Guidelines

For design of concrete and reinforced concrete structures made of heavy-weight and light-weight concrete without reinforcement prestress

(Addition to SNiP 2.03.01-84)

Moscow
Central Project Institute of Standard Designs
1989

1. GENERAL RECOMMENDATIONS

Basic Positions

Recommendations of the present Guidelines are applied to design of concrete and reinforced concrete structures produced without reinforcement priestess made of heavy-weight, fine and light-weight concrete and used by temperature no more than 50 Celsius degree above zero and no less than 70 Celsius degree below zero.

Notes: 1. Recommendations of the Guidelines are not applied to design of concrete and reinforced concrete structures for water development facilities, bridges, transport tunnels, pipes under filling dams, highways and aerodromes covering.

2. Definitions “heavy-weight concrete”, “fine concrete” and “light-weight concrete” are used in accordance with GOST 25192-82.

Light-weight concretes may have compact and porous structure that’s why in the present Guidelines there are used definitions “light-weight concrete” for light-weight concrete of compact structure and “porous concrete” for light-weight concrete of porous structure with inter-granular openings more than 6 percent.

Types of light-weight and porous concretes as well as their application fields are given in Annex 1.

Concrete and reinforced concrete members of buildings and structures for corrosive atmosphere and high humidity conditions should be designed considering requirements of SNiP 1.03.11-85.

(1.4) Prefabricated members structures must conform to requirements of mechanized production at specialized plants.

It is wise to enlarge elements of prefabricated structures as big as it is possible according to weight-lift ability of installing mechanisms, producing and transportation conditions.

(1.5) For monolithic structures it is necessary to use dimensions applicable for inventory form work as well as enlarged three-dimensional reinforcement cages.

(1.6) It is necessary to pay more attention to rigidity and working life of connections.

Joints and connection structures of members must provide reliable transferring of forces, durability of members in connection zones as well as connection of additional concrete in joints with concrete of structure by means of different structural and technological measures.

(1.7) Concrete members are used:

a) in structures being pressed by little eccentricities of longitudinal forces, not exceeding the values given in Item 3.4;
b) in specific cases in structures being pressed by larger eccentricities as well as in bending structures if their failure does note constitute a danger for human life and equipment safety (members base on solid base etc).

Note: Structures are considered as concrete ones if their durability during the use period is provided only by concrete

(1.8) Design winter temperature of outside air is taken as average air temperature of the coldest five-days week depending on the construction region according to SNiP 2.01.01-82. Design technological temperatures are settled in the project statement.

Environment air humidity is determined as average relative humidity of outside air of the hottest month according to the construction region in compliance with SNiP 2.01.01-82 or as relative air humidity of rooms of heated buildings.

Numerical values of given in the present document concrete and reinforcement design characteristics, limit values of crack openings and deflections are used only during design. For construction quality estimation it is necessary to follow the requirements of correspondent state standards and technical specifications.

Basic Calculation Requirements

(1.10) Concrete and reinforced concrete structures must meet the requirements of the load-carrying capacity calculation (first class limit states) and according to serviceability limit state (second class limit states).

a) *Calculation according to the first class limit states* must protect structures against:
  - Unstable, elastic or other failure (rigidity calculation considering deflection of the structure before failure);
  - Structure stable form failure or position failure.
  - Endurance rupture (endurance limit calculation of the structure which is under effect of repeated load – moving and pulsating);
  - Failures under influence of stresses and adverse environmental impacts (periodic or permanent aggressive influences, freezing and melting etc);

b) *Calculation according to the first class limit states* must protect structures against:
  - Exceeding crack opening (calculation of the crack opening);
  - Exceeding displacements – deflections, rotation angles, vibration (deformation calculation).

It is possible not to make calculation of concrete structures according to second class limit states as well as regarding the endurance limit.

Notes: 1. Calculations of repeated loads are made in compliance with the recommendations of "Guidelines for design of prestressed reinforced concrete structures made of heavy-weight and light-weight concrete" (Moscow, 1986).
2. Calculations of the form stability or position stability as well as calculations of influence of stresses and adverse environmental impacts are made according to normative documents or Guidelines.

(1.11) Design to limit state of the structure in general as well as of members of structure must be made as a rule for all stages – manufacturing, transportation, installing and use; at the same time design schemes must meet the accepted construction solutions.

(1.12) Loads and effects values, values of safety factors as regards the load γ_f, combinations coefficients as well as dividing of loads into dead loads and live loads must be taken according to requirements of SNiP 2.01.07-85.

Loads values must be multiplied by safety factors as regards the purpose taken according to "Registration rules of responsibility degree of buildings and structures during design" approved by Gosstroy of the USSR.

Loads considered during calculations of first class limit states (exploitative) must be taken according to Items 1.15 and 1.17. At the same time to long-term loads belong also a part of total value of short-term loads settled in SNiP 2.01.07-85 and short-term load inserted into the calculation must be taken as reduced by the value considered in long-term load (for example if snow load for the IIIrd region is s = 1000 H/m^2 so snow long-term load will be $s = 0.3 \times 1000 = 300$ H/m^2 and snow short-term load $s = 1000 - 300 = 700$ H/m^2).

Combinations coefficients belong to total value of short-term loads.

It is necessary to consider temperature climatic effects for structures not protected against solar irradiation for climatic sub-regions IVA according to SNiP 2.01.01-82.

(1.13) During calculation of members of prefabricated structures as regards the forces growing during their lifting, transportation and installation it is necessary to insert the load of the member weight with dynamic factor equal to:
1.60 – during transportation
1.40 – during lifting and installing

In this case it is also necessary to consider the load safety factor.

(1.15) Forces in statically indefinable reinforced concrete structures caused by loads and forced displacements (as result of changes of temperature, concrete moisture, supports displacements etc) as well as forces in statically indefinable reinforced concrete structures during their calculation as regards the deformation scheme must be determined considering inelastic concrete and reinforcement deformations and cracks presence.

It is possible to determine forces in statically indefinable reinforced concrete structures on the basis of their linear elasticity for structures whose calculation methods considering inelastic characteristics of reinforced concrete are not worked out as well as for intermediate stage of the calculation considering inelastic characteristics.

(1.16) Width of long-lived and short-lived crack openings for members used in non-aggressive conditions must not exceed values mentioned in Table 1.

Members mentioned in Position 1a of Table 1 can be designed without prestressing only by special justification

Table 1 (1, 2)

Work conditions of the structure	Limit width of crack opening, mm	
	Short-lived a_{crc1}	Long-lived a_{crc2}
1. members carrying the load of liquids or gases if the section is		
a) fully stretched	0.2	0.1
b) partly compressed	0.3	0.2
2. members carrying the load of granular materials	0.3	0.2
3. members used in the ground with variable ground-water elevations	0.3	0.2
4. other members	0.4	0.3

Note. By short-lived crack opening we shall basically understand opening under effect of dead loads, long-term and short-term loads; by long-lived crack opening we shall understand – only under effect of dead loads and long-term loads. At the same time safety factor is equal to 1.

(1.19) For under-reinforced concrete members whose load-carrying capacity becomes exhausted concurrent with crack opening in the stretched concrete zone, sectional area of longitudinal stretched reinforcement must be increased by no less than 15 percent in comparison with calculations requirements.

Such increase is to be fulfilled upon the following condition

$$M_{crc} \geq M_u ,$$

where M_{crc} is crack opening moment determined according to Item 4.2 replacing value $R_{bt,ser}$ by $1.2R_{bt,ser}$;

M_u is moment corresponding to load-carrying capacity exhaust and determined according to Items 3.15-3.80; for eccentric compressed and stretched members values are determined relating to the axis going through core point the most distant from the stretched zone (see Item 4.2).

This requirement can be applied to elements which rest on a solid base.

(1.20) Deflections of members of reinforced concrete structures must not exceed limit values settled considering the following requirements:

a) technological requirements (normal running conditions of cranes, process units, machines, etc);
b) structural requirements (neighbor elements influence; given grade of slope, etc);
c) esthetic requirements (a person’s impression of structure workability).

Deflection limits values are given in Table 2.

Deformation calculation must be made by technological or constructive requirements as regards dead loads, short-term and long-term loads; by esthetic loads as regards dead loads and long-term loads. At the same time it is taken $\gamma_f = 1.0$

By dead loads, short-term and long-term loads beams and slabs deflections must not exceed 1/150 of a span and 1/75 of an overhanging length in all cases.

Limit deflections values can be increased by the height of a camber if it is not restricted by technological or constructive requirements.

If in the lower room with plain ceiling there are partition walls (not supporting) located across the span of member l and if the distance between these partition walls is l_p so the deflection of the member within the distance l_p (counted from the line connecting top points of partition walls axes) can be taken up to $1/200l_p$, at the same time limit deflection must be no more than $1/500l$.

Table 2 (4)

Structure members	Deflection limits
1. Crane beams For manually operated cranes For electric cranes	 $\frac{l}{500}$ $\frac{l}{600}$
2. Floors with a plane ceiling and roof members (except the ones mentioned in position 4) if the span is: $l < 6$ $6 \le l \le 7.5$ $l > 7.5$	 $\frac{l}{200}$ 3 cm $\frac{l}{250}$
3. Floors with ribbed ceiling and stairs members if the span is: $l < 5$ $5 \le l \le 10$ $l > 10$	 $\frac{l}{200}$ 2.5 cm $\frac{l}{400}$
4. Roof elements of agricultural building for production purpose if the span is: $l < 6$ $6 \le l \le 10$ $l > 10$	 $\frac{l}{150}$ 4 cm $\frac{l}{250}$
5. Suspended wall panels if the span is: $l < 6$ $6 \le l \le 7.5$ $l > 7.5$	 $\frac{l}{200}$ 3 cm $\frac{l}{250}$
Symbols: l is beams or slabs span; for consoles it is necessary to take l equal to double overhanging length.	

(1.20) For not connected with neighbor members structures of floor slabs, flights of stairs, platforms etc it is necessary to run additional check as regards the instability: additional deflection caused by short-term center-point load 1000 H by the worst loading scheme must be no more than 0.7 mm.

(1.22) The distance between contraction joints must be settled according to the calculation. It is possible not to make the calculation if the distance between contraction joints by design if temperature of outside air 40 Celsius degrees below zero and higher doesn't exceed values given in Table 3. For framework buildings and structures without top-running bridge crane if in the considered direction there are bracings (stiffening diaphragms) the values given in Table 3 can be multiplied by the coefficient equal to:

$$\delta = \delta_{\Delta t}\delta_t\delta_f ,$$

but no less than one,

Where $\delta_{\Delta t}$ is the coefficient taken equal to $\delta_{\Delta t} = \dfrac{50 \cdot 10^{-5}}{10^{-5}\Delta t_w + \varepsilon}$ for heated buildings and

$\delta_{\Delta t} = \dfrac{60}{\Delta t_c}$ for not heated buildings and structures (here $\Delta t_w, \Delta t_c$ are design temperature changes in Celsius degrees determined in compliance with SNiP 2.01.07-85, ε – is coefficient of strain of longitudinal elements caused by vertical loads. For reinforced concrete elements it is possible to take $\varepsilon = 1 \cdot 10^{-4}$, for other members $\varepsilon = 3 \cdot 10^{-4}$);

$\delta_l = \dfrac{l/h}{9}$ (Here l is the length of the column between fixing points, h is the height of the column section in the direction under consideration);

$\delta_\varphi = 0.4 + \varphi_{ext}/100 \le 1$ (Here φ_{ext} is external air humidity in percents during the hottest month of the year, taken in accordance with SNiP 20.1.01-82).

When considering the coefficient δ the distance between contraction joints must be no more than 150 m for heated buildings made of prefabricated structures, 90 m – for heated buildings made of prefabricated-monolithic and monolithic structures; for not heated building and structures the mentioned above values must be reduced by 20 percent.

Table 3

Structures	Maximum distances in meters between contraction joints allowable without calculation for structures located		
	inside of heated buildings or in the ground	inside of not heated buildings	in the open
1. Concrete structures			
a) prefabricated	40	35	30
b) monolithic:			
by constructive reinforcement	30	25	20
without constructive reinforcement	20	15	10
2. reinforced concrete structures:			
a) prefabricated-frame structure:			
one-storey	72	60	48
multi-storey	60	50	40
b) prefabricated-monolithic and monolithic structures:			
frame structures	50	40	30
solid structures	40	30	25
Note: For reinforced concrete frame structures (pos. 2) the distances between contraction joints are determined without bracings or if bracings are located in the middle of the temperature block.			

During calculation of the floor as regards all limit states the weight of partition walls located along the slabs span is considered in the following manner:

a) The load from weight of blind rigid partition wall (for example reinforced concrete prefabricated wall made of horizontal members, reinforced concrete monolithic wall, stone wall, etc) is applied concentrated at the distance of 1/12 of the partition wall length from its edges;

b) If there is an opening in the rigid partition wall and the opening is located within one half of the partition wall so the load from the smaller pier (including the load of the half part above the opening) is applied concentrated at the distance 1/3 of the pier length and the load of weight of another part of the partition wall is applied at the distance 1/12 of the length of this part from the opening edge and from the partition wall edge; if the opening is arranged differently so the load is applied at the distance 1/18 of corresponding parts of a partition wall and of their edges;
c) If there are two and more openings in a partition wall so the load of the weight of this partition wall is applied concentrated on the centers of parts supported on the floor;
d) For other partition walls 60 percent of their weight is distributed along the partition wall length (on the parts between openings) and 40 percent of the weight is applied in compliance with sub-items "a" – "b".

Local load among members of prefabricated floors made of hollow-cored or solid slabs is spread in the following manner if the joints between slabs are grouted well:

a) By calculations as regards all limit states it is taken the following spread of load from the weight of partition walls located along the span of slabs with the same width:
   - If the partition wall is located within one plate so this plate carries 50 percent of the partition wall weight and two neighbor plates carry 25 percent of its weight;
   - If the partition wall is supported on two neighbor plates so the weight of the partition wall is spread among them.
b) By calculations of the second class limit states local concentrated loads located within a center third of the slab span are applied on the width no more than a length of the span; by the durability calculation such spread of concentrated loads can be applied only if neighbor plates are doweled (see Item 3.115).

Note. If the floor is formed of two slabs supported at three sides and the partition wall is located within one slab so this slab carries 75 percent of the partition wall weight; in this case the load from the partition wall is transferred according to Item 1.20 if the partition wall is located both along and across the slab.

2. MATERIALS FOR CONCRETE AND REINFORCED CONCRETE STRUCTURES

Concrete

(2.3) For concrete and reinforced concrete structures it is necessary to use the following materials:

a) *concrete class as regards the resistance against compression*
   - heavy-weight concrete – B3.5; B5; B7.5; B10; B12.5; B15; B20; B25; B30; B35; B40; B45; B50; B55; B60;
   - fine concrete groups:
     A – aging concrete or concrete tempered by pressure of air on the sand with fineness modulus more than 2.0 – B3.5; B5; B7.5; B10; B12.5; B15; B20; B25; B30;
     B (Rus. – Б) – the same with fineness modulus 2.0 and less – B3.5; B5; B7.5; B10; B12.5; B15; B20; B25; B30;
     C (Rus. – B) – autoclaved concrete – B15; B20; B25; B30; B40; B45; B50; B55; B60;
   - light-weight concrete if average density grades are the following:
     D800, D900 – B2.5; B3.5; B5; B7.5*
     D1000, D1100 – B2.5; B3.5; B5; B7.5; B10; B12.5*;
     D1200, D1300 – B2.5; B3.5; B5; B7.5; B10; B12.5; B15*;

D1400, D1500 – B3.5; B5; B7.5; B10; B12.5; B15; B20*; B25*; B30*;
D1600, D1700 – B5; B7.5; B10; B12.5; B15; B20; B25*; B30*; B35*;
D1800, D1900 – B10; B12.5; B15; B20; B25*; B30*; B35*; B40*;
D2000 – B20; B25; B30; B35*; B40*;

- porous concrete if average density grades are:
D800, D900, D1000 – B2.5; B3.5; B5; B7.5;
D1100, D1200, D1300, D1400 – B3.5; B5; B7.5

b) *concrete class as regards the resistance to frost:*
heavy-weight and fine concrete – F50; F75; F100; F150; F200; F300; F400; F500;
light-weight concrete – F25; F35; F50; F75; F100; F150; F200; F300; F400; F500;
porous concrete – F15; F25; F35; F50; F75; F100;

c) *concrete class as regards the water permeability* – W2; W4; W6; W8; W10; W12;

d) *concrete class as regards the average density:*
light-weight concrete – D800; D900; D1000; D1100; D1200; D1300; D1400; D1500; D1600; D1700; D1800; D1900; D2000;
porous concrete – D800; D900; D1000; D1100; D1200; D1300; D1400

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* The present grade of light-weight concrete based on natural aggregate, foamed slag and fly ash aggregate can be used only if it is approved by the manufacturing plant.

Notes:
1. It is necessary to take concrete grade according to resistance to axis tension for structures whose resistance to tension is the main characteristic in compliance with SNiP 2.03.01-84.
2. Definitions "concrete grade" and "concrete class" see in GOST 25192-82.
3. According the present Guidelines porous concrete can be used only for eccentric compressed concrete and reinforced concrete members.

(2.4) Concrete age conforming to its grade according to resistance to compression is taken in compliance with possible terms of structure loading by design loads, mode of building, concrete hardening conditions. In case if there is no this data concrete age is taken 28 days.

Concrete strength of members of prefabricated structures is taken according to GOST 13015.0-83.

(2.5) For reinforced concrete structures it is impossible to use:

- heavy-weight and fine concrete less than B7.5 concrete grade according to resistance to compression;
- light-weight concrete of grade B2.5 as regards the resistance to compression – for one-layer structures;
- concrete of grade no less than B25 – for heavily loaded reinforced concrete axial element (for example for columns carrying heavy crane loads and for columns of lower storeys of multistory buildings);
- concrete of grade no less than B15 for thin-walled reinforced concrete structures as well as for walls of buildings and structures built up in slip or traveling forms.

For concrete compressed members it is not recommended to use more than B30 concrete grade.

(2.8) For building-in of members joints of prefabricated reinforced concrete structures concrete grade must be taken according to work conditions of joined members but it must be no less than B7.5.

(2.9) Concrete grades as regards resistance to frost and to water of concrete and reinforced concrete structures (according to their use mode and design winter temperatures of outside air in the construction region) must be the following:

- no less than the ones shown in Table 4 – for buildings structures (except external walls of heated buildings);

- no less than the ones sown in Table 5 – for external walls of heated buildings.

Table 4 (9)

Structure work conditions		Concrete grade no less than					
mode characteristics	design winter temperature of external air, in Celsius degrees	according to resistance to frost			according to resistance to water		
		for building structures (except external walls of heated buildings) of responsibility degree					
		I	II	III	I	II	III
1. Alternate freezing and melting							
a) in waterlogged state (for example structures located in the ground layer seasonally melting in the permafrost region)	Lower than 40 degrees below zero	F300	F200	F150	W6	W4	W2
	Lower than 20 degrees below zero up to 40 degrees below zero	F200	F150	F100	W4	W2	Not regulated
	Lower than 5 degrees below zero up to 20 degrees below zero	F150	F100	F75	W2	Not regulated	
	5 degrees below zero and more	F100	F75	F50	Not regulated		
b) in conditions of occasional water saturation (for example overland structures exposed to atmospheric effects)	Lower than 40 degrees below zero	F200	F150	F100	W4	W2	Not regulated
	Lower than 20 degrees below zero up to 40 degrees below zero	F100	F75	F50	W2	Not regulated	
	Lower than 5 degrees below zero up to 20 degrees below zero	F75	F50	F35*	Not regulated		
	5 degrees below zero and more	F50	F35*	F25*	Not regulated		
c) in air humidity conditions without occasional water saturation (for example structures exposed to atmospheric effects but protected against atmospheric precipitation)	Lower than 40 degrees below zero	F150	F100	F75	W4	W2	Not regulated
	Lower than 20 degrees below zero up to 40 degrees below zero	F75	F50	F35*	Not regulated		
	Lower than 5 degrees below zero up to 20 degrees below zero	F50	F35*	F25*	Not regulated		
	5 degrees below zero and more	F35*	F25*	F15**	Not regulated		
2. Possible occasional temperature influence lower than 0 degree below zero							
a) in waterlogged state (for example structures located in the ground or under water)	Lower than 40 degrees below zero	F150	F100	F75	Not regulated		
	Lower than 20 degrees below zero up to 40 degrees below zero	F75	F50	F35	Not regulated		
	Lower than 5 degrees below zero up to 20 degrees below zero	F50	F35*	F25	Not regulated		
	5 degrees below zero and more	F35*	F25*	Not regulated	Not regulated		

b) in air humidity conditions (for example internal structures of heated buildings during construction and assembling)	Lower than 40 degrees below zero	F75	F50	F35*	Not regulated
	Lower than 20 degrees below zero up to 40 degrees below zero	F50	F35*	F25*	Not regulated
	Lower than 5 degrees below zero up to 20 degrees below zero	F35*	F25*	F15**	Not regulated
	5 degrees below zero and more	F25*	F15**	Not regulated	Not regulated

* For heavy-weight and fine concrete the grades as regards resistance to frost are not regulated.
** For heavy-weight, fine and light-weight concrete the grades as regards resistance to frost are not regulated.

Notes: 1. Concrete grades as regards resistance to frost and to water for water supply and sewer systems buildings as well as for piles and pile shells must be taken in compliance with requirements of corresponding normative documents.
2. Design winter temperatures of external air are taken according to instructions of Item 1.8.

Table 5 (10)

Structure work conditions		Minimum concrete grade according to resistance to frost of external walls of heated buildings made of					
relative degree of humidity of internal air inside of rooms φ_{int}, in percents	design winter temperature of external air, in Celsius degrees	light-weight, porous concrete			heavy-weight, fine concrete		
		for building structures (except external walls of heated buildings) of responsibility degree					
		I	II	III	I	II	III
1. $\varphi_{int} > 75$	Lower than 40 degrees below zero	F100	F75	F50	F200	F150	F100
	Lower than 20 degrees below zero up to 40 degrees below zero	F75	F50	F35	F100	F75	F50
	Lower than 5 degrees below zero up to 20 degrees below zero	F50	F35	F25	F75	F50	Not regulated
	5 degrees below zero and more	F35	F25	F15*	F50	Not regulated	
2. $60 < \varphi_{int} \leq 75$	Lower than 40 degrees below zero	F75	F50	F35	F100	F75	F50
	Lower than 20 degrees below zero up to 40 degrees below zero	F50	F35	F25	F50	Not regulated	
	Lower than 5 degrees below zero up to 20 degrees below zero	F35	F25	F15*	Not regulated		
	5 degrees below zero and more	F25	F15*	Not regulated			
3. $\varphi_{int} \leq 60$	Lower than 40 degrees below zero	F50	F35	F25	F75	F50	Not regulated
	Lower than 20 degrees below zero up to 40 degrees below zero	F35	F25	F15*	Not regulated		
	Lower than 5 degrees below zero up to 20 degrees below zero	F25	F15*	Not regulated			
	5 degrees below zero and more	F15*	Not regulated				

* For light-weight concretes the grades as regards resistance to frost are not regulated.

Notes: 1. If structures made of heavy-weight, fine and light-weight concretes have vapor- and hydro-insulation so their grades as regards resistance to frost shown in the present table must be decreased by one degree.
2. Design winter temperatures of external air are taken according to instructions of Item 1.8.

(2.10) For building-in of members joints of prefabricated reinforced concrete structures exposed to freezing temperature of external air during use period or assembling it is necessary to use concretes of design grades as regards resistance to frost and water no less than grades of concrete of joined members.

For light-weight concretes it is necessary to take concrete grades as regards average density in compliance with Table 6.

Table 6

Light-weight concrete grade as regards the resistance to compression	Grades regarding average density for				
	expanded-clay concrete shungite concrete	slag-pumeconcrete slag-concrete	perlite concrete	concrete of natural expanded aggregate	agloporite concrete
B2.5	D800–D1000	D1000–D1400	D800–D900	D800–D1200	D1000–D1200
B3.5	D800–D1100	D1100–D1500	D800–D1000	D900–D1300	D1100–D1300
B5	D800–D1200	D1200–D1600	D800–D1100	D1000–D1400	D1200–D1400
B7.5	D900–D1300	D1300–D1700	D900–D1200	D1100–D1500	D1300–D1500
B10	D1000–D1400	D1400–D1800	D1000–D1300	D1200–D1600	D1400–D1600
B12.5	D1000–D1400	D1400–D1800	D1000–D1400	D1200–D1600	D1400–D1600
B15	D1200–D1700	D1600–D1800	D1300–D1600	D1500–D1700	D1600–D1800
B20	D1300–D1800	D1700–D1900	–	D1600–D1800	D1700–D1900
B25	D1300–D1800	D1800–D1900	–	D1700–D1900	D1700–D1900
B27.5*	D1400–D1800	D1900–D2000	–	D1800–D2000	D1800–D2000
B30	D1500–D1800	–	–	D1900–D2000	D1900–D2000
B35	D1600–D1900	–	–	–	–
B40	D1700–D1900	–	–	–	–

* Is used with a view to economize cement in comparison with use of concrete of grade B30 and to save other technical-economical characteristics of the structure

Standard and Design Characteristics of Concrete

(2.11) Standard resistance of concrete is also resistance to centric compression of prism (prism strength) R_{bn} and resistance to centric tension R_{btn}.

Design resistances of concrete R_{bn} and R_{btn} according to concrete class B are given in Table 7.

(2.11, 2.13) Design resistances of concrete for first class limit states R_b and R_{bt} are determined by means of dividing of standard resistances into safety factors for concrete equal to: by tension $\gamma_{bc} = 1.3$; by compression $\gamma_{bt} = 1.5$.

Design concrete resistances R_b and R_{bt} are to be decreased (or increased) by means of multiplying by concrete work conditions coefficients γ_{bi} considering work conditions of the structure, process of manufacturing, sections dimensions etc.

Table 7 (12)

Resistance type	Concrete	Standard resistances of concrete R_{bn} and R_{btn} and design resistances for second class limit states $R_{b,ser}$ and $R_{bt,ser}$, in Mega Pascal (kilogram-force per cm[2]) if concrete grade as regards resistance to compression is							
		B2.5	B3.5	B5	B7.5	B10	B12.5	B15	B20
Axial compression (prism strength) R_{bn} and $R_{b,ser}$	heavy-weight, fine, light-weight	1.9 (19.4)	2.7 (27.5)	3.5 (35.7)	5.5 (56.1)	7.5 (76.5)	9.5 (96.9)	11.0 (112)	15.0 (153)
Axial tension R_{btn} and $R_{bt,ser}$	heavy-weight, fine[1], light-weight with dense aggregate	0.29 (2.96)	0.39 (4.00)	0.55 (5.61)	0.70 (7.14)	0.85 (8.67)	1.00 (10.2)	1.15 (11.7)	1.40 (14.3)
	Light-weight concrete with porous aggregate[2]	0.29 (2.96)	0.39 (4.00)	0.55 (5.61)	0.70 (7.14)	0.85 (8.67)	1.00 (10.2)	1.10 (11.2)	1.20 (12.2)
Resistance type	Concrete	Standard resistances of concrete R_{bn} and R_{btn} and design resistances for second class limit states $R_{b,ser}$ and $R_{bt,ser}$, in Mega Pascal (kilogram-force per cm[2]) if concrete grade as regards resistance to compression is							
		B25	B30	B35	B40	B45	B50	B55	B60
Axial compression (prism strength) R_{bn} and $R_{b,ser}$	heavy-weight, fine, light-weight	18.5 (189)	22.0 (224)	25.5 (260)	29.0 (296)	32.0 (326)	36.0 (367)	39.5 (403)	43.0 (438)
Axial tension R_{btn} and $R_{bt,ser}$	heavy-weight, fine[1], light-weight with dense aggregate	1.60 (16.3)	1.80 (18.4)	1.95 (19.9)	2.10 (21.4)	2.20 (22.4)	2.30 (23.5)	2.40 (24.5)	2.50 (25.5)
	Light-weight concrete with porous aggregate[2]	1.35 (13.8)	1.50 (15.3)	1.65 (16.8)	1.80 (18.4)	–	–	–	–

[1] For fine concrete of groups Б (see Item 2.1) values R_{btn} and $R_{bt,ser}$ are decreased by 15 percent.

[2] For expanded-clay perlite concrete on expanded perlite sand values R_{btn} and $R_{bt,ser}$ are decreased by 15 percent.

Note. For porous concrete values R_{bn} and $R_{b,ser}$ are taken the same as for light-weight concrete and values R_{btn} and $R_{bt,ser}$ are multiplied by coefficient 0.7.

Design resistances of concrete for second class limit states $R_{b,ser}$ and $R_{bt,ser}$ are taken equal to standard resistances and are inserted into the calculation with the concrete work condition coefficient $\gamma_{bi} = 1.0$.

Design resistances of concrete according to concrete resistance to compression are given: in Table 8 – for the first class limit states; in Table 7 – for the second class limit states.

Design resistances given in Table 8 include work condition coefficient γ_{b2} considering duration of loads action influence and strength gain of concrete; coefficient γ_{b2} usage order is given in Item 3.1.

In case of need design resistances of concrete given in Table 8 must be multiplied by work conditions coefficients according to Table 9.

(2.14) Concrete tangent modulus of elasticity values E_b by tension and compression are taken according to Table 11.

For concretes being permanently frozen and melted (see pos. 1 of Table 4) values E_b given in Table 11 must be multiplied by work condition coefficient γ_{b6} taken according to Table 10.

(2.15) Linear temperature deformation coefficient α_{bt} by temperature variation from 40 degree below zero up to 50 degree above zero is taken equal to:

- 1×10^{-5} °C^{-1} – for heavy-weight, fine and light-weight concrete with fine dense aggregate;
- 0.7×10^{-5} °C^{-1} – for light-weight concrete with fine porous aggregate.
- 0.8×10^{-5} °C^{-1} – for porous concrete.

(2.16) Prime coefficient of concrete deformation v (Poisson number) is taken equal to 0.2 for all concrete types and modulus of shear of concrete G is taken equal to 0.4, corresponding values E_b given in Table 11.

For determination of weight of reinforced concrete or concrete structures concrete density is taken equal to: 2400 kg/m^3 – for heavy-weight concrete; 2200 kg/m^3 – for fine concrete; for light-weight and porous concrete it is necessary to multiply concrete grade as regards average density D by 1.05 – for concrete grade B12.5 and more, and by $1+w/100$ (where w is gravimetric humidity of concrete during its use determined according to SNiP II-3-79**, it is possible to take w equal to 10 percent) – for concrete grade B10 and less. During calculation of structures at stage of manufacturing and transportation light-weight and porous concrete density is determined considering transport volume humidity ω by formula $D+\frac{\omega}{100}1000$ where ω = 15 and 20 percent correspondingly for light-weight and porous concrete grade B10 and less and ω = 10 percent for light-weight concrete of class B12.5 and more.

Reinforced concrete density by reinforcement content 3 percent and less can be taken more than concrete density by 100 kg/m^3; if reinforcement content is more than 3 percent so density is determined as a sum of concrete and reinforcement weight per unit of volume of reinforced concrete structure. At the same time weight of 1 m of reinforcement steel is taken according to Annex 4 and weight of strip iron, angle steel and section steel – according to state standards. During determination of external walling structures weight made of light-weight concrete of grade B100 and less it is necessary to consider high density of textured layers.

For determination of loads of dead weight of the structure it is possible to take its specific weight kN/m^3 equal to 0.01 of density kg/m^3.

Reinforcement

(2.19) As non-prestressed reinforcement of reinforced concrete structures (except the ones mentioned in Item 2.15):

it is necessary to use:

a) ribbed rod reinforcement A-III, and At-IIIC;
b) ribbed regular reinforcement wire of class Bp-I in welded meshes and frameworks

it is possible to use:

c) ribbed rod reinforcement A-II and plain reinforcement A-I for cross reinforcement as well as for working longitudinal reinforcement if other kinds of reinforcement can't be used;
d) regular reinforcement wire of class Bp-I for bound stirrups of beams up to 400 mm high and columns.

Reinforcement grade A-III, At-IIIC, A-II and A-I must be used in form of welded frameworks and welded meshes.

Under economical justification it is possible to use non-prestressed reinforcement A-IV, A-V and A-VI as pressed reinforcement, and reinforcement A-IV as stretched reinforcement. It is also possible to use reinforcement A-IIIв as stretched reinforcement. The elements with mentioned above reinforcement must be designed in compliance with "Guidelines for design of prestressed reinforced concrete structures made of heavy-weight and light-weight concrete" (Gosstroy USSR, 1986)

As constructive reinforcement of reinforced concrete structures it is also possible to use regular plain bars B-I.

Notes: 1. In the present document there is used the definition "bar" for reinforcement of any diameter, type and section.
2. Special purpose rod reinforcement A-II is lettered as Ac-II with the letter "c".

(2.20) In structures with non-prestressed reinforcement which are under gas or liquid pressure:

it is necessary to use:

a) rod reinforcement A-II and A-I;

it is possible to use:

b) rod reinforcement A-III and At-IIIC;
c) reinforcement wire Bp-I.

(2.23) When choosing type and grade of steel for reinforcement as well as rolled iron for embedded elements it is necessary to consider temperature conditions of use of the structure and loading schemes according to Table 12 and 13.

During installation works performed during cold seasons in climatic regions with design winter temperature less than 40 Celsius degrees below zero load-carrying capacity of structures with reinforcement which can be used only in heated buildings must be provided reasoning from design resistance of reinforcement with reduction factor 0.7 and from design load with safety factor $\gamma_f = 1.0$

(2.24) For lifting loops of members of prefabricated reinforced concrete and concrete structures it is necessary to use hot-rolled reinforcement steel Ac-II of grade 10ГТ and A-I of grade ВСт3сп2 and ВСт3пс2.

If the installation of structures is possible by design winter temperature lower than 40 Celsius degrees below zero so it is possible to use steel of grade ВСт3пс2 for lifting loops.

Table 8

Resistance type	Concrete	Work condition coefficient γ_{b2}	Design resistance of concrete for the first class limit states R_b and R_{bt}, Mega Pascal (kilogram-force/cm^2) if class of concrete as regards the resistance to compression is															
			B2.5	B3.5	B5	B7.5	B10	B12.5	B15	B20	B25	B30	B35	B40	B45	B50	B55	B60
Axial compression (prism strength) R_b	Heavy-weight, fine and light-weight	0.9	1.3 (13.3)	1.9 (19.4)	2.5 (25.5)	4.0 (4.08)	5.4 (55)	6.7 (68.5)	7.7 (78.5)	10.5 (107)	13.0 (133)	15.5 (158)	17.5 (178)	20.0 (204)	22.5 (230)	25.0 (230)	27.0 (275)	29.5 (300)
		1.0	1.5 (15.3)	2.1 (21.4)	2.8 (28.6)	4.5 (45.9)	6.0 (61.2)	7.5 (76.5)	8.5 (86.7)	11.5 (117)	14.5 (148)	17.0 (173)	19.5 (199)	22.0 (224)	25.0 (255)	27.5 (280)	30.0 (306)	33.0 (336)
		1.1	1.6 (16.3)	2.3 (23.4)	3.1 (32.6)	4.9 (50)	6.6 (67.3)	8.2 (83.5)	9.4 (96)	12.5 (128)	16.0 (163)	19.0 (194)	21.5 (219)	24.0 (245)	27.5 (280)	30.5 (310)	33.0 (334)	36.5 (370)
Axial tension R_{bt}	Heavy-weight, fine[1] and light-weight concrete with fine dense aggregate	0.9	0.18 (1.84)	0.23 (2.34)	0.33 (3.33)	0.43 (4.39)	0.51 (5.20)	0.59 (6.01)	0.67 (6.83)	0.80 (8.16)	0.95 (9.7)	1.10 (11.2)	1.15 (11.7)	1.25 (12.7)	1.30 (13.3)	1.40 (14.3)	1.45 (14.8)	1.50 (15.3)
		1.0	0.20 (2.04)	0.26 (2.65)	0.37 (3.77)	0.48 (4.89)	0.57 (5.81)	0.66 (6.73)	0.75 (7.65)	0.90 (9.18)	1.05 (10.7)	1.20 (12.2)	1.30 (13.3)	1.40 (14.3)	1.45 (14.8)	1.55 (15.8)	1.60 (16.3)	1.65 (16.8)
		1.1	0.22 (2.24)	0.29 (2.96)	0.41 (4.18)	0.53 (5.40)	0.63 (6.43)	0.73 (7.45)	0.82 (8.36)	1.00 (10.2)	1.15 (11.7)	1.30 (13.3)	1.45 (14.8)	1.55 (15.8)	1.60 (16.3)	1.70 (17.3)	1.75 (17.8)	1.80 (18.4)
	Light-weight concrete with fine porous aggregate[2]	0.9	0.18 (1.84)	0.23 (2.34)	0.33 (3.33)	0.43 (4.39)	0.51 (5.20)	0.59 (6.01)	0.66 (6.73)	0.72 (7.34)	0.81 (8.26)	0.90 (9.18)	1.00 (10.2)	1.10 (11.2)	–	–	–	–
		1.0	0.20 (2.04)	0.26 (2.65)	0.37 (3.77)	0.48 (4.89)	0.57 (5.81)	0.66 (6.73)	0.74 (7.55)	0.80 (8.16)	0.90 (9.18)	1.00 (10.2)	1.10 (11.2)	1.20 (12.2)	–	–	–	–
		1.1	0.22 (2.24)	0.29 (2.96)	0.41 (4.18)	0.53 (5.40)	0.63 (6.43)	0.73 (7.45)	0.81 (8.26)	0.90 (9.18)	1.00 (10.2)	1.10 (11.2)	1.2 (12.2)	1.30 (13.3)	–	–	–	–

[1] For fine concrete of group Б (see Item 2.1) values R_{bt} are decreased by 15 percent.
[2] For expanded-clay perlite concrete on expanded perlite sand values R_{bt} are decreased by 15 percent.

Notes: 1. For porous concrete values R_b are taken the same like for light-weight concrete and values R_{bt} are multiplied by the coefficient 0.7.

2. Application conditions of work condition coefficient γ_{b2} are given in Item 3.1.

3. Design concrete resistance with the work condition coefficient $\gamma_{b2} = 1.0$ are taken in compliance with Table 13 of SNiP 2.03.01-84.

Table 9 (15)

Factors providing work condition coefficient insertion	Work condition coefficient of concrete	
	graphical symbol	number identification
1. Concreting in vertical position (concreting layer height is more than 1.5 m)	γ_{b3}	0.85*
2. Concreting of monolithic poles and reinforced concrete columns with maximum section dimension less than 30 cm	γ_{b5}	0.85
3. Alternate freezing and melting	γ_{b6}	See Table 10
4. Use of not protected against solar radiation structures in climatic sub-region IVA according to SNiP 2.01.01-82	γ_{b7}	0.85
5. Concrete structures	γ_{b9}	0.90
6. Concrete structures of heavy-weight concrete B35 and higher or of fine concrete B25 and higher	γ_{b10}	$0.3+\omega\leq1$ (value ω see in Item 3.14)
7. Concrete for joints filling of prefabricated elements if thickness of the joint is less than 1/5 of the least dimension of the member section and less than 10 cm.	γ_{b12}	1.15

*For members of porous concrete $\gamma_{b3} = 0.80$

Notes: 1. Work condition coefficients according pos. 3-5 must be considered during determination of design resistances R_b and R_{bt}, according other positions only during determination of R_b.

2. Work conditions coefficients of concrete are inserted independently on each other but at the same time their product [including γ_{b2} (see Item 3.1)] must be no less than 0.45.

Table 10 (17)

Structure application conditions	Design winter temperature of external air, Celsius degrees	Work conditions coefficient of concrete γ_{b6} by alternate freezing and melting of the structure	
		for heavy-weight and fine concrete	for light-weight and porous concrete
Alternate freezing and melting a) in water saturated state (see pos. 1a of Table 4);	Lower than 40 degrees below zero	0.70	0.80
	Lower than 40 degrees below zero up to 40 degrees below zero	0.85	0.90
	Lower than 5 degrees below zero up to 20 degrees below zero	0.90	1.00
	5 degrees below zero and higher	0.95	1.00
b) in conditions of occasional water saturation (see pos. 1b of Table 4)	Lower than 40 degrees below zero	0.90	1.00
	40 degrees below zero and higher	1.00	1.00

Notes: 1. Design winter temperature of external air is taken according to Item 1.8.

2. If concrete grade as regards resistance to frost in comparison with a required one according to Table 4 the coefficient of the present table can be decreased by 0.05 according to each decrease step but they cannot be more than 1.

Table 11 (18)

Concrete	Prime concrete modulus of elasticity $E_b \cdot 10^{-3}$ Mega Pascal (kilogram-force/cm^2) if concrete class as regards resistance to compression is															
	B2.5	B3.5	B5	B7.5	B10	B12.5	B15	B20	B25	B30	B35	B40	B45	B50	B55	B60
Heavy-weigh:																
- of air hardening;	–	0.95 (96.9)	13.0 (133)	16.0 (163)	18.0 (184)	21.0 (214)	23.0 (235)	27.0 (275)	30.0 (306)	32.5 (331)	34.5 (352)	36.0 (367)	37.5 (382)	39.8 (398)	39.5 (403)	40.0 (408)
- exposed to thermal treatment by air pressure	–	8.5 (86.7)	11.5 (117)	14.5 (148)	16.0 (163)	19.0 (194)	20.5 (209)	24.0 (245)	27.0 (275)	29.0 (296)	31.0 (316)	32.5 (332)	34.0 (347)	35.0 (357)	35.5 (362)	36.0 (367)
Fine concrete of groups:																
A–of air hardening;	–	7.0 (71.4)	10.0 (102)	13.5 (138)	15.5 (158)	17.5 (178)	19.5 (199)	22.0 (224)	24.0 (245)	26.0 (265)	27.5 (280)	28.5 (291)	–	–	–	–
exposed to thermal treatment by air pressure	–	6.5 (66.3)	9.0 (92)	12.5 (127)	14.0 (143)	15.5 (158)	17.0 (173)	20.0 (204)	21.5 (219)	23.0 (235)	24.0 (245)	24.5 (250)	–	–	–	–
Б– of air hardening;	–	6.5 (66.3)	9.0 (91.8)	12.5 (127)	14.0 (143)	15.5 (158)	17.0 (173)	20.0 (204)	21.5 (219)	23.0 (235)	–	–	–	–	–	–
exposed to thermal treatment by air pressure	–	5.5 (56.1)	8.0 (81.6)	11.5 (117)	13.0 (133)	14.5 (148)	15.5 (158)	17.5 (178)	19.0 (194)	20.5 (209)	–	–	–	–	–	–
B–of autoclave hardening	–	–	–	–	–	–	16.5 (168)	18.0 (184)	19.5 (199)	21.0 (214)	22.0 (224)	23.0 (235)	23.5 (240)	24.0 (245)	24.5 (250)	25.0 (255)
Light-weight and porous of grade as regards average density D:																
800	4.0 (40.8)	4.5 (45.9)	5.0 (51.0)	5.5 (56.1)	–	–	–	–	–	–	–	–	–	–	–	–
1000	5.0 (51.0)	5.5 (56.1)	6.3 (62.4)	7.2 (73.4)	8.0 (81.6)	8.4 (85.7)	–	–	–	–	–	–	–	–	–	–
1200	6.0 (61.2)	6.7 (68.3)	7.6 (77.5)	8.7 (88.7)	9.5 (96.9)	10.0 (102)	10.5 (107)	–	–	–	–	–	–	–	–	–
1400	7.0 (71.4)	7.8 (79.5)	8.8 (89.7)	10.0 (102)	11.0 (112)	11.7 (119)	12.5 (127)	13.5 (138)	14.5 (148)	15.5 (158)	–	–	–	–	–	–
1600	–	9.0 (91.8)	10.0 (102)	11.5 (117)	12.5 (127)	13.2 (135)	14.0 (143)	15.5 (158)	16.5 (168)	17.5 (178)	18.0 (184)	–	–	–	–	–
1800	–	–	11.2 (114)	13.0 (133)	14.0 (143)	14.7 (150)	15.5 (158)	17.0 (173)	18.5 (189)	19.5 (199)	20.5 (209)	21.0 (214)	–	–	–	–
2000	–	–	–	14.5 (148)	16.0 (163)	17.0 (173)	18.0 (184)	19.5 (199)	21.0 (214)	22.0 (224)	23.0 (235)	23.5 (240)	–	–	–	–

Notes: 1. Fine concrete groups are given in Item 2.1.

2. For light-weight and porous concrete by intermediate values of concrete grade as regards average density initial elasticity modulus is taken according to linear interpolation.

3. For light-weight and porous concrete values E_b are given by use gravimetric humidity w which is 5 percent for concrete B12.5 and higher and 10 percent – for concrete B10 and lower. If for concrete B10 and lower gravimetric humidity w determined in compliance with SNiP II-3-79** is more than 10 percent so values E_b can be increased according to Table 11 if relative grade as regards average density D (100+w)/110 (where D is concrete grade as regards average density).

4. For heavy-weight concrete exposed to autoclave treatment values E_b given in Table 11 for natural hardening concrete must be multiplied by the coefficient 0.75.

5. For not protected against solar radiation structures designed for use in climatic sub-region IVA according to SNiP 2.01.01-82 E_b given in Table 11 must be multiplied by the coefficient 0.85

Table 12 (Annex 1)

Reinforcement types and documents regulating its quality	Reinforcement class	Steel grade	Reinforcement diameter, mm	Use conditions of the structure by									
				static load					dynamic and repeated load				
				in heated buildings	in open air and in not heated buildings by design temperature in Celsius degrees				in heated buildings	in open air and in not heated buildings by design temperature in Celsius degrees			
					up to 30 degrees below zero	lower than 30 degrees below zero up to 40 degrees below zero	lower than 40 degrees below zero up to 55 degrees below zero	lower than 55 degrees below zero up to 70 degrees below zero		up to 30 degrees below zero	lower than 30 degrees below zero up to 40 degrees below zero	lower than 40 degrees below zero up to 55 degrees below zero	lower than 55 degrees below zero up to 70 degrees below zero
Hot-rolled plain rod reinforcement, GOST 5781-82 and GOST 380-71	A-I	Ст3сп3	6–40	+	+	+	+	+[1]	+	+	–	–	–
		Ст3пс3	6–40	+	+	+	–	–	+	+	–	–	–
		Ст3кп3	6–40	+	+	–	–	–	+	+	–	–	–
		ВСт3сп2	6–40	+	+	+	+	+	+	+	+	+	+
		ВСт3пс2	6–40	+	+	+	+	–	+	+	+	–	–
		ВСт3кп2	6–40	+	+	–	–	–	+	+	–	–	–
		ВСт3Гпс2	6–18	+	+	+	+	+[1]	+	+	+	+	+[1]
Hot-rolled ribbed rod reinforcement	A-II	ВСт5сп2	10–40	+	+	+	+[1]	+[1]	+	+	+[1]	–	–
		ВСт5пс2	10–16	+	+	+	+[1]	–	+	+	+[1]	–	–
			18–40	+	+	–	–	–	+	+[1]	–	–	–
		18Г2С	40–80	+	+	+	+	+[1]	+	+	+	+	+[1]
	Ac-II	10ГТ	10–32	+	+	+	+	+	+	+	+	+	+
	A-III	35ГС	6–40	+	+	+	+[1]	–	+	+	+[1]	–	–
		25Г2С	6–8	+	+	+	+	+	+	+	+	+	–
			10–40	+	+	+	+	+[1]	+	+	+	+[1]	–
		32Г2Рпс	6–22	+	+	+	+[1]	–	+	+	+[1]	–	–
Ausform robbed rod reinforcement	Aт-IIIC	БСт5пс БСт5сп	10–22	+	+	+	+[1]	–	+	+	+[1]	–	–
Regular ribbed reinforcement wire	Bp-I	–	3–5	+	+	+	+	+	+	+	+	+	+

[1] Can be used only in bound framework meshes

Notes: 1. In the present table sign “+” means – allowable, sign “–” means not allowable.

2. Design temperature is taken according to instructions of Item 1.8.

3. In the present table the loads must be considered to be dynamic if quantity of these loads during calculation of the structure as regards the rigidity is more than 0.1 of static load; repeated loads are the loads which require calculation of the structure as regards robustness.

Table 13 (Annex 2)

Embedded elements characteristics	Design temperature, Celsius degrees			
	up to 30 degrees below zero		lower than 30 degrees below zero up to 40 degrees below zero	
	Steel grade according to GOST 380-71	sheet steel thickness, mm	Steel grade according to GOST 380-71	sheet steel thickness, mm
1. Calculated as regards the loads				
a) static;	ВСт3кп2	4–30	ВСт3пс6	4–25
b) dynamic and repeated	ВСт3пс6	4–10	ВСт3пс6	4–10
	ВСт3Гпс5	11–30	ВСт3Гпс5	11–30
	ВСт3сп5	11–25	ВСт3сп5	11–25
2. Constructive (not calculated as regards any forces)	БСт3кп2	4–10	БСт3кп2	4–10
	ВСт3кп2	4–30	БСт3кп2	4–30

Notes: 1. Design temperature is taken according to Item 1.8 instructions.

2. When using low-alloyed steel for example steel grade 10ГС2С1, 09ГС2С, 15 ХСНД as well as by design temperature lower than 40 Celsius degrees below zero choosing of steel grade and electrodes must be performed as for steel welded structures in compliance with requirements of SNiP II-23-81.

3. Design resistances of steel are taken according to SNiP II-23-81.

Table 14 (19, 20)

Type and class of reinforcement	Standard resistances against tension R_{sn} and design resistances against tension for the second class limit states $R_{s,ser}$, mega Pascal (kilogram-force/cm^2)	Type and class of reinforcement	Standard resistances against tension R_{sn} and design resistances against tension for the second class limit states $R_{s,ser}$, mega Pascal (kilogram-force/cm^2)
Rod reinforcement		Reinforcement wire of class Вр-I, diameter:	
A-I	235 (2400)	3 mm	410 (4200)
A-II	295 (3000)	4 mm	405 (4150)
A-III and Aт-IIIC	390 (4000)	5 mm	395 (4050)

Table 15 (22, 23)

Type and class of reinforcement	Design resistances of reinforcement for the first classes limit states, mega Pascal (kilogram-force/cm^2)		
	against tension		against compression R_{sc}
	of longitudinal reinforcement R_s	Of cross reinforcement (stirrups and bend-up bars) R_{sw}	
Rod reinforcement of classes:			
A-I	225 (2300)	175 (1800)	225 (2300)
A-II	280 (2850)	225 (2300)	280 (2850)
A-III with diameter:			
6–8 mm	355 (3600)	285 (5900)*	355 (3600)
10–40 mm	365 (3750)	290 (3000)*	365 (3750)
Aт-IIIC	365 (3750)	390 (3000)*	365 (3750)
Reinforcement wire of class Вр-II with diameter:			
3 mm	375 (3850)	270 (2750); 300 (3050)**	375 (3850)
4 mm	356 (3750)	265 (2700); 295 (3000)**	365 (3750)
5 mm	360 (3700)	260 (2650); 290 (2950)**	360 (3700)

* In welded frameworks for stirrups made of reinforcement A-III and Aт-IIIC with diameter less than 1/3 of diameter of longitudinal bars values R_{sw} are taken equal to 255 Mega Pascal (2600 kilogram-force/cm^2).

** For bound frameworks.

Standard and design characteristics of reinforcement

(2.25) For characteristic strength of reinforcement R_{sn} it is necessary to take the least controlled values:

- for rod reinforcement – physical yield limit;
- for regular reinforcement wire – stress equal to 0.75 of rapture strength.

Standard resistances R_{sn} for main types of non-prestressed reinforcement are given in Table 14.

(2.26) Design strength of reinforcement against tension and compression R_s and R_{sc} for the first class limit states are determined by means of dividing of characteristic strength into safety factor γ_s taken equal to:

a) 1.05 – for rod reinforcement A-I and A-II;
1.07 – for rod reinforcement Aт-IIIC and A-III with diameter 10–40 mm
1.10 – for rod reinforcement A-III with diameter 6–8 mm;
b) 1.10 – for reinforcement wire Bp-I.

Design extension strength of reinforcement for the second group limit states is taken equal to characteristic strength.

Design extension and compression strength of reinforcement used during calculation according to the first class limit states are given in the Table 15 and by calculations according to the second class limit states – in Table 14.

(2.28) Design strength of cross reinforcement (stirrups and bend-up bars) R_{sw} get decreased in comparison with R_s by means of multiplying by the work conditions coefficients γ_{s1} and γ_{s2}:

a) independently on type and class of reinforcement – by the coefficient $\gamma_{s1} = 0.8$ considering unevenness of forces spread in reinforcement in the length dimension of the section;
b) for rod reinforcement of class A-III and Aт-IIIC with diameter no less than 1/3 of diameter of longitudinal bars and for reinforcement wire of class Bp-I in welded frameworks – by the coefficient $\gamma_{s2} = 0.9$ considering the welded joint brittle failure possibility.

