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 not 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
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 $\gamma_f$, 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/m² so snow long-term load will be $s = 0.3 \times 1000 = 300$ H/m² and snow short-term load $s = 1000 - 300 = 700$ H/m²).

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)

<table>
<thead>
<tr>
<th>Work conditions of the structure</th>
<th>Limit width of crack opening, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Short-lived $a_{cr1}$</td>
</tr>
<tr>
<td>1. members carrying the load of liquids or gases if the section is</td>
<td></td>
</tr>
<tr>
<td>a)   fully stretched</td>
<td>0.2</td>
</tr>
<tr>
<td>b)   partly compressed</td>
<td>0.3</td>
</tr>
<tr>
<td>2. members carrying the load of granular materials</td>
<td>0.3</td>
</tr>
<tr>
<td>3. members used in the ground with variable ground-water elevations</td>
<td>0.3</td>
</tr>
<tr>
<td>4. other members</td>
<td>0.4</td>
</tr>
</tbody>
</table>
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)

<table>
<thead>
<tr>
<th>Structure members</th>
<th>Deflection limits</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1. Crane beams</strong></td>
<td></td>
</tr>
<tr>
<td>For manually operated cranes</td>
<td>( \frac{l}{500} )</td>
</tr>
<tr>
<td>For electric cranes</td>
<td>( \frac{l}{600} )</td>
</tr>
<tr>
<td><strong>2. Floors with a plane ceiling and roof members (except the ones mentioned in position 4) if the span is:</strong></td>
<td></td>
</tr>
<tr>
<td>( l &lt; 6 )</td>
<td>( \frac{l}{200} )</td>
</tr>
<tr>
<td>( 6 \leq l \leq 7.5 )</td>
<td>3 cm</td>
</tr>
<tr>
<td>( l &gt; 7.5 )</td>
<td>( \frac{l}{250} )</td>
</tr>
<tr>
<td><strong>3. Floors with ribbed ceiling and stairs members if the span is:</strong></td>
<td></td>
</tr>
<tr>
<td>( l &lt; 5 )</td>
<td>( \frac{l}{200} )</td>
</tr>
<tr>
<td>( 5 \leq l \leq 10 )</td>
<td>2.5 cm</td>
</tr>
<tr>
<td>( l &gt; 10 )</td>
<td>( \frac{l}{400} )</td>
</tr>
<tr>
<td><strong>4. Roof elements of agricultural building for production purpose if the span is:</strong></td>
<td></td>
</tr>
<tr>
<td>( l &lt; 6 )</td>
<td>( \frac{l}{150} )</td>
</tr>
<tr>
<td>( 6 \leq l \leq 10 )</td>
<td>4 cm</td>
</tr>
<tr>
<td>( l &gt; 10 )</td>
<td>( \frac{l}{250} )</td>
</tr>
<tr>
<td><strong>5. Suspended wall panels if the span is:</strong></td>
<td></td>
</tr>
<tr>
<td>( l &lt; 6 )</td>
<td>( \frac{l}{200} )</td>
</tr>
<tr>
<td>( 6 \leq l \leq 7.5 )</td>
<td>3 cm</td>
</tr>
<tr>
<td>( l &gt; 7.5 )</td>
<td>( \frac{l}{250} )</td>
</tr>
</tbody>
</table>

*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:
\[
\delta = \delta_\Delta \delta_\varepsilon \delta_j,
\]
but no less than one,

Where \( \delta_\Delta \) is the coefficient taken equal to 
\[
\delta_\Delta = \frac{60}{\Delta t_c}
\]
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, \( \varepsilon \) – 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 \( \varepsilon = 3 \cdot 10^{-4} \));

\[
\delta_j = \frac{l \cdot h}{9}
\]
(Here \( l \) is the length of the column between fixing points, \( h \) is the height of the column section in the direction under consideration);

\[
\delta_\varphi = 0.4 + \varphi_{\text{ext}} / 100 \leq 1
\]
(Here \( \varphi_{\text{ext}} \) 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 \( \delta \) 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>
<thead>
<tr>
<th>Table 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structures</td>
</tr>
<tr>
<td></td>
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<tr>
<td></td>
</tr>
<tr>
<td>-------------------------------</td>
</tr>
<tr>
<td>1. Concrete structures</td>
</tr>
<tr>
<td>a) prefabricated</td>
</tr>
<tr>
<td>b) monolithic:</td>
</tr>
<tr>
<td>by constructive reinforcement</td>
</tr>
<tr>
<td>without constructive reinforcement</td>
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<tr>
<td></td>
</tr>
<tr>
<td>2. reinforced concrete structures:</td>
</tr>
<tr>
<td>a) prefabricated-frame structure:</td>
</tr>
<tr>
<td>one-storey</td>
</tr>
<tr>
<td>multi-storey</td>
</tr>
<tr>
<td>b) prefabricated-monolithic and monolithic structures:</td>
</tr>
<tr>
<td>frame structures</td>
</tr>
<tr>
<td>solid structures</td>
</tr>
<tr>
<td>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.</td>
</tr>
</tbody>
</table>

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. – В) – 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

* 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)

<table>
<thead>
<tr>
<th>Structure work conditions</th>
<th>Concrete grade no less than</th>
<th>Concrete grade no less than</th>
</tr>
</thead>
<tbody>
<tr>
<td>design winter temperature of external air, in Celsius degrees</td>
<td>according to resistance to frost</td>
<td>according to resistance to water</td>
</tr>
<tr>
<td>for building structures (except external walls of heated buildings) of responsibility degree</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>mode characteristics</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>I</th>
<th>II</th>
<th>III</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Alternate freezing and melting</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a) in waterlogged state (for example structures located in the ground layer seasonally melting in the permafrost region)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lower than 40 degrees below zero</td>
<td>F300</td>
<td>F200</td>
<td>F150</td>
<td>W6</td>
<td>W4</td>
<td>W2</td>
</tr>
<tr>
<td>Lower than 20 degrees below zero up to 40 degrees below zero</td>
<td>F200</td>
<td>F150</td>
<td>F100</td>
<td>W4</td>
<td>W2</td>
<td>Not regulated</td>
</tr>
<tr>
<td>Lower than 5 degrees below zero up to 20 degrees below zero</td>
<td>F150</td>
<td>F100</td>
<td>F75</td>
<td>W2</td>
<td>Not regulated</td>
<td></td>
</tr>
<tr>
<td>5 degrees below zero and more</td>
<td>F100</td>
<td>F75</td>
<td>F50</td>
<td>Not regulated</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lower than 40 degrees below zero</td>
<td>F200</td>
<td>F150</td>
<td>F100</td>
<td>W4</td>
<td>W2</td>
<td>Not regulated</td>
</tr>
<tr>
<td>Lower than 20 degrees below zero up to 40 degrees below zero</td>
<td>F100</td>
<td>F75</td>
<td>F50</td>
<td>W2</td>
<td>Not regulated</td>
<td></td>
</tr>
<tr>
<td>Lower than 5 degrees below zero up to 20 degrees below zero</td>
<td>F75</td>
<td>F50</td>
<td>F35*</td>
<td>Not regulated</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 degrees below zero and more</td>
<td>F50</td>
<td>F35*</td>
<td>F25*</td>
<td>Not regulated</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. Possible occasional temperature influence lower than 0 degree below zero</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a) in waterlogged state (for example structures located in the ground or under water)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lower than 40 degrees below zero</td>
<td>F150</td>
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<td>F75</td>
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</tr>
<tr>
<td>Lower than 20 degrees below zero up to 40 degrees below zero</td>
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<td>F50</td>
<td>F35*</td>
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<td></td>
</tr>
<tr>
<td>Lower than 5 degrees below zero up to 20 degrees below zero</td>
<td>F50</td>
<td>F35*</td>
<td>F25*</td>
<td>Not regulated</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 degrees below zero and more</td>
<td>F35*</td>
<td>F25*</td>
<td>F15**</td>
<td>Not regulated</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1. Alternate freezing and melting

a) in waterlogged state (for example structures located in the ground layer seasonally melting in the permafrost region)

b) in conditions of occasional water saturation (for example overland structures exposed to atmospheric effects)

c) in air humidity conditions without occasional water saturation (for example structures exposed to atmospheric effects but protected against atmospheric precipitation)

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)
<table>
<thead>
<tr>
<th>Structure work conditions</th>
<th>Minimum concrete grade according to resistance to frost of external walls of heated buildings made of light-weight, porous concrete</th>
<th>Minimum concrete grade according to resistance to frost of external walls of heated buildings made of heavy-weight, fine concrete for building structures (except external walls of heated buildings) of responsibility degree</th>
</tr>
</thead>
<tbody>
<tr>
<td>relative degree of humidity of internal air inside of rooms $\varphi_{\text{int}}$, in percents</td>
<td>design winter temperature of external air, in Celsius degrees</td>
<td>I</td>
</tr>
<tr>
<td>1. $\varphi_{\text{int}} &gt; 75$</td>
<td>Lower than 40 degrees below zero</td>
<td>F100</td>
</tr>
<tr>
<td></td>
<td>Lower than 20 degrees below zero up to 40 degrees below zero</td>
<td>F75</td>
</tr>
<tr>
<td></td>
<td>Lower than 5 degrees below zero up to 20 degrees below zero 5 degrees below zero and more</td>
<td>F50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>F35</td>
</tr>
<tr>
<td>2. $60 &lt; \varphi_{\text{int}} \leq 75$</td>
<td>Lower than 40 degrees below zero</td>
<td>F75</td>
</tr>
<tr>
<td></td>
<td>Lower than 20 degrees below zero up to 40 degrees below zero</td>
<td>F50</td>
</tr>
<tr>
<td></td>
<td>Lower than 5 degrees below zero up to 20 degrees below zero 5 degrees below zero and more</td>
<td>F35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>F25</td>
</tr>
<tr>
<td>3. $\varphi_{\text{int}} \leq 60$</td>
<td>Lower than 40 degrees below zero</td>
<td>F50</td>
</tr>
<tr>
<td></td>
<td>Lower than 20 degrees below zero up to 40 degrees below zero</td>
<td>F35</td>
</tr>
<tr>
<td></td>
<td>Lower than 5 degrees below zero up to 20 degrees below zero 5 degrees below zero and more</td>
<td>F25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>F15*</td>
</tr>
</tbody>
</table>

* 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 | slag-pumice concrete | perlite concrete | concrete of natural expanded aggregate | aglopore concrete |
| B3.5 | D800–D1100 | D1100–D1500 | D800–D1000 | D900–D1300 | D1100–D1300 |
| B7.5 | D900–D1300 | D1300–D1700 | D900–D1200 | D1100–D1500 | D1300–D1500 |
| B12.5 | D1000–D1400 | D1400–D1800 | D1000–D1400 | D1200–D1600 | D1400–D1600 |
| 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 $R_{bt}$.

Design resistances of concrete $R_{bn}$ and $R_{bt}$ 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 $\gamma_{bt} = 1.3$; 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 $\gamma_{bi}$ considering work conditions of the structure, process of manufacturing, sections dimensions etc.
<table>
<thead>
<tr>
<th>Resistance type</th>
<th>Concrete</th>
<th>Standard resistances of concrete $R_{bn}$ and $R_{bt}$ and design resistances for second class limit states $R_{b,ser}$ and $R_{bt,ser}$, in Mega Pascal (kilogram-force per cm²) if concrete grade as regards resistance to compression is</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial compression (prism strength) $R_{bn}$ and $R_{b,ser}$</td>
<td>heavy-weight, fine, light-weight</td>
<td>$1.9$ $(19.4)$ $2.7$ $(27.5)$ $3.5$ $(35.7)$ $5.5$ $(56.1)$ $7.5$ $(76.5)$ $9.5$ $(96.9)$ $11.0$ $(112)$ $15.0$ $(153)$</td>
</tr>
<tr>
<td>Axial tension $R_{bn}$ and $R_{bt,ser}$</td>
<td>heavy-weight, fine¹, light-weight with dense aggregate</td>
<td>$0.29$ $(2.96)$ $0.39$ $(4.00)$ $0.55$ $(5.61)$ $0.70$ $(7.14)$ $0.85$ $(8.67)$ $1.00$ $(10.2)$ $1.15$ $(11.7)$ $1.40$ $(14.3)$</td>
</tr>
<tr>
<td>Light-weight concrete with porous aggregate²</td>
<td></td>
<td>$0.29$ $(2.96)$ $0.39$ $(4.00)$ $0.55$ $(5.61)$ $0.70$ $(7.14)$ $0.85$ $(8.67)$ $1.00$ $(10.2)$ $1.10$ $(11.2)$ $1.20$ $(12.2)$</td>
</tr>
<tr>
<td>Resistance type</td>
<td>Concrete</td>
<td>Standard resistances of concrete $R_{bn}$ and $R_{bt}$ and design resistances for second class limit states $R_{b,ser}$ and $R_{bt,ser}$, in Mega Pascal (kilogram-force per cm²) if concrete grade as regards resistance to compression is</td>
</tr>
<tr>
<td>-----------------</td>
<td>----------</td>
<td>-----------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Axial compression (prism strength) $R_{bn}$ and $R_{b,ser}$</td>
<td>heavy-weight, fine, light-weight</td>
<td>$18.5$ $(189)$ $22.0$ $(224)$ $25.5$ $(260)$ $29.0$ $(296)$ $32.0$ $(326)$ $36.0$ $(367)$ $39.5$ $(403)$ $43.0$ $(438)$</td>
</tr>
<tr>
<td>Axial tension $R_{bn}$ and $R_{bt,ser}$</td>
<td>heavy-weight, fine¹, light-weight with dense aggregate</td>
<td>$1.60$ $(16.3)$ $1.80$ $(18.4)$ $1.95$ $(19.9)$ $2.10$ $(21.4)$ $2.20$ $(22.4)$ $2.30$ $(23.5)$ $2.40$ $(24.5)$ $2.50$ $(25.5)$</td>
</tr>
<tr>
<td>Light-weight concrete with porous aggregate²</td>
<td></td>
<td>$1.35$ $(13.8)$ $1.50$ $(15.3)$ $1.65$ $(16.8)$ $1.80$ $(18.4)$ $–$ $–$ $–$ $–$</td>
</tr>
</tbody>
</table>

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

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

Note. For porous concrete values $R_{bn}$ and $R_{bt,ser}$ are taken the same as for light-weight concrete and values $R_{bt,ser}$ 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 $\gamma_{bc} = 1.0$.

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 $\gamma_{b2}$ considering duration of loads action influence and strength gain of concrete; coefficient $\gamma_{b2}$ 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 $\gamma_{bb}$ taken according to Table 10.

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

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

(2.16) Prime coefficient of concrete deformation $\nu$ (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 $1 + \frac{w}{100}$ (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 $\omega$ by formula $D + \frac{\omega}{1000}$ where $\omega = 15$ and 20 percent correspondingly for light-weight and porous concrete grade B10 and less and $\omega = 10$ 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-II 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 BCr3cn2 and BCr3пс2.

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 BCr3пс2 for lifting loops.
### Table 8

<table>
<thead>
<tr>
<th>Resistance type</th>
<th>Concrete</th>
<th>Work condition coefficient $\gamma_b$</th>
<th>Design resistance of concrete for the first class limit states $R_b$ and $R_{bt}$, Mega Pascal (kilogram-force/cm²) if class of concrete as regards the resistance to compression is</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>B.2.5</td>
<td>B.3.5</td>
</tr>
<tr>
<td>Axial compression (prism strength) $R_b$</td>
<td>Heavy-weight, fine and lightweight</td>
<td>0.9</td>
<td>1.3(13.3)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.0</td>
<td>1.5(15.3)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.1</td>
<td>1.6(16.3)</td>
</tr>
<tr>
<td>Axial tension $R_{bt}$</td>
<td>Heavy-weight, fine and lightweight concrete with fine dense aggregate</td>
<td>0.9</td>
<td>0.18(1.84)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.0</td>
<td>0.20(2.04)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.1</td>
<td>0.22(2.24)</td>
</tr>
<tr>
<td>Light-weight concrete with fine porous aggregate(^1)</td>
<td>0.9</td>
<td>0.18(1.84)</td>
<td>0.23(2.34)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.0</td>
<td>0.20(2.04)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.1</td>
<td>0.22(2.24)</td>
</tr>
</tbody>
</table>

1. For fine concrete of group Б (see Item 2.1) values $R_b$ are decreased by 15 percent.

2. For expanded-clay perlite concrete on expanded perlite sand values $R_b$ 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 $\gamma_b$ are given in Item 3.1.

3. Design concrete resistance with the work condition coefficient $\gamma_b = 1.0$ are taken in compliance with Table 13 of SNiP 2.03.01-84.
Table 9 (15)

<table>
<thead>
<tr>
<th>Factors providing work condition coefficient insertion</th>
<th>Work condition coefficient of concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>graphical symbol</td>
</tr>
<tr>
<td>1. Concreting in vertical position (concreting layer height is more than 1.5 m)</td>
<td>$\gamma_{b3}$</td>
</tr>
<tr>
<td>2. Concreting of monolithic poles and reinforced concrete columns with maximum section dimension less than 30 cm</td>
<td>$\gamma_{b5}$</td>
</tr>
<tr>
<td>3. Alternate freezing and melting</td>
<td>$\gamma_{b6}$</td>
</tr>
<tr>
<td>4. Use of not protected against solar radiation structures in climatic sub-region IVA according to SNiP 2.01.01-82</td>
<td>$\gamma_{b7}$</td>
</tr>
<tr>
<td>5. Concrete structures</td>
<td>$\gamma_{b9}$</td>
</tr>
<tr>
<td>6. Concrete structures of heavy-weight concrete B35 and higher or of fine concrete B25 and higher</td>
<td>$\gamma_{b10}$</td>
</tr>
<tr>
<td>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.</td>
<td>$\gamma_{b12}$</td>
</tr>
</tbody>
</table>

*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 $\gamma_{b2}$ (see Item 3.1)] must be no less than 0.45.

