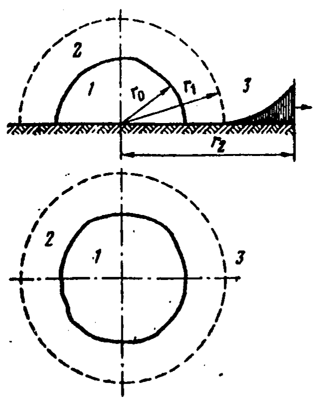


Fig. 110. Schematic of explosion. of a cloud of gas-air mixture (GVS)

1 -- Action zone of detonation wave within GVS cloud
2 -- Action zone of products of GVS explosion
3 -- Action zone of air shock wave
r_0 -- Initial radius of GVS cloud
r_1 -- Limiting radius of explosion products' dispersion
r_2 -- Distance to center of GVS explosion

Zone of Gas-Air Mixture Cloud

The initial radius of the GVS cloud equals: $r_0 = 17{,}5\sqrt[3]{Q}$, (142)
where Q is the amount of stored compressed hydrocarbon gases prior to the explosion, tons-force.

Acting in this zone is the detonation wave, with an overpressure in its front constantly within the CVS cloud and taken equal to $\Delta p_Д = 17$ kg/cm^2.

The effective action time θ of the detonation wave is determined from the formula

$$\theta = 0{,}47 \cdot 10^{-3}\, r_0 (1 + 0{,}4 r/r_0), \quad r \leqslant r_0, \qquad (143)$$

where r_0 and r represent the initial radius of the GVS cloud and the distance to the burst center, in meters.

When the detonation wave reflects from an obstacle, with the component located perpendicular to the direction of propagation of the detonation wave, pressure on the obstacle exceeds pressure in the detonation wave front by approximately 2.5

times and the effective action time of reflected overpressure is determined from the formula

$$\theta = 0{,}195 \cdot 10^{-3} r_0 (1 + 1{,}28 r/r_0), \quad r \leqslant r_0, \tag{144}$$

where r_0 and r are in meters.

Action Zone of GVS Explosion Products

This zone is limited by the radii $r_0 \leqslant r \leqslant r_1$, where r_1 is the limiting radius of dispersion of the explosion products (PV);

$$r_1 \approx 1{,}7 r_0. \tag{145}$$

Pressure in the shock front and effective action time depend on the distance and are determined from the formulas:

$$\Delta p_\Phi = 13 (r_0/r)^3 + 0{,}5, \quad r_0 < r \leqslant r_1; \tag{146}$$

$$\theta = 2{,}1 \cdot 10^{-4} r_0 \sqrt{(r/r_0)^5}, \quad r_0 < r \leqslant r_1, \tag{147}$$

where r and r_0 are in meters.

Action Zone of Air Shock Wave

The third zone begins at distances $r_2 \geqslant r_1 = 1{,}7 r_0$. Pressure in the front of the air shock wave is determined from the formulas:

$$\Delta p_\Phi = \frac{7}{3\left(\sqrt{1 + 29{,}8 R_2^3} - 1\right)}, \quad R_2 \leqslant 2; \tag{148}$$

$$\Delta p_\Phi = \frac{0{,}227}{R_2 \sqrt{\lg R_2 + 0{,}158}}, \quad R_2 > 2. \tag{149}$$

In these formulas R_2 is the dimensionless radius of the shock wave;

$$R_2 = 0{,}24 r_2/r_0, \quad r_2 \geqslant 1{,}7 r_0, \tag{150}$$

where r_2 is the distance to the center of the GVS explosion.

The effective action time of the air shock wave is determined from the formula

$$\theta = \frac{2{,}5 \cdot 10^{-4} r_2}{\Delta p_\Phi R_2^2}, \tag{151}$$

or, after substituting the value R_2, from the formula

$$\theta = \frac{4{,}34 \cdot 10^{-3} r_0^2}{\Delta p_\Phi r_2}. \tag{152}$$

In these formulas r_0 and r_2 are in meters and Δp_Φ is in kg/cm^2.

Parameters of the GVS explosion in all three zones also can be determined using the nomogram in Fig. 111.

Examples of Determining Parameters of GVS Explosion

We will determine from the nomogram and calculate from formulas the values of overpressure in the shock front and effective action time in all three zones in the burst of a GVS cloud formed during destruction of a container holding 1500 tons-force of compressed propane.

From the upper left quadrant of the nomogram for Q = 1500 tons-force we find on the ordinate axis the initial radius r_0 = 200 m. We will obtain the very same results from formula (142): $r_0 = 17{,}5 \sqrt[3]{1500} = 200$ м.

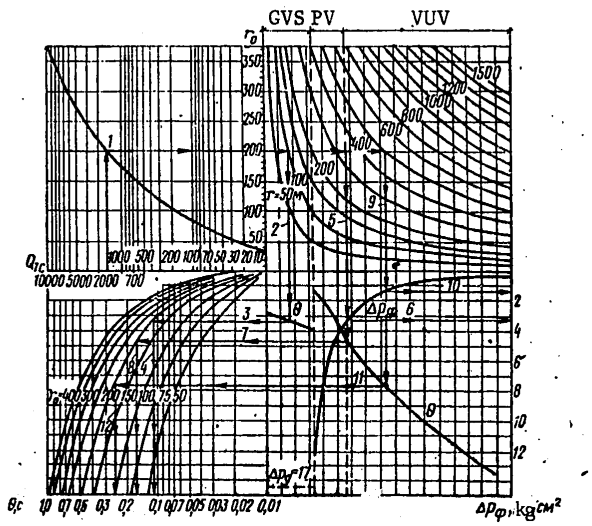


Fig. 111. Nomogram for determining pressure in the front and effective action time of shock wave of a gas-air mixture burst in relation to the distance to the burst center and amount of hydrocarbon gases in storage tanks

In the GVS zone at a distance r = 100 m from the burst center, the action time of the detonation wave with pressure in the front of $\Delta p_Д = 17$ kg/cm^2 will be, from formula (143), $\theta = 0{,}47 \cdot 10^{-3} \cdot 200\,(1 + 0{,}4 \cdot 100/200) = 0{,}113$ sec.

The determination of θ with r = 100 m is shown in the nomogram by arrows (1-2-3-4) and we read the answer on the abscissa axis in the lower left quadrant.

Reflected pressure acts on a structure located perpendicular to the direction of the detonation wave's propagation, with a maximum value equal to:

$$\Delta p_{отр} = 2{,}5\Delta p_Д = 2{,}5 \cdot 17 = 42{,}5 \text{ kg/см}^2,$$

and the effective action time from formula (144) equals:

$$\theta_{отр} = 0{,}195 \cdot 10^{-3} \cdot 200\,(1 + 1{,}28 \cdot 100/200) = 0{,}064 \text{ sec}.$$

The action zone of explosion products is limited by the dispersion radius $r_1 = 1.7 \cdot 200 = 340$ m. The boundaries between the GVS and PV zones and between the PV and action zone of the air shock wave (VUV) are shown in the nomogram by vertical broken lines in the upper right quadrant and are continued in the lower quadrant. The pressure $\Delta p_ф$ acts at the boundary of the dispersion zone of PV with $r = r_1 = 340$ m, and its value from formula (146) equals:

$$\Delta p_ф = 13\,(200/340)^3 + 0{,}5 = 3{,}15 \text{ kg/см}^2.$$

We will find the very same value from the nomogram (arrows 1-5-6) on the vertical scale $\Delta p_ф$, using the curve denoted by $\Delta p_ф$ in the lower right quadrant. The effective action time from formula (147) is $\theta = 2{,}1 \cdot 10^{-4} \cdot 200 \sqrt{1{,}75} = 0{,}158$ sec.

The sequence of determining this time is shown in the nomogram by arrows 1-5-7-8.

We determine parameters of the air shock wave with r_2 = 500 m in its zone of action from formulas (148) and (152) with the dimensionless radius of the shock wave $R_2 = 0.24\frac{50}{200} = 0.6$:

$$\Delta p_\Phi = \frac{7}{3\left(\sqrt{1+29{,}8\cdot 0{,}6^3}-1\right)} = 1{,}35 \text{ kg/cm}^2;$$

$$\theta = \frac{2{,}5\cdot 10^{-4}\cdot 500}{1{,}35\cdot 0{,}6^3} = 0{,}258 \text{ sec.}$$

In the nomogram the determination of Δp_Φ is shown by arrows 1-9-10, and that of θ by arrows 1-9-11-12.

Using the nomogram we can determine quickly the distance from the geometric center of areas with tanks of compressed hydrocarbon gases a shelter should be located with a given pressure acting on it.

Appendix 5 - Equivalent Static Loads from Effect of Inertial Forces on Internal Components and Devices for Fastening Internal Shelter Equipment

1. Inner walls and columns of shelters are calculated for normal forces from a load transmitted from the overhead cover and for a horizontal equivalent static load from inertial forces generated by displacement of element supports during structure movement, and inner partitions are calculated for the load from inertial forces.

2. Overhead cover of shelters which are intermediate floors are calculated for the vertical load from inertial forces and separately for normal forces transmitted from outer walls.

3. The equivalent static load $q_{ЭКВ}$ on internal components from the effect of inertial forces generated by the displacement of element supports during shelter movement is determined from the following formulas:

For vertical elements (internal walls, columns, partitions)

$$q_{экв} = G_к\, j_г\, \eta_{уд}\, k_д; \tag{153}$$

For horizontal elements (intermediate floors which are overhead cover)

$$q_{экв} = G_к\,(j_в\, \eta_{уд}\, k_д - 1). \tag{154}$$

Here G_K is the weight of a unit length (area) of the calculated component;
$j_Г$, $j_В$ is the maximum pulse amplitude of acceleration of component supports, m/sec^2 (indexes: "Г" -- horizontal, "В" -- vertical;
$\eta_{уд}$ is the coefficient of acceleration transfer to the calculated component;
$k_Д$ is the dynamic-response factor determined from the chart in Fig. 112 in relation to $\omega\tau$, where τ -- see formula (155), and ω is the cyclic free frequency of the calculated element.

4. The maximum vertical acceleration amplitude in "g" units is determined from the formulas:

For freestanding shelters

$$j_в = \frac{k^*_{отр}\,\sigma_m\cdot 10^4}{(A_п + kA_ф)\,\tau g}\left(1 - e^{-\frac{A_п + kA_ф}{m_в}\tau}\right), \tag{155}$$

where τ is the build-up time of acceleration amplitude to maximum in seconds, taken as equal to the build-up time of the load on the cover to a maximum value determined from formula (19), Paragraph 3.6.

For built-in shelters

$$j_в = \frac{\Delta P_{max}\cdot 10^4}{kA_ф\,\tau g}\left(1 - e^{-\frac{kA_ф}{m_в}\tau}\right), \tag{156}$$

where τ is taken as equal to: 0.15 sec for Class V shelters; 0.09 sec for Class IV; 0.06 sec for Class III and 0.04 sec for Class II shelters.

In formulas (155) and (156) g is the acceleration of gravity in m/sec^2; $k^*_{отр}$ is the reflection factor of the compression wave from the cover, determined from the chart in Fig. 63; σ_m is the maximum pressure in the compression wave at the level of the top of cover, kg/cm^2; Δp_{max} is the maximum pressure of the air shock wave on the cover of a built-in shelter, kg/cm^2; $k = F_ф/F_п$ is the ratio of the area F_ϕ of the foundation base to the area F_Π of the structure's cover; A_Π and A_ϕ are the acoustical resistances of the fill soil above the cover and under the foundation expressed by the formula $A = a_1\rho$, in which a_1 is the rate of propagation (m/sec) of elasticoplastic compression waves in the soil; ρ is the soil density (kg·sec²/m⁴) (see Table 9); m_B is the structure weight per 1 m² of base area, kg·sec²/m³.

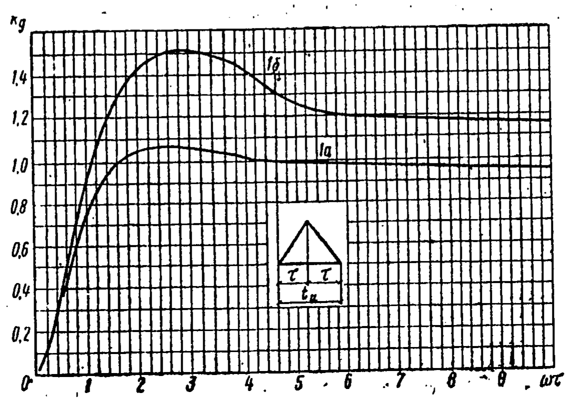


Fig. 112. Chart for determining dynamic-response factor when calculating components for inertial forces

1b -- For Case 1b
1a -- For Case 1a

5. The maximum horizontal acceleration amplitude in g units is determined from an approximation formula (without considering frictional forces on the foundation base)

$$j_r = \frac{p_{max}\cdot 10^4}{2A_c \tau g}\left(1 - e^{-\frac{2A_c\tau}{m_r}}\right), \qquad (157)$$

where p_{max} is the maximum value of the horizontal dynamic load on a shelter wall at the level of the mid-height of a wall, kg/cm² (determined in accordance with paragraphs 3.6-3.7);

A_c is the acoustical resistance of soil at the shelter walls, kg·sec/m³;

m_Γ is the mass of the structure per 1 m² of area of the shelter's vertical face receiving a horizontal load kg·sec²/m³; the mass of continuous footings (under the walls) and freestanding foundations (under the columns) is not considered in determining m_Γ.

The build-up time in acceleration amplitude to maximum is taken as equal to the lesser of two values determined from the formula

$$\tau = \left\{\begin{array}{l} \dfrac{H}{a_1}; \\ \dfrac{L}{D_\phi}, \end{array}\right\} \qquad (158)$$

where H is the shelter height, m;
L is the shelter dimension in the movement direction of the shock front, m;
a_1 is the velocity of elasticoplastic waves in the soil, m/sec (see Table 9);
D_ϕ is the velocity of the shock front, m/sec (see Table 11).

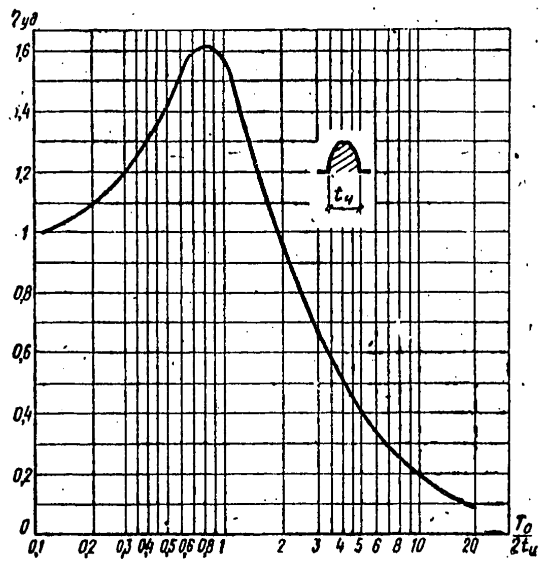


Fig. 113. Chart for determining coefficient of acceleration transmission to calculated component

6. The coefficient of acceleration transmission to the calculated component is determined from the chart in Fig. 113 in relation to the ratio of the half-period ($T_0/2$) of the component's natural oscillation to the duration $t_И$ of the acceleration impulse of component supports. The duration of acceleration impulse is taken as equal to

$$t_и = 2\tau. \qquad (159)$$

7. Stresses arising in the parts of rigid fastenings of internal equipment to components are determined from the formula

$$p_{экв} = G j \eta_{vд} k_д, \qquad (160)$$

where G is the equipment weight;
j is the maximum amplitude of the acceleration impulse of a component in the corresponding direction (determined from paragraphs 4 and 5 of this appendix);
$\eta_{vД}$ is the coefficient of acceleration transmission to the component to which the equipment is fastened;
$k_Д$ is the dynamic-response factor, taken equal to 1.2.

8. *Example.* A freestanding shelter has plan view dimensions of 42 x 60 m; height of walls (from top of cover to foundation) is 4.6 m; foundations under walls are continuous footings, and under columns are freestanding; total weight of the structure is 10,000 tons-force, including foundation weight (2,000 tons-force). The ratio of the foundations' area to the cover area is $k=F_\Phi/F_n=0{,}25$. the soil is fill loam: a_1 = 150 m/sec; a_0 = 250 m/sec; ρ = 160 kg·sec^2/m^4;

$k_б$ = 0.5. Embankment thickness is 1 m. For undisturbed load (under foundations): a_0 = 700 m/sec; a_1 = 350 m/sec;

ρ = 170 kg·sec^2/m^4. Pressure at the front of a passing shock wave $\Delta p\phi$ = 2 kg/cm^2; Dϕ = 555 m/sec. It is necessary to determine the vertical and horizontal loads and the equivalent static load on a reinforced brick partition with a thickness of one brick grade 100 with mortar grade 25, with a height of 360 cm.