Design strengths R_{sw} with consideration of the mentioned above work conditions coefficients γ_{s1} and γ_{s2} are given in Table 15.

Besides if the considered section is locates in anchor zone of reinforcement so design strengths R_s and R_{sc} are multiplied by work conditions coefficient γ_{s5} considering incomplete anchorage of reinforcement and determined according to Item 3.44.

For elements made of light-weight concrete B7.5 and less design resistances R_{sw} of cross reinforcement A-I and Bp-I are to be multiplied by work conditions coefficient $\gamma_{s7} = 0.8$.

(2.30) Values of reinforcement elasticity modulus E_s are taken equal to:
210 000 mega Pascal (2 100 000 kilogram-force/cm^2) – for reinforcement A-I and A-II
200 000 mega Pascal (2 000 000 kilogram-force/cm^2) – for reinforcement A-III and Aт-IIIC
170 000 mega Pascal (1 700 000 kilogram-force/cm^2) – for reinforcement Bp-I

3. CALCULATION OF CONCRETE AND REINFORCED CONCRETE MEMBERS AS REGARDS THE FIRST CLASS LIMIT STATES.

3.1. For registration of loads influence on the concrete strength it is necessary to calculate concrete and reinforced concrete members as regards their strength:

a) regarding dead loads, long-term and short-term loads except loads of short duration (wind loads, crane loads and other during production, transportation, installation, etc) as well as regarding special loads caused by deformation of collapsible, swelling, permanently frozen soils and soil of that kind; in that case design tension and compression strength of concrete R_b and R_{bt} are taken according to Table 8 if $\gamma_{b2} = 0.9$:
b) regarding all loads action including loads of short duration; in that case design strength of concrete R_b and R_{bt} are taken according to Table 8 by $\gamma_{b2} = 1.1$*

* If by consideration of special loads in compliance with instructions of norms it is necessary to insert a work conditions coefficient (for example when consideration of earthquake loads) so it is taken $\gamma_{b2} = 1.0$

If the structure is used in conditions favorable for concrete strength developing [hardening under the water, in humid soil or if surrounding air humidity is more than 75 percent (see Item 1.8)] so calculation according to case "a" is made by $\gamma_{b2} = 1.0$.

Strength conditions must be fulfilled as according to case "a" as according to case "b".

In case of absence of loads of short duration or emergency calculation is made only as according to case "b" if the following condition is met:

$$F_I < 0.82 F_{II} \tag{1}$$

where F_I is the force (moment M_I, cross force Q_I or longitudinal force N_I) from the loads used by the calculation according to case "a"; at the same time in the calculations of sections normal to longitudinal axis of eccentric loaded members moment M_I is taken relating to the axis going through the most stretched (or the least pressed) reinforcement rod, and for concrete members – relating to stretched or the leased compressed surface;

F_{II} is the force from the loads used by calculation according to case "b".

It is possible to make the calculation only according to case "b" if the condition (1) is not fulfilled, taking design resistances R_b and R_{bt} (by $\gamma_{b2} = 1.0$) with the coefficient $\gamma_{bl} = 0.9 F_{II} / F_I \leq 1.1$.

For eccentric pressed members calculation according to un-deformed scheme values F_I and F_{II} can be determined without considering member deflection.

For structures used in conditions favorable for concrete strength developing, condition (1) becomes $F_I < 0.9 F_{II}$ and the coefficient $\gamma_{bl} = F_{II} / F_I$.

CALCULATION OF CONCRETE MEMBERS STRENGTH

3.2. (3.1) Calculation of strength of concrete members must be made for sections normal to their longitudinal axis. According to work conditions of members they are calculated considering as well as without considering resistance of tensile zone of concrete.

Without consideration of resistance of tensile zone of concrete the calculation of eccentric pressed members mentioned in Item 1.7a considering that limit state is characterized by failure of compressed concrete.

With consideration of resistance of tensile zone of concrete the calculation of members mentioned in Item 1.7b as well as members for which the presence of cracks is not allowed according to use conditions of the structure (members under the pressure of water, cornices, parapets, etc). At the same time it is considered that limit state is characterized by failure of tensile concrete (crack formation).

In case if appearance of diagonal cracks is possible (for example members of T- or double T-section under lateral forces) it is necessary to make the calculation of concrete members according to condition (13).

Besides it is necessary to make the calculation as regards local compression in compliance with Item 3.93.

Eccentric Pressed Members

3.3. (3.2, 1.21) During calculation of eccentric pressed concrete members it is necessary to take into account the occasional eccentricity of longitudinal force e_a caused by not considered in the calculation factors. In any eccentricity e_a is taken no less than

- 1/600 of the member length or of distance between its sections fixed against displacement;
- 1/30 of the member height;
- 10 mm (for prefabricated members if there are no any other justified values e_a)

For members of statically non-definable structures the value of eccentricity of longitudinal force relating to center of gravity of the given section e_0 is taken equal to eccentricity of static calculation of the structure but no less than e_a.

In members of statically non-definable structures eccentricity e_0 is determined as a sum of eccentricities according to static calculation of the structure and occasional one.

3.4. (3.3) By elasticity of members $l_0 / i > 14$ (for rectangular sections by $l_0 / h > 4$) it is necessary to consider the influence of deflections in the eccentricity plane of longitudinal force and in the plane normal to it on the load-carrying capacity of members by means of multiplying of values e_0 by coefficient η (see Item 3.7). In the calculation from eccentricity plane of longitudinal force value e_0 is taken equal to occasional eccentricity.

Use of eccentric pressed concrete members (except the cases provided in Item 1.7b) is not allowed by eccentricities of longitudinal force considering deflections $e_0\ \eta$ which are more than:

a) according to the loads combinations
   - 0.9y………by basic combination;
   - 0.95y….......by special load combination;

b) according to concrete class:
   - y –10……..by B10 and higher;
   - y –20……..by B7.5 an lower

(here y is the distance from the center of gravity of the section to the most compressed concrete fiber).

3.5. (3.4) In eccentric compressed concrete members it is necessary to design constructive reinforcement in cases mentioned in Item 5.122.

3.6. (3.5) Calculation of eccentric compressed concrete members must be made without considering tensile concrete according to the following condition:

$$N \le R_b A_b \qquad (2)$$

where A_b area of compressed zone of concrete determined according to the condition that its center of gravity is congruent with point of external resultant forces (Draft 1).

Draft 1. Forces scheme and stress distribution across the cross-section of compressed concrete member without considering the tensile concrete resistance

1 – center of gravity of compressed zone area; *2* – the same of the whole section area.

For members of rectangular section A_b is determined by the following formula:

$$A_b = bh\left(1 - \frac{2e_0\eta}{h}\right) \qquad (3)$$

Eccentric compressed concrete elements which can not have any cracks according to use conditions (see Item 3.2) must be checked independently on calculation according to condition (2) but in compliance with the following condition:

$$N \le \frac{R_{bt} W_{pl}}{e_0\eta - r} \qquad (4)$$

For members of rectangular section condition (4) has the following view:

$$N \le \frac{1.75 R_{bt} bh}{\frac{6e_0\eta}{h} - \varphi} \qquad (5)$$

Calculation of eccentric pressed members mentioned in Item 1.7b must be made according to the condition (2) or (4).

In formulas (3)–(5):

η is the coefficient determined by the formula (8);

r is the distance from the center of gravity of the section to the heart point most distant from the tensile zone determined by the following formula:

$$r = \varphi \frac{W}{A} \tag{6}$$

$\varphi = 1.6 - \frac{\sigma_b}{R_{b,ser}}$ But is taken no more than 1.0;

σ_b – Maximum compression stress determined as for elastic body;

W_{pl} – is sectional modulus for end tensile fiber considering non-elastic deformations of tensile concrete determined by the following formula:

$$W_{pl} = \frac{2I_{b0}}{h - x} + S_{b0} \tag{7}$$

where I_{b0} is moment of inertia of concrete pressed zone section area relating to zero line;

S_{b0} is static moment of concrete pressed zone section area relating to zero line;

$h - x$ is the distance from the zero line to the tensile surface:

$$h - x = \frac{2S_{b1}}{A + A_{b1}};$$

A_{b1} is area of compressed zone of concrete supplemented in tensile zone with the rectangle with width b equal to the width of section along the zero line and with height $h - x$ (Draft 2);

S_{b1} is static moment of area A_{b1} relating to stretched surface.

Draft 2. To definition A_{b1}.

It is possible to determine W_{pl} by the following formula:

$$W_{pl} = \gamma W_0$$

where γ – see in Table 29.

3.7. (3.6) Coefficient η considering deflection influence on the eccentricity of longitudinal force e_0 must be determined by the following formula:

$$\eta = \frac{1}{1 - \frac{N}{N_{cr}}} \tag{8}$$

where N_{cr} is relative critical force determined by the following formula:

$$N_{cr} = \frac{6.4 E_b I}{\varphi_l (l_0 / h)^2} \left(\frac{0.11}{0.1 + \delta_e} + 0.1 \right) \tag{9}$$

(here I is moment of inertia of concrete section).

For elements of rectangular section formula (9) has the following view:

$$N_{cr} = \frac{0.533 E_b A}{\varphi_l (l_0 / h)^2} \left(\frac{0.11}{0.1 + \delta_e} + 0.1 \right) \tag{9a}$$

In formulas (9) and (9a):

φ_l – Coefficient considering influence of long duration of the load on the member deflection:

$$\varphi_l = 1 + \beta \frac{M_{1l}}{M_1} \qquad (10)$$

but no more than $1+\beta$

here β is coefficient taken by Table 16;

M_1 is the moment relating to tensile or the least compressed surface of the section caused by influence of dead loads, short-term and long-term loads;

M_{1l} is the same but caused by dead loads and long-term loads;

l_0 is determined according to Table 17;

δ_e – The coefficient taken equal to e_0 / h but no less than

$$\delta_{e,\min} = 0.5 - 0.01\frac{l_0}{h} - 0.01R_b$$

(Here R_b is in Mega Pascals).

Note. During calculation of the section according to cases "a" and "b" (see Item 3.1) it is possible to determine $\delta_{e,\min}$ only once taking R_b by $\gamma_{b2} = 0.1$.

Table 16 (30)

Concrete	Coefficient β in formula (10)
1. Heavy-weight concrete	1.0
2. Fine concrete:	
group A	1.3
group Б	1.5
group B	1.0
3. Light-weight concrete	
- with artificial coarse and fine aggregate:	
dense	1.0
porous	1.5
- with natural coarse aggregate	2.5
4. Porous concrete	2.0

Note: Fine concrete groups are given in Item 2.1.

Table 17 (31)

Walls and columns support character	Design length l_0 of eccentric pressed concrete members
1. with supports above and below:	
a) with hinges on both ends independently on displacement of supports;	H
b) by one end restraint and possible displacement of supports for	
- multi-span buildings	$1.25H$
- one-span buildings	$1.50H$
2. free supported	$2.00H$

Symbols in Table 17: H – the height of the column (wall) within the first storey except the thickness of the floor slab or the height of free supported structure.

3.8. The calculation considering deflection of eccentric pressed concrete members of rectangular section made of heavy-weight concrete of class no higher than B20 can be made due to the diagram (Draft 3). At the same time the following condition must be met:

$$N \le \alpha_n R_b bh$$

Where α_n is determined according to the diagram (Draft 3) in compliance with values e_0 / h and $\lambda = l_0 / h$.

Draft 3. Diagram of load carrying capacity of eccentric compressed concrete elements.

Explanation: ——— by $M_{1l} / M_1 = 1.0$

-------- By $M_{1l} / M_1 = 0.5$

Bending Elements

3.9. (3.8) Calculation of bending concrete elements must be made according to the following condition:

$$M \le R_{bt} W_{pl} \tag{11}$$

where W_{pl} is determined by Formula (7); for members of rectangular section W_{pl} is taken equal to:

$$W_{pl} = \frac{bh^2}{3.5} \tag{12}$$

Besides for members of T- and double T-section the following condition must be met:

$$\tau_{xy} \le R_{bt} \tag{13}$$

Where τ_{xy} – shear stresses determined as for elastic material at the level of center of gravity of the section.

Examples of Calculation

Example 1. *Given:* a concrete panel of the wall between apartments, thickness h = 200 mm, height H = 2.7 mm manufactured vertically (in the mounting) of expanded-clay concrete with glass sand of class B15, concrete grade as regards average density is D1600 (E_b = 14 000 Mega Pascal) total load per 1 m of the wall is N = 900 kN, including dead load and long-term loads N_l = 540 kN; no load of short duration.

It is required to test the strength of the wall panel.

Calculation is made according to Item 3.6 as regards the longitudinal force N = 900 kN applied with occasional eccentricity e_a determined according to Item 3.3.

As $\frac{h}{30} = \frac{200}{30} = 6.67$ mm < 10 mm occasional eccentricity is taken equal to 10 mm, which means $e_0 = 10$ mm. The connection of the panel above and below is considered to be hinge connection, so design length l_0 in compliance with Table 17 is $l_0 = H = 2.7$ m.

As panel elasticity $\frac{l_0}{h} = \frac{2.7}{0.2} = 13.5 > 4$ so the calculation is made with consideration of deflection in compliance with Item 3.7.

Coefficient φ_l is determined according to formula (10) by $\beta = 1.0$ (see Table 16). As eccentricity of longitudinal force doesn't depend on load characteristics so here it is possible to take $\frac{M_{1l}}{M} = \frac{N_l}{N} = \frac{540}{900} = 0.6$,

So $\varphi_l = 1 + \beta \frac{M_{1l}}{M_1} = 1 + 0.6 = 1.6$

As there are no loads of short duration so design concrete strength R_b in compliance with Item 3.1 is taken considering the coefficient $\gamma_{b2} = 0.90$ that is $R_b = 7.7$ mega

Pascal and in compliance with Table 9 considering work conditions coefficients $\gamma_{b3} = 0.85$ and $\gamma_{b9} = 0.90$ we get $R_b = 7.7 \times 0.85 \times 0.90 = 5.89$ mega Pascal.

As $\delta_{e,\min} = 0.5 - 0.01\frac{l_0}{h} - 0.01R_b = 0.5 - 0.01 \cdot 13.5 - 0.01 \cdot 5.89 = 0.306 > \frac{e_0}{h} = \frac{10}{200}$ so we take $\delta_e = \delta_{e,\min} = 0.306$.

Critical force N_{cr} is determined by formula (9a) taking section area A for 1 m of the wall length, that is A = 200×1000 = 200 000 mm^2:

$$N_{cr} = \frac{0.533E_bA}{\varphi_l(l_0/h)^2}\left(\frac{0.11}{0.1+\delta_e}+0.1\right) = \frac{0.533 \cdot 14 \cdot 10^3 \cdot 200000}{1.6 \cdot 13.5^2}\left(\frac{0.11}{0.1+0.306}+0.1\right) = 1898 \cdot 10^3\,\text{N} = 1898\text{kN}$$

from this $\eta = \frac{1}{1-\frac{N}{N_{cr}}} = \frac{1}{1-\frac{900}{1898}} = 1.902$

If we check condition (2) using formula (3):

$$R_bA_b = R_bbh\left(1-\frac{2e_0\eta}{h}\right) = 5.89 \cdot 200000\left(1-\frac{2 \cdot 10 \cdot 1.902}{200}\right) = 954000\,\text{N} = 954\text{ kN} > N = 900\text{ kN},$$

that is the strength of the panel is provided.

CALCULATION OF REINFORCED CONCRETE MEMBERS STRENGTH

3.10.(3.9) Calculation of reinforced concrete members as regards their strength must be made for the sections normal to their longitudinal axis as well as for inclined sections of the most dangerous direction. By torque moments it is necessary to check the strength of spatial sections in stretched zone bounded by torsion fracture of the most dangerous of all possible directions. Besides it is necessary to make the calculation of members as regards local loads (bearing stress, punching force, cleavage).

Bending Elements

3.11.(3.11) Calculation of sections normal to longitudinal axis of the member when bending moment acts in the plane of section symmetry axis and reinforcement is concentrated at surfaces perpendicular to the mentioned plane must be made in compliance with Items 3.15-3.23 according to the ratio between the value of relative height of concrete compressed zone $\xi = x/h_0$ determined according to requirements for equilibrium and the value of relative height of compressed concrete zone ξ_R (see Item 3.14) whereby limit state of limit state of the member comes at the same time with the stress equal to design strength R_s in the stretched reinforcement.

3.12. **(3.18)** Calculation of ring cross section bending elements if the ration of internal and external radii is $r_1/r_2 \geq 0.5$ with reinforcement evenly spread in a circumferential direction (if there are no less than 6 longitudinal bars) must be made as for eccentric compressed members in compliance with Items 3.69 and 3.70 by N = 0 and by bending moment value instead of Ne_0.

3.13. Calculation of normal sections not mentioned in Items 3.11, 3.12 and 3.24 is made by formulas of general case of normal section calculation in compliance with Item 3.76 taking N = 0 in formula (154) and replacing $N\overline{e}$ by $\overline{M}$ (projection of bending moment on the plane perpendicular to the straight line which bounds compression zone) in condition (153). If symmetry axis of the section is not congruent with the moment plane or is absent at all so location of the compressed zone bounding must conform to the additional condition of parallelism of moments planes of internal and external forces.

3.14. (3.12) Value ξ_R is determined by the following formula:

$$\xi_R = \frac{\omega}{1 + \frac{R_s}{\sigma_{sc,u}}\left(1 - \frac{\omega}{1.1}\right)} \qquad (14)$$

where ω is characteristic of concrete compressed zone determined by the following formula:

$$\omega = \alpha - 0.008 R_b \qquad (15)$$

here α is the coefficient equal to:

0.85……for heavy-weight concrete
0.80……for fine concrete (see Item 2.1) of group A
0.75……for fine concrete of groups Б and В
0.80……for light-weight and porous concrete

$\sigma_{sc,u} = 500$ Mega Pascal by coefficient $\gamma_{b2} = 0.9$ (see Item 3.1);

$\sigma_{sc,u} = 400$ Mega Pascal by coefficient $\gamma_{b2} = 1.0$ or $\gamma_{b2} = 1.1$;

R_s, R_b are in mega Pascals.

Values ω and ξ_R are given in table 18 – for members of heavy-weight concrete; in Table 19 – for members of fine concrete of group A, light-weight and fine concrete

Table 18

Concrete work conditions coefficient γ_{b2}	Class of tensile reinforcement	Symbol	Values ω, ξ_R, α_R and ψ_c for members of heavy-weight concrete of classes										
			B12.5	B15	B20	B25	B30	B35	B40	B45	B50	B55	B60
0.9	Any	ω	0.796	0.788	0.766	0.746	0.726	0.710	0.690	0.670	0.650	0.634	0.614
	A-III (Ø10–40) and BP-I (Ø4; 5)	ξ_R	0.662	0.652	0.627	0.604	0.582	0.564	0.542	0.521	0.500	0.484	0.464
		α_R	0.443	0.440	0.430	0.422	0.413	0.405	0.395	0.381	0.376	0.367	0.355
		ψ_c	4.96	4.82	4.51	4.26	4.03	3.86	3.68	3.50	3.36	3.23	3.09
	A-II	ξ_R	0.689	0.680	0.650	0.632	0.610	0.592	0.571	0.550	0.531	0.512	0.490
		α_R	0.452	0.449	0.439	0.432	0.424	0.417	0.408	0.399	0.390	0.381	0.370
		ψ_c	6.46	6.29	5.88	5.55	5.25	5.04	4.79	4.57	4.38	4.22	4.03
	A-I	ξ_R	0.708	0.698	0.674	0.652	0.630	0.612	0.591	0.570	0.551	0.533	0.510
		α_R	0.457	0.455	0.447	0.439	0.432	0.425	0.416	0.407	0.399	0.391	0.380
		ψ_c	8.04	7.82	7.32	6.91	6.54	6.27	5.96	5.68	5.46	5.25	5.01
1.0	Any	ω	0.790	0.782	0.758	0.734	0.714	0.694	0.674	0.650	0.630	0.610	0.586
	A-III (Ø10–40) and BP-I (Ø4; 5)	ξ_R	0.628	0.619	0.591	0.563	0.541	0.519	0.498	0.473	0.453	0.434	0.411
		α_R	0.431	0.427	0.416	0.405	0.395	0.384	0.374	0.361	0.350	0.340	0.327
		ψ_c	3.89	3.79	3.52	3.29	3.12	2.97	2.83	2.68	2.56	2.46	2.35
	A-II	ξ_R	0.660	0.650	0.623	0.593	0.573	0.551	0.530	0.505	0.485	0.465	0.442
		α_R	0.442	0.439	0.429	0.417	0.409	0.399	0.390	0.378	0.367	0.357	0.344
		ψ_c	5.07	4.94	4.6	4.29	4.07	3.87	3.69	3.49	3.34	3.21	3.06
	A-I	ξ_R	0.681	0.673	0.645	0.618	0.596	0.575	0.553	0.528	0.508	0.488	0.464

		α_R ψ_c	0.449 6.31	0.447 6.15	0.437 5.72	0.427 5.34	0.419 5.07	0.410 4.82	0.400 4.59	0.389 4.35	0.379 4.16	0.369 3.99	0.356 3.80
1.1	Any	ω	0.784	0.775	0.750	0.722	0.698	0.678	0.653	0.630	0.606	0.586	0.558
	A-III (Ø10–40) and BP-I (Ø4; 5)	ξ_R α_R ψ_c	0.621 0.428 3.81	0.610 0.424 3.71	0.581 0.412 3.44	0.550 0.399 3.19	0.523 0.386 3.00	0.502 0.376 2.86	0.481 0.365 2.73	0.459 0.351 2.65	0.429 0.346 5.52	0.411 0.327 2.35	0.385 0.312 2.23
	A-II	ξ_R α_R ψ_c	0.650 0.439 4.97	0.642 0.436 4.84	0.613 0.425 4.49	0.582 0.413 4.16	0.556 0.401 3.91	0.534 0.391 3.72	0.514 0.382 3.53	0.485 0.361 3.34	0.477 0.363 3.29	0.442 0.344 3.06	0.417 0.330 2.91
	A-I	ξ_R α_R ψ_c	0.657 0.447 6.19	0.665 0.444 6.02	0.636 0.434 5.59	0.605 0.422 5.17	0.579 0.411 4.86	0.558 0.402 4.63	0.537 0.393 4.42	0.509 0.379 4.16	0.500 0.375 4.09	0.464 0.356 3.80	0.439 0.343 3.62

$$\omega = 0.85 - 0.008R_b;\ \xi_R \frac{\omega}{1+\frac{R_s}{\sigma_{sc,u}}\left(1-\frac{\omega}{1.1}\right)};\ \alpha_R = \xi_R(1-0.5\xi_R);\ \psi_c = \frac{\sigma_{sc,u}}{R_s\left(1-\frac{\omega}{1.1}\right)}$$

Note: Values ω, ξ_R α_R and ψc given in Table 18 are calculated without considering coefficients γ_{bi} according to Table 9.

Table 19

Concrete work conditions coefficient γ_{b2}	Class of tensile reinforcement	Symbol	Values ω, ξ_R, α_R and ψ_c for members of fine concrete of group A, light-weight and porous concrete of classes									
			B5	B7.5	B10	B12.5	B15	B20	B25	B30	B35	B40
0.9	Any	ω	0.780	0.768	0.757	0.746	0.738	0.716	0.696	0.676	0.660	0.640
	A-III (Ø10–40) and BP-I (Ø4; 5)	ξ_R α_R ψ_c	0.643 0.436 4.71	0.629 0.431 4.54	0.617 0.427 4.39	0.604 0.422 4.26	0.595 0.418 4.16	0.571 0.408 3.92	0.551 0.399 3.75	0.528 0.388 3.55	0.510 0.380 3.42	0.490 0.370 3.28
	A-II	ξ_R α_R ψ_c	0.671 0.446 6.14	0.657 0.441 5.92	0.644 0.437 5.73	0.632 0.432 5.55	0.623 0.429 5.43	0.599 0.420 5.12	0.577 0.411 4.86	0.556 0.401 4.63	0.539 0.394 4.46	0.519 0.384 4.27
	A-I	ξ_R α_R ψ_c	0.690 0.452 7.64	0.676 0.488 7.36	0.664 0.444 7.13	0.652 0.439 6.91	0.643 0.436 6.75	0.619 0.427 6.37	0.597 0.419 6.05	0.576 0.410 5.76	0.559 0.403 5.56	0.539 0.394 5.31
1.1	Any	ω	0.774	0.761	0.747	0.734	0.725	0.700	0.672	0.648	0.628	0.608
	A-III (Ø10–40) and BP-I (Ø4; 5)	ξ_R α_R ψ_c	0.609 0.424 3.70	0.594 0.418 3.56	0.578 0.411 3.42	0.563 0.405 3.29	0.553 0.400 3.22	0.526 0.388 3.01	0.496 0.373 2.82	0.471 0.360 2.67	0.451 0.349 2.55	0.432 0.339 2.45
	A-II	ξ_R α_R ψ_c	0.641 0.436 4.82	0.626 0.430 4.64	0.610 0.424 4.45	0.595 0.418 4.29	0.585 0.414 4.19	0.558 0.402 3.67	0.528 0.389 3.48	0.503 0.377 3.30	0.482 0.366 3.33	0.463 0.356 3.19
	A-I	ξ_R α_R ψ_c	0.663 0.443 6.00	0.648 0.438 5.71	0.633 0.433 5.54	0.618 0.427 5.34	0.608 0.423 5.21	0.581 0.412 4.89	0.551 0.399 4.57	0.526 0.388 4.33	0.506 0.378 4.14	0.486 0.368 3.97

$$\omega = 0.80 - 0.008R_b;\ \xi_R \frac{\omega}{1+\frac{R_s}{\sigma_{sc,u}}\left(1-\frac{\omega}{1.1}\right)};\ \alpha_R = \xi_R(1-0.5\xi_R);\ \psi_c = \frac{\sigma_{sc,u}}{R_s\left(1-\frac{\omega}{1.1}\right)}$$

Note: Values ω, ξ_R, α_R and ψ_c given un Table 19are calculated without considering coefficients according to Table 9.

RECTANGULAR SECTIONS

3.15. Calculation of rectangular sections with reinforcement concentrated at compressed and tensile surface of the member (Draft 4), is made in the following manner according to the height of compressed zone:

$$x = \frac{R_s A_s - R_{sc} A_s^{'}}{R_b b} \tag{16}$$

a) by $\xi = \frac{x}{h_0} \le \xi_R$ – for the condition

$$M \le R_b bx(h_0 - 0.5x) + R_{sc} A_s^{'}(h_0 - a') \tag{17}$$

b) by $\xi > \xi_R$ – for the condition

$$M \le \alpha_R R_b bh_0^2 + R_{sc} A_s^{'}(h_0 + a') \tag{18}$$

Where $\alpha_R = \xi_R(1 - 0.5\xi_R)$

At the same time design load-carrying capacity of the section can be increased by means of replacing of value α_R by $0.8\alpha_R + 0.2\alpha_m$ in the condition (18) where by $\xi \le 1$ $\alpha_m = \xi(1 - 0.5\xi)$ or according to table 20. Values ξ_R and α_R are determined according to table 18 and 19. If $x \le 0$ so the strength is checked according to the following condition

$$M \le R_s A_s(h_0 - a') \tag{19}$$

Note. If the height of compressed zone determined considering of a half of compressed reinforcement, $x = \frac{R_s A_s - 0.5 R_{sc} A_s^{'}}{R_b b} \le a'$ so design load carrying capacity of the section can be increased if the calculation will be made by formulas (16) and (17) without considering compressed reinforcement $A_s^{'}$.

Draft 4. Loads scheme in rectangular cross section of bending reinforced concrete element.

3.16. It is recommended to design bending elements so that to provide the fulfillment of the condition $\xi < \xi_R$. It is possible not to meet this condition only in case when the section area of stretched reinforcement is determined according to the calculation as regards the second class limit states or if it’s taken on the grounds of constructive solutions.

3.17. Checking of rectangular sections strength with single reinforcement is made

- by $x < \xi_R h_0$ in compliance with the condition:

$$M \le R_s A_s(h_0 - 0.5x) \tag{20}$$

Where height of compressed zone is $x = \frac{R_s A_s}{R_b b}$

- by $x \ge \xi_R h_0$ in compliance with the condition:

$$M \le \alpha_R R_b bh_0^2 \tag{21}$$

at the same time design load carrying capacity of the section can be increased using recommendations of Item 3.15b [ξ_R, α_R - see formula (4) or Table 18 and 19].

3.18. Choosing of longitudinal reinforcement is made in the following manner. It is necessary to calculate the following value:

$$\alpha_m = \frac{M}{R_b bh_0^2} \tag{22}$$

If $\alpha_m \le \alpha_R$ (see Table 18 and 19) so that means that compressed reinforcement is not required.

If there is no compressed reinforcement so section area of tensile reinforcement is determined by the following formula:

$$A_s = \frac{M}{R_b \zeta h_0} \qquad (23)$$

Where ζ is determined according to Table 20 according to value α_m.

If $\alpha_m > \alpha_R$ so it is necessary to enlarge the section or to increase the concrete grade, or to fix compressed reinforcement in compliance with Item 3.19.

By consideration of the concrete work conditions coefficient $\gamma_{b2} = 0.9$ (see Item 3.1) tensile reinforcement can be chosen according to Annex 2.

Table 20

ξ	ζ	α_m	ξ	ζ	α_m	ξ	ζ	α_m
0.01	0.995	0.010	0.26	0.870	0.226	0.51	0.745	0.380
0.02	0.990	0.020	0.27	0.865	0.234	0.52	0.740	0.385
0.03	0.985	0.030	0.28	0.860	0.241	0.53	0.735	0.390
0.04	0.980	0.039	0.29	0.855	0.243	0.54	0.730	0.394
0.05	0.975	0.049	0.30	0.850	0.255	0.55	0.725	0.399
0.06	0.970	0.058	0.31	0.845	0.262	0.56	0.720	0.403
0.07	0.965	0.068	0.32	0.840	0.269	0.57	0.715	0.407
0.08	0.960	0.077	0.33	0.835	0.276	0.58	0.710	0.412
0.09	0.955	0.086	0.34	0.830	0.282	0.59	0.705	0.416
0.10	0.950	0.095	0.35	0.825	0.289	0.60	0.700	0.420
0.11	0.945	0.104	0.36	0.820	0.295	0.62	0.690	0.428
0.12	0.940	0.113	0.37	0.815	0.302	0.64	0.680	0.435
0.13	0.935	0.122	0.38	0.810	0.308	0.66	0.670	0.442
0.14	0.930	0.130	0.39	0.805	0.314	0.68	0.660	0.449
0.15	0.925	0.139	0.40	0.800	0.320	0.70	0.650	0.455
0.16	0.920	0.147	0.41	0.795	0.326	0.72	0.640	0.461
0.17	0.915	0.156	0.42	0.790	0.332	0.74	0.630	0.466
0.18	0.910	0.164	0.43	0.785	0.338	0.76	0.620	0.471
0.19	0.905	0.172	0.44	0.780	0.343	0.78	0.610	0.476
0.20	0.900	0.180	0.45	0.775	0.349	0.80	0.600	0.480
0.21	0.895	0.188	0.46	0.770	0.354	0.85	0.575	0.489
0.22	0.890	0.196	0.47	0.765	0.360	0.90	0.550	0.495
0.23	0.885	0.204	0.48	0.760	0.365	0.95	0.525	0.499
0.24	0.880	0.211	0.49	0.755	0.370	1.00	0.500	0.500
0.25	0.875	0.219	0.50	0.750	0.375	–	–	–

For bending moments of rectangular section:

$$\xi = \frac{R_s A_s - R_{sc} A_s^{'}}{R_b b h_0}; \quad \alpha_m \frac{M - R_{sc} A_s^{'} (h_0 - a')}{R_b b h_0^2}; \quad \alpha_m = \xi (1 - 0.5\xi); \; \zeta = 1 - 0.5\xi .$$

3.19. Cross sections areas of tensile A_s and compressed $A_s^{'}$ reinforcement corresponding to minimum of their sum for members of concrete of class B30 and lower should be determined if compressed reinforcement is required according to the calculation (see Item 3.18), by the following formulas:

$$A_s^{'} = \frac{M - 0.4 R_b b h_0^2}{R_{sc} (h_0 - a')} \qquad (24)$$

$$A_s = \frac{0.55 R_b b h_0}{R_s} + A_s^{'} \qquad (25)$$

If taken section area of compressed reinforcement $A_s^{'}$ far exceeds the value calculated by formula (24) so section area of tensile reinforcement is determined according to actual value of area $A_s^{'}$ by the following formula:

$$A_s = \xi b h_0 \frac{R_b}{R_s} + A_s^{'} \qquad (26)$$

Where ξ is determined according to Table 20 depending on the value $\alpha_m = \frac{M - R_{sc} A_s^{'} (h_0 - a')}{R_b b h_0^2} \geq 0$ which must conform to requirement $\alpha_m \leq \alpha_R$ (see table 18 and 19).

T- AND DOUBLE T-SECTIONS

3.20. Calculation of sections which have a flange in compressed zone (T-sections and double T-sections, etc) must be made depending on the compressed zone bounding position:

a) if the bounding of compressed zone goes in the flange (Draft 5a) that is the following condition is met:

$$R_s A_s \leq R_b b_f^{'} h_f^{'} + R_{sc} A_s^{'} \qquad (27)$$

The calculation is made as for rectangular section which is $b_f^{'}$ wide in compliance with Items 3.15 and 3.17;

b) if the bounding of compressed zone goes in the rib (Draft 5b) that is condition (27) is not met, so the calculation is made according to the following condition

$$M \leq R_b b x (h_0 - 0.5x) + R_b (b_f^{'} - b) h_f^{'} (h_0 - 0.5h_f^{'}) + R_{sc} A_s^{'} (h_0 - a') \qquad (28)$$

At the same time concrete compressed zone height x is determined by the following formula:

$$x = \frac{R_s A_s - R_{sc} A_s^{'} - R_b (b_f^{'} - b) h_f^{'}}{R_b b} \qquad (29)$$

And it is taken no more than $\xi_R h_0$ (see Table 18 and 19).

If $x \geq \xi_R h_0$ so condition (28) can be written in the following form:

$$M \leq \alpha_R R_b b h_0^2 + R_b (b_f^{'} - b) h_f^{'} (h_0 - 0.5h_f^{'}) + R_{sc} A_s^{'} (h_0 - a') \qquad (30)$$

Where α_R – see in Table 18 and 19.

At the same time it is necessary to consider the recommendations of Item 3.16.

Notes: 1. by variable height of a flange overhang it is possible to take the value $h_f^{'}$ equal to average height of overhangs.

2. Compressed flange width $b_f^{'}$ inserted into the calculation must not exceed the values given in Item 3.23.

Draft 5. Compressed zone bounding position in T-section of bending reinforced concrete element.
a – in a flange; b – in a rib

3.21. Required section area of compressed reinforcement is determined by the following formula:

$$A_s^{'} = \frac{M - \alpha_R R_b b h_0^2 - R_b (b_f^{'} - b) h_f^{'} (h_0 - 0.5h_f^{'})}{R_{sc} (h_0 - a')} \qquad (31)$$

where α_R – see in Table 18 and 19.

3.22. Required section area of tensile reinforcement is determined in the following manner:

a) if compressed zone border goes in a flange, that is the following condition is met:

$$M \le R_b b_f^{'} h_f^{'} \left(h_0 - 0.5h_f^{'}\right) + R_{sc} A_s^{'} \left(h_0 - a'\right) \tag{32}$$

so section area of tensile reinforcement is determined as for rectangular cross section $b_f^{'}$ wide in compliance with Items 3.18 and 3.19;

b) if compressed zone border goes in a rib that is condition (32) is not met so cross section of tensile section is determined by the following formula:

$$A_s \frac{R_b \left[\xi b h_0 + \left(b_f^{'} - b\right)h_f^{'}\right] + R_{sc} A_s^{'}}{R_s} \tag{33}$$

where ξ is determined according to Table 20 depending on the value

$$\alpha_m = \frac{M - R_b \left(b_f^{'} - b\right)h_f^{'} \left(h_0 - 0.5h_f^{'}\right) - R_{sc} A_s^{'} \left(h_0 - a'\right)}{R_b b h_0^2} \tag{34}$$

At the same time the condition $\alpha_m \le \alpha_R$ must be met (see Table 18 and 19).

3.23. **(3.16)** Value $b_f^{'}$ inserted into the calculation is taken according to the condition that the width of an overhang to each side of the rib must be no less than 1/6 of the span of the member and no more than:

a) by cross ribs or by $h_f^{'} \ge 0.1h - 1/2$ of the clear distance between longitudinal ribs;

b) without cross ribs (or if the distance between them is more than the distance between longitudinal ribs) and $h_f^{'} < 0.1h - 6h_f^{'}$;

c) by console overhangs of a flange:
By $h_f^{'} \ge 0.1h - 6h_f^{'}$;
By $0.05h \le h_f^{'} < 0.1h - 3h_f^{'}$;
By $h_f^{'} < 0.05h$ - overhangs are not taken into account.

Examples of Calculation

Rectangular section

Example 2. *Given:* the section with dimensions b = 300 mm, h = 600 mm, a = 40 mm; $\gamma_{b2} = 0.9$ (no loads of short duration); bending moment M = 200 KN·m; heavy-weight concrete B15 ($R_b = 7.7$ Mega Pascal); reinforcement A-II (R_s = 280 Mega Pascal).

It is required to determine the cross section area of longitudinal reinforcement.

Calculation. $h_0 = 600 - 40 = 560$ mm. Longitudinal reinforcement is chosen according to Item 3.18. Value α_m is determined by Formula (22):

$$\alpha_m = \frac{M}{R_b b h_0^2} = \frac{200 \cdot 10^6}{7.7 \cdot 300 \cdot 560^2} = 0.276$$

According to Table 18 for a member of concrete B15 with reinforcement A-II by $\gamma_{b2} = 0.9$, we find that $\alpha_R = 0.449$.

As $\alpha_m = 0.276 < \alpha_R = 0.449$ so that means that compressed reinforcement is not required.

According to table 20 by $\alpha_m = 0.276$ we find $\zeta = 0.835$

Required cross section of tensile reinforcement is to be determined by formula (23):

$$A_s = \frac{M}{R_s \zeta h_0} = \frac{200 \cdot 10^6}{280 \cdot 0.835 \cdot 560} = 1528 \text{ mm}^2.$$

It is taken 2Ø28 + 1Ø25 ($A_s = 1598$ mm^2)

Example 3. *Given:* a section with dimensions b = 300 mm; h = 800 mm; a = 70 mm; tensile reinforcement A-III (R_s = 365 Mega Pascal); its section area $A_s = 2945$ mm^2 (6Ø25); $\gamma_{b2} = 0.9$ (no loads of short duration); heavy-weight concrete B25 (R_b = 13 Mega Pascal); bending moment M = 550 kN·m.

It is required to check the section strength.

Calculation. $h_0 = 800 - 70 = 730$ mm. Section strength is calculated according to Item 3.17.

Value x is determined in the following manner:

$$x = \frac{R_s A_s}{R_b b} = \frac{365 \cdot 2945}{13 \cdot 300} = 276 \text{ mm}$$

According to table 18 for concrete B25 with reinforcement A-III by $\gamma_{b2} = 0.9$, we find $\xi_R = 0.604$.

As $\xi = \frac{x}{h_0} = \frac{276}{730} = 0.38 < \xi_R = 0.604$

so the strength is to be checked according to condition (20):

$R_s A_s (h_0 - 0.5x) = 365 \cdot 2945 (730 - 0.5 \cdot 276) = 636.4 \cdot 10^6$ N· mm = 636.4 kN·m > M = 550 kN ·m,

that means the strength is corresponding to norms.

Example 4. *Given:* a section with dimensions b = 300 mm; h = 800 mm; a = 50 mm; reinforcement A-III ($R_s = R_{sc} = 365$ Mega Pascal); bending moment with consideration of crane load $M_{II} = 780$ kN·m; moment without consideration crane load $M_I = 670$ kN·m; heavy-weight concrete B15 ($R_b = 8.5$ Mega Pascal by $\gamma_{b2} = 1.0$).

It is required to determine the section area of longitudinal reinforcement.

Calculation is made as regards the total load correcting design resistance of concrete according to Item 3.1.

As $\gamma_{bl} = 0.9 \frac{M_{II}}{M_I} = 0.9 \frac{780}{670} = 1.05 < 1.1$ so we take $R_b = 8.5 \cdot 1.05 = 8.93$ Mega Pascal.

We calculate $h_0 800 - 50 = 750$ mm.

We determined required area of longitudinal reinforcement according to Item 3.18. Value α_m is determined according to Formula (22):

$$\alpha_m = \frac{M}{R_b b h_0^2} = \frac{780 \cdot 10^6}{8.93 \cdot 300 \cdot 750^2} = 0.518$$

As $\alpha_m = 0.518 > \alpha_R = 0.42$ (see Table 18 by $\gamma_{b2} = 1.0$) by given dimensions of the section and concrete class it is required compressed reinforcement. The following calculation is made according to Item 3.19.

Taking $a' = 30$ mm we determine required section of compressed and tensile reinforcement by formulas (24) and (25):

$$A_s' = \frac{M - 0.4 R_b b h_0^2}{R_{sc}(h_0 - a')} = \frac{780 \cdot 10^{16} - 0.4 \cdot 8.93 \cdot 300 \cdot 750^2}{365(750 - 30)} = 674 \text{ mm}^2$$

$$A_s = \frac{0.55 b h_0 R_b}{R_s} + A_s' = \frac{0.55 \cdot 300 \cdot 750 \cdot 8.93}{365} + 674 = 3702 \text{ mm}^2.$$

We take $A_s' = 763$ mm^2 (3Ø18); A_s = 4021 mm^2 (5Ø32).

Example 5. *Given:* a section with dimensions b = 300 mm; h = 700 mm; a = 50 mm; a' = 30 mm; heavy-weight concrete B30 (R_b = 15.5 MPa by $\gamma_{b2} = 0.9$); reinforcement A-III ($R_s = 365$ MPa); section area of compressed reinforcement $A_s^{'} = 942\,\text{mm}^2$ (3Ø20); bending moment M = 580 kN·m.
It is required to determined section area of tensile reinforcement.
Calculation: $h_0 = 700 - 50 = 650$ mm. The calculation is made considering the area of compressed reinforcement according to Item 3.19.

Value α_m is determined in the following manner:

$$\alpha_m = \frac{M - R_{sc}A_s^{'}\left(h_0 - a'\right)}{R_b b h_0^2} = \frac{580 \cdot 10^6 - 365 \cdot 942(650 - 30)}{15.5 \cdot 300 \cdot 650^2} = 0.187\,;$$

$\alpha_m = 187 < \alpha_R = 0.413$ (See Table 18)
According to Table 20 by $\alpha_m = 187$ we find $\xi = 0.21$. Required area of tensile reinforcement is determined by Formula (26):

$$A_s = \frac{\xi b h_0 R_b}{R_s} + A_s^{'} = \frac{0.21 \cdot 300 \cdot 650 \cdot 15.5}{365} + 942 = 2680\ \text{mm}^2$$

We take 3Ø36 (R_s = 3054 mm^2).

Example 6. *Given:* a section with dimensions b = 300 mm; h = 700 mm; a = 70 mm; a' = 30 mm; heavy-weight concrete B25 (R_b = 13 MPa by $\gamma_{b2} = 0.9$); reinforcement A-III ($R_s = R_{sc} = 365$ MPa); section area of stretched reinforcement $A_s = 4862\ \text{mm}^2$ (6Ø32), of tensile reinforcement $A_s^{'} = 339\ \text{mm}^2$ (3Ø12); bending moment M = 600 kN·m.
It is required to check the section strength.
Calculation: $h_0 = 700 - 70 = 630$ mm. The section strength is checked in compliance with Item 3.15.
The height of compressed zone x is determined by Formula (16):

$$x = \frac{R_s A_s - R_{sc} A_s^{'}}{R_b b} = \frac{365\left(4826 - 339\right)}{13 \cdot 300} = 420\ \text{mm}$$

We find $\xi_R = 0.604$ and $\alpha_R = 0.422$ according to table 18.
As $x = 420\,\text{mm} > \xi_R h_0 = 0.604 \cdot 630 = 380\,\text{mm}$ so section strength is to be checked according to condition (18):
$\alpha_R R_b b h_0^2 + R_{sc} A_s^{'}\left(h_0 - a'\right) = 0.422 \cdot 13 \cdot 300 \cdot 630^2 + 365 \cdot 339\left(630 - 30\right) = 727.4 \cdot 10^6$ N·mm = 727.4 kN·m > M = 600 kN·m,
that is section strength is provided.

T-SECTIONS AND DOUBLE T-SECTIONS

Example 7. *Given:* a section with dimensions $b_f^{'} = 1500$ mm, $h_f^{'} = 50$ mm, *and* b = 200 mm, h = 400 mm, a = 40 mm; heavy-weight concrete B25 (R_b = 13 MPa by $\gamma_{b2} = 0.9$); reinforcement A-III (R_s = 365 MPa); bending moment M = 300 kN·m.
It is required to determine the section area of longitudinal reinforcement.
Calculation: $h_0 = 400 - 40 = 360$ mm. The calculation is made according to Item 3.22 on the hypothesis that compressed reinforcement is not required according to the calculation.

We check the condition (32) taking $A_s^{'} = 0$; $R_b b_f^{'} h_f^{'} (h_0 - 0.5h_f^{'}) = 13 \cdot 1500 \cdot 50(360 - 0.5 \cdot 50) = 326.6 \cdot 10^6$ N·mm = 326.6 kN·m, that is the border of compressed zone goes in the flange and the calculation is made as for rectangular section with the width $b = b'_f$ = 1500 mm in compliance with Item 3.18.
We determine the value α_m:

$$\alpha_m = \frac{M}{R_b b h_0^2} = \frac{300 \cdot 10^6}{13 \cdot 1500 \cdot 360^2} = 0.119 < \alpha_R = 0.422 \text{ (See Table 18)},$$

that is compressed reinforcement is not required.
The section area of stretched reinforcement is calculated by formula (23). For that according to Table 20 by $\alpha_m = 0.119$ we find $\zeta = 0.938$ and

$$A_s = \frac{M}{R_s \zeta h_0} = \frac{300 \cdot 10^6}{365 \cdot 0.938 \cdot 360} = 2434 \text{ mm}^2$$

We take 4Ø28 (A_s = 2463 mm^2).

Example 8. *Given:* a section with dimensions $b_f^{'} = 400$ mm, $h_f^{'} = 120$ mm, *and* b = 200 mm, h = 600 mm, a = 60 mm; heavy-weight concrete B15 (R_b = 7.7 MPa by $\gamma_{b2} = 0.9$); reinforcement A-III (R_s = 365 MPa); bending moment M = 270 kN·m.
It is required to determined section area of tensile reinforcement.
Calculation: $h_0 = 600 - 60 = 540$ mm. The calculation is made in compliance with Item 3.22 on the hypothesis that compressed reinforcement is not required.
As $R_b b_f^{'} h_f^{'} (h_0 - 0.5h_f^{'}) = 7.7 \cdot 400 \cdot 120(540 - 0.5 \cdot 120) = 177.4 \cdot 10^6$ N·mm = 177.4 kN·m < M = = 270 kN·m, that is the border of compressed zone goes in the rib, section area of stretched reinforcement is calculated by formula (33).
For that we determine the value α_m:

$$\alpha_m = \frac{M - R_b (b_f^{'} - b) h_f^{'} (h_0 - 0.5h_f^{'})}{R_b b h_0^2} = \frac{270 \cdot 10^6 - 7.7(400 - 200) \cdot 120(540 - 0.5 \cdot 120)}{7.7 \cdot 200 \cdot 540^2} = 0.404 < \alpha_R = 0.44$$

(see Table 18), so compressed reinforcement is not required.
According to Table 20 by $\alpha_m = 0.404$ we find $\xi = 0.563$, then

$$A_s = \left[\xi b h_0 + (b_f^{'} - b) h_f^{'}\right] \frac{R_b}{R_s} = \left[0.563 \cdot 200 \cdot 540 + (400 - 200)120\right] \frac{7.7}{365} = 1789 \text{ mm}^2.$$

We take 4Ø25 (A_s = 1964 mm^2).

Example 9. *Given:* a section with dimensions $b_f^{'} = 400$ mm, $h_f^{'} = 100$ mm, b = 200 mm, h = 600 mm, a = 70 mm; heavy-weight concrete B25 (R_b = 13 MPa by $\gamma_{b2} = 0.9$); tensile reinforcement A-III (R_s = 365 MPa), its section area A_s = 1964 mm^2 (4Ø25); $A_s^{'} = 0$; bending moment M = 300 kN·m.
It is required to check the strength of the section.
Calculation: $h_0 = 600 - 70 = 530$ mm. The section strength is checked in compliance with Item 3.20, taking $A_s^{'} = 0$. As $R_s A_s = 365 \cdot 1964 = 716860$ N > $R_b b_f^{'} h_f^{'} = 13 \cdot 400 \cdot 100 = 520000$ N, the border of compressed zone goes in the rib. The section strength is checked according to condition (28).
For that we determine the height of compressed zone x by Formula (29):

$$x = \frac{R_s A_s - R_b (b_f^{'} - b) h_f^{'}}{R_b b} = \frac{365 \cdot 1964 - 13(400 - 200)100}{13 \cdot 200} =$$

= 176 mm $< \xi_R h_0 = 0.604 \cdot 530 = 320$ mm (ξ_R is found according to table 18);

$$R_b bx(h_0 - 0.5x) + R_b (b_h^{'} - b) h_f^{'} (h_0 - 0.5 h_f^{'}) = 13 \cdot 200 \cdot 176 (530 - 0.5 \cdot 176) +$$

$+ 13(400 - 200)100(530 - 0.5 \cdot 100) = 327.1 \cdot 10^6$ N·mm = 327.1 kN·m > M =300 kN·m, that is the strength of the section is provided.

MEMBERS WORKING IN SKEW BENDING

3.24. Calculation of rectangular sections, T-sections, double T- and L-sections of members working in skew bending can be made taking the form of compressed zone according to Draft 6, at the same time the following condition must be met:

$$M_x \le R_b \left[A_{web} (h_0 - x_1 / 3) + S_{ov,x} \right] + R_{sc} S_{sx} \tag{35}$$

where M_x is a component of a bending moment in plane of axes x (for axes x and y we take to perpendicular axes going trough the center of gravity of section of tensile reinforcement parallel to the section sides; for a section with a flange axis x is taken parallel to the rib plane);

$$A_{web} = A_b - A_{ov} \tag{36}$$

A_b compressed concrete zone area equal to:

$$A_b = \frac{R_s A_s - R_{sc} A_s^{'}}{R_b} \tag{37}$$

A_{ov} is the area of the most compressed overhang of a flange;

x_1 the measurements of compressed zone along the most compressed lateral side of the section determined by the following formula:

$$x_1 = \frac{2}{3} \frac{R_b A_{web}^2}{R_b (b_0 A_{web} + S_{ov,y}) + R_{sc} S_{sy} - M_y} \tag{38}$$

b_0 is the distance from the center of gravity of the section of tensile reinforcement to the most compressed lateral face of the rib (side);

$S_{ov,y}$ is static moment of the area A_{ov} in the plane of axis y relating to axis x;

S_{sy} is static moment of the area $A_s^{'}$ in the plane of axis y relating to axis x;

M_y is a component of bending moment in the plane of axis y;

$S_{ov,x}$ is static moment of the area A_{ov} in the plane of axis x relating to axis x;

S_{sx} is static moment of the area $A_s^{'}$ in the plane of axis x relating to axis y.

Draft 6. Form of compressed zone in cross section of reinforced concrete element working in biaxial bending

a – T-section; *b* – rectangular section; *1* – plane of bending moment; *2* – center of gravity of tensile reinforcement section.

If considered in the calculation tensile reinforcement rods are located in plane of axis x (Draft 7) value x_1 is determined by the following formula:

$$x_1 = -t + \sqrt{t^2 + 2 A_{web} ctg\beta} \tag{39}$$

Where $t = 1.5 \left(\frac{S_{ov,y} ctg\beta - S_{ov,x}}{A_{web}} + b_0 ctg\beta - h_0 \right)$;

β is angle of dip of the bending moment plane to axis x that is ctgβ = M_x / M_y.

Drafts 7. Section with tensile reinforcement rods in the plane of axis *x*.

Formula (39) must be also used independently on reinforcement location if it is necessary to determine limit value of bending moment by given angle β.

During calculation of rectangular sections values A_{ov}, $S_{ov,x}$ and $S_{ov,y}$ in formulas (35), (36), (38) and (39) are taken equal to zero.

If $A_b < a_{ov}$ or $x_1 < 0.2h_f^{'}$ so that means that the calculation is made as for rectangular section $b = b_f^{'}$ wide.

If the following condition is met:

$$x_1 < \frac{1.5A_{web}}{b = b_{ov}} \tag{40}$$

(where b_{ov} the width of the least compressed overhang of the flange), so the calculation is made without considering skew bending that is according to formulas of Items 3.15 and 3.20 as regards moment $M = M_x$ at the same time it is necessary to check the condition (41) taking x_1 as by skew bending.