Table 10 (17)

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

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>
<thead>
<tr>
<th>Concrete</th>
<th>Prime concrete modulus of elasticity $E_b$ $10^{-3}$ Mega Pascal (kilogram-force/cm²) if concrete class as regards resistance to compression is</th>
</tr>
</thead>
<tbody>
<tr>
<td>B2.5</td>
<td>B3.5</td>
</tr>
<tr>
<td>Heavy-weigh:</td>
<td></td>
</tr>
<tr>
<td>- of air hardening:</td>
<td></td>
</tr>
<tr>
<td>(96.9)</td>
<td>(133)</td>
</tr>
<tr>
<td>- exposed to thermal treatment by air pressure</td>
<td></td>
</tr>
<tr>
<td>Fine concrete of groups:</td>
<td></td>
</tr>
<tr>
<td>A–of air hardening;</td>
<td></td>
</tr>
<tr>
<td>(71.4)</td>
<td>(102)</td>
</tr>
<tr>
<td>- exposed to thermal treatment by air pressure</td>
<td></td>
</tr>
<tr>
<td>(66.3)</td>
<td>(92)</td>
</tr>
<tr>
<td>B– of air hardening;</td>
<td></td>
</tr>
<tr>
<td>(66.3)</td>
<td>(91.8)</td>
</tr>
<tr>
<td>- exposed to thermal treatment by air pressure</td>
<td></td>
</tr>
<tr>
<td>(56.1)</td>
<td>(81.6)</td>
</tr>
<tr>
<td>B–of autoclave hardening</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Light-weight and porous of grade as regards average density D:</td>
<td></td>
</tr>
<tr>
<td>800</td>
<td></td>
</tr>
<tr>
<td>(40.8)</td>
<td>(45.9)</td>
</tr>
<tr>
<td>1000</td>
<td></td>
</tr>
<tr>
<td>(51.0)</td>
<td>(56.1)</td>
</tr>
<tr>
<td>1200</td>
<td></td>
</tr>
<tr>
<td>(61.2)</td>
<td>(68.3)</td>
</tr>
<tr>
<td>1400</td>
<td></td>
</tr>
<tr>
<td>(71.4)</td>
<td>(79.5)</td>
</tr>
<tr>
<td>1600</td>
<td></td>
</tr>
<tr>
<td>(91.8)</td>
<td>(102)</td>
</tr>
<tr>
<td>1800</td>
<td></td>
</tr>
<tr>
<td>-</td>
<td></td>
</tr>
<tr>
<td>(114)</td>
<td>(133)</td>
</tr>
<tr>
<td>2000</td>
<td></td>
</tr>
<tr>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

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)

<table>
<thead>
<tr>
<th>Reinforcement types and documents regulating its quality</th>
<th>Reinforcement class</th>
<th>Steel grade</th>
<th>Reinforcement diameter, mm</th>
<th>Use conditions of the structure by static load</th>
<th>Dynamic and repeated load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>in heated buildings up to 30 degrees below zero</td>
<td>lower than 30 degrees below zero up to 40 degrees below zero</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>in open air and in not heated buildings by design temperature in Celsius degrees</td>
<td>lower than 40 degrees below zero up to 55 degrees below zero</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>lower than 55 degrees below zero up to 70 degrees below zero</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hot-rolled plain rod reinforcement, GOST 5781-82 and GOST 380-71</td>
<td>A-I</td>
<td>Cr3сп3</td>
<td>6–40</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cr3пс3</td>
<td>6–40</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cr3кп3</td>
<td>6–40</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ВСт3сп2</td>
<td>6–40</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ВСт3пс2</td>
<td>6–40</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ВСт3кп2</td>
<td>6–40</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ВСт3Гпс2</td>
<td>6–18</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Hot-rolled ribbed rod reinforcement</td>
<td>A-II</td>
<td>ВСт5сп2</td>
<td>10–40</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ВСт5пс2</td>
<td>10–16</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td></td>
<td></td>
<td>18Г2С</td>
<td>18–40</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td></td>
<td></td>
<td>18Г2С</td>
<td>10–22</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Ac-II</td>
<td>10ГТ</td>
<td>10–32</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>A-III</td>
<td>35ГС</td>
<td>6–40</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td></td>
<td>25Г2С</td>
<td>6–8</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td></td>
<td>10–40</td>
<td>+</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td></td>
<td>32Г2Рпс</td>
<td>6–22</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Ausform robbed rod reinforcement</td>
<td>Ar-IIIC</td>
<td>БСт5пс2</td>
<td>10–22</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td></td>
<td></td>
<td>БСт5пс</td>
<td>10–22</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Regular ribbed reinforcement wire</td>
<td>Bp-I</td>
<td>–</td>
<td>3–5</td>
<td>+</td>
<td>+</td>
</tr>
</tbody>
</table>

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)

<table>
<thead>
<tr>
<th>Embedded elements characteristics</th>
<th>Design temperature, Celsius degrees</th>
<th>Steel grade according to GOST 380-71</th>
<th>sheet steel thickness, mm</th>
<th>Steel grade according to GOST 380-71</th>
<th>sheet steel thickness, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>up to 30 degrees below zero</td>
<td></td>
<td></td>
<td>lower than 30 degrees below zero up to 40 degrees below zero</td>
<td></td>
</tr>
<tr>
<td>1. Calculated as regards the loads</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a) static;</td>
<td></td>
<td>BC3kr2</td>
<td>4–30</td>
<td>BC3nc6</td>
<td>4–25</td>
</tr>
<tr>
<td>b) dynamic and repeated</td>
<td></td>
<td>BC3nc6</td>
<td>4–10</td>
<td>BC3nc6</td>
<td>4–10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>BC3Gnc5</td>
<td>11–30</td>
<td>BC3Gnc5</td>
<td>11–30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>BC3Gn5</td>
<td>11–25</td>
<td>BC3Gn5</td>
<td>11–25</td>
</tr>
<tr>
<td>2. Constructive (not calculated as regards any forces)</td>
<td></td>
<td>BC3kr2</td>
<td>4–10</td>
<td>BC3kr2</td>
<td>4–10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>BC3kr2</td>
<td>4–30</td>
<td>BC3kr2</td>
<td>4–30</td>
</tr>
</tbody>
</table>

Notes: 1. Design temperature is taken according to Item 1.8 instructions.
2. When using low-alloyed steel for example steel grade 10ГС2C1, 09ГС2C, 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)

<table>
<thead>
<tr>
<th>Type and class of reinforcement</th>
<th>Standard resistances against tension $R_{sn}$ and design resistances against tension for the second class limit states $R_{s,ser}$, mega Pascal (kilogram-force/cm²)</th>
<th>Type and class of reinforcement</th>
<th>Standard resistances against tension $R_{sn}$ and design resistances against tension for the second class limit states $R_{s,ser}$, mega Pascal (kilogram-force/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rod reinforcement</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-I</td>
<td>235 (2400)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-II</td>
<td>295 (3000)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-III and Ar-IIIC</td>
<td>390 (4000)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Reinforcement wire of class Bp-I, diameter:</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3 mm</td>
<td>410 (4200)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4 mm</td>
<td>405 (4150)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5 mm</td>
<td>395 (4050)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 15 (22, 23)

<table>
<thead>
<tr>
<th>Type and class of reinforcement</th>
<th>Design resistances of reinforcement for the first classes limit states, mega Pascal (kilogram-force/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>of longitudinal reinforcement $R_l$</td>
</tr>
<tr>
<td>Rod reinforcement of classes:</td>
<td>R</td>
</tr>
<tr>
<td>A-I</td>
<td>225 (2300)</td>
</tr>
<tr>
<td>A-II</td>
<td>280 (2850)</td>
</tr>
<tr>
<td>A-III with diameter:</td>
<td>R</td>
</tr>
<tr>
<td>6–8 mm</td>
<td>355 (3600)</td>
</tr>
<tr>
<td>10–40 mm</td>
<td>365 (3750)</td>
</tr>
<tr>
<td>Ar-IIIC</td>
<td>365 (3750)</td>
</tr>
<tr>
<td></td>
<td>R</td>
</tr>
<tr>
<td></td>
<td>3 mm</td>
</tr>
<tr>
<td></td>
<td>4 mm</td>
</tr>
<tr>
<td></td>
<td>5 mm</td>
</tr>
</tbody>
</table>

* In welded frameworks for stirrups made of reinforcement A-III and Ar-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/cm²).
** 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 $\gamma_s$ 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 $\gamma_{s1}$ and $\gamma_{s2}$:

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 $\gamma_{s1}$ and $\gamma_{s2}$ 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 $\gamma_{s5}$ 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 $\gamma_{b2} = 0.9$:

\[ R_b = \gamma_b R_{b0} \]

\[ R_{bt} = \gamma_{bt} R_{bt0} \]

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:

\[ F_I < 0.82 F_{II} \]  \hfill (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 $\eta$ (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:

\[
N \leq R_h A_b
\]  

(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:

\[
A_b = bh \left(1 - \frac{2e_0 \eta}{h}\right)
\]  

(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:

\[
N \leq \frac{R_h W_{pl}}{e_0 \eta - r}
\]  

(4)

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

\[
N \leq \frac{1.75R_w bh}{6e_0 \eta \left(\frac{h}{\phi}\right)}
\]  

(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):

$\eta$ is the coefficient determined by the formula (8);
**Draft 2. To definition \( A_{b1} \).**

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

\[
W_{pl} = \gamma W_0
\]

where \( \gamma \) – see in Table 29.

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

\[
\eta = \frac{1}{1 - \frac{N}{N_{cr}}}
\]

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

\[
N_{cr} = \frac{6.4 E_b I}{\varphi (l_0 / h)^2} \left( \frac{0.11}{0.1 + \delta} + 0.1 \right)
\]

(9)

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

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

\[
N_{cr} = \frac{0.533 E_b A}{\varphi (l_0 / h)^2} \left( \frac{0.11}{0.1 + \delta} + 0.1 \right)
\]

(9a)

In formulas (9) and (9a):

\( \varphi \) – Coefficient considering influence of long duration of the load on the member deflection:

\[
r = \varphi \frac{W}{A}
\]

(6)

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

\( \sigma_b \) – 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:

\[
W_{pl} = \frac{2 I_{b0}}{h - x} + S_{b0}
\]

(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:

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

\( A_{b1} \) 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.

\( r \) 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:

\[r = \varphi \frac{W}{A}\]
\[ \varphi_l = 1 + \beta \frac{M_{ll}}{M_1} \]

but no more than 1+β

here \( \beta \) 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;

\( M_{ll} \) is the same but caused by dead loads and long-term loads;

\( l_0 \) is determined according to Table 17;

\( \delta_e \) – The coefficient taken equal to \( e_0/h \) but no less than

\[ \delta_{e, \text{min}} = 0.5 - 0.01 \frac{l_0}{h} - 0.01 R_b \]

(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 \( \delta_{e, \text{min}} \) only once taking \( R_b \) by \( \gamma_{b,3} = 0.1 \).

### Table 16 (30)

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Coefficient ( \beta ) in formula (10)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Heavy-weight concrete</td>
<td>1.0</td>
</tr>
<tr>
<td>2. Fine concrete:</td>
<td></td>
</tr>
<tr>
<td>group A</td>
<td>1.3</td>
</tr>
<tr>
<td>group B</td>
<td>1.5</td>
</tr>
<tr>
<td>group B</td>
<td>1.0</td>
</tr>
<tr>
<td>3. Light-weight concrete</td>
<td></td>
</tr>
<tr>
<td>- with artificial coarse and fine aggregate:</td>
<td></td>
</tr>
<tr>
<td>dense</td>
<td>1.0</td>
</tr>
<tr>
<td>porous</td>
<td>1.5</td>
</tr>
<tr>
<td>- with natural coarse aggregate</td>
<td>2.5</td>
</tr>
<tr>
<td>4. Porous concrete</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Note: Fine concrete groups are given in Item 2.1.

### Table 17 (31)

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

*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:
\[
N \leq \alpha_n R_b bh
\]

Where \( \alpha_n \) 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 \( M_{ul} / M_1 = 1.0 \)
- By \( M_{ul} / M_1 = 0.5 \)

**Bending Elements**

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

\[
M \leq R_b W_{pl}
\]

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

\[
W_{pl} = \frac{bh^2}{3.5}
\]

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

\[
\tau_{xy} \leq R_{bt}
\]

Where \( \tau_{xy} \) – 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 \( (E_b = 14 \text{,}000 \text{ Mega Pascal}) \) 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 \( \phi_i \) 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 \( \frac{M_{ul}}{M} = \frac{N_l}{N} = \frac{540}{900} = 0.6 \),

So \( \phi_i = 1 + \beta \frac{M_{ul}}{M_1} = 1 + 0.6 = 1.6 \)

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 \( \gamma_{bt} = 0.90 \) that is \( R_b = 7.7 \) mega
Pascal and in compliance with Table 9 considering work conditions coefficients $\gamma_{o3} = 0.85$ and $\gamma_{o9} = 0.90$ we get $R_b = 7.7 \times 0.85 \times 0.90 = 5.89$ mega Pascal.

As $\delta_{e,\min} = 0.5 - 0.01 \frac{l_0}{h} - 0.01R_b = 0.5 - 0.01 \cdot 13.5 - 0.01 \cdot 5.89 = 0.306 > \frac{e_o}{h} = \frac{10}{200}$ 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 $A = 200 \times 1000 = 200 000 \text{ mm}^2$:

$$N_{cr} = \frac{0.533E_bA}{\varphi_1(l_0/h)^2} \left( \frac{0.11}{0.1 + \delta_e} + 0.1 \right) = \frac{0.533 \cdot 14 \cdot 10^4 \cdot 200000}{1.6 \cdot 13.5^2} \left( \frac{0.11}{0.1 + 0.306} + 0.1 \right) = 1898 \cdot 10^3 \text{ N} = 1898 \text{ kN}$$

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

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

$$R_b A_b = R_b b h \left( 1 - \frac{2e_o \eta}{h} \right) = 5.89 \cdot 200000 \left( 1 - \frac{2 \cdot 10 \cdot 1.902}{200} \right) = 954000 \text{ N} = 954 \text{ kN} > N = 900 \text{ kN}$$

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 $\xi_R$ (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 $r_i/r_S \geq 0.5$ 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 $Ne$ by $\bar{M}$ (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 $\xi_R$ is determined by the following formula:

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

where $\omega$ is characteristic of concrete compressed zone determined by the following formula:

$$\omega = \alpha - 0.008 R_b$$

where $\alpha$ 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 B and B
- 0.85 for light-weight and porous concrete

$\sigma_{sc,u} = 500$ Mega Pascal by coefficient $\gamma_b = 0.9$ (see Item 3.1);
$\sigma_{sc,u} = 400$ Mega Pascal by coefficient $\gamma_b = 1.0$ or $\gamma_b = 1.1$;
$R_s, R_b$ are in mega Pascals.

Values $\omega$ and $\xi_R$ 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>
<thead>
<tr>
<th>Concrete work conditions coefficient $\gamma_{b2}$</th>
<th>Class of tensile reinforcement</th>
<th>Symbol</th>
<th>Values $\omega$, $\xi_R$, $\alpha_R$ and $\psi_c$ for members of heavy-weight concrete of classes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Any</td>
<td>$\omega$</td>
<td></td>
<td>B12.5</td>
</tr>
<tr>
<td>0.9</td>
<td></td>
<td></td>
<td>0.796 0.788 0.766 0.746 0.726 0.710 0.690 0.670 0.650 0.634 0.614</td>
</tr>
<tr>
<td>A-III (Ø10–40) and BP-I (Ø4; 5)</td>
<td>$\xi_R$</td>
<td>$\alpha_R$</td>
<td>0.662 0.652 0.652 0.604 0.582 0.564 0.542 0.521 0.500 0.484 0.464</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\psi_c$</td>
<td>0.443 0.443 0.440 0.422 0.413 0.405 0.395 0.381 0.380 0.367 0.355</td>
</tr>
<tr>
<td>A-II</td>
<td>$\xi_R$</td>
<td>$\alpha_R$</td>
<td>0.690 0.698 0.698 0.652 0.630 0.612 0.591 0.570 0.550 0.531 0.512 0.490</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\psi_c$</td>
<td>0.452 0.449 0.449 0.439 0.432 0.424 0.417 0.408 0.399 0.380 0.361 0.337</td>
</tr>
<tr>
<td>A-I</td>
<td>$\xi_R$</td>
<td>$\alpha_R$</td>
<td>0.708 0.708 0.708 0.652 0.630 0.612 0.591 0.570 0.550 0.531 0.512 0.490</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\psi_c$</td>
<td>0.457 0.455 0.455 0.447 0.439 0.432 0.425 0.416 0.407 0.399 0.380 0.337</td>
</tr>
<tr>
<td>1.0</td>
<td>$\omega$</td>
<td></td>
<td>B15</td>
</tr>
<tr>
<td>Any</td>
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<td></td>
<td>0.790 0.782 0.758 0.734 0.714 0.694 0.674 0.650 0.630 0.610 0.586</td>
</tr>
<tr>
<td>A-III (Ø10–40) and BP-I (Ø4; 5)</td>
<td>$\xi_R$</td>
<td>$\alpha_R$</td>
<td>0.684 0.681 0.684 0.652 0.630 0.612 0.591 0.570 0.550 0.531 0.512 0.490</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\psi_c$</td>
<td>0.457 0.455 0.449 0.447 0.439 0.432 0.425 0.416 0.407 0.399 0.380 0.337</td>
</tr>
<tr>
<td>A-II</td>
<td>$\xi_R$</td>
<td>$\alpha_R$</td>
<td>0.684 0.681 0.684 0.652 0.630 0.612 0.591 0.570 0.550 0.531 0.512 0.490</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\psi_c$</td>
<td>0.457 0.455 0.449 0.447 0.439 0.432 0.425 0.416 0.407 0.399 0.380 0.337</td>
</tr>
</tbody>
</table>
Table 19

Concrete work conditions coefficient \( \gamma_{bi} \)

<table>
<thead>
<tr>
<th>Concrete work conditions coefficient ( \gamma_{bi} )</th>
<th>Class of tensile reinforcement</th>
<th>Symbol</th>
<th>Values of ( \omega, \xi_R, \alpha_R ) and ( \psi_c ) for members of fine concrete of group A, light-weight and porous concrete of classes</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.9</td>
<td>Any</td>
<td>( \omega )</td>
<td>0.780</td>
</tr>
<tr>
<td>0.9</td>
<td>Any</td>
<td>( \xi_R )</td>
<td>0.643</td>
</tr>
<tr>
<td>0.9</td>
<td>Any</td>
<td>( \alpha_R )</td>
<td>0.436</td>
</tr>
<tr>
<td>0.9</td>
<td>Any</td>
<td>( \psi_c )</td>
<td>0.764</td>
</tr>
<tr>
<td>1.1</td>
<td>Any</td>
<td>( \omega )</td>
<td>0.774</td>
</tr>
<tr>
<td>1.1</td>
<td>Any</td>
<td>( \xi_R )</td>
<td>0.609</td>
</tr>
<tr>
<td>1.1</td>
<td>Any</td>
<td>( \alpha_R )</td>
<td>0.424</td>
</tr>
<tr>
<td>1.1</td>
<td>Any</td>
<td>( \psi_c )</td>
<td>0.370</td>
</tr>
</tbody>
</table>

\[
\omega = 0.85 - 0.008 R_y; \quad \xi_R = \frac{\omega}{1 + \frac{R_y}{\sigma_{sc,u}} \left(1 - \frac{\omega}{1.1}\right)}; \quad \alpha_R = \xi_R \left(1 - 0.5 \xi_R\right); \quad \psi_c = \frac{\sigma_{sc,u}}{R_y \left(1 - \frac{\omega}{1.1}\right)}
\]

Note: Values \( \omega, \xi_R, \alpha_R \) and \( \psi_c \) given in Table 18 are calculated without considering coefficients \( \gamma_{bi} \) according to Table 9.
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:

\[ x = \frac{R_s A_s - R_y A_y}{R_s b} \]  

(16)

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

\[ M \leq R_s b x (h_0 - 0.5x) + R_s A_y \left( h_0 - a' \right) \]  

(17)

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

\[ M \leq \alpha_R R_s b h_0^2 + R_s A_y \left( h_0 + a' \right) \]  

(18)

Where \( \alpha_R = \xi_R \left( 1 - 0.5 \xi_R \right) \)

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

\[ M \leq R_s A_y (h_0 - a') \]  

(19)

Note. If the height of compressed zone determined considering of a half of compressed reinforcement,

\[ \xi \leq \frac{b - b_{ssc}}{2} \]  

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 \( A_s \).