Structure weight per 1 m^2 of base area $m_в=\dfrac{10\,000\cdot10^3}{(42\cdot60)\,9{,}81}=405$ kg·sec^2/m^3.

We will determine horizontal acceleration in the direction of the shelter's lesser side; consequently, the weight per 1 m^2 of wall is $m_r=\dfrac{8000\cdot10^3}{(60\cdot4{,}6)9{,}81}=2960$ kg·sec^2/m^3.

Let us determine the values included in formula (155) for vertical acceleration: $\sigma_m=\Delta p_\Phi=2$ kg/cm^2; taking $\sigma_*=1{,}5$ kg/cm^2, from the chart in Fig. 63 with the values $\sigma_m/\sigma_*=2/1{,}5=1{,}33$ and $a_0/a_1=\dfrac{250}{150}=1{,}7$ we find $k^*_{отр}$= 1.68.

From formula (19):
$$\tau=\frac{1}{150}\left(1-\frac{150}{250}\right)=0{,}003\ \text{sec}.$$

The soil acoustical resistances equal:

Embankment and at walls $A_n=A_c=150\cdot160=2{,}4\cdot10^4$ kg·sec/m^3;
Under the foundation $A_\Phi=350\cdot170=6\cdot10^4$ kg·sec/m^3.

Maximum vertical acceleration amplitude is
$$j_в=\frac{1{,}68\cdot2\cdot10^4}{(2{,}4\cdot10^4+0{,}25\cdot6\cdot10^4)\,0{,}003\cdot9{,}81}\times\left[1-e^{-\frac{(2{,}4+0{,}25\cdot6)\cdot10^4}{405}\cdot0{,}003}\right]=29{,}2\,(1-0{,}748)=7{,}4.$$

The amplitude build-up time to maximum for horizontal acceleration in formula (158):
$$\tau=\begin{cases}4{,}6/150=0{,}031\ \text{sec}\\ 42/555=0{,}076\ \text{sec}\end{cases}\quad \tau_{min}=0{,}031\ \text{sec}.$$

The horizontal load on a wall: $p_{max}=k_б\Delta p_\Phi=0{,}5\cdot2=1$ kg/cm^2. The amplitude of horizontal acceleration in g units from formula (157) is
$$j_r=\frac{1\cdot10^4}{2\cdot2{,}4\cdot10^4\cdot0{,}031\cdot9{,}81}\left(1-e^{-\frac{2\cdot2{,}4\cdot10^4\cdot0{,}031}{2960}}\right)=0{,}69\,(1-0{,}6)\approx0{,}28.$$

We take the hinged fastening of a partition in height. With a modulus of deformation of the masonry
$$E=0{,}8E_0=0{,}8\,(1000\cdot13)\approx10\,000\ \text{kg/cm}^2$$
$$I=\frac{bh^3}{12}=\frac{1\cdot25^3}{12}=1300\ \text{cm}^4.$$

Oscillation frequency is
$$\omega=\frac{\pi^2}{360^2}\sqrt{\frac{10\,000\cdot1300}{40{,}7\cdot10^{-6}}}=43\ \text{sec}^{-1}$$

(here $40{,}7\cdot 10^{-6}\frac{\text{kg}\cdot\text{sec}^2}{\text{см}^2}$ is the linear mass of the partition).

Duration of the horizontal acceleration impulse from formula (159) equals $t_и=2\tau=2\cdot 0{,}031=0{,}062$ sec. The natural oscillation period of the partition is $T_0=\frac{2\pi}{\omega}=\frac{2\cdot 3{,}14}{43}=0{,}146$. With the ratio $T_0/2t_и=\frac{0{,}146}{2\cdot 0{,}062}=1{,}18$ we find from the chart of Fig. 113 the coefficient of acceleration transmission to the partition $\eta_{уд}=1.5$. The dynamic-response factor with $\omega\tau=43\cdot 0{,}031=1{,}33$ from Fig. 112 (curve 1b) equals $k_д=1.2$.

The weight of a unit length of partition with $\gamma_{кл}=1{,}6\cdot 10^{-3}$ kg/cm^3 is $G_к=1\cdot 25\cdot 1{,}6\cdot 10^{-3}=0{,}04$ kg/cm. From formula (153) the equivalent static load per unit length of partition with width of loaded zone b = 1 cm equals:

$$q_{экв}=0{,}04\cdot 0{,}28\cdot 1{,}5\cdot 1{,}2=2\cdot 10^{-2}\ \text{kg}/\text{см}^2.$$

Appendix 6 - Determination of Optimum Percentage of Reinforcement

When calculating bending precast and precast-monolithic reinforced concrete components of protective structures for a special load combination (for Case 1a), the optimum percentage of reinforcement of components $\mu_{ОПТ}$ is recommended for determination based on values of the factor $\alpha_{ОПТ}$, obtained from the nomogram in Fig. 114.

$$\mu_{опт} = \alpha_{опт} \frac{R_и}{R_а}. \qquad (161)$$

With $\mu = \mu_{опт}$ the sections of reinforced concrete elements will be most economical for cost figures. The following notations are used in the nomogram:

$R_И$, R_a -- Strength indicators of materials (concrete, steel) from tables 9 and 11 of SN 405-70. The strength of the compressed sectional zone of concrete is taken in calculating precast-monolithic components;

C_a is the cost of 1 m^3 of reinforcement (on the job) (with the construction of reinforcement cages and meshes and their placement);

$C_б$ is the cost of 1 m^3 of precast concrete "on the job";

Φ_a is the reinforcement use factor equal to the ratio of the weight of all the element's reinforcement to the weight of the longitudinal working reinforcement.

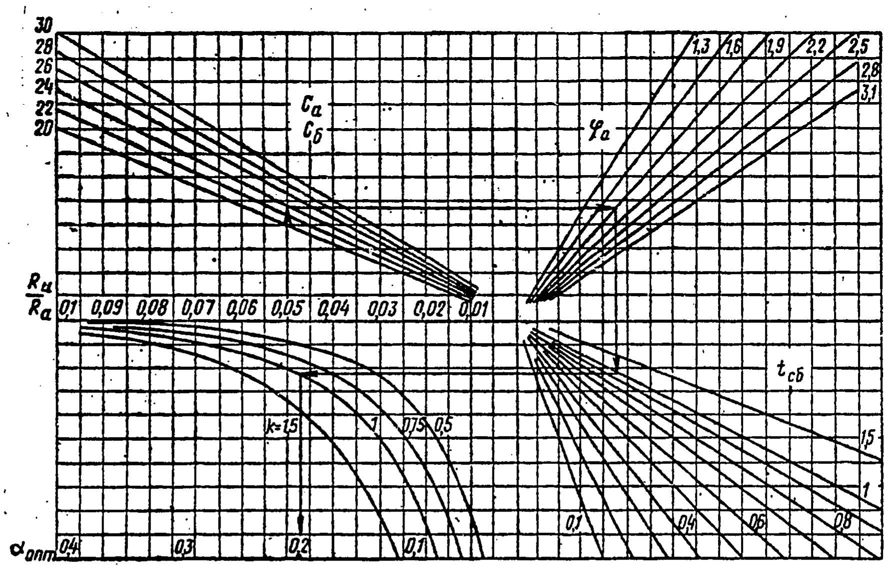


Fig. 114. Nomogram for determining optimum value of coefficient a

Tentative values of coefficient Φ_a can be assumed: 1.4 for precast slabs of continuous sections; 1.5 for hollow slabs; 1.8 for flanged slabs; 1.6 for collar beams with rectangular sections; and 1.6 for continuous precast-monolithic slabs of overhead cover; $t_{сб}$ is the section coefficient of a precast element determined from the formula

$$t_{сб} = \frac{F}{bh}, \qquad (162)$$

where F is the area of concrete in the element's cross section;
b is the width of the concrete's compression zone;
h is the height of the section.

For components with a continuous rectangular section $t_{c6} = 1$; for T sections with a flange in the compression zone, and for hollow sections $t_{c6} < 1$; for T sections with flange in the tensile zone $t_{c6} > 1$.

In precast-monolithic overhead cover the collar beam section in the span, when determining t_{c6}, should be taken in the form of a Tee with width of the section's compression zone equal to b_{Π} (flange width). The optimum percentage of reinforcement found from formula (161) relates to b_{Π}.

Curves for determining optimum values of the coefficient $a_{o\Pi T}$ are given for different values of the parameter k, determined from the formula

$$k = \frac{C_{6.M}}{C_6 t_{c6}} \qquad (163)$$

where $C_{6.M}$ is the cost of 1 m^3 of monolithic concrete "on the job." In determining the coefficient $a_{o\Pi T}$ for precast reinforced concrete cover, the value of coefficient k is taken equal to 1.

Appendix 7 - Charts for Determining Dynamic-Response Factors:
Basic Symbols

Solid curves -- dynamic-response factors for displacements of supports $k_{\Pi}^{O\Pi}$;

Broken curves -- dynamic-response factors for displacements of spans $k_{\Pi}^{\Pi p}$;

k_{Π} -- dynamic-response factor for displacements;

$k_M^{O\Pi}$ -- dynamic-response factor bending moment of supports;

$k_M^{\Pi p}$ -- dynamic-response factor for bending moment of spans;

θ -- effective action time of shock wave [formula (17)];

ω_i -- cyclic natural oscillation frequency;

ω_1^H -- cyclic natural oscillation frequency of central spans of continuous beams;

ω_2^H -- cyclic natural oscillation frequency of beam with one hinged support and the other support fixed (end spans of continuous beam);

β_1 -- factor determined from formula (107);

β_2 -- factor determined from formula (112).

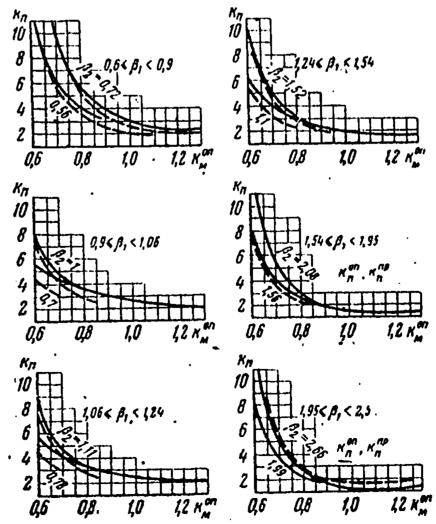


Fig. 115. Fixed beam. Class A-I, A-II, A-III steel; $\omega_1\theta$ = 25

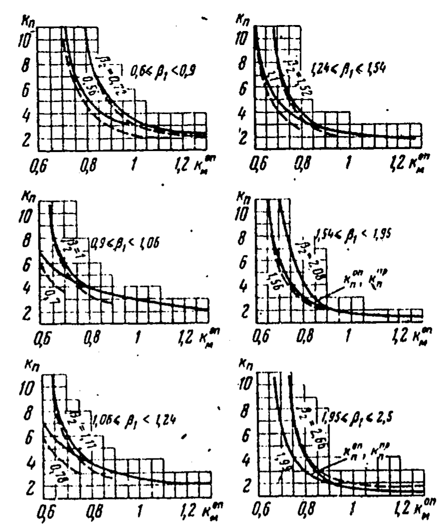


Fig. 116. Fixed beam. Class A-I, A-II, A-III steel; $\omega_1\theta$ = 50

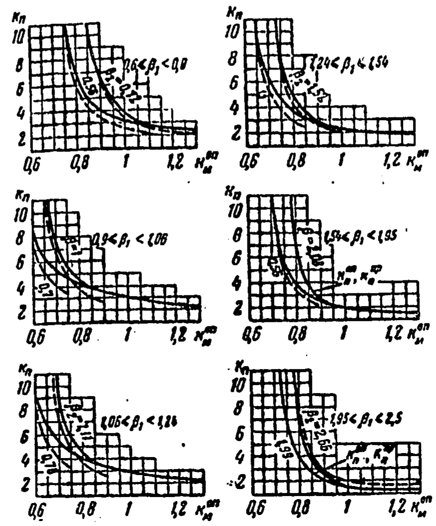


Fig. 117. Fixed beam. Class A-I, A-II, A-III steel; $\omega_1\theta$ = 100

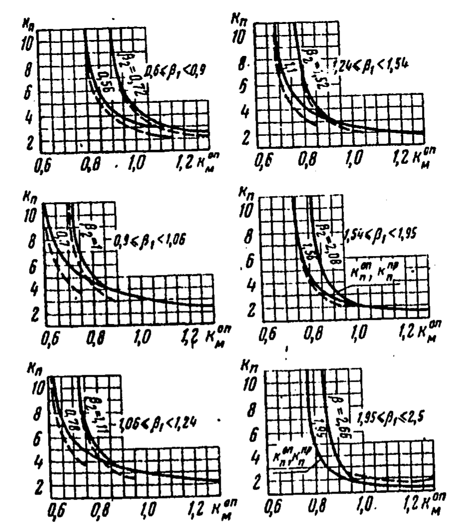


Fig. 118. Fixed beam. Class A-I, A-II, A-III steel; $\omega_1\theta$ = 500

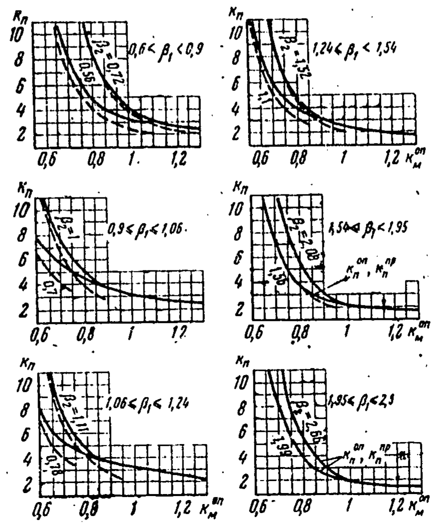


Fig. 119. Fixed beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1\theta$ = 25

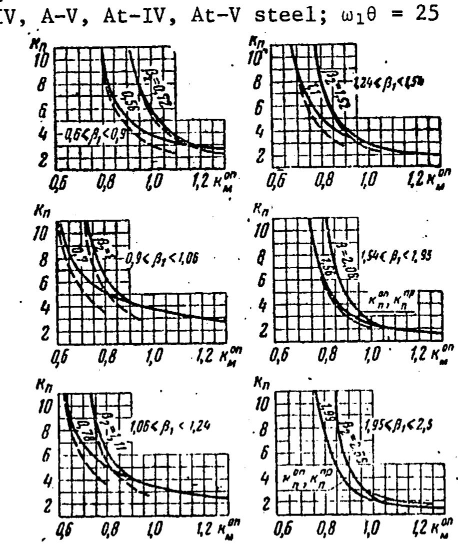


Fig. 120. Fixed beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V; $\omega_1\theta$ = 50

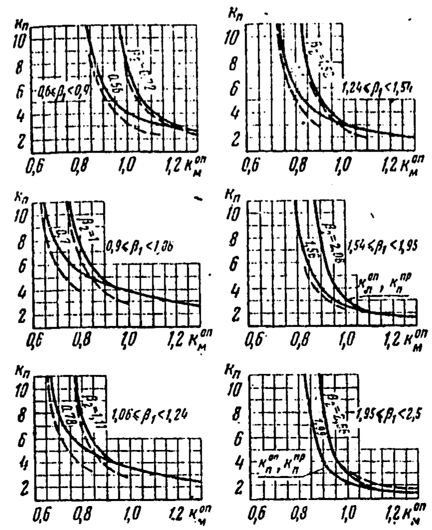


Fig. 121. Fixed beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1\theta$ = 100

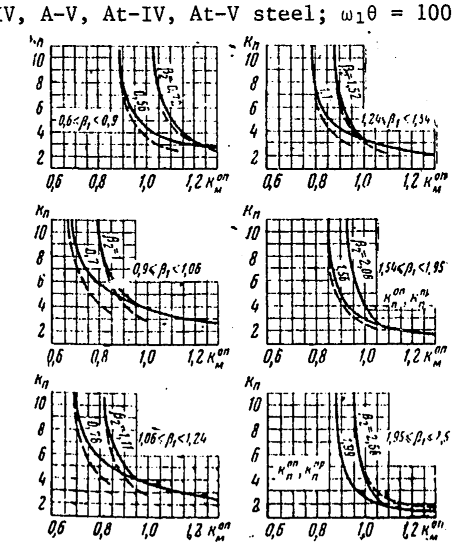


Fig. 122. Fixed beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1\theta$ = 500