During determination of the value A_b by formula (37) the stress in the closest to compressed zone border tensile bar must be no less than R_s that corresponds to the following condition:

$$\xi_i = \frac{b_{ov}^{'} tg\theta = x_1}{\left(b_{0i} = b_{ov}^{'}\right) tg\theta + h_{0i}} \leq \xi_R \tag{41}$$

Where ξ_R – see Tables 18 and 19

b_{0i}, h_{0i} are the distances from the rod under consideration to the most compressed and lateral surface of the rib (side) and to the most compressed surface normal to axis x (see Draft 4);

$b_{ov}^{'}$ – The width of the most compressed overhang;

θ – Angle of slope of the line bounding the compressed zone to axis y; value of tgθ is determined by the following formula:

$$tg\theta = \frac{x_1^2}{2A_{web}}.$$

If condition (41) is not met so the calculation of the section is made by means of step-by-step approximation and replacing in formula (37) value R_s for each tensile rod by stress values equal to:

$\sigma_{si} \psi_c (\omega / \xi_i - 1) R_s$ But no more than R_s,

Where ψ_c, ω are taken according to Table 18 and 19, at the same time axes x and y must be drawn through resultant of forces in tensile rods.

During design of structures value ξ_i must not exceed value ξ_R more than by 20 percent, at the same time it is possible to make only one repeated calculation with replacement of values R_s in formula (37) for tensile rods for which $\xi_i > \xi_R$ by stresses equal to :

$$\sigma_{si} \frac{\psi_c (\omega / \xi_i - 1) + 2}{3} R_s \tag{42}$$

By repeated calculation value x_1 is determined by formula (39) independently on location of tensile rods.

The calculation as regards skew bending is made according to Item 3.27 if the following conditions are met:

- for rectangular sections, T- and L-sections with a flange in compressed zone

$$x_1 > h \tag{43}$$

- for double T-, T- and L-sections with a flange in tensile zone

$$x_1 > h - h_f - b_{ov,t} tg\theta \tag{44}$$

where h_f, $b_{ov,t}$tgθ is the height and the width of the least tensile overhang of a flange (Draft 8).

Draft 8. T-section with compressed zone going into the least tensile overhang of a flange.

When using formula (37) it is recommended to take reinforcement located near tensile surface which s parallel to axis *y* for tensile reinforcement with the area A_s, and to take reinforcement located near tensile surface which s parallel to axis *y* but on one the most compressed side of axis *x* (see Draft 6) for compressed reinforcement with the area $A_s^{'}$.

3.25. It is recommended to determine required quantity of tensile reinforcement by skew bending for rectangular section, T- and L-section elements with a flange in compressed zone by means of Draft 9. For that α_s is determined by means of the diagram depending on the following values:

$$\alpha_{mx} \frac{M_x - R_b S_{ov,x} - R_{sc} S_{sx}}{R_b b_0 h_0^2};$$

$$\alpha_{my} = \frac{M_y - R_b S_{ov,y} - R_{sc} S_{sy}}{R_b b_0^2 h_0}$$

[symbols see in Formulas (35)–(38)].

If $\alpha_{mx} < 0$ so the calculation is made as for rectangular section taking $b = b_f^{'}$.

If value α_s on the diagram is located on the left side of the curve corresponding to parameter $\frac{b_{ov} + b}{b_0}$, choosing of reinforcement is made without considering skew bending that is according to Items 3.18, 3.19 and 3.22 as regards the moment $M = M_x$.

Draft 9. Diagram of bearing capacity of rectangular, T- and L-sections for members working in skew bending

Required area of tensile reinforcement by work condition of its total design resistance is determined by the following formula:

$$A_s = (\alpha_s b_0 h_0 + A_{ov}) \frac{R_b}{R_s} + A_s^{'} \tag{45}$$

where A_{ov} – see Formula (36).

Center of gravity of the section of actual tensile reinforcement must be distant from tensile surfaces no more than the taken in the calculation center of gravity. Otherwise the calculation is made one more time taking the new center of gravity of tensile reinforcement.

Work condition of tensile reinforcement with total design resistance is the fulfillment of condition (41).

For members made of concrete B25 and lower condition (41) is always met if value α_s on Draft 9 is located inside the area bounded by coordinate axes and a curve corresponding to parameter $b_{ov}^{'} / b_0$.
If condition (41) is not met so it is necessary to put (increase) compressed reinforcement or increase class of concrete or to increase dimensions of the section (especially dimensions of the most compressed overhang).

Values α_s on the diagram must not be located between axis α_{my} and a curve corresponding to parameter h_0 / h. Otherwise x is more than h and the calculation is made according to Item 3.27.

3.26. The calculation of skew bending of rectangular and double T-shaped symmetric sections with symmetrically located reinforcement can by made according to Item 3.76 taking $N = 0$.

3.27. For sections not settled in Items 3.24–3.26 as well as if conditions (43) and (44) are met and if reinforcement is spread on the section that disturbs the determination of values A_s and $A_s^{'}$ and location of centers of gravity of tensile and compressed reinforcement, calculation as regards skew bending must be made using the formula of general case of normal section calculation (see Item 3.76) considering instructions of Item 3.13.

Order of use of formulas of general case is the following:
1) two perpendicular to each other axes x and y are drawn through the center of gravity of the section of the most tensile rod if possible parallel to section sides;
2) By means of step-by-step approximation it is chosen the location of a line bounding compressed zone so that the equation (154) by $N = 0$ was met after insertion into it the value σ_{si} determined by formula (155). At the same time angle of slope of this line θ is taken as a permanent one and equal to angle of slope of neutral axis determined as for elastic body;
3) there are determined moments of internal stresses relating to axes x and y – M_{yu} and M_{xu}.

If both these moments are more or less then correspondent components of external moment (M_y or M_x) so the strength of the section is correspondingly provided (if more) and not provided (if less).

If one of moments (for example M_{yu}) is less than the correspondent component of external moment M_y and the other moment is more than the component of external moment (that is $M_{xu} < M_x$) so it is taken a larger angle of slope θ (more than the one taken earlier) and the same calculation is made one more time.

Examples of Calculation

Example 10. *Given:* reinforced concrete collar beam of the roof with the slope 1:4 (ctgβ = 4); section and location of reinforcement is corresponding to Draft 10; $\gamma_{b2} = 0.9$ (no loads of short duration); heavy-weight concrete B25 (R_b = 13 MPa); stretched reinforcement class A-III (R_s = 365 MPa); its section area A_s = 763 mm^2 (3Ø18); $A_s^{'} = 0$; bending moment in a vertical plane M = 82.6 kN·m.

It is required to check the strength of the section.

Calculation. In compliance with Draft 10 it is:

$$h_0 = 400 - 30 - \frac{1 \cdot 30}{3} = 360 \text{ mm};$$

$$b_0 = \frac{2 \cdot 120 + 1 \cdot 30}{3} = 90 \text{ mm};$$

$$b_{ov}^{'} = b_{ov} = \frac{300 - 150}{2} = 75 \text{ mm};$$

$$h_f^{'} = 80 + \frac{20}{2} = 90 \text{ mm}$$

Draft 10. To calculation example 10.

1 – bending moment plane; *2* – center of gravity of tensile reinforcement section

According to Formula (37) we determine the area of compressed zone of concrete A_b:

$$A_b = \frac{R_s A_s}{R_b} = \frac{365 \cdot 763}{13} = 21420 \text{ mm}^2.$$

Area of the most compressed overhang of a flange and static moments of its area relating to axes x and y are correspondingly equal to:

$$A_{ov} b_{ov}^{'} h_f^{'} = 75 \cdot 90 = 6750 \text{ mm}^2;$$

$$S_{ov,y} = A_{ov}\left(b_0 + b_{ov}^{'}/2\right) = 6750(90 + 75/2) = 86.06 \cdot 10^4 \text{ mm}^3;$$

$$S_{ov,x} = A_{ov}\left(h_0 + h_f^{'}/2\right) = 6750(360 + 90/2) = 212.6 \cdot 10^4 \text{ mm}^3.$$

As $A_b > A_{ov}$ so the calculation is continued as for T-section.

$$A_{web} = A_b A_{ov} = 21420 - 6750 = 14670 \text{ mm}^2.$$

The components of bending moment in the plane of axes y and x are correspondingly equal to (by ctgβ = 4):

$$M_y = M \sin\beta = \frac{M}{\sqrt{1 + ctg^2\beta}} \frac{82.6}{\sqrt{1 + 4^2}} = 20 \text{ kN·m};$$

$$M_x = M_y ctg\beta = 20 \cdot 4 = 80 \text{ kN·m}$$

According to formula (38) we determine dimensions of compressed zone of concrete x_1 relating to the most compressed side of the section, taking $S_{sy} = 0$:

$$x_1 = \frac{2}{3} \frac{R_b A_{web}^2}{R_b\left(b_0 A_{web} + S_{ov,y}\right) - M_y} = \frac{2}{3} \frac{13 \cdot 14670^2}{13(90 \cdot 14676 + 860600) - 2000000} = 223 \text{ mm}$$

Let's check condition (40):

$$\frac{1.5 A_{web}}{b + b_{ov}} = \frac{1.5 \cdot 14670^2}{150 + 75} = 97.8 \text{ mm} < x_1 = 233 \text{ mm}$$

So the calculation is to be continued by formulas foe skew bending.

Let's check condition (41) for the least tensile rod. According to Draft 10 $b_{0i} = 30$ mm, $h_{0i} = 400 - 30 = 370$ mm:

$$tg\theta = \frac{x_2^1}{2A_{web}} = \frac{223^2}{2 \cdot 14670} = 1.695;$$

$$\xi_i = \frac{b_{ov} tg\theta = x_1}{(b_{0i} + b_{ov}) tg\theta + h_{0i}} = \frac{75 \cdot 1.695 + 223}{(30+75)1.695 + 370} = 0.64 > \xi_R = 0.604$$

(see Table 18).

Condition (41) is not met. Let's make one more calculation; in formula (37) we replace value R_s for the least tensile rod by stress σ_s determined by formula (42), correcting values h_0 and b_0.

From Table 18 we have $\omega = 0.746$ and $\psi_c = 4.26$.

$$\sigma_s = \frac{\psi_c (\omega / \xi_i - 1) + 2}{3} R_s = \frac{4.26(0.746/0.64 - 1) + 2}{3} R_s = 0.902 R_s$$

As all rods have the same diameter so new values A_b, b_0 and h_0 will be equal to:

$$A_b = 21420 \frac{2 + 0.902}{3} = 20720 \text{ mm}^2;$$

$$b_0 = \frac{2 \cdot 120 + 0.902 \cdot 30}{2 + 0.902} = 92.0 \text{ mm};$$

$$h_0 = 400 - 30 - \frac{1 \cdot 30}{2 + 0.902} = 359.7 \text{ mm}$$

Then let's determine values $S_{ov,y}$, $S_{ov,x}$ and A_{web}:

$$S_{ov,y} = 6750(92 + 75/2) = 87.41 \cdot 10^4 \text{ mm}^3;$$

$$S_{ov,x} = 6750(359.7 + 90/2) = 212.4 \cdot 10^4 \text{ mm}^3;$$

$$A_{web} = 20720 - 6750 = 13970 \text{ mm}^2.$$

Value x_1 we determine by formula (39):

$$t = 1.5\left(\frac{S_{ov,y} ctg\beta - S_{ov,x}}{A_{web}} + b_0 ctg\beta - h_0\right) = 1.5\left(\frac{874100 \cdot 4 - 2124000}{13970} + 92 \cdot 4 - 359.7\right) = 159.8 \text{ mm}$$

$$x_1 = -t + \sqrt{t^2 + 2A_{web} ctg\beta} = -159.8 + \sqrt{159.8^2 + 2 \cdot 13970 \cdot 4} = 210.7 \text{ mm}$$

Let's check the section strength according to condition (35) taking $S_{sx} = 0$:

$$R_b \left[A_{web}(h_0 - x_1/3) + S_{ov,x}\right] = 13\left[13970\left(359.7 - \frac{210.7}{3}\right) + 212.4 \cdot 10^4\right] = 80.2 \cdot 10^6 \text{ N·mm} >$$

$$> M_x = 80 \cdot 10^6 \text{ N·mm},$$

that is the section strength is provided.

Example 11. According to data of Example 10 it is necessary to choose the area of tensile reinforcement by moment in vertical plane $M = 64$ kN·m.

Calculation. Components of bending moment in the plane of axes y and x are equal to:

$$M_y = M \sin\beta = \frac{M}{\sqrt{1 + ctg^2\beta}} = \frac{64 \cdot 10^6}{\sqrt{1 + 4^2}} = 15.5 \cdot 4 = 62 \text{ kN·m}$$

We determine required quantity of reinforcement according to Item 3.25.

Taking values $b_0, h_0, S_{0v,x}, S_{0v,y}, S_{sy} = S'_{sy} = 0$ according to Example 10 we find values α_{mx} and α_{my}:

$$\alpha_{mx} = \frac{M_x - R_b S_{ov,x}}{R_b b_0 h_0^2} = \frac{62 \cdot 10^6 - 13 \cdot 212.4 \cdot 10^4}{13 \cdot 90 \cdot 360^2} = 0.227;$$

$$\alpha_{my}=\frac{M_y-R_bS_{ov,y}}{R_bb_0^2h_0}=\frac{15.5\cdot10^6-13\cdot86.06\cdot10^4}{13\cdot90^2\cdot360}=0.114.$$

As $\alpha_{mx}>0$ so the calculation is continued as for T-section.

As the point with coordinates $\alpha_{mx}=0.227$ and $\alpha_{my}=0.114$ on Drawing 9 is located on the right side of the curve corresponding to parameter $\frac{b+b_{ov}}{b_0}=\frac{150+75}{90}=2.5$ and on the left side of the curve corresponding to parameter $\frac{b_{ov}}{b_0}=\frac{75}{90}=0.83$ so the reinforcement will work with total design resistance that is condition (41) is met. Required area of tensile reinforcement is determined according to formula (45).

According to Draft 9 by $\alpha_{mx}=0.227$ and $\alpha_{my}=0.114$ we find $\alpha_s=0.25$

Then by $A_s^{'}=0$ we have

$$A_s=(\alpha_sb_0h_0+A_{ov})\frac{R_b}{R_s}=(0.25\cdot90\cdot360+6750)\frac{13}{365}=529 \text{ mm}^2.$$

We take rods 3Ø16 ($A_s=603$ mm^2) and arrange them as it is shown on Draft 10.

Example 12. *Given:* hinged wall plate of a public building with a span 5.8 m with the cross section according to Draft 11; light-weight concrete B3.5, average density grade D1100; reinforcement A-III; load on the panel at stage of use – dead weight and weight of located above glass (including the pier) 3 m height 3.93 kN/m^2, from plane of the panel – wind load 0.912 kN/m^2.

It is required to check the strength of the panel at stage of use.

Calculation. First we determine bending moments in the middle section of the panel in the plane and out of the plane of the panel.

According to Item 2.13 we determine the load of dead weight of the panel. As class of light weight concrete is lower than B12.5 so density of concrete is $\gamma=1.1D=1.1\cdot1100=1210$ kg/m^3. So the load of dead weight of the panel will be:

$$q_w=bh\gamma\cdot0.01=0.34\cdot1.2\cdot1210\cdot0.01=4.94 \text{ kN/m}$$

And considering safety factor $\gamma_f=1.2$ (as $\gamma<1800$ kg/m^3)

$$q_w=1.2\cdot4.94=5.92 \text{ kN/m}$$

Load of located above glazing weight is $q_g=3.93\cdot3=11.8$ kN/m.

Total load in the panel plane is:

$$q_x=q_w+q_g=5.92+11.8=17.72 \text{ kN/m},$$

and moment of this load in the middle of the panel is:

$$M_x=\frac{q_xl^2}{8}=\frac{17.72\cdot5.8}{8}=74.5 \text{ kN·m}$$

Wind load per 1 m of the panel length considering the load of located above and below glazing is:

$$q_y=0.912(1.2+3)=3.83 \text{ kN/m}$$

and moment of this load is:

$$M_y=\frac{q_yl^2}{8}=\frac{3.83\cdot5.8^2}{8}=16.1 \text{ kN·m}$$

As reinforcement is spread unevenly on the section so the strength is to be checked by formulas of a general case in compliance with Item 3.76 (considering Item 3.13).

We'll give numbers to all rods as it is shown on Draft 11. Through the center of the most stretched rod *1* we draw axis *x* parallel to dimension h = 1195 mm and axis *y* parallel to dimension b = 340 mm.

Draft 11. To example of calculation 12

! – 8 – rods

Angle θ between axis y and the straight line bounding the compressed zone is taken as for the calculation of elastic body as regards biaxial bending:

$$tg\theta = \frac{M_y}{M_x}\frac{I_x}{I_y} = \frac{M_y}{M_x}\left(\frac{h}{b}\right)^2 = \frac{16.1}{74.5}\left(\frac{1.195}{0.34}\right)^2 = 2.67\,.$$

In the first approximation we determine the area of concrete compressed zone by formula (37) that is taking all rods with total design strength, at the same time rod *8* is taken as a compressed one and other ones are taken as tensile ones.

For rods *1, 2, 7, 8* (Ø10) we have $R_s = R_{sc}$ = 365 MPa, and for rods *3 – 6* (Ø6) – R_s = 355 MPa, then:

$$R_s R_s = 365 \cdot 236 + 355 \cdot 113 = 126250 \text{ N};$$

$$R_{sc} A_s^{'} = 365 \cdot 78.5 = 28650 \text{ N}.$$

As there is a wind load so value R_b is taken considering the coefficient $\gamma_{b2} = 1.1$ that is $R_b = 2.3$ MPa.

$$A_b = \frac{R_s A_s - R_{sc} A_s^{'}}{R_b} = \frac{126250 - 28650}{2.3} = 42440 \text{ mm}^2$$

Area of compressed zone on the assumption that it's of triangular form is determined by formula $A_b = \dfrac{x_1^2}{2tg\theta}$ where x_1 is dimension of compressed zone by the section side *h* so x_1 is:

$$x_1 = \sqrt{2tg\theta A_b} = \sqrt{2 \cdot 26.7 \cdot 42440} = 476 \text{ mm} < h = 1195 \text{ mm}.$$

Dimension y_1 of compressed zone by section side *b* is:

$$y_1 = \frac{x_1}{tg\theta} = \frac{476}{2.76} = 172 \text{ mm} < b = 340 \text{ mm},$$

that is compressed zone is of triangular form in fact.

If we put these dimension on Draft 11 so we can see that rod *8* is located in compressed zone and all other rods are in tensile zone. Let's check the stress σ_{si} in the closest to tensile zone border rods that is in rods *6 – 8* by formula (155), determining the ratios $\xi = \dfrac{x}{h_{0i}}$ by formula $\xi_i = \dfrac{x_1}{a_{yi} tg\theta = a_{xi}}$ where a_{xi} and a_{yi} are the distances from *i*-rod to the most compressed side of the section in direction of axes *x* and *y*.

Taking $\sigma_{sc,u} = 400$ MPa, $\omega = 0.8 - 0.008 R_b = 0.8 - 0.008 \cdot 2.3 = 0.782$ we get

$$\sigma_{si} = \frac{\sigma_{sc,u}}{1 - \frac{\omega}{1.1}}\left(\frac{\omega}{\xi_1} - 1\right) = \frac{400}{1 - \frac{0.782}{1.1}}\left(\frac{0.782}{\xi_i} - 1\right) = \frac{1082}{\xi_i} - 1384 \text{ (MPa)}$$

The calculations we summarize in the following Table:

Rod number	A_{si}, mm^2	a_{yi}, mm	a_{xi}, mm	$a_{yi} tg\theta + a_{xi}$	ξ_i	σ_{si} <> R_s

				, mm		MPa
6	28.3	40	555	662	0.719	120.9<355
7	78.5	300	80	881	0.54	620>365
8	78.5	40	80	187	2.545	-959<-365

During determination of A_b it was taken incorrect stress only for rod *6*: 355 MPa instead of 120.9 MPa. In this rod we take stress larger than the calculated one, – $\sigma_{s6} = 160$ MPa. From equation (150) by $N = 0$ we determine value A_b:

$$A_b = \frac{\sum \sigma_{si} A_{si}}{R_b} = \frac{365 \cdot 263 + 355 \cdot 85 + 160 \cdot 28.3 - 28650}{2.3} = 40080 \text{ mm}^2.$$

By a similar way we determine $x_1 = \sqrt{2 \cdot 2.67 \cdot 40080} = 463$ mm.
So for rod *6* we have:

$$\xi_6 = \frac{x_1}{a_{yi} tg\theta + a_{xi}} = \frac{463}{662} = 0.699;$$

$$\sigma_{s6} = \frac{1082}{0.699} - 1384 = 164 \text{ MPa},$$

That is value σ_{s6} is close to the excepted one and so it is not necessary to calculate values A_b and x_1 one more time.
Let's determine moments of internal forces relating to axes *y* and *x* – M_{xu} and M_{yu}.

$$M_{xu} = R_b A_b \left(a_{x1} - \frac{x_1}{3} \right) - \Sigma \sigma_{si} A_{si} \left(a_{x1} - a_{xi} \right) = 2.3 \cdot 40080 \left(1115 - \frac{463}{3} \right) -$$
$$- 355 \cdot 2 \cdot 28.3 (1115 - 1015) - (355 + 160) \cdot 28.3 (1115 - 555) -$$
$$- (365 \cdot 78.5 - 365 \cdot 78.5)(1115 - 80) = 78.4 \cdot 10^6 \text{ N·mm} =$$
$$+ 78.4 \text{ kN·m} > M_x = 74.5 \text{ kN·m};$$

$$y_1 = \frac{x_1}{tg\theta} = \frac{463}{2.67} = 173 \text{ mm};$$

$$M_{yu} = R_b A_b \left(a_{y1} - \frac{y_1}{3} \right) - \Sigma \sigma_{si} A_{si} \left(a_{y1} - a_{yi} \right) = 2.3 \cdot 40080 \left(300 - \frac{173}{3} \right) -$$
$$- (365 \cdot 78.5 + 355 \cdot 28.3 + 160 \cdot 28.3 - 365 \cdot 78.5)(300 - 40) =$$
$$= 18.55 \cdot 10^6 \text{ N·mm} = 18.5 \text{ kN·m} > M_y = 16.1 \text{ kN·m}.$$

As both internal moments exceed both components of external moment so the section strength is provided.

CALCULATION OF SECTIONS INCLINED TO LONGITUDINAL AXIS OF THE MEMBER

3.28. (3.29) It is necessary to calculate inclined sections of reinforced concrete members to provide the strength:

- against lateral force along inclined band between inclined cracks in compliance with Item 3.30;
- against lateral force along inclined crack for members with cross reinforcement in compliance with Items 3.31-3.39, for members without cross reinforcement - in compliance with Items 3.40 and 3.41;
- against bending moment along inclined crack in compliance with Items 3.42-3.47.

Short consoles of columns are calculated as regards the lateral forces along inclined compressed band between the load and the support in compliance with Item 3.99.

Beams loaded by one or two point forces located no farther than h_0 from the support as well as short beams with a span $l \leq 2h_0$ are to be calculated as regards lateral force considering the strength of inclined compressed band between the load and the support considering proper recommendations. It is possible to calculate such beams as members without cross reinforcement according to Item 3.40.

Note. In the present document under cross reinforcement we understand stirrups and bend-up bars. Definition "stirrups" includes cross rods of welded frameworks and stirrups of bound frameworks.

3.29. Distances between stirrups s, between a support and a bend-up bar end s_1 as well as between the end of a previous and beginning of the next bending s_2 (Draft 12) must be no more than value s_{max}:

$$s_{\max} = \frac{\varphi_{b4} R_{bt} b h_0^2}{Q} \tag{46}$$

Where φ_{b4} – see table 21.

Besides, these distances must correspond to constructive requirements of Items 5.69 and 5.71.

Draft 12. Distances between stirrups, support and bendings.

By linear width b variation along the height it is necessary to insert [into the formula (46) and the following ones] the width of the member at the level of the middle of the section height (without considering flanges).

CALCULATION OF MEMBERS AS REGARDS LATERAL FORCE ALONG INCLINED COMPRESSED BAND

3.30. (3.30) Calculation of reinforced concrete members as regards lateral force to provide the strength along inclined stripe between inclined cracks must be made according to the following condition:

$$Q \leq 0.3 \varphi_{w1} \varphi_{b1} R_b b h_0 \tag{47}$$

where Q is lateral force in a normal section taken at the distance from the support no less than h_0;

φ_{w1} – Coefficient considering influence of stirrups normal to the member axis and determined by the following formula:

$$\varphi_{w1} = 1 + 5\alpha\mu_w \tag{48}$$

but no more than 1.3;

Here $\mu_w = \frac{A_{sw}}{bs}$;

φ_{b1} – Coefficient determined by the following formula:

$$\varphi_{b1} = 1 - \beta R_b \tag{49}$$

here β is the coefficient taken equal to 0.01 – for heavy-weight and fine concrete; 0.02 – for light-weight concrete;

R_b is in MPa.

CALCULATION OF INCLINED SECTION AS REGARDS LATERAL FORCE ALONG INCLINED CRACK

Members with constant height reinforced by stirrups without bend-up bars

3.31. The strength of inclined section as regards lateral force along inclined crack (Draft 13) is made according to the following condition:

$$Q \le Q_b + Q_{sw} \tag{50}$$

where Q is lateral force of external load located on one side of inclined section under review; by vertical load on the top surface of the member value Q is taken in the normal section going through the most distant from the support end of inclined section; by the load on the bottom surface of the member or within the height of its section it is also possible to take value Q as the most distant from the support end of inclined section if stirrups installed according to Item 3.97 are not considered in the present calculation; at the same time it is necessary to consider absence of live load within the inclined section;

Q_b is cross force taken by concrete and equal to:

$$Q = \frac{M_b}{c}, \tag{51}$$

$$M_b = \varphi_{b2}\left(1+\varphi_f\right)R_{bt}bh_0^2; \tag{52}$$

φ_{b2} – Coefficient considering concrete type and determined by Table 21;

φ_f – Coefficient considering influence of compressed flanges of T- and double-T-section elements and determined by the following formula:

$$\varphi_f = 0.75\frac{\left(b_f^{'} - b\right)h_f^{'}}{bh_0} \tag{53}$$

but no more than 0.5,

At the same time value $(b_f^{'} - b)$ is taken no more than $3h_f^{'}$;

reinforcement in the rib is anchored in the flange where there is cross reinforcement connecting flange overhangs with a rib;

c is projection length of inclined axis upon member determined according to Item 3.32.

Draft 13. Forces model in inclined section of elements with stirrups during its calculation as regards lateral force

Table 21

Concrete	Coefficients		
	φ_{b2}	φ_{b3}	φ_{b4}
Heavy-weight concrete	2.00	0.6	1.5
Fine concrete	1.70	0.5	1.2
Light-weight concrete			
- D1900	1.90	0.5	1.2
- D1800 and lower by fine aggregate			
dense	1.75	0.4	1.0
porous	1.50	0.4	1.0

Value Q_b is taken no less than $Q_{b,\min} = \varphi_{b3}\left(1+\varphi_f\right)R_{bt}bh_0$ (φ_{b3} - see Table 21);

Q_{sw} is cross force carried by stirrups equal to:

$$Q_{sw} = q_{sw} c_0 \qquad (54)$$

here q_{sw} is force in stirrups per length unit of the member within inclined section determined by the following formula:

$$q_{sw} = \frac{R_{sw} A_{sw}}{s}; \qquad (55)$$

c_0 is projection length of inclined crack upon longitudinal axis of the member taken equal to:

$$c_0 = \sqrt{\frac{M_b}{q_{sw}}} \qquad (56)$$

But no more than c and no more than $2h_0$ as well as no less than h_0 if $c > h_0$

At the same time for stirrups installed according to the calculation (that is when requirements of Items 3.40 and 3.41 are not met) it is necessary to follow the following requirement:

$$q_{sw} \geq \frac{Q_{b,\min}}{2h_0} \qquad (57)$$

It is possible not to follow the requirement (57) if in formula (52) it is considered such reduced value $R_{bt}b$ when condition (57) becomes an equation that is if $M_b = 2h_0^2 q_{sw} \dfrac{\varphi_{b2}}{\varphi_{b3}}$;

in that case it is always $c_0 = 2h_0$ but no more than c.

3.32. By checking of condition (50) in general case it is taken a row of inclined sections by different values c not exceeding the distance from the support to the section with maximum bending moment and no more than $(\varphi_{b2} / \varphi_{b3}) h_0$.

If point loads act on the element so values c are taken equal to the distances from the support to the point of the force impact (Draft 14).

Draft 14. Location of design inclined sections by point loads

1 – inclined section checked as regards the lateral force Q_1; *2* – the same of force Q_2

By calculation of the member as regards the distributed load q value c is taken equal to $\sqrt{\dfrac{M_b}{q_{sw}}}$ and if $q_1 > 0.65 q_{sw}$ so it is necessary to take $c = \sqrt{\dfrac{M_b}{q_1 + q_{sw}}}$ where q_1 is determined in the following manner:

a) if there is actual distributed load $q_1 = q$;
b) If load q includes live load which reduces to equivalent distributed load v (when diagram of moment M of accepted in the calculation load v always bends round the diagram M of any actual live load), $q_1 = g + v/2$ (where g is dead continuous load).

At the same time value Q is taken equal to $Q_{\max} - q_1 c$ where $Q_{\max}$ is lateral force in support section.

3.33. Required density of stirrups shown by means of q_{sw} (see Item 3.31) is determined in the following manner:

a) by point forces located at the distances c_i of the support acting on the element, for each inclined section with the projection length c_i not exceeding the distance to the section with maximum bending moment value q_{sw} is determined according to the coefficient $\chi_i = \dfrac{Q_i - Q_{bi}}{Q_{bi}}$ by one of the following formulas:

If $æ_i < æ_{0i} = \dfrac{Q_{b,\min}}{Q_{bi}} \dfrac{c_0}{2h_0}$,

$$q_{sw(i)} = \frac{Q_i}{c_0} \frac{æ_{0i}}{æ_{0i} + 1}; \tag{58}$$

If $æ_{0i} \le æ_i \le \dfrac{c_i}{c_0}$,

$$q_{sw(i)} = \frac{Q_i - Q_{bi}}{c_0}; \tag{59}$$

If $\dfrac{c_i}{c_0} < æ_i \le \dfrac{c_i}{h_0}$,

$$q_{sw(i)} = \frac{(Q_i - Q_{bi})^2}{M_b}; \tag{60}$$

If $æ_i > \dfrac{c_i}{h_0}$,

$$q_{sw(i)} = \frac{Q_i - Q_{bi}}{h_0}; \tag{61}$$

(here h_0 is taken no more than c_i).

Finally it is taken maximum value $q_{sw(i)}$.

In formulas of Item 3.33:

- Q is lateral force in the normal section located at the distance c_i from the support;
- Q_{bi} is determined by formula (51) by $c = c_i$;
- $Q_{b,\min}, M_b$ See Item 3.31;
- c_0 is taken equal to c_i but no more than $2h_0$;

b) by distributed load q required density of stirrups is determined by the following formulas:

By $Q_{\max} \le \dfrac{Q_{bi}}{0.6}$

$$q_{sw} = \frac{Q_{\max}^2 - Q_{b1}^2}{4M_b}; \tag{62}$$

By $\dfrac{M_b}{h_0} + Q_{b1} > Q_{\max} > \dfrac{Q_{b1}}{0.6}$

$$q_{sw} = \frac{(Q_{\max} - Q_{b1})^2}{M_b}; \tag{63}$$

In both cases q_{sw} is taken no less than $\left(\frac{Q_{\max} - Q_{b1}}{2h_0}\right)$;

By $Q_{\max} \geq \frac{M_b}{h_0} + Q_{b1}$

$$q_{sw} = \frac{Q_{\max} - Q_{b1}}{h_0} \tag{64}$$

In case if calculated value q_{sw} doesn't correspond to condition (57) so it must be calculated by the following formula:

$$q_{sw} = \frac{Q_{\max}}{2h_0} + \frac{\varphi_{b2}}{\varphi_{b3}} q_1 - \sqrt{\left(\frac{Q_{\max}}{2h_0} + \frac{\varphi_{b2}}{\varphi_{b3}} q_1\right)^2 - \left(\frac{Q_{\max}}{2h_0}\right)^2},$$

Here $Q_{b1} = 2\sqrt{M_b q_1}$

$Q_{\max}$ – Lateral force in the support section;

$M_b, \varphi_{b2}, \varphi_{b3}$ – see in Item 3.31;

q_1 see in Item 3.32.

3.34. If from the support to the span density of stirrups reduces from q_{sw1} to q_{sw2} (for example by stirrups spacing increase) it is necessary to check condition (50) by values c increasing l_1 the length of the member part with stirrups quantity q_{sw1} (Draft 15). At the same time value Q_{sw} is taken equal to:

By $c - l_1 < c_{01}$

$$Q_{sw} = q_{sw1}c_{01} - (q_{sw1} - q_{sw2})(c - l_1);$$

By $c_{02} > c - l_1 > c_{01}$

$$Q_{sw} = q_{sw2}(c - l_1);$$

By $c - l_1 > c_{02}$

$$Q_{sw} = q_{sw2}c_{02},$$

Where c_{01}, c_{02} are determined by formula (56) by q_{sw} equal to q_{sw1} and q_{sw2}.

Draft 15. to the calculation of inclined sections by density of stirrups alterations.

By distributed load acting on the element the length of the part with density q_{sw1} is taken no less than value l determined in the following manner:

If $q_1 > q_{sw1} - q_{sw2}$,

$$l_1 = c - \frac{M_b / c + q_{sw1}c_{01} - Q_{\max} + q_1 c}{q_{sw1} - q_{sw2}},$$

Where $c = \sqrt{\frac{M_b}{q_1 - (q_{sw1} - q_{sw2})}}$ but no more than $\frac{\varphi_{b2}}{\varphi_{b3}} h_0$;

At the same time if $q_1 > 1.56 q_{sw1} - q_{sw2}$

So $c = \sqrt{\frac{M_b}{q_1 + q_{sw2}}}$;

If $q_1 \le q_{sw1} - q_{sw2}$

$$l_1 = \frac{Q_{\max} - \left(Q_{b,\min} + q_{sw2} c_{01}\right)}{q_1} - c_{01}$$

Here q_1 - see Item 3.32.

If condition (57) is not met for the value q_{sw2} so the length l_1 is calculated by values $M_b = 2h_0^2 q_{sw2} \varphi_{b2} / \varphi_{b3}$ and $Q_{b,\min} = 2h_0 q_{sw2}$ corrected according to Item 3.31, at the same time the sum $Q_{b,\min} + q_{sw2} c_{01}$ is taken no less than the uncorrected value $Q_{b,\min}$.

Members with a constant height reinforced by bend-up bars

3.35. Strength test of inclined section against lateral force for members with bend-up bars is made according to condition (50) with addition to the right part of the condition (50) the following value:

$$Q_{s,imc} = R_{sw} A_{s,inc} \sin\theta \qquad (65)$$

Where $A_{s,inc}$ – section area of bend-up bars crossing dangerous inclined crack with the projection length c_0;

θ is angle of slope of bend-up bars to the longitudinal axis of the member.

Value c_0 is taken equal to the length of the member part within the inclined section under review for which the expression $Q_{sw} + Q_{s,inc} + Q_b = q_{sw} c_0 + Q_{s,inc} + \frac{M_b}{c_0}$ gets minimum value. For that it is necessary to consider the parts from the end of inclined section or from the end of bend-up bar within the length *c* to the beginning of the bend-up bar which is close to the support (Draft 16) at the same time the length of the part is taken no more than c_0 determined by formula (56) and inclined cracks not crossing bend-up bars by values c_0 less than the ones determined by formula (56) are not considered in the calculation.

Draft 16. For determination of the most dangerous inclined section for members with bend-up bars by calculation as regards lateral force.
1 – 4 – possible inclined sections; *5* – considered inclined section

On Draft 16 the most dangerous inclined crack corresponds to minimum value of the following expressions:

$1 - q_{sw} c_{01} + R_{sw} A_{s,inc1} \sin\theta + M_b / c_{01}$;

$2 - q_{sw} c_{02} + R_{sw} A_{s,inc2} \sin\theta + M_b / c_{02}$;

$3 - q_{sw} c_{03} + R_{sw} A_{s,inc3} \sin\theta + M_b / c_{03}$;

$4 - q_{sw} c_0 + R_{sw} (A_{s,inc1} + A_{s,inc2}) \sin\theta + M_b / c_{02}$

[here c_0 - see formula (56)].

Values *c* are taken equal to the distances from the support to the ends of bend-up bars as well as to points of concentrated forces; besides it is necessary to examine inclined sections which cross the last plane of bend-up bars and end at the distance c_0 determined by formula (56) from the beginning of the last and next to the last planes of bend-up bars (Draft 17).

Location of bend-up bars must meet the requirements of Items 3.29, 5.71 and 5.72.

Draft 17. Location of inclined sections of the member with bend-up bars
1 – 4 – calculated inclined sections.

Variable height elements with cross reinforcement

3.36. (3.33) Calculation of members with inclined compressed surfaces as regards lateral forces is made according to Items 3.31, 3.32, 3.34 and 3.35 considering recommendations of Items 3.37 and 3.38 taking maximum value h_0 as working height (Draft 18, a).

It is also recommended to make calculation of members with inclined stretched sections as regards lateral forces in compliance with Items 3.31, 3.32, 3.34 and 3.35 taking maximum value h_0 within inclined section (Draft 18, b).

Draft 18. Beams with variable height and inclined surface
a – compressed; *b* – tensile

Angle β between compressed and tensile surfaces of member must meet the requirement $tg\beta < 0.4$.

3.37. For beams without bend-up bars with height evenly increasing from the support to the span (see Draft 18) calculated as regards distributed load q inclined section is tested according to condition (50) by the most disadvantageous value c determined in the following manner:

if the following equation is met:

$$q_1 < 0.56 q_{sw} - 2.5\sqrt{q_{inc} q_{sw}} \quad (66)$$

so value c is determined in by the following formula

$$c = \sqrt{\frac{M_{b1}}{q_{inc} + \sqrt{q_{inc} q_{sw}} + q_1}} \quad (67)$$

if condition (66) is not met so value c is calculated by the following formula:

$$c = \sqrt{\frac{M_{b1}}{q_{inc} + q_{sw} + q_1}} \text{ (At the same time } c_0 = c\text{)} \quad (68)$$

As well as if $q_{sw} < M_{b1}/(4h_{01}^2)$,

$$c = \sqrt{\frac{M_{b1}}{q_{inc} + 2q_{sw} tg\beta + q_1}} \text{ (At the same time } c_0 = 2h_0\text{)} \quad (69)$$

Here $q_{inc} = \varphi_{b2} R_{bt} b tg^2 \beta$;

M_{b1} – Value M_b determined by formula (52) as for support section of beam with working height h_{01} without considering enlarged footing b;

β – Angle between compressed and tensile surfaces of the beam;

q_1 – See Item 3.32.

Working height h_0 is taken equal to $h_0 = h_{01} + ctg\beta$.

By decrease of density of stirrups from q_{sw1} at the support to q_{sw2} in the span it is necessary to examine condition (50) by values *c* exceeding l_1 the length of the part of the member with density of stirrups q_{sw1}, at the same time value Q_{sw} is determined according to Item 3.34.

Parts of beams with permanent decrease of working height h_0 must not be less than the accepted value *c*.
By point loads acting on the beam, it is necessary to examine inclined sections by values *c* taken in compliance with Item 3.32 as well as if $tg\beta > 01$ determined by formula (68) by $q_1 = 0$.

3.38. For consoles without bend-up bars with height evenly exceeding from the free supported beam to the support (Draft 19) in general case it is necessary to examine condition (50) taking inclined sections with values *c*, determined by formula (68) by $q_1 = 0$ and taken no more than the distances from the beginning of the inclined section in tensile zone to the support. At the same time for h_{01} and *Q* there are taken working height and shear at the beginning of the inclined section in tensile zone. Besides it is necessary to examine inclined sections carried on to the support if $c_2 < c$.

Draft 19. Console with the height decreasing from the support to the free supported end

By distributed loads acting on the console the inclined section is located in tensile zone of normal sections going through the points of application of these loads (see Draft 19).

By distributed load or linear increasing to the support the console is calculated as an element with the constant height according to Items 3.31 and 3.32 taking working height h_0 in support section.

Members with cross reinforcement by biaxial bending

3.39. Calculation as regards shear force of members with rectangular section exposed to biaxial bending is made according to the following condition:

$$\left[\frac{Q_x}{Q_{bw(x)}}\right]^2 + \left[\frac{Q_y}{Q_{bw(y)}}\right]^2 \leq 1 \qquad (70)$$

Where Q_x, Q_y are components of shear force acting in the plane of symmetry *x* and in the normal to it plane *y* at the most distant from the support end of inclined section;

$Q_{bw(x)}, Q_{bw(y)}$ Are limit shear forces on inclined section by acting of these forces in planes *x* and *y* and taken equal to the right part of the condition (50).

By distributed load acting on the member it is possible to determine value *c* according to Item 3.32 for each plane *x* and *y*.

Note. Bend-up bars are not considered by calculation as regards the shear force by biaxial bending.

Members without cross reinforcement

3.40. (3.32) Calculation of members without cross reinforcement as regards the shear force is made according to the following conditions:

a) $Q_{max} \leq 2.5R_{bt}bh_0$ (71)

Where Q_{max} – maximum shear force at the support surface;

b) $Q \leq \dfrac{\varphi_{b4}R_{bt}bh_0^2}{c}$ (72)

where Q is shear force at the end of inclined section;

φ_{b4} – Coefficient determined by Table 21;

c – Is projection length of inclined section starting from the support; value c is taken no more than $c_{max} = 2.5h_0$.

In continuous flat slabs with constrained edges (connected with other elements or having supports) it is possible to divide the mentioned value c_{max} by the coefficient α:

$$\alpha = 1 + 0.05b/h \quad (73)$$

but no more than 1.25.

By testing of condition (72) in general case it is taken value c no more than c_{max}.

By point loads acting on the element values c are taken equal to the distances from the support to the application points of these loads (Draft 20).

Draft 20. Location of the most disadvantageous sections in elements without cross reinforcement.
1 – inclined section tested as regards shear force action Q_1; *2* – the same for force Q_2

During calculation of the element as regards distributed load if the following condition is met:

$$q_1 \leq \frac{\varphi_{b4}R_{bt}b}{(c_{max}/h_0)^2} \quad (74)$$

so value c in condition (72) is taken equal to c_{max} and if condition (74) is not met so

$$c = h_0\sqrt{\frac{\varphi_{b4}R_{bt}b}{q_1}} \quad (75)$$

here q_1 is taken by distributed loads in compliance with Item 3.32 and by continuous load with linear variable density it is taken equal to average density of the support part with the length equal to a quarter of the beam (slab) span or to a half of the console overhang but no more than c_{max}.

3.41. For elements with variable height of the section during testing of condition (71) value h_0 is taken in support section and during testing of condition (72) as average value h_0 within inclined section.

For elements with the section height increasing by increase of shear force value c_{max} is taken equal to $c_{max} = \dfrac{2.5h_{01}}{1+1.25tg\beta}$ at the same time for continuous flat plates mentioned in Item 3.40 $c_{max} = \dfrac{2.5h_{01}}{\alpha+1.25tg\beta}$,

Where h_{01} – working height in support section;

β is angle between tensile and compressed surfaces of the element;

α see formula (73) where h can be taken according to support section.

By distribution load acting on such element value c in condition (72) is taken equal to:

$$c = h_{01}\sqrt{\frac{1}{tg^2\beta/4 + q_1/(\varphi_{b2}R_{bt}b)}} \quad (76)$$

but no more than $c_{\max}$ where q_1 – see Item 3.40.

CALCULATION OF INCLINED SECTIONS AS REGARDS BENDING MOMENT

3.42. (3.35) Calculation of elements as regards bending moment to provide the strength along inclined crack (Draft 21) must be made according to the following condition:

$$M \le R_s A_s z_s + \Sigma R_{sw} A_{sw} z_{sw} + \Sigma R_{sw} A_{s,inc} z_{s,inc} \quad (77)$$

where M is moment of external load located on one side of considered inclined section relating to the axis which is perpendicular to the moment action plane and going through the point of application of resultant force N_b in compressed zone (Draft 22);

$R_s A_s z_s$ – Moments sum relating to the same axis from forces in longitudinal
$\Sigma R_{sw} A_{sw} z_{sw}$ Reinforcement, stirrups and bend-up bars crossing stretched zone of
$\Sigma R_{sw} A_{s,inc} z_{s,inc}$ inclined section;

$z_s, z_{sw}, z_{s,inc}$ – The distances from the planes of longitudinal reinforcement, stirrups and bend-up bars to the mentioned axis.

Draft 21. Forces scheme in inclined section by its calculation as regards bending moment.

Draft 22. Determination of design value of the moment by calculation of inclined section
a – for a free supported beam; *b* – for a console

The height of compressed zone measured by a line normal to longitudinal axis of the member is determined according to the requirements of equilibrium of forces projections in concrete of compressed zone and in reinforcement crossing the inclined section onto a longitudinal axis of the member according to Items 3.15 and 3.20. If there are bend-up bars in the member so in the numerator for value x it is necessary to add value $\Sigma R_{sw} A_{s,inc} \cos\theta$ (where θ is angle of inclination of bend-up bars to the longitudinal axis of the member).

Value z_s can be taken equal to $h_0 - 0.5x$ but considering compressed reinforcement no more than $h - a'$.

Value $\Sigma R_{sw} A_{sw} z_{sw}$ by constant stirrups quantity is determined by the following formula:

$$\Sigma R_{sw} A_{sw} z_{sw} = 0.5 q_{sw} c^2 \quad (78)$$

Where q_{sw} is force in the stirrups per unit of length (see Item 3.31);

c is the length of the projection of incline section onto the longitudinal axis of the member measured between points of application of resultant forces in tensile reinforcement and compressed zone (see Item 3.45).

Values $z_{s,inc}$ for each plane of bend-up bars are determined by the following formula:

$$z_{s,inc} = z_s \cos\theta + (c - a_1)\sin\theta \qquad (79)$$

where a_1 is the distance from the beginning of the inclined section to the beginning of a bend-up bar in tensile zone (see Draft 21).

3.43. (3.35) Calculation of inclined sections as regards the moment is made in points of break or bending of longitudinal reinforcement as well as at surfaces of end free support of a beams and at free end of consoles if there is no special anchors of longitudinal reinforcement.

Besides, calculation of inclined sections as regards the moment is made in points of abrupt change of the element configuration.

It is possible not to make calculation of inclined sections as regards the moment if conditions (71) and (72) are met by multiplying their right parts by 0.8 and by values c no more than $0.8c_{\max}$.

3.44. If inclined section crosses longitudinal inclined reinforcement without anchors so design strength of this reinforcement R_s within anchorage zone must be decreased by means of multiplying it by the work condition coefficient γ_{s5} equal to:

$$\gamma_{s5} = \frac{l_x}{l_{an}} \qquad (80)$$

where l_x is the distance from the end of reinforcement to the cross point of inclined section with longitudinal reinforcement;

l_{an} anchorage zone length determined by the following formula:

$$l_{an} = \left(\omega_{an}\frac{R_s}{R_b} + \Delta\lambda_{an}\right)d \qquad (81)$$

Here $\omega_{an}, \Delta\lambda_{an}$ are coefficients taken equal to:

For end free supports of beams $\omega_{an} = 0.5, \Delta\lambda_{an} = 8$

For free ends of consoles $\omega_{an} = 0.7; \Delta\lambda_{an} = 11$

In case of use of plain rods coefficient ω_{an} is taken equal to 0.8 for supports of beams and to 1.2 for ends of consoles.

If end free supports have confinement or cross reinforcement bounding longitudinal reinforcement without welding coefficient ω_{an} must be divided by value $1+12\mu_v$ and coefficient $\Delta\lambda_{an}$ is decreased by the value $10\sigma_b / R_b$ here μ_v is volume coefficient of reinforcement determined as for welded meshes by formula (99), for stirrups – by formula $\mu_v = \frac{A_{sw}}{2as}$ (where A_{sw} and s – are correspondingly section area of a bounding stirrup and its spacing), in any case value μ_v is taken no more than 0.06.

Concrete compression stress on the support σ_b is determined by means of dividing of support reaction by support area of the element and is taken no more than $0.5R_b$.

The length l_{an} is taken no less than 20d or 250 mm for free ends of consoles, at the same time the length of anchorage l_{an} can be determined considering the data of Table 45 (Pos.1).

In case if cross reinforcement or distribution bars are welded to longitudinal tensile bars so considered in the calculation strengthening of longitudinal reinforcement R_sA_z is decreased by the following value:

$$N_w = 0.7 n_w \varphi_w d_w^2 R_{bt} \tag{82}$$

Taken no less than $0.8 R_s d_w^2 n_w$

In formula (82):

n_w – Number of welded rods along the length l_x;

φ_w – Coefficient taken by Table 22;

d_w – Diameter of welded bars.

Table 22

d_w	6	8	10	12	14
φ_w	200	150	120	100	80

Finally value R_sA_z is taken no more than value R_sA_z without considering γ_{s5} and N_w.

3.45. For free supported beams the most disadvantageous inclined section begins from the surface support and has the projection length c for beams with permanent section height equal to:

$$c = \frac{Q - F_i - R_{sw} A_{s,inc} \sin\theta}{q_{sw} + q} \tag{83}$$

but no more than maximum length of support part beyond which condition (72) is met multiplying the right part by 0.8 and by c no more than $0.8c_{max}$.

In formula (83):

Q – shear force in support section;

$F_{i,q}$ – point load and distributed load within inclined section;

$A_{s,inc}$ – section area of bend-up bars crossing the inclined section;

θ – angle of slope of bend-up bars to longitudinal axis of the element;

q_{sw} – the same like in formula (55).

If value c determined considering point load F_i will be less than the distance to the surface of the support to force F_i and value c determined without considering force F_i – more than this distance so it is necessary to take the distance to force F_i for value c.

If within the length c the stirrups change their density from q_{sw1} at the beginning of the inclined section to q_{sw2} so value c is determined by formula (83) by $q_{sw} = q_{sw2}$ and if numerator is decreased by the value $(q_{sw1} - q_{sw2})l_1$ (where l_1 is the length of the part with stirrups quantity q_{sw1}).

For beams loaded by distributed load q with constant density of stirrups without bends condition (77) can be replaced by the following condition:

$$Q \le \sqrt{2(R_s A_s z_s - M_0)(q_{sw} + q)} \tag{84}$$

where Q – is shear force in support sectionж

M_0 – is the moment in the section along the surface of the support.

For consoles loaded by point forces (Draft 22, b) the most disadvantageous inclined section begins from the point of application of point forces near free end and has the projection length c for consoles with a constant height equal to:

$$c = \frac{Q_1 - R_{sw} A_{s,inc} \sin\theta}{q_{sw}} \tag{85}$$

but no more than the distance from the beginning of the inclined section to the support (here Q_1 is shear force in the beginning of the inclined section).

For consoles loaded by only by distributed load q the most disadvantageous inclined section ends at the support section and has the projection length c equal to:

$$c = \frac{R_s A_s z_s}{l_{an}(q_{sw} + q)} \tag{86}$$

At the same time if $c < l - l_{an}$ so it is possible not to make the calculation of inclined section.

In formula (86):

A_s – is section area of reinforcement going to the free end;
z_s – see Item 3.42; value z_s is determined for support section;
l_{an} – length of the anchorage zone (see Item 3.44).

For members with the section height increasing at the same time with the increase of bending moment by determination of the projection length of the most disadvantageous section by formulas (83) and (85) numerators of these formulas are to be decreased by value $R_sA_s\text{tg}\beta$ – by inclined compressed surface, and by value $R_sA_s\sin\beta$ – by inclined tensile section (where β is angle of slope of the surface to the horizontal line).

3.46. To provide the strength of inclined sections to bending moment in members of constant height with stirrups longitudinal tensile bars break in the span must be get to the point of break in theory (that is behind the normal section where external moment becomes equal to the bearing capacity of the section without considering broken bars, Draft 23) by length no less than value w determined by the following formula:

$$w = \frac{Q - R_{sw} A_{s,inc} \sin\theta}{2q_{sw}} + 5d \tag{87}$$

where Q is shear force in the normal section going through the theoretical break point;
$A_{s,inc}, \theta$ The same like in formula (83);
b is diameter of a broken bar;
q_{sw} see in Item 3.31.

For beams with inclined compressed surface, numerator of formula (87) is decreased by $R_sA_s\text{tg}\beta$ and for beams with inclined tensile surface – by $R_sA_s\sin\beta$ (where β is angle of slope of the surface to the horizontal line). Besides, it is necessary to consider requirements of Item 5.44.

For members without cross reinforcement value w is taken equal to $10d$, at the same time point of theoretical break must be located on the part of the element on which condition (72) is met multiplying the right part by 0.8 and by value c no more than $0.8c_{max}$.