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:

\[ M \leq R_s A_y \left( h_0 - 0.5x \right) \]  

(20)

Where height of compressed zone is \( x = \frac{R_s A_y}{R_s b} \)

- by \( x \geq \xi_R h_0 \) in compliance with the condition:

\[ M \leq \alpha_R R_s b h_0^2 \]  

(21)

at the same time design load carrying capacity of the section can be increased using recommendations of Item 3.15b \( [\xi_R, \alpha_R] \) – 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:

\[ \alpha_m = \frac{M}{R_s b h_0^2} \]  

(22)

If \( \alpha_m \leq \alpha_R \) (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:

\[ A_s = \frac{M}{R_s \xi h_0} \]  

(23)

Where \(\xi\) is determined according to Table 20 according to value \(\alpha_m\).

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 \(\gamma_{k2} = 0.9\) (see Item 3.1) tensile reinforcement can be chosen according to Annex 2.

<p>| Table 20 |
|------------------|------------------|------------------|------------------|------------------|------------------|------------------|------------------|</p>
<table>
<thead>
<tr>
<th>(\xi)</th>
<th>(\xi')</th>
<th>(\alpha_m)</th>
<th>(\xi)</th>
<th>(\xi')</th>
<th>(\alpha_m)</th>
<th>(\xi)</th>
<th>(\xi')</th>
<th>(\alpha_m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01</td>
<td>0.995</td>
<td>0.010</td>
<td>0.26</td>
<td>0.870</td>
<td>0.226</td>
<td>0.51</td>
<td>0.745</td>
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<tr>
<td>0.02</td>
<td>0.990</td>
<td>0.020</td>
<td>0.27</td>
<td>0.865</td>
<td>0.234</td>
<td>0.52</td>
<td>0.740</td>
<td>0.385</td>
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<tr>
<td>0.03</td>
<td>0.985</td>
<td>0.030</td>
<td>0.28</td>
<td>0.860</td>
<td>0.241</td>
<td>0.53</td>
<td>0.735</td>
<td>0.390</td>
</tr>
<tr>
<td>0.04</td>
<td>0.980</td>
<td>0.039</td>
<td>0.29</td>
<td>0.855</td>
<td>0.243</td>
<td>0.54</td>
<td>0.730</td>
<td>0.394</td>
</tr>
<tr>
<td>0.05</td>
<td>0.975</td>
<td>0.049</td>
<td>0.30</td>
<td>0.850</td>
<td>0.255</td>
<td>0.55</td>
<td>0.725</td>
<td>0.399</td>
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<tr>
<td>0.06</td>
<td>0.970</td>
<td>0.058</td>
<td>0.31</td>
<td>0.845</td>
<td>0.262</td>
<td>0.56</td>
<td>0.720</td>
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<tr>
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<td>0.965</td>
<td>0.068</td>
<td>0.32</td>
<td>0.840</td>
<td>0.269</td>
<td>0.57</td>
<td>0.715</td>
<td>0.407</td>
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<td>0.960</td>
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<td>0.835</td>
<td>0.276</td>
<td>0.58</td>
<td>0.710</td>
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</tr>
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<td>0.955</td>
<td>0.086</td>
<td>0.34</td>
<td>0.830</td>
<td>0.282</td>
<td>0.59</td>
<td>0.705</td>
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<td>0.10</td>
<td>0.950</td>
<td>0.095</td>
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<td>0.825</td>
<td>0.289</td>
<td>0.60</td>
<td>0.700</td>
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<td>0.805</td>
<td>0.314</td>
<td>0.64</td>
<td>0.685</td>
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<td>0.70</td>
<td>0.650</td>
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<td>0.640</td>
<td>0.460</td>
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<td>0.630</td>
<td>0.466</td>
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<td>0.343</td>
<td>0.78</td>
<td>0.610</td>
<td>0.476</td>
</tr>
<tr>
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<td>0.180</td>
<td>0.45</td>
<td>0.775</td>
<td>0.349</td>
<td>0.80</td>
<td>0.600</td>
<td>0.481</td>
</tr>
<tr>
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<td>0.895</td>
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<td>0.770</td>
<td>0.354</td>
<td>0.85</td>
<td>0.575</td>
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</tr>
<tr>
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<td>0.47</td>
<td>0.765</td>
<td>0.360</td>
<td>0.90</td>
<td>0.550</td>
<td>0.495</td>
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<tr>
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<td>0.885</td>
<td>0.204</td>
<td>0.48</td>
<td>0.760</td>
<td>0.365</td>
<td>0.95</td>
<td>0.525</td>
<td>0.499</td>
</tr>
<tr>
<td>0.24</td>
<td>0.880</td>
<td>0.211</td>
<td>0.49</td>
<td>0.755</td>
<td>0.370</td>
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<td>0.500</td>
<td>0.500</td>
</tr>
<tr>
<td>0.25</td>
<td>0.875</td>
<td>0.219</td>
<td>0.50</td>
<td>0.750</td>
<td>0.375</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

For bending moments of rectangular section:

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

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:

\[ A_s' = \frac{M - 0.4 R_s h_0^2}{R_s (h_0 - a')} \]  

(24)

\[ A_s = \frac{0.55 R_s h_0}{R_s} + A_s' \]  

(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:

$$A_s = \xi bh_0 \frac{R_b}{R_s} + A_s'$$  \hspace{1cm} (26)

Where $\xi$ is determined according to Table 20 depending on the value

$$\alpha_m = \frac{M - R_c A_s (h_0 - a')}{R_b bh_0^2} \geq 0$$

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:

$$R_s A_s \leq R_b b_f h_f + R_{sc} A_s'$$  \hspace{1cm} (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

$$M \leq R_b bx(h_0 - 0.5x) + R_b \left( b_f - b \right) h_f \left( h_0 - 0.5h_f \right) + R_{sc} A_s' (h_0 - a')$$  \hspace{1cm} (28)

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

$$x = \frac{R_s A_s - R_{sc} A_s' - R_b \left( b_f - b \right) h_f}{R_b b}$$  \hspace{1cm} (29)

And it is taken no more than $\xi \frac{h_x}{h_0}$ (see Table 18 and 19).

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

$$M \leq \alpha_R R_b bh_0^2 + R_b \left( b_f - b \right) h_f \left( h_0 - 0.5h_f \right) + R_{sc} A_s' (h_0 - a')$$  \hspace{1cm} (30)

Where $\alpha_R$ – 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.


a – in a flange; b – in a rib

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

$$A_s' = \frac{M - \alpha_R R_b bh_0^2 - R_b \left( b_f - b \right) h_f \left( h_0 - 0.5h_f \right)}{R_{sc} (h_0 - a') \right)}  \hspace{1cm} (31)$$
where $\alpha_R$ – 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:

$$M \leq R_p b_f' h_f' (h_0 - 0.5h_f') + R_{sc} A_s (h_0 - a')$$  \hspace{1cm} (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:

$$A_s = \frac{R_p \left[ \frac{2bh_0}{b_f} + \left( b_f' - b \right) h_f' \right] + R_{sc} A_s}{R_s}$$  \hspace{1cm} (33)

where $\xi$ is determined according to Table 20 depending on the value

$$\alpha_m = \frac{M - R_p \left( b_f' - b \right) h_f' (h_0 - 0.5h_f') - R_{sc} A_s (h_0 - a')}{R_p b h_0^2}$$  \hspace{1cm} (34)

At the same time the condition $\alpha_m \leq \alpha_R$ 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 $h_f' \geq 0.1h - 1/2$ 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 $h_f' \geq 0.1h - 6h_f'$;

By $0.05h \leq h_f' < 0.1h - 3h_f'$;

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 ($R_p = 7.7$ 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 $\alpha_m$ is determined by Formula (22):

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

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 $\xi = 0.835$

Required cross section of tensile reinforcement is to be determined by formula (23):
\[ A_s = \frac{M}{R_s \xi h_0} = \frac{200 \cdot 10^6}{280 \cdot 0.835 \cdot 560} = 1528 \text{ mm}^2. \]

It is taken \(2\Omega 28 + 1\Omega 25\) (\(A_s = 1598 \text{ mm}^2\))

**Example 3.** *Given:* a section with dimensions \(b = 300 \text{ mm}; h = 800 \text{ mm}; a = 70 \text{ mm};\) tensile reinforcement A-III \((R_s = 365 \text{ Mega Pascal});\) its section area \(A_s = 2945 \text{ mm}^2\) \((6\Omega 25);\) \(\gamma_{b2} = 0.9\) (no loads of short duration); heavy-weight concrete B25 \((R_b = 13 \text{ Mega Pascal});\) bending moment \(M = 550 \text{ kN} \cdot \text{m}.

*It is required* to check the section strength.

**Calculation.** \(h_0 = 800 - 70 = 730 \text{ mm}.\) 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):

\[ R_s A_s (h_0 - 0.5x) = 365 \cdot 2945 (730 - 0.5 \cdot 276) = 636.4 \cdot 10^6 \text{ N} \cdot \text{mm} = 636.4 \text{ kN} \cdot \text{m} > M = 550 \text{ kN} \cdot \text{m}, \]

that means the strength is corresponding to norms.

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

*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 \(\gamma_{b2} = 0.9 \frac{M_\mu}{M_I} = 0.9 \frac{780}{670} = 1.05 < 1.1\) so we take \(R_b = 8.5 \cdot 1.05 = 8.93 \text{ Mega Pascal}.\)

We calculate \(h_0, 800 - 50 = 750 \text{ mm}.\)

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

\[ \alpha_m = \frac{M}{R_b bh_0^2} = \frac{780 \cdot 10^6}{8.93 \cdot 300 \cdot 750^2} = 0.518 \]

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 \(a' = 30 \text{ mm}\) we determine required section of compressed and tensile reinforcement by formulas (24) and (25):

\[ A_s = \frac{M - 0.4R_b bh_0^2}{R_{sc}(h_0 - a')} = \frac{780 \cdot 10^6 - 0.4 \cdot 8.93 \cdot 300 \cdot 750^2}{365(750 - 30)} = 674 \text{ mm}^2 \]

\[ A_s = \frac{0.55bh_0 R_b}{R_s} + A_s = \frac{0.55 \cdot 300 \cdot 750 \cdot 8.93}{365} + 674 = 3702 \text{ mm}^2. \]

We take \(A_s = 763 \text{ mm}^2\) \((3\Omega 18); A_s = 4021 \text{ mm}^2\) \((5\Omega 32).\)
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 $A_s = 942$ mm$^2$ $(3\Phi20)$; bending moment $M = 580$ kN·m.

It is required to determine 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 $\alpha_m$ is determined in the following manner:

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

$\alpha_m = 187 < \alpha_R = 0.413$ (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 $3\Phi36$ ($R_s = 3054$ 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 $A_s = 4862$ mm$^2$ $(6\Phi32)$, of tensile reinforcement $A_s = 339$ mm$^2$ $(3\Phi12)$; 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):

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

We find $\xi = 0.604$ and $\alpha_R = 0.422$ according to table 18.

As $x = 420$ mm $> \xi h_0 = 0.604 \cdot 630 = 380$ mm so section strength is to be checked according to condition (18):

$$\alpha_R R_s b h_0^2 + R_{sc} A_s (h_0 - a') = 0.422 \cdot 13 \cdot 300 \cdot 630^2 + 365 \cdot 339(630 - 30) = 727.4 \cdot 10^6 \text{ N·mm} = 727.4 \text{ kN·m} > M = 600 \text{ kN·m},$$

that is section strength is provided.

T-SECTIONS AND DOUBLE T-SECTIONS

Example 7. Given: a section with dimensions $b' = 1500$ mm; $h' = 50$ 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 $A'_s = 0$; 
$$R_b b_j h_j (h_0 - 0.5 h_j) = 13 \cdot 1500 \cdot 50 (360 - 0.5 \cdot 50) = 326.6 \cdot 10^6 \text{N-mm} = 326.6 \text{kN-m},$$
that is the border of compressed zone goes in the flange and the calculation is made as for rectangular section with the width $b = b'_f = 1500 \text{mm}$ in compliance with Item 3.18.

We determine the value $\alpha_m$:
$$\alpha_m = \frac{M}{R_b bh_0^2} = \frac{300 \cdot 10^6}{13 \cdot 1500 \cdot 360^2} = 0.119 < \alpha_R = 0.422$$  (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 $4\phi 28$ ($A_s = 2463 \text{ mm}^2$).

**Example 8.** Given: a section with dimensions $b'_f = 400 \text{ mm}$, $h_j = 120 \text{ mm}$, and $b = 200 \text{ mm}$, $h = 600 \text{ mm}$, $a = 60 \text{ mm}$; heavy-weight concrete B15 ($R_b = 7.7 \text{ MPa}$ by $\gamma_{h2} = 0.9$); reinforcement A-III ($R_s = 365 \text{ MPa}$); bending moment $M = 270 \text{ kN-m}$.  

*It is required* to determined section area of tensile reinforcement.  

**Calculation:**
$$h_0 = 600 - 60 = 540 \text{ mm}.$$  
The section strength is checked in compliance with Item 3.22 on the hypothesis that compressed reinforcement is not required.

As $R_b b'_f h_j (h_0 - 0.5 h_j) = 7.7 \cdot 400 \cdot 120 (540 - 0.5 \cdot 120) = 177.4 \cdot 10^6 \text{N-mm} = 177.4 \text{kN-m} < M = 270 \text{kN-m}$, 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 $\alpha_m$:
$$\alpha_m = \frac{M - R_b (b'_f - b) h_j (h_0 - 0.5 h_j)}{R_b bh_0^2} = \frac{270 \cdot 10^6 - 7.7 (400 - 200) \cdot 120 (540 - 0.5 \cdot 120)}{7.7 \cdot 200 \cdot 540^2} = 0.404 < \alpha_R = 0.44$$
(see Table 18), so compressed reinforcement is not required.

According to Table 20 by $\alpha_m = 0.404$ we find $\zeta = 0.563$, then
$$A_s = \frac{M}{R_s \zeta h_0} = \frac{300 \cdot 10^6}{365 \cdot 0.563 \cdot 200 \cdot 540 + (400 - 200) \cdot 120 \cdot 7.7}{365} = 1789 \text{ mm}^2.$$  
We take $4\phi 25$ ($A_s = 1964 \text{ mm}^2$).

**Example 9.** Given: a section with dimensions $b'_f = 400 \text{ mm}$, $h_j = 100 \text{ mm}$, $b = 200 \text{ mm}$, $h = 600 \text{ mm}$, $a = 70 \text{ mm}$; heavy-weight concrete B25 ($R_b = 13 \text{ MPa}$ by $\gamma_{h2} = 0.9$); tensile reinforcement A-III ($R_s = 365 \text{ MPa}$), its section area $A_s = 1964 \text{ mm}^2$ (4Ø25); $A'_s = 0$; bending moment $M = 300 \text{ kN-m}$.  

*It is required* to check the strength of the section.  

**Calculation:**
$$h_0 = 600 - 70 = 530 \text{ mm}.$$  
The section strength is checked in compliance with Item 3.20, taking $A'_s = 0$. As $R_s A_s = 365 \cdot 1964 = 716860 \text{ N} > R_b b'_f h_j = 13 \cdot 400 \cdot 100 = 520000 \text{ N}$, 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):
\[
x = \frac{R_b A_x - R_y (b_f' - b) h_f'}{R_b b} = \frac{365 \cdot 1964 - 13(400 - 200) \cdot 100}{13 \cdot 200} = 176 \text{ mm} < \xi h_0 = 0.604 \cdot 530 = 320 \text{ mm} \quad (\xi \text{ is found according to table 18});
\]

\[R_b b (h_0 - 0.5x) + R_y (b_f' - b) h_f' (h_0 - 0.5h_f') = 13 \cdot 200 \cdot 176 (530 - 0.5 \cdot 176) + \]
\[+ 13(400 - 200) \cdot 100 (530 - 0.5 \cdot 100) = 327.1 \cdot 10^6 \text{ N-mm} = 327.1 \text{ kN-m} > M = 300 \text{ kN-m}, \]
that is the strength of the section is provided.

MEMBERS WORKING IN SKEW BENDING

3.2.4. 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:

\[M_x \leq R_x \left[ A_{web} (h_0 - x_i / 3) + S_{ov,x} \right] + R_y S_{xx} \quad (35) \]

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 through 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);

\[A_{web} = A_b - A_{ov} \quad (36)\]

\[A_b \quad \text{compressed concrete zone area equal to:} \]

\[A_b = \frac{R_b A_x - R_y A_x'}{R_b} \quad (37) \]

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

\[x_i \quad \text{the measurements of compressed zone along the most compressed lateral side of the section determined by the following formula:} \]

\[x_i = \frac{2}{3} \frac{R_b A_{web}^2}{R_b b (b_f A_{web} + S_{ov,y}) + R_y S_{yy} - M_y} \quad (38) \]

\(b_0\) 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 \(x\);

\[S_{yy}\] is static moment of the area \(A_x\) 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 \(x\);

\[S_{xx}\] is static moment of the area \(A_x\) 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; \(I\) – 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 \(x_i\) is determined by the following formula:

\[x_i = -t + \sqrt{t^2 + 2A_{web} \tan \beta} \quad (39) \]

Where \(t = 1.5 \left( \frac{S_{ov,y} \tan \beta - S_{ov,x} A_{web}}{S_{yy} A_{web}^2} + b_b \tan \beta - h_b \right) ; \]

\(\beta\) is angle of dip of the bending moment plane to axis \(x\) that is \(\tan \beta = M_y / M_x\).
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 \( \beta \).