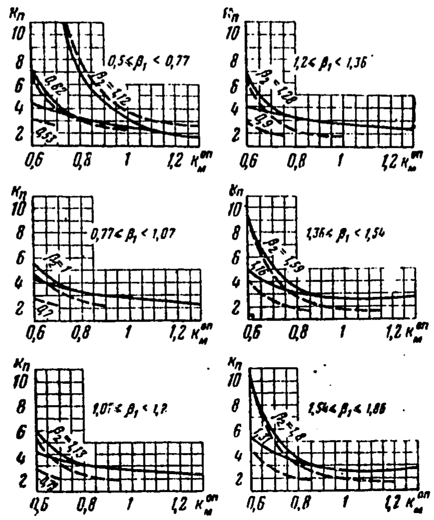


Fig. 123. Continuous double-span beam with supported ends. Class A-I, A-II, A-III steel; $\omega_2''\theta = 25$

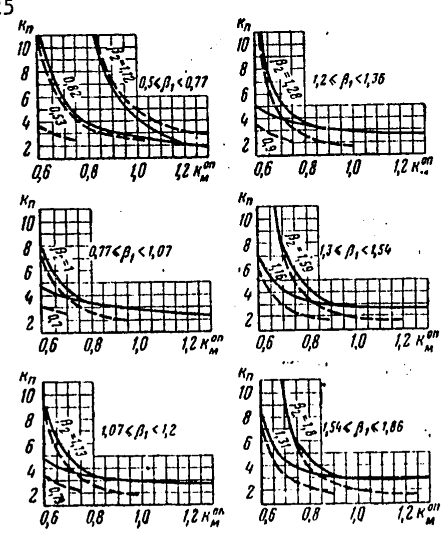


Fig. 124. Continuous double-span beam with supported ends. Class A-I, A-II, A-III steel; $\omega_2''\theta = 50$

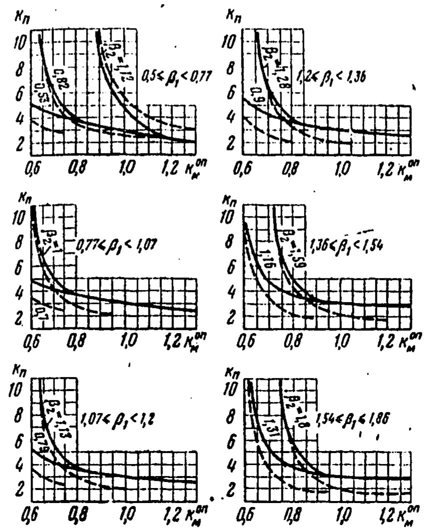


Fig. 125. Continuous double-span beam with supported ends. Class A-I, A-II, A-III steel; $\omega_2^{H}\theta$= 100

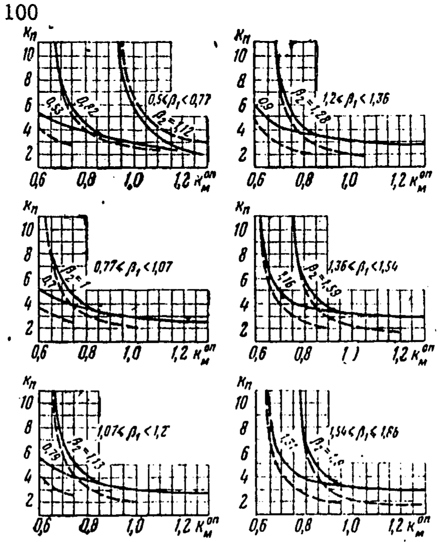


Fig. 126. Continuous double-span beam with supported ends. Class A-I, A-II, A-III steel; $\omega_2^{H}\theta$= 500.

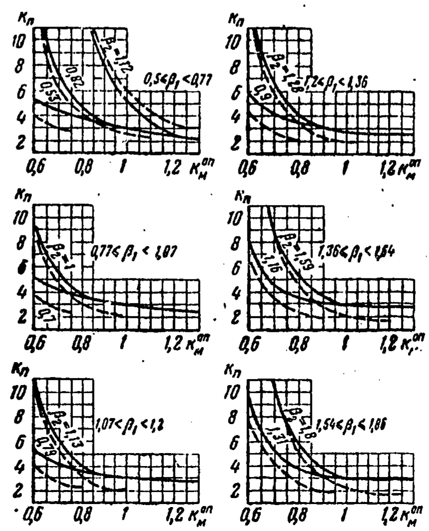


Fig. 127. Continuous double-span beam with supported ends. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_2^{H}0 = 25$

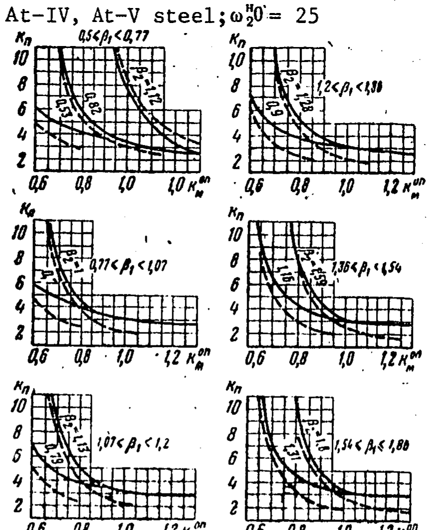


Fig. 128. Continuous double-span beam with supported ends. Class A-IIv, A-IIIv, A-IV. A-V, At-IV, At-V steel; $\omega_2^{H}0 = 50$

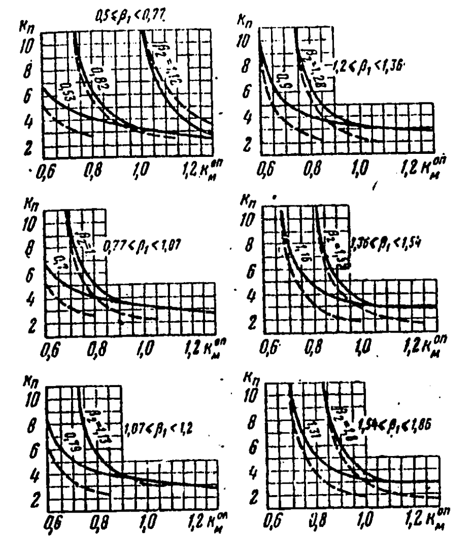


Fig. 129. Continuous double-span beam with supported ends. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_2^{н}0 = 100$

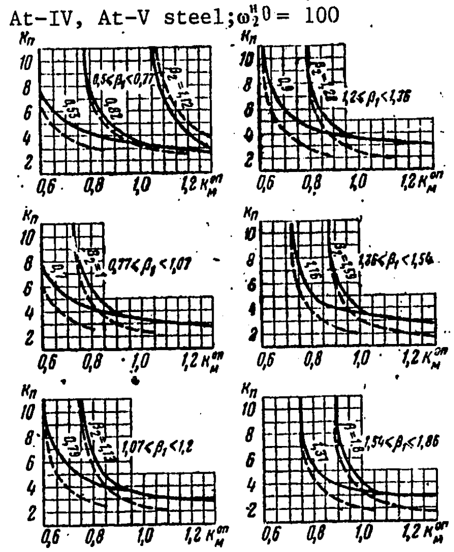


Fig. 130. Continuous double-span beam with supported ends. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_2^{н}0 = 500$

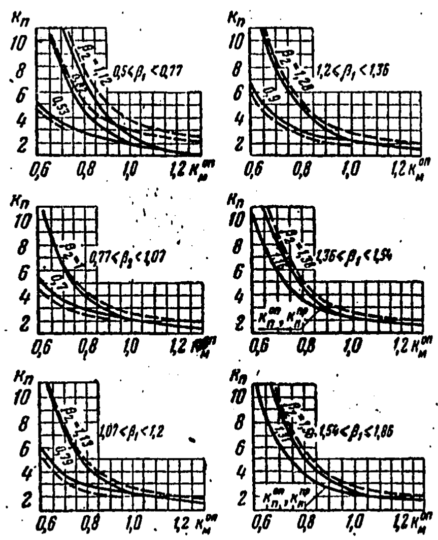


Fig. 131. End spans of continuous triple-span beam. Class A-I, A-II, A-III steel; $\omega_2^H 0 = 25$

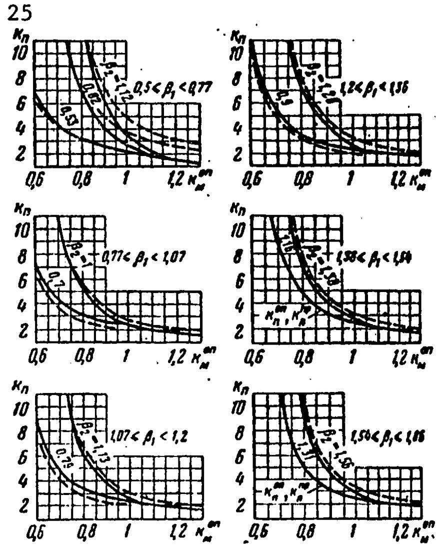


Fig. 132. End spans of continuous triple-span beam. Class A-I, A-II, A-III steel; $\omega_2^H 0 = 50$

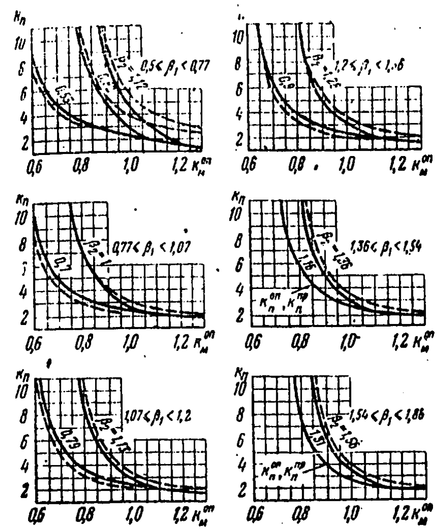


Fig. 133. End spans of continuous triple-span beam. Class A-I, A-II, A-III steel; $\omega_2^{\text{H}}\theta = 100$

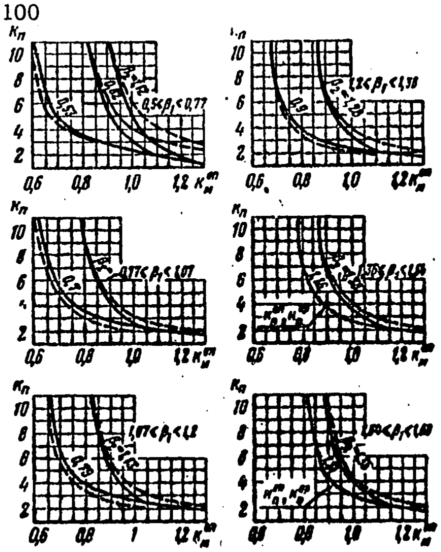


Fig. 134. End spans of continuous triple-span beam. Class A-I, A-II, A-III steel; $\omega_2^{\text{H}}\theta = 500$

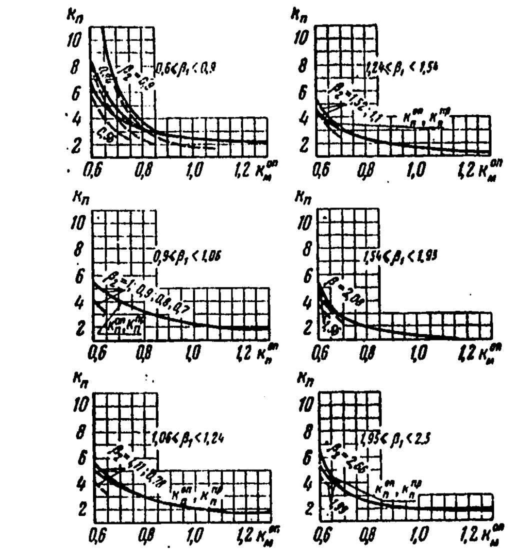


Fig. 135. Central span of continuous triple-span beam. Class A-I, A-II, A-III steel; $\omega_1^{''}\theta = 25$

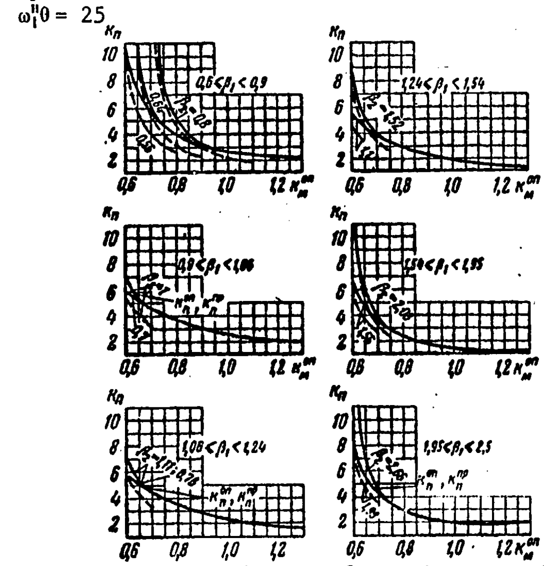


Fig. 136. Central span of continuous triple-span beam. Class A-I, A-II, A-III steel; $\omega_1^{''}\theta = 50$

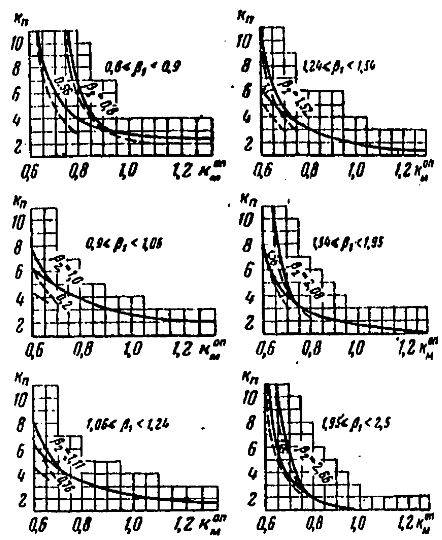


Fig. 137. Central span of continuous triple-span beam. Class A-I, A-II, A-III steel; $\omega_1^H\theta = 100$

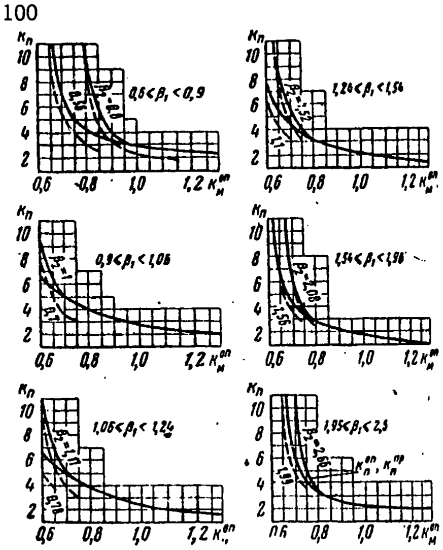


Fig. 138. Central span of continuous triple-span beam. Class A-I, A-II, A-III steel; $\omega_1^H\theta = 500$

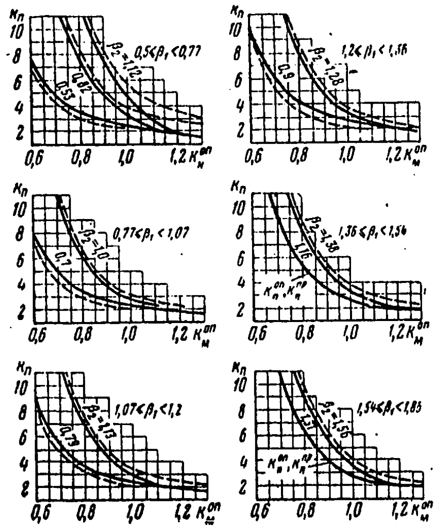


Fig. 139. End spans of continuous triple-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_2^H\theta = 25$

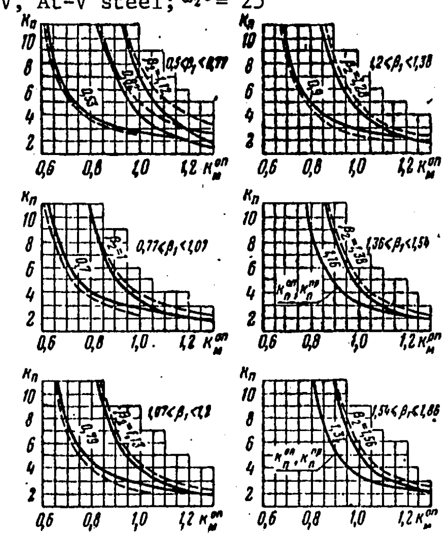


Fig. 140. End spans of continuous triple-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_2^H\theta = 50$

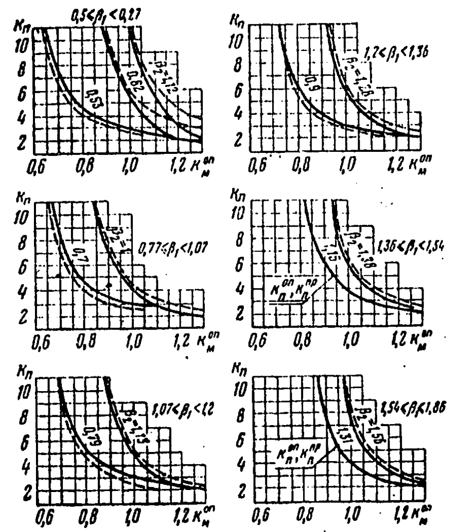


Fig. 141. End spans of continuous triple-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_2^{''}0 = 100$