Draft 23. Break of tensile bars in the span

1 – point of theoretical break; *2* – diagram *M*

3.47. To provide strength of inclined sections against bending moment the beginning of bar bending in tensile zone must be distant from the normal section where bend-up bar is used according to the moment no less than by $h_0/2$ and the end of the bar bending must be located no closer than that normal section where bar bending is not required according to the calculation.

CALCULATION OF INCLINED SECTIONS IN UNDERCUTS

3.48. For members with sharply-changing section height (for example for beams and consoles with undercuts) it is necessary to make calculation as regards shear force for inclined sections going at the console support created by a undercut (Draft 24) according to Items 3.31-3.39, at the same time it is necessary to insert working height h_{01} of the short console formed by an undercut into the calculation formulas.

Stirrups necessary for inclined section strength must be fixed behind the end of undercut on the part no less than w_0 determined by formula (88).

Draft 24. The most disadvantageous inclined sections n members with undercut
1 – inclined compressed stripe; *2* – by calculation as regards shear force; *3* – the same as regards bending moment; *4* – the same as regards bending moment beyond undercut

3.49. For free supported beams with undercuts it is necessary to make the calculation as regards bending moment in inclined section going through re-entrant angle of undercut (see Draft 24) according to Items 3.42-3.45. At the same time longitudinal tensile reinforcement in the short reinforcement formed by the undercut must be get behind the end of the undercut at the length no less than l_{an} (see Item 5.44) and no less than w_0 equal to:

$$w_0 = \frac{2\left(Q_1 - R_{sw}A_{sw1} - R_{sw}A_{s,inc}\sin\theta\right)}{q_{sw}} + a_0 + 10d \tag{88}$$

where Q_1 is shearing force in the normal section at the end of undercut;
A_{sw1} is section area of additional stirrups located at the end of the undercut on the part no more than $h_{01}/4$ long and which are not considered by determination of stirrups quantity q_{sw} at the undercut;
$A_{s,inc}$ is section area of bend-up bars going through the re-entrant angle of undercut;
a_0 is the distance from the console support to the end of the undercut;
d is diameter of a broken bar.

Stirrups and bend-up bars fixed at the end of the undercut must conform to the following requirement:

$$R_{sw}A_{sw1} + R_{sw}A_{s,inc}\sin\theta \geq Q_1\left(1 - h_{01}/h_0\right) \tag{89}$$

Where h_{01}, h_0 – working height in the short console of the undercut and in the beam beyond the undercut.

If bottom reinforcement of the element has no anchorage so according to Items 3.42-3.45 it is also necessary to check the strength of inclined section located beyond the undercut and beginning behind the mentioned stirrups at the distance no less than $h_0 - h_{01}$ from the end face (see Draft 24). At the same time in the calculation it is not considered longitudinal reinforcement of short console and projection length c is taken no less than the distance from the inclined section to the end of mentioned reinforcement. Besides anchorage length l_{an} for bottom reinforcement is taken as for free ends of consoles.

Calculation of the short console of undercut is made according to Items 3.99 and 3.100 taking direction of compressed strip from external edge of area of bearing to resultant force in additional stirrups with the section area A_{sw1} at the level of compressed reinforcement of beams that is by $\sin^2\theta = \frac{(h_{01}-a')^2}{(h_{01}-a')^2+(l_{\sup}+a_x)^2}$ (where l_{sup} – see Item 3.99, a_x – see Draft 24); at the same time in Formula (207) coefficient 0.8 is replaced by 1.0.

EXAMPLES OF CALCULATION

Calculation of inclined sections as regards lateral force

Example 13. *Given*: reinforced concrete floor slab with dimensions of cross section according to Draft 25; heavy-weight concrete B15 ($R_b = 7.7$ MPa $R_{bt} = 0.67$ MPa by $\gamma_{b2} = 0.9$; $E_b = 20.5 \cdot 10^3$ MPa); rib of the slab is reinforced by plane welded framework with cross reinforcement rods A-III, diameter 8 mm ($A_{sw} = 50.3$ mm^2; $R_{sw} = 285$ MPa; $E_s = 2 \cdot 10^5$ MPa), with spacing $s = 100$ mm; equivalent live load v = 18 kN/m; gravity load of the slab and floor g = 3.9 kN/m; cross force on the support $Q_{\max} = 62$ kN.

It is required to test the strength of inclined strip of the rib between inclined cracks as well as strength of inclined sections to shear force.

Draft 25. For example of calculation 13

Calculation. $h_0 = 350 - 58 = 292$ mm. Strength of inclined strip is to be calculated according to condition (47).

We determine coefficients φ_{w1} and φ_{b1}:

$$\mu_w = \frac{A_{sw}}{bs} = \frac{50.3}{85 \cdot 100} = 0.0059;$$

$$\alpha = \frac{E_s}{E_b} = \frac{2 \cdot 10^5}{20.5 \cdot 10^3} = 9.76;$$

So $\varphi_{w1} = 1 + 5\alpha\mu_w = 1 + 5 \cdot 9.76 \cdot 0.0059 = 1.29 < 1.3$;

For heavy-weight concrete $\beta = 0.01$;

$$\varphi_{b1} = 1 - \beta R_b = 1 - 0.01 \cdot 7.7 = 0.923,$$

So $0.3\varphi_{w1}\varphi_{b1}R_b bh = 0.3 \cdot 1.29 \cdot 0.923 \cdot 7.7 \cdot 85 \cdot 292 = 68300$ N > Q_{max} = 62 kN, that is the strength of inclined section is provided.

Strength of inclined section to shear force is to be tested according to condition (50).
We determine values M_b and q_{sw}:

$$\varphi_{b2} = 2.0 \text{ (See Table 21)};$$

As $b'_f - b = 475 - 85 = 390$ mm > $3h'_f = 3 \cdot 50 = 150$ mm so we take $b'_f - b = 150$, and

$$\varphi_f = 0.75\frac{(b'_f - b)h'_f}{bh_0} = 0.75\frac{150 \cdot 50}{85 \cdot 292} = 0.227 < 0.5;$$

$$M_b = \varphi_{b2}(1+\varphi_f)R_{bt}bh_0^2 = 2(1+0.227)0.67 \cdot 85 \cdot 292^2 = 11.92 \cdot 10^6 \text{ N·mm} = 11.92 \text{ kN·m};$$

$$q_{sw}\frac{R_{sw}A_{sw}}{s} = \frac{285 \cdot 50.3}{100} = 143 \text{ N/mm (kN/m)}$$

We determine value $Q_{b,\min}$ taking $\varphi_{b3} = 0.6$:

$$Q_{b,\min} = \varphi_{b3}(1+0.227)0.67 \cdot 85 \cdot 292 = 12240 \text{ N} = 12.24 \text{ kN}.$$

As $\frac{Q_{b,\min}}{2h_0} = \frac{12.24}{2 \cdot 0.292} = 21$ kN/m < q_{sw} = 143 kN/m,

so condition (57) is met that means it is not necessary to correct value M_b.

According to Item 3.32 we determine projection length of the most disadvantageous inclined section c:

$$q_1 = g + v/2 = 3.9 + 18/2 = 12.9 \text{ kN/m (N/mm)},$$

As $0.56q_{sw} = 0.56 \cdot 143 = 80$ kN/m > $q_1 = 12.9$ kN/m so value c is to be determined only by the following formula:

$$c = \sqrt{\frac{M_b}{q_1}} = \sqrt{\frac{11.92}{12.9}} = 0.962 \text{ m};$$

So $$Q_b = \frac{M_b}{c} = \frac{11.92}{0.962} = 12.4 \text{ kN} > Q_{b,\min} = 12.2 \text{ kN};$$

$$Q = Q_{\max} - q_1 c = 62 - 12.9 \cdot .962 = 49.3 \text{ kN}$$

Projection length of inclined crack is equal to:

$$c_0 = \sqrt{\frac{M_b}{q_{sw}}} = \sqrt{\frac{11.9}{143}} = 0.288 \text{ m} < 2h_0$$

As $c_0 = 0.288 < h_0 = 0.292$ m so we take $c_0 = h_0 = 0.292$ m, so $Q_{sw} = q_{sw}c_0 = 143 \cdot 0.292 = 41.8$ kN.

Let's check condition (50):

$$Q_b = Q_{sw} = 12.4 + 41.8 = 54.2 \text{ kN} > Q = 49.3 \text{ kN},$$

that is strength of inclined section to shear force is provided. Besides it is necessary to meet a requirement of Item 3.29:

$$s_{\max} = \frac{\varphi_{b4}R_{bt}bh_0^2}{Q_{\max}} = \frac{1.5 \cdot 0.67 \cdot 85 \cdot 292^2}{62 \cdot 10^3} = 117.5 \text{ mm} > s = 100\text{mm}.$$

Conditions of Item 5.69 $s < h/2 = 350/2 = 175$ mm and s < 150 mm are also met.

Example 14. *Given*: free supported reinforced concrete beam of the floor with the span l = 5.5 m; equivalent distributed live load on the beam v = 36 kN/m; dead load g = 14 kN/m; dimensions of the cross section b = 200 mm, h = 400 mm, $h_0 = 370$ mm; heavy-weight concrete B15 ($R_b = 7.7$ MPa $R_{bt} = 0.67$ MPa by $\gamma_{b2} = 0.9$); stirrups of reinforcement A-I ($R_{sw} = 175$ MPa).

It is required to determine diameter and spacing of stirrups at the support as well as to find out how it is possible to increase spacing of stirrups.

Calculation. The largest shear force in the support section is equal to:

$$Q_{\max} = \frac{ql}{2} = \frac{50 \cdot 5.5}{2} = 137.5 \text{ kN},$$

Where $q = v + g = 36 + 14 = 50$ kN/m

We determine required density of stirrups of support part according to Item 3.33b.

By means of formula (52) by $\varphi_f = 0$ and $\varphi_{b2} = 2.0$ (see table 21) we get:

$$M_b = \varphi_{b2} R_{bt} b h_0^2 = 2 \cdot 0.67 \cdot 200 \cdot 370^2 = 36.7 \cdot 10^6 \text{ N·mm} = 36.7 \text{ kN·m}.$$

According to Item 3.32

$$q_1 = g + v/2 = 14 + 36/2 = 32 \text{ kN/m (N/mm)};$$

$$Q_{b1} = 2\sqrt{M_b q_1} = 2\sqrt{36.7 \cdot 32} = 68.4 \text{ kN}$$

As $\dfrac{Q_{b1}}{0.6} = \dfrac{68.4}{06} = 114$ kN < Q_{max} = 137.5 kN, and $Q_{\max} < \dfrac{M_b}{h_0} + Q_{b1} = \dfrac{36.7}{0.37} + 68.4 = 167$ kN,

Stirrups quantity is to be determined by formula (63):

$$q_{sw} = \frac{(Q_{\max} - Q_{b1})^2}{M_b} = \frac{(137.5 - 68.4)^2}{36.7} = 130 \text{ kN/m (N/mm)}$$

At the same time as $\dfrac{Q_{\max} - Q_{b1}}{2h_0} = \dfrac{(137.5 - 68.4)10^3}{2 \cdot 370} = 93.44$ N/mm < 130 N/mm,

So $q_{sw} = 130$ N/mm

According to Item 5.69 spacing s_1 at the support must be no more than $h/2$ = 200 and 150 mm and in the span – $\dfrac{3}{4}h = 300$ and 500 mm. Maximum allowable spacing at the support according to Item 3.29 is equal to:

$$s_{\max} = \frac{\varphi_{b4} R_{bt} b h_0^2}{Q_{\max}} = \frac{1.5 \cdot 0.67 \cdot 200 \cdot 370^2}{137.5 \cdot 10^3} = 200 \text{ mm}$$

We take stirrups spacing at the support $s_1 = 150$ mm and in the span – $2s_1 = 300$ mm so:

$$A_{sw1} = \frac{q_{sw} s_1}{R_{sw}} = \frac{130 \cdot 150}{175} = 111 \text{ mm}^2.$$

We take two stirrups with diameter 10 mm in the cross section (A_{sw} = 157 mm^2).

So excepted density of stirrups at he support and in the span will be equal to:

$$q_{sw1} = \frac{R_{sw} A_{sw}}{s_1} = \frac{175 \cdot 157}{150} = 183.2 \text{ N/mm};$$

$$q_{sw2} = 0.5 q_{sw1} = 0.5 \cdot 183.2 = 91.6 \text{ N/mm}$$

Let's check condition (57) by means of calculation of $Q_{b,\min}$:

$$Q_{b,\min} = \varphi_{b3}(1 + \varphi_f) R_{bt} b h_0 = 0.6 \cdot 0.67 \cdot 200 \cdot 370 = 29750 \text{ N}.$$

Then $\dfrac{Q_{b,\min}}{2h_0} = \dfrac{29750}{2 \cdot 370} = 40.2$ N/mm < $q_{sw1} = 183.2$ N/mm;

$$\frac{Q_{b,\min}}{2h_0} = 40.2 \text{ N/mm} < q_{sw2} = 96.1 \text{ N/mm}$$

So values q_{sw1} and q_{sw2} are not to be corrected.

We determine the length of part l_1 with stirrups quantity q_{sw1} according to Item 3.34. As $q_{sw1} - q_{sw2} = q_{sw2} = 91.6$ N/mm > $q_1 = 32$ N/mm so value l_1 is to be calculated according to the following formula:

$$l_1 = \frac{Q_{max} - Q_{b,min} - q_{sw2}c_{01}}{q_1} - c_{01} = \frac{137.5 \cdot 10^3 - 29750 - 91.6 \cdot 448}{32} - 448 =$$

$$= 1637 \text{ mm} > \frac{l}{4} = \frac{5.5}{4} = 1.375 \text{ m}$$

(Here $c_{01} = \sqrt{\frac{M_b}{q_{sw1}}} = \sqrt{\frac{36.7 \cdot 10^6}{183.2}} = 448\,\text{mm}$)

We take the length of the part with the stirrups spacing $s_1 = 150$ mm equal to 1.64 m.

Example 15. *Given*: reinforced concrete beam of the floor loaded by point forces as it's shown on Draft 26a; dimensions of cross section – according to Draft 26b; heavy-weight concrete B15 ($R_{bt} = 0.67$ MPa by $\gamma_{b2} = 0.9$); stirrups of reinforcement A-I ($R_{sw} = 175$ MPa).

It is required to determine diameter and spacing of stirrups as well as to find out how it is possible to increase spacing of stirrups.

Draft 26. For example of calculation 15

Calculation. First we determine value M_b according to Item 3.31:

$$\varphi_{b2} = 2 \text{ (See Table 21);}$$

$$h_f^{'} = 100 + 60/2 = 130 \text{ mm (see Draft 26b);}$$

$$b_f^{'} - b = 220 - 80 = 140 \text{ mm} < 3h_f^{'};$$

$$h_0 = 890 - 80 = 810 \text{ mm;}$$

$$\varphi_f = 0.75\frac{(b_f^{'} - b)h_f^{'}}{bh_0} = 0.75\frac{140 \cdot 130}{80 \cdot 810} = 0.211 < 0.5;$$

$$M_b = \varphi_{b2}(1 + \varphi_f)R_{bt}bh_0^2 = 2(1 + 0.211)0.67 \cdot 80 \cdot 810^2 = 85.2 \cdot 10^6 \text{ N·mm} = 85.2 \text{ kN·m}.$$

Let's determine required density of stirrups according to Item 3.33a, taking projection length of inclined section c equal to the distance from the support to the first weight – c_1 = 1.35 m.
Shear force at the distance c_1 from the support is equal to $Q_1 = 105.2$ kN (see Draft 26).
From formula (51) we have:

$$Q_{b1} = \frac{M_b}{c_1} = \frac{85.2}{1.35} = 63.11 \text{ kN} > Q_{b,min} = \varphi_{b3}(1 + \varphi_f)R_{bt}bh_0 =$$

$$= 0.6(1 + 0.211)0.67 \cdot 80 \cdot 810 = 31.55 \text{ kN}$$

Then $æ_1 = \frac{Q_1 - Q_{b1}}{Q_{b1}} = \frac{105.2 - 63.11}{63.11} = 0.667$

As $c_1 = 1.35 \text{ m} < 2h_0 = 2 \cdot 0.81 = 1.62$ m so we take $c_0 = c_1 = 1.35$ m;

$$æ_{01} = \frac{Q_{b,min}}{Q_{b1}}\frac{c_0}{2h_0} = \frac{31.55}{63.11}\frac{1.35}{2 \cdot 0.81} = 0.417$$

As $æ_{01}$ = 0.417 < $æ_1$ = 0.667 < c_1/c_0 = 1 so value $q_{sw(1)}$ is to be determined by the following formula:

$$q_{sw(1)} = \frac{Q_1 - Q_{b1}}{c_0} = \frac{105.2 - 63.11}{1.35} = 31.18 \text{ kN/m}$$

Let's determine q_{sw} by value c equal to the distance from the support to the second weight – c_2 = 2.85 m.

$$Q_{b2} = \frac{M_b}{c_2} = \frac{85.2}{2.85} = 29.9 \text{ kN} < Q_{b,min} = 31.55 \text{ kN}$$

We take $Q_{b2} = Q_{b,\min} = 31.55$ kN.

Corresponding shear force is equal to $Q_2 = 58.1$ kN. As c_2 = 2.85 m > $2h_0 = 1.62$ m we take $c_0 = 2h_0 = 1.62$ m.

$$æ_2 = \frac{Q_2 - Q_{b2}}{Q_{b2}} = \frac{58.1 - 31.55}{31.55} = 0.842 < æ_{02} = \frac{Q_{b,\min}}{Q_{b2}} \frac{c_0}{2h_0} = 1$$

Therefore value $q_{sw(2)}$ is to be determined by formula (58):

$$q_{sw(2)} = \frac{Q_2}{c_0} \frac{æ_{02}}{æ_{02} + 1} = \frac{58.1}{1.62} \frac{1}{2} = 17.93 \text{ kN/m}$$

So value $q_{sw(2)}$ is determined by formula (58):

$$q_{sw(2)} = \frac{Q_2}{c_0} \frac{æ_{02}}{æ_{02} + 1} = \frac{58.1}{1.62} \frac{1}{2} = 17.93 \text{ kN/m}$$

We take maximum value $q_{sw} = q_{sw(1)} = 31.18$ kN/m.

According to welding conditions (see Item 5.13) we take diameter of stirrups 6 mm (A_{sw} = 28.3 mm^2) then spacing of stirrups in support part is:

$$s_1 = \frac{R_{sw} A_{sw}}{q_{sw}} = \frac{175 \cdot 28.3}{31.18} = 159 \text{ mm}$$

We take $s_1 = 150$ mm. Spacing of stirrups in the span we take equal to $s_2 = 2s_1 = 2 \cdot 150 = 300$.

The length of the part with spacing s_1 is determined according to the condition of strength in compliance with Item 3.34; at the same time

$$q_{sw1} = \frac{R_{sw} A_{sw}}{s_1} = \frac{175 \cdot 28.3}{150} = 33 \text{ N/mm;}$$

$$q_{sw2} = 0.5 q_{sw1} = 16.5 \text{ N/mm;}$$

$$q_{sw1} - q_{sw2} = q_{sw2} = 16.5 \text{ N/mm}$$

Let's take the length of the part with stirrups spacing s_1 equal to the distance from the support to the first weight l_1 = 1.35 m; let's check condition (50) by value c equal to the distance from the support to the second weight c = 2.85 m > l_1. Value c_{01} is to be determined by formula (56) by $q_{sw1} = 33$ kN/m:

$$c_{01} = \sqrt{\frac{M_b}{q_{sw1}}} = \sqrt{\frac{85.1}{33}} = 1.6 \text{ m} < 2h_0 = 1.62 \text{ m}$$

As $c - l_1 = 2.85 - 1.35 = 1.5$ m $c_{01} = 1.6$ m so value Q_{sw} in condition (50) is to be taken equal to:

$$Q_{sw} = q_{sw1} c_{01} - (q_{sw1} - q_{sw2})(c - l_1) = 33 \cdot 1.6 - 16.5 \cdot 1.5 = 28.05 \text{ kN;}$$

$$Q_b = Q_{b,\min} = 31.55 \text{ kN;}$$

$$Q_b + Q_{sw} = 31.55 + 28.05 = 59.6 \text{ kN} > Q_2 = 58.1 \text{ kN,}$$

that is strength of this inclined section is provided.

The larger value c is not taken into consideration as by this value shear force sharply reduces. So the length of the part with stirrups spacing $s_1 = 150$ mm is to be taken equal to $l_1 = 1.35$ m.

Example 16. *Given*: reinforced concrete beam of monolithic floor with cross section dimensions according to Draft 27a; location of bend-up bars – according to Draft 27b; equivalent live load on the beam is v = 96 kN/m, dead load is g = 45 kN/m; shear force at the support Q_{max} = 380 kN/m; heavy-weight concrete B15 ($R_{bt} = 0.67$ MPa by $\gamma_{b2} = 0.9$); two-leg stirrups with diameter 6 mm (A_{sw} = 57 mm^2) of reinforcement A-I ($R_{sw} = 175$ MPa), spacing

$s = 150$ mm; bend-up bars A-II ($R_{sw} = 225$ MPa), with the section area: of the first plane – $A_{,s,inc1} = 628\,\text{mm}^2$ (2Ø20), of the second plane – $A_{,s,inc2} = 402\,\text{mm}^2$ (2Ø16).

It is required to examine strength of inclined sections as regards shear force.

Calculation: $h_0 = 600 - 40 = 560\,\text{mm}$. According to Item 3.31 we find values M_b and q_{sw}:

$$\varphi_{b2} = 2 \text{ (See Table 21);}$$

$$b_f^{'} - b = 3h_f^{'} = 3 \cdot 100 = 300 \text{ mm;}$$

$$\varphi_f = 0.75\frac{\left(b_f^{'} - b\right)h_f^{'}}{bh_0} = 0.75\frac{300 \cdot 100}{300 \cdot 560} = 0.134 < 0.5;$$

$$M_b = 2\left(1 + \varphi_f\right)R_{bt}bh_0^2 = 2(1 + 0.134)0.67 \cdot 300 \cdot 560^2 = 143 \cdot 10^6 \text{ kN·m;}$$

$$q_{sw} = \frac{R_{sw}A_{sw}}{s} = \frac{175 \cdot 57}{150} = 66.5 \text{ N/mm}$$

According to tem 3.32 we find $q_1 = g + v/2 = 45 + 96/2 = 93$ kN/m.

According to condition (50) considering formula (65) let's check inclined section with the projection length equal to the distance from the support to the end of the second plane of bend-up bars, that is by c = 50 + 520 + 300 = 870 mm = 0.87 m.

Shear force at the distance $c = 0.87$ m from the support is:

$$Q = Q_{\max} - q_1 c = 380 - 93 \cdot 0.87 = 299.1 \text{ kN}$$

Let's determine projection of dangerous inclined crack c_0 in compliance with Item 3.35.

First we determine maximum value c_0 by formula (56):

$$c_{0,\max} = \sqrt{M_b / q_{sw}} = \sqrt{143/66.5} = 1.466 \text{ m} > 2h_0 = 2 \cdot 0.56 = 1.12\,\text{m;}$$

We take $c_{0,\max} = 1.12\,\text{m}$. As $c = 0.87\,\text{m} < c_{0,\max} = 1.12\,\text{m}$ so we take $c_0 = c = 0.87$ for this inclined section. Inclined crack located between the end of the second and the beginning of the first bending plane (that is not crossing bend-up bars) is not considered in the calculation as $c_0 = 0.30\,\text{m} < c_{0,\max}$.

For the first plane of bend-up bars:

$$Q_{s,inc1} = A_{s,inc}1R_{sw}\sin\theta = 628 \cdot 225 \cdot 0.707 = 99.9 \cdot 10^3\,\text{N} = 99.9 \text{ kN.}$$

So $\dfrac{M}{c} + q_{sw}c_0 + Q_{s,inc1} = \dfrac{143}{0.87} + 66.5 \cdot 0.87 + 99.9 = 322.1 \text{ kN} > Q = 299.1$ kN

that is strength of the present section is provided.

Let's check inclined section that ends at the distance $c_0 = 1.12\,\text{m}$ from the beginning of the first plane of bend-up bars that is by $c = 0.05 + 0.52 + 1.12 = 1.69\,\text{m}$.

Shear force at the distance $c = 1.69$ m from the support is $Q = 380 - 93 \cdot 1.69 = 222.8\,\text{kN}$.

For the second plane of bend-up bars:

$$Q_{s,inc2} = A_{s,inc2}R_{sw}\sin\theta = 402 \cdot 225 \cdot 0.707 = 63.9 \cdot 10^3\,\text{N} = 63.9 \text{ kN.}$$

For this section we take inclined crack going from the end of inclined section to the beginning of the first plane of bend-up bars that is $c_0 = c_{0,\max} = 1.12\,\text{m}$. Inclined cracks going from the end of inclined section to the support and to the beginning of the second plane of bend-up bars are not considered as in the first case $c_0 = c = 1.69\,\text{m} > c_{0,\max} = 1.12\,\text{m}$ and in the second case the crack crosses bend-up bars by $c_0 < c_{0,\max}$.

So $\frac{M_b}{c} + q_{sw}c_0 + Q_{s,inc2} = \frac{143}{1.69} + 66.5 \cdot 1.12 + 63.9 = 223$ kN > Q =222.8 kN, that is strength of the present section is provided.

Let's check inclined section which ends at the distance $c_{0,\max} = 1.12$ m from the beginning of the second plane of bend-up bars that is by $c = 0.05 + 0.52 + 0.30 + 0.52 + 1.12 = 2.51$ m.

Shear force at the distance c = 2.51 m from the support is equal to Q = 380 – 93·2.51 = 146.6 kN.

For this section it is obvious that $c_0 = c_{0,\max} = 1.12$ m and the inclined crack doesn't cross bend-up bars, that is $Q_{s,inc} = 0$. As c = 2.51 m > $\frac{\varphi_{b2}}{\varphi_{b3}} h_0 = \frac{2}{0.6} 0.56 = 1.87$ m so we take

$$Q_b = Q_{b,\min} = \frac{M_b}{\frac{\varphi_{b2}}{\varphi_{b3}} h_0} = \frac{143}{1.87} = 76.5 \text{ kN}.$$

So $Q_b + q_{sw}c_0 + Q_{s,inc} = 76.5 + 66.5 \cdot 1.12 + 0 = 151$ kN > Q = 146.6 kN that is strength of any inclined sections is provided.

In compliance with Item 3.29 let's check the distance between the beginning of the first plane of bend-up bars and the end of the second plane taking shear force at the end of the second plane of bend-up bars Q = 299.1 kN and $\varphi_{b4} = 1.5$:

$$\frac{\varphi_{b4} R_{bt} b h_0^2}{Q} = \frac{1.5 \cdot 0.67 \cdot 300 \cdot 560^2}{299.1 \cdot 10^3} = 316.1 \text{ mm} < 300\text{mm},$$

that is the requirement of Item 3.29 is met.

Example 17. *Given:* a reinforced concrete corner beam with a span 8.8 m; continuous distributed load q = 46 kN/m (Draft 28a); dimensions of the cross section – according to Draft 28b; heavy-weight concrete B25 ($R_{bt} = 0.95$ MPa by $\gamma_{b2} = 0.9$); stirrups made of reinforcement A-I ($R_{sw} = 175$ MPa) diameter 8 mm ($A_{sw} = 50.3$ mm^2), spacing s = 150 mm.
It is required to examine strength of inclined sections as regards shear force.

Draft 28. For calculation example 17

Calculation. The calculation is made according to Item 3.37.

Working height of support section is $h_{01} = 600 - 80 = 520$ mm (Draft 28b).
Let's determine values φ_{f1} and M_{b1} by formulas (53) and (52) as for a support section:

$$h'_f = 150 + 100/2 = 200 \text{ mm};$$

$$b'_f - b = 300 - 100 = 200 \text{ mm} < 3h'_f;$$

$$\varphi_{f1} = 0.75 \frac{(b'_f - b)h'_f}{bh_{01}} = 0.75 \frac{200 \cdot 200}{100 \cdot 520} = 0.577 > 0.5,$$

We take $\varphi_{f1} = 0.5$; $\varphi_{b2} = 2$ (see Table 21);

$$M_{b1} = \varphi_{b2}(1 + \varphi_{b1})R_{bt}bh_{01}^2 = 2(1 + 0.5)0.95 \cdot 100 \cdot 520^2 = 77.06 \cdot 10^6 \text{ N·mm} = 77.06 \text{ kN·mm}.$$

Let's determine value q_{inc} taking tgβ = 1/12:

$$q_{inc} = \varphi_{b2} R_{bt} b tg^2 \beta = 2 \cdot 0.95 \cdot 100 / 12^2 = 1.32 \text{ N/mm (kN/m)}$$

As there is continuous load so we take $q_1 = q = 46$ kN/m.

Let's check condition (66):

$$0.56 q_{sw} - 2.5\sqrt{q_{sw} q_{inc}} = 0.56 \cdot 58.7 - 2.5\sqrt{58.7 \cdot 1.32} = 10.9 \text{ kN/m} < q_1 = 46 \text{kN/m}$$

Condition (66) is not met so value c is to be determined by formula (68):

$$c = \sqrt{\frac{M_{b1}}{q_{inc} + q_{sw} + q_1}} = \sqrt{\frac{77.06}{1.32 + 58.7 + 46}} = 0.853 \text{m},$$

At the same time $c_0 = c = 0.853$ m

Working height of the cross section h_0 at the distance $c = 0.853$ m from the support is:

$$h_0 = h_{01} + c tg \beta = 0.52 + 0.853/12 = 0.591 \text{m}.$$

Let's determine value M_b by $h_0 = 591$ mm:

$$\varphi_f = 0.75 \frac{(b_f' - b) h_f'}{b h_0} = 0.75 \frac{200 \cdot 200}{100 \cdot 591} = 0.508 > 0.5;$$

We take $\varphi_f = 0.5$;

$$M_b = \varphi_{b2}(1 + \varphi_f) R_{bt} b h_0^2 = 2(1 + 0.5) 0.95 \cdot 100 \cdot 591^2 = 99.55 \cdot 10^6 \text{N·mm} = 99.55 \text{ kN·m}.$$

Let's check condition (50) taking shear force at the end of inclined section equal to:

$$Q = Q_{\max} - q_1 c = \frac{ql}{2} - q_1 c = \frac{46 \cdot 8.8}{2} - 46 \cdot 0.853 = 163.2 \text{kN};$$

$$Q_b + Q_{sw} = \frac{M_b}{c} + q_{sw} c_0 = \frac{99.55}{0.853} + 58.7 \cdot 0.853 = 166.8 \text{kN} > \text{Q} = 163.2 \text{ kN},$$

that is strength of inclined sections as regards shear force is provided.

Example 18. *Given:* a console with dimensions according to Draft 29; concentrated force F = 300 kN located at the distance 0.8 m from the support; heavy-weight concrete B15 ($R_{bt} = 0.67$ MPa by $\gamma_{b2} = 0.9$); two-leg stirrups with diameter 8 mm ($A_{sw} = 101 \text{mm}^2$) of reinforcement A-I ($R_{sw} = 175$ MPa), spacing s = 200 mm.
It is required to check the strength of inclined sections as regards shear force.

Draft 29. For the calculation example 18

Calculation. In compliance with Item 3.38 according to condition (50) let's check the inclined section going from the point of application of concentrated force by value c determined by formula (68).

Working height at the point of application of concentrated force is equal to $h_{01} = 650 - (650 - 300)\frac{800}{950} - 50 = 305$ mm (see Draft 29).

By formula (52) let's determine value M_{b1} taking $\varphi_{b2} = 2$ (see Table 21) and $\varphi_f = 0$:

$$M_{b1} = \varphi_{b2}(1 + \varphi_f) R_{bt} b h_{01}^2 = 2 \cdot 1 \cdot 0.67 \cdot 400 \cdot 305^2 = 49.9 \cdot 10^6 \text{ N·mm}$$

Value q_{sw} is: $q_{sw} = \frac{R_{sw}A_{sw}}{s} = \frac{175 \cdot 101}{200} = 88.4$ N/mm (kN/m).

Taking tgβ = $\frac{650-300}{950}$ = 0.369 (see Draft 29) we determine q_{inc}:

$$q_{inc} = \varphi_{b2} R_{bt} b tg^2 \beta = 2 \cdot 0.67 \cdot 400 \cdot 0.369^2 = 73 \text{ N/mm},$$

therefore taking $q_1 = 0$ we have

$$c = \sqrt{\frac{M_{b1}}{q_{inc} + q_{sw}}} = \sqrt{\frac{49.9 \cdot 10^6}{73 + 88.4}} = 556 \text{kN}$$

As value c does not exceed the distance from the load to the support so we take c = 556 mm and determine working height h_0 at the end of inclined section:

$$h_0 = h_{01} + ctg\beta = 305 + 556 \cdot 0.369 = 510 \text{mm}$$

As $2h_0 = 2 \cdot 510 \text{mm} > c_0 = 558 \text{mm}$ so we take $c_0 = 556$ mm.

Value M_b is

$$M_b = \varphi_{b2} R_{bt} b h_0^2 = 2 \cdot 0.67 \cdot 400 \cdot 510^2 = 139.4 \cdot 10^6 \text{N·mm} = 139.4 \text{ kN·m}$$

Therefore $Q_b + Q_{sw} = \frac{M_b}{c} + q_{sw} c_0 = \frac{139.4}{0.556} + 88.4 \cdot 0.556 = 299.9 \text{ kN} \approx Q = 300$ kN,

that is strength of the present section is provided.

For inclined section located from the load to the support we determine value c_0 by formula (56) taking $h_0 = 650 - 50 = 600$ mm:

$$M_b = 2 \cdot 0.67 \cdot 400 \cdot 600^2 = 193 \cdot 10^6 \text{ N·mm};$$

$$c_0 = \sqrt{\frac{M_b}{q_{sw}}} = \sqrt{\frac{193 \cdot 10^6}{88.4}} = 1478 \text{mm} > 2h_0 = 2 \cdot 600 = 1200 \text{mm},$$

We take $c_0 = 2h_0 = 1200$ mm.

As $c_0 = 1200 \text{mm} > c = 800$ mm so it is possible not to examine the mentioned above inclined section. So section of any inclined section is provided.

Example 19. *Given:* continuous floor slab without cross reinforcement 3x6 m, h = 160 mm thick, monolithically connected with beams along the perimeter; equivalent distributed live load v = 50 kN/m^2; load of dead weight and floor $g = 9$ kN/m^2; a = 20 mm; heavy-weight concrete B25 ($R_{bt} = 0.95$ MPa by $\gamma_{b2} = 0.9$).

It is required to examine the slab strength as regards shear force.

Calculation: $h_0 = h - a = 160 - 20 = 140$mm. The calculation is made for the strip b = 1m = = 1000 mm with a span l = 3m; total load on the slab is $q = v + g = 50 + 9 = 59$ kN/m.

Shear force on the support is

$$Q_{\max} = \frac{ql}{2} = \frac{59 \cdot 3}{2} = 88.5 \text{kN}$$

Let's check condition (71):

$$2.5 R_{bt} b h_0 = 2.5 \cdot 0.95 \cdot 1000 \cdot 140 = 333 \cdot 10^3 \text{N} > Q_{\max} = 88.5 \text{kN}$$

Let's check condition (72). As lateral edges of the slab are connected with beams so value c_{max} is to be determined considering coefficient $\alpha = 1 + 0.05b/h = 1 + 0.05 \cdot 6/0.16 > 1.25$ (here b = 6 m – distance between lateral edges of the slab) that is $\alpha = 1.25$:

$$c_{\max} = \frac{2.5}{\alpha} h_0 = \frac{2.5}{1.25} h_0 = 2h_0 = 2 \cdot 140 = 280 \text{mm}$$

According to Item 3.32 we have :

$$q_1 = g + v/2 = 9 + 50/2 = 34\,\text{kN/m} = 34\ \text{N/mm};$$
$$\varphi_{b4} = 1.5\ \text{(See Table 21)}$$

As $\frac{\varphi_{b4} R_{bt} b}{(c_{\max}/h_0)^2} = \frac{1.5 \cdot 0.95 \cdot 1000}{2^2} = 356\,\text{N/mm} > q_1 = 34$ N/mm so we take $c = c_{\max} = 280$ mm $= 0.28$ m.

Shear force at the end of inclined section is $Q = Q_{\max} - q_1 c = 88.5 - 34 \cdot 0.28 = 79$ kN.

$$\frac{\varphi_{b4} R_{bt} b h_0^2}{c} = \frac{1.5 \cdot 0.95 \cdot 1000 \cdot 140^2}{280} = 99.75 \cdot 10^3\ \text{N} = 99.75\ \text{kN} > Q = 79\ \text{kN},$$

that is the strength of the slab as regards shear force is provided.

Example 20. *Given:* a panel of a tank with variable thickness from 262 (point of embedment) to 120 mm (at free supported end), overhang length is 4.25 m; lateral earth pressure is considering loads of vehicle on the ground surface decreases linearly from $q_0 = 69\,\text{kN/m}^2$ at the point of embedment to q = 7 kN/m^2 at the free supported end; a = 22 mm; heavy-weight concrete B15 ($R_{bt} = 0.82$ MPa by $\gamma_{b2} = 1.1$).

It is required to examine the strength of the panel as regards shear force.

Calculation. Working height of the panel at the point of embedment is $h_{01} = 262 - 22 = 240$ mm.

Let's determine tgβ (β is the angle between the tensile and the compressed surfaces):

$$tg\beta = \frac{262 - 120}{4250} = 0.0334.$$

The calculation is made for the strip of the panel b = 1 m = 1000 mm wide.

Let's check conditions of Item 3.40. Lateral force at the point of embedment is:

$$Q_{\max} = \frac{69 + 7}{2} 4.25 = 161.5\,\text{kN}$$

Let's check condition (71) taking $h_0 = h_{01} = 240\,\text{mm}$:

$$2.5 R_{bt} b h_0 = 2.5 \cdot 0.82 \cdot 1000 \cdot 240 = 492\,\text{kN} > Q_{max} = 117\ \text{kN},$$

that is the condition is met.

As panels are connected with each other and the width of the tank side is more than 5h so we determine value c_{max} considering the coefficient $\alpha = 1.25$:

$$c_{\max} = \frac{2.5 h_{01}}{\alpha + 1.25 tg\beta} = \frac{2.5 \cdot 240}{1.25 + 1.25 \cdot 0.0334} = 464\,\text{mm}$$

Average load density at the support part $c_{\max} = 464\,\text{mm}$ long is $q_1 = 69 - (69 - 7)\frac{464}{4250 \cdot 2} = 65.6$ N/mm. From table 21 $\varphi_{b4} = 1.5$

As $c = h_{01}\sqrt{\frac{1}{tg^2\beta/4 + q_1/(\varphi_{b4} R_{bt} b)}} = 240\sqrt{\frac{1}{0.0334^2/4 + 65.4/(1.5 \cdot 0.82 \cdot 1000)}} = 1037\,\text{mm}$ $> c_{\max} = 464$ mm so we take $c = c_{\max} = 464\,\text{mm}$.

Let's determine working height of the section at the distance $\frac{c}{2}$ from the support (that is average value h_0 within the length c):

$$h_0 = h_{01} - \frac{c}{2} tg\beta = 240 - \frac{464}{2} 0.0334 = 232\,\text{mm}$$

Shear force at the distance c = 464 mm from the support is:

$$Q = Q_{\max} - q_1 c = 161.5 - 65.6 \cdot 0.464 = 131.1 \text{kN}$$

Let's check condition (72):

$$\frac{\varphi_{b4} R_{bt} b h_0^2}{c} = \frac{1.5 \cdot 0.82 \cdot 1000 \cdot 232^2}{464} = 142.7 \cdot 10^3 \text{N} > Q = 131.1 \text{ kN},$$

that is the strength of the panel as regards lateral force is provided.

Calculation of inclined sections as regards bending moment

Example 21. *Given:* a free supported reinforced concrete beam with a span l = 5.5 m with the distributed load q = 29 kN/m; the structure of the support part of the beams is taken according to Draft 30; heavy-weight concrete of class B15 ($R_b = 7.7$ MPa; $R_{bt} = 0.67$ MPa by $\gamma_{b2} = 0.9$); longitudinal reinforcement A-III ($R_s = 365$ MPa) without anchors, section area is $A_s = 982\,\text{mm}^2$ (2Ø25) and $A_s^{'} = 226\,\text{mm}^2$ (2Ø12); stirrups of reinforcement A-I ($R_{sw} = 175$ MPa) with diameter 6 mm, spacing s = 150 mm are welded to longitudinal rods.
It is required to test the strength of inclined sections as regards bending moment.

Draft 30. For the calculation of example 21

Calculation: $h_0 = h - a = 400 - 40 = 360\,\text{mm}$. As tensile reinforcement has no anchors so the calculation of inclined sections as regards the moment is necessary.
Let's take the beginning of inclined section at the surface of the support. Therefore $l_x = l_{\sup} - 10\,\text{mm}$ = 280 – 10 = 270 mm (see Draft 30).
By formula (81) we determine the anchorage length l_{an} taking $\omega_{an} = 0.5$ and $\Delta\lambda_{an} = 8$:

$$l_{an} = \left(\omega_{an} \frac{R_s}{R_b} + \Delta\lambda_{an}\right) d = \left(0.5 \frac{365}{7.7} + 8\right) 25 = 793\,\text{mm}$$

As $l_x < l_{an}$ so design resistance of tensile reinforcement is decreased by means of its multiplying by the coefficient $\gamma_{s5} = \frac{l_x}{l_{an}} = \frac{270}{793} = 0.340$, therefore $R_s = 365 \cdot 0.340 = 124.1$ MPa.
As within the length l_x four vertical and two horizontal cross rods are welded to tensile rods (see Draft 30) so we increase force $R_s A_s = 124.1 \cdot 982 = 121.9 \cdot 10^3$ N by value N_w.
Taking $d_w = 6\,\text{mm}, n_w = 6$, $\varphi_w = 200$ (see Table 22) we get

$$N_w = 0.7 n_w \varphi_w d_w^2 R_{bt} = 0.7 \cdot 6 \cdot 200 \cdot 6^2 \cdot 0.67 = 20.26 \cdot 10^3 \text{ N}.$$

Therefore $R_s A_s = 121.9 + 20.26 = 142.2$ kN
As this value does not exceed value $R_s A_s$ determined without considering γ_{s5} and N_w that is it's equal to $365 \cdot 982 = 358 \cdot 10^3$ N so we take $R_s A_s = 142.2$ kN.
The height of compressed zone is determined by formula (16):

$$x = \frac{R_s A_s - R_{sc} A_s^{'}}{R_b b} = \frac{142.2 \cdot 10^3 - 365 \cdot 226}{7.7 \cdot 200} = 39\,\text{mm} < 2a' = 2 \cdot 35\,\text{mm}$$

In compliance with Item 3.42 we take $z_s = h_0 - a' = 360 - 35 = 325$ mm.
Let's determine value q_{sw} by formula (55):

$$q_{sw} = \frac{R_{sw} A_{sw}}{s} = \frac{175 \cdot 57}{150} = 68.4 \text{ N/mm}$$

Let's determine the projection length of the most disadvantageous section by formula (83) taking value Q equal to support reaction of the beam, that is $Q = \frac{ql}{2} = \frac{29 \cdot 5.5}{2} = 80$ kN as well as $F_i = 0$ and $A_{s,inc} = 0$:

$$c = \frac{Q}{q_{sw} + q} = \frac{80 \cdot 10^3}{68.4 + 29} = 821 \text{ mm}$$

Let's determine maximum length l_s of the support part behind which condition (72) is met with multiplying of the right part by 0.8 and by $c = c_1 \le 0.8c_{\max} = 2h_0$; that is according to the following equation:

$$Q = Q_{\max} = ql_s = 0.8\varphi_{b4} R_{bt} bh_0^2 / c_1 .$$

Supposing that $l_s > 2h_0$ we take maximum value $c_1 = 2h_0$

Then by $\varphi_{b4} = 1.5$ we get:

$$l_s = \frac{Q_{\max} - \frac{0.8\varphi_{b4} R_{bt} bh_0^2}{2h_0}}{q} = \frac{80 \cdot 10^3 - \frac{0.8 \cdot 1.5 \cdot 0.67 \cdot 200 \cdot 360^2}{2 \cdot 360}}{29} = 1760 \text{ mm} >$$

$$> 2h_0 = 2 \cdot 360 = 720 \text{ mm}$$

As $l_s = 1760$ mm $> c$ =821 mm so we take c = 821 mm.

Moment of external forces relating to the axis located in the middle of the height of inclined section here is equal to the bending moment in the normal section going through the mentioned axis; that is at the distance $l_1 + c = l_{\sup}/3 + c = 280/3 + 821 = 914$ mm from the point of application of support reaction:

$$M = Q(l_1 + c) - \frac{q(l_1 + c)^2}{2} = 80 \cdot 0.914 - \frac{29 \cdot 0.914^2}{2} = 61 \text{ kN·m}$$

Let's check the strength according to condition (77) considering formula (78):

$$R_s A_s z_s + 0.5q_{sw}c^2 = 142.2 \cdot 10^3 \cdot 325 + 0.5 \cdot 68.4 \cdot 821^2 = 46.6 + 23.05 = 69.4 \text{ kN·m} >$$

$$> M = 61 \text{ kN·m},$$

that is the strength of inclined sections as regards bending moment is provided.

As the beam has no bend-up bars and is loaded by distributed load so the strength of inclined section can be also checked according to the more simple formula (84) taking $M_0 = Ql_1 = 80\times$ $\times 10^3 \cdot 93 = 7.4 \cdot 10^6$ N·mm:

$$\sqrt{2(R_s A_s z_s - M_0)(q_{sw} + q)} = \sqrt{2(46.3 \cdot 10^6 - 7.4 \cdot 10^6)(68.4 + 29)} = 87.1 \cdot 10^3 \text{ N} = 87 \text{ kN} >$$

$$> Q = 80 \text{ kN}.$$

Example 22. *Given:* a collar beam of a multi-storey frame with diagrams of bending moments and shear forces of distributed load q = 228 kN/m according to Draft 31; heavy-weight concrete B25; cross reinforcement and longitudinal reinforcement A-III ($R_s = 365$ MPa; $R_{sw} = 290$ MPa); cross section of the support part – according to Draft 31; stirrups with diameter 10 mm, spacing s = 150 mm ($A_{sw} = 236$ mm^2).

It is required to determine the distance from the left support to the break point of the first rod of top reinforcement.

Draft 31. For the calculation example 32

Calculation. Let's determine limit bending moment which stretches support reinforcement without considering a broken rod according to the condition (19) as $A_s = 1609\,\text{mm}^2 < A_s^{'}$, that is $x < 0$:

$$M_u = R_s A_s (h_0 - a') = 365 \cdot 1609(740 - 50) = 405 \text{ kN·m}$$

According to the moment diagram we determine distance x from the support to the point of theoretical break of the first rod in compliance with the following equation:

$$M = M_{\sup} - \frac{M_{\sup} - M_{\sup}^{'}}{l}x - \frac{ql}{2}x + \frac{q}{2}x^2 = M_u,$$

So $x = \left(\frac{l}{2} + \frac{M_{\sup} - M_{\sup}^{'}}{ql}\right) - \sqrt{\left(\frac{l}{2} + \frac{M_{\sup} - M_{\sup}^{'}}{ql}\right)^2 - 2\frac{(M_{\sup} - M_u)}{q}} = \left(\frac{4.9}{2} + \frac{600 - 300}{228 \cdot 4.9}\right) -$

$$- \sqrt{\left(\frac{4.9}{2} + \frac{600 - 300}{228 \cdot 4.2}\right)^2 - \frac{2(600 - 405)}{228}} = 0.334 \text{ m.}$$

Shear force at the o\point of theoretical break is:

$$Q = Q_{\max} - qx = 620 - 228 \cdot 0.334 = 554 \text{ kN}$$

Let's determine value q_{sw}:

$$q_{sw} = \frac{R_{sw}A_{sw}}{s} = \frac{290 \cdot 236}{150} = 456 \text{ N/mm}$$

By formula (87) let's determine length w:

$$w = \frac{Q}{2q_{sw}} + 5d = \frac{544 \cdot 10^3}{2 \cdot 456} + 5 \cdot 32 = 756 \text{ mm}$$

So the distance from the support to the point of rod break can be taken equal to $x + w = 334 + 756 = 1090$ mm.

Let's determine required distance l_{an} from the point of rod break to the vertical section where it is fully used, according to Table 45:

$$l_{an} = 29d = 29 \cdot 32 = 930\,\text{mm} < 1090\,\text{mm}$$

That is the rod is to be broken at the distance 1090 mm from the support.

Example 23. *Given:* adjoining of a prefabricated reinforced concrete floor beam to the collar beam by means of cutting as it's shown on Draft 32a; heavy-weight concrete B25 ($R_b = 13$ MPa; $R_{bt} = 0.95$ by $\gamma_{b2} = 0.9$); stirrups and bend-up bars of reinforcement A-III with diameter 12 and 16 mm ($A_{sw} = 452$ mm^2; $A_{s,inc} = 804$ mm^2); section area of additional stirrups of cuttings $A_{sw1} = 402$ mm^2 (2Ø16); longitudinal reinforcement A-III according to Draft 32b; shear force at the support Q = 640 kN.

It is required to examine the strength of inclined sections.

Draft 32. To the calculation example 23

Calculation. Let's determine the strength of inclined section of the cutting as regards shear force according to Item 3.31 taking $h_0 = 370$ mm, b = 730 mm (see Draft 32), $\varphi_{b2} = 2$ (see Table 21):

$$M_b = \varphi_{b2} R_{bt} b h_0^2 = 2 \cdot 0.95 \cdot 730 \cdot 370^2 = 190 \cdot 10^6 \text{ N·mm}$$

By value c equal to the distance from the support to the first load – c = 1.5 m we have

$$Q_b = \frac{M_b}{c} = \frac{190\cdot 10^6}{1500} = 126.7\cdot 10^3 \text{ N} < Q_{b,\min} =$$
$$= \varphi_{b3} R_{bt} b h_0 = 0.6\cdot 0.95\cdot 730\cdot 370 = 154\cdot 10^3 \text{ N}$$
$$(\varphi_{b3} = 0.6 \text{ – see Table 21}),$$

So we take $Q_b = 154\cdot 10^3$ N;

$$q_{sw} = \frac{R_{sw} A_{sw}}{s} = \frac{255\cdot 452}{100} = 1152 \text{ T/mm}$$
$$c_0 = \sqrt{\frac{M_b}{q_{sw}}} = \sqrt{\frac{190\cdot 10^6}{1152}} = 406 \text{ mm} < 2h_0$$

At the same time $c_0 < c = 1.5$ m and $c_0 > h_0$.

Thereafter $Q_b + q_{sw} c_0 + R_{sw} A_{sw1} = 154\cdot 10^3 + 1152\cdot 406 + 290\cdot 402 = 738\cdot 10^3 \text{N} > Q$ = 640 kN, that is even without considering bend-up bars the strength of cutting as regards the shear force is provided.

Let's check if there are enough additional stirrups and bend-up bars according to condition (89). According to Draft 32 $\theta = 45$ degrees; $h_0 = 700 - 60 - 80/2 = 600$ mm; $h_{01} = 370$ mm;

$$R_{sw} A_{sw1} + R_{sw} A_{s,inc} \sin 45° = 290\cdot 402 + 290\cdot 804\cdot 0.707 = 281\cdot 10^3 \text{ N} > \left(1 - \frac{h_{01}}{h_0}\right) = 460\times$$
$$\times\left(1 - \frac{370}{600}\right) = 245 \text{ kN}$$

Let's check the strength of the inclined section going through the reentrant angle of cutting as regards bending moment.

The most disadvantageous value *c* is determined by formula (83) considering bend-up bars and additional stirrups in the numerator and taking $F_i = 0$ and $q = 0$:

$$c = \frac{Q - \left(R_{sw} A_{sw1} + R_{sw} A_{s,inc} \sin\theta\right)}{q_{sw}} = \frac{640\cdot 10^3 - 281\cdot 10^3}{1152} = 312 \text{ mm}$$

As longitudinal reinforcement of the short console is anchored in the support so we consider this reinforcement with total design strength; that is with $R_s = 365$ MPa.

According to Draft 32 we have $A_s = A_s^{'} = 1256 \text{mm}^2$ (4Ø20). As $A_s = A_s^{'}, x = 0$ so $z_s = h_{01} - a' = 370 -$
$- 50 = 320 \text{ mm}$

According to (79) taking $a_1 = 30$ mm we get

$$z_{s,inc} = z_s \cos\theta + (c - a_1)\sin\theta = 320\cdot 0.707 + (312 - 30)0.707 = 425 \text{ mm}$$

Let's check condition (77) taking:

$$\Sigma R_{sw} A_{sw} z_{sw} = 0.5 q_{sw} c^2 + R_{sw} A_{sw1}(c - a_1) = 0.5\cdot 1152\cdot 312^2 + 290\cdot 402(312 - 30) = 88.8\cdot 10^6 \text{ N·mm};$$
$$M = Q(a_0 + c) = 640\cdot 10^3(130 + 312) = 283\cdot 10^6 \text{ N·mm};$$
$$R_s A_s z_s + \Sigma R_{sw} A_{sw} z_{sw} + \Sigma R_{sw} A_{s,inc} z_{s,inc} = 365\cdot 1256\cdot 320 + 88.8\cdot 10^6 + 290\cdot 804\cdot 425 =$$
$$334.6\cdot 10^6 \text{ N·mm} > M = 283 \text{ kN·m},$$

that is the strength of the inclined section is provided.