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:

\[
x_1 < \frac{1.5A_{web}}{b = b_{ov}}
\]

(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:

\[
\xi_i = \frac{b_{ov} \tan \theta = x_1}{(b_{ov} = b_{ov}) g \theta + h_{ov}} \leq \xi_R
\]

(41)

Where \( \xi_R \) – 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;

\( \theta \) – Angle of slope of the line bounding the compressed zone to axis \( y \); value of \( \tan \theta \) is determined by the following formula:

\[
\tan \theta = \frac{x_1^2}{2A_{web}}.
\]

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 \( R_i \) for each tensile rod by stress values equal to:

\[
\sigma_i \psi_c (\omega / \xi_i - 1) R_i \] But no more than \( R_i \).

Where \( \psi_c, \omega \) 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 \( \xi_i \) must not exceed value \( \xi_R \) more than by 20 percent, at the same time it is possible to make only one repeated calculation with replacement of values \( R_i \) in formula (37) for tensile rods for which \( \xi_i > \xi_R \) by stresses equal to:

\[
\sigma_i \psi_c (\omega / \xi_i - 1) + \frac{2}{3} R_i
\]

(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
  \[ x_j > h \]  (43)
- for double T-, T- and L-sections with a flange in tensile zone
  \[ x_j > h - h_f - b_{ov,\text{tg}} \theta \]  (44)
where \( h_f, b_{ov,\text{tg}} \theta \) 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 \( A_j \).

**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 \( \alpha_s \) is determined by means of the diagram depending on the following values:

\[
\alpha_{mx} = \frac{M_x - R_y S_{ov,x} - R_{s,y} S_{ss}}{R_y b_0 h_0^2},
\]
\[
\alpha_{my} = \frac{M_y - R_x S_{ov,y} - R_{s,x} S_{sy}}{R_x b_0^2 h_0}
\]
[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 \( \alpha_s \) 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:

\[ A_s = \left( \alpha_s b_0 h_0 + A_{ov} \right) \frac{R_y}{R_s} + A_j \]  (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 $\alpha_s$ 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 $\alpha_s$ on the diagram must not be located between axis $\alpha_{my}$ 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 $\sigma_s$ determined by formula (155). At the same time angle of slope of this line $\theta$ 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 $\theta$ (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 (ctg$\beta = 4$); section and location of reinforcement is corresponding to Draft 10; $\gamma_{d2} = 0.9$ (no loads of short duration); heavy-weight concrete B25 ($R_b = 13$ MPa); stretched...
reinforcement class A-III \( (R_s = 365 \text{ MPa}) \); its section area \( A_s = 763 \text{ mm}^2 \) \( (3\Omega18) \); \( A'_i = 0 \);
bending moment in a vertical plane \( M = 82.6 \text{ kN} \cdot \text{m} \).

It is required to check the strength of the section.

Calculation. In compliance with Draft 10 it is:

\[
\begin{align*}
\frac{h_0}{3} &= 400 - 30 - \frac{1 \cdot 30}{3} = 360 \text{ mm}; \\
b_0 &= \frac{2 \cdot 120 + 1 \cdot 30}{3} = 90 \text{ mm}; \\
b_{ov}' &= \frac{300 - 150}{2} = 75 \text{ mm}; \\
h_f' &= 80 + \frac{20}{2} = 90 \text{ mm}
\end{align*}
\]

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:

\[
\begin{align*}
A_{ov}' & = b_{ov}' h_f' = 75 \cdot 90 = 6750 \text{ mm}^2; \\
S_{ov,y} & = A_{ov}' \left( b_0 + b_{ov}' / 2 \right) = 6750(90 + 75 / 2) = 86.06 \cdot 10^4 \text{ mm}^3; \\
S_{ov,x} & = A_{ov}' \left( h_0 + h_f' / 2 \right) = 6750(360 + 90 / 2) = 212.6 \cdot 10^4 \text{ mm}^3.
\end{align*}
\]

As \( A_b > A_m \), so the calculation is continued as for T-section.

\[
A_{web} = A_{ov}' A_{ov} = 21420 - 6750 = 14670 \text{ mm}^2.
\]

The components of bending moment in the plane of axes \( y \) and \( x \) are correspondingly equal to (by \( \text{ctg}\beta = 4 \)):

\[
\begin{align*}
M_y &= M \sin \beta = \frac{M}{\sqrt{1+\text{ctg}^2\beta}} \frac{82.6}{\sqrt{1+4^2}} = 20 \text{ kN} \cdot \text{m}; \\
M_x &= M \cdot \text{ctg}\beta = 20 \cdot 4 = 80 \text{ kN} \cdot \text{m}
\end{align*}
\]

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

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

Let’s check condition (40):

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

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

Let’s check condition (41) for the least tensile rod. According to Draft 10 \( b_{0i} = 30 \text{ mm} \),

\[
h_{0i} = 400 - 30 = 370 \text{ mm;}
\]

\[
\text{tg} \theta = \frac{x_2^l}{2 A_{web}} = \frac{223^2}{2 \cdot 14670} = 1.695;
\]

42
\[
\xi_i = \frac{b_0 \tan \theta - x_i}{(b_0 + b_\infty) \tan \theta + h_0} = \frac{75 \cdot 1.695 + 223}{(30 + 75) \cdot 1.695 + 370} = 0.64 > \xi_R = 0.604
\]
(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 \( \sigma_s \) determined by formula (42), correcting values \( h_0 \) and \( b_0 \).

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

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

As all rods have the same diameter so new values \( A_p, b_0 \) and \( h_0 \) will be equal to:

\[
A_p = 21420 \frac{2 + 0.902}{3} = 20720 \text{ mm}^2;
\]
\[
b_0 = 2 \cdot 120 + 0.902 \cdot 30 = 92.0 \text{ mm};
\]
\[
h_0 = 400 - 30 - \frac{1 \cdot 30}{2 + 0.902} = 359.7 \text{ mm}
\]

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):

\[
t = 1.5 \left( \frac{S_{ov,y} \tan \beta - S_{ov,x}}{A_{web}} + b_0 \tan \beta - h_0 \right) = 1.5 \left( \frac{874100 \cdot 4 - 2124000}{13970} + 92 - 4 - 359.7 \right) = 159.8 \text{ mm}
\]
\[
x_1 = -t + \sqrt{t^2 + 2A_{web} \tan \beta} = -159.8 + \sqrt{159.8^2 + 2 \cdot 13970 \cdot 4} = 210.7 \text{ mm}
\]

Let’s check the section strength according to condition (35) taking \( S_{it} = 0 \):

\[
R_b \left[ A_{web} (h_0 - x_1 / 3) + S_{ov,x} \right] = 13 \left[ 13970 \left( 359.7 - \frac{210.7}{3} \right) + 212.4 \cdot 10^4 \right] = 80.2 \cdot 10^6 \text{ N-mm} > M_x = 80 \cdot 10^6 \text{ N-mm},
\]
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 \( M = 64 \text{ kN-m} \).

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

\[
M_y = M \sin \beta = \frac{64 \cdot 10^6}{\sqrt{1 + \tan^2 \beta}} = 15.5 \cdot 4 = 62 \text{ kN-m}
\]

We determine required quantity of reinforcement according to Item 3.25.

Taking values \( b_0, h_0, S_{ov,x}, S_{ov,y}, S_{xy} = S_{yx} = 0 \) according to Example 10 we find values \( \alpha_{mx} \) and \( \alpha_{my} \):

\[
\alpha_{mx} = \frac{M_x - R_b S_{ov,x}}{R_b b_0 h_0} = \frac{62 \cdot 10^6 - 13 \cdot 212.4 \cdot 10^4}{13 \cdot 90 \cdot 360^2} = 0.227;
\]
As \( \alpha_{xx} > 0 \) so the calculation is continued as for T-section.

As the point with coordinates \( \alpha_{xx} = 0.227 \) and \( \alpha_{yy} = 0.114 \) on Drawing 9 is located on the right side of the curve corresponding to parameter \( \frac{b + b_w}{b_0} = \frac{150 + 75}{90} = 2.5 \) and on the left side of the curve corresponding to parameter \( \frac{b_w}{b_0} = \frac{75}{90} = 0.83 \) 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 \( \alpha_{xx} = 0.227 \) and \( \alpha_{yy} = 0.114 \) we find \( \alpha_s = 0.25 \)

Then by \( A_s = 0 \) we have

\[
A_s = \left( \alpha_s b_0 h_0 + A_m \right) \frac{R_b}{R_s} = \left( 0.25 \cdot 90 \cdot 360 + 6750 \right) \frac{13}{365} = 529 \text{ mm}^2.
\]

We take rods 3Ø16 (\( A_j = 603 \text{ mm}^2 \)) 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/m², from plane of the panel – wind load 0.912 kN/m².

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 \( \gamma = 1.1 \cdot 1100 = 1210 \text{ kg/m}^3 \). So the load of dead weight of the panel will be:

\[
q_w = bh \gamma \cdot 0.01 = 0.34 \cdot 1.2 \cdot 1210 \cdot 0.01 = 4.94 \text{ kN/m}
\]

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

\[
q_w = 1.2 \cdot 4.94 = 5.92 \text{ kN/m}
\]

Load of located above glazing weight is \( q_g = 3.93 \cdot 3 = 11.8 \text{ kN/m} \).

Total load in the panel plane is:

\[
q_s = q_w + q_g = 5.92 + 11.8 = 17.72 \text{ kN/m},
\]

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

\[
M_x = \frac{q_s l^2}{8} = \frac{17.72 \cdot 5.8}{8} = 74.5 \text{ kN}\cdot\text{m}
\]

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

\[
q_s = 0.912(1.2 + 3) = 3.83 \text{ kN/m}
\]

and moment of this load is:

\[
M_y = \frac{q_s l^2}{8} = \frac{3.83 \cdot 5.8^2}{8} = 16.1 \text{ kN}\cdot\text{m}
\]
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 \( h = 1195 \text{ mm} \) and axis \( y \) parallel to dimension \( b = 340 \text{ mm} \).

**Draft 11. To example of calculation 12**

\( l - 8 \) – rods

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

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

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 \( R_s = R_{sc} = 365 \text{ MPa} \), and for rods 3 – 6 (Ø6) – \( R_s = 355 \text{ MPa} \), then:

\[
R_s A_s = 365 \cdot 236 + 355 \cdot 113 = 126250 \text{ N};
\]

\[
R_{sc} A_s = 365 \cdot 78.5 = 28650 \text{ N}.
\]

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

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

Area of compressed zone on the assumption that it’s of triangular form is determined by formula \( A_b = \frac{x_1^2}{2 \tan \theta} \) where \( x_1 \) is dimension of compressed zone by the section side \( h \) so \( x_1 \) is:

\[
x_1 = \sqrt{2 \tan \theta A_b} = \sqrt{2 \cdot 2.67 \cdot 42440} = 476 \text{ mm} < h = 1195 \text{ mm}.
\]

Dimension \( y_1 \) of compressed zone by section side \( b \) is:

\[
y_1 = \frac{x_1}{\tan \theta} = \frac{476}{2.76} = 172 \text{ mm} < b = 340 \text{ mm},
\]

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 \( \sigma_{si} \) in the closest to tensile zone border rods that is in rods 6 – 8 by formula (155), determining the ratios \( \xi = \frac{x}{h_{ij}} \) by formula \( \xi_i = \frac{x_i}{a_i \tan \theta = a_{si}} \) where \( a_{si} \) 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 \( \sigma_{sc,aw} = 400 \text{ MPa} \), \( \omega = 0.8 - 0.008 R_b = 0.8 - 0.008 \cdot 2.3 = 0.782 \) we get

\[
\sigma_{si} = \frac{\sigma_{sc,aw} \left( \omega \xi - 1 \right)}{1 - \frac{\omega}{1.1} \left( \xi_i - 1 \right)} = \frac{400 \left( 0.782 \xi_i - 1 \right)}{1 - 0.782 \xi_i} = 1082 \xi_i - 1384 \text{ (MPa)}
\]

The calculations we summarize in the following Table:

<table>
<thead>
<tr>
<th>Rod number</th>
<th>( A_{si}, \text{ mm}^2 )</th>
<th>( a_{yi}, \text{ mm} )</th>
<th>( a_{si}, \text{ mm} )</th>
<th>( a_{yi} \tan \theta + a_{si} )</th>
<th>( \xi_i )</th>
<th>( \sigma_{si} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
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 $A_b$:

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

By a similar way we determine $x_1 = \sqrt{2} \cdot 2.67 \cdot 40080 = 463 \text{ mm}$.

So for rod 6 we have:

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

$$\sigma_{s6} = \frac{1082}{0.699} = 1544 \text{ MPa},$$

That is value $\sigma_{s6}$ 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 $x - M_{xy}$ and $M_{yy}$.

$$M_{xy} = R_b A_b \left( a_{xi} - \frac{x_1}{3} \right) - \Sigma \sigma_i A_i \left( a_{xi} - a_{yi} \right) = 2.3 \cdot 40080 \left( 1115 - \frac{463}{3} \right) -$$

$$- 355 \cdot 2.83(1115 - 1015) - (355 + 160) \cdot 28.3(1115 - 555) -$$

$$- (356.5 - 365) (1115 - 80) = 78.4 \times 10^6 \text{ N-mm} =$$

$$+ 78.4 \text{ kN-m} > M_x = 74.5 \text{ kN-m};$$

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

$$M_{yy} = R_b A_b \left( a_{yi} - \frac{y_1}{3} \right) - \Sigma \sigma_i A_i \left( a_{yi} - a_{xi} \right) = 2.3 \cdot 40080 \left( 300 - \frac{173}{3} \right) -$$

$$- (365 \cdot 78.5 + 355 \cdot 28.3 + 160 \cdot 28.3 - 365 \cdot 78.5)(300 - 40) =$$

$$= 18.5 \times 10^6 \text{ N-mm} = 18.5 \text{ kN-m} > M_y = 16.1 \text{ kN-m}.$$

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 \( s_{\text{max}} \):

\[
s_{\text{max}} = \frac{\varphi_{b4} R_w b h_0^2}{Q}
\]

(46)

Where \( \varphi_{b4} \) – 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:

\[
Q \leq 0.3 \varphi_{w1} \varphi_{b1} R_b b h_0
\]

(47)

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

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

\[
\varphi_{w1} = 1 + 5 \alpha \mu_w
\]

(48)

but no more than 1.3;

Here \( \mu_w = \frac{A_{sw}}{b_s} \);

\( \varphi_{b1} \) – Coefficient determined by the following formula:

\[
\varphi_{b1} = 1 - \beta R_b
\]

(49)

here \( \beta \) 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:

\[ Q \leq Q_b + Q_{sw} \]  

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:

\[ Q = \frac{M_b}{c}, \]  

\[ M_b = \varphi_{b2}(1 + \varphi_f)R_{st}b_h^2; \]  

\( \varphi_{b2} \) – Coefficient considering concrete type and determined by Table 21;

\( \varphi_f \) – Coefficient considering influence of compressed flanges of T- and double-T-section elements and determined by the following formula:

\[ \varphi_f = 0.75 \left( \frac{b_f - b}{b} \right) \]  

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

<table>
<thead>
<tr>
<th>Concrete</th>
<th>( \varphi_{b2} )</th>
<th>( \varphi_{b3} )</th>
<th>( \varphi_{b4} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heavy-weight concrete</td>
<td>2.00</td>
<td>0.6</td>
<td>1.5</td>
</tr>
<tr>
<td>Fine concrete</td>
<td>1.70</td>
<td>0.5</td>
<td>1.2</td>
</tr>
<tr>
<td>Light-weight concrete</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- D1900</td>
<td>1.90</td>
<td>0.5</td>
<td>1.2</td>
</tr>
<tr>
<td>- D1800 and lower by fine aggregate dense</td>
<td>1.75</td>
<td>0.4</td>
<td>1.0</td>
</tr>
<tr>
<td>- porous</td>
<td>1.50</td>
<td>0.4</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Value \( Q_b \) is taken no less than \( Q_{b,\min} = \varphi_{b3}(1 + \varphi_f)R_{st}b_h^2 \ (\varphi_{b3} - \text{see Table 21});

\( Q_{sw} \) is cross force carried by stirrups equal to:
\[ Q_{sw} = q_{rw} c_0 \]  
(54)

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

\[ q_{sw} = \frac{R_{sw} A_{sw}}{s} ; \]  
(55)

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

\[ c_0 = \sqrt{\frac{M_b}{q_{rw}}} \]  
(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:

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

It is possible not to follow the requirement (57) if in formula (52) it is considered such reduced value \( R_{sw} b \) when condition (57) becomes an equation that is if \( M_b = 2h_0^2 q_{rw} \frac{\phi_{b,2}}{\phi_{b,3}} \);

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 \( \frac{3h}{bb \phi \phi} \).