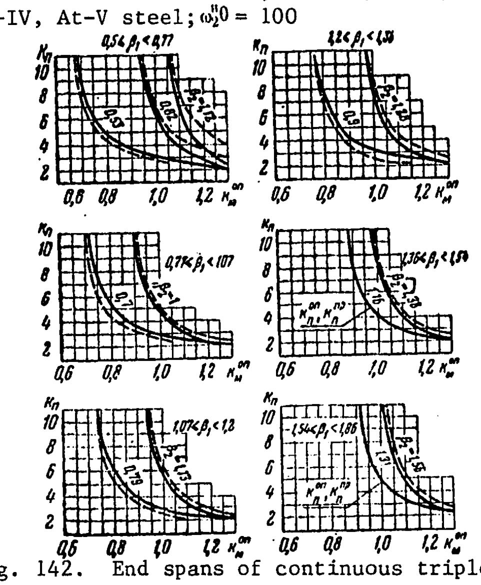


Fig. 142. End spans of continuous triple-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_2^{''}0 = 500$

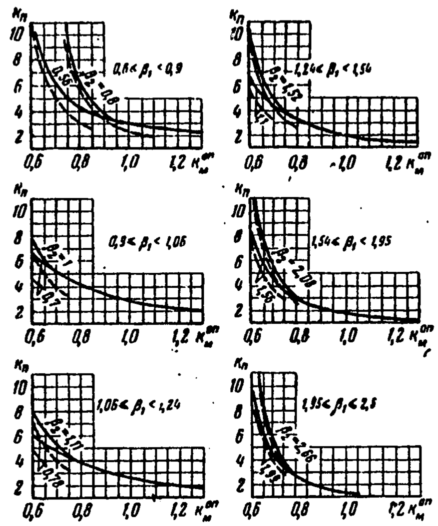


Fig. 143. Central span of continuous triple-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1^{H}0 = 25$

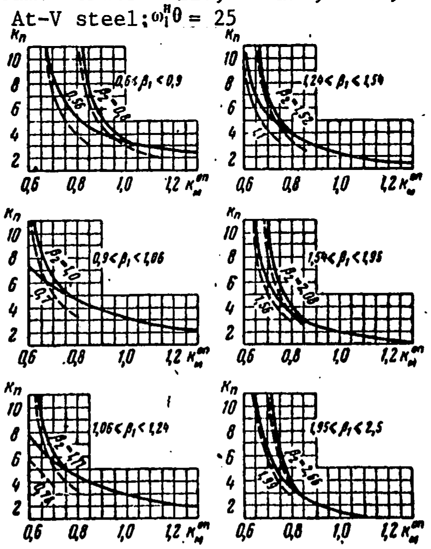


Fig. 144. Central span of continuous triple-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1^{H}0 = 50$

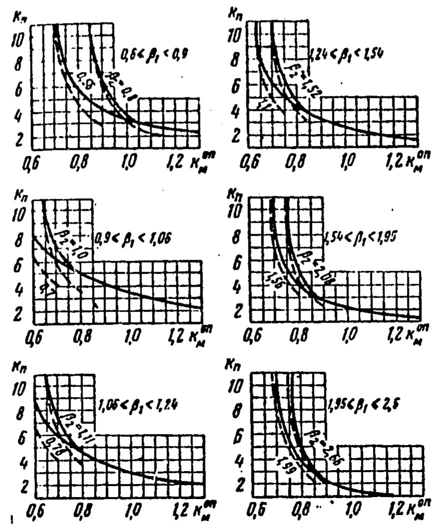


Fig. 145. Central span of continuous triple-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1^{H}\theta = 100$

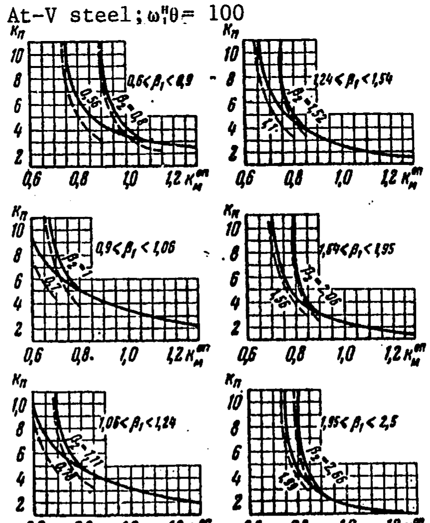


Fig. 146. Central span of continuous triple-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1^{H}\theta = 500$

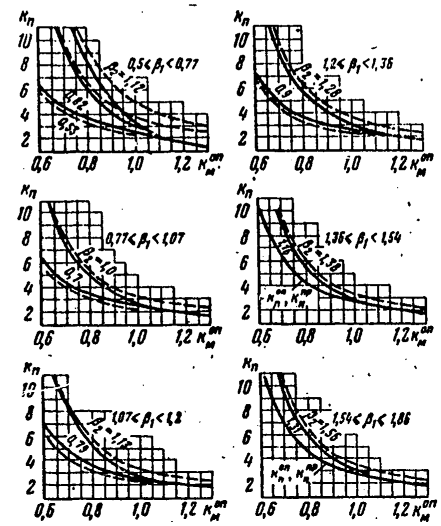


Fig. 147. End span of continuous beam with more than three spans. Class A-I, A-II, A-III steel; $\omega_2^{H}\theta = 25$

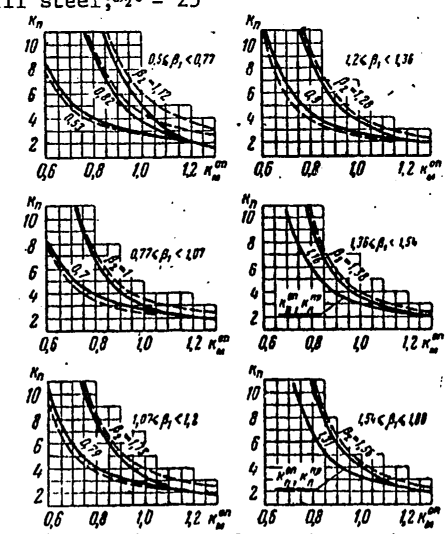


Fig. 148. End span of continuous beam with more than three spans. Class A-I, A-II, A-III steel; $\omega_2^{H}\theta = 50$

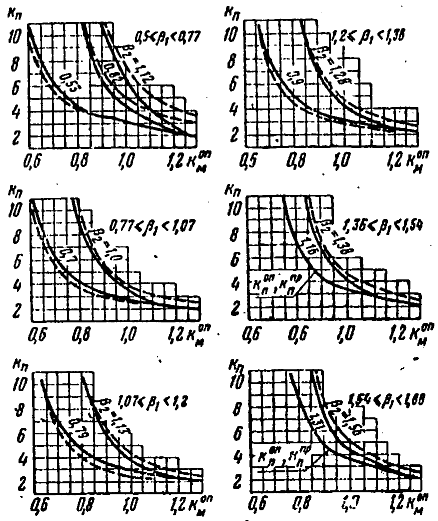


Fig. 149. End span of continuous beam with more than three spans. Class A-I, A-II, A-III steel; $\omega_2^H \theta = 100$

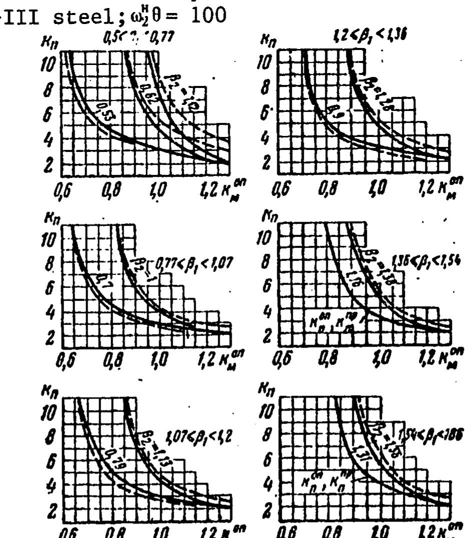


Fig. 150. End span of continuous beam with more than three spans. Class A-I, A-II, A-III steel; $\omega_2^H \theta = 500$

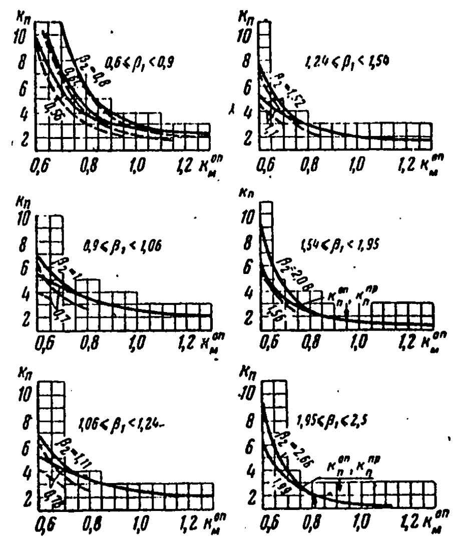


Fig. 151. Second and third spans of continuous four-span beam. Class A-I, A-II, A-III steel; $\omega_1^{H}\theta = 25$

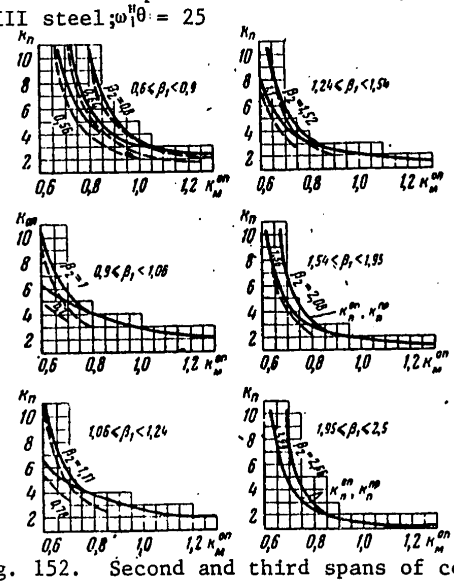


Fig. 152. Second and third spans of continuous four-span beam. Class A-I, A-II, A-III steel; $\omega_1^{H}\theta = 50$

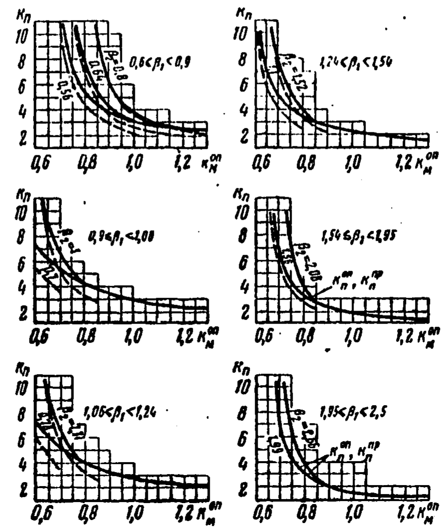


Fig. 153. Second and third spans of continuous four-span beam. Class A-I, A-II, A-III steel; $\omega_1^{н}0 = 100$

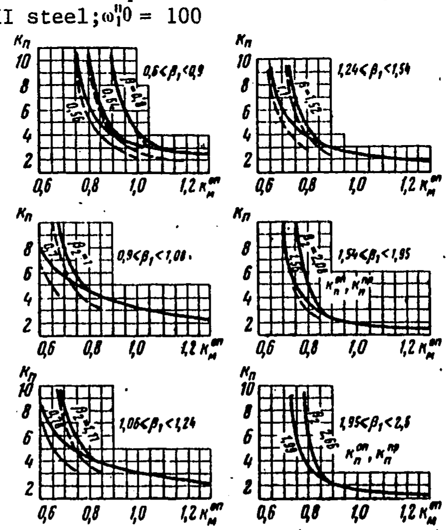


Fig. 154. Second and third spans of continuous four-span beam. Class A-I, A-II, A-III steel; $\omega_1^{н}0 = 500$

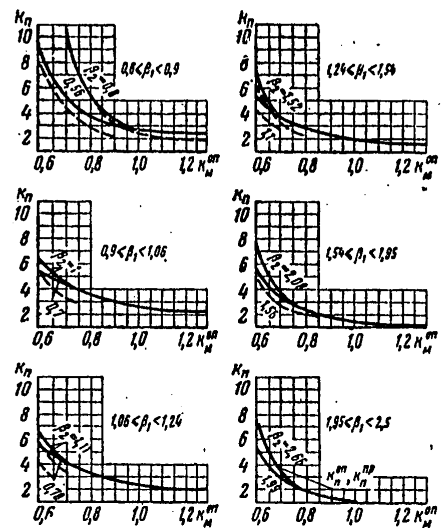


Fig. 155. Second span of continuous five-span beam. Class A-I, A-II, A-III steel; $\omega_I''\theta = 25$

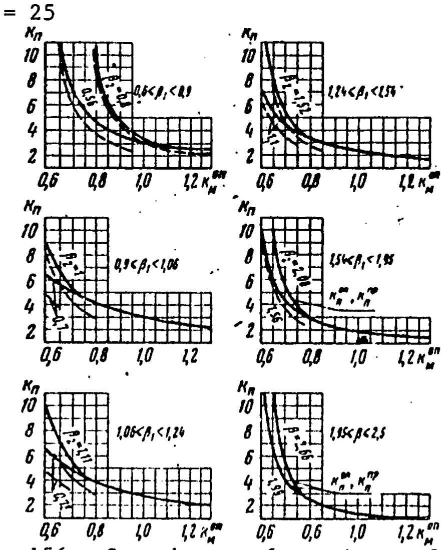


Fig. 156. Second span of continuous five-span beam. Class A-I, A-II, A-III steel; $\omega_I''\theta = 50$

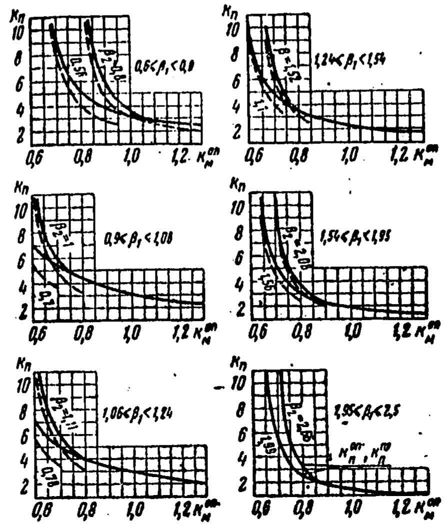


Fig. 157. Second span of continuous five-span beam. Class A-I, A-II, A-III steel; $\omega_1^H\theta = 100$

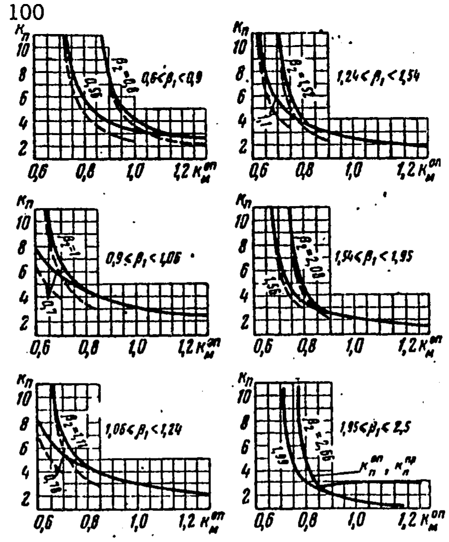


Fig. 158. Second span of continuous five-span beam. Class A-I, A-II, A-III steel; $\omega_1^H\theta = 500$

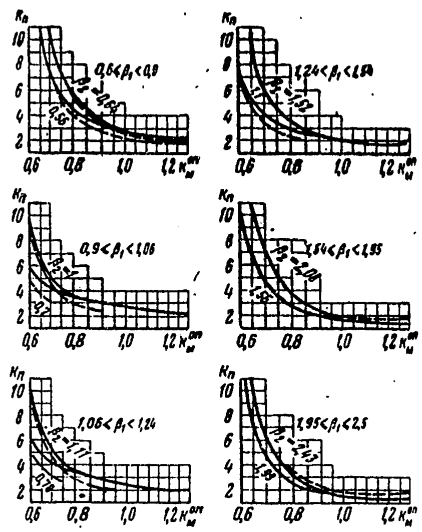


Fig. 159. Third span of continuous five-span beam. Class A-I, A-II, A-III steel; $\omega_I^H 0 = 25$