Let's determine required length of longitudinal tensile reinforcement going behind the cutting end by the following formula:

$$w_0 = \frac{2\left(Q_1 - R_{sw} A_{sw1} - R_{sw} A_{s,inc} \sin\theta\right)}{q_{sw}} + a_0 + 10d =$$

$$= \frac{2\left(640\cdot 10^3 - 281\cdot 10^3\right)}{1152} + 130 + 10\cdot 20 = 953\,\text{mm} > l_{an} = 30\cdot 20 = 600 \text{ mm}$$

Let’s find out if it is necessary to install anchors for bottom reinforcement of the beam. For that let’s check inclined section located beyond the cutting and beginning at the distance $h_0 - h_{01} = 600 - 370 = 230$ mm from the end of the beam. Then $l_x = 230 - 10 = 220$ mm

The length of anchorage for bottom reinforcement is determined according to pos. 1 of Table 45, where for concrete B25 and reinforcement A-III there is $\lambda_{an} = 29$ thereafter $l_{an} = 29\cdot 40 = 1160\,\text{mm} > l_x = 220$.

Design strength of bottom reinforcement is to be decreased by means of multiplying by the coefficient $\gamma_{s5} = \frac{l_x}{l_{an}} = \frac{220}{1160} = 0.19$; that is $R_s = 365\cdot 0.19 = 69.2$ MPa.

According to Draft 32 $A_s = 5027\,\text{mm}^2$ (4Ø40)

Taking the fact into account that within the length $l_x = 220$ mm two top rods have two welded vertical rods and two bottom rods have two vertical and one horizontal welded rod, we increase the force $R_s A$ by value N_w determined by formula (82) taking $n_w = 10$, $d_w = 12$ mm, $\varphi_w = 100$ (see Table 22):

$$N_w = 0.7 n_w \varphi_w d_w^2 R_{bt} = 0.7\cdot 10\cdot 100\cdot 12^2\cdot 0.95 = 95760\,\text{N} < 0.8 R_s d_w^2 n_w = 0.8\cdot 365\cdot 12^2\cdot 10 =$$
$$= 420000 \text{ N}$$

Thereafter $R_s A_s = 69.2\cdot 5027 + 95760 = 443600\,\text{N} < 365\cdot 5027 = 1835\cdot 10^3\,\text{N}$

Taking $b = b_f^{'} = 730$ mm we determine the height of the compressed zone *x*:

$$x = \frac{R_s A_s - R_{sc} A_s^{'} + R_{sw} A_{s,inc}\cos\theta}{R_b b} = \frac{443600 - 365\cdot 1256 + 290\cdot 804\cdot 0.707}{13\cdot 730} =$$
$$= 15.8\,\text{mm} < 2a' = 2\cdot 50 = 100 \text{ mm}$$

And so $z_s = h_0 - a' = 600 - 50 = 550$ mm

The most disadvantageous value *c* is equal to:

$$c = \frac{Q}{q_{sw}} = \frac{640000}{1152} = 555\,\text{mm} < w_0 - \left(h_0 - h_{01}\right) = 953 - 230 = 723 \text{ mm}$$

that is by such value *c* the inclined section crosses longitudinal reinforcement of the short console. We take the end of inclined section at the end of the mentioned above reinforcement that is at the distance $w_0 = 953$ mm from the cutting; at the same time *c* = 723 mm. Design moment *M* in the section going through the end of inclined section is:

$$M = Q\left(a_0 + w_0\right) = 460(0.13 + 0.953) = 693\,\text{kN·m};$$

$$z_{s,inc} = z_s\cos\theta + \left(c - a_1\right)\sin\theta = 550\cdot 0.707 + (723 - 70)0.707 = 851\,\text{mm}$$

[Where $a_1 = 300 - 230 = 70$ mm (see Draft 32)]

Let’s check condition (77):

$$R_s A_s z_s + \frac{q_{sw} c^2}{2} + R_{sw} A_{s,inc} z_{s,inc} = 443600\cdot 550 + \frac{1152\cdot 723^2}{2} + 290\cdot 804\cdot 851 =$$
$$= 743.5\cdot 10^6\,\text{N·mm} > M = 693\,\text{kN·m},$$

that is the strength of the inclined section is provided and anchors for bottom reinforcement are not required.

Let’s check the strength of the short console of the cutting according to Items 3.99 and 3.100 considering Item 3.31.

Let’s check condition (207) taking $l_{\sup} = 130$ mm, $a_x = 90$ mm, $h_{01} - a' = 370 - 50 = 320$ mm (see Draft 32) thereafter

$$\sin^2\theta = \frac{(h_{01}-a')^2}{(h_{01}-a')^2+(l_{sup}+a_x)^2} = \frac{320^2}{320^2+(130+90)^2} = 0.679\,.$$

Taking $\mu_w = 0$ and replacing 0.8 by 1.0 we have $R_b b l_{sup}\sin^2\theta = 13\cdot 730\cdot 130\cdot 0.679 =$ $= 838\cdot 10^3\,\text{N} < 3.5R_{bt}bh_{01} = 3.5\cdot 0.95\cdot 730\cdot 370 = 898\cdot 10^3\,\text{N}$; that is the right part of condition (207) is equal to 838 kN.

As Q = 640 kN < 838 kN so the strength of the compressed zone is provided.

Let's check condition (208) taking $l_1 = l_{sup} + a_x = 220\,\text{mm}$, $h_0 = 320\,\text{mm}$, $A_s = 1256\,\text{mm}^2$ (4Ø20):

$$Q\frac{l_1}{h_0} = 640\cdot 10^3\frac{220}{320} = 440\cdot 10^3\,\text{N} < R_sA_s = 365\cdot 1256 = 458\cdot 10^3\,\text{N},$$

that is there is enough longitudinal reinforcement in the short console.

Eccentric Compressed Members

GENERAL POSITIONS

3.50. (1.21) During calculation of eccentric pressed reinforced concrete members it is necessary to consider accidental eccentricity e_a resulting from not considered in the calculation factors. Anyway eccentricity e_a is taken no less than:

- 1/600 of the member length or of the distance between its fixed sections;
- 1/30 of the section height;
- 10 mm (for structures formed of prefabricated members if there are no any other experiment justified values e_a).

For members of statically undeterminable structures (including columns of frame work buildings) longitudinal force eccentricity value relating to the center of gravity of the given section e_0 is taken equal to the eccentricity calculated according to static calculation of the structure, but no less than e_a.

In members of statically determinable structures (for example formwork poles, electric power line supports) eccentricity e_0 is calculated as a sum of eccentricities – determined according to static calculation and accidental one.

3.51. Calculation of eccentric compressed members is to be made considering the deflection influence in the longitudinal force eccentricity plane (in the bending plane) and in the normal to it plane. In the last case it is assumed that longitudinal force is applied with the eccentricity e_0 equal to the accidental eccentricity e_a (see Item 3.50).

The deflection influence is considered according to Items 3.54 and 3.55.

It is possible not to make calculation as regards the bending plane if the member elasticity l_0/i (for rectangular section $-l_0/h$) in the bending plane is more than the elasticity in the plane normal to the bending plane.

If there are design eccentricities in two directions exceeding accidental eccentricities e_a so it is necessary to make calculation as regards the skew eccentric compression (see Items 3.73 – 3.75).

3.52. For frequent types of compressed members (rectangular section; double-T section with symmetrical reinforcement; round and ring section with reinforcement distributed along the perimeter) the normal sections strength calculation is made according to Items 3.61-3.75).

For other kinds of sections and by unspecified location of longitudinal reinforcement the calculation of normal sections is made by formulas of the general case of the normal section calculation of eccentric compressed member according to Item 3.76. During calculation by means of computers it is recommended to follow the instructions of Item 3.76.

If condition $A_s^{'} > 0.02A_b$ is met, so it is necessary to consider decrease of actual concrete area by value $A_s^{'}$ in formulas of Items 3.61-3.76.

3.53. Strength of inclined sections of eccentric compressed members is calculated similar to calculation of bending moments in compliance with Items 3.28-3.49. At the same time value M_b is determined by the following formula:

$$M_b = \varphi_{b2}\left(1+\varphi_f+\varphi_n\right)R_{bt}bh_0^2 \quad (90)$$

Where $\varphi_n = 0.1\frac{N}{R_{bt}bh_0}$ but no more than 0.5; value $Q_{b,\min}$ is taken equal to $\varphi_{b3}\left(1+\varphi_f+\varphi_n\right)R_{bt}bh_0$ and in formulas (72)–(76) coefficient φ_{b4} is replaced by $\varphi_{b4}\left(1+\varphi_n\right)$

Total coefficient $1+\varphi_f+\varphi_n$ is taken no more than 1.5.

Longitudinal forces influence is not considered if they cause bending moments which have the same signs like moments caused by lateral load. For eccentric compressed members of statically undeterminable structures for which longitudinal force is located in the center of gravity of the section it is possible always to consider longitudinal forces influence.

If there is no lateral load within the span of eccentric compressed member so it is possible not to calculate the strength of inclined sections if there are no normal cracks [that is if condition (233) is met with replacing of $R_{bt,ser}$ by R_{bt}].

Member deflection influence

3.54.(3.24, 3.6) During calculation of eccentric compressed members it is necessary to consider the deflection influence on the bearing capacity as a rule by means of calculation of the structure according to deformed scheme considering non-elastic concrete and reinforcement deformations as well as crack formation.

It is possible to make calculation of the structure according to non-deformed scheme considering the member deflection influence by means of multiplying of the eccentricity e_0 by the coefficient η determined by the following formula:

$$\eta = \frac{1}{1 - \frac{N}{N_{cr}}} \tag{91}$$

Where N_{cr} – relative critical force determined by the following formulas:
for members of any section:

$$N_{cr} = \frac{6.4E_b}{l_0^2}\left[\frac{I}{\varphi_l}\left(\frac{0.11}{0.1+\delta_e}+0.1\right)+\alpha I_s\right] \tag{92}$$

for members of rectangular section:

$$N_{cr} = \frac{1.6E_b bh}{(l_0/h)}\left[\frac{\frac{0.11}{0.1+\delta_e}+0.1}{3\varphi_1}+\mu\alpha\left(\frac{h_0-a'}{h}\right)^2\right] \tag{93}$$

In formulas (92) and (93):

I, I_s – Inertia moments of concrete section and section of all reinforcement relating to the center of gravity of concrete section;

φ_l – Coefficient considering long-term action of the load on the member deflection in the limit state and equal to:

$$\varphi_l = 1 + \beta\frac{M_{1l}}{M_1}, \tag{94}$$

But no more than $1+\beta$ (here β – see Table 16);

M_1, M_{1l} – are moments of external forces relating to the axis parallel to the line bounding the compressed zone and going through the center of the most stretched or the least compressed reinforcement rod (by the whole compressed zone) and caused by the total load and load and by dead loads and long-term loads. For members calculated according to Items 3.61, 3.62, 3.65–3.68 it is possible to determine M_1 and M_{1l} relating to the axis going through the center of gravity of all reinforcement *S*. If bending moments (or eccentricities) caused by total loads or by the sum of dead loads and long-term loads have different sighs so by absolute value of the total load eccentricity $e_0 > 0.1h$ it is taken $\varphi_l = 1.0$; if this condition is met so value φ_l is to be taken equal to $\varphi_l = \varphi_{l1} + 10(1-\varphi_{l1})\times$ $\times e_0/h$ where φ_{l1} is determined by formula (94), taking M_1 equal to the product of the longitudinal force *N* caused by the total load by the distance from the center of gravity of the section to the axis going through the center of the most stretched (the least compressed) by dead loads and by long-term loads reinforcement rod;

δ_e – Coefficient taken equal to e_0/h but no less than

$$\delta_{e,\min} = 0.5 - 0.01\frac{l_0}{h} - 0.01R_b \tag{95}$$

(Here R_b is given in mega-Pascal, it is possible to take $\gamma_{b2} = 1.0$; for round and ring sections value *h* is replaced by *D*);

l_0 – is taken according to Item 3.55;

$$\mu\alpha = \frac{A_s + A_s^{'}}{bh}\frac{E_s}{E_b} \quad (96)$$

During calculation of rectangular sections with reinforcement located along the height of the section according to Item 3.63 in value $A_s + A_s^{'}$ it is not considered 2/3 of reinforcement located at the surfaces parallel to the bending plane ($2A_{sl}$), and value $\frac{h_0 - a'}{h}$ in formula (93) is taken equal to $1 - 2\delta_1$.

For members made of fine concrete of group Б it is necessary to insert numbers 5, 6 and 1, 4 instead of 6, 4 and 1, 6 in formulas (92) and (93).

Eccentricity e_0 used in the present Item can be determined relating to the center of gravity of concrete section.

By elasticity of the element $l_0 / i < 14$ (for rectangular sections $l_0 / h < 4$) it is taken $\eta = 1$.

By of the element $14 \le l_0 / i < 35$ ($4 \le l_0 / h < 10$) and by $\mu = \frac{A_s + A_s^{'}}{A} \le 0.025$ it is possible to take:

For rectangular sections:

$$N_{cr} = 0.15\frac{E_b A}{(l_0 / h)^2},$$

For other sections:

$$N_{cr} = \frac{2E_b I}{l_0^2}.$$

By $N > N_{cr}$ it is necessary to increase the section dimensions.

By design eccentricities in two directions coefficient η can be determined for each direction and multiplied by the corresponding eccentricity.

3.55.(3.25) It is recommended to determine design length l_0 of eccentric compressed reinforced concrete members as for frame structure members considering its deformed state by the most disadvantageous for the present element load distribution considering inelastic deformations of materials and cracks.

For members of frequent structures it is possible to take l_0 equal to:

a) for columns of multistory buildings by no less than two spans and connections of collar-beams to columns calculated as fixed connections by:
   - Prefabricated floor structures........ $l_0 = H$
   - Cast-in-situ floor structures .... $l_0 = 0.7H$

   [where H is the height of the storey (the distance between the centers of joints)];
b) for columns of one-storey buildings with hinge connection of bearing structures of the roof hard in their plane (able to transfer horizontal forces) as well as for trestles – according to Table 23;

c) for trusses and arches elements – according to table 24.

Table 23 (32)

Buildings and columns characteristics					Design length l_0 for columns of one-storey buildings if they are calculated in: the plane of the cross frame or plane perpendicular to the trestle axis	the plane perpendicular to the cross frame or parallel to the trestle axis: if there are bracings in the plane of longitudinal row of columns or anchor supports	the plane perpendicular to the cross frame or parallel to the trestle axis: if there are no bracings in the plane of longitudinal row of columns or anchor supports
Buildings	With crane bridges	Considering the load of canes	Bottom part of columns by crane runaway beams	Semi-infinite	1.5 H_1	0.8 H_1	1.2 H_1
				simply supported	1.2 H_1	0.8 H_1	0.8 H_1
			Top part of columns by crane runaway beams	Semi-infinite beam	2.0 H_2	1.5 H_2	2.0 H_2
				simply supported beam	2.0 H_2	1.5 H_2	1.5 H_2
		Without considering the load of cranes	Bottom part of building beams	one-span beam	1.5 H	0.8 H_1	1.2 H
				multi-span beam	1.2 H	0.8 H_1	1.2 H
			Top part of columns by crane runaway beams	Semi-infinite beam	2.5 H_2	1.5 H_2	2.0 H_2
				simply supported beam	2.0 H_2	1.5 H_2	1.5 H_2
	Without crane bridges	Tapered columns	Bottom part of building beams	one-span beam	1.5 H	0.8 H	1.2 H
				multi-span beam	1.2 H	0.8 H	1.2 H
			Top part of columns		2.5 H_2	2.0 H_2	2.5 H_2
		Columns of constant section of buildings		one-span beam	1.5 H	0.8 H	1.2 H
				multi-span beam	1.2 H	0.8 H	1.2 H
Trestles	Crane trestle		by crane runaway beams	Semi-infinite	2.0 H_1	0.8 H_1	1.5 H_1
				simply supported	1.5 H_1	0.8 H_1	H_1

	Trestle for pipelines	By connection of columns to a span structure	Hinge connection	2.0 H	H	2.0 H
			Fixed connection	1.5 H	0.7 H	1.5 H

Symbols of Table 23

H – total height of the column from the top of the foundation to the horizontal structure in the corresponding plane;

H_1 – the height of the bottom part of the column from the top of the foundation to the bottom of the crane runaway beam;

H_2 – the height of the top part of the column from the step of the column to a horizontal structure of the corresponding plane.

Note. If there are bracings up to the top of columns in buildings with bridge cranes so design length of the bottom part of columns in the plane of the axis of longitudinal column row is taken H_2.

Table 24 (33)

Elements	Design length l_0 of trusses and arches members
1. Trusses elements:	
a) top chord of truss during calculation:	
- in the truss plane:	
by $e_0 < 1/8h_1$	0.9l
by $e_0 \geq 1/8h_1$	0.8l
- out of the truss plane	
for the part under the skylight (if the width of the skylight is 12 m and more);	0.8l
in other cases	0.9l
b) inclined braces and poles during calculation:	
- in the truss plane	0.8l
- out of the truss plane	
by $b_1/b_2 < 1.5$	0.9l
by $b_1/b_2 \geq 1.5$	0.8l
2. Arches	
a) during calculation in the arch plane:	
- triple-hinged	0.580L
- double-hinged	0.540L
- hingeless	0.365L
b) during calculation out of the arch plane (any)	L

Symbols of Table 24:

l – the length of the element between centers of adjoining connections; for top chord of the truss during calculation out of the arch plane this is the distance between its fixing points;

L – the length of the arch along its geometrical axis; during calculation out of the arch plane this is the length of the arch between its fixing points according to the arch plane;

h_1 – section height of the top chord;

b_1, b_2 – the width of the section of the top chord and of the pole (inclined brace) of the truss.

3.56. Columns deflection influence of multistory framework buildings are to be considered taking moments M in support sections of columns equal to:

$$M = M_v \eta_v + M_h \eta_h + M_t , \qquad (97)$$

Where M_v – moment caused by vertical loads on the floors;

η_v – Coefficient equal to one and in embedment into foundation it is determined by formula (91) by $l_0 = 0.??$ (*H* is the height of the storey) and by considering only vertical loads;

M_h – Moment caused by horizontal loads (wind loads and seismic loads);

η_h – is coefficient η determined according to Items 3.54 and 3.55 considering all loads;

M_t – Moments caused by forced horizontal displacements (for example temperature deformation of floors, displacement of hard bracing diaphragms).

Moments caused by all loads for sections in the middle third of the column length are multiplied by the coefficient determined according to Items 3.54 and 3.55 and moments in other sections are determine by linear interpolation.
Values of moments in support columns sections determined by formula (97) must be considered by determination of moments in adjoining to the column elements (foundations, collar-beams with fixed connections).

Confinement reinforcement influence

3.57.(3.22) Calculation of solid-section elements of heavy-weight and fine concrete with confinement reinforcement in the shape of welded meshes, spiral or ring reinforcement (Draft 33) must be determined according to Items 3.61-3.68, 3.71-3.76 inserting into the calculation only a part of concrete section A_{ef} bounded by axes of end rods of a mesh or of a spiral and replacing R_b by changed strength of concrete $R_{b,red}$ and calculating the compressed concrete zone characteristics ω considering the influence of confinement by formula (104).

Draft 33. Compressed elements with confinement reinforcement.
a – in the shape of welded meshes; *b* – in the shape of spiral reinforcement.

The deflection influence of the element with confinement reinforcement on the eccentricity of longitudinal force is considered according to Item 3.58.

Elasticity l_0 / i_{ef} of elements with confinement reinforcement must be no more than:

- 55 – by confinement reinforcement by means of meshes (for rectangular sections $l_0 / h_{ef} \leq 16$);
- 35 – By confinement reinforcement by means of spirals (for round sections $l_0 / d_{ef} \leq 9$) where i_{ef}, h_{ef}, d_{ef} are correspondingly inertia radius, height and diameter of the section part inserted into the calculation.

Value $R_{b,red}$ is determined by the following formulas:

a) by reinforcement by welded cross meshes

$$R_{b,red} = R_b + \varphi \mu_{xy} R_{s,xy}, \tag{98}$$

Where $R_{s,xy}$ – design strength of reinforcement meshes

$$\mu_{xy} = \frac{n_x A_{sx} l_x + n_y A_{sy} l_y}{A_{ef} s} \tag{99}$$

Here n_x, A_{sx}, l_x are: quantity of rods, cross section area and length of a mesh rod (calculating in axes of end rods) in one direction;

n_y, A_{sy}, l_y – The same in another direction;

A_{ef} is section area of reinforcement within the meshes contours;

s is distance between meshes;

φ is efficiency factor of confinement reinforcement determined by the following formula:

$$\varphi = \frac{1}{0.23 + \psi} \qquad (100)$$

$$\psi = \frac{\mu_{xy} R_{s,xy}}{R_b + 10} \qquad (101)$$

$R_{s,xy}, R_b$ Are given in MPa

For members made of fine concrete efficiency factor φ must be taken no more than 1.

b) By reinforcement of spiral or ring reinforcement

$$R_{b,red} = R_b + 2\mu_{cir} R_{s,cir} \left(1 - \frac{7.5 e_0}{d_{ef}}\right) \qquad (102)$$

Where $R_{s,cir}$ – design strength of the spiral;

μ_{cir} –reinforcement factor equal to:

$$\mu_{cir} = \frac{4 A_{s,cir}}{d_{ef} s} \qquad (103)$$

Here $A_{s,cir}$ – cross-section area of spiral reinforcement;

d_{ef} – Section diameter inside of the spiral;

s – Is spiral spacing

e_0 – Eccentricity of longitudinal force application (without considering deflection influence).

Reinforcement coefficients values which are determined by formulas (99) and (103) for members of fine concrete must be no more than 0.04.

During determination of limit value of relative height of compressed zone for sections with confinement reinforcement it is necessary to insert the following value into formula (14):

$$\omega = \alpha - 0.008 R_b + \delta_2 \le 0.9 \qquad (104)$$

Where α – coefficient taken according to Item 3.14 instructions;

δ_2 – Coefficient equal to 10μ but taken no more than 0.15 [here μ is reinforcement coefficient μ_{xy} or μ_{cir} determined by formulas (99) and (103) correspondingly for meshes and spirals].

Confinement reinforcement is considered in the calculation on conditions that bearing capacity of the element determined according to the instructions of the present Item (using A_{ef} and $R_{b,red}$) is more than its bearing capacity determined according to its total section A and to the value of concrete design strength R_b without considering confinement reinforcement. Besides confinement reinforcement must meet the constructive requirements of Items 5.78-5.80.

3.58.(3.22). During calculation of elements with confinement reinforcement according to undeformed scheme member deflection influence on the longitudinal force eccentricity is considered according to Items 3.54-3.56. At the same time value N_{cr} determined by formula (92) or (93) is to be multiplied by coefficient $\varphi_1 = 0.25 + 0.05 l_0 / c_{ef} \le 1.0$ and value $\delta_{e,\min}$ is determined by formula $\delta_{e,\min} = 0.5 + 0.01 l_0 / c_{ef} (1.0 - 0.1 l_0 / c_{ef}) - 0.01 R_b$ where c_{ef} is the height or diameter of considered part of the section.

Besides during determination of N_{cr} dimensions of the section are taken according to the considered part of the section.

3.59.(3.22). In elements made of heavy-weight concrete with confinement reinforcement at the shape of meshes it is recommended to use longitudinal high-strength reinforcement A-V and A-VI using its increased strength equal to:

$$R_{sc,red} = R_{sc} \frac{1 + \delta_3 \lambda_1}{1 + \delta_3 \lambda_2} \le R_s \qquad (105)$$

Where λ_1, λ_2 – see Table 25;

R_{sc}, R_s

$\delta_3 = 1.6 \theta \psi$;

Here $$\theta = 0.8 + 25 \frac{A_{s,tot}}{A_{ef}} \left(1 - \frac{R_b}{100}\right)$$

(106)

but no more than 1.0 and no more than 1.6;

ψ, A_{ef} – see Item 3.57;

$A_{s,tot}$ – Section area of all longitudinal high-strength reinforcement;

R_b – In MPa.

Table 25

Reinforcement class	λ_1, λ_2 and R_{sc} in MPa by coefficient γ_{b2} (see Item 3.1) equal to						R_s MPa	$R_{s,ser}$ MPa
	0.9			1.0 or 1.1				
	λ_1	λ_2	R_{sc}	λ_1	λ_2	R_{sc}		
A-V	1.25	0.53	500	2.78	1.03	400	680	785
A-VI	2.04	0.77	500	3.88	1.25	400	815	980

$$\lambda_1 = \left[\left(\frac{R_s}{R_{sc}}\right)^2 - 1\right] \frac{1000}{R_s}; \ \lambda_2 = \left(\frac{R_s}{R_{sc}} - 1\right) \frac{1000}{R_s}.$$

Value $\sigma_{sc,u}$ in formulas (14) and (155) is taken equal to $\sigma_{sc,u} = 380 + 1000 \delta_3$ but no more than 1200 MPa.

The mentioned elements of rectangular section with reinforcement concentrated at the most or the least compressed surfaces are calculated according to Items 3.65 and 3.61 if the height of compressed zone *x* determined by formula (107a) or (110a) is more than the limit value $\xi_R h_0$ by replacement R_s by $0.8 R_s$ in the calculation formula. Otherwise the calculation is made according to Item 3.41 of the "Manual for design of prestressed reinforced concrete structures of heavy-weight and light-weight concrete" taking $\sigma_{sp} = 0$. In this case use of confinement reinforcement and high-strength reinforcement is inefficient.

3.60.(3.23). During calculation of eccentric compressed elements with confinement reinforcement along with calculation as regards the strength according to Item 3.57 instructions it is necessary to make calculations providing crack resistance of protection cover of concrete.

The calculation is made in compliance with instructions of Items 3.61-3.68 and 3.71-3.76 according to performance values of design loads ($\gamma_f = 1.0$) considering total area of concrete section and taking design resistances $R_{b,ser}$ and $R_{s,ser}$ for limit states of the second group and reinforcement design resistance against compression equal to value $R_{s,ser}$ but no more than 400 MPa.

During determination of value ξ_R in formulas (14) and (155) it is taken $\sigma_{sc,u} = 400$ MPa and in formula (15) coefficient 0.008 is replaced by 0.006.

When considering the elasticity influence it is necessary to use Item 3.54 instructions determining value $\delta_{e,\min}$ by formula (95) replacing $0.010R_b$ by $0.008R_{b,ser}$.

Calculation of members of symmetrical section by location of the longitudinal force in the symmetry plane.

RECTANGULAR SECTIONS WITH SYMMETRICAL REINFORCEMENT

3.61. Strength examination of rectangular sections with symmetrical reinforcement concentrated at the most compressed and tensile (the least compressed) surfaces of the element is made in the following manner according to the compressed zone height *x*:

$$x = \frac{N}{R_b b}: \tag{107}$$

a) by $x \le \xi_R h_0$ (Draft 34) – according to the following condition

$$N_e \le R_b bx(h_0 - 0.5x) + R_{sc} A_s^{'}(h_0 - a'); \tag{108}$$

Draft 34. Forces scheme in rectangular cross-section of eccentric compressed element.

b) By $x > \xi_R h_0$ – according to condition (108) taking the height of compressed zone equal to $x = \xi h_0$ where ξ is determined by following formulas:
   - For elements of concrete of class B30 and lower:

$$\xi = \frac{\alpha_n(1-\xi_R) + 2\alpha_s\xi_R}{1-\xi_R + 2\alpha_s} \tag{109}$$

   - For members of concrete of class more than B30:

$$\xi = -\frac{\alpha_s + \psi_c\alpha_s - \alpha_n}{2} + \sqrt{\left(\frac{\alpha_s + \psi_c\alpha_s + \alpha_n}{2}\right)^2 + \psi_c\alpha_s\omega} \tag{110}$$

In formulas (109) and (110):

$$\alpha_n = \frac{N}{R_b b h_0}; \ \alpha_s = \frac{R_s A_s}{R_b b h_0}; \ \psi_c = \frac{\sigma_{sc,u}}{R_s\left(1 - \frac{\omega}{1.1}\right)}:$$

ξ_R, ψ_s, ω – See Table 18 and 19.

Value e is determined by the following formula:

$$e = e_0 + \frac{h_0 - a'}{2} \tag{111}$$

At the same time eccentricity of longitudinal force e_0 relating to the center of gravity of the section is determined considering the deflection of the element according to Items 3.54-3.56.

Notes. 1. If the height of compressed zone which is determined considering a half of compressed reinforcement is $x = \frac{N + R_s A_s / 2}{R_b b} < a'$, so design bearing capacity of the section can be increased using condition (108) by $A_s^{'} = 0$ and $x = \frac{N + R_s A_s}{R_b b}$.

2. Formula (110) can be also used during determination of elements of concrete B30 and lower class.

3.62. Required quantity of symmetrical reinforcement is determined in the following manner according to relative value of longitudinal force $\alpha_n = \frac{N}{R_b b h_0}$:

a) by $\alpha_n \le \xi_R$

$$A = A_s^{'} = \frac{R_b b h_0}{R_s} \frac{\alpha_{m1} - \alpha_n (1 - \alpha_n / 2)}{1 - \delta} \tag{112}$$

b) by $\alpha_n > \xi_R$

$$A_s = A_s^{'} = \frac{R_b b h_0}{R_s} \frac{\alpha_{m1} - \xi (1 - \xi / 2)}{1 - \delta} \tag{113}$$

Where ξ – relative height of compressed zone determined by formula (109) or (110).

Value α_s in formula (109) can be determined by the following formula:

$$\alpha_s = \frac{\alpha_{m1} - \alpha_n (1 - \alpha_n / 2)}{1 - \delta} \tag{114}$$

And in formula (110) it's determined by formula (114) replacing α_n by $(\alpha_n + \xi_R)/2$.

In formulas (112)–(114):

$$\alpha_{m1} = \frac{N_e}{R_b b h_0^2}; \ \delta = \frac{a'}{h_0}.$$

Value e is determined by formula (111).

If value a' does not exceed $0.15 h_0$ so required quantity of reinforcement can be determined by the diagram of Draft 35 using the following formula:

$$A_s = A_s^{'} = \alpha_s \frac{R_b b h_0}{R_s},$$

Where α_s is determined according to the diagram of Draft 35 in relation to the following values:

$$\alpha_m = \frac{M}{R_b b h_0^2}; \ \alpha_n = \frac{N}{R_b b h_0}$$

At the same time value of moment M relating to the center of gravity is determined considering the element deflection in compliance with Items 3.54-3.56.

Draft 35. Bearing capacity diagrams of eccentric compressed elements of rectangular section with symmetrical reinforcement

$$\alpha_n = \frac{N}{R_b b h_0};\ \alpha_m = \frac{M}{R_b b h_0^2};\ \alpha_s = \frac{R_s A_s}{R_b b h_0}$$

By static calculation as regards the undeformed scheme and by using the coefficient $\eta > 1$ reinforcement is chosen according to the mentioned formulas and to the diagram of Draft 35 in general case by means of step-by-step approximation.

For members of heavy-weight concrete of class B15-B50 as well as if light-weight concrete B10-B40 by average density grade no less than D1800 by $\lambda = l_0 / h \leq 25$ and by a' no more than $0.15h_0$ reinforcement can be chosen without step-by-step approximation by means of diagrams of Annex 3, at the same time there are used values *M* without considering coefficient η.

3.63. If there is reinforcement located along the height of the section so calculation of eccentric compressed elements can be made by formulas (117) and (118) considering all reinforcement as evenly distributed along the rods centers of gravity lines (Draft 36). At the same time section area of reinforcement A_{sl}, located at one of the surfaces parallel to the bending plane taken equal to:

$$A_{sl} = A_{s1,l}(n_l + 1) \tag{115}$$

Where $A_{s1,l}$ is section area of one intermediate rod; by different diameters it is taken average section area of the rod;

n_l – The number of intermediate rods.

Draft 36. The scheme taken by calculation of the eccentric compressed element of rectangular section with reinforcement located along the height of the section

Section area of reinforcement A_{st} located at one of the surfaces perpendicular to the plane of bending is:

$$A_{st} = \frac{A_{s,tot}}{2} - A_{sl}, \tag{116}$$

Where $A_{s,tot}$ – section of total reinforcement in the section of element.

The section strength is to be examined according to the relative height of compressed zone $\xi = \frac{x}{h} = \frac{\alpha_{n1} + \alpha_{sl}}{1 + 2\alpha_{sl}/\omega}$:

a) By $\xi \leq \xi_R$ section strength is checked according to the following condition:

$$Ne_0 \leq R_b b h^2 \left[0.5\xi(1-\xi) + \alpha_{sl}(\xi_1 - \delta_1)(1 - \xi_1 - \delta_1) - 0.05\alpha_{sl}\xi_1^2 + \alpha_{st}(1 - 2\delta_1)\right], \tag{117}$$

Where $\xi_1 = \frac{\xi}{\omega}$; $\alpha_{n1} = \frac{N}{R_b b h}$;

$$\alpha_{sl} = \frac{R_s A_{sl}}{R_b b h (0.5 - \delta_1)};\ \alpha_{st} = \frac{R_s A_{st}}{R_b b h};\ \delta_1 = \frac{a_1}{h} \text{ (see Draft 36)}$$

b) By $\xi > \xi_R$ section strength is checked according to the following condition:

$$Ne_0 \le R_{bt} bh^2 \alpha_m R \frac{\alpha_{na} - \alpha_{n1}}{\alpha_{na} - \alpha_{nR}}, \tag{118}$$

Where $\alpha_{na} = 1 + \frac{R_s A_{s,tot}}{R_b bh}$ – relative value of longitudinal force by even compression of the whole section;

α_{mR}, α_{nR} – Relative values of a bending moment and of longitudinal force by compression zone height $\xi_R h$:

$$\alpha_{mR} = 0.5\xi_R(1-\xi_R) + \alpha_{sl}(\xi_{1R} - \delta_1)(1 - \xi_{1R} - \delta_1) - 0.05\alpha_{sl}\xi_{1R}^2 + \alpha_{st}(1 - 2\delta_1);$$

$$\alpha_{nR} = \xi_R + \alpha_{sl}(2\xi_{1R} - 1);$$

$$\xi_{1R} = \frac{\xi_R}{\omega}; \ \xi_R, \omega \text{ – see table 18 and 19.}$$

Eccentricity of longitudinal force e_0 is determined considering the element deflection according to Items 3.54-3.56.

Note. By location of reinforcement within end quarters of height $h - 2a_1$ (see Draft 36) the calculation is made in compliance with Items 3.61 and 3.62 considering reinforcement *S* and *S'* as concentrated along their centers of gravity.

3.64. Calculation of compressed elements made of heavy-weight concrete B15-B40 or of light-weight concrete B12.5-B30 and by average density grade no less than D1800 as regards longitudinal force applied with the eccentricity taken according to item 3.50 equal to accidental eccentricity $e_a = h/30$ by $l_0 \le 20h$ can be made according to the following condition:

$$N \le \varphi(R_b A + R_{sc} A_{s,tot}), \tag{119}$$

Where φ is the coefficient determined by the following formula:

$$\varphi = \varphi_b + 2(\varphi_{sb} - \varphi_b)\alpha_s \tag{120}$$

But it's taken no more than φ_{sb}

Here φ_b, φ_{sb} are coefficients taken according to Tables 26 and 27;

$$\alpha_s = \frac{R_s A_{s,tot}}{R_b A};$$

$A_{s,tot}$ – See Item 3.63;

By $\alpha_s > 0.5$ it is possible to take $\varphi = \varphi_{sb}$ without using formula (120).

Table 26

Concrete	$\frac{N_l}{N}$	Coefficient φ_b by l_0/h							
		6	8	10	12	14	16	18	20
Heavy-weight	0	0.93	0.92	0.91	0.90	0.89	0.88	0.86	0.84
	0.5	0.92	0.91	0.90	0.89	0.86	0.82	0.78	0.72
	1.0	0.92	0.91	0.89	0.86	0.82	0.76	0.69	0.61

Light-weight	0 0.5 1.0	0.92 0.92 0.91	0.91 0.90 0.90	0.90 0.88 0.86	0.88 0.84 0.80	0.86 0.79 0.71	0.82 0.72 0.62	0.77 0.64 0.54	0.72 0.55 0.45

Table 27

Concrete	$\frac{N_l}{N}$	Coefficient φ_b by l_0/h							
		6	8	10	12	14	16	18	20
A. By $a = a' < 0.15h$ and if there are no intermediate rods (see a sketch) or by section area of these rods less than $A_{s,tot}/3$									
Heavy-weight	0 0.5 1.0	0.93 0.92 0.92	0.92 0.92 0.91	0.91 0.91 0.90	0.90 0.89 0.89	0.89 0.88 0.87	0.88 0.86 0.84	0.86 0.83 0.79	0.84 0.79 0.74
Light-weight	0. 0.5 1.0	0.92 0.92 0.92	0.92 0.91 0.91	0.91 0.90 0.90	0.89 0.88 0.88	0.88 0.86 0.85	0.85 0.83 0.80	0.82 0.77 0.74	0.77 0.71 0.67
B. By $0.25h > a = a' \geq 0.15h$ or by section are of no intermediate rods (see a sketch) is equal or more than $A_{s,tot}/3$ independently on value *a*									
Heavy-weight	0 0.5 1.0	0.92 0.92 0.92	0.92 0.91 0.91	0.91 0.90 0.89	0.89 0.88 0.86	0.87 0.85 0.82	0.85 0.81 0.77	0.82 0.76 0.70	0.79 0.71 0.63
Light-weight	0. 0.5 1.0	0.92 0.92 0.91	0.91 0.91 0.90	0.90 0.89 0.88	0.88 0.86 0.84	0.85 0.81 0.76	0.81 0.73 0.68	0.76 0.65 0.60	0.69 0.57 0.52

Symbols in Table 26 and 27:

N_l – Longitudinal force of dead loads and long-term loads:

N – Longitudinal force of all loads

Sketch
1 – Considered plane*;*
2 – Intermediate rods.

RECTANGULAR SECTIONS WITN ASYMMETRICAL REINFORCEMENT

3.65. The strength of rectangular sections with asymmetrical reinforcement concentrated at the most compressed and tensile (the least compressed) surfaces of the element is calculated according to Item 3.61, at the same time formulas (107), (109) and (110) have the following form:

$$x = \frac{N + R_s A_s - R_{sc} A_s'}{R_b b}; \quad (107a)$$

$$\xi = \frac{\alpha_n (1 - \xi_R) + (\alpha_s + \alpha_s')\xi_R + (\alpha_s - \alpha_s')}{1 - \xi_R + 2\alpha_s}; \quad (109a)$$

$$\xi = -\frac{\alpha_s' + \psi_c \alpha_s - \alpha_n}{2} + \sqrt{\left(\frac{\alpha_s' + \psi_c \alpha_s - \alpha_n}{2}\right)^2 + \psi_c \alpha_s \omega}; \quad (110a)$$

Where $\alpha_s' = \frac{R_{sc} A_s'}{R_b b h_0}$

3.66. Areas of sections of compressed and tensile reinforcement corresponding to minimum of their sum is determined by following formulas:
For elements made of concrete B30 and lower:

$$A_s^{'} = \frac{Ne - 0.4 R_b b h_0^2}{R_{sc}(h_0 - a')} \geq 0; \qquad (121)$$

$$A_s = \frac{0.55 R_b b h_0 - N}{R_s} + A_s^{'}; \qquad (122)$$

For elements of concrete more than B30:

$$A_s^{'} = \frac{Ne - \alpha_R R_b b h_0^2}{R_{sc}(h_0 - a')} \geq 0; \qquad (123)$$

$$A_s = \frac{\xi_R R_b b h_0 - N}{R_s} + A_s^{'} \qquad (124)$$

Where α_R, ξ_R are determined according to Tables 18 and 19 and are taken no more than 0.4 and 0.55.

By negative value A_s determined by formula (122) and (124) it is taken minimum section area of reinforcement *S* according to constructive requirements but no less than the following value:

$$A_{s,\min} = \frac{N(h_0 - a' - e) - R_b bh(h/2 - a')}{R_{sc}(h_0 - a')}; \qquad (125)$$

And the section area of reinforcement *S'* is determined in the following manner:

- by negative value of $A_{s,\min}$ – by the following formula:

$$A_s^{'} = \frac{(N - R_b ba') - \sqrt{(N - R_b ba')^2 - N(N - 2R_b bh_0 + 2R_b be)}}{R_{sc}}, \qquad (126)$$

- by positive value of $A_{s,\min}$ – by the following formula:

$$A_s^{'} = \frac{N - R_b bh}{R_{sc}} - A_{s,\min}; \qquad (127)$$

If accepted section area of compressed reinforcement $A_{s,fact}^{'}$ is more than the value calculated by formula (121) or (123) (for example by negative value $A_s^{'}$) so section area of tensile reinforcement can be decreased according to the following formula:

$$A_s = \frac{\xi R_b b h_0 - N + R_{sc} A_{s,fact}^{'}}{R_s}; \qquad (128)$$

Where ξ is determined according to table 20 in compliance with the following value:

$$\alpha_m = \frac{Ne - R_{sc} A_{s,fact}^{'}(h_0 - a')}{R_b b h_0^2}; \qquad (129)$$

If there is no compressed reinforcement or it's not considered in the calculation so section area of tensile reinforcement is always determined by formula (128) but at the same time it is necessary to meet the requirement $\alpha_m < \alpha_R$.

I-SECTIONS WITH SYMMETRICAL REINFORCEMENT

3.67. The strength of I-sections with symmetrical reinforcement concentrated in flanges (see Draft 37) is made in the following manner.

Draft 37. Loads scheme in the cross-I-section of eccentric compressed element

If the following condition is met (that is the border of compressed zone lies in the flange) so the calculation is made as for a rectangular section $b_f^{'}$ wide in compliance with Item 3.61:

$$N \le R_b b_f^{'} h_f^{'} \qquad (130)$$

If condition (130) is not met (that is the border of compressed zone goes in the rib) so the calculation is made according to the compressed zone height $x = \dfrac{N - R_b A_{ov}}{R_b b}$:

a) by $x \le \xi_R h_0$ section area is examined according to the following condition:

$$Ne \le R_b bx(h_0 - x/2) + R_b A_{ov}\left(h_0 - h_f^{'}/2\right) + R_{sc} A_s^{'}(h_0 - a') \qquad (131)$$

b) by $x > \xi_R h_0$ section area is examined according to condition (131) determining the height of compressed zone by the following formula:

$$x = h_0\left[-\frac{\alpha_s + \psi_c\alpha_s + \alpha_{ov} - \alpha_n}{2} + \sqrt{\left(\frac{\alpha_s + \psi_c\alpha_s + \alpha_{ov} - \alpha_n}{2}\right)^2 + \psi_c\alpha_s\omega}\right] \qquad (132)$$

Where $\alpha_s = \dfrac{R_s A_s}{R_b b h_0}$; $\alpha_n = \dfrac{N}{R_b b h_0}$; $\alpha_{ov} = \dfrac{A_{ov}}{b h_0}$

ψ_c, ξ_R, ω – See table 18 and 19

A_{ov} – Area of compressed flange overhangs equal to $A_{ov} = \left(b_f^{'} - b\right)h_f$

If value x determined by formula (132) is more than $h - h_f$ (that is the border of compressed zone goes along the least compressed flange) it is possible to consider the increase of the bearing capacity due to the less compressed flange. In that case (if $b_f^{'} = b_f$) the calculation is made by formulas (131) and (132) replacing b_f by $b_f^{'}$ and $h_f^{'}$ by $(h + h_f^{'} - h_f)$ taking $A_{ov} = -\left(b_f - b\right)\left(h - h_f^{'} - h_f\right)$.

Note. By variable height of the flange overhangs values h_f and $h_f^{'}$ are taken equal to the average height of overhangs.

3.68. Required quantity of symmetrical reinforcement of I-sections is determined in the following manner.

If condition (130) is met so reinforcement is chosen as for rectangular sections $b_f^{'}$ wide according to Item 3.62.

If condition (130) is not met so reinforcement is chosen according to relative height of compressed zone ξ:

$$\xi = \alpha_n - \alpha_{ov} \qquad (133)$$

a) by $\xi \le \xi_R$

$$A_s = A_s^{'} = \frac{R_b b h_0}{R_s}\frac{\alpha_{m1} - \xi(1 - \xi/2) - \alpha_{m,ov}}{1 - \delta} \qquad (134)$$

b) by $\xi > \xi_R$

$$A_s = A_s^{'} = \frac{R_b b h_0}{R_s}\frac{\alpha_{m1} - \xi_1(1 - \xi_1/2) - \alpha_{m,ov}}{1 - \delta} \qquad (135)$$

Where relative height of compressed zone $\xi_1 = x / h_0$ is determine according to formula (132) by:

$$\alpha_s = \frac{\alpha_{m1} - \xi(1-\xi) - \alpha_{m,ov}}{1-\delta} \tag{136}$$

In formulas (133)–(136):

α_n, α_{ov} – See Item 3.67;

$$\alpha_{m,1} = \frac{Ne}{R_b b h_0^2}; \ \delta = a' / h_0;$$

$$\alpha_{m,ov} = \alpha_{ov}\left(1 - 0.5 h_f^{'} / h_0\right).$$

RING CROSS-SECTIONS

3.69. The strength of ring cross-sections (Draft 38) by the ratio of inner and outer radius $r_1 / r_2 \geq 0.5$ and reinforcement distributed along the circle (by no less than 6 longitudinal rods) is calculated in the following manner according to relative area of compressed zone of concrete ξ_{cir}:

$$\xi_{cir} = \frac{N + R_s A_{s,tot}}{R_b A + 2.7 R_s A_{s,tot}} \tag{137}$$

a) by $15 < \xi_{cir} < 0.6$ – according to the following condition:

$$Ne_0 \leq \left(R_b A r_m + R_s A_{s,tot} r_s\right)\frac{\sin \pi \xi_{cir}}{\pi} + R_s A_{s,tot} r_s \left(1 - 1.7\xi_{cir}\right)\left(0.2 + 1.3\xi_{cir}\right) \tag{138}$$

b) by $\xi_{cir} \leq 0.15$ – according to the following condition

$$Ne_0 \leq \left(R_b A r_m + R_s A_{s,tot} r_s\right)\frac{\sin \pi \xi_{cir1}}{\pi} + 0.29 R_s A_{s,tot} r_s \tag{139}$$

Where $\xi_{cir1} = \dfrac{N + 0.75 R_s A_{s,tot}}{R_b A + R_s A_{s,tot}}$ (140)

c) By $\xi_{cir} \geq 0.6$ – according to the following condition

$$Ne_0 \leq \left(R_b A r_m + R_s A_{s,tot} r_s\right)\frac{\sin \pi \xi_{cir2}}{\pi} \tag{141}$$

Where $\xi_{cir2} = \dfrac{N}{R_b A + R_s A_{s,tot}}$ (142)

In formulas (137)–(142):

$A_{s,tot}$ – Section area of total longitudinal reinforcement;

$r_m = \dfrac{r_1 + r_2}{2}$;

r_s – Circle radius going through the center of gravity of considered reinforcement.

Eccentricity of longitudinal reinforcement e_0 is determined considering the element deflection according to Items 3.54–3.56.

Draft 38. The scheme taken by calculation of the ring cross-section of eccentric compressed element

3.70. It is possible to check the strength and to determine required quantity of reinforcement of ring sections mentioned in Item 3.69 by $r_s \approx r_m$ by means of diagrams of Draft 39 using the following formulas:

$$Ne_0 \le \alpha_m R_b r_m A \quad (143)$$

$$A_{s,tot} = \alpha_s \frac{R_b A}{R_s} \quad (144)$$

$$\alpha_n = \frac{N}{R_b A};\ \alpha_m = \frac{Ne_0}{R_b A r_m};\ \alpha_s = \frac{R_s A_{s,tot}}{R_b A}$$

Where values α_m and α_s are determined in compliance with the diagram according to values $\alpha_s = \frac{R_s A_{s,tot}}{R_b A}$ and $\alpha_m = \frac{Ne_0}{R_b A r_m}$ as well as $\alpha_n = \frac{N}{R_b A}$. At the same time eccentricity e_0 is determined considering the deflection of the element according to Items 3.54–3.56.

Draft 39. Bearing capacity diagrams of eccentric compressed elements of ring-section

ROUND SECTIONS

3.71. The strength of round sections (Draft 40) with reinforcement distributed along the circle (no less than 60 longitudinal rods) by concrete class no less than B30 is checked according to the following condition:

$$Ne_0 \le \frac{2}{3} R_b A r \frac{\sin^3 \pi\xi_{cir}}{\pi} + R_s A_{s,tot}\left(\frac{\sin \pi\xi_{cir}}{\pi} + \varphi\right) r_s \quad (145)$$

Where r – radius of the cross section;

ξ_{cir} – Relative area of the compressed concrete zone determined in the following manner:

If the following condition is met:

$$N \le 0.77 R_b A + 0.645 R_s A_{s,tot} \quad (146)$$

According to the following equation:

$$\xi_{cir} = \frac{N + R_s A_{s,tot} + R_b A \frac{\sin 2\pi\xi_{cir}}{2\pi}}{R_b A + R_s A_{s,tot}}; \quad (147)$$

If equation (146) is not met so it's determined according to the following equation:

$$\xi_{cir} = \frac{N + R_b A \frac{\sin 2\pi\xi_{cir}}{2\pi}}{R_b A + R_s A_{s,tot}}; \quad (148)$$

φ – Coefficient considering the work of tensile reinforcement and taken equal to; if condition (146) is met $\varphi = 1.6(1 - 1.55\xi_{cir})\xi_{cir}$ but no more than one; if condition (146) is not met so $\varphi = 0$;

$A_{s,tot}$ – Section area of the total longitudinal reinforcement;

r_s – Circle radius going through the center of gravity of longitudinal reinforcing rods.

Eccentricity of longitudinal force e_0 is determined considering the deflection of the element according to Items 3.54–3.56.

Draft 40. The scheme taken by the calculation of the round section of the eccentric compressed element

3.72. It is possible to check the strength and to determine required quantity of reinforcement of round sections mentioned in Item 3.71 by means of diagrams of Draft 41 using the following formulas:

$$Ne_0 \le \alpha_m R_b A r\,; \tag{149}$$

$$A_{s,tot} = \alpha_s \frac{R_b A}{R_s}\,, \tag{150}$$

Where values α_m and α_s are determined by Draft 41 according to values $\alpha_s = \dfrac{R_s A_{s,tot}}{R_b A}$ and $\alpha_m = \dfrac{Ne_0}{R_b A r}$ as well as $\alpha_n = \dfrac{N}{R_b A}$. At the same time eccentricity e_0 is determined considering the deflection of the element according to Items 3.54–3.56.

Draft 41. Bearing capacity diagrams of eccentric compressed elements of round section

CALCULATION OF ELEMENTS WORKING IN BIAXIAL ECCENTRIC COMPRESSION

3.73. Calculation of normal sections of elements working in biaxial eccentric compression is made in general case according to Item 3.76 determining location of the straight line which bounds the compressed zone by means of step-by-step approximation.

3.74. Calculation of elements of rectangular section with symmetrical reinforcement as regards biaxial compression can be made by means of diagrams of Draft 42.

Draft 42. Diagrams of bearing capacity of rectangular section elements with symmetrical reinforcement working in biaxial eccentric compression

a – by $\alpha_s = 0.2$; *b* – by $\alpha_s = 0.4$; *c* – by $\alpha_s = 0.6$; *d* – by $\alpha_s = 1.0$ (where $\alpha_s = \dfrac{R_s A_{s,tot}}{R_b bh}$)

Section strength is considered to be provided if points with coordinates M_x / M_x^0 and M_y / M_y^0 on the diagram corresponding to parameter α_s are located inside of the part bounded by a curve corresponding to parameter α_{n1} and by axes of coordinates.

Values M_x and M_y are represented by bending moments caused by external loads relating to the center of gravity of the section and acting in symmetry planes *x* and *y*. Influence of the element deflection is considered by means of multiplying of moments M_x and M_y by coefficients η_x and η_y determined for planes *x* and *y* according to Item 3.54 by the longitudinal force *N*.

Values M_x^0 and M_y^0 are represented by limit bending moments which can be taken by the section in symmetry planes *x* and *y* considering longitudinal force *N* applied in the center of gravity of the section.