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 \( \sqrt{\frac{M_b}{q_{sw}}} \) and if \( q_{1} > 0.65 q_{sw} \) so it is necessary to take \( c = \sqrt{\frac{M_b}{q_{1} + q_{sw}}} \) 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 \( Q_{\max} - q_{w} c \) 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 $\chi_i = \frac{Q_i - Q_{bi}}{Q_{bi}}$ by one of the following formulas:

If $\varepsilon_i < \varepsilon_{0i}$,

$$q_{sw(i)} = \frac{Q_i - Q_{bi}}{2h_0} c_0;$$

(58)

If $\varepsilon_{0i} \leq \varepsilon_i \leq \frac{c_i}{c_0}$,

$$q_{sw(i)} = \frac{Q_i - Q_{bi}}{c_0} \varepsilon_{0i};$$

(59)

If $\frac{c_i}{c_0} < \varepsilon_i \leq \frac{c_i}{h_0}$,

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

(60)

If $\varepsilon_i > \frac{c_i}{h_0}$,

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

(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 $c = c_i$;
- $Q_{bi,min}, M_b$ 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 $Q_{max} \leq \frac{Q_{bi}}{0.6}$

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

(62)

By $\frac{M_b}{h_0} + Q_{bi} > Q_{max} > \frac{Q_{bi}}{0.6}$

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

(63)
In both cases $q_{sw}$ is taken no less than \( \frac{Q_{\text{max}} - Q_{b1}}{2h_0} \):

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

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

(64)

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

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

Here

- $Q_{b1} = 2\sqrt{M_b q_1}$
- $Q_{\text{max}}$ – 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 $q_1 > q_{sw1} - q_{sw2}$,

$$l_1 = c - \frac{M_b}{l c + q_{sw1} c_{01} - Q_{\text{max}} + q_1 c}{q_{sw1} - q_{sw2}}.$$  

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 \( q_1 \leq q_{sw1} - q_{sw2} \)

\[
l_1 = \frac{Q_{\text{max}} - (Q_{b,\text{min}} + q_{sw2}c_{01})}{q_1} - c_{01}
\]

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

\[
M_b = 2h_0^2 q_{sw2} \varphi_{b2} / \varphi_{b3} \quad \text{and} \quad Q_{b,\text{min}} = 2h_0 q_{sw2} \text{ corrected according to Item 3.31}.
\]

At the same time the sum \( Q_{b,\text{min}} + q_{sw2}c_{01} \) is taken no less than the uncorrected value \( Q_{b,\text{min}} \).

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:

\[
Q_{r,\text{inc}} = R_{sw} A_{s,\text{inc}} \sin \theta
\]

Where \( A_{s,\text{inc}} \) — section area of bend-up bars crossing dangerous inclined crack with the projection length \( c_0 \);

\( \theta \) 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 \( Q_{sw} + Q_{r,\text{inc}} + Q_b = q_{sw}c_0 + Q_{r,\text{inc}} + \frac{M_b}{c_0} \) 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:

\[
1 - q_{sw}c_{01} + R_{sw} A_{s,\text{inc1}} \sin \theta + \frac{M_b}{c_{01}}
\]

\[
2 - q_{sw}c_{02} + R_{sw} A_{s,\text{inc2}} \sin \theta + \frac{M_b}{c_{02}}
\]

\[
3 - q_{sw}c_{03} + R_{sw} A_{s,\text{inc3}} \sin \theta + \frac{M_b}{c_{03}}
\]

\[
4 - q_{sw}c_0 + R_{sw} (A_{s,\text{inc1}} + A_{s,\text{inc2}}) \sin \theta + \frac{M_b}{c_{02}}
\]

[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 $\beta$ between compressed and tensile surfaces of member must meet the requirement $\tan \beta < 0.4$.

**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:

$$q_1 < 0.56 q_{sw} - 2.5 \sqrt{q_{inc} q_{sw}}$$

so value $c$ is determined in by the following formula

$$c = \frac{M_{bi}}{\sqrt{q_{inc} + \sqrt{q_{inc} q_{sw} + q_1}}}$$

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

$$c = \frac{M_{bi}}{\sqrt{q_{inc} + q_{sw} + q_1}}$$  \hspace{1cm} \text{(At the same time } c_0 = c)$$

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

$$c = \frac{M_{bi}}{\sqrt{q_{inc} + 2 q_{sw} \tan \beta + q_1}}$$  \hspace{1cm} \text{(At the same time } c_0 = 2 h_{01})$$

Here $q_{inc} = \varphi_{h2} R_b \tan \beta$;

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

$\beta$ – Angle between compressed and tensile surfaces of the beam;

$q_1$ – See Item 3.32.

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

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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 $Q_{tw}$ 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 $t g \beta > 0.1$ 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:

$$\left[ \frac{Q_x}{Q_{bw(x)}} \right]^2 + \left[ \frac{Q_y}{Q_{bw(y)}} \right]^2 \leq 1$$  \hspace{1cm} (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:
\[ Q_{\text{max}} \leq 2.5 R_{\text{tr}} b h_0 \]  \hspace{1cm} (71)

Where \( Q_{\text{max}} \) – maximum shear force at the support surface;

\[ Q \leq \frac{\varphi_{\text{h}_4} R_{\text{h}_0} b h_0^2}{c} \]  \hspace{1cm} (72)

where \( Q \) is shear force at the end of inclined section;
\( \varphi_{\text{h}_4} \) – Coefficient determined by Table 21;
\( c \) – Is projection length of inclined section starting from the support; value \( c \) is taken no more than \( c_{\text{max}} = 2.5 h_0 \).

In continuous flat slabs with constrained edges (connected with other elements or having supports) it is possible to divide the mentioned value \( c_{\text{max}} \) by the coefficient \( \alpha \):
\[
\frac{h_b}{0.5 \pi} + \alpha = \alpha
\]  \hspace{1cm} (73)

but no more than 1.25.

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

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:
\[ q_1 \leq \frac{\varphi_{\text{h}_4} R_{\text{h}_0} b}{(c_{\text{max}} / h_0)^2} \]  \hspace{1cm} (74)

so value \( c \) in condition (72) is taken equal to \( c_{\text{max}} \) and if condition (74) is not met so
\[ c = h_0 \frac{\varphi_{\text{h}_4} R_{\text{h}_0} b}{q_1} \]  \hspace{1cm} (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 \( c_{\text{max}} \).

**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 \( c_{\text{max}} \) is taken equal to \( c_{\text{max}} = \frac{2.5 h_{01}}{1 + 1.25 \tan \beta} \) at the same time for continuous flat plates mentioned in Item 3.40 \( c_{\text{max}} = \frac{2.5 h_{01}}{\alpha + 1.25 \tan \beta} \),

Where \( h_{01} \) – working height in support section;
\( \beta \) is angle between tensile and compressed surfaces of the element;
\( \alpha \) 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:

\[
c = h_{01} \frac{1}{\tan^2 \beta l + q_1 / (\varphi_b R_{b1} b)}
\]  

(76)

but no more than \( c_{\text{max}} \) 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:

\[
M \leq R_s A_s z_s + \Sigma R_{sw} A_{sw} z_{sw} + \Sigma R_{sw} A_{sw} z_{sw} z_{s,inc}
\]

(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 reinforcement, stirrups and bend-up bars crossing stretched zone of inclined section;

\( \Sigma R_{sw} A_{sw} z_{sw}, \Sigma R_{sw} A_{sw} z_{sw} z_{s,inc} \) – 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 \( \Sigma R_{sw} A_{sw} z_{sw,inc} \cos \theta \) (where \( \theta \) is angle of inclination of bend-up bars to the longitudinal axis of the member).

Value \( z_s \) can be taken equal to \( h_b - 0.5x \) 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:

\[
\Sigma R_{sw} A_{sw} z_{sw} = 0.5 q_{sw} c^2
\]

(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:
\[ z_{x, \text{inc}} = z_x \cos \theta + (c - a_1) \sin \theta \]  

(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 0.8\( c_{\text{max}} \).

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 \( \gamma_{s5} \) equal to:

\[ \gamma_{s5} = \frac{l_x}{l_{an}} \]  

(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:

\[ l_{an} = \left( \omega_{an} \frac{R_s}{R_b} + \Delta \lambda_{an} \right) d \]  

(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 \( \omega_{an} \) 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 \( \omega_{an} \) 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 \( \mu_v \) is volume coefficient of reinforcement determined as for welded meshes by formula (99), for stirrups – by formula \( \mu_v = \frac{A_{sw}}{2ds} \) (where \( A_{sw} \) and \( s \) – are correspondingly section area of a bounding stirrup and its spacing), in any case value \( \mu_v \) is taken no more than 0.06.

Concrete compression stress on the support \( \sigma_b \) is determined by means of dividing of support reaction by support area of the element and is taken no more than 0.5\( R_b \).

The length \( l_{an} \) is taken no less than 20\( d \) 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:

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

(82)

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

In formula (82):
- $n_w$ – Number of welded rods along the length $l$;
- $\varphi_w$ – Coefficient taken by Table 22;
- $d_w$ – Diameter of welded bars.

Table 22

<table>
<thead>
<tr>
<th>$d_w$</th>
<th>6</th>
<th>8</th>
<th>10</th>
<th>12</th>
<th>14</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varphi_w$</td>
<td>200</td>
<td>150</td>
<td>120</td>
<td>100</td>
<td>80</td>
</tr>
</tbody>
</table>

Finally value $R_sA_z$ is taken no more than value $R_sA_z$ without considering $\gamma_{s5}$ 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:

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

(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;
- $F_i$ – point load and distributed load within inclined section;
- $A_{s,inc}$ – section area of bend-up bars crossing the inclined section;
- $\theta$ – 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:

$$Q \leq \sqrt{2(R_s A_z z_s - M_0)(q_{sw} + q)}$$

(84)

where $Q$ – is shear force in support section.
$M_0$ – 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:

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

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:

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

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 $R_s A_s \tan \beta$ – by inclined compressed surface, and by value $R_s A_s \sin \beta$ – by inclined tensile section (where $\beta$ 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:

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

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 $R_s A_s \tan \beta$ and for beams with inclined tensile surface – by $R_s A_s \sin \beta$ (where $\beta$ 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 $0.8c_{max}$.
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:

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

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 $h_0/4$ 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:

$$R_{sw}A_{sw1} + R_{sw}A_{s,inc} \sin \theta \geq Q_1(1 - h_0/h_0) \quad (89)$$

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 \( \sin^2 \theta = \frac{(h_{01} - a')^2}{(h_{01} - a')^2 + (l_{sup} + a')^2} \) (where \( l_{sup} \) – see Item 3.99, \( a' \) – 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 \( R_b = 7.7 \text{ MPa} \) \( R_{tu} = 0.67 \text{ MPa} \) by \( \gamma_{b_2} = 0.9 \); \( E_b = 20.5 \cdot 10^3 \text{ MPa} \); rib of the slab is reinforced by plane welded framework with cross reinforcement rods A-III, diameter 8 mm \( A_{sw} = 50.3 \text{ mm}^2 \); \( R_{sw} = 285 \text{ MPa} \); \( E_s = 2 \cdot 10^5 \text{ MPa} \), with spacing \( s = 100 \text{ mm} \); equivalent live load \( v = 18 \text{ kN/m} \); gravity load of the slab and floor \( g = 3.9 \text{ kN/m} \); cross force on the support \( Q_{max} = 62 \text{ kN} \).

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. \( h_0 = 350 - 58 = 292 \text{ mm} \). Strength of inclined strip is to be calculated according to condition (47).

We determine coefficients \( \varphi_{w1} \) and \( \varphi_{1} \):

\[
\mu_w = \frac{A_{sw}}{b s} = \frac{50.3}{85 \cdot 100} = 0.0059;
\]

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

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 \( 0.3 \varphi_{w1} \varphi_{b1} R_b b h = 0.3 \cdot 1.29 \cdot 0.923 \cdot 7.7 \cdot 85 \cdot 292 = 68300 \text{ N} > Q_{max} = 62 \text{ kN} , \)

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 \( b_f = 475 - 85 = 390 \text{ mm} > 3 h_f = 3 \cdot 50 = 150 \text{ mm} \) so we take \( b_f = b = 150 \), and
\[
\varphi_f = 0.75 \frac{(v_f - b))v_f}{bh_0} = 0.75 \frac{150 \cdot 50}{85 \cdot 292} = 0.227 < 0.5;
\]
\[
M_b = \varphi_{b,2}(1 + \varphi_f)R_{sh}bh_0^2 = 2(1 + 0.227)0.67 \cdot 85 \cdot 292^2 = 11.92 \cdot 10^6 \text{ N-mm} = 11.92 \text{ kN-m};
\]
\[
q_{sw} = \frac{R_{sw}A_{sw}}{s} = \frac{285 \cdot 50.3}{100} = 143 \text{ N/mm (kN/m)}
\]
We determine value \(Q_{b,\min}\) taking \(\varphi_{b,3} = 0.6\):
\[
Q_{b,\min} = \varphi_{b,3}(1 + 0.227)0.67 \cdot 85 \cdot 292 = 12240 \text{ N} = 12.24 \text{ kN}.
\]
As \(\frac{Q_{b,\min}}{2h_0} = \frac{12.24}{2 \cdot 0.292} = 21 \text{ kN/m} < q_{sw} = 143 \text{ kN/m},\)
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 \(0.56q_{sw} = 0.56 \cdot 143 = 80 \text{ kN/m} > q_1 = 12.9 \text{ kN/m}\) 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
\[
Q_b = \frac{M_b}{c} = \frac{11.92}{0.962} = 12.4 \text{ kN} > Q_{b,\min} = 12.2 \text{ kN};
\]
\[
Q = Q_{\max} - q_c = 62 - 12.9 \cdot 0.962 = 49.3 \text{ kN}
\]
Projection length of inclined crack is equal to:
\[
c_0 = \sqrt{\frac{M_b}{q_{sw}}} = \sqrt{\frac{111.9}{143}} = 0.288 \text{ m} < 2h_0
\]
As \(c_0 = 0.288 < h_0 = 0.292 \text{ m}\) so we take \(c_0 = h_0 = 0.292 \text{ m}, \) so \(Q_{sw} = q_{sw}c_0 = 143 \cdot 0.292 = 41.8 \text{ 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:
\[
s_{\max} = \frac{\varphi_{b,4}R_{sh}bh_0^2}{Q_{\max}} = \frac{1.5 \cdot 0.67 \cdot 85 \cdot 292^2}{62 \cdot 10^3} = 117.5 \text{ mm} > s = 100 \text{ mm}.
\]
Conditions of Item 5.69 \(s < h/2 = 350/2 = 175 \text{ mm}\) and \(s < 150 \text{ mm}\) are also met.

**Example 14.** Given: free supported reinforced concrete beam of the floor with the span \(l = 5.5 \text{ m} \); equivalent distributed live load on the beam \(v = 36 \text{ kN/m} \); dead load \(g = 14 \text{ kN/m} \);
dimensions of the cross section \(b = 200 \text{ mm}, h = 400 \text{ mm}, \) \(h_0 = 370 \text{ mm} \);
heavy-weight concrete B15 ( \(R_b = 7.7 \text{ MPa} \) \(R_{sh} = 0.67 \text{ MPa} \) by \(\gamma_{sh} = 0.9 \) ); stirrups of reinforcement A-I ( \(R_{sw} = 175 \text{ MPa} \)).

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:
\[ Q_{\text{max}} = \frac{ql}{2} = \frac{50 \cdot 5.5}{2} = 137.5 \text{ kN}, \]

Where \( q = v + g = 36 + 14 = 50 \text{ kN/m} \)

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:

\[ M_b = \varphi_{b2} R_m b_h h_0^2 = 2 \cdot 0.67 \cdot 200 \cdot 370^2 = 36.7 \cdot 10^6 \text{ N-mm} = 36.7 \text{ kN-m}. \]

According to Item 3.32

\[ q_1 = g + v/2 = 14 + 36/2 = 32 \text{ kN/m (N/mm)}; \]

\[ Q_{bl} = 2\sqrt{M_b q_1} = 2\sqrt{36.7 \cdot 32} = 68.4 \text{ kN}. \]

As \( \frac{Q_{bl}}{0.6} = \frac{68.4}{0.6} = 114 \text{ kN} < Q_{\text{max}} = 137.5 \text{ kN}, \) and \( Q_{\text{max}} < \frac{M_b}{h_0} + Q_{bl} = 36.7 + 68.4 = 167 \text{ kN}, \)

Stirrups quantity is to be determined by formula (63):

\[ q_{sw} = \frac{(Q_{\text{max}} - Q_{bl})^2}{M_b} = \frac{(137.5 - 68.4)^2}{36.7} = 130 \text{ kN/m (N/mm)}; \]

At the same time as \( \frac{Q_{\text{max}} - Q_{bl}}{2 h_0} = \frac{(137.5 - 68.4)^3}{2 \cdot 370} = 93.44 \text{ N/mm} < 130 \text{ N/mm}, \)

So \( q_{sw} = 130 \text{ N/mm} \)

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 \( - \frac{3}{4} h = 300 \) and 500 mm. Maximum allowable spacing at the support according to Item 3.29 is equal to:

\[ s_{\text{max}} = \frac{\varphi_{b1} R_m b_h h_0^2}{Q_{\text{max}}} = \frac{1.5 \cdot 0.67 \cdot 200 \cdot 370^2}{137.5 \cdot 10^3} = 200 \text{ mm} \]

We take stirrups spacing at the support \( s_1 = 150 \) mm and in the span \( - 2 s_1 = 300 \) mm so:

\[ A_{rsw1} = \frac{q_{sw} s_1}{R_m} = \frac{130 \cdot 150}{175} = 111 \text{ mm}^2. \]

We take two stirrups with diameter 10 mm in the cross section \( (A_{sw} = 157 \text{ mm}^2). \)

So excepted density of stirrups at he support and in the span will be equal to:

\[ q_{rsw1} = \frac{R_m A_{rsw}}{s_1} = \frac{175 \cdot 157}{150} = 183.2 \text{ N/mm}; \]

\[ q_{rsw2} = 0.5 q_{rsw1} = 0.5 \cdot 183.2 = 91.6 \text{ N/mm}. \]

Let’s check condition (57) by means of calculation of \( Q_{b, \text{min}} \):

\[ Q_{b, \text{min}} = \varphi_{b3} (1 + \varphi_f) R_b b_h h_0 = 0.6 \cdot 0.67 \cdot 200 \cdot 370 = 29750 \text{ N}. \]

Then

\[ \frac{Q_{b, \text{min}}}{2 h_0} = \frac{29750}{2 \cdot 370} = 40.2 \text{ N/mm} < q_{rsw1} = 183.2 \text{ N/mm}; \]

\[ \frac{Q_{b, \text{min}}}{2 h_0} = 40.2 \text{ N/mm} < q_{rsw2} = 96.1 \text{ N/mm}. \]

So values \( q_{rsw1} \) and \( q_{rsw2} \) are not to be corrected.