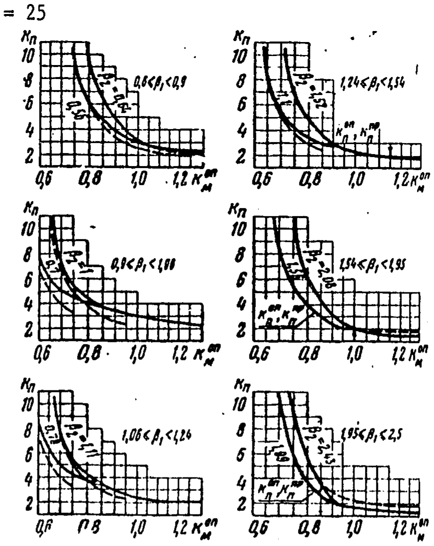


Fig. 160. Third span of continuous five-span beam. Class A-I, A-II, A-III steel; $\omega_I^H 0 = 50$

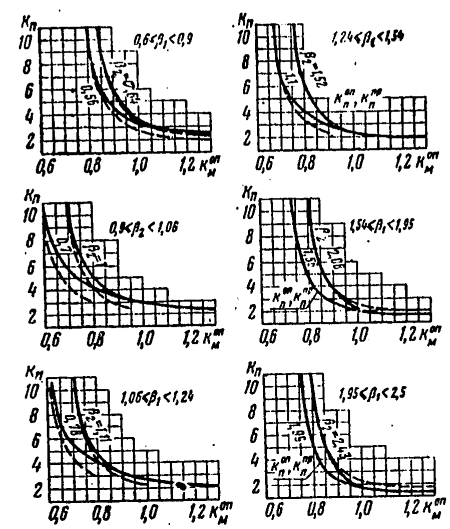


Fig. 161. Third span of continuous five-span beam. Class A-I, A-II, A-III steel; $\omega_1''\theta = 100$

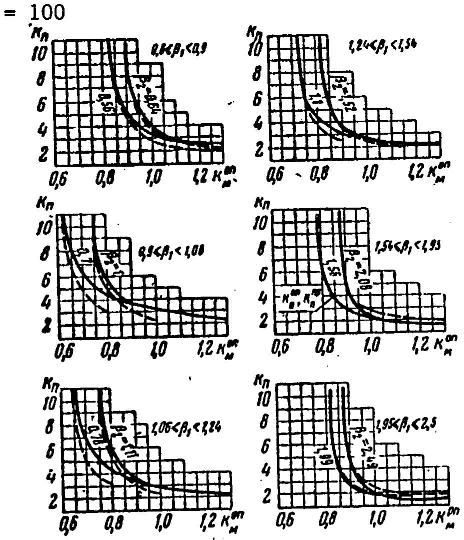


Fig. 162. Third span of continuous five-span beam. Class A-I, A-II, A-III steel; $\omega_1''\theta = 500$

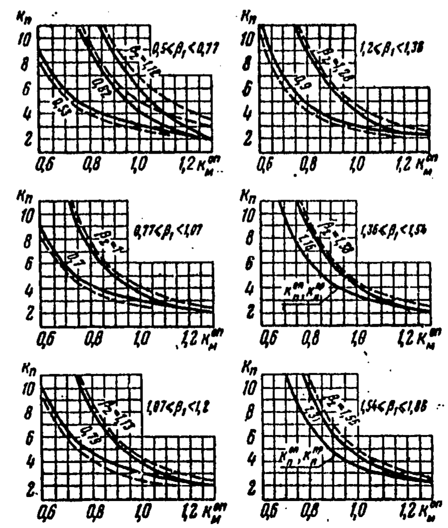


Fig. 163. End span of continuous beam with more than three spans. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_2^{n}0 = 25$

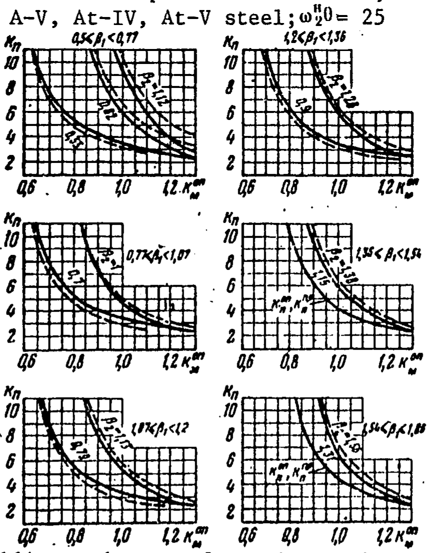


Fig. 164. End span of continuous beam with more than three spans. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_2^{n}0 = 50$

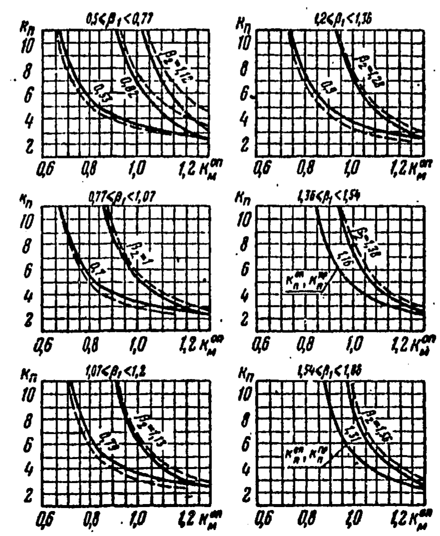


Fig. 165. End span of continuous beam with more than three spans. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_2^{II}\theta = 100$

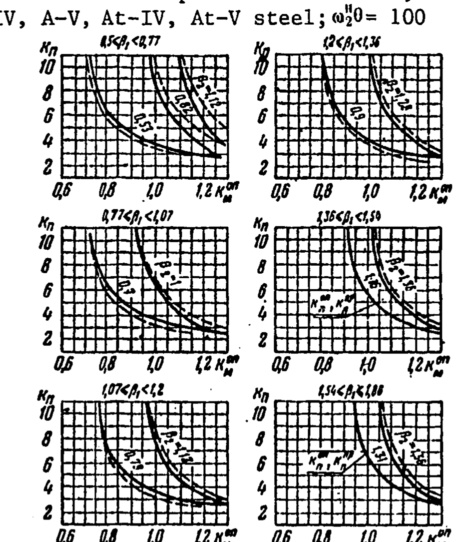


Fig. 166. End span of continuous beam with more than three spans. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_2^{II}\theta = 500$

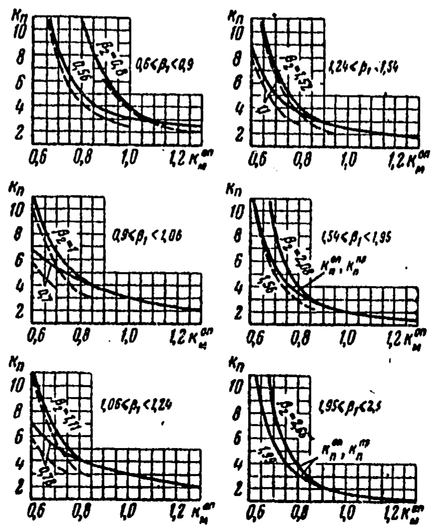


Fig. 167. Second and third spans of continuous four-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1^{H}\theta = 25$

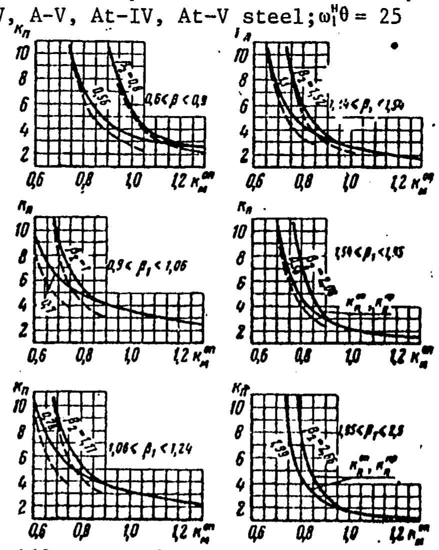


Fig. 168. Second and third spans of continuous four-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1^{H}\theta = 50$

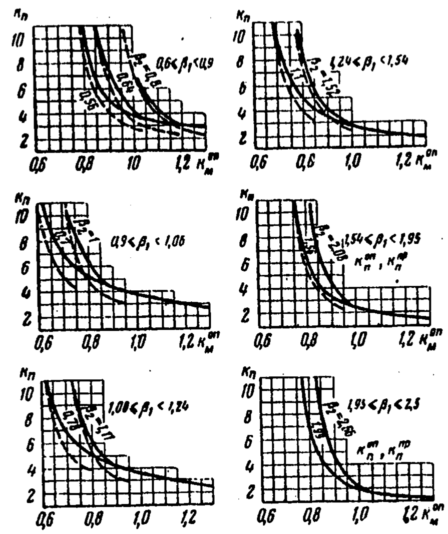


Fig. 169. Second and third spans of continuous four-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_2^{\prime\prime}0 = 100$

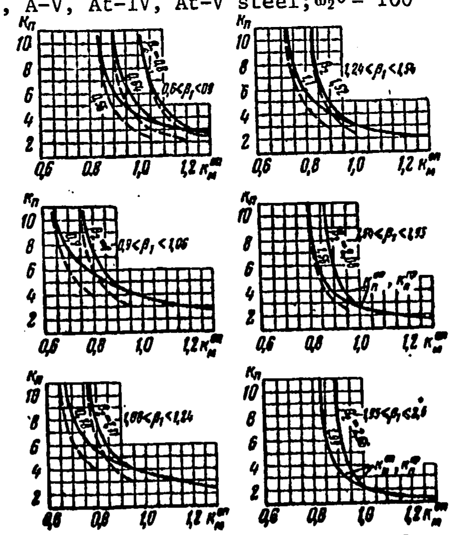


Fig. 170. Second and third spans of continuous four-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1^{\prime\prime}0 = 500$

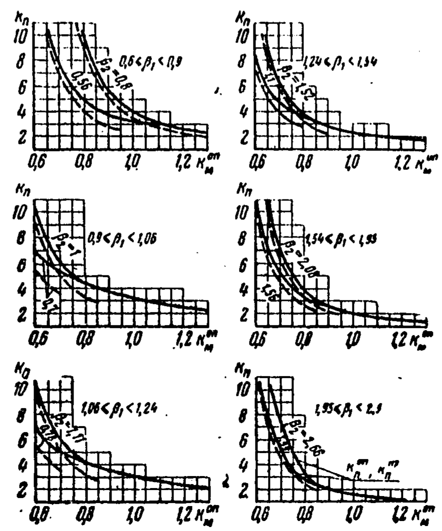


Fig. 171. Second span of continuous five-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1^{H}\theta = 25$

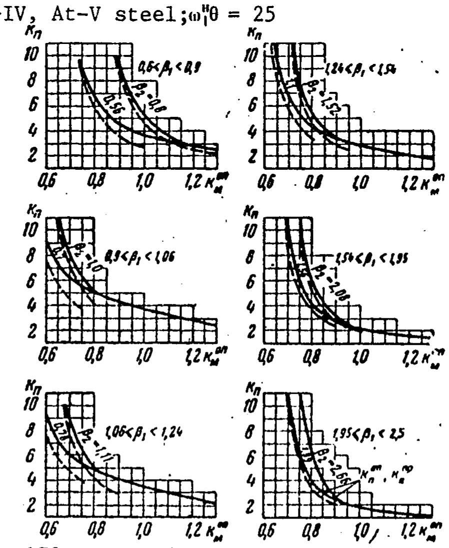


Fig. 172. Second span of continuous five-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1^{H}\theta = 50$

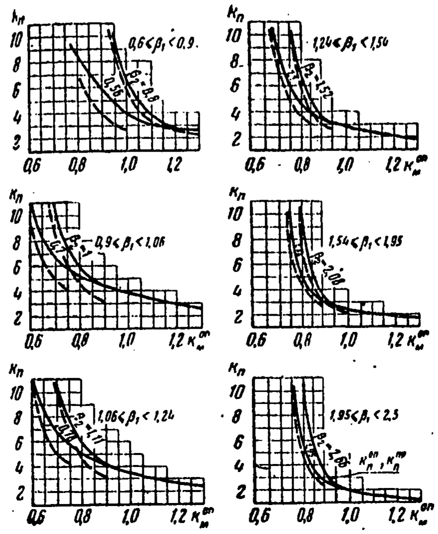


Fig. 173. Second span of continuous five-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1^H\theta = 100$

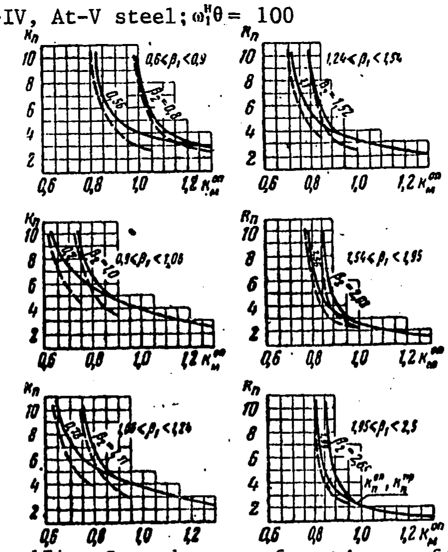


Fig. 174. Second span of continuous five-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1^H\theta = 500$

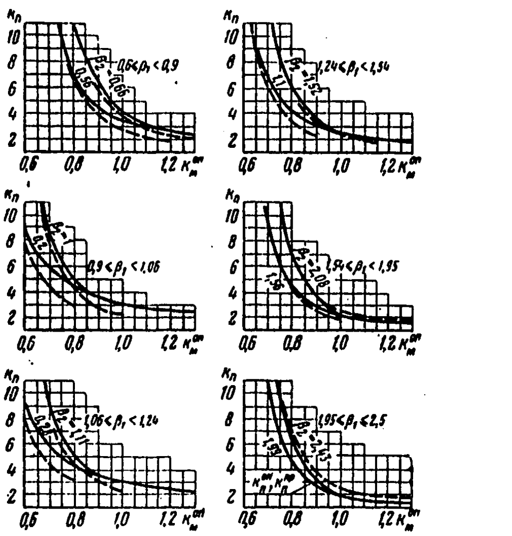


Fig. 175. **First span of continuous five-span** beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1^{\text{н}}\theta = 25$

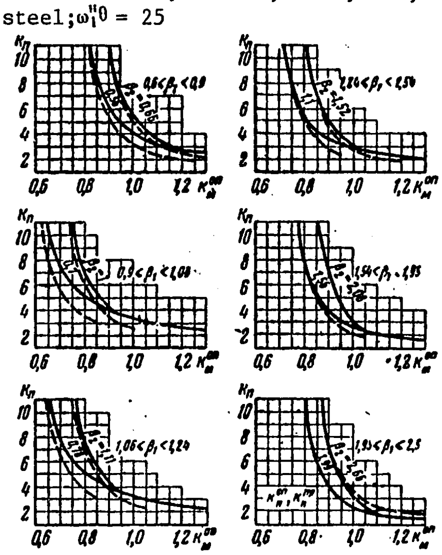


Fig. 176. First span of continuous five-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1^{\text{н}}\theta = 50$

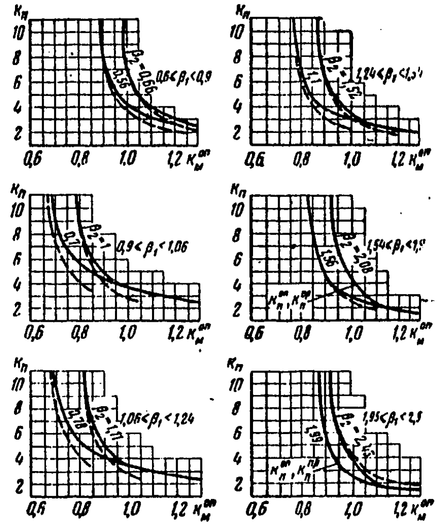


Fig. 177. First span of continuous five-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1^{II}\theta = 100$

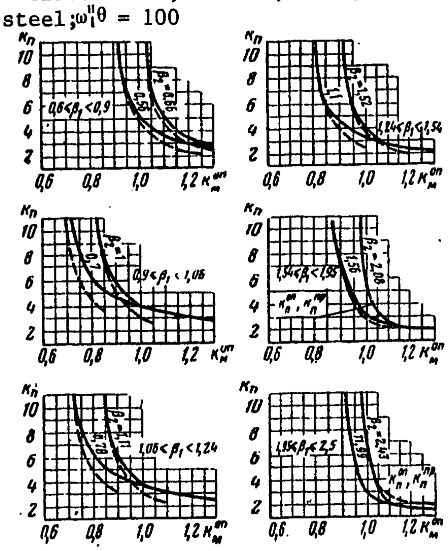


Fig. 178. First span of continuous five-span beam. Class A-IIv, A-IIIv, A-IV, A-V, At-IV, At-V steel; $\omega_1^{II}\theta = 500$

Appendix 8 - Sample Calculation of Basic Elements of Freestanding Buried Protective Structure

Given the calculation for bearing power (Case 1a) of basic elements of frame-panel construction with soil fill over overhead cover 80 cm thick to the effect of loads from a shock wave with pressure at the front of Δp = 1.5 kg/cm^2 with effective time of action θ = 0.8 sec.