Values of limit moments M_x^0 and M_y^0 are represented by right parts of equations (117) and (118). At the same time discretely located reinforcement rods are replaced by distributed reinforcement.

$$A_{sx} = A_{s1,x}(n_x + 1) + (2A_{s0} - A_{s1,x} - A_{s1,y})\frac{\beta}{1+\beta}; \quad (151)$$

$$A_{sy} = \frac{A_{s,tot}}{2} - A_{sx} \quad (152)$$

Where A_{sx}, A_{sy} – area of reinforcement located at surfaces normal to symmetry axes *x* and *y* (Draft 43);

$A_{s1,x}, A_{s1,y}$ – Area of each intermediate rod located at surfaces normal to symmetry axes *x* and *y*;

n_x – Number of intermediate rods with the area $A_{s1,x}$ located along one side of the section;

A_{s0} – Angle rod area;

$\beta = \frac{M_x}{M_y}\frac{h_y}{h_x}$;

h_x, h_y – Section height by eccentric compression in planes *x* and *y*;

$A_{s,tot}$ – Section area of total longitudinal reinforcement.

Parameters α_s and α_{n1} are determined by following formulas:

$$\alpha_s = \frac{R_s A_{s,tot}}{R_b bh}; \ \alpha_{n1} = \frac{N}{R_b bh}$$

Draft 43. Location of reinforcement in rectangular section by calculation as regards biaxial eccentric compression

a – actual; *b* – design

3.75. Calculation of elements of symmetrical I-section by $b_f / b = 3 - 5$ and $h_f / h = 0.15 - 0.25$ with symmetrical reinforcement located in flanges of the section as regards biaxial compression can be made by means of diagrams of bearing capacity given on Draft 44.

Draft 44. Bearing capacity of elements of symmetrical I-section working in biaxial eccentric compression

a – by $\alpha_s = 0.2$; *b* – by $\alpha_s = 0.4$; *c* – by $\alpha_s = 1.0$; *d* – by $\alpha_s = 1.4$ (where $\alpha_s = \frac{R_s A_{s,tot}}{R_b bh}$); *e* – by $\alpha_s = 1.8$; *f* – by $\alpha_s = 2.8$ (where $\alpha_s = \frac{R_s A_{s,tot}}{R_b bh}$)

The calculation is made similar to the calculation given in Item 3.74 for elements of rectangular section.

Taken by the section in the symmetry axis *x* going in the rib limit moment M_x^0 represented by the right part of condition (131) decreased by $N(h_0 - a')/2$; limit moment M_y^0 can be determined as for rectangular section made up of two flanges according to Item 3.63.

GENERAL CASE OF CALCULATION OF NORMAL SECTIONS OF ECCENTRIC COMPRESSED ELEMENTS (BY ANY SECTIONS, EXTERNAL FORCES AND BY ANY REINFORCEMENT)

3.76.(3.28) Calculation of the eccentric compressed element in general case (Draft 45) must be made according to the following equation:

$$N\bar{e} \le R_b S_b - \Sigma\sigma_{si} S_{si} \tag{153}$$

Where $\bar{e}$ – the distance of longitudinal force to the axis parallel to the line bounding compressed zone and going through the center of gravity of a tensile rod which is most distant from the mentioned line;

S_b – Static moment of the section area of compressed zone of concrete relating to the mentioned axis;

S_{si} – Static moment of the section area of *i*-rod of longitudinal reinforcement relating to the mentioned axis;

σ_{si} – Stress in the *i*-rod of longitudinal reinforcement determined according to the present item instructions.

The height of compressed zone x and stresses σ_{si} are determined according to the following equations:

$$R_b A_b - \Sigma\sigma_{si} A_{si} - N = 0 \tag{154}$$

$$\sigma_{si} = \frac{\sigma_{sc,u}}{1 - \frac{\omega}{1.1}} \left(\frac{\omega}{\xi_i} - 1 \right) \tag{155}$$

In formulas (154) and (155):

A_{si} – Section area of *i*-rod of longitudinal reinforcement;

ξ_i – Relative height of compressed concrete zone equal to $\xi_i = \frac{x}{h_{0i}}$ where h_{0i} is the distance from the axis going through the center of gravity of the *i*-rod section and parallel to the line bounding the compressed zone to the most distant point of compressed zone of the section (see draft 45);

ω – Characteristics of concrete compressed zone determined by formulas (15) or (104);

$\sigma_{sc,u}$ – See Items 3.14 and 3.59.

Draft 45. Forces scheme and stresses diagram in the section normal to the longitudinal axis of the reinforced concrete element, in general case according to the calculation as regards the strength

I – I the plane parallel to the bending moment plane or the plane going through the point of application of the longitudinal force and resultant force of internal compression and tension forces; A – pint of application of resultant forces in compressed reinforcement and in concrete of compressed zone; Б – the same in tensile reinforcement; 1 – 8 – rods.

Stress σ_{si} is inserted into the calculation with the sign determined by formula (155), at the same time stresses with sign “plus” symbolize tension stress and are taken no more than R_{si} and stresses with sign “minus” symbolize compression stresses and are taken no more than R_{sc}.

To determine the location of the compressed zone by biaxial eccentric compression except formulas (154) and (155) it is necessary to meet the additional requirement: points of application of external longitudinal force, resultant force of compression forces in concrete and reinforcement and resultant force in tensile reinforcement must belong to one straight line (see Draft 45).

If it is possible to identify the specific axis (for example symmetry axis or axis of the rib of a L-section) by biaxial eccentric compression it is necessary to make the calculation according to two conditions: (153) determining values $\bar{e}$, S_b and S_{si} relating to axis x going through the center of gravity of the most tensile rod parallel to the mentioned above specific axis and according to condition (153) determining values $\bar{e}$, S_b and S_{si} relating to axis y which crosses axis x at right angle in the center of the most tensile rod. At the same time the location of the straight line bounding the compressed zone is chosen by means of step-by-step approximation according to equations (154) and (155) taking the angle of slope of this line θ constant and equal to the angle of slope of the neutral axis determined as for elastic material.

Section strength will be provided only if condition (153) is met relating to both axes (x and y). If equation (153) is not met during all examinations so the strength of the section is not provided and it is necessary to increase reinforcement, dimensions of the section or to increase concrete class. If the condition is met only relating to one axis so it is necessary to determine the shapes of compressed zone one more time by different angle θ and to make similar calculation one more time.

EXAMPLES OF CALCULATION

RECTANGULAR SECTIONS WITH SYMMETRICAL REINFORCEMENT

Example 24. *Given:* a column with the frame work, section b = 400 mm, h = 500 mm, $a = a' = 40$ mm; heavy-weight concrete B25 ($E_b = 2.7 \cdot 10^4$ MPa); reinforcement A-III ($R_s = R_{sc} = 365$ MPa; $E_s = 2 \cdot 10^5$ MPa); its section area is $A_s = A_s^{'} = 1232\,\text{mm}^2$ (2Ø28); longitudinal forces and bending moments: from dead loads and long-term loads $N_l = 650$ kN, $M_l = 140$ kN·m; from wind load $N_{sh} = 50$ kN, $M_{sh} = 73$ kN·m; design length of the column $l_0 = 6$ m.

It is required to check the strength of the column section.

Calculation: $h_0 = 500 - 40 = 460$ mm. As there are forces from the short-term loads (wind load) so the calculation is made according to case “a” in compliance with Item 3.1.

Forces from wind load are equal to:

$N = 650 + 50 = 700$ kN; $M = 140 + 73 = 213$ kN·m

Let’s determine moments of external forces relating to tensile reinforcement M_I and M_{II} calculated considering and without consideration a short-term load (wind load):

$$M_{II} = M_1 = M + N\frac{h_0 - a'}{2} = 213 + 700\frac{0.46 - 0.04}{2} = 360\ \text{kN·m}$$

$$M_I = M_{1l} = M_l + N_l\frac{h_0 - a'}{2} = 140 + 560\frac{0.46 - 0.04}{2} = 276.5\ \text{kN·m}$$

As $0.82M_{II} = 0.82 \cdot 360 = 295$ kN·m so the calculation is made only according to case “b” (see Item 3.1) that is as regards all loads taking $R_b = 16$ MPa (by $\gamma_{b2} = 1.1$).

As $l_0 / h = 6/0.5 = 12 > 10$ so the calculation is made considering the column deflection according to Item 3.54, calculating N_{cr} by formula (93).

For that we determine:

$$\varphi_l = 1 + \beta \frac{M_{1l}}{M_1} = 1 + 1\frac{276.5}{360} = 1.77$$

[Here $\beta = 1.0$ for heavy-weight concrete (see Table 16)];

$$\mu\alpha = \frac{A_s + A_s^{'}}{bh}\frac{E_s}{E_b} = \frac{2 \cdot 1232 \cdot 2 \cdot 10^5}{400 \cdot 500 \cdot 2.7 \cdot 10^4} = 0.0913;$$

$$e_0 = \frac{N}{M} = \frac{213 \cdot 10^6}{700 \cdot 10^3} = 304\,\text{mm} > e_a = h/30$$

So that means accidental eccentricity is not to be taken into account.

As $\frac{e_0}{h} = \frac{304}{500} = 0.608 > \delta_{e,\min} = 0.5 - 0.01\frac{l_0}{h} - 0.01R_b = 0.5 - 0.01 \cdot 12 \cdot 16 = 0.22$ so we take $\delta_e = \frac{e_0}{h} 0.608$.

Coefficient η is to be determined by formula (91):

Value e is:

$e = e_0\,\eta + \frac{h_0 - a'}{2} = 304 \cdot 1{,}115 + \frac{460 - 40}{2} = 549\ mm \cong 0{,}55\ м.$

Let's determine the height of compressed zone x by formula (107):

mm.

ξ_R = 0.55 (See Table 18).

As x = 109,4 mm < $\xi_R h_0$ = 0,55 · 460 = 253 mm so section strength is to be checked according to condition (108):

$R_b bx(h_0 - 0{,}5x) + R_{sc}A_s'(h_0 - a') = 16 \cdot 400 \cdot 109{,}4(460 - 0{,}5 \cdot 109{,}4) + 365 \cdot 1232 \times$

$\times (460 - 40) = 472{,}6 \cdot 10^6\ N \cdot mm = 472{,}6\ kN \cdot m > Ne = 700 \cdot 0{,}55 = 385\ kN \cdot m,$

т. е. прочность сечения обеспечена.

Example 25. *Given:* section of the element with dimensions b = 400 mm, h = 500 mm; $a = a'$ = = 40 mm; heavy-weight concrete B25 (E_b = 2,7 · 10^4 MPa); symmetrical reinforcement A-III (R_s = = R_{sc} = 365 MPa; E_s = 2 · 10^5 MPa); longitudinal forces and bending moments: from dead loads and long-term loads N_l = 600 kN, M_l = 170 kN·m; from wind load N_{sh} = 200 kN, M_{sh} = 110 kN·m; design length l_0 = 8 m.

It is required to determine section area of reinforcement.

Calculation: h_0 = 500 – 40 = 460 mm. As there are wind load forces so let's check condition (1). For that we determine:

kN·m;

kN·m;

kN;

kN·m.

As 0,82 M_{II} = 0,82 · 448 = 368 kN·m > M_{I} = 296 kN·m so the calculation is made only according to case „b", that is as regards all loads, taking R_b = 16 MPa (by γ_{b2} = 1,1).

As l_0/h = 8000/500 = 16 > 10 so the calculation is made considering the element deflection according to Item 3.54, calculating N_{cr} by formula (93).

For that we determine:

[β = 1,0 – See Table 16];

$e_0 = \frac{M}{N} = \frac{280 \cdot 10^6}{800 \cdot 10^3} = 350\ mm > e_a = h/30$

(See Item 3.50)

As e_0/h = 350/500 = 0,7 > $\delta_{e,min}$ = 0,5 – 0,01 – 0,01R_b, so we take δ_e = = 0,7.

On the first approximation we take μ = 0,01, = 7,4,

So

$$N_{cr} = \frac{1{,}6E_b bh}{(l_0/h)^2}\left[\frac{\frac{0{,}11}{0{,}1+\delta_e}+0{,}1}{3\varphi_l}+\mu\alpha\left(\frac{h_0-a'}{h}\right)^2\right] = \frac{1{,}6\cdot 2{,}7\cdot 10^4\cdot 400\cdot 500}{16^2}\times$$

$$\times\left[\frac{\frac{0{,}11}{0{,}1+0{,}7}+0{,}1}{3\cdot 1{,}66}+0{,}01\cdot 7{,}4\left(\frac{460-40}{500}\right)^2\right] = 33{,}75\cdot 10^6\,(0{,}0477+0{,}0522) = 3372\cdot 10^3\ H = 3372\ kN$$

Coefficient η is:

Considering the element deflection value e is:

 mm.

Required reinforcement we determine according to Item 3.62.

Let's determine the following values:

By Table 18 we find ξ_R = 0,55.

As $\alpha_n < \xi_R$, so value A_s = is to be determined by Formula (112):

$$A_s = A'_s = \frac{R_b bh_0}{R_s}\ \frac{\alpha_{m1}-\alpha_n(1-\alpha_n/2)}{1-\delta} =$$

$$= \frac{16\cdot 400\cdot 460}{365}\ \frac{0{,}395-0{,}272(1-0{,}272/2)}{1-0{,}087} = 1413\ mm^2,$$

From which

As the present reinforcement is more than reinforcement taken during determination of N_{cr} (μ = = 0,01) so value A_s = 1413 mm^2 is determined „on the safe side" and it can be decreased if value μ. is specified more exact

We take μ = (0,01 + 0,014)/2 = 0,012 and determine value A_s = :

$$N_{cr} = 33{,}75\cdot 10^6\left[0{,}0477+0{,}012\cdot 7{,}4\times\right.$$

$$\left.\times\left(\frac{460-40}{500}\right)^2\right] = 3724\cdot 10^3\ H = 3724\ kN;$$

 mm;

 mm^2.

Finally we take A_s = = 1362 mm^2 (2 ∅ 25 + 1 ∅ 22).

Example 26. According to Example 25 data it is necessary to determine reinforcement area, using diagrams of Annex 3.

Calculation. In compliance with Example 25: N = 800 kN; M = 280 kN·m; = 16; = 0,66.

Let's determine values α_n и α_m:

By diagram *б* of Annex 3 by α_n = 0,272, α_m = 0,207 and λ = 15 we find α_s = 0,16.

By diagram *в* of Annex 3 by α_n = 0,272, α_m = 0,207 and λ = 20 we find α_s = 0,2.

Value α_s corresponding to λ = 16 is to be determined by means of linear interpolation:

So reinforcement area is:
mm^2.
We take A_s = = 1362 mm^2 (2 ∅ 25 + 1 ∅ 22).

Example 27. *Given:* a column with the multistory framework with the section dimensions b = = 400 mm, h = 500 mm; $a = a'$ = 40 mm; heavy-weight concrete B25 (E_b = 2,7 · 10^4 MPa); symmetrical reinforcement A-III ($R_s = R_{sc}$ = 365 MPa; E_s = 2 · 10^5 MPa); longitudinal forces and bending moments in the support section of the column: from dead loads and long-term loads on the floors N_l = 2200 kN, M_l = 259 kN·m; from wind loads N_{sh} = 0, M_{sh} = 53,4 kN·m; no short-term loads on the floor; design length of the column is l_0 = 6 м.
It is required to determine reinforcement area.
Calculation. $h_0 = h - a$ = 500 − 40 = 460 mm. As there is the wind load force we determine condition (1). We determine:
kN·m;
kN;
kN·m;
kN·m.
As 0,82 M_{II} = 0,82 · 784,4 = 643 кН·м < M_I = 721 kN·m so condition (1) is not met and we make the calculation twice: according to case „a" — as regards dead loads and long-term loads by R_b = 13 MPa (that is by γ_{b2} = 0,9) and according to case „b" — as regards all loads by R_b = = 16 МПа (that is by γ_{b2} = 1,1). The calculation is made for the support section.
Calculation according to case „a". As l_0/h = 6000/500 = 12 > 4 according to Item 3.54 so it is necessary to consider the column deflection. But in compliance with Item 3.56 coefficient η_v for columns and multistorey frameworks taken for the moment M_v caused by loads on the floors is taken equal to 1,0 and moment $M_h = M_{sh}$ from wind loads is not considered in the present calculation, that's why design moment is $M = M_v\,\eta_v$ = 259 kN·m.
Design longitudinal force is $N = N_l$ = 2200 kN, so
= 118 mm > = 16,7 mm. We take e_0 = 118 mm.
By formula (111) we determine $e = e_0 + (h_0 - a')/2$ = 118 + (460 − 40)/2 = 328 mm.
Required reinforcement is to be determined according to Item 3.62. Let's determine the following values:

By Table 18 we find ξ_R = 0,604.
As α_n = 0.92 > ξ_R = 0,604 so value A_s = is to be determined by formula (113). For that we calculate values α_s and ξ by formulas (114) and (109):

$$A_s = A'_s = \frac{R_b b h_0}{R_s}\,\frac{\alpha_{m1} - \xi(1-\xi/2)}{1-\delta} = \frac{13\cdot 400\cdot 460}{365}\,\frac{0{,}656 - 0{,}772(1-0{,}772/2)}{1-0{,}087} = 1304\ mm^2.$$

Calculation by case „b". In compliance with Item 3.54 we determine coefficient η, taking reinforcement, calculated according to case „a", that is:

[β = 1,0 – See Table 16];
mm.
As e_0/h = = 0,293 > $\delta_{e,min}$ = 0,5 − 0,01 l_0/h − 0,01 R_b = 0,5 −0,01 · 12 − 0,01 · 16 = 0,22, we take $\delta_e = e_0/h$ = 0,293;

By formula (93) we determine N_{cr}:

$$N_{cr} = \frac{1{,}6E_b bh}{(l_0/h)^2}\left[\frac{\frac{0{,}11}{0{,}1+\delta_e}+0{,}1}{3\varphi_l}+\mu\alpha\left(\frac{h_0-a'}{h}\right)^2\right] = \frac{1{,}6\cdot 2{,}7\cdot 10^4\cdot 400\cdot 500}{12^2}\times$$

$$\times\left[\frac{\frac{0{,}11}{0{,}1+0{,}293}+0{,}1}{3\cdot 1{,}92}+0{,}096\times\left(\frac{460-40}{500}\right)^2\right] = 8021\cdot 10^3\ N,$$

So coefficient η is:

According to Item 3.56 coefficient $\eta = \eta_h = 1{,}38$ is multiplied by the wind loads moment M_{sh} = = M and coefficient η_v = 1,0, that's why considering the element deflection the moment is equal to:

kN·m.

Required reinforcement we determine by Item 3.62 similar to the calculation according to case „a" taking R_b = 16 MPa:

mm;

According to Table 18 we find $\xi_R = 0{,}55$.

As $\alpha_n > \xi_R$ so we determine value A_s = by formula (113):

So

$$A_s = A_s' = \frac{R_b bh_0}{R_s}\,\frac{\alpha_{m1}-\xi(1-\xi/2)}{1-\delta} = \frac{16\cdot 400\cdot 460}{365}\,\frac{0{,}586-0{,}675(1-0{,}675/2)}{1-0{,}087} =$$

$$= 1228\ мм^2 < 1304\ mm^2.$$

Finally we take A_s = = 1362 mm^2 (2 ∅ 25 + 1 ∅ 22) >1304 mm^2.

Example 28. *Given:* the element section with dimensions b = 400 mm, h = 600 mm; heavy-weight concrete B25 (R_b = 16 MPa by γ_{b2} = 1,1; $E_b = 2{,}7 \cdot 10^4$ MPa); reinforcement A-III (R_s = R_{sc} = 365 MPa; $E_s = 2 \cdot 10^5$ MPa) located in the section as it's shown on Draft 46; longitudinal forces and bending moments: from all loads N = 500 kN, M = 500 kN·m; from dead loads and long-term loads N_l = 350 kN, M_l = 350 kN·m; design length l_0 = 10 м.

It is required to examine the section strength.

Черт. 46. For the calculation example 28

The calculation is to be made according to Item 3.63. Taking $A_{s1,l}$ = 491 mm^2 (∅ 25), η_l = 2 and $A_{s,tot}$ = 6890 mm^2 (8 ∅ 28 + 4 ∅ 25) we find reinforcement area A_{sl} и A_{st}:

mm^2;

mm^2.

According to Draft 46 we have a_1 = 45 mm, so

As l_0/h = 10/0,6 = 16,7 > 10 the calculation is to be made considering the element deflection according to Item 3.54 determining value N_{cr} by formula (93).

For that we determine:

[β = 1.0 (See Table 16)];

m.
As e_0/h = = 1,67 > $\delta_{e,min}$ = 0,5 – 0,01 l_0/h – 0,01 R_b, so we take δ_e = e_0/h = 1,67.
Value $\mu\alpha$ we determine as for the section with reinforcement located along the height of the section in compliance with Item 3.54:

So

$$N_{cr} = \frac{1{,}6 E_b bh}{(l_0/h)^2} = \left[\frac{\frac{0{,}11}{0{,}1+\delta_e}+0{,}1}{3\varphi_l} + \mu\alpha\left(\frac{h_0-a'}{h}\right)^2\right] = \frac{1{,}6\cdot 2{,}7\cdot 10^4\cdot 400\cdot 600}{16{,}7^2}\times$$

$$\times\left(\frac{\frac{0{,}11}{0{,}1+1{,}67}+0{,}1}{3\cdot 1{,}7}+0{,}11\right) = 5271\cdot 10^3 \ H = 5271 \ kN.$$

Coefficient η is:

Let's determine the following values:

According to Table 18 we find ω = 0,722 and ξ_R = 0,55.
As 0.24 < ξ_R = 0.55 so section strength is to be determined by formula (117):

$$R_b bh^2\left[0{,}5\xi(1-\xi)+\alpha_{sl}(\xi_1-\delta_1)(1-\xi_1-\delta_1)-0{,}05\alpha_{sl}\xi_1^2+\alpha_{st}(1-2\delta_1)\right]=16\cdot 400\cdot 600^2\times$$
$$\times\left[0{,}5\cdot 0{,}24(1-0{,}24)+0{,}329(0{,}332-0{,}075)\times(1-0{,}332-0{,}075)-0{,}05\cdot 0{,}329\cdot 0{,}332^2+\right.$$
$$\left.+0{,}187(1-2\cdot 0{,}075)\right]=688\cdot 10^6 \ H\cdot мм = 688 \ кН\cdot м > Ne_0\,\eta = 500\cdot 1{,}105 = 552 \ kN\cdot m$$

That is the section strength is provided.

Example 29. *Given:* column section with dimensions b = 600 mm, h = 1500 mm; heavy-weight concrete B30 (R_b = 19 MPa by γ_{b2} = 1,1); reinforcement A-III (R_s = 365 MPa) located as it's shown on Draft 47; longitudinal forces and bending moments determined according to the frame calculation as regards deformed scheme: from all loads N = 12 000 kN, M = 5000 kN·m; from dead loads and long-term loads N_l = 8500 kN, M_l = 2800 kN·m; design length of the column in the bending plane l_0 = 18м, out of the bending plane l_0 = 12 м; actual column length l = 12 m.
It is required to examine the section strength.

Черт. 47. For the calculation example 29

<u>The calculation in the bending plane</u> is made according to Item 3.63.
Taking $A_{s1,l}$ = 615, 8 mm^2 (∅ 28), η_l = 5 and $A_{s,tot}$ = 17 417 mm^2 (14 ∅ 32 + 10 ∅ 28), we find reinforcement area A_{sl} and A_{st}: A_{sl} = $A_{s1,l}$ (η_l + 1) = 615, 8 (5 + 1) = 3695 mm^2, mm^2.
Center of gravity of reinforcement located at tensile surface (7 ∅ 32) is distant from this surface
mm,
So

Let's determine the following values:

From Table 18 we find $\omega = 0{,}698$ and $\xi_R = 0{,}523$. As $0{,}584 > \xi_R = 0{,}523$so section strength is to be checked by formula (118).
For that we determine:

$$R_b bh^2 \alpha_{mR} \frac{\alpha_{na} - \alpha_{n1}}{\alpha_{na} - \alpha_{nR}} = 19 \cdot 600 \cdot 1500^2 \cdot 0{,}24 \times$$
$$\times \frac{1{,}372 - 0{,}702}{1{,}372 - 0{,}61} = 5413 \cdot 10^6 \ H \cdot мм = 5413 \ кH \cdot м > M = 5000 \ kN \cdot m,$$

That is the section strength in the bending plane is provided.

<u>The calculation out of the bending plane.</u> As design length out of the bending plane l_0 = 12 m and ratio l_0/b = 12/0,6 = 20 is more than ration l_0/h = 18/1,5 = 12, corresponding to the column calculation in the bending plane according to 3.51, so it is necessary to calculate the column out of the bending plane taking eccentricity e_0 equal to occasional eccentricity e_a. At the same time we replace symbols h and b by b and h, that is we take the dimension of the section out of the bending plane h = 600 мм instead of the section height.

As according to Item 3.50 the accidential eccentricity is equal to $e_a = \frac{h}{30} = \frac{600}{30} = 20 \ mm \geq \frac{l}{600} = \frac{12\ 000}{600}$ and l_0 = 12 m ≤ 20h, so the calculation is made according to Item 3.64.

Section area of intermediate rods located along the short sides is equal to $A_{s,int}$ = 4826 мм^2 (6 ∅ 32). As = 5800 mm^2 > $A_{s,int}$ = 4876 mm^2 и a = 50 mm < 0,15h = 0,15 · 600 = 90 mm so we use Table 27 in the calculation (Part A). According to Tables 26 and 27 by and we find φ_b = 0,674 and φ_{sb} = 0,77.

Value

By formula (120) we determine coefficient φ:

Let's check condition (119):

$$\varphi(R_b A + R_s A_{s,tot}) = 0{,}746(19 \cdot 600 \cdot 1500 + 365 \cdot 17\ 417) = 17\ 500 \ кH > N = 12\ 000 \ kN,$$

That is the section strength out of the bending plane is provided.

Example 30. *Given:* a column with the section 400X400 mm; design length is equal to the actual length $l = l_0$ = 6 m; heavy-weight concrete B25 (R_b = 13 MPa by γ_{b2} = 0,9); longitudinal reinforcement A-III (R_{sc} = 365 MPa); centric applied forces: from dead loads and long-term loads N_l = 1800 kN; from short-term loads N_{sh} = 200 kN.

It is required to determine section area of longitudinal reinforcement.

Calculation: in compliance with Item 3.50the calculation is made considering occasional eccentricuty e_a.

As h/30 = 400/30 = 13,3 mm **>** = 10 mm so occasional eccentricity is taken equal to $e_a = h/30$, so the calculation can be made according to Item 3.64, taking $N = N_l + N_{sh} = 1800 + 200 = 2000$ kN.

According to Tables 26 and 27 for heavy-weight concrete by N_l/N = 1800/2000 = 0,9, l_0/h = =6000/400 = 15, supposing that there are no intermediate rods by $a = a' < 0{,}15\ h$, we find φ_b = 0,8 и φ_{sb} = 0,858.

Taking in the first approximation $\varphi = \varphi_{sb}$ = 0,858 according to condition (119) we find:

$$R_s A_{s,tot} = \frac{N}{\varphi} - R_b A = \frac{2000 \cdot 10^3}{0{,}858} - 13 \cdot 400 \cdot 400 = 2331 \cdot 10^3 - 2080 \cdot 10^3 = 251 \cdot 10^3 \ N.$$

So

As $\alpha_s < 0{,}5$ we specify value φ more exact calculating it by formula (120):

In a similar manner we determine

$$R_s A_{s,tot} = \frac{2000 \cdot 10^3}{0{,}814} - 2080 \cdot 10^3 = 377 \cdot 10^3 \ N.$$

Calculated value $R_s A_{s,tot}$ is more than the accepted in the first approximation value so we determine this value one more time:

$$R_s A_{s,tot} = \frac{2000 \cdot 10^3}{0{,}821} - 2080 \cdot 10^3 = 360 \cdot 10^3 \ N.$$

As determined value $R_s A_{s,tot}$ is close to the value accepted in the second approximation so total area of reinforcement section is taken equal to:

mm^2.

Finally we take $A_{s,tot}$ = 1018 mm^2 (4 ∅ 18).

RECTANGULAR SECTIONS WITH ASYMETRICAL REINFORCEMENT

Example 31. *Given:* element section with dimensions b = 400 mm, h = 500 mm; $a = a'$ = 40 mm; heavy-weight concrete B25 (R_b = 13 MPa by γ_{b2} = 0,9; E_b = 2,7 · 10^4); reinforcement A-III ($R_s = R_{sc}$ = 365 MPa); longitudinal force N = 800 kN; its eccentricity relating to the center of gravity of concrete section e_0 = 500 мм; design length l_0 = 4,8 m.

It is required to determine areas of reinforcement section S and S'.

Calculation. h_0 = 500 − 40 = 460 mm. As 4 < l_0/h = 4,8/0,5 = 9,6 < 10so the calculaiton is made considering the element deflection according to Item 3.54. At the same time supposing that $\mu \leq 0{,}025$we determine value N_{cr} by a simplified formula:

$$N_{cr} = 0{,}15 \frac{E_b A}{(l_0 / h)^2} = 0{,}15 \frac{2{,}7 \cdot 10^4 \cdot 400 \cdot 500}{9{,}6^2} = 9110 \cdot 10^3 \ H = 9110 \ kN.$$

Coefficient η is to be determined by formula (91):

Value e considering the element deflection:

mm.

Required section area of reinforcement S' and S we determine by formulas (121) and (122):

$$A'_s = \frac{Ne - 0{,}4 R_b b h_0^2}{R_{sc}(h_0 - a')} = \frac{800 \cdot 10^3 \cdot 758 - 0{,}4 \cdot 13 \cdot 400 \cdot 460^2}{365(460 - 40)} = 1085 \ mm^2 > 0;$$

$$A_s = \frac{0{,}55 R_b b h_0 - N}{R_s} + A'_s = \frac{0{,}55 \cdot 13 \cdot 400 \cdot 460 - 800 \cdot 10^3}{365} + 1085 = 2498 \ mm^2.$$

As 0,018 < 0,025 so values A_s and are not to be specified more exact.

We take = 1232 mm^2 (2 ∅ 28), A_s = 2627 mm^2 (2 ∅ 32 + 1 ∅ 36).

ELEMENTS WITH CONFINEMENT REINFORCEMENT

Example 32. *Given:* a column of a bracing framework with the section dimensions and location of reinforcement according to Draft 48; heavy-weight concrete B40 (R_b = 20 MPa by γ_{b2} = 0,9; $R_{b,ser}$ = 29 MPa; E_b = 3,25 · 10^4 MPa); longitudinal reinforcement A-VI; confinement reinforcement meshes of rods A-III, diameter 10 mm ($R_{s,xy}$ = 365 MPa), located with spacing s = 130 mm along the whole length of the column; longitudinal force by γ_f > 1,0: from all loads

N = 6600 kN, from dead loads and long-term loads N_l = 4620 kN; the same by γ_f = 1,0: N = 5500 kN and N_l = 3850 kN; primary eccentricity of longitudinal force $e_0 = e_a$ = 13,3 mm; design length of the column l_0 = 3,6 m.
It is required to examine the strength of the column.

Draft. 48. For the calculation example 32

Calculation. Let's check the section strength within the meshes contours considering confinement reinforcement according to Item 3.57. Design dimensions of the section $h_{ef} = b_{ef} = 350$ mm. As l_0/h_{ef} = 3600/350 = 10,3 < 16 so confinement reinforcement can be considered in the calculation; at the same time it is necessary to consider the column deflection in compliance with Items 3.54 and 3.58 as $l_0/h_{ef} > 4$.
Taking $l_0/c_{ef} = l_0/h_{ef}$ = 10,3 and $h = h_{ef}$ = 350 mm we get

Therefore we take $\delta_e = \delta_{e,min}$ = 0,297. As intermediate rods of confinement reinforcement are located in end quarters of the distance between end rods equal to $h - 2a_1$ = 350 – 2 · 22 = 306 mm [58 mm < = 76,5 mm (see Draft 48)] so according to the note to Item 3.63 we take reinforcement S and S' as concentrated along the lines of their center of gravity. Then considering that all rods have the same diameter we have:
mm;
mm.
Coefficient φ_l is to be determined by formula (94) taking β = 1,0 (see Table 16) and

Critical force N_{cr} is to be determined by formula (93), taking
mm^2 (6 ∅ 25),

and multiplying the calculated value by coefficient φ_1 = 0,25 + 0,05 = 0,25 + 0,05 · 10,3 = =0,764:

$$N_{cr} = \frac{1{,}6E_b bh}{(l_0/h)^2}\left[\frac{\frac{0{,}11}{0{,}1+\delta_e}+0{,}1}{3\varphi_l}+\mu\alpha\left(\frac{h_0-a'}{h}\right)^2\right]\varphi_1 = \frac{1{,}6\cdot 3{,}25\cdot 10^4\cdot 350\cdot 350}{10{,}3^2}\times$$

$$\times\left[\frac{\frac{0{,}11}{0{,}1+0{,}297}+0{,}1}{3\cdot 1{,}7}+0{,}281\left(\frac{309-41}{350}\right)^2\right]\times 0{,}764 = 10\ 900\cdot 10^3\ H = 10\ 900\ kN.$$

Coefficient η is equal to:

So in compliance with formula (111),
mm.
Let's determine prism strength $R_{b,red}$ according to Item 3.57.
Taking $A_{sx} = A_{sy}$ = 78,5 mm^2 (∅ 10), $n_x = n_y = 5$, $l_x = l_y$ = 350 mm and $A_{ef} = h_{ef}b_{ef}$ = 350 · 350= = 122 500 mm^2 (see Draft 48) we determine coefficient

so

MPa
As there is used high-strength reinforcement A-VI so design resistance of reinforcement against compression is to be determined according to Item 3.59:
mm^2;

We take $\theta = 1{,}6$.

From Table 25 $\lambda_1 = 2{,}04$, $\lambda_2 = 0{,}77$, $R_{sc} = 500$ MPa, $R_s = 815$ MPa, so

$$R_{sc,red} = R_{sc}\frac{1+\delta_3\,\lambda_1}{1+\delta_3\,\lambda_2} = 500\frac{1+0{,}54\cdot 2{,}04}{1+0{,}54\cdot 0{,}77} = 742\ M\Pi a < R_s = 815\ MPa$$

Section strength is to be examined according to condition (108), determining the height of compressed zone $x = \xi h_0$ by formula(110a).
For that we determine value ω by formula (104). As $10\mu_{xy} = 10 \cdot 0{,}0173 = 0{,}173 > 0{,}15$, we take $\delta_2 = 0{,}15$, then $\omega = 0{,}85 - 0{,}008\ R_b + \delta_2 = 0{,}85 - 0{,}008 \cdot 20 + 0{,}15 = 0{,}84 \le 0{,}9$.
According to Items 3.61 and 3.65 we determine required coefficients α_n, α_s, and ψ_c, taking $R_b = R_{b,red} = 34{,}3$ MPa; $\sigma_{sc,u} = 380 + 1000\ \delta_3 = 380 + 1000 \cdot 0{,}54 = 920$ MPa < 1200 MPa иand $R_{sc} = R_{sc,red} = 742$ MPa:

$$R_b b h_0 = 34{,}3\cdot 350\cdot 309 = 3710\cdot 10^3\ N;$$

So

Value ξ_R with replacing R_s by $0{,}8R_s$ is:

that means it was necessary to use formula (110a);
mm;

$$R_b bx(h_0 - 0{,}5x) + R_{sc}A'_s(h_0 - a') = 34{,}3\cdot 350\cdot 284(309 - 0{,}5\cdot 284) +$$

$$+ 742\cdot 2945(309 - 41) = 1154{,}5\cdot 10^6\ H\cdot м > Ne = 6600\cdot 0{,}168 = 1109\ kN\cdot m,$$

that is section strength is provided.

Let's examine resistance to strength of the column protection layer by means of similar calculation as regards the force $N = 5500$ kN (by $\gamma_f = 1{,}0$) taking $R_b = R_{b,ser} = 29$ MPa, $R_s = R_{s,ser} = 980$ МПа, $R_{sc} = 400$ МПа, $\sigma_{sc,u} = 400$ МПа, $\omega = 0{,}85 - 0{,}006\ R_{b,ser} = 0{,}85 - 0{,}006 \cdot 29 = 0{,}679$ according to Item 3.60 and considering total section of the column, that means $b = h = 400$ mm, $a = a' = 41 + 25 = 66$ mm, $h_0 = 400 - 66 = 334$ mm.
Critical force N_{cr} is to be determined by formula (93) taking $l_0/h = 3600/400 = 9$, $e_0/h = 13{,}3/400 = 0{,}033$, $\delta_{e,min} = 0{,}5 - 0{,}01 - 0{,}008\ R_{b,ser} = 0{,}5 - 0{,}01 \cdot 9 - 0{,}008 \cdot 29 = 0{,}178 > e_0/h$, that means $\delta_e = \delta_{e,min} = 0{,}178$.

During determination of coefficient φ_l we consider longitudinal forces N and N_l by $\gamma_f = 1.0$, that means
so $\varphi_l = 1 + 0{,}7 = 1{,}7$;

$$N_{cr}=\frac{1{,}6E_b bh}{(l_0/h)^2}\left[\frac{\frac{0{,}11}{0{,}1+\delta_e}+0{,}1}{3\varphi_l}+\mu\alpha\left(\frac{h_0-a'}{h}\right)^2\right]=$$

$$=\frac{1{,}6\cdot 3{,}25\cdot 10^4\cdot 400\cdot 400}{9^2}\left[\frac{\frac{0{,}11}{0{,}1+0{,}178}+0{,}1}{3\cdot 1{,}7}+0{,}215\left(\frac{334-66}{400}\right)^2\right]=$$

$=19900\cdot 10^3\cdot H=19900 \text{ ħн}.$

Coefficient η is:

mm.

Let's make a calculation similar to the calculation as regards the strength:

$R_b bh_0=29\cdot 400\cdot 3=3880\cdot 10^3 N;$

mm;

$R_b bx(h_0-0{,}5x)+R_{sc}A_s'(h_0-a')=29\cdot 400\cdot 305\times$

$\times(334-0{,}5\cdot 0{,}305)+400\cdot 2945(334-66)=$

$=957{,}8\cdot 10^6 N\cdot mm=957{,}8 kN\cdot m>Ne=$

$=5500\cdot 0{,}1524=838 kN\cdot m,$

That means resistance to cracks of the column protection layer is provided.

I-SECTIONS

Example 33. *Given*: section dimensions and location of reinforcement — according to Draft 49; heavy-weight concrete B30 (E_b = 2,9 · 10^4 MPa; R_b = 19 MPa by γ_{b2} = 1,1); reinforcement A-III (R_s = R_{sc} = 365 MPa); its cross-section area A_s = A'_s = 5630 mm^2 (7 ∅ 32); longitudinal forces and bending moments: from dead loads and long-term loads N_l = 2000 kN, M_l = 2460 kN·m; from all loads N = 2500 kN, M = 3700 kN·m; design length of the element: in the bending plane l_0 = 16,2 m, out of the bending plane l_0 = 10,8 m; actual length of the element l = 10,8 m.

It is required to check the section strength.

Draft 49. For the calculation examples 33, 34 and 39

<u>The calculation in the bending plane</u>. We tae design thickness of the flange equal to the average height of overhangs h'_f = h_f = 200 + 30/2 = 215 mm.

Let's determine the area and inertial moment of concrete section:

mm^2;

$$I=\frac{200\cdot 1500^3}{12}+2\frac{400\cdot 215^3}{12}+2\cdot 400\cdot 215\times$$

$$\times\left(\frac{1500}{2}+\frac{215}{2}\right)^2=1279\cdot 10^8 \, mm^4.$$

Radius of inertia of the section is mm.

As l_0/i = 16 200/520 = 31,1 < 35 and l_0/i > 14 so the calculation is made considering the element deflection in compliance with Item 3.54 taking value N_{cr} equal to:

$$N_{cr} = \frac{2E_b I}{l_0^2} = \frac{2 \cdot 2{,}9 \cdot 10^4 \cdot 1279 \cdot 10^8}{16200^2} =$$

$$= 28270 \cdot 10^3\, H = 28270 kN.$$

Coefficient η is to be determined by formula (91):

Center of gravity of the reinforcement area A_s and A'_s is distant from the nearest surface at a = =a'= mm, and $h_0 = h - a$ = 1500 – 79 = 1421 mm.

Value e considering the element deflection is:

$$e = e_0\eta + \frac{h_0 - a'}{2} = \frac{3700 \cdot 10^6 \cdot 1{,}096}{2500 \cdot 10^3} + \frac{1421 - 79}{2} = 2293 mm.$$

Let's check condition (130):

$$R_b b'_f h'_f = 19 \cdot 600 \cdot 215 = 2451 \cdot 10^3\, H = 2451 kN < N = 2500 kN,$$

That means that the calculation is made as for the I-section.

Area of compressed flange overhangs is equal to:

mm^2.

Let's determine the area of compressed zone:

mm.

From Table 18 we find ξ_R = 0,523. As x = 228 mm < $\xi_R h_0$ = 0,523 · 1421 = 743 mm, section strength is to be checked according to condition (131):

$$R_b bx\left(h_0 - \frac{x}{2}\right) + R_b A_{ov}\left(h_0 - \frac{h'_f}{2}\right) + R_{sc}A'_s(h_0 - a') = 19 \cdot 200 \cdot 228\left(1421 - \frac{228}{2}\right) +$$

$$+ 19 \cdot 86000\left(1421 - \frac{215}{2}\right) + 365 \cdot 5630(1421 - 79) = 5847 \cdot 10^6\, N \cdot mm = 5847\, kN \cdot m >$$

$$> Ne = 2500 \cdot 2{,}293 = 5725\, kN \cdot m,$$

That means the section strength in the bending plane is provided.

Calculation out of the bending plane. Let's determine radius of the section out of the bending plane:

mm^4;

mm.

As elasticity out of the bending plane l_0/i = 10 800/134 = 80 is more than elasticity in the bending plane l_0/i = 31.1 so according to Item 3.51 we check the section strength out of the bending plane taking eccentricity e_0 equal to occasional eccentricity e_a. At the same time the section height is h = 600 мм.

As in compliance with Item 3.50 occasional eccentricity is e_a = mm> mm, we take e_a = that allows to make the calculation by in compliance with Item 3.64 as for the rectangular section without considering the „reserve" of the rib section, that means taking b = 2 · 215 = 430 mm.

Section area of intermediate rods located along both flanges is $A_{s,int}$ = 4826 mm^2 (6 ∅ 32), and section area of all rods is $A_{s,tot}$ = 11 260 mm^2 (14 ∅ 32). As $A_{s,tot}/3$ = 11 260/3 = 3750 mm^2 < <$A_{s,int}$ = 4826 mm^2 so we use Table 27 (Part Б) in the calculation. From Table 27 for heavy-weight concrete by N_l/N = 2000/2500 = 0,8 и l_0/h = 10,8/0,6 = 18 we find φ_{sb} = 0,724.

Value So, $\varphi = \varphi_{sb}$ = 0,724.

Let's check condition (119):

$$\varphi(R_b A + R_{sc}A_{s,tot}) = 0.724(19 \cdot 430 \cdot 600 + 365 \cdot 11260) = 6525 \cdot 10^3\,\text{N} > N = 2500\,\text{kN}$$

That means the section strength out of the bending plane is provided.

Example 34. *Given:* section dimensions and location of reinforcement — according to Draft 49; heavy-weight concrete B30 (R_b = 19 MPa by γ_{b2} = 1,1; E_b = 2,9 · 10^4 MPa); symmetrical reinforcement A-III (R_s = R_{sc} = 365 MPa); longitudinal force N = 6000 kN; bending moment M = 3100 kN·m; design length of the element: in the bending plane l_0 = 16,2 m, out of the bending planel_0 = 10,8 m.

It is required to determine the section strength of the element.

<u>The calculation in the bending plane</u>. From example 33 we have: h'_f = 15 mm; h_0= =1421 mm; a'= 79 mm; N_{cr} = 28 270 kN.

By formula (91) we determine coefficient η:

Considering the element deflection value e is equal to:

$$e = e_0\eta + \frac{h_0 - a'}{2} = \frac{M}{N}\eta + \frac{h_0 - a'}{2} = \frac{3100\cdot 10^6}{6000\cdot 10^3}1{,}27 + \frac{1421-79}{2} = 1327mm.$$

Let's check condition (130):

$$R_b b'_f h'_f = 19\cdot 600\cdot 215 = 2451\cdot 10^3\,H = 2451kN < N = 6000kN,$$

That means the calculation is to be made as for the I-section.

Area of compressed flange overhangs is equal to:

mm^2.

Let's determine values α_n, α_{m1}, α_{ov}, $\alpha_{m,ov}$, δ.

From Table 18 we find ξ_R = 0,523.

As $\xi = \alpha_n - \alpha_{ov}$ = 1,111 – 0,302 = 0,809 > ξ_R = 0,523 so reinforcement area is to be determined by formula (135). For that we determine values α_s and $\xi_1 = \frac{x}{h_0}$ by formulas (136) and (132)

$$\alpha_s = \frac{\alpha_{m1} - \xi(1-\xi/2) - \alpha_{m,ov}}{1-\delta} = \frac{1{,}037 - 0{,}809(1-0{,}809/2) - 0{,}279}{1-0{,}055} = 0{,}292.$$

From Table 18 we find ψ_c = 3,0 and ω = 0,698.

$$\frac{\alpha_s + \psi_c\alpha_s + \alpha_{ov} - \alpha_n}{2} = \frac{0{,}292 + 3\cdot 0{,}292 + 0{,}302 - 1{,}111}{2} = 0{,}18;$$

$$\xi_1 = -\frac{\alpha_s + \psi_c\alpha_s + \alpha_{ov} - \alpha_n}{2} + \sqrt{\left(\frac{\alpha_s + \psi_c\alpha_s + \alpha_{ov} - \alpha_n}{2}\right)^2 + \alpha_s\psi_c\omega} = -0{,}18 +$$

$$+\sqrt{0{,}18^2 + 0{,}292\cdot 3\cdot 0{,}698} = 0{,}602,$$

So

$$A_s = A'_s = \frac{R_b b h_0}{R_s}\frac{\alpha_{m1} - \xi_1(1-\xi_1/2) - \alpha_{m,ov}}{1-\delta} = \frac{19\cdot 200\cdot 1421}{365}\frac{1{,}037 - 0{,}602(1-0{,}602/2) - 0{,}279}{1-0{,}055} = 5278mm^2.$$

We take $A_s = A'_s$ = 5630 mm^2 (7 ∅ 32).

The calculation out of the bending plane is to be made similar to Example 33.

RING SECTIONS

Example 35. *Given:* section with internal radius r_1 = 150 mm, external radius — r_2 = 250 mm; heavy-weight concrete B25 (R_b = 16 MPa by γ_{b2} = 1,1); longitudinal reinforcement A-III (R_s = =R_{sc} = 365 MPa); its section area $A_{s,tot}$ = 1470 mm^2 (13 ∅ 12); longitudinal force from total load N = 1200 kN, its eccentricity relating to the center of gravity of the section considering the element deflection is e_0 = 120 mm.
It is required to check the section strength.
Calculation. Let's calculate the area of the ring section:
mm^2
Relative area of concrete compressed zone:

$$\xi_{cir} = \frac{N + R_s A_{s,tot}}{R_b A + 2{,}7 R_s A_{s,tot}} = \frac{1200 \cdot 10^3 + 365 \cdot 1470}{16 \cdot 125600 + 2{,}7 \cdot 365 \cdot 1470} = 0{,}502;$$

mm.
As 0,15 < ξ_{cir} = 0,502 < 0,6 so section strength is to be checked according to condition (138):

$$\left(R_b A r_m + R_s A_{s,tot} r_s\right)\frac{\sin \pi \xi_{cir}}{\pi} + R_s A_{s,tot} r_s \left(1 - 1{,}7\xi_{cir}\right)\left(0{,}2 + 1{,}3\xi_{cir}\right) =$$

$$= \left(16 \cdot 125600 \cdot 200 + 365 \cdot 1470 \cdot 200\right)\frac{\sin\left(180^\circ \cdot 0{,}502\right)}{3{,}14} + 365 \cdot 1470 \cdot 200\left(1 - 1{,}7 \cdot 0{,}502\right)\left(0{,}2 + 1{,}3 \cdot 0{,}502\right) =$$

$$= 175{,}4 \cdot 10^6\, N \cdot mm = 175{,}4\, kN \cdot m > Ne_0 = 1200 \cdot 0{,}12 = 144\, kN \cdot m,$$

that is section strength is provided.

ROUND SECTIONS

Example 36. *Given:* a section with diameter D = 400 mm; a = 35 mm; heavy-weight concrete B25 (R_b = 13 MPa by γ_{b2} = 0,9; E_b = 2,7 · 10^4 MPa); longitudinal reinforcement A-III (R_s =R_{sc}= = 365 MPa; E_s = 2 · 10^5 MPa); its section area $A_{s,tot}$ = 3140 mm^2 (10 ∅ 20); longitudinal forces and bending moments: from dead loads and long-term loads N_l = 400 kN·m; from all loads N = =600 kN, M = 140 kN·m; design length of the element l_0 = 4 m.
It is required to examine the section strength.
Calculation. Let's calculate:

Area of the round section $A = \frac{\pi D^2}{4} = \frac{3{,}14 \cdot 400^2}{4} = 125\,600$ mm^2;

Radius of the section inertia $i = \frac{D}{4} = \frac{400}{4} = 100$ mm;

Element elasticity $l_0 / i = \frac{4000}{100} = 40 > 14.$

So the calculation is made considering deflection of the element according to 3.54 and value N_{cr} is to be determined by formula (92). For that we determine:

$$r_s = \frac{D}{2} - a = \frac{400}{2} - 35 = 165 \text{ mm};$$

$$\varphi_l = 1 + \beta\frac{M_{1l}}{M_1} = 1 + \beta\frac{M_l + N_l r_s}{M + N r_s} = 1 + 1\frac{100 + 400 \cdot 0{,}165}{140 + 600 \cdot 0{,}165} = 1{,}695$$

[здесь β = 1,0 (see Table 16)];

$e_0 = M / N = 140 / 600 = 0{,}233\text{“} = 233$ mm

As $e_0 / D = \frac{233}{400} = 0{,}583$ > $\delta_{e,min}$ = 0,5 – 0,01 l_0/D – 0,01 R_b so we take δ_e = e_0/D = 0,583.

Inertia moments of the concrete section and all reinforcement are:

$$I = \frac{\pi D^4}{64} = \frac{3{,}14 \cdot 400^4}{64} = 1256 \cdot 10^6 \text{ mm}^4;$$

$$I_s = \frac{A_{s,tot} r_s^2}{2} = \frac{3140 \cdot 165^2}{2} = 42{,}74 \cdot 10^6 \text{ mm}^4;$$

$$\alpha = \frac{E_s}{E_b} = \frac{2 \cdot 10^5}{2{,}7 \cdot 10^4} = 7{,}4.$$

Then

$$N_{cr} = \frac{6{,}4E_b}{l_0^2}\left[\frac{I}{\varphi_l}\left(\frac{0{,}11}{0{,}1+\delta_e}+0{,}1\right)+\alpha I_s\right] = \frac{6{,}4 \cdot 2{,}7 \cdot 10{,}4}{4000^2}\left[\frac{1256 \cdot 10^6}{1{,}695}\left(\frac{0{,}11}{0{,}1+0{,}583}+0{,}1\right)+\right.$$

$$\left.+7{,}4 \cdot 42{,}74 \cdot 10^6\right] = 5505 \cdot 10^3\, H = 5505\, kN.$$

Coefficient η we determine by formula (91):

$$\eta = \frac{1}{1-\dfrac{N}{N_{cr}}} = \frac{1}{1-\dfrac{600}{5505}} = 1{,}12.$$

Section strength is to be examined by means of the diagram of Draft 41.

According to values $\alpha_n = \dfrac{N}{R_b A} = \dfrac{600 \cdot 10^3}{13 \cdot 125\,600} = 0{,}367,$

$\alpha_s = \dfrac{R_s A_{s,tot}}{R_b A} = \dfrac{365 \cdot 3140}{13 \cdot 125\,600} = 0{,}702$ и $\dfrac{a}{D} = \dfrac{35}{400} = 0{,}0875$ we find at the diagram $\alpha_m = 0{,}51$.

As $\alpha_m R_b A r$ = 0,51 · 13 · 125 600 · 200 = 167 · 10^6 N·mm = 167 kN·m > $Ne_0\eta = 600 \cdot 0.233 \times$ $\times 1.12 = 156.6$ kN·m, the section strength is provided.

Example 37. Due to the data of example 36 it is necessary to choose reinforcement using the diagram of Draft 41.

Calculation. From example 36 i = 100 mm, A = 125 600 mm^2, r_s = 165 mm. As l_0/i = =4000/100 40 > 35 so we choose reinforcement considering the element deflection determining value N_{cr} by formula (92).

In the first approximation we have $A_{s,tot}$ = 0,01 A = 1256 m^2, thereafter

$$I_s = \frac{A_{s,tot} r_s^2}{2} = \frac{1256 \cdot 165^2}{2} = 17{,}1 \cdot 10^6 \text{ mm}^4.$$

From example 36 φ_l = 1,695, δ_e = 0,583, I = 1256 · 10^6 mm^4.