We determine the length of part \( l_1 \) with stirrups quantity \( q_{rsw1} \) according to Item 3.34. As

\[ q_{rsw1} - q_{rsw2} = q_{rsw2} = 91.6 \text{ N/mm} > q_1 = 32 \text{ N/mm} \]

so value \( l_1 \) is to be calculated according to the following formula:
\[ I_1 = \frac{Q_{\text{max}} - Q_{b,\text{min}} - q_{sw}c_{201}}{q_1} - c_{01} = \frac{137.5 \cdot 10^3 - 29750 - 91.6 \cdot 448}{32} - 448 = 1637 \text{ mm} > \frac{l}{4} = \frac{5.5}{4} = 1.375 \text{ m} \]

(Here \( c_{01} = \sqrt{\frac{M_b}{q_{sw}}} = \sqrt{\frac{36.7 \cdot 10^6}{183.2}} = 448 \text{ mm} \))

We take the length of the part with the stirrups spacing \( s_1 = 150 \text{ mm} \) 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 \((R_m = 0.67 \text{ MPa by } \gamma_{b2} = 0.9)\); stirrups of reinforcement A-I \((R_{sw} = 175 \text{ MPa})\).

*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:

\[ \varphi_{b2} = 2 \text{ (See Table 21)}; \]

\[ h_f' = 100 + 60/2 = 130 \text{ mm (see Draft 26b)}; \]

\[ b_f' - b = 220 - 80 = 140 \text{ mm} < 3h_f'; \]

\[ h_0 = 890 - 80 = 810 \text{ mm}; \]

\[ \varphi_f = 0.75 \frac{(b_f' - b)h_f'}{bh_0} = 0.75 \frac{140 \cdot 130}{80 \cdot 810} = 0.211 < 0.5; \]

\[ M_b = \varphi_{b2}(l + \varphi_f)R_{bw}bh_0^2 = 2(l + 0.211)0.67 \cdot 80 \cdot 810^2 = 85.2 \cdot 10^6 \text{ N-mm} = 85.2 \text{ kN-m}. \]

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 – \( c_1 = 1.35 \text{ m} \).

Shear force at the distance \( c_1 \) from the support is equal to \( Q_1 = 105.2 \text{ kN} \) (see Draft 26).

From formula (51) we have:

\[ Q_{b1} = \frac{M_b}{c_1} = \frac{85.2}{1.35} = 63.11 \text{ kN} > Q_{b,\text{min}} = \varphi_{b3}(l + \varphi_f)R_{bw}bh_0 = \]

\[ = 0.6(l + 0.211)0.67 \cdot 80 \cdot 810 = 31.55 \text{ kN} \]

Then \( \alpha_1 = \frac{Q_1 - Q_{b1}}{Q_{b1}} = \frac{105.2 - 63.11}{63.11} = 0.667 \)

As \( c_1 = 1.35 \text{ m} < 2h_0 = 2 \cdot 0.81 = 1.62 \text{ m} \) so we take \( c_0 = c_1 = 1.35 \text{ m}; \)

\[ \alpha_{01} = \frac{Q_{b,\text{min}}c_0}{Q_{b1}2h_0} = \frac{31.55 \cdot 1.35}{63.11 \cdot 2 \cdot 0.81} = 0.417 \]

As \( \alpha_{01} = 0.417 < \alpha_1 = 0.667 < c_1/c_0 = 1 \) 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 – \( c_2 = 2.85 \text{ m} \).

\[ Q_{b2} = \frac{M_b}{c_2} = \frac{85.2}{2.85} = 29.9 \text{ kN} < Q_{b,\text{min}} = 31.55 \text{ kN} \]
We take \( Q_{b2} = Q_{b,\min} = 31.55 \text{ kN} \). Corresponding shear force is equal to \( Q_2 = 58.1 \text{ kN} \). As \( c_2 = 2.85 \text{ m} > h_0 = 1.62 \text{ m} \) we take \( c_0 = 2h_0 = 1.62 \text{ m} \).

\[
\varepsilon_2 = \frac{Q_2 - Q_{b2}}{Q_{b2}} = \frac{58.1 - 31.55}{31.55} = 0.842 < \varepsilon_02 = \frac{Q_{b,\min}}{Q_{b2}} \frac{c_0}{2h_0} = 1
\]

Therefore value \( q_{sw(2)} \) is to be determined by formula (58):

\[
q_{sw(2)} = \frac{Q_2}{c_0} \frac{\varepsilon_02}{\varepsilon_02 + 1} = \frac{58.1}{1.62} = 17.93 \text{ kN/m}
\]

So value \( q_{sw(2)} \) is determined by formula (58):

\[
q_{sw(2)} = \frac{Q_2}{c_0} \frac{\varepsilon_02}{\varepsilon_02 + 1} = \frac{58.1}{1.62} = 17.93 \text{ kN/m}
\]

We take maximum value \( q_{sw} = q_{sw(1)} = 31.18 \text{ kN/m} \).

According to welding conditions (see Item 5.13) we take diameter of stirrups 6 mm (\( A_{sw} = 28.3 \text{ mm}^2 \)) 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 \( s_1 = 150 \text{ mm} \). Spacing of stirrups in the span we take equal to \( s_2 = 2s_1 = 2 \cdot 150 = 300 \text{ mm} \).

The length of the part with spacing \( s \) is determined according to the condition of strength in compliance with Item 3.34; at the same time

\[
q_{sw1} = \frac{R_{sw} A_{sw}}{s_1} = \frac{175 \cdot 28.3}{150} = 33 \text{ N/mm};
q_{sw2} = 0.5q_{sw1} = 16.5 \text{ N/mm};
q_{sw1} - q_{sw2} = q_{sw2} = 16.5 \text{ N/mm}
\]

Let’s take the length of the part with stirrups spacing \( s_1 \) equal to the distance from the support to the first weight \( l_1 = 1.35 \text{ m} \); let’s check condition (50) by value \( c \) equal to the distance from the support to the second weight \( c = 2.85 \text{ m} > l_1 \). Value \( c_{01} \) is to be determined by formula (56) by \( q_{sw1} = 33 \text{ kN/m} \):

\[
c_{01} = \sqrt{\frac{M_p}{q_{sw1}}} = \sqrt{\frac{85.1}{33}} = 1.6 \text{ m} < 2h_0 = 1.62 \text{ m}
\]

As \( c - l_1 = 2.85 - 1.35 = 1.5 \text{ m} \) \( c_{01} = 1.6 \text{ m} \) so value \( Q_{sw} \) in condition (50) is to be taken equal to:

\[
Q_{sw} = q_{sw1} c_{01} - (q_{sw1} - q_{sw2})(c - l_1) = 33 \cdot 1.6 - 16.5 \cdot 1.5 = 28.05 \text{ kN};
Q_p = Q_{b,\min} = 31.55 \text{ kN};
Q_p + Q_{sw} = 31.55 + 28.05 = 59.6 \text{ kN} > Q_2 = 58.1 \text{kN},
\]

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 \( s_1 = 150 \text{ mm} \) is to be taken equal to \( l_1 = 1.35 \text{ m} \).

**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 \( v = 96 \text{ kN/m} \), dead load is \( g = 45 \text{ kN/m} \); shear force at the support \( Q_{max} = 380 \text{ kN/m} \); heavy-weight concrete B15 (\( R_{bt} = 0.67 \text{ MPa} \) by \( \gamma_{bt} = 0.9 \)); two-leg stirrups with diameter 6 mm (\( A_{sw} = 57 \text{ mm}^2 \)) of reinforcement A-I (\( R_{sw} = 175 \text{ MPa} \)), spacing...
s = 150 mm; bend-up bars A-II \( (R_{sw} = 225\text{MPa}) \), with the section area: of the first plane – \( A_{s,inc1} = 628\text{mm}^2 \) \( (2\times20) \), of the second plane – \( A_{s,inc2} = 402\text{mm}^2 \) \( (2\times16) \).

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 \( q_{sw} \):

\[
\varphi_{b2} = 2 \quad (\text{See Table 21});
\]

\[
b'_{j} - b = 3h_{j} = 3 \cdot 100 = 300\text{ mm};
\]

\[
\varphi_{j} = 0.75 \frac{(b'_{j} - b)h_{j}}{bh_{0}} = 0.75 \frac{300 \cdot 100}{300 \cdot 560} = 0.134 < 0.5;
\]

\[
M_b = 2(1 + \varphi_{j})R_{sw}bh_{0}^2 = 2(1 + 0.134)0.67 \cdot 300 \cdot 560^2 = 143 \cdot 10^6 \text{ kN-m};
\]

\[
q_{sw} = \frac{R_{sw}A_{sw}}{s} = \frac{175 \cdot 57}{150} = 66.5 \text{ N/mm}
\]

According to tem 3.32 we find \( q_1 = g + v/2 = 45 + 96/2 = 93 \text{ kN/m} \).

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 \( c = 50 + 520 + 300 = 870\text{ mm} = 0.87\text{ m} \).

Shear force at the distance \( c = 0.87 \text{ m} \) from the support is:

\[
Q = Q_{max} - q_1c = 380 - 93 \cdot 0.87 = 299.1 \text{ kN}
\]

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):

\[
c_{0,max} = \sqrt{M_b / q_{sw}} = \sqrt{143/66.5} = 1.466 > 2h_0 = 2 \cdot 0.56 = 1.12\text{ m};
\]

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 \( c_0 = c = 0.87 \text{ for this inclined section}. \) 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:

\[
Q_{s,inc1} = A_{s,inc1}R_{sw} \sin \theta = 628 \cdot 225 \cdot 0.707 = 99.9 \cdot 10^3 \text{ N} = 99.9 \text{ kN}.
\]

So \( \frac{M}{c} + q_m c_0 + Q_{s,inc1} = \frac{143}{0.87} + 66.5 \cdot 0.87 + 99.9 = 322.1 \text{ kN} > Q = 299.1 \text{ kN} \)

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 \( c = 1.69 \text{ m} \) 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 \[ \frac{M_b}{c} + q_w c_0 + Q_{s\text{inc}} = \frac{143}{1.69} + 66.5 \cdot 1.12 + 63.9 = 223 \text{ kN} > Q = 222.8 \text{ kN}, \]
that is strength of the present section is provided.

Let’s check inclined section which ends at the distance \( c_{0,\text{max}} = 1.12 \text{ m} \) from the beginning of the second plane of bend-up bars that is by \( c = 0.05 + 0.52 + 0.30 + 0.52 + 1.12 = 2.51 \text{ m} \).

Shear force at the distance \( c = 2.51 \text{ m} \) from the support is equal to \( Q = 380 - 93 \cdot 2.51 = 146.6 \text{ kN} \).

For this section it is obvious that \( c_0 = c_{0,\text{max}} = 1.12 \text{ m} \) and the inclined crack doesn’t cross bend-up bars, that is \( Q_{s\text{inc}} = 0 \). As \( c = 2.51 \text{ m} > \frac{\varphi_{b2}}{\varphi_{b3}} h_0 = \frac{2}{0.56} = 1.87 \text{ m} \) so we take
\[ Q_b = Q_{b,\text{min}} = \frac{M_b}{\frac{\varphi_{b2}}{\varphi_{b3}} h_0} = \frac{143}{1.87} = 76.5 \text{ kN}. \]

So \( Q_b + q_w c_0 + Q_{s\text{inc}} = 76.5 + 66.5 \cdot 1.12 + 0 = 151 \text{ kN} > Q = 146.6 \text{ kN} \) 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 \( Q = 299.1 \text{ kN} \) and \( \varphi_{b3} = 1.5 \):
\[ \frac{\varphi_{b3} R_b b h_0^2}{Q} = \frac{1.5 \cdot 0.67 \cdot 300 \cdot 560^2}{299.1 \cdot 10^3} = 316.1 \text{ mm} < 300 \text{ mm}, \]
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 \( q = 46 \text{ kN/m} \) (Draft 28a); dimensions of the cross section – according to Draft 28b; heavy-weight concrete B25 (\( R_r = 0.95 \text{ MPa} \) by \( \varphi_{b2} = 0.9 \)); stirrups made of reinforcement A-I (\( R_{sw} = 175 \text{ MPa} \)) diameter 8 mm (\( A_{sw} = 50.3 \text{ mm}^2 \)), spacing \( s = 150 \text{ mm} \).

*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 \( h_{01} = 600 - 80 = 520 \text{ mm} \) (Draft 28b).

Let’s determine values \( \varphi_{f1} \) and \( M_{b1} \) by formulas (53) and (52) as for a support section:
\[ h_f' = 150 + 100/2 = 200 \text{ mm} ; \]
\[ b_f' - b = 300 - 100 = 200 \text{ mm} < 3h_f' ; \]
\[ \varphi_{f1} = 0.75 \cdot \frac{b_f' - b}{b h_{01}} = 0.75 \cdot \frac{200 \cdot 200}{100 \cdot 520} = 0.577 > 0.5 , \]

We take \( \varphi_{f1} = 0.5 ; \varphi_{b2} = 2 \) (see Table 21);
\[ M_{b1} = \varphi_{b2} (1 + \varphi_{b1}) R_{sw} b h_0^2 = 2(1 + 0.5)0.95 \cdot 100 \cdot 520^2 = 77.06 \cdot 10^6 \text{ N-mm} = 77.06 \text{ kN-mm}. \]
Let’s determine value $q_{\text{inc}}$ taking $\tan \beta = 1/12$:

$$q_{\text{inc}} = \varphi_{R_c} R_{\gamma} b tg \beta = 2 \cdot 0.95 \cdot 100 / 12^2 = 1.32 \text{ N/mm (kN/m)}$$

As there is continuous load so we take $q_1 = q = 46 \text{ kN/m}$.

Let’s check condition (66):

$$0.56 q_{sw} - 2.5 \sqrt{q_{sw} q_{\text{inc}}} = 0.56 \cdot 58.7 - 2.5 \sqrt{58.7 \cdot 1.32} = 10.9 \text{ kN/m} < q_1 = 46 \text{ kN/m}$$

Condition (66) is not met so value $c$ is to be determined by formula (68):

$$c = \sqrt{\frac{M_{bl}}{q_{\text{inc}} + q_{sw} + q_1}} = \sqrt{\frac{77.06}{1.32 + 58.7 + 46}} = 0.853 \text{ m}$$

At the same time $c_0 = c = 0.853 \text{ m}$

Working height of the cross section $h_0$ at the distance $c = 0.853 \text{ m}$ from the support is:

$$h_0 = h_{01} + ctg \beta = 0.52 + 0.853 / 12 = 0.591 \text{ m}$$

Let’s determine value $M_b$ by $h_0 = 591 \text{ mm}$:

$$\varphi_f = 0.75 \cdot \left( \frac{b_f - b}{bh_0} \right) = 0.75 \cdot \frac{200 - 200}{100 \cdot 591} = 0.508 > 0.5$$

We take $\varphi_f = 0.5$;

$$M_b = \varphi_{\gamma_2} (1 + \varphi_f) R_{\gamma} bh_0^2 = 2 (1 + 0.5) 0.95 \cdot 100 \cdot 591^2 = 99.55 \cdot 10^6 \text{ N-mm} = 99.55 \text{ kN-m}$$

Let’s check condition (50) taking shear force at the end of inclined section equal to:

$$Q = Q_{\max} - q_1 c = \frac{ql}{2} - q_1 c = \frac{46 \cdot 8.8}{2} - 46 \cdot 0.853 = 163.2 \text{ kN};$$

$$Q_{bw} + Q_{sw} = \frac{M_b}{c} + q_{sw} c_0 = \frac{99.55}{0.853} + 58.7 \cdot 0.853 = 166.8 \text{ kN} > Q = 163.2 \text{ kN},$$

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 $F = 300 \text{ kN}$ located at the distance 0.8 m from the support; heavy-weight concrete B15 ($R_{\gamma} = 0.67 \text{ MPa}$ by $\gamma_{\gamma_2} = 0.9$); two-leg stirrups with diameter 8 mm ($A_{sw} = 101 \text{ mm}^2$) of reinforcement A-I ($R_{\gamma} = 175 \text{ MPa}$), spacing $s = 200 \text{ mm}$.  

*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

$$h_{01} = 650 - (650 - 300) \frac{800}{950} = 50 = 305 \text{ mm}$$

(see Draft 29).

By formula (52) let’s determine value $M_{b1}$ taking $\varphi_{\gamma_2} = 2$ (see Table 21) and $\varphi_f = 0$:

$$M_{b1} = \varphi_{\gamma_2} (1 + \varphi_f) R_{\gamma} bh_{01}^2 = 2 \cdot 0.67 \cdot 400 \cdot 305^2 = 49.9 \cdot 10^6 \text{ N-mm}$$
Value $q_{sw}$ is:
\[ q_{sw} = \frac{R_{sw} A_{sw}}{s} = \frac{175 \cdot 101}{200} = 88.4 \text{ N/mm (kN/m)}. \]

Taking $\tan \beta = \frac{650 - 300}{950} = 0.369$ (see Draft 29) we determine $q_{inc}$:
\[ q_{inc} = \varphi_{b2} R_{sw} \beta \tan \beta = 2 \cdot 0.67 \cdot 400 \cdot 0.369^2 = 73 \text{ N/mm}, \]

therefore taking $q_1 = 0$ we have
\[ c = \sqrt{\frac{M_{bl}}{q_{inc} + q_{sw}}} = \sqrt{\frac{49.9 \cdot 10^6}{73 + 88.4}} = 556 \text{ kN} \]

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:
\[ h_0 = h_{01} + \tan \beta = 305 + 556 \cdot 0.369 = 510 \text{ mm} \]

As $2h_0 = 2 \cdot 510 \text{ mm} > c = 556 \text{ mm}$ so we take $c_0 = 556$ mm.

Value $M_b$ is
\[ M_b = \varphi_{b2} R_{sw} h_0^2 = 2 \cdot 0.67 \cdot 400 \cdot 510^2 = 139.4 \cdot 10^6 \text{ N-mm} = 139.4 \text{ kN-m} \]

Therefore
\[ Q_b + Q_{sw} = \frac{M_b}{c} + q_{sw} c_0 = \frac{139.4}{0.556} + 88.4 \cdot 0.556 = 299.9 \text{ kN} \approx Q = 300 \text{ kN}, \]

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 $c_0 = 1200 \text{ mm} > c = 800 \text{ mm}$ 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 $v = 50 \text{ kN/m}^2$; load of dead weight and floor $g = 9 \text{ kN/m}^2$; $a = 20$ mm; heavy-weight concrete B25 ($R_{bc} = 0.95 \text{ MPa by } \gamma_{b2} = 0.9$).

It is required to examine the slab strength as regards shear force.

Calculation: $h_0 = h - a = 160 - 20 = 140 \text{ mm}$. The calculation is made for the strip $b = 1 \text{ m} = 1000 \text{ mm}$ with a span $l = 3 \text{ m}$; total load on the slab is $q = v + g = 50 + 9 = 59 \text{ kN/m}$.