The structural-design decision of the structure is shown in Fig. 179.

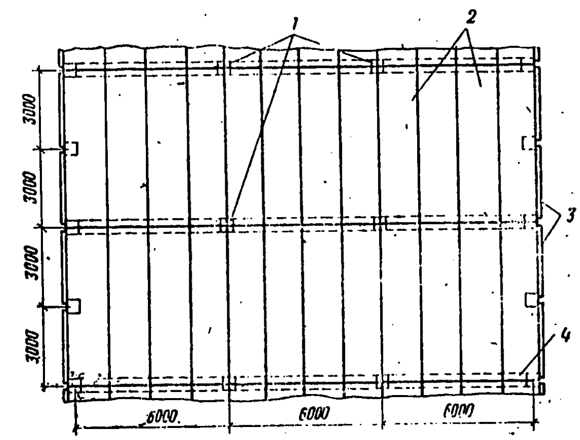


Fig. 179. Schematic of structural-design decision of shelter (plan view)

1. Columns
2. Slabs
3. Wall panels
4. Collar beams

Determination of Effective Loads

In accordance with paragraphs 3.1-3.3 of SN 405-70, we draw up a special load combination on the overhead cover.

The load from the earth fill is $q_{\text{гр}} = h_{\text{обс}}\gamma_{\text{гр}} = 80\cdot 1,8\cdot 10^{-3} \doteq 0,144$ kg/cm^2.

The load from the components' own weight with a previously assumed thickness of the overhead cover slab equal to $h_{\text{ПЛ}}$ = 25 cm is

$$q_{\text{с.в}} = h_{\text{пл}}\gamma_{\text{жб}} = 25\cdot 2,5\cdot 10^{-3} = 0,063 \text{ kg/cm}^2.$$

The cumulative static load equals: $q_{\text{ст}} = q_{\text{гр}} + q_{\text{с.в}} = 0,144 + 0,063 \approx 0,21$ kg/cm^2.

The equivalent static load from formula [41(4)] of SN 405-70 with $k_Д = 1.2$ (see Table 15) equals:

$$q_{\text{экв}} = k_Д \Delta p = 1,2 \cdot 1,5 = 1,8 \text{ kg/cm}^2,$$

The special load combination is

$$p = q_{\text{ст}} + q_{\text{экв}} = 0,21 + 1,8 = 2,01 \text{ kg/cm}^2.$$

Calculation of Overhead Cover Slab

The overhead cover of a precast-monolithic component is being designed. Slabs with a flanged section covered above by a layer of monolithic concrete are taken as the basic bearing elements of the overhead cover. The slabs are placed with 10 cm intervals filled with mass concrete having reinforcement (Fig. 180).

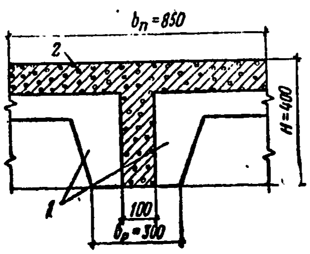


Fig. 180. Design section of overhead cover slabs

1. Precast flanged slabs
2. Monolithic concrete

The design schematic of the slab is a multispan continuous beam component with identical spans.

The effective span of the slab is taken in accordance with Paragraph 3.34 as equal to 530 cm; width of the precast slab is 75 cm; grade of slab concrete is 300; $R_И$= 160 kg/cm^2; grade of monolithic concrete 200; $R_И$= 100 kg/cm^2. In accordance with Paragraph 3.22, $k_y = 1.2$; $k_t = 1.1$.

The longitudinal working reinforcement is Class A-III steel; $R_a = 3400$ kg/cm^2.

Let us compute the bending moments:

In the end spans $M_1^{\text{пр}} = k_p pl^2 = 0,078 \cdot 2,01 \cdot 530^2 = 4,41 \cdot 10^4$ kg·cm;

Above the second supports from the end $M_в = -k_p pl^2 = -0,105 \cdot 2,01 \cdot 530^2 = 5,93 \cdot 10^4$ kg·cm. Here k_p represents the tabular coefficient for bending moments in continuous beams;

On central supports $M_0 = -k_p pl^2 = -0,079 \cdot 2,01 \cdot 530^2 = 4,46 \cdot 10^4$ kg·cm;

In the second span $M_2^{\text{пр}} = 0,033 \cdot 2,01 \cdot 530^2 = 1,86 \cdot 10^4$ kg·cm;

In the third span $M_3^{\text{пр}} = 0,046 \cdot 2,01 \cdot 530^2 = 2,6 \cdot 10^4$ kg·cm,

We construct moment diagrams (Fig. 181). We perform a redistribution of stresses. We reduce the value of the support moment by 25 percent above the second support from the end to equalize it with the moment on the central support, for which we add a triangular diagram to the moment diagram (Fig. 181b) with an ordinate above the support of $+1.5 \cdot 10^4$kg·cm.

The moment diagram obtained as a result of stress redistribution is shown in Fig. 181c.

Let us determine the effective section height for the moment in the end span after stress redistribution: $M_1 = 5,04 \cdot 10^4$ kg·cm,

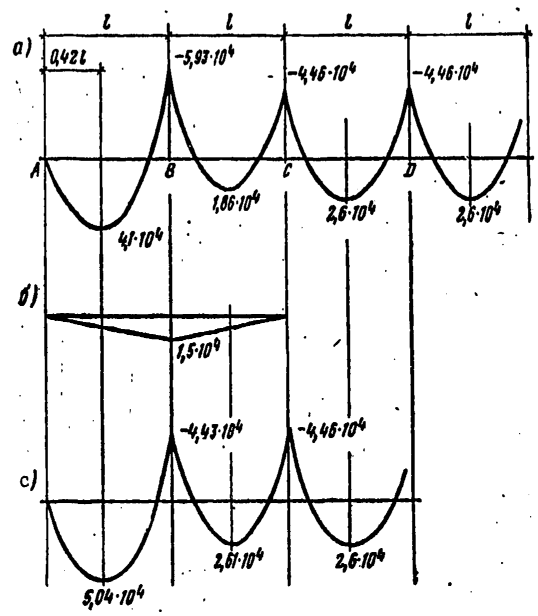


Fig. 181. Moment diagrams for slab

a. Before redistribution of stresses
b. For redistributed stresses above supports
c. After redistribution of stresses

In accordance with Paragraph 3.41, we assume $\alpha^{\text{пр}} = 0.36$.

The effective section height of the slab equals:

$$h_0 = \sqrt{\frac{M_1}{\alpha(1-0{,}5\alpha)\,k_y R_{\text{п}} k_t}} = \sqrt{\frac{5{,}04\cdot 10^4}{0{,}36\,(1-0{,}5\cdot 0{,}36)\,1{,}2\cdot 100\cdot 1{,}1}} \approx 37 \text{ см}.$$

In the first approximation in Table 19(10) of Paragraph 3.24 with $\omega_2^{\text{п}} \approx 100$ $\sec^{-1}$ we take the dynamic hardening factor of reinforced steel to be $k_y = 1.3$. With consideration of the work condition factor $m_a = 1.1$ (see Paragraph 3.23), the effective dynamic resistance of longitudinal working reinforcement is

$$R_a^y = m_a\, k_y\, R_a = 1{,}1\cdot 1{,}3\cdot 3400 = 4870 \text{ kg/см}^2.$$

The design resistance obtained is less than the tensile strength with a static load equal to 5,000 kg/cm^2.

We determine the reinforcement factor in the end span by using formula (8) of SN 405-70:

$$\mu_{np} = \alpha_{np} \frac{k_y k_l R_н}{R_a^y} = 0{,}36 \frac{1{,}1\cdot 1{,}2\cdot 100}{4870} \approx 0{,}01.$$

From formula (77) we determine the rigidity of a T-section slab with flange in the compression zone for the span:

$$\xi_{np} = \frac{1}{1{,}8 + \frac{1}{10n\mu_{np}}} = \frac{1}{1{,}8 + \frac{1}{10\cdot 7{,}55\cdot 0{,}01}} = 0{,}319;$$

$$\beta_{np} = \frac{1 - 0{,}5\xi_{np}}{1 + \frac{1{,}8n\mu_{np}}{\xi_{np}}} = \frac{1 - 0{,}5\cdot 0{,}319}{1 + \frac{1{,}8\cdot 7{,}55\cdot 0{,}01}{0{,}319}} = 0{,}588;$$

$$B^{np} = E_a \mu_{np} b_n h_0^3 \beta = 2\cdot 10^6\cdot 0{,}01\cdot 85\cdot 37^2\cdot 0{,}588 = 5{,}05\cdot 10^{10}\ \text{kg}\cdot\text{см}^2.$$

We calculate that the amount of compressed reinforcement ($R_{a:c}$) below in the support section equals:

$$F_a' = 0{,}5 b_n h_0 \mu_{np} = 0{,}5\cdot 85\cdot 37\cdot 0{,}01 = 15{,}8\ \text{см}^2,$$

We take $\alpha_{оп}$= 0.4 for the support. The width of the flange of the T section in the compression zone on the support must be at least

$$b_{np} = \frac{M^{оп} b_п - R_{a.c} F_a'(h_0 - a)}{k_l k_y R_н h_0^2 \alpha (1 - 0{,}5\alpha)} =$$

$$= \frac{4{,}43\cdot 10^4\cdot 85 - 3400\cdot 15{,}8(37-3)}{1{,}1\cdot 1{,}2\cdot 140\cdot 37^2\cdot 0{,}4(1 - 0{,}5\cdot 0{,}4)} = 24{,}2\ \text{см}.$$

We take the width of the T section flange to be 30 cm. With a width of the precast slab legs and space between slabs 10 cm each. The value n for the assumed T section is determined in proportion to the dimensions of the precast slab legs (grade 300 concrete) and the monolithic space (grade 200 concrete) and equals:

$$n = \frac{E_a}{E_б^{cp}} = \frac{2\cdot 10^6}{{}^2/_3 3{,}15\cdot 10^5 + {}^1/_3 2{,}65\cdot 10^5} = 6{,}72;$$

$$\mu_{оп} = \alpha_{оп} \frac{R_н^y}{R_a^y} = 0{,}4 \frac{1{,}1\cdot 1{,}2\cdot 140}{4870} = 0{,}0153;$$

$$\xi_{оп} = \frac{1}{1{,}8 + \frac{1}{10\cdot 6{,}72\cdot 0{,}0153}} = 0{,}365;$$

$$\beta_{оп} = \frac{1 - 0{,}5\cdot 0{,}365}{1 + \frac{1{,}8\cdot 6{,}72\cdot 0{,}0153}{0{,}365}} = 0{,}535.$$

From formula (77) $B^{on} = E_a \mu_{on} b_p h_0^3 \beta_{on} = 2\cdot 10^6\cdot 0{,}0153\cdot 30\cdot 37^3\cdot 0{,}535 = 2{,}50$ kg·cm².

We find the ratio of rigidity at the support and in the span from formula (129):

$$\beta_{1,1} = \frac{B^{on}}{B^{np}} = \frac{2{,}50\cdot 10^{10}}{5{,}05\cdot 10^{10}} = 0{,}5.$$

The factor k_2 from formula (117) equals:

$$k_2 = \frac{0{,}26 + 0{,}74\cdot 0{,}5}{0{,}578 + 0{,}422\cdot 0{,}5} = 0{,}8.$$

For end spans $k_2^{н} = 0{,}8\cdot k_2 = 0{,}64.$

We find the component's natural oscillation frequency from formula (71):

$$\omega_2^{н} = \frac{20{,}7}{530^2}\sqrt{\frac{5{,}05\cdot 10^{10}}{1{,}78\cdot 10^{-3}}} = 120\ 1/\text{sec}.$$

where

$$m = \frac{q_{ст}}{g} = \frac{0{,}144\cdot 85 + 20\cdot 85\cdot 2{,}4\cdot 10^{-3} + 20\cdot 24\cdot 2{,}4\cdot 10^{-3}}{981} = 1{,}78\cdot 10^{-3}\ \text{kg}\cdot\text{sec}^2/\text{cm}^2$$

The dynamic hardening factor of reinforced steel from the chart in Fig. 76 equals $k_y = 1.32$.

The moments from a linear static load with consideration of stress redistribution are:

In the span $M_{ст}^{np} = 0{,}125\bar{q}_{ст} l^2\left(1 - \frac{k_2^{н}}{4}\right) =$

$$= 0{,}125\cdot 0{,}21\cdot 85\cdot 530^2\left(1 - \frac{0{,}64}{4}\right) = 0{,}44\cdot 10^6\ \text{kg}\cdot\text{cm};$$

At the support $M_{ст}^{on} = 0{,}125\bar{q}_{ст} l^2 k_2^{н} = 0{,}125\cdot 0{,}21\cdot 85\cdot 530^2\cdot 0{,}64 = 0{,}40\cdot 10^6$ kg·cm;

Moments of internal stresses are:

In the span $M_0^{np} = R_a^y \mu_{np} b_n h_0^2(1 - 0{,}5\alpha_{np}) = 4950\cdot 0{,}01\cdot 85\cdot 37^2(1 - 0{,}5\cdot 0{,}36) = 4{,}62\cdot 10^6$ kg·cm;

At the support $M_0^{on} = R_a^y \mu_{on} b_p h_0^2(1 - 0{,}5\alpha_{np}) = 4950\cdot 0{,}0153\cdot 30\cdot 37^2(1 - 0{,}5\cdot 0{,}4) = 2{,}5\cdot 10^6$ kg·cm;

Moments of internal stresses with the subtraction of static moments are:

In the span $\bar{M}_0^{np} = M_0^{np} - k_y M_{ст}^{np} = 4{,}62\cdot 10^6 - 1{,}32\cdot 0{,}44\cdot 10^6 = 4{,}04\cdot 10^6$ kg·cm;

At the support $\bar{M}_0^{on} = M_0^{on} - k_y M_{ст}^{on} = 2{,}5\cdot 10^6 - 1{,}32\cdot 0{,}404\cdot 10^6 = 2\cdot 10^6$ kg·cm

The ratio of moments equals: $\beta_3 = \frac{\bar{M}_0^{on}}{\bar{M}_0^{np}} = \frac{2\cdot 10^6}{4{,}04\cdot 10^6} \approx 0{,}5.$

We check the dynamic strength conditions [formula (132) and (133)]. For this we compute the moment from the dynamic load at the support:

$$M_p^{on} = 0{,}125\Delta p b_n l^2 k_2^{н} = 0{,}125\cdot 1{,}5\cdot 85\cdot 530^2\cdot 0{,}64 = 2{,}88\cdot 10^6\ \text{kg}\cdot\text{cm}$$

and the dynamic-response factor for the bending moment for the support:

$$k_м^{on} = \frac{\bar{M}_0^{on}}{M_p^{on}} = \frac{2\cdot 10^6}{2{,}88\cdot 10^6} = 0{,}74.$$

For this value $k_M^{оп}$ and with the values $\beta_{1,1}=0{,}5$; $\beta_2 = 0{,}5$ и $\omega_2^н \theta = 120 \cdot 0{,}8 \approx 100$ from the chart of Appendix 7 we find $k_П^{оп}= 5$; $k_П^{пр}= 4.8$. From Table 30 we find $k_Q^{оп}= 1.44$ and $k_Q^{пр} = 1.4$.

Check for fulfillment of strength condition. The limiting expansion angles in hinges of plasticity from formula (58) are $\psi_п^{пр} = 0{,}035 + \frac{0{,}003}{\alpha_{пр}} = 0{,}0433$; $\psi_п^{оп} = 0{,}0425$.

The expansion angle in the hinge of plasticity is:

At the support from formula (120) $\psi^{оп} = \frac{\Delta p l^3 k_п^{оп}}{106{,}8 B^{пр}} = \frac{1{,}5 \cdot 85 \cdot 530^3 \cdot 5}{106{,}8 \cdot 5{,}05 \cdot 10^{10}} = 1{,}77 \cdot 10^{-2} < 0{,}5 \psi_п^{оп} = 2{,}125 \cdot 10^{-2}$;

In the span from formula (121)

$$\psi^{пр} = \frac{\Delta p l^3 k_п^{пр}}{44 B^{пр}} = \frac{1{,}5 \cdot 85 \cdot 530^3 \cdot 4{,}8}{44 \cdot 5{,}05 \cdot 10^{10}} = 4{,}1 \cdot 10^{-2} < \psi_п^{пр} = 4{,}33 \cdot 10^{-2}.$$

The found parameters of the slab satisfy strength conditions in the span and at the support.

Third Span

All central spans of a continuous multispan slab are designed for a third span.