Then

$$N_{cr} = \frac{6{,}4E_b}{l_0^2}\left[\frac{I}{\varphi_l}\left(\frac{0{,}11}{0{,}1+\delta_e}+0{,}1\right)+\alpha I_s\right] = \frac{6{,}4 \cdot 2{,}7 \cdot 10^4}{4000^2}\left[\frac{1256 \cdot 10^6}{1{,}695}\left(\frac{0{,}11}{0{,}1+0{,}583}+0{,}1\right)+7{,}4 \cdot 17{,}1 \cdot 10^6\right] =$$

$$= 0{,}0108\left(193{,}4 \cdot 10^6 + 126{,}5 \cdot 10^6\right) = 3455 \cdot 10^3\, H = 3455\, kN.$$

Coefficient is $\eta = \dfrac{1}{1-\dfrac{N}{N_{cr}}} = \dfrac{1}{1-\dfrac{600}{3455}} = 1{,}21.$

Due to values $\alpha_n = \dfrac{N}{R_b A} = \dfrac{600 \cdot 10^3}{13 \cdot 125\,600} = 0{,}367,$

$\alpha_m = \dfrac{Ne_0\,\eta}{R_b A r} = \dfrac{600 \cdot 10^3 \cdot 233 \cdot 1{,}21}{13 \cdot 125\,600 \cdot 200} = 0{,}518$ we find α_s = 0,74, thereafter

$$A_{s,tot} = \alpha_s \frac{R_b A}{R_s} = 0{,}74 \frac{13 \cdot 125\,600}{365} = 3310 \text{ mm}^2.$$

As determined reinforcement is more than the one accepted in the first approximation ($A_{s,tot}$ = =1256 mm^2) so value $A_{s,tot}$ = 3310 mm^2 is determined with „reserve" and it can be a bit decreased after value N_{cr} is specified more exact.

We take $A_{s,tot} = \dfrac{1256+3310}{2} = 2283$ mm^2 and make similar calculation:

$$I_s = \frac{2283 \cdot 165^2}{2} = 31{,}08 \cdot 10^6 \text{ mm}^4;$$

$$N_{cr} = 0{,}0108\left(193{,}4 \cdot 10^6 + 7{,}4 \cdot 31{,}08 \cdot 10^6\right) = 4573 \text{ kN};$$

$$\eta = \frac{1}{1 - \frac{600}{4573}} = 1{,}151.$$

Due to values $\alpha_m = 0{,}518 \frac{1{,}151}{1{,}21} = 0{,}493$, $\alpha_n = 0{,}367$ and $\frac{a}{D} = 0{,}1$ at the diagram of Draft 41 we find $\alpha_s = 0{,}68$.

$$A_{s,tot} = 0{,}68 \frac{13 \cdot 125\,600}{365} = 3042 \text{ mm}^2.$$

We take $A_{s,tot} = 3142$ mm^2 (10 ∅ 20).

ELEMENTS WORKING IN SKEW BENDING

Example 38. *Given:* rectangular section of the column with dimensions b = 400 mm, h = 600 mm; heavy-weight concrete B25 (R_b = 16 MPa by γ_{b2} = 1,1); longitudinal reinforcement A-III ($R_s = R_{sc}$ = 365 MPa) located in the section due to Draft 50; in the section act both longitudinal force N = 2600 kN and bending moments at the same time: in the plane parallel to the dimension h, – M_x = 240 kN·m and in the plane parallel to dimension b, – M_y = 182,5 kN·m; moments M_x and M_y are given considering the column deflection.
It is required to examine the section strength.

Черт. 50. For the calculation examples 38 and 40
I — borders of the compressed zone in the first approximation; *II* — final border of the compressed zone

Calculation. The strength is to be checked according to Item 3.74. Symmetry axes which are parallel to dimensions h and b we symbolize x and y. Let's determine limit moments M_x^0 and M_y^0. For that we determine distributed reinforcement A_{sx} и A_{sy}. Due to Draft 50 $A_{s1,x} = 0$, n_x = =0, A_{s0} = 804,3 mm^2 (∅ 32), $A_{s1,y}$ = 314,2 mm^2 (∅ 20),

$$\beta = \frac{M_x}{M_y}\frac{h_x}{h_y} = \frac{240}{182{,}5}\frac{400}{600} = 0{,}877.$$

$$A_{sx} = A_{s1,x}\left(n_x + 1\right) + \left(2A_{s0} - A_{s1,x} - A_{s1,y}\right)\frac{\beta}{1+\beta} = 0 + \left(2 \cdot 804{,}3 - 324{,}2\right)\frac{0{,}877}{1 + 0{,}877} = 605 mm^2;$$

$A_{s,tot} = 3845$ mm^2 (4 ∅ 32 + 2 ∅ 20);

$$A_{sy} = \frac{A_{s,tot}}{2} - A_{sx} = \frac{3845}{2} - 605 = 1318 \text{ mm}^2.$$

When determining moment M_x^0 which acts in the plane of axis x due to Item 3.63 we take: $A_{sl} = A_{sy}$ = 1318 mm^2; $A_{st} = A_{sx}$ = 605 mm^2; h = 600 mm; b = 400 mm.

$$\delta_1 = a_1 / h = 50/600 = 0{,}083;$$

$$R_b bh = 16 \cdot 400 \cdot 600 = 3840 \cdot 10^3 \text{ N};$$

$$\alpha_{sl} = \frac{R_s A_{sl}}{R_b bh\left(0{,}5 - \delta_1\right)} = \frac{365 \cdot 1318}{3840 \cdot 10^3\left(0{,}5 - 0{,}083\right)} = 0{,}3;$$

$$\alpha_{st} = \frac{R_s A_{st}}{R_b bh} = \frac{365 \cdot 605}{3840 \cdot 10^3} = 0{,}06;$$

$$\alpha_{n1} = \frac{N}{R_b bh} = \frac{2600 \cdot 10^3}{3840 \cdot 10^3} = 0{,}677.$$

From Table 18 we find ω = 0,722, ξ_R = 0,55.

As $\xi = \frac{\alpha_{n1} + \alpha_{sl}}{1 + 2\alpha_{sl} / \omega} = \frac{0,677 + 0,3}{1 + 2 \cdot 0,3 / 0,722} = 0,534$ < ξ_R = 0,55 so value M_x^0 is to be determined by formula (117) due to $\xi_1 = \frac{\xi}{\omega} = \frac{0,534}{0,722}$ = 0,74:

$$M_x^0 = R_b bh^2 \left[0,5\xi(1 - 0,5\xi) + \alpha_{sl}(\xi_1 - \delta_1) \times (1 - \xi_1 - \delta_1) - 0,05\alpha_{sl}\xi_1^2 + \alpha_{st}(1 - 2\delta_1)\right] =$$
$$= 3840 \cdot 10^3 \cdot 600[0,5 \cdot 0,534(1 - 0,5 \cdot 0,534) + 0,3(0,74 - 0,083)(1 - 0,74 - 0,083) -$$
$$- 0,05 \cdot 0,3 \cdot 0,74^2 + 0,06(1 - 2 \cdot 0,083)] = 464,7 \cdot 10^6\, N \cdot mm = 464,7\, kN \cdot m.$$

When determining moment M_y^0 which acts in the plane of axis y, we take: A_{sl} = A_{sx} = 605 mm^2; A_{st} = A_{sy} = 1318 mm^2; h = 400 mm; b = 600 mm; $\delta_1 = \frac{a_1}{h} = \frac{50}{400} = 0,125.$

$$\alpha_{sl} = \frac{R_s A_{sl}}{R_b bh(0,5 - \delta_1)} = \frac{365 \cdot 605}{3840 \cdot 10^3 (0,5 - 0,125)} = 0,153;$$

$$\alpha_{st} = \frac{R_s A_{st}}{R_b bh} = \frac{365 \cdot 1318}{3840 \cdot 10^3} = 0,125.$$

As $\xi = \frac{\alpha_{n1} + \alpha_{sl}}{1 + 2\alpha_{sl} / \omega} = \frac{0,677 + 0,153}{1 + 2 \cdot 0,153 / 0,722} = 0,583$ > ξ_R = 0,55 so value M_y^0 is to be determined by formula (118) calculating:

$$\alpha_{na} = 1 + \frac{R_s A_{s,tot}}{R_b bh} = 1 + \frac{365 \cdot 3845}{3840 \cdot 10^3} = 1,365;$$

$$\xi_{1R} = \frac{\xi_R}{\omega} = \frac{0,55}{0,722} = 0,762;$$

$$\alpha_{mR} = 0,5\xi_R(1 - \xi_R) + \alpha_{sl}(\xi_{1R} - \delta_1) \times (1 - \xi_{1R} - \delta_1) - 0,05\alpha_{sl}\xi_{1R}^2 + \alpha_{st}(1 - 2\delta_1) =$$
$$= 0,5 \cdot 0,55(1 - 0,55) + 0,153(0,762 - 0,125) \times (1 - 0,762 - 0,125) - 0,05 \cdot 0,153 \cdot 0,762^2 +$$
$$+ 0,125(1 - 2 \cdot 0,125) = 0,224;$$

$$\alpha_{nR} = \xi_R + \alpha_{sl}(2\xi_{1R} - 1) = 0,55 + 0,153(2 \cdot 0,762 - 1) = 0,63.$$

$$M_y^0 = R_b bh^2 \alpha_{mR} \frac{\alpha_{na} - \alpha_{n1}}{\alpha_{na} - \alpha_{nR}} = 3840 \cdot 10^3 \cdot 400 \cdot 0,224 \frac{1,365 - 0,677}{1,365 - 0,63} = 322 \cdot 10^6\, N \cdot mm = 322\, kN \cdot m.$$

As $\alpha_s = \frac{R_s A_{s,tot}}{R_b bh} = 0,365$ so section strength is to be checked according to the diagram of Draft 42, *a*, *б* corresponding to α_s = 0,2 and α_s = 0,4. At both diagrams the point with coordinates M_x / M_x^0 = 240/464,7 = 0,516 and M_y / M_y^0 = 182,5/322 = 0,566 lies inside of the area bounded by means of the curve corresponding to parameter α_{n1} = 0,677 and by coordinate axes.
That means the section strength is provided.

Example 39. *Given:* the column section, materials characteristics and values of longitudinal forces from all loads — see Example 33; in the section at the same time there are bending moments in the plane parallel to the dimension h, – M_x = 3330 kN·m and in the plane parallel to dimension b, – M_y = 396 kN·m; moments M_x and M_y are given considering the column deflection.
It is required to check the section strength.
Calculation. The strength is to be checked according to Item 3.75. Let's determine limit moment M_x^0 acting in the plane of the symmetry axis x going in the rib. Due to Example 33 the right part of condition (131) is 5847 kN·m, and so

$$M_x^0 = 5847 - \frac{N(h_0 - a')}{2} = 5847 - 2500\frac{1,421 - 0,079}{2} = 4170\, kN \cdot m.$$

Limit moment M_y^0 acting in the plane of the symmetry axis y normal to the rib we determine as for a rectangular section which consists of two flanges due to Item 3.63. So according to Draft 49 we have: h = 600 mm; b = 2 · 215 = 430 mm.
Let's determine distributed reinforcement A_{sl} и A_{st}:
$A_{s1,l} = 804{,}3$ mm^2 (∅ 32); $\eta_l = 3$;
$A_{s,tot} = 11\,260$ mm^2 (14 ∅ 32);
$A_{sl} = A_{s1,l}(n_l + 1) = 804{,}3\ (3 + 1) = 3220$ mm^2;
$A_{st} = A_{s,tot}/2 - A_{sl} = 11\,260/2 - 3220 = 2410$ mm^2.
From Table 18 we find $\omega = 0{,}698$ and $\xi_R = 0{,}523$.
$R_b bh = 19 \cdot 430 \cdot 600 = 4902 \cdot 10^3$ N;
$\delta_1 = a_1/h = 0{,}083$;

$$\alpha_{sl} = \frac{R_s A_{sl}}{R_b bh(0{,}5-\delta_1)} = \frac{365 \cdot 3220}{4902 \cdot 10^3(0{,}5-0{,}083)} = 0{,}576;$$

$$\alpha_{n1} = \frac{N}{R_b bh} = \frac{2500 \cdot 10^3}{4902 \cdot 10^3} = 0{,}51;$$

$$\alpha_{st} = \frac{R_s A_{st}}{R_b bh} = \frac{365 \cdot 2410}{4902 \cdot 10^3} = 0{,}179;$$

$$\xi = \frac{\alpha_{n1} + \alpha_{sl}}{1 + 2_{\ sl}/\omega} = \frac{0{,}51 + 0{,}576}{1 + 2 \cdot 0{,}576/0{,}698} = 0{,}41 < \xi_R = 0{,}523.$$

Value M_y^0 is to be determine by formula (117) after calculating $\xi_1 = \frac{\xi}{\omega} = \frac{0{,}41}{0{,}698} = 0{,}59$:

$$M_y^0 = R_b bh^2\left[0{,}5\xi(1-\xi) + \alpha_{sl}(\xi_1 - \delta_1)\times(1-\xi_1-\delta_1) - 0{,}05\alpha_{sl}\xi_1^2 + \alpha_{st}(1-2\delta_1)\right] =$$
$$= 19 \cdot 430 \cdot 600^2\left[0{,}5 \cdot 0{,}41(1-0{,}41) + 0{,}576(0{,}59-0{,}083)(1-0{,}59-0{,}083) -\right.$$
$$\left. - 0{,}05 \cdot 0{,}576 \cdot 0{,}59^2 + 0{,}179(1-2 \cdot 0{,}083)\right] = 1029 \cdot 10^6\ H \cdot мм = 1029\ kN \cdot m$$

Let's check the section strength taking b = 200 mm, h = 1500 mm.
As $\alpha_s = \frac{R_s A_{s,tot}}{R_b bh} = \frac{365 \cdot 11\,260}{19 \cdot 200 \cdot 1500} = 0{,}721,$ so section strength is to be checked according to diagrams of Draft 44, *б*, *в*, corresponding to $\alpha_s = 0{,}6$ и $\alpha_s = 1{,}0$.
At both diagrams the point with coordinates M_x / M_x^0 = 3330/4170 = 0,8 and M_y / M_y^0 = =396/1029 = 0,385 lies within the area which is bounded by a curve corresponding to the parameter $\alpha_{n1} = N/(R_b bh) = 2500 \cdot 10^3/(19 \cdot 200 \cdot 1500) = 0{,}44$ and by coordinate axes.
That means the section strength is provided.

Example 40. *Given:* rectangular section of the column with dimensions b = 400 mm, h = 600 mm; heavy-weight concrete B25 (R_b = 16 MPa by $\gamma_{b2} = 1{,}1$); longitudinal reinforcement A-III (R_s = 365 MPa) due to Draft 50; in the section at the same time there is the longitudinal force N = 2600 kN and bending moments acting in the plane parallel to dimension h, $- M_x$ = 250 kN·m and in the plane parallel to dimension b, M_y = 200 kN·m; bending moments M_x and M_y are given considering the column deflection.
It is required to examine the section strength using formulas of Item 3.76 for the general calculation case.
Calculation. All rods symbolized by numbers as it's shown on Draft 50. Through the center of gravity of rod *5* we draw an axis x parallel to dimension h = 600 mm and axis y parallel to dimension b.
Angle θ between axis y and the line bounding the compressed zone we take as by the calculation of elastic body as regards the; that means:

$$tg\,\theta = \frac{M_y}{M_x}\frac{I_x}{I_y} = \frac{M_y}{M_x}\left(\frac{h}{b}\right)^2 = \frac{200}{250}\left(\frac{600}{400}\right)^2 = 1{,}8.$$

Taking value x_1 — with dimensions of the compressed zone along the section side h for each rod it is possible to determine the ration $\xi_i = x/h_{0i}$ by formula $\xi_i = \dfrac{x_1}{a_{yi}tg\theta + a_{xi}}$, where a_{xi} and a_{yi} are distances from the i-rod to the most compressed side of the section in the lines of axes x and y.

Due to values ξ_i we determine stress σ_{si}, taking $\sigma_{sc,u} = 400$ MPa, $\omega = 0{,}722$ (see Table 18):

$$\sigma_{si} = \frac{\sigma_{sc,u}}{1-\dfrac{\omega}{1{,}1}}\left(\frac{\omega}{\xi_i}-1\right) = \frac{400}{1-\dfrac{0{,}722}{1{,}1}} \times \left(\frac{0{,}722}{\xi_i}-1\right) = 1160\left(\frac{0{,}722}{\xi_i}-1\right) \text{ (MPa)}$$

At the same time if $\sigma_{si} > R_s = 365$ MPa, what is equivalent to condition $\xi_i < \xi_R = 0{,}55$ (see Table 18) so we take $\sigma_{si} < R_s = 365$ MPa.

If $\sigma_{si} < -R_{sc} = -365$ MPa we take $\sigma_{si} = -365$ MPa.

The last condition after we insert the equations for σ_{si} into it looks like:

$$\xi_i > \frac{0{,}722}{1-\dfrac{365}{1160}} = 1{,}054.$$

Then we determine the sum of forces in all rods $\sum A_{si}\sigma_{si}$.

We take equation $x_1 = h = 600$ mm in the first approximation and make mentioned calculations; the results of the calculations are given in the following table:

Number of the rod	A_{si}, mm^2	a_{yi}, mm	a_{xi}, mm	a_{yi}tg$\theta + a_{xi}$, mm (tg$\theta = 1{,}8$)	$x_1 = 600$ mm			$x_1 = 660$ mm		
					ξ_i	σ_{si}, MPa	$A_{si}\sigma_{si}$, H	ξ_i	σ_{si}, MPa	$A_{si}\sigma_{si}$, N
1	804,3	350	50	680	0,882	–210	–168 900	0,971	–297	–238 877
2	804,3	50	50	140	4,29	–365	–293 570	4,714	–365	–293 570
3	314,2	350	300	930	0,645	138	43 360	0,71	20	6284
4	314,2	50	300	390	1,54	–365	–114 683	1,692	–365	–114 683
5	804,3	350	550	1180	0,508	365	293 570	0,56	339	272 658
6	804,3	50	550	640	0,937	266	213 944	1,031	–348	–279 896
							$\sum A_{si}\sigma_{si} = -26\,280$ N			$\sum A_{si}\sigma_{si} = -648\,080$ N

As $\dfrac{x_1}{tg\theta} = \dfrac{600}{1{,}8} = 333$ mm < b = 400 mm so compressed zone has triangle form and its area is:

$$A_b = \frac{x_1^2}{2tg\theta} = \frac{600^2}{2\cdot 1{,}8} = 100\,000 \text{ mm}^2.$$

Let's check condition (154):

$$R_b A_b - \Sigma A_{si}\sigma_{si} = 16\cdot 100000 + 26280 = 1626\cdot 10^3\,N = 1626\,kN < N = 2600\,kN,$$

That means area of compressed zone is decreased.

Let's increase value x_1 up to 660 mm and determine $\sum A_{si}\sigma_{si}$ (see the table of the present example).

By $x_1 > h$ and $x_1/\text{tg}\theta = 660/1{,}8 = 367$ mm < b = 400 mm the compressed zone has a trapezoid form and its area is:

$$A_b = \frac{x_1^2}{2tg\theta} - \frac{(x_1 - h)^2}{2tg\theta} = \frac{660 \cdot 367}{2} - \frac{(660 - 600^2)}{2 \cdot 1{,}8} = 121100 - 1000 = 120100 mm^2.$$

As $R_bA_b - \sum A_{si}\sigma_{si}$ = 16 · 120 100 + 648 080 = 2570 · 10^3 N = 2570 кН ≈ N = 2600 kN, so condition (154) is met.

Let's determine moments of internal forces relating to axes y and x. For that let's determine static moments of the section area of compressed zone relating to these axes:

$$S_{bx} = \frac{x_1^2}{2tg\theta}\left(a_{x5} - \frac{x_1}{3}\right) + \frac{(x_1 - h)^2}{2tg\theta}\left(- a_{x5} + \right.$$

$$\left. + \frac{x_1 - h}{3} + h\right) = 121100\left(550 - \frac{660}{3}\right) + 1000\left(-550 + \frac{660 - 600}{3} + 600\right) = 40036000 mm^3;$$

$$S_{by} = \frac{x_1^2}{2tg\theta}\left(a_{y5} - \frac{x_1 / tg\theta}{3}\right) - \frac{(x_1 - h)^2}{2tg\theta}\left(a_{y5} - \frac{x_1 - h}{3tg\theta}\right) =$$

$$= 121100\left(350 - \frac{367}{3}\right) - 100\left(350 - \frac{660 - 600}{3 \cdot 1{,}8}\right) = 27912000 mm^3.$$

Then $M_{xu} = R_bS_{bx} - \Sigma A_{si}\sigma_{si}(a_{x5} - a_{xi}) = 16 \cdot 40036000 - [-238877(550 - 50) - 293570 \times$
$\times (550 - 50) + 6284(550 - 300) - 114683(550 - 300)] = 933.9 \cdot 10^6$ N · mm = 934 kN · m;

$$M_{yu} = R_bS_{by} - \Sigma A_{si}\sigma_{si}(a_{y5} - a_{yi}) = 16 \cdot 27912000 - [-293570(350 - 50) -$$
$$- 114683(350 - 50) - 279896(350 - 50)] = 653 \cdot 10^6\ N \cdot mm = 653 kN \cdot m.$$

Moments of external forces relating to axes y and x are:

$$M_{x1} = M_x + N\left(\frac{h}{2} - 50\right) = 250 \cdot 10^6 + 2600 \cdot 10^3\left(\frac{600}{2} - 50\right) = 900 \cdot 10^6\ N \cdot mm;$$

$$M_{y1} = M_y + N\left(\frac{b}{2} - 50\right) = 200 \cdot 10^6 + 2600 \cdot 10^3\left(\frac{400}{2} - 50\right) = 590 \cdot 10^6\ N \cdot mm.$$

As $M_{xu} > M_{x1}$, a $M_{yu} > M_{y1}$ so section strength is provided.

CACLULATION OF INCLINED SECTIONS

Example 41. *Given:* a column of a multistory frame work with the section dimensions b = 400 mm, h = 600 mm; $a = a'$ = 50 mm; heavy-weight concrete B25 (R_{bt} = 0,95 MPa by γ_{b2} = 0,9); stirrups located along the column surfaces made of reinforcement A-III, diameter 10 mm (R_{sw} = 255 MPa; A_{sw} = 157 mm^2), spacing s = 400 mm; bending moments in the top and the bottom support sections are M_{sup} = 350 kN·m, M_{inf} = 250 kN·m and stretch the left and the right surface of the column; longitudinal force N = 572 kN; column length (distance between support sections) l = 2,8 m.

It is required the strength of inclined sections of the column as regards the shear force.

Calculation. $h_0 = h - a$ = 600 − 50 = 550 mm. The calculation is made according to Item 3.31 considering recommendations of Item 3.53.

Shear force in the in the column is:

$$Q = \frac{M_{\sup} + M_{\inf}}{l} = \frac{350 + 250}{2{,}8} = 214 \text{ kN}.$$

As shear force is constant along the column length so the projection length of inclined section is taken maximum possible; tat is equal to

$$c_{\max} = \frac{\varphi_{b2}}{\varphi_{b3}}h_0 = \frac{2}{0{,}6}0{,}55 = 1{,}833\text{“} < l = 2{,}8 \text{ m}$$

Let's determine coefficient φ_n:

$$\varphi_n = 0{,}1\frac{N}{R_{bt}bh_0} = 0{,}1\frac{572\ 000}{0{,}95\cdot 400\cdot 550} = 0{,}27 < 0{,}5; \quad \varphi_f = 0.$$

As $c = c_{max}$, $Q_b = Q_{b,min} = \varphi_{b3}(1 + \varphi_n)R_{bt}bh_0 = 0{,}6\ (1 + 0{,}27)0{,}95 \cdot 400 \cdot 550 = 159{,}2 \cdot 10^3$ H < <Q = 214 kN so the stirrups are required due to the calculation.

Value q_{sw} is to be determined by formula (55):

$$q_{sw} = \frac{R_{sw}A_{sw}}{s} = \frac{255\cdot 157}{400} = 100{,}1 \text{ N/mm.}$$

Let's check condition (57):

$$\frac{Q_{b,\min}}{2h_0} = \frac{159{,}2\cdot 10^3}{2\cdot 550} = 144{,}7\,н/\text{“ “} > q_{sw} = 100{,}1 \text{ N·mm}$$

As condition (57) is not met so we determine value by the following formula

$M_b = 2h_0^2\, q_{sw}\, \varphi_{b2} / \varphi_{b3}$,

So

$$Q_b = \frac{M_b}{c} = \frac{2h_0^2\, q_{sw}\, \varphi_{b2} / \varphi_{b3}}{h_0\, \varphi_{b2} / \varphi_{b3}} = 2h_0 q_{sw} = 2\cdot 550\cdot 100{,}1 = 110{,}1\cdot 10^3 \text{ N}$$

c_0 is taken equal to $c_0 = 2h_0 = 2 \cdot 550 = 1100$ mm, and $Q_{sw} = q_{sw}c_0 = 100{,}1 \cdot 1100 = 110{,}1 \cdot 10^3$ N.

Let's check condition (50):

$Q_b + Q_{sw} = 110{,}1\cdot 10^3 + 110{,}1\cdot 10^3 = 220{,}2\cdot 10^3\, H > Q = 214$ kN

That means the section strength as regards the shear force is provided.

Центрально- и внецентренно растянутые элементы

CENTRALLY TENSILE ELEMENTS

- **(3.26).** During calculation of eccentric tensile reinforced concrete elements the following condition must be met:

  $$N \le R_s A_{s,tot}, \qquad (156)$$

  where $A_{s,tot}$ — section area of total longitudinal reinforcement.

ECCENTRICTENSILE ELEMENTS

CALCULATION OF RECTANGULAR SECTIONS NORMAL TO THE LONGITUDINAL AXIS OF THE ELEMENT, IF LONGITUDINAL FORCE IS LOCATED IN THE SYMMETRY AXIS

- **(3.27).** Calculation of rectangular sections of eccentric compressed elements with reinforcement concentrated at the most tensile and at compressed (the lest tensile) surfaces must be made according to the location of longitudinal force N:

  a) if longitudinal force N is applied between resultant forces in reinforcement S and S' (Draft 51, *a*), that means by $e' \le h_0 - a'$, — so the calculation is made due to the following conditions:

  $$Ne' \le R_s A_s\left(h_0 - a'\right); \qquad (157)$$

  $$Ne \le R_s A_s'\left(h_0 - a'\right); \qquad (158)$$

b) if longitudinal force N is applied beyond the area between resultant forces in reinforcement S and S' (Draft51, *б*), that means by $e' > h_0 - a'$, — so the calculation is made due to the following condition

$$Ne \le R_b bx\left(h_0 - 0{,}5x\right) + R_{sc}A'_s\left(h_0 - a'\right), \qquad (159)$$

at the same time the height of compressed zone x is determined by formula

$$x = \frac{R_s A_s - R_{sc}A'_s - N}{R_b b}. \qquad (160)$$

Draft 51. Forces scheme and diagram of stresses in the section normal to longitudinal axis of eccentric tensile reinforced concrete element by its calculation as regards the strength

a — longitudinal force N applied between resultant forces in reinforcement S and S';

б — the same, beyond the area between resultant forces in reinforcement S and S'

If calculated by formula (160) value $x > \xi_R h_0$, so it is necessary to insert value $x = \xi_R h_0$ where ξ_R is determined by Tables 18 and 19 into the formula (159).

If $x < 0$so section strength is to be checked according to condition (157).

By symmetrical reinforcement the strength is examined according to condition (157) independently of value e'.

Note. If by $e' > h_0 - a'$ the height of compressed zone determined without considering compressed reinforcement $x = \frac{R_s A_s - N}{R_b b}$, is less than $2a'$ so it is possible to increase design bearing capacity after calculations by formulas (159) and (160) without considering compressed reinforcement.

- Required quantity of longitudinal reinforcement is determined in the following manner:
  a) by $e' \le h_0 - a'$ it's determined section area of reinforcement S and S' by the following formulas:

$$A = \frac{Ne'}{R_s\left(h_0 - a'\right)}; \qquad (161)$$

$$A'_s = \frac{Ne}{R_s\left(h_0 - a'\right)}; \qquad (162)$$

б) by $e' > h_0 - a'$ it's determined section area of stretched reinforcement A_s by formula

$$A_s = \frac{\xi b h_0 R_b + N}{R_s} + A'_s\frac{R_{sc}}{R_s}, \qquad (163)$$

where ξ is taken by Table 20 due to value

$$\alpha_m = \frac{Ne - R_{sc}A'_s\left(h_0 - a'\right)}{R_b b h_0^2}. \qquad (164)$$

At the same time condition $\alpha_m \le \alpha_R$ must be met (see Table 18 and 19). Otherwise it is necessary to increase the section of compressed reinforcement A'_s, to increase concrete class or to increase the section dimensions.

If $\alpha_m < 0$ so section area of tensile reinforcement A_s is determined by formula (161).

Symmetrical reinforcement area independently of value e' is chosen due to formula (161).

Note. By $e' > h_0 - a'$ required quantity of reinforcement determined by formula(161) can be decreased if value ξ, determined due to Table 20 without considering compressed reinforcement, that means due to value $\alpha_m = \frac{Ne}{R_b b h_0^2}$, is less than $2a'/h_0$. In that case section area of stretched reinforcement A_s is determined by the following formula

$$A_s = \frac{N\left(e + \zeta h_0\right)}{R_s \zeta h_0}, \tag{165}$$

where ζ is determined by formula 20 according to value $\alpha_m = \frac{Ne}{R_b b h_0}$.

GENERAL CASE OF CALCULATION OF NORMAL SECTIONS OF ECCENTRIC COMPRESSED ELEMENT (BY ANY SECTIONS, EXTERNAL FORCES AND ANY REINFORCEMENT)

- Calculation of sections of the eccentric tensile element in general case (see Draft 45) must be made due to the following condition

$$N\bar{e}' \leq \Sigma \sigma_{si} S_{si} - R_b S_b, \tag{166}$$

where $\bar{e}'$ — distance from the longitudinal force N to the axis parallel to the line which bounds the compressed zone and going through the most distant from the mentioned line point of compressed zone;

S_b — static moment of the concrete compressed zone area relating to the mentioned axis;

S_{si} — static moment of the section area of the longitudinal reinforcement *i*-rod relating to the mentioned axis;

σ_{si} — stress in the *i*-rod of longitudinal reinforcement.

Compressed zone height x and stresses σ_{si} are determined due to the combined solution of equations (154) and (155) replacing sign „minus" by sign „plus" in front of N.

Except formulas (154) and (155) by skew eccentric tension it is necessary to meet additional requirement to determine the location the concrete zone borders: points of application of external longitudinal force, the resultant of compression forces in concrete and reinforcement and the resultant of forces in tensile reinforcement must belong to the straight line (see Draft 45).

CALCULATION OF SECTIONS INCLINED TO THE LONGITUDINAL AXIS OF THE ELEMENT

- Calculation of inclined sections of eccentric tensile elements as regards the shear force is made as for bending moments according to Items 3.28—3.41. At the same time value M_b in Item 3.31 is determined by the following formula

$$M_b = \varphi_{b2}\left(1 + \varphi_f - \varphi_n\right) R_{bt} b h_0^2, \tag{167}$$

where $\varphi_n = 0{,}2\frac{N}{R_{bt} b h_0}$, but no more than 0,8;

value $Q_{b,min}$ is taken equal to φ_{b3} $(1 + \varphi_f - \varphi_n) R_{bt} b h_0$. Besides in all formulas of Items 3.29, 3.40 and 3.41 coefficient φ_{b4} is replaced by φ_{b4} $(1 - \varphi_n)$.

Calculation of inclined sections of eccentric tensile elements as regards bending moment is made as for bending elements in compliance with Items 3.42—3.45. At the same time the height of compressed zone in inclined section is determined considering tensional force N by formula (160) or due to Item 3.80.

In case if condition $e' < h_0 - a'$ is met design moment in inclined section can be determined as the moment of external forces located on one side of the inclined section under review relating to the axis going through the center of gravity of reinforcement S'.

EXAMPLES OF CALCULATION

Example 42. *Given:* a stretched leg of a two-leg column with the cross section dimensions b = 500 mm, h = 200 mm; $a = a'$ = 40 mm; longitudinal reinforcement A-III ($R_s = R_{sc}$ = 365 MPa); its section area $A_s = A'_s$ = 982 mm^2 (2 ∅ 25); heavy-weight concrete B25 (R_b = 16 MPa by γ_{b2} = 1,1); longitudinal force N = 44 kN; maximum bending moment M = 43 kN · m.
It is required to examine the strength of the normal section.

Calculation. h_0 = 200 – 40 = 160 mm;

$$e_0 = \frac{M}{N} = \frac{43 \cdot 10^6}{44 \cdot 10^3} = 977 \text{ mm};$$

$$e' = e_0 + \frac{h}{2} - a' = 977 + \frac{200}{2} - 40 = 1037 \text{ mm};$$

$$e = e_0 - \frac{h}{2} + a = 977 - \frac{200}{2} + 40 = 917 \text{ mm}.$$

As there is symmetrical reinforcement so the strength is examined due to condition (157):

$$R_s A_s (h_0 - a') = 365 \cdot 982 (160 - 40) = 40{,}1 \cdot 10^6 \; N \cdot mm < Ne' = 44 \cdot 10^3 \cdot 1037 = 45{,}6 \cdot 10^6 \; N \cdot mm$$

That means condition (157) is not met. As e' = 1037 mm > $h_0 - a'$ = 120 mm, and the height of compressed zone x, determined by formula (160) without considering compressed reinforcement is

$$x = \frac{R_s A_s - N}{R_b b} = \frac{365 \cdot 982 - 44 \cdot 10^3}{16 \cdot 500} = 40\text{“ “} < 2a' = 2 \cdot 40 = 80 mm$$

according to the note to Item 3.78 we check the strength according to Formula (159), taking x = 40 mm and A'_s = 0:

$$R_b b x (h_0 - 0{,}5x) = 16 \cdot 500 \cdot 40 (160 - 0{,}5 \cdot 40) = 40{,}6 \cdot 10^6 \; N \cdot mm > Ne = 44 \cdot 10^3 \cdot 917 = 40{,}4 \cdot 10^6 \; N \cdot mm$$

that is the strength the normal section is provided.

Example 43. *Given:* a rectangular section with dimensions b = 1000 mm, h = 200 мм; $a = a'$ = =35 mm; heavy-weight concrete B15 (R_b = 7,7 MPa by γ_{b2} = 0,9); longitudinal reinforcement A-III ($R_s = R_{sc}$ = 365 MPa); reinforcement section area S' A'_s = 1005 mm^2; tension force N = =160 kN; bending moment M = 116 kN·m.
It is required to determine section area of reinforcement S.
Calculation. h_0 = 200 – 35 = 165 mm;

$$e_0 = \frac{M}{N} = \frac{116 \cdot 10^6}{160 \cdot 10^3} = 725 \text{ mm};$$

$$e = e_0 - \frac{h}{2} + a = 725 - \frac{200}{2} + 35 = 660 \text{ mm};$$

$$e' = e_0 + \frac{h}{2} - a' = 725 + \frac{200}{2} - 35 = 790 \text{ mm}.$$

As e' = 790 mm $h_0 - a'$ = 165 – 35 = 130 mm so let’s determine required section area of stretched reinforcement according to Item 3.796.

Let’s determine the following value:

$$\alpha_m = \frac{Ne - R_{sc} A'_s (h_0 - a')}{R_b b h_0^2} = \frac{160 \cdot 10^3 \cdot 660 - 365 \cdot 1005 (165 - 35)}{7{,}7 \cdot 1000 \cdot 165^2} = 0{,}276.$$

As 0 < α_m < α_R = 0,44 (see Table 18) so value A_s is to be determined by formula (163). For that we find ξ = 0,33 by α_m = 0,276 according to Table 20.

$$A_s = \frac{\xi b h_0 R_b + N}{R_s} + A_s' \frac{R_{sc}}{R_s} = \frac{0{,}33 \cdot 1000 \cdot 165 \cdot 7{,}7 + 160 \cdot 10^3}{365} + 1005 = 2592 mm^2.$$

We take A_s = 3079 mm² (5 ∅ 28).

Example 44. *Given:* a rectangular section with dimensions b = 1000 mm, h = 200 мм; $a = a'$ = =40 mm; heavy-weight concrete B15 (R_b = 7,7 MPa by γ_{b2} = 0,9); longitudinal reinforcement A-III ($R_s = R_{sc}$ = 365 MPa); tension force N = 532 kN; bending moment M = 74 kN·m.

It is required to determine section area of symmetrical longitudinal reinforcement.

Calculation. $h_0 = h - a$ = 200 − 40 = 160 mm;

$$e_0 = \frac{M}{N} = \frac{74 \cdot 10^6}{532 \cdot 10^3} = 139 \text{ mm};$$

$$e = e_0 - \frac{h}{2} + a = 139 - \frac{200}{2} + 40 = 79 \text{ mm};$$

$$e' = e_0 + \frac{h}{2} - a' = 139 + \frac{200}{2} - 40 = 199 \text{ mm}.$$

As it is symmetrical reinforcement so section area is to be determined by formula (161):

$$A_s = A_s' = \frac{Ne'}{R_s(h_0 - a')} = \frac{532 \cdot 10^3 \cdot 199}{365(160 - 40)} = 2417 \text{ mm}^{2.}$$

As e' = 199 mm > $h_0 - a'$ = 120 mm so according to note of Item 3.79 it is possible to decrease value A_s.

Let's determine value ξ without considering compressed reinforcement. For that we determine value α_m:

$$\alpha_m = \frac{Ne}{R_b b h_0^2} = \frac{532 \cdot 10^3 \cdot 79}{7{,}7 \cdot 1000 \cdot 160^2} = 0{,}213.$$

Due to Table 20 by α_m = 0,213 we find ξ = 0,24 and ζ = 0,88. As $\xi = 0{,}24 < \frac{2a'}{h_0} = \frac{2 \cdot 40}{160} = 0{,}5,$ we determine value A_s by formula (165):

$$A_s = A_s' = \frac{N(e + \zeta h_0)}{R_s \zeta h_0} = \frac{532 \cdot 10^3 (79 + 0{,}88 \cdot 160)}{365 \cdot 0{,}88 \cdot 160} = 2275 \text{ mm}^2.$$

We take $A_s = A'_s$ = 2281 mm² (6 ∅ 22).

Example 45. *Given:* a stretched leg of the two-leg column with the section dimensions b = 500 mm, h = 200 mm; $a = a'$ = 40 mm; heavy-weight concrete B25 (R_{bt} = 1,15 MPa by γ_{b2} = 1,1); stirrups located along the leg surfaces made of reinforcement A-III (R_{sw} = 285 MPa); longitudinal tension force N = 44 kN; shear force Q = 143 kN; the distance between the connecting strips of the two-leg column is l = 600 mm.

It is required to determine diameter and spacing of stirrups.

Calculation. $h_0 = h - a$ = 200 − 40 = 160 mm. The calculation is made according to Item 3.33a considering recommendations of Item 3.81.

Value M_b is to be determined by formula (167) by φ_{b2} = 2 (see Table 21), φ_f = 0 and

$$\varphi_n = 0{,}2 \frac{N}{R_{bt} b h_0} = 0{,}2 \frac{44\,000}{1{,}15 \cdot 500 \cdot 160} = 0{,}096 < 0{,}8:$$

$$M_b = \varphi_{b2}(1 - \varphi_n) R_{bt} b h_0^2 = 2(1 - 0{,}096) 1{,}15 \cdot 500 \cdot 160^2 = 26{,}6 \cdot 10^6 \text{ N·mm}$$

As it is constant shear force between the connection strips of the two-leg column so the projection length of inclined section is taken maximum as it's possible, that is:

$$c = c_{max} = \frac{\varphi_{b2}}{\varphi_{b3}} h_0 = \frac{2}{0{,}6} 160 = 533 \text{ мм} < l = 600 \text{ mm}.$$

Then

$Q_b = \frac{M_b}{c} = \frac{26{,}6 \cdot 10^6}{533} = 49{,}9 \cdot 10^3 \text{H} = Q_{b,\min}$

As $2h_0 = 2 \cdot 160 = 320$ mm < $c = 533$ mm so we take $c_0 = 2h_0 = 320$ mm.

Определим коэффициент æ :

$æ = \frac{Q - Q_b}{Q_b} = \frac{143 - 49{,}9}{49{,}9} = 1{,}866.$

As $\frac{c}{c_0} = \frac{533}{320} = 1{,}667 <$ æ $= 1{,}866 < \frac{c}{h_0} = \frac{533}{160} = 3{,}33$ so stirrups quantity is determined by formula (63):

$q_{sw} = \frac{\left(Q - Q_b\right)^2}{M_b} = \frac{(143 - 49{,}9)^2}{26{,}6} = 325{,}9$ kN/m.

Maximum allowable stirrups quantity according to Item 3.30 is:

$s_{\max} = \frac{\varphi_{b4}\left(1 - \varphi_n\right) R_{bt} b h_0^2}{Q} = \frac{1{,}5(1 - 0{,}096)1{,}15 \cdot 500 \cdot 160^2}{143 \cdot 10^3} = 139{,}6$ mm

Besides in compliance with Item 5.58 the stirrups quantity must be no more than $2h = 2 \cdot 200 =$
$=400$ mm.

We take stirrups quantity $s = 100$ mm < s_{max}, so

$A_{sw} = \frac{q_{sw} s}{R_{sw}} = \frac{325{,}9 \cdot 100}{290} = 112{,}4$ mm^2.

We take two stirrups with diameter 10 mm ($A_{sw} = 157$ mm^2).

Elements working in torsion with bending (spatial sections calculation)

ELEMENTS OF RECTANGULAR SECTION

- **(3.37)** During calculation of elements working in torsion with bending the following requirement must be met:

$$T \leq 0{,}1 R_b b^2 h, \tag{168}$$

where b, h — are the larger and the less dimensions of the element surfaces.

At the same time value R_b for concrete more than B30 is taken as for concrete class B30.

- Spatial sections are calculated as regards combined action of torsion and bending moments by location of compressed zone at the element surface perpendicular to the plane of acting of bending moment (Scheme 1 of Draft 52).

Besides, spatial sections are calculated as regards combined action of torsion moments and shear forces by location of compressed zone at the element surface parallel to the plane of acting of bending moment (Scheme 2 of Draft 53).

Draft 52. Forces scheme in spatial section of the 1st scheme

Draft 53. Forces scheme in spatial section of the 2nd scheme

- Calculation of the spatial section according to the 1st scheme is made due to the following condition

$$T + M\frac{b}{c_1} \leq \left(R_s A_{s1}\frac{b}{c_1} + q_{sw1} c_1 \delta_1\right)\left(h_0 - 0{,}5x_1\right), \tag{169}$$

At the same time value R_sA_{s1} is taken no more than $2q_{sw1}b+\frac{M}{h_0-0{,}5x_1}$, and value q_{sw1} — no more than $\frac{1{,}5}{b}\left(R_sA_{s1}-\frac{M}{h_0-0{,}5x_1}\right)$.

In condition (169):

c_1 — The length of the projection onto the longitudinal axis of the element of the line bounding the compressed zone of the spatial section; the most disadvantageous value c_1 in general case is determined by means of step-by-step approximations and is taken no more than $2h + b$ and no more than $b\sqrt{\frac{2}{\delta_1}}$, at the same time the spatial section must not be beyond the element borders and its part with one-valued and zero value T;

A_{s1} — section area of all longitudinal rods located at the stretched surface with width b;

q_{sw1} — the force in cross rods located at the stretched surface with the width b per a unit length of the element equal to:

$$q_{sw1}=\frac{R_{sw}A_{sw1}}{s_1}, \tag{170}$$

where A_{sw1} — is section area of one cross rod;

s_1 — distance between cross rods;

$$\delta_1=\frac{b}{2h+b}. \tag{171}$$

Torsion moment T and bending moment M are taken in the cross section going through the center of the spatial section (Draft 54, *a*).

The height of compressed zone x_1 is determined by formula

$$x_1=\frac{R_sA_{s1}-R_{sc}A'_{s1}}{R_bb}, \tag{172}$$

where A'_{s1} — section area of all compressed rods located at the surface with the width b.

If $x_1 < 2a'$ in condition (169) so it's taken $x_1 = 2a'$. If $x_1 > \xi_R h_0$ (where ξ_R — see item 3.14) so it is necessary to examine the strength of the normal section according to Item 3.15.

Condition (169) must be also met if in the quality of values A_{s1} and A_{sw1} we take section areas of longitudinal and cross reinforcement located in the compressed by bending zone; in that case value M is taken with sign „minus”.

Note. Limitation for value R_sA_{s1} in condition (169) can be used in formula (172) that can cause the increase of design bearing capacity.

Draft 54. Determination of the bending and torsion moments of the shear force acting in the spatial section
a — 1st scheme; *б* — 2nd scheme

- The strength as regards longitudinal reinforcement located at stretched by bending surface (1st Scheme) should be examined:
  a) for continuous beams and consoles by location of the spatial section at the support as well as for any elements loaded by concentrated forces and torsion moments by location

of the spatial section at points of application of these forces and moments on the side of the part with larger bending moments (Draft 55) — according to the following condition

$$R_s A_{s1}\left(h_0 - 0{,}5x_1\right) \geq M_{\max} + \frac{\left(T - 0{,}5Qb\right)^2}{4\delta_1 q_{sw1} b\left(h_0 - 0{,}5x_1\right)}, \tag{173}$$

where M_{max} — is maximum bending moment at the beginning of the spatial section;

T, Q — torsion moment and shear force in the section with the larger bending moment.

At the same time $q_{sw1}\, b(h_0 - 0{,}5x_1)$ is taken no more than $\frac{0{,}6T}{\sqrt{\delta_1}}$;

Draft 55. Location of design spatial sections of the 1st scheme in the beam loaded by concentrated forces
1, 2 — design spatial sections;
M_1, T_1, Q_1 — design forces for the spatial section *1*;
M_2, T_2, Q_2 — the same for the spatial section *2*

б) for elements loaded only by distributed load q if in the span section with maximum bending moment M_{max} there is a bending moment T_0, — due to the following condition

$$R_s A_{s1}\left(h_0 - 0{,}5x_1\right) \geq M_{\max} + \frac{T_0^2}{4\delta_1 q_{sw1} b\left(h_0 - 0{,}5x_1\right) - 2t^2 / q}, \tag{174}$$

where t — distributed torsion moment per a unit length of the element.

The strength as regards longitudinal reinforcement located at the compressed by bending surface should be examined for free supported beams according to the condition (173), taking forces T and Q in the support section by $M_{max} = 0$.

If on the considered parts the following condition is not met

$$T < 0{,}5Qb, \tag{175}$$

so longitudinal reinforcement can be examined only according to the condition of clear bending (see Item 3.15).

The strength as regards cross reinforcement located at any surface with the width b should be examined due to the following condition

$$q_{sw1} b\left(h_0 - 0{,}5x_1\right) \geq \frac{T}{2\sqrt{2\delta_1}}. \tag{176}$$

Note. Longitudinal reinforcement determined due to the condition (173) can be decreased if the most disadvantageous spatial section with the projection length c_1, equal to:

$$c_1 = 2b\frac{R_s A_{s1}\left(h_0 - 0{,}5x_1\right) - M_{\max}}{T - 0{,}5Qb}, \tag{177}$$

goes beyond the length of the element or its part with one-valued or zero values T. In that case the calculation is made by means of general method in compliance with Item 3.84 by decreased projection length c_1.

- Calculation of the spatial section as regards the 2nd scheme (see Draft 53) is made due to the following condition

$$T + 0{,}5Qb \leq \left(R_s A_{s2}\frac{h}{c_2} + q_{sw2} c_2 \delta_2\right)\left(b - 2a_2\right), \tag{178}$$

At the same time $R_s A_{s2}$ value is taken no more than $2q_{sw2}h$ and value q_{sw2} — no more than $\frac{1{,}5R_s A_{s2}}{h}$.

In condition (178):

A_{s2} — section area of all tensile longitudinal rods located at the surface wit the width h, parallel to the bending plane;

c_2 — projection length on the longitudinal axis of the element of the line bounding compressed zone of the spatial section; the most disadvantageous value c_2 is determined by the following formula

$$c_2 = 2h\frac{R_s A_{s2}\left(b-2a_2\right)}{T+0{,}5Qb} \qquad (179)$$

and it's taken no more than $h\sqrt{\frac{2}{\delta_2}}$ and no more than $2b + h$; at the same time spatial section must not go beyond the element and its part with one-valued and zero value T;

$$q_{sw2} = \frac{R_{sw} A_{sw2}}{s_2}; \qquad (180)$$

where A_{sw2} — section area of one cross rod located at the surface with the width h;

s_2 — the distance between cross rods located at the surface with the width h;

$$\delta_2 = \frac{h}{2b+h}; \qquad (181)$$

a_2 — the distance from the surface with the width h to the axis of longitudinal rods located at this surface.

Torsion moment T and cross force Q are taken in the cross section going through the center of gravity of the spatial section (see Draft 54, *б*).

In case if condition (175) is met calculation of the spatial section due to the 2nd scheme is not made. Instead of this calculation it is necessary to make a calculation of inclined sections according to Items 3.31–3.38 without considering bend-up bars. At the same time in corresponding formulas to shear force Q it is added value $\frac{3T}{b}$ (where T — is torsion moment in the same cross section like Q) and value q_1 is multiplied by coefficient $1+3\frac{e_q}{b}$ (where e_q — is eccentricity of lateral distributed load q which cause the element torsion). In case if $T < 0{,}25Qb$ so it is possible to consider bend-up bars during calculation of inclined sections.

- Required by the calculation as regards the 2nd scheme density of stirrups $\frac{A_{sw2}}{s_2}$ can be determined by the following formulas:

by $\varphi_t = \dfrac{T+0{,}5Qb}{R_s A_{s2}(b-2a_2)\sqrt{2\delta_2}} \le 1$

$$\frac{A_{sw2}}{s_2} = 0{,}5\frac{R_s A_{s2}}{R_{sw} h}\varphi_t; \qquad (182)$$

by $1{,}75 \ge \varphi_t > 1$

$$\frac{A_{sw2}}{s_2} = 0{,}5\frac{R_s A_{s2}}{R_{sw} h}\varphi_t^2, \qquad (183)$$

where T, Q — is maximum value of corresponding torsion moment and shear force on the part under review.

By $\varphi_t > 1б75$ it is necessary to increase the section area of reinforcement A_{s2} or dimension of the section b so that condition $\varphi_t \le 1{,}75$ was met.

If lateral load is applied within the height of the section and acts towards the stretched zone so vertical stirrups quantity must be increased in comparison with the value determined by formulas (182) and (183) in compliance with the calculation as regards the break according to Item 3.97.

T-, I- AND OTHER SECTIONS WITH RE-ENTRANT CORNER

- Cross section of the element must be divided into several rectangles (Draft 56) at the same time if the height of flange overhangs or the width of the rib are variable so it is necessary to take their average values.

Draft 56. Dividing into rectangles of sections with re-entrant angles during calculation as regards the torsion with bending

Dimensions of the cross section must meet the following requirement

$$T \le 0{,}1 R_b \Sigma b_i^2 h_i, \tag{184}$$

where h_i, b_i — are the larger and the less dimensions of each rectangle.

Besides it is necessary to meet requirement of Item 3.30.

If within the section height there are flanges whose top or bottom surfaces are not the prolongations of corresponding surfaces of the element so the calculation of spatial sections is made without considering these flanges as for the element of rectangular section in compliance with Items 3.83–3.87.

- Calculation of a spatial section as regards combined action of torsion and bending moments (the 1st scheme of Draft 57) is made due to the following condition

$$T + M\frac{b'_f}{c_1} \le R_s A_{s1}\frac{b'_f}{c_1}(h_0 - 0{,}5x_1) + q_{sw1} b_f (h_{0w} - 0{,}5x_1), \tag{185}$$

at the same time value $R_s A_{s1}$ is taken no more than $2q_{sw1}b_f + \dfrac{M}{h_0 - 0{,}5x_1}$.

In condition (185):

b'_f, b_f — The width of compressed and stretched surfaces normal to the bending plane;

c_1 — the length of projection onto the longitudinal axis of the element of the line bounding compressed zone of the spatial section; value c_1 is taken corresponding to the value of the slope angle of a spatial crack to the element axis 45° on all surfaces of the element (without considering x_1) by formula

$$c_1 = 2h + b + b_f + (b_f - b) + (b'_f - b) = 2h + 2b_f + b'_f - 2b,$$

At the same length c_1 must not go beyond the element and its part with one-valued or zero values T;

A_{s1} — section area of all longitudinal rods located in the stretched by the bending zone;

x_1 — the height of compressed zone determined as for the flat cross section of bending moment (see Item 3.20);

$$q_{sw1} = \frac{R_{sw} A_{sw1}}{s_1}; \tag{186}$$

A_{sw1}, s_1 — area of cross rods located in one plane in the stretched by bending, and a spacing of these rods;

h_{0w} — the distance from the compressed zone to the resultant of forces in cross rods of the stretched zone.