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):
\[ 2.5 R_{bc} h_0 = 2.5 \cdot 0.95 \cdot 1000 \cdot 140 = 333 \cdot 10^3 \text{ N} > Q_{max} = 88.5 \text{ kN} \]

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 $b = 6 \text{ m}$ – 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:
\[ q_i = g + \nu/2 = 9 + 50/2 = 34 \text{kN/m} = 34 \text{N/mm}; \]
\[ \varphi_{b4} = 1.5 \text{ (See Table 21)} \]

As \[ \frac{\varphi_{b4} R_{ts} b}{(c_{\text{max}}/h_{0i})^2} = \frac{1.5 \cdot 0.95 \cdot 1000}{2^2} = 356 \text{N/mm} > q_i = 34 \text{ N/mm} \] so we take \[ c = c_{\text{max}} = 280 \text{ mm} = 0.28 \text{ m}. \]

Shear force at the end of inclined section is \[ Q = Q_{\text{max}} - q_i c = 88.5 - 34 \cdot 0.28 = 79 \text{kN}. \]

\[ \frac{\varphi_{b4} R_{ts} b h_{0i}^2}{c} = \frac{1.5 \cdot 0.95 \cdot 1000 \cdot 140^2}{280} = 99.75 \cdot 10^3 \text{N} = 99.75 \text{kN} > Q = 79 \text{kN}, \]
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 mm (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 \( q = 7 \text{kN/m}^2 \) at the free supported end; \( a = 22 \text{ mm} \); heavy-weight concrete B15 (\( R_{b4} = 0.82 \text{ MPa by } \gamma_{b4} = 1.1 \)).

*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 \( h_{01} = 262 - 22 = 240 \text{ mm}. \)

Let’s determine \( \tan \beta \) (\( \beta \) is the angle between the tensile and the compressed surfaces):

\[ \tan \beta = \frac{262 - 120}{4250} = 0.0334. \]

The calculation is made for the strip of the panel \( b = 1 \text{ m} = 1000 \text{ mm} \) wide.

Let’s check conditions of Item 3.40. Lateral force at the point of embedment is:

\[ Q_{\text{max}} = \frac{69 + 7}{2} \cdot 4.25 = 161.5 \text{kN} \]

Let’s check condition (71) taking \( h_0 = h_{01} = 240 \text{ mm} \):

\[ 2.5 R_{ts} b h_0 = 2.5 \cdot 0.82 \cdot 1000 \cdot 240 = 492 \text{kN} > Q_{\text{max}} = 117 \text{kN}, \]
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 \( c_{\text{max}} \) considering the coefficient \( \alpha = 1.25 : \)

\[ c_{\text{max}} = \frac{2.5 h_{01}}{\alpha + 1.25 \tan \beta} = \frac{2.5 \cdot 240}{1.25 + 1.25 \cdot 0.0334} = 464 \text{ mm} \]

Average load density at the support part \( c_{\text{max}} = 464 \text{ mm} \) long is \( q_i = 69 -(69-7) \cdot \frac{464}{4250 \cdot 2} = 65.6 \text{ N/mm}. \) From table 21 \( \varphi_{b4} = 1.5 \)

As \[ c = h_{01} \sqrt{\frac{1}{\tan \beta/4 + q_i/(\varphi_{b4} R_{ts} b)}} = 240 \sqrt{\frac{1}{0.0334^2/4 + 65.4/(1.5 \cdot 0.82 \cdot 1000)}} = 1037 \text{ mm} \]

\[ > c_{\text{max}} = 464 \text{ mm} \] so we take \( c = c_{\text{max}} = 464 \text{ mm}. \)

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 \( c \)):

\[ h_0 = h_{01} - \frac{c}{2} \tan \beta = 240 - \frac{464}{2} \cdot 0.0334 = 232 \text{ mm} \]

Shear force at the distance \( c = 464 \text{ mm} \) from the support is:
Let’s check condition (72):

$$Q = Q_{\text{max}} - q_c = 161.5 - 65.6 \cdot 0.464 = 131.1 \text{kN}$$

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 (\( R_s = 7.7 \) MPa; \( R_{bh} = 0.67 \) MPa by \( \gamma_{bh} = 0.9 \)); longitudinal reinforcement A-III (\( R_s = 365 \) MPa) without anchors, section area is \( A_1 = 982 \text{mm}^2 \) (2Ø25) and \( A_2 = 226 \text{mm}^2 \) (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: \( 360 \cdot 40 \cdot 400 \cdot 0 = - = - = \text{ahh mm}. \) 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 \( l_x = l_{\text{sup}} - 10 \text{mm} = 280 - 10 = 270 \text{mm} \) (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 \( R_s A_x = 124.1 \cdot 982 = 121.9 \cdot 10^3 \) N by value \( N_w \).

Taking \( d_w = 6 \text{mm}, n_w = 6 \cdot \varphi_w = 200 \) (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 \( R_s A_x = 121.9 + 20.26 = 142.2 \text{kN} \)

As this value does not exceed value \( R_s A_s \) determined without considering \( \gamma_{s5} \) and \( N_w \) that is it’s equal to \( 365 \cdot 982 = 358 \cdot 10^3 \) N so we take \( R_s A_x = 142.2 \text{kN} \).

The height of compressed zone is determined by formula (16):

$$x = \frac{R_s A_x - R_{sw} A_{sw}}{R_s b} = \frac{142.2 \cdot 10^3 - 365 \cdot 226}{7.7 \cdot 200} = 39 \text{mm} < 2a' = 2 \cdot 35 \text{mm}$$

In compliance with Item 3.42 we take \( z_x = h_0 - a' = 360 - 35 = 325 \text{mm} \).

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 \( Q = \frac{q l}{2} = \frac{29 \cdot 5.5}{2} = 80 \text{ kN} \) as well as \( F_j = 0 \) and \( A_{s,inc} = 0 \):

\[
Q = \frac{80 \cdot 10^3}{q_{sw} + q} = 821 \text{ mm}
\]

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 \( c = c_1 \leq 0.8c_{\text{max}} = 2h_0 \); that is according to the following equation:

\[
Q = Q_{\text{max}} = q l_s = 0.8\phi_{ba} R_{st} b h^2_0 / c_1 .
\]

Supposing that \( l_s > 2h_0 \) we take maximum value \( c_1 = 2h_0 \).

Then by \( \phi_{ba} = 1.5 \) we get:

\[
l_s = \frac{0.8\phi_{ba} R_{st} b h^2_0}{2h_0} = \frac{80 \cdot 10^3}{q} = \frac{0.8 \cdot 1.5 \cdot 0.67 \cdot 200 \cdot 360^2}{29} = 1760 \text{ mm} > 2h_0 = 2 \cdot 360 = 720 \text{ mm}
\]

As \( l_s = 1760 \text{ mm} > c = 821 \text{ mm} \) so we take \( c = 821 \text{ mm} \).

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 \( l_1 + c = l_{\text{sup}} / 3 + c = 280 / 3 + 821 = 914 \text{ mm} \) from the point of application of support reaction:

\[
M = Q(l_1 + c) - \frac{q(l_1 + c)^2}{2} = 80 \cdot 0.914 - \frac{29 \cdot 0.914^2}{2} = 61 \text{ kN\cdot m}
\]

Let’s check the strength according to condition (77) considering formula (78):

\[
R_s A_{s,z} + 0.5q_{sw} c^2 = 142.2 \cdot 10^3 \cdot 325 + 0.5 \cdot 68.4 \cdot 821^2 = 46.6 + 23.05 = 69.4 \text{ kN\cdot m} > M = 61 \text{ kN\cdot m},
\]

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 \( M_0 = Ql_1 = 80 \times 10^3 \times 93 = 7.4 \cdot 10^6 \text{ N\cdot mm} \):

\[
\sqrt{2(R_s A_{s,z} - M_0)q_{sw} + q} = \sqrt{2(46.3 \cdot 10^6 - 7.4 \cdot 10^6)(68.4 + 29)} = 87.1 \cdot 10^3 \text{ N} = 87 \text{ kN} > Q = 80 \text{ kN}.
\]

**Example 22.** Given: a collar beam of a multi-storey frame with diagrams of bending moments and shear forces of distributed load \( q = 228 \text{ kN/m} \) according to Draft 31; heavy-weight concrete B25; cross reinforcement and longitudinal reinforcement A-III (\( R_s = 365 \text{ MPa} \); \( R_{sw} = 290 \text{ MPa} \)); cross section of the support part – according to Draft 31; stirrups with diameter 10 mm, spacing \( s = 150 \text{ mm} \) (\( A_{sw} = 236 \text{ mm}^2 \)).

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 \( x < 0 \):

\[
M_u = R_s A_s (h_0 - a') = 365 \cdot 1609 (740 - 50) = 405 \text{ kN} \cdot \text{m}
\]

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:

\[
M = M_{sup} - \frac{M_{sup} - M_{sup}'}{l} x - \frac{q l}{2} x + \frac{q}{2} x^2 = M_u,
\]

So \( x \) is:

\[
x = \left( \frac{l}{2} + \frac{M_{sup} - M_{sup}'}{ql} \right) - \left[ \frac{1}{2} + \frac{M_{sup} - M_{sup}'}{ql} \right]^2 - \frac{2(M_{sup} - M_u)}{q} = \left( \frac{4.9}{2} + \frac{600 - 300}{228 \cdot 4.9} \right) -
\]

\[
= \frac{2(600 - 405)}{228} = 0.334 \text{ m}.
\]

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 \( q_{sw} \):

\[
q_{sw} = \frac{R_s A_{sw}}{s} = \frac{290 \cdot 236}{150} = 456 \text{ N/mm}
\]

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 \( x + w = 334 + 756 = 1090 \text{ mm} \).

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 (\( A_{sw} = 452 \text{ mm}^2 \); \( A_{s,inc} = 804 \text{ mm}^2 \)); section area of additional stirrups of cuttings \( A_{sw1} = 402 \text{ mm}^2 \) (2016); longitudinal reinforcement A-III according to Draft 32b; shear force at the support \( Q = 640 \text{ kN} \).

*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 \( h_0 = 370 \text{ mm}, b = 730 \text{ mm} \) (see Draft 32), \( \varphi_{b2} = 2 \) (see Table 21):

\[
M_b = \varphi_{b2} R_{bt} b h_0^2 = 2 \cdot 0.95 \cdot 730 \cdot 370^2 = 190 \cdot 10^6 \text{ N} \cdot \text{mm}
\]

By value \( c \) equal to the distance from the support to the first load – \( c = 1.5 \text{ m} \) we have
\[ Q_b = \frac{M_b}{c} = \frac{190 \cdot 10^6}{1500} = 126.7 \cdot 10^3 \text{ N} < Q_{b,\min} = \]
\[ = \varphi_{b,3} R_{n} b h_0 = 0.6 \cdot 0.95 \cdot 730 \cdot 370 = 154 \cdot 10^3 \text{ N} \]
\[ (\varphi_{b,3} = 0.6 \text{ – see Table 21}), \]

So we take \( Q_b = 154 \cdot 10^3 \text{ N}; \)
\[ q_{sw} = \frac{R_{sw} A_{sw}}{s} = \frac{255 \cdot 452}{100} = 1152 \text{ T/mm} \]
\[ c_0 = \sqrt{\frac{M_b}{q_{sw}}} = \sqrt{\frac{190 \cdot 10^6}{1152}} = 406 \text{ mm} < h_0 \]

At the same time \( c_0 < c = 1.5 \text{ m} \) and \( c_0 > h_0 \). Thereafter \( Q_b + q_{sw} c_0 + R_{sw} A_{sw1} = 154 \cdot 10^3 + 1152 \cdot 406 + 290 \cdot 402 = 738 \cdot 10^3 \text{ N} > Q = 640 \text{ kN}, \)
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 \( \theta = 45^\circ \); \( h_0 = 700 - 60 - 80/2 = 600 \text{ mm} \); \( h_{01} = 370 \text{ mm}; \)
\[ R_{sw} A_{sw1} + R_{sw} A_{s,inc} \sin 45^\circ = 290 \cdot 402 + 290 \cdot 804 \cdot 0.707 = 281 \cdot 10^3 \text{ N} \]
\[ \times \left(1 - \frac{370}{600}\right) = 245 \text{ kN} \]

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 \( R_s = 365 \text{ MPa}. \)

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 \( a_i = 30 \text{ mm} \) we get \( z_{s,inc} = z_s \cos \theta + (c - a_i) \sin \theta = 320 \cdot 0.707 + (312 - 30) \cdot 0.707 = 425 \text{ mm} \)

Let’s check condition (77) taking:
\[ \Sigma R_{sw} A_{sw} z_{sw} = 0.5 q_{sw} c e^2 + R_{sw} A_{sw1} (c - a) = 0.5 \cdot 1152 \cdot 312^2 + 290 \cdot 402(312 - 30) = 88.8 \cdot 10^6 \text{ N-mm}; \]
\[ M = Q(a_0 + c) = 640 \cdot 10^3 (130 + 312) = 283 \cdot 10^6 \text{ N-mm}; \]
\[ R_s A_s z_{s} + \Sigma R_{sw} A_{sw} z_{sw} + \Sigma R_{sw} A_{s,inc} z_{s,inc} = 365 \cdot 1256 \cdot 320 + 88.8 \cdot 10^6 + 290 \cdot 804 \cdot 425 = \\
334.6 \cdot 10^6 \text{ N-mm} > M = 283 \text{ kN-m}, \]
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:
\[ w_0 = \frac{2(Q - R_{sw} A_{sw1} - R_{sw} A_{s,inc} \sin \theta)}{q_{sw}} + a_0 + 10d = \]
Let’s check inclined section located beyond the cutting and beginning at the distance
\[ h_0 - h_{01} = 600 - 370 = 230 \text{ mm} \]
from the end of the beam. Then \( l_s = 230 - 10 = 220 \text{ mm} \)
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
\[ l_{an} = 29 \cdot 40 = 1160 \text{ mm} > l_s = 220 \text{ mm} \]
Design strength of bottom reinforcement is to be decreased by means of multiplying by the
coefficient \( \gamma_s = \frac{l_s}{l_{an}} = \frac{220}{1160} = 0.19 \); that is \( R_s = 365 \cdot 0.19 = 69.2 \text{ MPa} \).
According to Draft 32 \( A_s = 5027 \text{ mm}^2 \) (4Ø40)
Taking the fact into account that within the length \( l_s = 220 \text{ mm} \) two top rods have two welded
vertical rods and two bottom rods have two vertical and one horizontal welded rod, we increase
the force \( R_sA_s \) by value \( N_w \) determined by formula (82) taking \( n_w = 10, d_w = 12 \text{ mm} \),
\( \varphi_w = 100 \) (see Table 22):
\[
N_w = 0.7n_w\varphi_w d_w^2 R_{bt} = 0.7 \cdot 10 \cdot 12^2 \cdot 0.95 = 95760 \text{ N} < 0.8R_s d_w^2 n_w = 0.8 \cdot 365 \cdot 12^2 \cdot 10 = 420000 \text{ N}
\]
Thereafter \( R_s A_s = 69.2 \cdot 5027 + 95760 = 443600 \text{ N} < 365 \cdot 5027 = 1835 \cdot 10^3 \text{ N} \)
Taking \( b = b' = 730 \text{ mm} \) we determine the height of the compressed zone \( x \):
\[
x = \frac{R_s A_s - R_w A_w + R_{sw} A_{sw,inc} \cos \theta}{R_s b} = \frac{443600 - 365 \cdot 1256 + 290 \cdot 804 \cdot 0.707}{13 \cdot 730} = 15.8 \text{ mm} < 2a' = 2 \cdot 50 = 100 \text{ mm}
\]
And so \( z_s = h_0 - a' = 600 - 50 = 550 \text{ mm} \)
The most disadvantageous value \( c \) is equal to:
\[
c = \frac{Q}{q_{sw}} = \frac{640000}{1152} = 555 \text{ mm} < w_0 - (h_0 - h_{01}) = 953 - 230 = 723 \text{ mm}
\]
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 \( w_0 = 953 \text{ mm} \) from the cutting; at the same time \( c = 723 \text{ mm} \). Design
moment \( M \) in the section going through the end of inclined section is:
\[
M = Q(a_0 + w_0) = 460(0.13 + 0.953) = 693 \text{kN·m};
\]
\[
z_{s,inc} = z_s \cos \theta + (c - a) \sin \theta = 550 \cdot 0.707 + (723 - 70) \cdot 0.707 = 851 \text{ mm}
\]
[Where \( a_1 = 300 - 230 = 70 \text{ mm} \) (see Draft 32)]
Let’s check condition (77):
\[
R_s A_s z_s + \frac{q_{sw} c^2}{2} + R_{sw} A_{sw,inc} z_{s,inc} = 443600 \cdot 550 + \frac{1152 \cdot 723^2}{2} + 290 \cdot 804 \cdot 851 = 743.5 \cdot 10^6 \text{ N·mm} > M = 693 \text{kN·m},
\]
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 \( l_{sup} = 130 \text{ mm}, a_s = 90 \text{ mm}, h_{01} - a' = 370 - 50 = 320 \text{ mm} \)
(see Draft 32) thereafter
Taking $\mu_w = 0$ and replacing 0.8 by 1.0 we have $R_p b l_{sup} \sin^2 \theta = 13 \cdot 730 \cdot 130 \cdot 0.679 = 838 \cdot 10^3 \text{N} < 3.5 R_y b h_{01} = 3.5 \cdot 0.95 \cdot 730 \cdot 370 = 898 \cdot 10^3 \text{N}$; that is the right part of condition (207) is equal to 838 kN.

As $Q = 640 \text{kN} < 838 \text{kN}$ so the strength of the compressed zone is provided.

Let’s check condition (208) taking $l_i = l_{sup} + a_x = 220 \text{mm}$, $h_0 = 320 \text{mm}$, $A_s = 1256 \text{mm}^2$ (4Ø20):

$$Q \frac{l}{h_0} = 640 \cdot 10^3 \cdot \frac{220}{320} = 440 \cdot 10^3 \text{N} < R_y A_s = 365 \cdot 1256 = 458 \cdot 10^3 \text{N},$$

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 framework 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 $A'_i > 0.02A_b$ is met, so it is necessary to consider decrease of actual concrete area by value $A'_i$ 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 $M_n$ is determined by the following formula:

$$M_n = \varphi_{b2} \left( l + \varphi_f + \varphi_n \right) R_{bs} b h_0^2$$

Where $\varphi_n = 0.1 \frac{N}{R_{bs} b h_0}$ but no more than 0.5; value $Q_{b,\text{min}}$ is taken equal to $\varphi_{b3} \left( l + \varphi_f + \varphi_n \right) R_{bs} b h_0$ and in formulas (72)–(76) coefficient $\varphi_{b4}$ 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 $R_{bs,\text{ser}}$ by $R_{bs}$].