We reduce the reinforcement percentage in the third span in comparison with the first and assume:

$$\mu_{пр} = 0{,}009;\quad \alpha_{пр} = 0{,}009 \frac{4950}{1{,}1 \cdot 1{,}2 \cdot 100} = 0{,}335;$$

$$\xi_{пр} = \frac{1}{1{,}8 + \frac{1}{7{,}55 \cdot 0{,}009}} = 0{,}305;$$

$$\beta_{пр} = \frac{1 - 0{,}5 \cdot 0{,}305}{1 + \frac{1{,}8 \cdot 7{,}55 \cdot 0{,}009}{0{,}335}} = 0{,}625;$$

$$B^{пр} = E_а \mu_{пр} b_п h_0^3 \beta_{пр} = 2 \cdot 10^6 \cdot 0{,}009 \cdot 85 \cdot 37^3 \cdot 0{,}625 = 4{,}86 \cdot 10^{10} \text{ kg} \cdot \text{см}^2.$$

The ratio of rigidity at the support and in the span

$$\beta_{1,3} = \frac{B_{оп}}{B_{пр}} = \frac{2{,}5 \cdot 10^{10}}{4{,}86 \cdot 10^{10}} = 0{,}515$$

from formula (129)

$$k_{1,3} = \frac{0{,}269 + 0{,}731 \cdot 0{,}515}{0{,}46 + 0{,}54 \cdot 0{,}515} = 0{,}91;$$

$$k_{1,3}^н = 0{,}948 \cdot 0{,}91 = 0{,}855.$$

Moments of internal stresses equal:

$$M_0^{пр} = 4950 \cdot 0{,}009 \cdot 85 \cdot 37^2 (1 - 0{,}5 \cdot 0{,}335) = 4{,}37 \cdot 10^6 \text{ kg} \cdot \text{см};$$

$M_0^{оп} = 2{,}5 \cdot 10^6$ kg·cm (as in the first span).

Moments from a linear static load with consideration of stress redistribution:

In the span

$$M_{ст}^{пр} = \frac{\bar{q}_{ст} l^2 (3 - 2k_{1,3}^н)}{24} = \frac{0{,}21 \cdot 85 \cdot 530^2 (3 - 2 \cdot 0{,}855)}{24} = 0{,}268 \cdot 10^6 \text{ kg} \cdot \text{см};$$

At the support $$M_{\text{ст}}^{\text{оп}} = \frac{\bar{q}_{\text{ст}}\, l^2 k_{1,3}^{\text{н}}}{12} = \frac{0{,}21\cdot 85\cdot 530^2\cdot 0{,}855}{12} = 0{,}355\cdot 10^6 \text{ kg}\cdot\text{см}.$$

Moments of internal stresses with subtraction of static moments
$$\bar{M}_0^{\text{пр}} = M_0^{\text{пр}} - M_{\text{ст}}^{\text{пр}} k_y = 4{,}37\cdot 10^6 - 0{,}268\cdot 10^6\cdot 1{,}32 =$$
$$= 4{,}02\cdot 10^6 \text{ kg}\cdot\text{см};$$
$$\bar{M}_0^{\text{оп}} = M_0^{\text{оп}} - M_{\text{ст}}^{\text{оп}} k_y = 2{,}5\cdot 10^6 - 0{,}355\cdot 10^6\cdot 1{,}32 = 2{,}03\cdot 10^6 \text{ kg}\cdot\text{см},$$

The ratio of moments equals:
$$\beta_2 = \frac{\bar{M}_0^{\text{оп}}}{\bar{M}_0^{\text{пр}}} = \frac{2{,}03\cdot 10^6}{4{,}02\cdot 10^6} = 0{,}506.$$

The moment from the dynamic load at the support
$$M_p^{\text{оп}} = 0{,}835\Delta p b_n l^2 k_{1,3}^{\text{н}} = 0{,}835\cdot 1{,}5\cdot 85\cdot 530^2\cdot 0{,}855 = 2{,}50\cdot 10^6 \text{ kg}\cdot\text{см}$$
and the dynamic-response factor for the support $$k_{\text{м}}^{\text{оп}} = \frac{\bar{M}_0^{\text{оп}}}{M_p^{\text{оп}}} = \frac{2{,}03\cdot 10^6}{2{,}50\cdot 10^6} = 0{,}82.$$

For this value $k_M^{\text{оп}}$ and with the values $\beta_1 = 0{,}515$; $\beta_2^2 = 0{,}506$; $\omega\theta = 100$ from the chart of Appendix 7 we find $k_{\text{п}}^{\text{оп}} = 7$ and $k_{\text{п}}^{\text{пр}} = 7$. From Table 24 $k_Q^{\text{оп}} = 1{,}25$.

Check fulfillment of the strength condition.

The limiting expansion angle in the hinge of plasticity at the support from formula (58)
$$\psi_{\text{п}}^{\text{оп}} = 0{,}035 + \frac{0{,}003}{0{,}4} = 4{,}25\cdot 10^{-2}.$$

From formula (110) $$\psi^{\text{оп}} = \frac{1{,}5\cdot 85\cdot 530^3\cdot 7}{192\cdot 4{,}86\cdot 10^{10}} = 1{,}44\cdot 10^{-2} < 0{,}5\psi_{\text{п}}^{\text{оп}} = 2{,}125\cdot 10^{-2}.$$

In the span $$\psi_{\text{п}}^{\text{пр}} = 0{,}035 + \frac{0{,}003}{0{,}335} = 4{,}4\cdot 10^{-2}.$$

From formula (111) $$\psi^{\text{пр}} = \frac{1{,}5\cdot 85\cdot 530^3\cdot 7}{96\cdot 4{,}86\cdot 10^{10}} = 2{,}9\cdot 10^{-2} < \psi_{\text{п}}^{\text{пр}} = 4{,}4\cdot 10^{-2}.$$

We perform the calculation for lateral force. The greatest value of lateral force will be at the face of the second support in the first span. From formula (46) with consideration of expression (127)
$$Q_1 = k_Q^{\text{оп}}\Delta p b_{\text{п}}\frac{l}{2} + k_t k_y\cdot 0{,}605(p_{\text{св}} + p_{\text{гр}}) b_{\text{п}} l = 1{,}44\cdot 1{,}5\cdot 85\frac{530}{2} + 1{,}1\cdot 1{,}2\cdot 0{,}605(0{,}068 + 0{,}144)\,85\cdot 530 = 5{,}55\cdot 10^4 \text{ kg}$$
We check the slab section for condition (10) of Paragraph 3.37:
$$Q_{\text{экв}} \leqslant 0{,}35 R_{\text{н}}^{\text{у}} b_{\text{р}} h_0;\ 5{,}55 < 0{,}35(1{,}1\cdot 1{,}2\cdot 140)\,30\cdot 37 = 7{,}2\cdot 10^4 \text{ kg}.$$

Condition (10) is satisfied.

The calculated value of the transverse force from formula (68) equals:
$$Q_p = Q_1 - 0{,}8\Delta p b_{\text{п}} C_0 - 0{,}5(p_{\text{с.в}} + p_{\text{гр}}) b_{\text{п}} C_0.$$

We first assume the length of the projection of the nonoptimum oblique section C_0 on the element's longitudinal axis to be equal to 40 cm from the condition of the extension of a diagonal crack in the section at an angle of 45°.

Then the calculated value of the transverse force equals:

$$Q_p = 5{,}55\cdot 10^4 - 0{,}8\cdot 1{,}5\cdot 85\cdot 40 - 0{,}5\cdot 0{,}21\cdot 85\cdot 40 = 5{,}1\cdot 10^4 \text{ kg}.$$

We determine the value of the limiting stress received by lateral rods:

$$q_x = \frac{Q_p^2}{0{,}6 R_{\text{н}}^{\text{y}} b_{\text{p}} h_0^2} = \frac{(5{,}1\cdot 10^4)^2}{0{,}6\,(1{,}1\cdot 1{,}2\cdot 140)\,30\cdot 37^2} = 575 \text{ kg/cm}.$$

From formula (67) of SNiP II-B.1-62* we refine the value

$$C_0 = \sqrt{\frac{0{,}15 R_{\text{н}}^{\text{y}} b h_0^2}{q_x}} = \sqrt{\frac{0{,}15\cdot 1{,}1\cdot 1{,}2\cdot 140\cdot 30\cdot 37^2}{575}} = 45 \text{ cm}.$$

We keep the previously assumed value C_0. We find the sectional area of lateral rods per 1 m of the component's length:

$$f_x = \frac{575\cdot 100}{2700\cdot 1{,}32} = 16{,}2 \text{ cm}^2.$$

The lateral reinforcement is placed in and between the flanges of precast elements.

We perform a check of the support section for shear stresses arising in the horizontal plane where precast slabs are connected with the layer of monolithic concrete placed above from formula (69).

$$\tau_{max} = \frac{Q_p}{0{,}9 b_{\text{п}} h_0} = \frac{5{,}1\cdot 10^4}{0{,}9\cdot 85\cdot 37} = 18 \text{ kg/cm}^2.$$

In accordance with Paragraph 3.40 $k_{\text{пов}}$= 0.8 for precast elements with a processed surface. Inasmuch as the grade of precast elements is 300 and that of monolithic concrete 200, we assume the average value of prism strength to be

$$R_{\text{пр}} = \frac{130+80}{2} = 105 \text{ kg/cm}^2.$$

From formula (70) the calculated shearing strength of concrete along the joint is

$$[\tau] = 0{,}25 R_{\text{пр}} k_{\text{пов}} = 0{,}25\cdot 105\cdot 0{,}8 = 21 \text{ kg/cm}^2.$$

Since $\tau<[\tau]$, the joint's shear strength is sufficient.

Calculation of Overhead Cover Collar Beam

In accordance with the accepted structual design schematic, the collar beam represents a triple-span continuous beam. The collar beam's performance at intermediate supports as a continuous component is ensured by placement of necessary reinforcement in the layer of monolithic concrete. The concrete grade in the precast portion of the collar beam is 300:

$$R_{\text{н}} = 160 \text{ kg/cm}^2;\ k_{\text{y}} = 1{,}2;\ k_t = 1{,}1.$$

The collar beam's cross section in the span is shown in Fig. 182. The effective width of the collar beam's flange (width of concrete compression zone in cross section) is assumed equal to 270 cm in conformity with Paragraph 7.18 of SNiP II-B.1-62*.

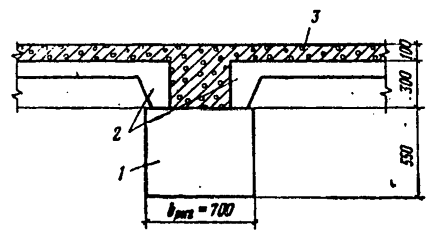


Fig. 182. Calculated section of collar beam in the span

1. Collar beam of precast reinforced concrete
2. Precast reinforced concrete channel slabs
3. Monolithic concrete

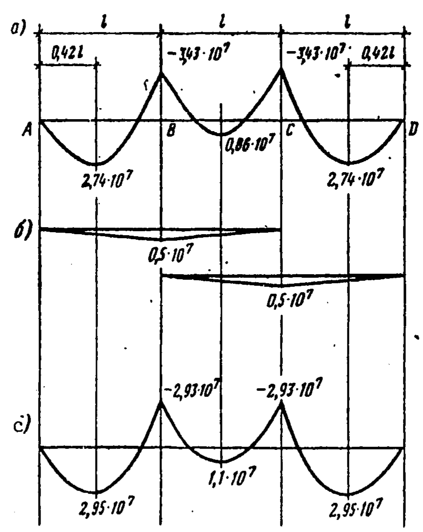


Fig. 183. Moment diagram for beam

a. Before stress redistribution
b. Redistribution of stresses above supports
c. After stress redistribution

We designate the effective span of the collar beam based on the condition of the previously assumed column section of 70 x 70 cm.

The cumulative load acting on the collar beam equals:

$$p_{\text{pir}} = pl_{\text{пл}} = 2{,}01 \cdot 600 = 1206 \text{ kg/cm.}$$

Considering the collar beam's own weight $p_{рнг} = 1220$ kg/cm.

We compute the bending moment:

In the end spans $M_1^{пр} = kp_{рнг} l^2 = 0{,}08 \cdot 1220 \cdot 530^2 = 2{,}74 \cdot 10^7$ kg·см;

Above the second supports from the end

$$M_a^{оп} = -k_p p_{рнг} l^2 = -0{,}1 \cdot 1220 \cdot 530^2 = 3{,}43 \cdot 10^7 \text{ kg·см};$$

In the second span $M_2^{пр} = 0{,}025 \cdot 1220 \cdot 530^2 = 0{,}86 \cdot 10^7$ kg·см.

We construct moment diagrams (Fig. 183). We perform a redistribution of stresses. We reduce the magnitude of the support moment above the second support from the end by 15 percent in order to equalize it with the moment in the first span, for which we add a triangular diagram with ordinate above the support of $+0.5 \cdot 10^7$kg·cm to the moment diagram (Fig. 183b). The moment diagram obtained as a result of stress redistribution is shown in Fig. 183c.

We determine the effective height of the collar beam's section for the moment in the end span after stress redistribution:

$$M_1 = 2{,}95 \cdot 10^7 \text{ kg·см}.$$

We set the reinforcement area in the span and at the support as $F_a = 75$ cm^2 and provisionally assume a dynamic hardening coefficient of reinforced steel $k_y = 1.3$ with $\omega = 100$ 1/sec:

$$R_a^y = m_a k_y R_a = 1{,}1 \cdot 1{,}3 \cdot 3400 = 4870 \text{ kg/см}^2.$$

$$H_0 = \frac{M_1}{F_a R_a^y \gamma} = \frac{2{,}95 \cdot 10^7}{75 \cdot 4870 \cdot 0{,}9} = 90 \text{ см}.$$

Determine the component's reinforcement factor in the span:

$$\mu_{пр} = \frac{F_a}{b_п H_0} = \frac{75}{270 \cdot 90} = 0{,}0031;$$

$$n = \frac{E_a}{E_б} = \frac{2 \cdot 10^6}{2{,}65 \cdot 10^5} = 7{,}55;$$

$$\xi_{пр} = \frac{1}{1{,}8 + \frac{1}{10 \cdot 7{,}55 \cdot 0{,}0031}} = 0{,}161;$$

$$\beta_{пр} = \frac{1 - 0{,}5 \cdot 0{,}161}{1 + \frac{1{,}8 \cdot 7{,}55 \cdot 0{,}0031}{0{,}161}} = 0{,}733.$$

Find the rigidity of the collar beam in the span

$$B_{пр} = 2 \cdot 10^6 \cdot 0{,}0031 \cdot 270 \cdot 90^3 \cdot 0{,}733 = 0{,}9 \cdot 10^{12} \text{ kg·см}^2$$

Determine the component's natural oscillation frequency from the formula

$$\omega_2^п = \frac{18{,}5}{l^2} \sqrt{\frac{B^{пр}}{\bar{m}}} = \frac{18{,}5}{530^2} \sqrt{\frac{0{,}9 \cdot 10^{12}}{0{,}14}} = 167 \text{ 1/sec},$$

where

$$\bar{m} = \frac{0{,}21 \cdot 600 + 70 \cdot 50 \cdot 2{,}4 \cdot 10^{-3}}{981} = 0{,}14 \text{ kg·sec}^2/\text{cm}^2$$

The dynamic hardening factor of reinforced steel from the chart in Fig 76 equals $k_y = 1.35$.