Draft 57. Location scheme of the compressed zone in the spatial section of the 1st scheme of the reinforced concrete element of T- and I-sections working in torsion with bending
C — center of gravity of longitudinal stretched reinforcement

Torsion moment T and bending moment M in condition (185) are taken in the cross section going through the center of the spatial section.

In case of changing of cross rods spacing s_1 within the length c_1 it is necessary to consider average spacing on the part with the length b_f located symmetrically relating to the cross section going through the spatial section.

Besides it is necessary to check the strength of the normal section in compliance with Item 3.20.

Note. Limitation for value R_sA_{s1} by using of condition (185), can be taken into account during calculation of the compressed zone height x_1 that will cause the decrease of design bearing capacity.

- Calculation of the spatial section as regards combined action of the torsion moment and shear force (2nd scheme, Draft 58) is made due to the condition

$$T + 0{,}5Qb_{f,\min} \le R_s A_{s2} \frac{h}{c_2}(b_0 - 0{,}5x_2) + q_{sw2}h(b_{0w} - 0{,}5x_2), \qquad (187)$$

at the same time condition R_sA_{s2} is taken no more than $2q_{sw2}h$.

In condition (187):

$b_{f,min}$ — the less width of the element flange or if there is one flange – the width of the rib

A_{s2} — area of all longitudinal rods located in the stretched zone by the present scheme;

c_2 — the length of the projection onto the longitudinal axis of the element of the line bounding compressed zone of the spatial section determined by formula:

$$c_2 = 2b_{f,\min} + h + 2b_{ov},$$

where b_{ov} — width of the flange overhang located in the stretched zone, at the same time length c_2 must not go beyond the element or its part with one-valued or with the zero values of T;

x_2 — the height of compressed zone determined as for the flat cross section of bending element by the present scheme of compressed zone location, at the same time compressed overhang of the flange is not taken into account if it sticks out beyond the surface of flange which has less width or beyond the surface of the rib if there is one flange;

$$q_{sw2} = \frac{R_{sw}A_{sw2}}{s_2}; \qquad (188)$$

A_{sw2}, s_2 — section area of one cross rod located in the stretched zone by the present scheme along the total height h, and its spacing;

b_0, b_{0w} — distance from the lateral compressed surface of the flange with the width $b_{f,min}$ to the resultant of forces in longitudinal rods with the area A_{s2} and in the cross rods with the area A_{sw2}.

Draft 58. Location schemes of the compressed zone in the spatial section of the 2nd scheme of reinforced concrete element of T-, I- and L-sections working in torsion with bending
C — center of gravity of longitudinal stretched reinforcement

Torsion moment T and shear force Q in condition (187) are taken in the cross section going through the center of the spatial section.

In case of changing of the cross rods spacing s_2 within the length c_2 it is necessary to consider average spacing on the part with the length h located symmetrically relating to the cross section going through the center of the spatial section.

Besides it is necessary to check the strength of inclined section in compliance with Item 3.31.

RING SECTION ELEMENTS WITH LONGITUDINAL REINFORCEMENT DISTRIBUTED ALONG THE CIRCLE

- Dimensions of the ring cross section of the element must meet the following requirement

$$T \le 0{,}08 R_b \, \pi\left(r_2^3 - r_1^3\right), \qquad (189)$$

where r_1, r_2 — are internal and external radius of the ring section.

The calculation of the spatial section (Draft 59) is made due to the following condition

$$T + M\frac{b}{c} \le M_u \frac{b}{c} + q_{sr} r_s c\beta, \qquad (190)$$

where b, c - is the projection length of the line bounding compressed zone onto the cross section of the element and on its longitudinal axis (see Draft 59). Value b is taken equal to

$$b = 2\sqrt{r_2^2 - \left(r_s \cos\pi\,\xi_{cir}\right)^2}, \qquad (191)$$

value c is determined due to Item 3.91;

Draft 59 Spatial section of the reinforced concrete element of the ring cross section working in torsion with bending

ξ_{cir} - relative area of the concrete compressed zone determined by formula (137), or by $\xi_{cir} < 0{,}15$ – by formula (140) by $N = 0$;

M_u - limit bending moment by clear bending taken equal to the right part of condition (138) or (139);

$$q_{sr} = \frac{R_{sw} A_{sr}}{s}; \ (192)$$

A_{sr}, s – section area of the rod of spiral (ring) reinforcement and spacing of the spiral lapping (rings spacing);

β - coefficient determined by formula

$$\beta = 1 - \frac{b}{2\pi r_s\left(1-\xi_{cir}\right)} \times \left[\frac{\sin\pi\xi_{cir}}{\pi\left(1-\xi_{cir}\right)} + \cos\pi\xi_{cir}\right] \ (193)$$

or according to Draft 60.

Draft 60. Diagram for determination of coefficient β during calculation of elements of ring cross-section as regards torsion with bending

Torsion moment T and bending moment M in condition (190) are taken in the cross section going through the center of the spatial section.
Besides, it is necessary to check condition (190) as for clear torsion multiplying value M_u by the ration $\frac{4\pi r_s q_{sr}}{R_s A_{s,\,tot}}$, where $A_{s,\,tot}$ is section area of total longitudinal reinforcement.

Value q_{sr} in condition (190) is taken no more than $\frac{1{,}5 R_s A_{s,tot}}{2\pi r_s}\left(1 - \frac{M}{M_u}\right)$.

- Condition (190) is checked for spatial sections where projection length c doesn't go beyond the part with one-valued and zero value T and is no more than $c_{\max} = 2\pi r_2(1-\xi_{cir})$.

For elements with the constant section as regards the length it is recommended to check spatial sections beginning from the normal section with maximum value T, and by constant values T – from the section with maximum value $M=M_{мах}$. In the last case the most disadvantageous value c is:

$$c = 2b\frac{M_u - M_{\max}}{T - Qb/2}.$$

For elements with variable section as regards the length it is recommended to check several spatial sections located in different places on the length and by values c, equal to:

$$c = 2b\frac{M_u - M}{T},$$

At the same time the projection length must not go beyond the length of the element. Dimensions of the cross section are taken corresponding to the center of the spatial section.

CALCULATION EXAMPLES

Example 46. *Дано*: a collar beam of the end frame floor of the multistory production building loaded by distributed load q=154,4 kN/m and distributed torsion moments t=34,28 kN·m/m; cross section of the collar beam at the support – due to Draft 61, *a*; diagram of torsion moments caused by vertical dead loads and long-term loads – due to Draft 61, *б*; diagram of bending moments and cross forces caused by the most disadvantageous for the support section combination of vertical loads and the wind load – due to Draft 61, *в*, *г*; diagram of bending moments caused by the most disadvantageous combination of vertical loads – due to Draft 61, *д*; heavy-weight concrete B25; longitudinal and cross reinforcement A-III (R_s=R_{sc}=365 MPa; R_{sw}=290 MPa).

Draft 61. For the calculation example 46

It is required to choose vertical and horizontal cross rods and to check the strength of the collar-beam as regards combined action of torsion and bending.

Calculation. As the section has re-entrant angles so we check condition (184) dividing the section into two rectangles with dimensions 800x320 and 155x250 mm and taking R_b = 13 MPa (that is by γ_{b2} = 0,9);

$$0{,}1R_b\sum b_i^2 h_i = 0{,}1\cdot 13\left(320^2\cdot 800 + 155^2\cdot 250\right) = 114{,}3\cdot 10^6 \text{ Н·мм} > \text{T} = 84 \text{ кН·м},$$

that means condition (184) is met.

The calculation of spatial sections is made as for a rectangle sections with dimensions b = 300 mm и h = 800 mm, as bottom surface of the collar-beam and the flange form an angle.

As for the support section 0,5Qb = 0,5·460·0,3 = 69 kN·m < T = 84 kN·m according to Items 3.85 and 3.86 so the calculation of the support section according to the 1st and the 2nd scheme is required.

Required quantity of vertical rods according to the calculation as regards the 2nd scheme is to be determined due to Item 3.87.

First of all we determine coefficients δ_2 и φ_t:

$$\delta_2 = \frac{h}{2b+h} = \frac{800}{2\cdot 300 + 800} = 0{,}571;$$

$$\varphi_t = \frac{T + 0{,}5Qb}{R_s A_{s2}(b - 2a_2)\sqrt{2\delta_2}} = \frac{84\cdot 10^6 + 0{,}5\cdot 460\cdot 10^3\cdot 300}{365\cdot 2304(300 - 2\cdot 50)\sqrt{2\cdot 0{,}571}} = 0{,}851,$$

where A_{s2} = 1609 + 314 + 380 = 2304 mm^2 (2∅32 + ∅20 + ∅22).

As $\varphi_t < 1$ so stirrups quantity is to be determined by formula (182):

$$\frac{A_{sw2}}{s_2} = 0{,}5\frac{R_s A_{s2}}{R_{sw} h}\varphi_t = 0{,}5\frac{365\cdot 2304}{290\cdot 800}0{,}851 = 1{,}54\,\text{mm}.$$

Taking the spacing of vertical stirrups s_2 = 100 mm, we find the area of one stirrup:

$$A_{sw2} = 1{,}54\cdot 100 = 154\ \text{mm}^2.$$

We take stirrups with diameter 14 mm (A_{sw2} = 154 mm^2).

Let's check the strength as regards the longitudinal reinforcement installed at the top stretched surface of the support part of the collar-beam according to Item 3.85a (the 1st scheme).

Due to Draft 61, *a* we find A_{s1} = 3217 mm^2 (4∅32) and A'_{s1}= 1388 mm^2 (2∅20 + 2∅22), a' = =68 mm.

By formula (172) we determine the height of compressed zone x_1 taking R_b = 16 MPa (that is by γ_{b2}= 1.1 as wind load is taken into account):

$$x_1 = \frac{R_s A_{s1} - R_{sc}A'_{s1}}{R_b b} = \frac{365(3217 - 1388)}{16\cdot 300} = 139\,\text{мм} > 2a' = 2\cdot 68 = 136\ \text{мм}.$$

Spacing and diameter of horizontal cross rods of the support part we take the same as for vertical stirrups, that means s_1=100 mm, A_{sw1}=154 mm^2 so

$$q_{sw1} = \frac{R_{sw}A_{sw1}}{s_1} = \frac{290\cdot 154}{100} = 446{,}6\,\text{N/m};$$

$$\delta_1 = \frac{b}{2h + b} = \frac{300}{2\cdot 800 + 300} = 0{,}158;$$

$$h_0 = 800 - 80 = 720\ \text{mm}.$$

Let's check the equation $q_{sw1}b(h_0\text{-}0{,}5x_1)$ = 446,6·300(720-0,5·139)=

= 87,2·10^6 N·mm < $\frac{0{,}6T}{\sqrt{\delta_1}} = \frac{0{,}6\cdot 84\cdot 10^6}{\sqrt{0{,}158}} = 126{,}8\cdot 10^6$ N·mm. So q_{sw} is not to be changed.

Let's check condition (173):

$$M_{\max} + \frac{(T - 0{,}5Qb)^2}{4\delta_1 q_{sw1} b(h_0 - 0{,}5x_1)} = 490\cdot 10^6 + \frac{(84\cdot 10^6 - 0.5\cdot 460\cdot 10^3\cdot 300)^2}{4\cdot 0.158\cdot 87.2\cdot 10^6} =$$

$$494\cdot 10^6\,\text{N·mm} < R_s A_s(h_0 - 0.5x_1) = 365\cdot 3217(720 - 0.5\cdot 139) = 763.8\cdot 10^6\ \text{N·mm}$$

that means there is installed enough longitudinal reinforcement according to the strength condition.

Due to condition (176) let's check the strength as regards horizontal cross reinforcement located on the support part:

$q_{sw1}b(h_0 - 0{,}5x_1)$ = 446,6·300(720-0,5·139)=87,2·10^6 Н·мм > $\frac{T}{2\sqrt{2\delta_1}} = \frac{84}{2\sqrt{2\cdot 0{,}158}} = 74{,}7$ kN·m

That means there is installed enough horizontal cross reinforcement on the support part.

As it's shown on Draft 61, *б*, *д*, in the section with maximum span bending moment there is a torsion moment that's why it is necessary to check the strength as regards longitudinal reinforcement installed at the bottom stretched surface in the middle part of the collar-beam span according to condition (174).

For this part of the collar-beam two top rods ∅ 32 are broken that's why due to Draft 61, *a*, we have A'_{sw} = 1609 mm^2 (2 ∅ 32); a' = 62 mm; A_{s1} = 1388 mm^2 (2 ∅ 20 + 2 ∅22); a = 68 mm.

Let's determine the height of compressed zone x_1, taking R_b = 13 MPa (that is by γ_{b2} =0,9, as wind load is not taken into account):

$$x_1 = \frac{R_s A_{s1} - R_{sc} A'_{s1}}{R_b b} = \frac{365(1388 - 1609)}{13 \cdot 300} < 0.$$

We take $x_1 = 2a'$, so $h_0 - 0{,}5\ x_1 = h - a - a' = 800 - 68 - 62 = 670$ mm.
Horizontal cross rods in the middle part of the span are taken with diameter 14 mm (A_{sw1} = 154 mm^2) and spacing s_1 = 200 mm, so

$$q_{sw1} = \frac{R_{sw} A_{sw1}}{s_1} = \frac{290 \cdot 154}{200} = 223.3 \text{ N/mm}$$

From Draft 61, *б*, *д* we have:

$$T_0 = \frac{2{,}71 - 2{,}45}{2{,}45} 84 = 8{,}9 \text{ kN·m};$$

$$M_{max} = 321 \text{ kN·m}$$

Let's check condition (174):

$$M_{\max} + \frac{T_0^2}{4\delta_1 q_{sw1} b\left(h_0 - 0{,}5x_1\right) - \dfrac{2t^2}{q}} = 321 \cdot 10^6 +$$

$$+ \frac{\left(8{,}9 \cdot 10^6\right)^2}{4 \cdot 0{,}158 \cdot 223{,}3 \cdot 300 \cdot 670 - \dfrac{2 \cdot 34{,}28^2}{154{,}4} 10^6} = 326{,}9 \cdot 10^6 \text{ N} \cdot \text{mm} <$$

$$< R_s A_{s1}\ (h_0 - 0{,}5x_1) = 365 \cdot 1388 \cdot 670 = 339{,}4 \cdot 10^6 \text{ N·mm}$$

That means that according to the strength conditions there is installed enough bottom longitudinal reinforcement.
Let's determine distance l_x from the zero point of the diagram T possible for spacing of horizontal cross rods 200 mm using condition (176). Taking $T = tl_x$ we have $q_{sw1} b(h_0 - 0{,}5\ x_1) =$ $= \dfrac{tl_x}{2\sqrt{2\delta_1}}$, so

$$l_x = \frac{2\sqrt{2\delta_1}\, q_{sw1} b\left(h_0 - 0{,}5x_1\right)}{t} = \frac{2\sqrt{2 \cdot 0{,}158} \cdot 223{,}3 \cdot 0{,}67}{34{,}28} = 1{,}47m$$

Therefore spacing of horizontal rods 100 mm on support parts can be 2,45-1,47 $\approx$ 1 m long.

Example 47. *Given:* a floor beam with the cross section – due to Draft 62, *a*; location of the loads, diagrams of torsion and bending moments as well as diagram of cross forces – due to Draft 62, *б*; heavy-weight concrete B25 (R_b = 13 MPa by γ_{b2} = 0,9); longitudinal and cross reinforcement A-III ($R_s = R_{sc}$ = 365 MPa; R_{sw} = 290 MPa).
It is required to check the strength of the beam as regards combined action of torsion and bending.
Calculation. Let's divide the section into two rectangles 200x400 and 350x400 mm and check condition (184):

$$0{,}1 R_b \sum b_i^2 h_i = 0{,}1 \cdot 13\left(200^2 \cdot 400 + 350^2 \cdot 400\right) = 84{,}5 \cdot 10^6 \text{ Н·мм} > \text{T} = 40 \text{ kN·m}.$$

Черт. 62. For the calculation example 47

Due to Draft 62, *a* we have h_0 = 800 – 50 = 750 mm.
First let's check the strength of the spatial section according to the 2nd scheme due to Item 3.90. At the same time as point loads applied in the middle of the section height cause the break of

the stretched zone of the beam it is necessary to take into account that some vertical stirrups bear the tearing force which is equal (according to Item 3.97).:

$$F_1 = F\left(1-\frac{h_s}{h_0}\right) = 280\left(1-\frac{350}{750}\right) = 149{,}3\text{kN}$$

(where $h_s = 400 - 50 = 350$ mm).

The force per a unit length of the beam in vertical stirrups located at the right surface caused by tearing force F is to be determined by distributing the tearing force on two branches of stirrups and taking the width of the support platform of force F $b = 300$ mm, then

$$a = 2h_s + b = 2\cdot350 + 300 = 1000 \text{ мм} = 1 \text{ м},$$

that means

$$q_{swa} = \frac{F_1/2}{a} = \frac{149{,}3/2}{1} = 74{,}6\text{kN/m}=74{,}6\text{N/mm}.$$

So considered by the calculation of the spatial section value q_{sw2} by $A_{sw2} = 154$ mm^2 (1∅14) and $s_2 = 100$ mm (see Draft 62, *a*) will be:

$$q_{sw2} = \frac{R_{sw}A_{sw2}}{s_2} - q_{swa} = \frac{290\cdot154}{100} - 74{,}6 = 372\text{N}/mm$$

Due to Draft 58, *в* and 62, *a*, we take $b_{f,min} = 200$ mm, $h = 800$ mm, b_{ov}=0, $A_{s2} = 1071$ mm^2 (1∅32 + 1∅12+1∅14).

Then value c_2 will be:

$$c_2 = 2b_{f,\min} + h + 2b_{ov} = 2\cdot200 + 800 = 1200mm$$

Spatial section is located at the support of the beam. As $c_2 < 1{,}94$ m that means spatial section doesn't go beyond the borders of a part with the zero values T so we live $c_2 = 1{,}2$ m.

Design values Q and T are taken at the distance $\frac{c_2}{2}$ from the support that means

$$Q = Q_{\max} - q\frac{c_2}{2} = 297 - 5{,}7\frac{1{,}2}{2} = 293{,}5\,\text{кН};\ \text{T} = 40 \text{ kN·m}$$

As $R_sA_{s2} = 365\cdot1071 = 391\cdot10^3$ N $< 2q_{sw2}h = 2\cdot372\cdot800 = 595\cdot10^3$ N so we live $R_sA_{s2} = 391$ kN.

The height of compressed zone x_2 is determined as for the rectangular section according to Item 3.20, taking for the present scheme $h_0 = b_0 = 200 - 50 = 150$ mm and $b = h = 800$ mm (compressed overhang of a flange is not taken into account).

As $a' = 50$ mm is a great part of $h_0 = 150$ mm so value x_2 is determined without considering compressed zone:

$$x_2 = \frac{R_sA_{s2}}{R_bb} = \frac{391\,000}{13\cdot800} = 37{,}6\text{ мм} < a' = 50 \text{ mm}.$$

Let's check condition (187) taking $b_{ow} = b_o = 150$ mm:

$$R_sA_{s2}\frac{h}{c_2}(b_0 - 0{,}5x_2) + q_{sw2}h(b_{ow} - 0{,}5x_2) = 391\,000\frac{800}{1200}(150 - 0{,}5\cdot37{,}6) + 372\cdot800\times$$

$$\times(150 - 0{,}5\cdot37{,}6) = 73{,}2\cdot10^6\text{ N}\cdot\text{mm} > T + 0{,}5\,Qb_{f,min} = 40 + 0{,}5\cdot293{,}5\cdot0{,}2 = 69{,}35 \text{ kN·m},$$

That means the strength as regards the 2nd scheme is provided.

Let's check the strength of the spatial section as regards the 1st scheme according to по 3.89. Let's take $b'_f = b = 200$mm; $b_f = 350$mm; $A_{s1} = 2526$ mm^2 (3∅32 + 1∅12); $A'_{s1} = 308$ mm^2 (2∅14); $A_{sw1} = 154$ mm^2 (1∅14); $s_1 = 200$ mm.

Let's determine the projection length c_1:

$$c_1 = 2h + 2b_f + b'_f - 2b = 2\cdot800 + 2\cdot350 + 200 - 2\cdot200 = 2100 \text{ mm}.$$

Spatial section is located on the part between the support and the first load near the point of application of this load. As $c_1 > 1{,}94$ m that means that the spatial section goes beyond the beam and we take c_1 = 1,94 m. Design values M and T are taken at the distance $\frac{c_1}{2}$ from the support that means $M = 297 \cdot 0{,}97 - \frac{5{,}7 \cdot 0{,}97^2}{2} = 285{,}4$ kN·m; T = 40 kN·m.

The height of compressed zone is determined as for the rectangular section:

$$x_1 = \frac{R_s A_{s1} - R_{sc} A'_{s1}}{R_b b} = \frac{365(2526 - 308)}{13 \cdot 200} = 311\,\text{mm}$$

At the same time x_1 = 311 mm **<** $\xi_R\, h_o$ = 0,604 · 750 = 453 mm (where ξ_R – see Table. 18);

$$q_{sw1} = \frac{R_{sw} A_{sw1}}{s_1} = \frac{290 \cdot 154}{200} = 223\,\text{N/mm}\,.$$

As $2q_{sw1}\, b_f + \frac{M}{h_0 - 0{,}5x_1}$ = 2 · 223 · 350 · + $\frac{285{,}4 \cdot 10^6}{750 - 0{,}5 \cdot 311}$ = 636,2 · 10^3 N < $R_s\, A_{s1}$ = 365 · 2526 = =922 · 10^3 N, we take $R_s\, A_{s1}$ = 636,9 · 10^3 N.

Let's check condition (185) taking $h_{ow} = h_o$ = 750 mm:

$$R_s A_{s1} \frac{b'_f}{c_1}(h_0 - 0{,}5x_1) + q_{sw1} b_f (h_{ow} - 0{,}5x_1) = 636{,}2 \cdot 10^3 \frac{200}{1940}(750 - 0{,}5 \cdot 311) + 223 \cdot 350 \times (750 - 0{,}5 \cdot 311) =$$

$$= 85{,}4 \cdot 10^6 \text{ N} \cdot \text{mm} > T + M \frac{b'_f}{c_1} = 40 + 285{,}4 \frac{0{,}2}{1{,}94} = 69{,}4\,\text{kN} \cdot \text{m},$$

That means the strength as regards the 1st scheme is provided.

Calculation of reinforced concrete elements as regards local loads

LOCAL COMPRESSION CALCULATION

- **(3.39).** During the calculation as regards the local compression of elements without cross reinforcement the following condition must be met:

$$N \le \psi R_{b,loc} A_{loc1}\,, \tag{194}$$

where N — longitudinal compression force caused by local;

A_{loc1} — compression area (see Draft 63);

ψ — coefficient пequal to:

1.0 – by local load distributed on the compression area;

0.75 by local load uneven distributed on the compression area (under ends of beams, girders, and connection beams);

$R_{b,\,loc}$ — design resistance of concrete against compression determined by formula

$$R_{b,\,loc} = \alpha\varphi_b\, R_b, \tag{195}$$

here $\alpha\varphi_b \ge 1.0$;

α = 1,0 for concrete grade less than B25;

α = 13,5 R_{bt}/R_b for concrete grade B25 and more;

$$\varphi_b = \sqrt[3]{A_{loc2} / A_{loc1}}\,,$$

but no more than the following values:

by the load application scheme due to Draft 63, *а, в, г, е*, and for concrete:

heavy-weight, fine and light-weight concrete of class:

more than B7,5 2,5

B3,5; B5; B7,5 1,5

Light weight concrete of class B2,5 1,2

by the load application scheme due to Draft 63, *б, д, ж* independently on concrete class — 1,0;

R_b, R_{bt} — taken as for (see Position 5 Table 9);

A_{loc2} — расчетная площадь смятия, определяемая в соответствии с п. 3.94.

If condition (194) is not met so it is recommended to use confinement reinforcement in form of welded meshes and to calculate the element in compliance with Item 3.95.

- A part which is symmetrical as regards the compression area is included into the design area A_{loc2} (Draft 63). At the same time the following rules must be followed:

By local loads into design area along the whole width *b* it is necessary to include a part with the length no more than *b* to each side of the local load border Draft 63, *а*);
By local edge load along the whole width of the element design area A_{loc2} is equal to the compression area A_{loc1} (Draft 63, *б*);
By local loads in the support points of collar-beams and beams ends into the design area it is included the with the width equal to the depth of setting of the collar beam or and with the length no more than the distance between the centers of spans adjoining to the beam (Draft 63, *в*);
If the distance between beams is more than a double width of the element so the length of design area is determined as a sum of the beam width and of the double width of the element (Draft 63, *г*);
By local edge load on the element angle (Draft 63, *д*) design area A_{loc2} is equal to the compression area A_{loc1};

Draft 63. Determination of design area A_{loc2} by the calculation as regards local compression by a local load
а — along the whole width of the element; *б* — edge load along the whole width of the element; *в*, *г* — in support points of of the ends of beams and collar-beams; *д* — edge load on the element angle; *е* — on the part of the length and the width of the element; *ж* — edge load в within the wall pier; *и* — on the section of irregular shape; *I* — minimum zone reinforced by meshes by which confinement reinforcement is considered in the calculation

By local load applied on the part of the length and the width of the element design length is taken due to Draft 63, *е*. If there are several loads of the mentioned type design areas are bounded by the lines going through the center of distances between points of application of two neighbor loads;
By local edge load located within the wall pier or an I-section separation wall design area A_{loc2} is equal to the compression area A_{loc1} (черт. 63, *ж)*;
By determination of design area for intricate shape sections it is not necessary to consider the parts which are not connected with the loaded parts and whose safety is not provided (Draft 63, *и*).

Note. By local loads from beams, collar-beams and connection beams working in bending the considered in the calculation depth of the support during determination of A_{loc1} и A_{loc2} is taken no more than 20 cm.

- **(3.41)**. By determination as regards local compression of the elements made of heavy-weight concrete with confinement reinforcement in form of welded cross meshes the following condition must be met:

$$N \le R^*_{b,loc} A_{loc1}, \qquad (196)$$

where A_{loc1} — compression area;

$R^*_{b,loc}$ — prism strength of concrete to local compression determined by the following formula

$$R^*_{b,loc} = R_b \varphi_b + \varphi \mu_{xy} R_{s,xy} \varphi_s, \qquad (197)$$

here $R_{s,xy}$, φ, μ_{xy} — are the same symbols like in Item 3.57;

$\varphi_b = \sqrt[3]{A_{loc2} / A_{loc1}}$, but no more than 3,5;

φ_s — coefficient considering the influence of confinement reinforcement in the local compression zone; for diagrams of Drafts 63, *б, д, ж* it is taken

φ_s = 1.0, at the same time confinement reinforcement is considered in the calculation by the condition that cross meshes are installed on the area no less than the area bounded by a dotted line on the corresponding schemes of Draft 63; for schemes of Draft 63, *а, в, г, е, и* coefficient φ_s is determined by formula

$$\varphi_s = 4{,}5 - 3{,}5\frac{A_{loc1}}{A_{ef}},$$

A_{ef} — concrete area within the contours of confinement reinforcing meshes if they are calculated according to end rods for which the following condition must be met $A_{loc1} < A_{ef} \leq A_{loc2}$.

If the compression area border goes beyond the contours of confinement reinforcing meshes (for example see Draft 63, *а - д, ж, и*) by determination of values A_{loc1} and A_{loc2} the area occupied by a protection layer is not taken into account.

The least depth of confinement reinforcement meshes installation must be determined by formulas:

by loading schemes due to Draft 63, *в - е*

$$h_d = \varphi_d\left(\sqrt{\frac{N}{R_b}} - \sqrt{A_{loc1}}\right); \qquad (198)$$

by loading schemes due to Draft 63, *а, б, ж, и*

$$h_d = \frac{\varphi_d}{b}\left(\frac{N}{R_b} - A_{loc1}\right). \qquad (199)$$

In formulas (198) and (199):

$\varphi_d = 0{,}5$ — by loading schemes due to Draft 63, *а, е, и*;

$\varphi_d = 0{,}75$ — by loading schemes due to Draft 63, *в, г*;

$\varphi_d = 1{,}0$ — by loading schemes due to Draft 63, *б, д, ж*.

The number of meshes is taken no less than two. Besides it is necessary to meet constructive requirements of Item 5.79. At the same time if dimensions of a mesh cell are more than 100 mm or more than 1/4 of the less side of the section, so rods of this mesh of this direction are not taken into account during determination of μ_{xy}.

CALCULATION EXAMPLES

Example 48. *Given:* a steel pole, supported on the reinforced concrete foundation and centrally loaded by force N = 1000 kN (Draft 64); foundation of heavy-weight concrete B 12,5 (R_b = 6,7 MPa by γ_{b2} = 0,9).

It is required to examine the strength of concrete under the pole as regards local compression.

Draft 64. For the calculation 48

The calculation is made according to instructions of Items 3.93 and 3.94.

Design area A_{loc2} is to be determined in compliance with Draft 63, *е*.

Due to Draft 64, we have c_1 = 200 mm < b = 800 mm; a_1 = 200 · 2+300 = 700 mm; $b_1 = 200 \cdot 2 + 200 = 600$ мм; $A_{loc2} = a_1 b_1$ = 700 · 600 = 420000 mm².

Compression area is A_{loc1} = 300 · 200 = 60000 mm². As concrete class is less than B25, α = 1,0.

Coefficient φ_b is:

$$\varphi_b = \sqrt[3]{\frac{A_{loc2}}{A_{loc1}}} = \sqrt[3]{\frac{420000}{60000}} = 1{,}9 < 2{,}5.$$

Let's determine design resistance of concrete against compression by formula (195), taking R_b considering γ_{b9} = 0,9 (see Table 9) as for the concrete structure: R_b = 6,7 · 0,9 = 6,03 MPa:

$$R_{b,\,loc} = \alpha\varphi_b\, R_b = 1 \cdot 9 \cdot 6{,}03 = 11{,}5 \text{ MPa}$$

(where $\alpha\varphi_b$ = 1 · 1,9 = 1,9 > 1,0).

Let's check condition (194), taking ψ = 1,0 as by even distribution of the local load

$$\psi R_{b,loc} A_{loc1} = 1 \cdot 11{,}5 \cdot 60000 = 690000 \text{ N} = 690 \text{ кН} < N = 1000 \text{ kN},$$

that means concrete strength as regards local compression is not provided and that means that it is necessary to use confinement reinforcement. We take confinement reinforcement in form of meshes made of reinforcing wire Bp-1, diameter 3 mm, dimensions of a cell 100×100 mm and spacing along the height s = 100 mm ($R_{s,xy}$ = 375 MPa).

Let's check the strength according to Item 3.95. As φ_b = 1.9 < 3.5, so φ_b = 1.9 is inserted into the calculation.

Coefficient of confinement reinforcement by meshes μ_{xy} is determined by formula (99).

Due to Draft 64 we have: n_x = 5; l_x = 300 mm; n_y = 4; l_y = 400 mm; $A_{sx} = A_{sy}$ = 7,1 mm^2 (∅3); $A_{ef} = l_x\, l_y$ = 300 · 400 = 120000 mm^2 > A_{loc1} = 60 000 mm^2, so

$$\mu_{xy} = \frac{n_x A_{sx} l_x + n_y A_{sy} l_y}{A_{ef}\, s} = \frac{5 \cdot 7{,}1 \cdot 300 + 4 \cdot 7{,}1 \cdot 400}{120000 \cdot 100} = 0{,}00183.$$

By formulas (101) and (100) we determine ψ and φ:

$$\psi = \frac{\mu_{xy} R_{s,xy}}{R_b + 10} = \frac{0{,}00183 \cdot 375}{6{,}7 + 10} = 0{,}041;$$

$$\varphi = \frac{1}{0{,}23 + \psi} = \frac{1}{0{,}23 + 0{,}041} = 3{,}69.$$

Coefficient φ_s is:

$$\varphi_s = 4{,}5 - 3{,}5\, A_{loc1}/A_{ef} = 4{,}5 - 3{,}5 \cdot 60000/120000 = 2{,}75.$$

Specified concrete strength $R^*_{b,loc}$ is determined by formula (197):

$$R^*_{b,loc} = R_b\, \varphi_b + \varphi\mu_{xy}\, R_{s,xy}\, \varphi_s = 6{,}7 \cdot 1{,}9 + 3{,}69 \cdot 0{,}00183 \cdot 375 \cdot 2{,}75 = 19{,}7 \text{ МПа}.$$

Let's check condition (196):

$$R^*_{b,loc}\, A_{loc1} = 19{,}7 \cdot 60000 = 1182 \cdot 10^3\, H > N = 1000 \text{ kN},$$

That means concrete strength is provided.

Let's determine the least depth of meshes setting by formula (198), taking φ_d = 0,5:

$$h_d = \varphi_d \left(\sqrt{\frac{N}{R_b}} - \sqrt{A_{loc1}} \right) = 0{,}5 \left(\sqrt{\frac{1000 \cdot 10^3}{6{,}7}} - \sqrt{60000} \right) = 70{,}7 \text{ mm} < s = 100 \text{ mm}$$

that means it's enough to install two meshes.

CALCULATION AS REGARDS THE PRESSING THROUGH

- **(3.42).** Calculation of slab structures (without cross reinforcement) as regards pressing through by forces evenly distributed on the restricted area must be made according to the following condition

$$F \le \alpha R_{bt}\, u_m\, h_o, \tag{200}$$

where F — pressing through force;

α — coefficient taken equal to:

for heavy-weight concrete 1,00

for fine concrete .... 0,85 for light-weight concrete 0,80

u_m — arithmetic mean value of perimeters of upper base and lower base of the pyramid which is formed by pressing through within the working height of the section.

By determination of u_m and F it's supposed that the punching through takes place on the lateral surface of the pyramid whose less base is the area of application of pressing force and lateral surfaces are inclined at the angle 45° to the horizontal line (Draft 65, *a*).

Pressing through force F is taken equal to the force acting on the pressing pyramid except the loads applied on the larger base of the pressing pyramid (calculation as regards the plane where stretched reinforcement is located) and resisting against the pressing.

If supporting scheme is so that pressing can take place only on the surface of the pyramid with the angle of lateral surfaces inclination more than 45° [for example in foundation pile caps (Draft 65, *б*)] so the right part of condition (200) is determined for actual punching pyramid and is multiplied by h_o/c (where c — is the length of horizontal projection of lateral surface of the pressing pyramid). At the same time value of the bearing capacity is taken no less than the value corresponding to the pyramid by $c = 0{,}4h_o$.

Draft 65. Scheme of the pressing pyramid by angle of inclination of its lateral surfaces to the horizontal line
а —45°; *б* —more than 45°

by installation of stirrups normal to the slab plane within the punching pyramid the calculation must be made due to the following pyramid

$$F \le F_b + 0{,}8\, F_{SW}, \tag{201}$$

But no more than $2F_b$,
where F_b — is the right part of condition (200);
$F_{sw} = 175\Sigma A_{sw}$ — the sum of all cross forces, taken by stirrups which cross lateral surfaces of pressing pyramid (175 MPa — limit pressure in stirrups).

When considering cross reinforcement value F_{sw} must be no less than 0,5 F_b.
It is possible to consider the least value F_{sw} in the calculation by replacement of the right part of condition (201) by $2{,}8F_{sw}$, but no less than F_b.

By location of stirrups on the restricted area close to the point load it is necessary to make additional calculation as regards the pressing of the pyramid with upper base located along the part contours with cross reinforcement according to condition (200) without considering cross reinforcement.
Cross reinforcement must meet requirements of Item 5.75.

CALCULATION AS REGARDS BREAK

- **(3.43).** Calculation of reinforced concrete elements as regards the break caused by the load applied on its bottom surface or within its section height (Draft 66) must be made due to the following condition

$$F\left(1-\frac{h_s}{h_0}\right) \le \Sigma R_{sw} A_{sw}, \tag{202}$$

where F – break force;

h_s – расстояние от уровня передачи отрывающей силы на элемент до центра тяжести сечения продольной арматуры S; при передаче нагрузки через монолитно связанные балки или консоли принимается, что нагрузка передается на уровне центра тяжести сжатой зоны элемента, вызывающего отрыв;

$\Sigma R_{sw}A_{sw}$– sum of cross forces taken by stirrups which are installed in addition to the stirrups required by the calculation of inclined or spatial section in compliance with Items 3.31—3.39, 3.86, 3.87 and 3.90; these stirrups are located along the break zone length equal to:

$$a = 2\, h_s + b, \qquad (203)$$

here b – width of the break force силы F transfer area.

By evenly distributed load q, applied within the section height required stirrups intensity is increased by value $q(1 - h_s/h_o)/R_{sw}$.

- Re-entrant angles in the stretched zone of elements reinforced by intersecting longitudinal rods (Draft 67) must have cross reinforcement enough to take:

a) resultant of forces in longitudinal stretched rods not going into the stretched zone equal to:

$$F_1 = 2R_s A_{s1} \cos\frac{\beta}{2}; \qquad (204)$$

б) 35 percent of resultant forces in all longitudinal stretched rods equal to:

$$F_2 = 0{,}7 R_s A_{s1} \cos\frac{\beta}{2}. \qquad (205)$$

Required by these calculations stretched reinforcement must be located along the length $s = h\ tg\ \frac{3}{8}\beta$.

Sum of forces projections in cross rods (stirrups) located along this length onto the bisectrix of the angle must be no less than sum $F_1 + F_2$,

That means $\Sigma R_{sw}\, A_{sw}\ \cos\theta \geq F_1 + F_2$. (206)

In formulas (204) — (206):

A_s — section area of all longitudinal stretched rods;

A_{s1} — section area of longitudinal stretched rods not anchored in the compressed zone;

β — re-entrant angle in the stretched zone of the element;

ΣA_{sw} — cross section of longitudinal reinforcement within the length s;

θ — angle of slope of cross rods onto the bisectrix of angle β.

Draft 66. Scheme for determination of the break zone length

а — by adjoining of beams; *б* — adjoining of consoles; *I* — center of gravity of compressed zone of the section of adjoining element

Draft 67. Reinforcement of re-entrant angle located in the stretched zone of reinforced concrete element

Calculation of short consoles

- **(3.34).** Calculation of short consoles of columns [$l_1 \leq 0{,}9\ h_0$; (Draft 68)] as regards the cross force to provide the strength of inclined compressed strip between the load and the support must be made due to the following condition

$$Q \leq 0{,}8\ R_b\ b\ l_{sup} \sin^2 \theta (1 + 5\ \alpha\mu_w), \qquad (207)$$

Where right part is taken no less than $3{,}5R_{bt}bh_0$ and no less than $2{,}5R_{bt}bh_0$.

In condition (207):

l_{sup} — the length of the area of the load supporting along the console overhanging length;

θ — angle of slope of design compressed strip onto the horizontal line расчетной $\left(\sin^2\theta = \frac{h_0^2}{h_0^2 + l_1^2}\right)$;

$\mu_w = \frac{A_{sw}}{bs_w}$ — reinforcement coefficient by stirrups located along the console height;

here s_w — the distance between stirrups measured along the normal line to them.

In the calculation it is necessary to consider the stirrups horizontal and inclined at the angle no more than 45 degrees to the horizontal line.

Compression stress in the points of application of the load on the console must be no more than $R_{b,loc}$ (see Item 3.93).

For short consoles included into the hard joint of frame structure value l_{sup} in equation (207) is taken equal to the console overhanging length l_1 if the following conditions are met $M/Q \geq 0{,}3$ m and $l_{sup}/l_1 \geq 2/3$ (where M and Q — are the moment which stretches the top surface of the collar-beam and the in the normal section of the collar beam along the edge of the console). In this case the right part of condition (207) is taken no more than $5R_{bt}bh_0$.

Draft 68. Desigh scheme for the short console by cross force action

By hinge support of prefabricated beam going along the console overhanging length on the short console, if there are no special embedded details fixing the support area (Draft 69) value l_{sup} in condition (207) is taken equal to 2/3 of the length of actual support area.

Cross reinforcement of short consoles must meet the requirements of Item 5.77.

Draft 69. Design scheme for the short console by hinge connection of the prefabricated beam going along the console overhanging length

- By hinge support of the beam on the column console longitudinal reinforcement of the console is examined due to the following condition

$$Q\frac{l_1}{h_o} \leq R_s A_s, \tag{208}$$

where l_1, h_o — see Draft 68.

At the same time longitudinal reinforcement of the console must be installed up to the free supported end of the console and have anchorage (see Items 5.44 and 5.45).

By fixed connection of the collar-beam and the column ригеля with monolithing of the joint and welding of lower reinforcement of the collar-beam to reinforcement of the console by means of embedded elements longitudinal reinforcement of the console is examined due to the following condition:

$$Q\frac{l_1}{h_o} - N_s \leq R_s A_s, \tag{209}$$

where l_1, h_0 — overhanging length and working height of the short console;

N_s — horizontal force acting on the top of the console from the collar-beam equal to:

$$N_s = \frac{M + Ql_{sup}/2}{h_{ob}} \tag{210}$$

and taken no more than 1,4 $k_f l_w R_{wf}$ + 0,3 Q (where k_f and l_w — the height and the length of the angle joint of welding of embedded details of the collar-beam and console; R_f — design resistance of angle joints against the cutting of the joint metal determined in compliance with SNiP II-23-81, with electrodes Э42 R_{wf} = 180 MPa; 0,3 — steel to steel friction coefficient), as well as no more than $R_{sw} A_{sw}$ (where R_{sw} and A_{sw} — are design resistance and section area of the top reinforcement of the collar-beam).

In formulas (209) and (210):

M, Q — bending moment and cross force in the normal section of the collar beam along the edge of the console; if moment *M* stretches lower surface of the collar-beam so value *M* is considered in formula (210) with the sign "minus";

l_{sup} — actual length of the load support area along the console overhanging length;

h_{ob} — working height of the collar beam.

CALCULATION EXAMPLES

Example 49. *Given:* a free supported prefabricated beam lies on the short console of the column (Draft 70); the length of supporting area $l_{sup,f}$ = 300 mm; console length b = 400 mm; height of the column and overhanging length of the column h = 700 mm, l_1 = 350 mm; heavy weight concrete of the column B25 (R_b = 13 MPa, R_{bt} = 0,95 MPa by γ_{b2} = 0,9; $E_b = 27 \cdot 10^3$ MPa); longitudinal reinforcement A-III (R_s = 365 МПа); load on the console Q = 700 kN.
It is required to check the strength of the column as regards the cross force and to determine the cross section of longitudinal reinforcement and stirrups.

Draft 70. For the calculation example 49

Calculation. $H_0 = h - a$ = 700 - 30 = 670 mm. As $3.5R_{bt}bh_0 = 3.5 \cdot 0.95 \cdot 400 \cdot 670 =$ $= 891.1 \cdot 10^3\text{N} = 891.1\text{kN}$ > Q = 700 kN and at the same time $2.5R_{bt}bh_0 = 2.5 \cdot 0.95 \cdot 400 \cdot 670 = 636.5\,\text{кН}$ < Q = 700 кН so the console strength is to be checked according to condition (207).

Due to Item 3.99 design length of the support area is to be taken equal to:

$$l_{sup} = 2/3\ l_{sup,f} = 2/3 \cdot 300 = 200 \text{ mm}$$

Due to Item 5.77 we take the stirrups spacing equal to

$$s_w = 150 \text{ мм} < \frac{h}{4} = \frac{700}{4} = 175 \text{ mm}$$

By two-leg stirrups with diameter 10 mm we have A_{sw} = 157 mm², so

$$\mu_w = \frac{A_{sw}}{bs_w} = \frac{157}{400 \cdot 150} = 2{,}62 \cdot 10^{-3};$$

$$\alpha = \frac{E_s}{E_b} = \frac{20 \cdot 10^4}{2{,}7 \cdot 10^4} = 7{,}4;$$

$$\sin^2\theta = \frac{h_o^2}{h_o^2 + l_1^2} = \frac{670^2}{670^2 + 350^2} = 0{,}786;$$

$$0{,}8\ R_b\ bl_{sup}\ sin^2\theta(1 + 5\ \alpha\mu_w) = 0{,}8 \cdot 13 \cdot 400 \cdot 200 \cdot 0{,}786\ (1+5 \cdot 7{,}4 \cdot 2{,}62 \cdot 10^{-3}) =$$
$$= 717 \cdot 10^3 \text{ H} > Q = 700 \text{ kN},$$

That means the strength of the console as regards the cross force is provided.

Due to condition (208) let's determine required section area of the console longitudinal reinforcement:

$$A_s = \frac{Ql_1}{h_o R_s} = \frac{700 \cdot 10^3 \cdot 350}{670 \cdot 365} = 1002\,\text{mm}^2.$$

We take 3 ∅ 22 (A_s = 1140 mm²).

Calculation of embedded elements and connection details

CALCULATION OF EMBEDDED ELEMENTS

- **(3.44).** Calculation of normal anchors welded to the flat details of steel embedded elements as regards bending moments, normal and shearing force caused by static load located in one plane of symmetry of the embedded detail (Draft 71), must be made due to the following condition

$$A_{an} = \frac{1{,}1\sqrt{N_{an}^2 + \left(\frac{Q_{an}}{\lambda\delta}\right)^2}}{R_s}, \tag{211}$$

where A_{an} — total area of the anchors cross section of the most stressed row;

N_{an} — maximum stretching force in one row of anchors equal to:

$$N_{an} = \frac{M}{z} + \frac{N}{n_{an}}; \tag{212}$$

Q_{an} — shearing force on one anchors row equal to:

$$Q_{an} = \frac{Q - 0{,}3N'_{an}}{n_{an}}; \tag{213}$$

N'_{an} — maximum compression force in one row of anchors determined by formula

$$N'_{an} = \frac{M}{z} - \frac{N}{n_{an}}; \tag{214}$$

In formulas (211) — (214):

M, N, Q — moment, normal and shearing forces acting on the embedded element; the moment is determined relating to the axis located in the area of the external surface of the plate and going through the center of gravity of all;

z — distance between the end rows of anchors;

n_{an} — number of anchors rows along the shearing force direction; if even transfer of shearing force Q on all anchor rows is not provided, so by determination of shearing force Q_{an} it is necessary to consider no more than four rows;

λ — coefficient determined for anchor rods with diameter 8 – 25 mm for heavy-weight and fine concrete B12,5 – B50 and light-weight concrete B12,5 – B30 by the following formula

$$\lambda = \frac{4{,}75\sqrt[3]{R_b}}{(1 + 0{,}15A_{an1})\sqrt{R_s}}\beta, \tag{215}$$

but taken no more than 0.7; for heavy-weight and fine concrete of class more than B50 coefficient λ is taken as for class B50, and for light-weight concrete of class more than B30 — as for class B30. For heavy-weight concrete coefficient λ can be determined by Table. 28.

In formula (215):

R_b, R_s, – in MPa;

By determination of R_b coefficient γ_{b2} (see Item 3.1) is taken equal to 1.0;

A_{an1} – section area of the anchor rod of the most stressed row наиболее, cm^2;

β – coefficient taken equal to:

for heavy-weight concrete 1,0
for fine concrete of group:
А 0,8
Б and В 0,7
For light-weight concrete .... ρ_m/2300
(ρ_m — average concrete density, kg/m^3);

δ – coefficient determined by formula

$$\delta = \frac{1}{\sqrt{1+\omega}}, \qquad (216)$$

but taken no less than 0,15;

here $\omega = 0,3\ \frac{N_{an}}{Q_{an}}$ by $N'_{an} > 0$ (with pressure);

$\omega = 0,6\ \frac{N}{Q}$ by $N'_{an} \le 0$ (no pressure);

if there are no tension forces in anchors so coefficient в δ is taken equal to 1,0.

Section area of anchors of other rows must be taken equal to the section area of anchors of the most stressed row.
In formulas (212) and (214) normal force N is considered to be positive if it's directed from the embedded element (see Draft 71), and negative — if it's directed towards to it. In case if normal forces N_{an} and N'_{an}, as well as shearing force Q_{an} during calculation by formulas (212) – (214) get negative value (211), (213) and (216) they are taken equal to zero. Besides if N_{an} gets negative value so in formula (213) it's taken $N'_{an} = N$.

By location of the embedded element on the top surface of the detail (during concreting) coefficient λ is decreased by 20 percent and value N'_{an} in formula (213) is taken equal to zero.

Draft 71. Scheme of forces acting on the embedded element

- Calculation of normal anchors of embedded elements as regards bending moments and shear forces located in two symmetry planes of the embedded elements as well as normal force and torsion moments is made in compliance with "Recommendations for design of steel embedded elements for reinforced concrete structures" (Moscow, Stroyizdat, 1984).

- **(3.45).** In the embedded element with anchors welded with overlapping at the angle from 15 to 30 degrees (see Item 5.111) inclined anchors located symmetrically relating to the plane of the shearing force is calculated as regards this shearing force (by $Q > N$, where N — break force) by formula

$$A_{an,inc} = \frac{Q - 0,3N'_{an}}{R_s}, \qquad (217)$$

where $A_{an,inc}$ — total area of the cross section of inclined anchors;
N_{an} — see Item 3.101.
At the same time it is necessary to install normal anchors calculated by formula (211) by δ = 1.0 and by values Q_{an}, equal to 0.1 of shearing force determined by formula (213). It is possible to decrease the section area due to transfer of shearing force equal to $Q - 0.9 R_s A_{an,inc}$ on normal anchors. In that case δ is determined by formula (216).

Таблица 28

Anchor diameter мм	Coefficient λ for calculation of normal anchors of embedded elements according to class of heavy-weight concrete and reinforcement																	
	B15			B20			B25			B30			B40			≥ B50		
	A-I	A-II	A-III	A-I	A-II	A-III	A-I	A-II	A-III	A-I	A-II	A-III	A-I	A-II	A-III	A-I	A-II	A-III
8	0,60	—	0,48	0,66	—	0,53	0,70	—	0,57	0,70	—	0,60	0,70	—	0,66	0,70	—	0,70
10	0,58	0,52	0,45	0,64	0,57	0,50	0,69	0,62	0,54	0,70	0,65	0,57	0,70	0,70	0,63	0,70	0,70	0,66
12	0,55	0,50	0,43	0,61	0,55	0,48	0,66	0,59	0,52	0,70	0,62	0,55	0,70	0,69	0,60	0,70	0,70	0,63
14	0,53	0,47	0,41	0,58	0,52	0,46	0,63	0,56	0,49	0,66	0,59	0,52	0,70	0,65	0,57	0,70	0,69	0,60
16	0,50	0,45	0,39	0,55	0,49	0,43	0,59	0,53	0,47	0,63	0,56	0,49	0,69	0,62	0,54	0,70	0,65	0,57
18	0,47	0,42	0,37	0,52	0,46	0,41	0,56	0,50	0,44	0,59	0,53	0,46	0,65	0,58	0,51	0,68	0,61	0,54
20	0,44	0,39	0,34	0,49	0,44	0,38	0,52	0,47	0,41	0,55	0,50	0,43	0,61	0,54	0,48	0,64	0,58	0,50
22	0,41	0,37	0,32	0,46	0,41	0,36	0,49	0,44	0,39	0,52	0,46	0,41	0,57	0,51	0,45	0,60	0,54	0,47
25	0,37	0,33	0,29	0,41	0,37	0,32	0,44	0,40	0,35	0,47	0,42	0,37	0,51	0,46	0,40	0,54	0,49	0,43

Notes: 1. For concrete class B 12,5 coefficient λ is to be decreased by 0,02 in comparison with coefficient λ for concrete class B15.
2. Values of coefficient λ are given by γ_{bi} = 1,00.

- On welded to the plate supports made of strip steel or reinforcement lugs (see Item 5.114) it is possible to transfer no more than 30 percent of shearing force acting on the detail by stresses equal to R_b in concrete under supports. At the same time values of shearing force transferred on the anchors of the embedded element is decreased.

- **(3.46).** The structure of welded embedded elements with welded to them details which transfer the load on the embedded elements must provide including into work of anchor rods in compliance with the accepted design. External elements of the embedded details and their welded connections are calculated due to SNiP II-23-81. Durinf calculation of plates and corrugated steel as regards break force it is recommended to take that they have hinge connection with normal anchor rods. If the element which transfers the load is welded to the plate along the line of location of one of the anchor rows so during calculation it is recommended to decrease break force by value $n_a A_{an1} R_s$ (where n_a — number of anchors in the present row).

Besides the thickness of plate t of design embedded element must be examined due to the condition

$$t \geq 0{,}25\ d_{an} \frac{R_s}{R_{sq}}, \qquad (218)$$

where d_{an} — diameter of the anchor rod required due to the calculation;
R_{sq} — Design resistance of rolled-steel against the shearing equal to 0.58 R_y (where R_y — see SNiP II-23-81).

For welded connections types which provide the larger zone of including the plate into work by pulling out of anchor rods out of it (see position 6 of Table 52) the correction of condition (218) is possible for decrease of the plate thickness. If shearing force Q acts on the embedded element with decreased plate thickness total section area (perpendicular to this force) of the section with welded to it elements in the location zone of anchor rods along the force Q is taken no less than the section area of the plate determined by formula (218).

- If the following condition is met

$$N'_{an} \leq 0, \qquad (219)$$

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