Determine the collar beam's rigidity at the support:

$$\mu_{\text{оп}} = \frac{75}{70 \cdot 90} = 0{,}0119;$$

$$\xi_{\text{оп}} = \frac{1}{1{,}8 + \dfrac{1}{10 \cdot 7{,}55 \cdot 0{,}0119}} = 0{,}342;$$

$$\beta_{\text{оп}} = \frac{1 - 0{,}5 \cdot 0{,}342}{1 + \dfrac{1{,}8 \cdot 7{,}55 \cdot 0{,}0119}{0{,}342}} = 0{,}563;$$

$$B^{\text{оп}} = 2 \cdot 10^6 \cdot 0{,}0119 \cdot 70 \cdot 90^3 \cdot 0{,}563 = 0{,}685 \cdot 10^{12} \text{ kg} \cdot \text{см}^2.$$

The ratio of rigidity at the support and in the span equals:

$$\beta_1 = \frac{0{,}685 \cdot 10^{12}}{0{,}9 \cdot 10^{12}} = 0{,}76.$$

The factor k_2 equals: $k_2 = \dfrac{0{,}26 + 0{,}74\,\beta_1}{0{,}578 + 0{,}422\,\beta_1} = \dfrac{0{,}26 + 0{,}74 \cdot 0{,}76}{0{,}578 + 0{,}422 \cdot 0{,}76} = 0{,}916;$

$$k_2^{\text{н}} = 0{,}8\,k_2 = 0{,}8 \cdot 0{,}916 = 0{,}733.$$

Determine moments of internal stresses:

In the span $\qquad \alpha_{\text{пр}} = \dfrac{0{,}0031\,(1{,}35 \cdot 1{,}1 \cdot 3400)}{1{,}1 \cdot 1{,}2 \cdot 100} = 0{,}118;$

$$M_0^{\text{пр}} = 5050 \cdot 0{,}0031 \cdot 270 \cdot 90^2 (1 - 0{,}5 \cdot 0{,}118) = 3{,}2 \cdot 10^7 \text{ kg} \cdot \text{см};$$

At the support $\qquad \alpha_{\text{оп}} = \dfrac{0{,}0119 \cdot 5050}{1{,}1 \cdot 1{,}2 \cdot 160} = 0{,}285;$

$$M_0^{\text{оп}} = 5050 \cdot 0{,}0119 \cdot 70 \cdot 90^2 (1 - 0{,}5 \cdot 0{,}285) = 2{,}89 \cdot 10^7 \text{ kg} \cdot \text{см}.$$

Moments from the linear static load are:

In the span $\qquad M_{\text{ст}}^{\text{пр}} = 0{,}125 \left(1 - \dfrac{k_2^{\text{н}}}{4}\right)^2 q_{\text{ст}}\, l^2 = 0{,}125 \left(1 - \dfrac{0{,}733}{4}\right)^2 137 \cdot 530^2 =$

$$= 0{,}322 \cdot 10^7 \text{ kg} \cdot \text{см};$$

At the support $\qquad M_{\text{ст}}^{\text{оп}} = 0{,}125\, k_2^{\text{н}}\, q_{\text{ст}}\, l^2 = 0{,}125 \cdot 0{,}733 \cdot 137 \cdot 530^2 = 0{,}354 \cdot 10^7 \text{ kg} \cdot \text{см}.$

Determine the internal stress moments with subtraction of static moments:

In the span $\qquad \overline{M}_0^{\text{пр}} = M_0^{\text{пр}} - k_y M_{\text{ст}}^{\text{пр}} = 3{,}2 \cdot 10^7 - 1{,}35 \cdot 0{,}322 \cdot 10^7 = 2{,}765 \cdot 10^7 \text{ kg} \cdot \text{см}$

At the support $\qquad \overline{M}_0^{\text{оп}} = M_0^{\text{оп}} - k_y M_{\text{ст}}^{\text{оп}} = 2{,}89 \cdot 10^7 - 1{,}35 \cdot 0{,}354 \cdot 10^7 = 2{,}4 \cdot 10^7 \text{ kg} \cdot \text{см}.$

The ratio of moments equals: $\qquad \beta_2 = \dfrac{2{,}4 \cdot 10^7}{2{,}765 \cdot 10^7} = 0{,}867.$

Compute the moment from dynamic load at the support

$$M_p^{\text{оп}} = 0{,}125\, l_{\text{пл}}\, l_p^2\, \Delta p\, k_2^{\text{н}} = 0{,}1 \cdot 1{,}5 \cdot 600 \cdot 530^2 \cdot 0{,}733 = 2{,}31 \cdot 10^7 \text{ kg} \cdot \text{см}.$$

The dynamic-response factor for bending moment for the support equals:

$$k_{м}^{оп} = \frac{\overline{M}_0^{оп}}{M_p^{оп}} = \frac{2,4\cdot10^7}{2,31\cdot10^7} = 1,04.$$

With the values $\beta_1=0,76$; $\beta_2=0,867$; $\omega_2^{н}\theta=167\cdot0,8=133,6>100$ we take $\omega_2^{н}\theta=500$ and $k_{м}^{оп}=1,04$, and find from the chart of Appendix 7 $k_{п}^{оп}\approx3$ и $k_{п}^{пр}\approx3,5$.

From Table 28 we find $k_Q^{оп}= 1.45$.

The expansion angle equals:

At the support

$$\psi_{п}^{оп} = 0,035 + \frac{0,003}{0,285} = 4,55\cdot10^{-2};$$

$$\psi^{оп} = \frac{1,5\cdot600\cdot530^2\cdot3}{106,8\cdot0,9\cdot10^{12}} = 0,422\cdot10^{-2} \ll 0,5\,\psi_{п}^{оп};$$

In the span

$$\psi_{п}^{пр} = 0,035 + \frac{0,003}{0,118} = 6\cdot10^{-2};$$

$$\psi^{пр} = \frac{1,5\cdot600\cdot530^2\cdot3,5}{44\cdot0,9\cdot10^{12}} = 1,19\cdot10^{-2} < \psi_{п}^{пр}.$$

Calculation of the central span of a continuous triple-span collar beam is performed in a similar manner.

Calculation for Lateral Force

The greatest value of lateral force will be at the face of the second support and equals:

$$Q = k_Q^{оп}\,\Delta p l_{пл}\,\frac{l_p}{2} + k_t\,k_y\,(p_{c.в} + p_{гр})\,\frac{l_p}{2} = 1,45\cdot1,5\cdot600\cdot\frac{540}{2} + 1,1\cdot1,35\cdot137\cdot\frac{540}{2} = 4\cdot10^5\text{ kg}.$$

We check the collar beam cross section for the condition of Paragraph 3.37

$$Q = 0,35\cdot1,1\cdot1,2\cdot160\cdot70\cdot90 = 4,65\cdot10^5\text{ kg}.$$

The collar beam's section satisfies the strength condition for transverse force.

We find the design value of transverse force from formula (68):

$$Q_p = Q - 0,8\,\Delta p l_{пл}\,C_0 - 0,5\,(p_{c.в} + p_{гр})\,C_0.$$

We assume the value C_0 equal to 90 cm;

$$Q_p = 4\cdot10^5 - 0,8\cdot1,5\cdot600\cdot90 - 0,5\cdot137\cdot90 = 3,4\cdot10^5\text{ kg}.$$

We determine the value q_x:

$$q_x = \frac{(3,4\cdot10^5)^2}{0,6\cdot1,1\cdot1,2\cdot160\cdot70\cdot90^2} = 1620\text{ kg/см}.$$

We refine the value C_0

$$C_0 = \sqrt{\frac{0,15\cdot1,1\cdot1,2\cdot160\cdot70\cdot90^2}{1620}} = 105\text{ см}.$$

We keep the previously assumed value C_0.

We find the sectional area of lateral reinforcement:

$$f_x = \frac{1620 \cdot 100}{2700 \cdot 1{,}35} = 44{,}4 \text{ см}^2.$$

We perform reinforcement of the precast-monolithic collar beam in the following manner.

Five reinforcement cages with double working rods are placed in the collar beam's precast portion. Three reinforcement cages are led into the layer of monolithic concrete from the collar beam's precast portion.

Reinforcement is placed above the support in the layer of monolithic concrete. We assume the lateral reinforcement to be Ø 16 mm with a spacing of 20 cm (F_a = 50 cm^2).

Calculation of Inner Column

The protective structure is located on soft soil. Loams serve as the base under the foundation.

The density of the loams ρ in conformity with Table 9 is assumed equal to 170 kg·sec^2/m^4; the rate of propagation of the compression wave is 350 m/sec; grade 200 concrete; Class A-II reinforcement.

From formulas (48) and (49) we determine the parameters $\overline{q_1}$ and r:

$$\bar{q}_1 = \frac{a_1 \rho F_\Phi}{2m} \text{ sec}^{-1}.$$

We tentatively determine the foundation area beneath the column, assuming standard base soil resistance of 2.5 kg/cm^2 with consideration of paragraphs 3.13 and 3.27:

$$F_\Phi = \frac{k_д \Delta p l_{пл} l_p + q_{ст} l_{пл} l_p}{[\sigma_{гр}] k_y} =$$

$$= \frac{1{,}2 \cdot 1{,}5 \cdot 600 \cdot 600 + 0{,}21 \cdot 600 \cdot 600}{2{,}5 \cdot 5} \approx 58\,000 \text{ см}^2.$$

We take a foundation height of 1.5 m and a column height of 2.6 m.

We find the cumulative weight of the structure's cell of 6·6 m in plan view from which the load on the column is collected:

$$m = m_\Phi + m_к + m_п;$$

$$m_п = 84 \text{ кгс} \cdot \text{с}^2/\text{см}^2; \; m_к = \frac{70 \cdot 70 \cdot 260 \cdot 2{,}4 \cdot 10^{-}}{981} = 3{,}1 \text{ kg} \cdot \text{sec}^2/\text{cm}^2;$$

$$m_\Phi = \frac{240 \cdot 240 \cdot 150 \cdot 2{,}4 \cdot 10^{-3}}{981} = 21{,}2 \text{ kg} \cdot \text{sec}^2/\text{cm}^2;$$

$$m = 108{,}3 \text{ kg} \cdot \text{sec}^2/\text{cm}^2;$$

$$\bar{q}_1 = \frac{350 \cdot 10^2 \cdot 170 \cdot 240^2}{2 \cdot 108{,}3 \cdot 10^9} = 16 \text{ sec}^{-1}.$$

We find the factor r from the formula

$$r=\sqrt{\frac{a_1}{\bar{q}_1 D}-1}=\sqrt{\frac{350\cdot 10^3}{16{,}0\cdot 240}-1}=2{,}83.$$

From the chart in Fig. 69 with r = 2.83 and the value $\theta\bar{q}_1=0{,}8\cdot 16=12{,}8$ we find $k_{д}^{N}=1{,}17$.

The design value of the normal force acting on the column, from formula (50), must be $N_p \leqslant N_в$, where $N_p=N_{экв}+1{,}2\,N_{ст}$. We take the buckling factor equal to 1;

$$N_{экв}=l_{пл}\,l_p\,k_{д}^{N}\,\Delta p=600\cdot 600\cdot 1{,}17\cdot 1{,}5=6{,}3\cdot 10^5\ \text{kg};$$

$$N_{ст}=q_{ст}\,l_{пл}\,l_p=0{,}21\cdot 600\cdot 600=0{,}756\cdot 10^5\ \text{kg};$$

$$N_p=6{,}3\cdot 10^5+1{,}2\cdot 0{,}756\cdot 10^5=7{,}2\cdot 10^5\ \text{kg}.$$

We determine the requisite area of compressed reinforcement:

$$F_а=\frac{N_p-k_y k_t R_{пр} F_б}{R_{а.с}}=\frac{7{,}2\cdot 10^5-1{,}2\cdot 1{,}1\cdot 130\cdot 4900}{2700}<0.$$

Column reinforcement is accomplished structurally in accordance with Paragraph 12.13 of SNiP II-B.1-62*.

Calculation of Foundation under Inner Column

The foundation area with a centrally applied load on the column is determined from the formula

$$F_ф=\frac{N_p}{R_{гр}^{y}}=\frac{72\cdot 10^5}{12{,}5}=5{,}8\cdot 10^4\ \text{см}^2.$$

The side of the base of a square foundation equals:

$$a=\sqrt{5{,}8\cdot 10^4}=240\ \text{см}.$$

We assume a double-benched foundation. Width of the upper bench is 150 cm.

We determine the tentative foundation height based on the condition of the column pressing upon it.

Grade of concrete for the foundation is 200.

$$H_ф=\frac{7{,}2\cdot 10^5}{2\cdot 140\cdot 20}=130\ \text{см}.$$

We determine the sectional moment for the column face:

$$M=\frac{R_{гр}^{y}\,a\,(a-b_к)^2}{8}=\frac{12{,}5\cdot 240\,(240-70)^2}{8}=0{,}87\cdot 10^7\ \text{kg}\cdot\text{см}.$$

We determine the requisite section of stretched reinforcement:

$$A_0=\frac{M}{b h_0^2 R_и^{y}}=\frac{0{,}87\cdot 10^7}{150\cdot 125^2\cdot 1{,}1\cdot 1{,}2\cdot 100}=0{,}028;$$

$$\gamma_0=0{,}98.$$

We find the cyclic natural oscillation frequency of the foundation with consideration of the added mass (overhead cover, columns):

$$\omega = \sqrt{\frac{k_z}{m}}; \quad k_z = C_z F;$$

$$C_z = \varkappa_z \frac{E}{1-\mu^2} \cdot \frac{1}{\sqrt{F}};$$

$$\varkappa_z = 1{,}14; \quad F = 5{,}8 \cdot 10^4 \text{ cm}^2, \quad E = 1000 \text{kg/cm}^2;$$

$$\mu = 0{,}35;$$

$$C_z = 1{,}14 \frac{1000}{(1-0{,}35^2)\sqrt{5{,}8 \cdot 10^4}} = 5{,}4 \text{ kg/cm}^3;$$

$$k_z = 5{,}4 \cdot 5{,}8 \cdot 10^4 = 3{,}14 \cdot 10^5 \text{ kg/cm};$$

$$\omega = \sqrt{\frac{3{,}14 \cdot 10^5}{108{,}3}} = 51{,}5 \text{ 1/sec}.$$

The dynamic hardening factor for reinforced steel (Class A-II) equals 1.25;

$$F_a = \frac{0{,}87 \cdot 10^7}{0{,}98 \cdot 12{,}5 \cdot 1{,}1 \cdot 1{,}25 \cdot 2700} = 19 \text{ cm}^2.$$

Table of Relationships Among Selected Physical Units to be Eliminated and SI [International System of] Unit

Measure	Unit				Relationship of Units
	To be Eliminated		SI		
	Designation	Notation	Designation	Notation	
Force; load; weight	kg-force ton-force gram-force	kgs ts gs	Newton	N	1 kgs ~ 9.8 N ~ 10 N 1 ts ~ $9.8 \cdot 10^3$ N ~ 10 kN 1 gs ~ $9.8 \cdot 10^{-3}$ N ~ 10 mN
Linear load	kg-force per meter	kgs/m	Newtons per meter	N/m	1 kgs/m ~ 10 N/m
Surface load	kg-force per square meter	kgs/m^2	Newtons per square meter	N/m^2	1 kgs/m^2 ~ 10 N/m^2
Pressure	kg-force per square centimeter	kgs/cm^2	Pascal	Pa	1 kgs/cm^2 ~ $9.8 \cdot 10^4$ Pa ~ 10^5 Pa ~ 0.1 MPa
	millimeters of water column	mm vod. st.			1 mm vod. st. ~ 9.8 Pa ~ 10 Pa
	millimeters of Mercury	mm rt. st.			1 mm rt. st. ~ 133.3 Pa
Mechanical stress	kg-force per square millimeter	kgs/mm^2	Pascal	Pa	1 kgs/mm^2 ~ $9.8 \cdot 10^6$ Pa ~ 10^7 Pa ~ 10 MPa
Young's modulus; shear modulus; bulk modulus	kg-force per square centimeter	kgs/cm^2			1 kgs/cm^2 ~ $9.8 \cdot 10^4$ Pa ~ 10^5 Pa ~ 0.1 MPa
Force moment; moment of a couple	kg-force-meter	kgs·m	Newton-meter	N·m	1 kgs·m ~ 9.8 N·m ~ 10 N·m

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Table of Relationships [Continued]

Measure	Unit				Relationship of Units
	To be Eliminated		SI		
	Designation	Notation	Designation	Notation	
Work (energy)	kg-force-meter	kgs·m	Joule	J	1 kgs·m ~ 9.8 J ~ 10 J
Quantity of heat	calorie kilo-calorie	kal kkal	Joule	J	1 cal ~ 4.2 J 1 kcal ~ 4.2 kJ
Power	kg-force-meter per second horsepower calories per second kilo-calories per hour	kgs·m/s l.s. kal/s kkal/ch	Watts	W	1 kgs·m/sec ~ 9.8 W ~ 10 W 1 hp ~ 735.5W 1 cal/sec ~ 4.2 W 1 kcal/hr ~ 1.16 W
Specific heat	calories per gram-degree Celsius kilo-calories per kilo-gram-degree Celsius	kal/(g·°C) kkal/(kg·°C)	Joules per kg-Kelvin	J/(kg·K)	1 cal/(g·°C) ~ 4.2·10^3 J/(kg·K) 1 kcal/(kg·C) ~ 4.2 kJ/(kg·K)
Heat conduction	calories per second per centi-meter-degree Celsius kilo-calories per hour per meter-degree Celsius	kal/(s·cm·°C) kkal/(ch·m·°C)	Watts per meter-Kelvin	W/(m·K)	1 cal/(sec·cm·°C) ~ 420 W/(m·K) 1 kcal/(hr·m·°C) ~ 1.16 W/(m·K)

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Table of Relationships [Continued]

Measure	Unit				Relationship of Units
	To be Eliminated		SI		
	Designation	Notation	Designation	Notation	
Heat exchange (heat emission) coefficient; heat transfer coefficient	calories per second per square centimeter-degree Celsius	kal/(s·cm^2·°C)	Watts per square meter-Kelvin	W/(m^2·K)	1 cal/(sec·cm^2·°C) ~ 42 KW/(m^2·K)
	kilocalories per hour per square meter-degree Celsius	kkal/(ch·m^2·°C)			1 kcal/(hr·m ·°C) ~ 1.16 KW/(m^2·K)

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