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 $\eta$ determined by the following formula:

$$\eta = \frac{1}{1 - \frac{N}{N_{cr}}}$$ (91)

Where $N_{cr}$ – relative critical force determined by the following formulas:

for members of any section:

$$N_{cr} = \frac{6.4E_b}{l_0^2} \left[ I \left( \frac{0.11}{0.1 + \delta_e} + 0.1 \right) + \alpha \delta \right]$$ (92)

for members of rectangular section:

$$N_{cr} = \frac{1.6E_b bh}{(l_0 / h)} \left[ \frac{0.11}{0.1 + \delta_e} + 0.1 \right] + \mu \alpha \left( \frac{h_0 - a}{h} \right)^2$$ (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;

$\varphi_l$ – Coefficient considering long-term action of the load on the member deflection in the limit state and equal to:

$$\varphi_l = 1 + \beta \frac{M_{ll}}{M_l}$$ (94)

But no more than $1 + \beta$ (here $\beta$ – see Table 16);

$M_l, M_{ll}$ – 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 $M_l$ and $M_{ll}$ 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 signs 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 $\varphi_l$ is to be taken equal to $\varphi_l = \varphi_{ll} + 10(1 - \varphi_{ll}) \times e_0 / h$ where $\varphi_{ll}$ 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;

$\delta_e$ – Coefficient taken equal to $e_0 / h$ but no less than

$$\delta_{e, min} = 0.5 - 0.01 \frac{l_0}{h} - 0.01R_p$$ (95)

(Here $R_p$ is given in mega-Pascal, it is possible to take $\gamma_{h_2} = 1.0$; for round and ring sections value $h$ is replaced by $D$);

$l_0$ – is taken according to Item 3.55;
\[ \mu \alpha = \frac{A_s + A_e}{bh} \frac{E_s}{E_b} \] 

(96)

During calculation of rectangular sections with reinforcement located along the height of the section according to Item 3.63 in value \( A_s + A_e \) it is not considered 2/3 of reinforcement located at the surfaces parallel to the bending plane (2\( A_o \)), and value \( \frac{h_o - a'}{h} \) in formula (93) is taken equal to \( 1 - 2\delta \).

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 \( l_o / i < 14 \) (for rectangular sections \( l_o / h < 4 \)) it is taken \( \eta = 1 \).

By of the element \( 14 \leq l_o / i < 35 \) (\( 4 \leq l_o / h < 10 \)) and by \( \mu = \frac{A_s + A_e}{A} \leq 0.025 \) it is possible to take:

For rectangular sections:

\[ N_{cr} = 0.15 \frac{E_b A}{(l_o / h)^2} \]

For other sections:

\[ N_{cr} = \frac{2E_b I}{l_o^2} \]

By \( N > N_{cr} \) it is necessary to increase the section dimensions.

By design eccentricities in two directions coefficient \( \eta \) can be determined for each direction and multiplied by the corresponding eccentricity.

3.55.(3.25) It is recommended to determine design length \( l_o \) 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 \( l_o \) 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……..\( l_o = H \)
- Cast-in-situ floor structures ……..\( l_o = 0.7H \)

[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>
<thead>
<tr>
<th>Buildings and columns characteristics</th>
<th>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</th>
<th>if there are</th>
<th>if there are bracings in the plane of longitudinal row of columns or anchor supports</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Buildings</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With crane bridges</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Considering the load of canes</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bottom part of columns by crane runaway beams</td>
<td>Semi-infinite simply supported 1.5 $H_1$ 0.8 $H_1$ 1.2 $H_1$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top part of columns by crane runaway beams</td>
<td>Semi-infinite beam simply supported 2.0 $H_2$ 1.5 $H_2$ 2.0 $H_2$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Without considering the load of cranes</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bottom part of building beams</td>
<td>One-span beam 1.5 $H$ 0.8 $H_1$ 1.2 $H$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Multi-span beam</td>
<td>1.2 $H$ 0.8 $H_1$ 1.2 $H$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top part of columns by crane runaway beams</td>
<td>Semi-infinite beam simply supported 2.5 $H_2$ 1.5 $H_2$ 2.0 $H_2$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Without crane bridges</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tapered columns</td>
<td>Bottom part of building beams</td>
<td></td>
<td></td>
</tr>
<tr>
<td>One-span beam</td>
<td>1.5 $H$ 0.8 $H$ 1.2 $H$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Multi-span beam</td>
<td>1.2 $H$ 0.8 $H$ 1.2 $H$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top part of columns</td>
<td>2.5 $H_2$ 2.0 $H_2$ 2.5 $H_2$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Columns of constant section of buildings</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>One-span beam</td>
<td>1.5 $H$ 0.8 $H$ 1.2 $H$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Multi-span beam</td>
<td>1.2 $H$ 0.8 $H$ 1.2 $H$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crane trestle</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bottom part of columns by crane runaway beams</td>
<td>Semi-infinite simply supported 2.0 $H_2$ 0.8 $H_1$ 1.5 $H_1$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Simply supported</td>
<td>1.5 $H_1$ 0.8 $H_1$ $H_1$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### 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)

<table>
<thead>
<tr>
<th>Elements</th>
<th>Design length $l_0$ of trusses and arches members</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Trusses elements:</td>
<td></td>
</tr>
<tr>
<td>a) top chord of truss during calculation:</td>
<td></td>
</tr>
<tr>
<td>- in the truss plane:</td>
<td></td>
</tr>
<tr>
<td>- by $e_0 &lt; 1/8h_1$</td>
<td>$0.9l$</td>
</tr>
<tr>
<td>- by $e_0 \geq 1/8h_1$</td>
<td>$0.8l$</td>
</tr>
<tr>
<td>- out of the truss plane for the part under the skylight (if the width of the skylight is 12 m and more):</td>
<td>$0.8l$</td>
</tr>
<tr>
<td>- in other cases</td>
<td>$0.9l$</td>
</tr>
<tr>
<td>b) inclined braces and poles during calculation:</td>
<td></td>
</tr>
<tr>
<td>- in the truss plane</td>
<td>$0.8l$</td>
</tr>
<tr>
<td>- out of the truss plane</td>
<td></td>
</tr>
<tr>
<td>- by $b_1 / b_2 &lt; 1.5$</td>
<td>$0.9l$</td>
</tr>
<tr>
<td>- by $b_1 / b_2 \geq 1.5$</td>
<td>$0.8l$</td>
</tr>
<tr>
<td>2. Arches</td>
<td></td>
</tr>
<tr>
<td>a) during calculation in the arch plane:</td>
<td></td>
</tr>
<tr>
<td>- triple-hinged</td>
<td>$0.580L$</td>
</tr>
<tr>
<td>- double-hinged</td>
<td>$0.540L$</td>
</tr>
<tr>
<td>- hingeless</td>
<td>$0.365L$</td>
</tr>
<tr>
<td>b) during calculation out of the arch plane (any)</td>
<td>$L$</td>
</tr>
</tbody>
</table>

**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:

$$M = M_v \eta_v + M_h \eta_h + M_f,$$

Where $M_v$ – moment caused by vertical loads on the floors;
\( \eta_v \) – 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);

\( \eta_h \) – is coefficient \( \eta \) 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 \( R_p \) by changed strength of concrete \( R_{b,rod} \) and calculating the compressed concrete zone characteristics \( \omega \) 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 \( R_{b,rod} \) is determined by the following formulas:

a) by reinforcement by welded cross meshes

\[
R_{b,rod} = R_b + \varphi \mu_{sy} R_{s,sy},
\]

(98)

Where \( R_{s,sy} \) – design strength of reinforcement meshes

\[
\mu_{sy} = \frac{n_x A_{sx} l_x + n_y A_{sy} l_y}{A_{ef} s}
\]

(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;
$n_y, A_{sy}, I_y$ – The same in another direction;

$A_{ef}$ is section area of reinforcement within the meshes contours;

$s$ is distance between meshes;

$\varphi$ is efficiency factor of confinement reinforcement determined by the following formula:

$$\varphi = \frac{1}{0.23 + \psi}$$

$$\psi = \frac{\mu_{xy} R_{s,xy}}{R_b + 10}$$

$R_{s,xy}, R_b$ are given in MPa

For members made of fine concrete efficiency factor $\varphi$ must be taken no more than 1.

b) By reinforcement of spiral or ring reinforcement

$$R_{b,red} = R_b + 2\mu_{cir} R_{s,cir} \left(1 - \frac{7.5e_0}{d_{ef}}\right)$$

Where $R_{s,cir}$ – design strength of the spiral;

$\mu_{cir}$ – reinforcement factor equal to:

$$\mu_{cir} = \frac{4A_{x,cir}}{d_{ef} s}$$

Here $A_{x,cir}$ – 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):

$$\omega = \alpha - 0.008R_b + \delta_2 \leq 0.9$$

Where $\alpha$ – coefficient taken according to Item 3.14 instructions;

$\delta_2$ – Coefficient equal to 10$\mu$ but taken no more than 0.15 [here $\mu$ is reinforcement coefficient $\mu_{sy}$ or $\mu_{cir}$ 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 \( \varphi_1 = 0.25 + 0.05l_0/c_{ef} \leq 1.0 \) and value \( \delta_{cr,min} \) is determined by formula \( \delta_{cr,min} = 0.5 + 0.01l_0/c_{ef} \left( 1.0 - 0.1l_0/c_{ef} \right) - 0.01R_b \) 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:

\[
R_{sc,red} = R_{sc} \frac{1 + \delta_3 \lambda_1}{1 + \delta_3 \lambda_2} \leq R_s
\]

(105)

Where \( \lambda_1, \lambda_2 \) – see Table 25;
\[
\delta_3 = 1.6\psi \theta ;
\]

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;

\( \psi, A_{ef} \) – see Item 3.57;

\( A_{s,tot} \) – Section area of all longitudinal high-strength reinforcement;

\( R_b \) – In MPa.

Table 25

<table>
<thead>
<tr>
<th>Reinforcement class</th>
<th>( \lambda_1 ), ( \lambda_2 ) and ( R_{sc} ) in MPa by coefficient ( \gamma_{b,2} ) (see Item 3.1) equal to</th>
<th>( R_s ) MPa</th>
<th>( R_{s,ser} ) MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.9</td>
<td>1.0 or 1.1</td>
<td></td>
</tr>
<tr>
<td>A-V</td>
<td>\begin{align*} \lambda_1 &amp; = 1.25, \lambda_2 &amp; = 0.53, R_{sc} &amp; = 500, \lambda_1 &amp; = 2.78, \lambda_2 &amp; = 1.03, R_{sc} &amp; = 400 \end{align*}</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>\begin{align*} R_s &amp; = 680, R_{s,ser} &amp; = 785 \end{align*}</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-VI</td>
<td>\begin{align*} \lambda_1 &amp; = 2.04, \lambda_2 &amp; = 0.77, R_{sc} &amp; = 500, \lambda_1 &amp; = 3.88, \lambda_2 &amp; = 1.25, R_{sc} &amp; = 400 \end{align*}</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>\begin{align*} R_s &amp; = 815, R_{s,ser} &amp; = 980 \end{align*}</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[
\lambda_1 = \left[ \left( \frac{R_s}{R_{sc}} \right)^2 - 1 \right] \frac{1000}{R_s} ; \lambda_2 = \left( \frac{R_s}{R_{sc}} - 1 \right) \frac{1000}{R_s} .
\]

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 \( \xi_h h_0 \) by replacement \( R_s \) by \( 0.8R_s \) 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.
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 \(\xi\) 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

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 \(x\):

\[
x = \frac{N}{R_b b} : 
\]

a) by \(x \leq \xi h_0\) (Draft 34) – according to the following condition

\[
N_e \leq R_b b x(h_0 - 0.5x) + R_{sc} A_s(h_0 - a') ;
\]

Draft 34. Forces scheme in rectangular cross-section of eccentric compressed element.

b) By \(x > \xi h_0\) – according to condition (108) taking the height of compressed zone equal to \(x = \xi h_0\) where \(\xi\) 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:

\[
\xi = \frac{-\alpha_n + \psi \alpha_s - \alpha_n}{2} + \sqrt{\left(\frac{\alpha_s + \psi \alpha_n + \alpha_n}{2}\right)^2 + \psi \alpha_s \omega} \tag{110}
\]

In formulas (109) and (110):

\[
\alpha_n = \frac{N}{R_b b h_0} ; \quad \alpha_s = \frac{R_A}{R_b b h_0} ; \quad \psi = \frac{\sigma_{sc,u}}{R_f \left(1 - \frac{\omega}{1.1}\right)} ;
\]

\(\xi_R, \psi, \omega\) – 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 $x = \frac{N + R_s A_e}{R_b b} < a'$, so design bearing capacity of the section can be increased using condition (108) by $A'_s = 0$ and $x = \frac{N + R_s A_e}{R_b b}$.

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 $\alpha_n = \frac{N}{R_b bh_0}$:

a) by $\alpha_n \leq \xi_R$

$$A = A'_s = \frac{R_b bh_0}{R_s} \alpha_{m1} - \alpha_e \left(1 - \frac{\alpha_e}{2}\right) \frac{1}{1 - \delta} \tag{112}$$

b) by $\alpha_e > \xi_R$

$$A_s = A'_s = \frac{R_b bh_0}{R_s} \alpha_{m1} - \xi \left(1 - \frac{\xi}{2}\right) \frac{1}{1 - \delta} \tag{113}$$

Where $\xi$ – relative height of compressed zone determined by formula (109) or (110).

Value $\alpha_e$ in formula (109) can be determined by the following formula:

$$\alpha_e = \frac{\alpha_{m1} - \alpha_e \left(1 - \frac{\alpha_e}{2}\right)}{1 - \delta} \tag{114}$$

And in formula (110) it’s determined by formula (114) replacing $\alpha_n$ by $(\alpha_n + \xi_R)/2$.

In formulas (112)–(114):

$$\alpha_{m1} = \frac{N_e}{R_b bh_0^2}; \quad \delta = \frac{a'}{h_0}.$$ 

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:

$$A_s = A'_s = \alpha_s \frac{R_b bh_0}{R_s},$$

Where $\alpha_s$ is determined according to the diagram of Draft 35 in relation to the following values:

$$\alpha_m = \frac{M}{R_b bh_0^2}; \quad \alpha_n = \frac{N}{R_b bh_0}.$$ 

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

\[ \alpha_n = \frac{N}{R_p bh_0}; \quad \alpha_m = \frac{M}{R_p bh_0}; \quad \alpha_s = \frac{R_p A_s}{R_p bh_0} \]

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 0.15\( h_0 \) 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 \( \eta \).

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:

\[ A_{sl} = A_{sl,1}(n_t + 1) \]

Where \( A_{sl,1} \) is section area of one intermediate rod; by different diameters it is taken average section area of the rod;

\( n_t \) – 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 \( A_{sl} \) located at one of the surfaces perpendicular to the plane of bending is:

\[ A_{sl} = \frac{A_{s,rot}}{2} - A_{sl}, \]

Where \( A_{s,rot} \) – 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:

\[ N_{ei} \leq R_p bh^2 \left[ 0.5\xi(1-\xi) + \alpha_{sl} (\xi_1 - \delta_1)(1-\xi_1 - \delta_1) - 0.05\alpha_{sl} \xi_1^2 + \alpha_{st}(1-2\delta_1) \right], \]

Where

\[ \xi_1 = \frac{\xi}{\omega}; \quad \alpha_{n1} = \frac{N}{R_p bh}; \quad \alpha_{sl} = \frac{R_p A_{sl}}{R_p bh(0.5 - \delta_1)}; \quad \alpha_{st} = \frac{R_p A_{st}}{R_p bh}; \quad \delta_1 = \frac{d_1}{h} \] (see Draft 36)
b) By $\xi > \xi_R$, section strength is checked according to the following condition:

$$Ne_0 \leq R_n b h^2 \alpha_{na} R \frac{\alpha_{na} - \alpha_{nR}}{\alpha_{na} - \alpha_{nR}},$$

(118)

Where $\alpha_{na} = 1 + \frac{R_n A_{x,tot}}{R_n b h}$ – relative value of longitudinal force by even compression of the whole section;

$\alpha_{nR}, \alpha_{nR}$ – Relative values of a bending moment and of longitudinal force by compression zone height $\xi_R h$:

$$\alpha_{nR} = 0.5\xi_R (1 - \xi_R) + \alpha_{sl} (\xi_{1R} - \delta) (1 - \xi_{1R} - \delta) - 0.05\alpha_{sl} \xi_{2R} + \alpha_{sl} (1 - 2\delta);$$

$$\alpha_{nR} = \xi_R + \alpha_{sl} (2\xi_{1R} - 1);$$

$$\xi_{1R} = \frac{\xi_R}{\alpha}; \xi_R, \omega \text{ – see table 18 and 19.}$$

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 $l_0 \leq 20h$ can be made according to the following condition:

$$N \leq \varphi \left( R_n A + R_{nR} A_{x,tot} \right),$$

(119)

Where $\varphi$ is the coefficient determined by the following formula:

$$\varphi = \varphi_b + 2(\varphi_{sb} - \varphi_b) \alpha_s$$

(120)

But it’s taken no more than $\varphi_{sb}$

Here $\varphi_b, \varphi_{sb}$ are coefficients taken according to Tables 26 and 27;

$$\alpha_s = \frac{R_n A_{x,tot}}{R_n A};$$

$A_{x,tot}$ – See Item 3.63;

By $\alpha_s > 0.5$ it is possible to take $\varphi = \varphi_{sb}$ without using formula (120).

Table 26

<table>
<thead>
<tr>
<th>Concrete</th>
<th>$N_1$</th>
<th>Coefficient $\varphi_b$ by $l_0/h$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>6</td>
</tr>
<tr>
<td>Heavy-weight</td>
<td>N/1</td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0.93</td>
<td>0.92</td>
</tr>
<tr>
<td>0.5</td>
<td>0.92</td>
<td>0.91</td>
</tr>
<tr>
<td>1.0</td>
<td>0.92</td>
<td>0.91</td>
</tr>
<tr>
<td>Concrete</td>
<td>$N_l$</td>
<td>6</td>
</tr>
<tr>
<td>----------</td>
<td>------</td>
<td>---</td>
</tr>
<tr>
<td>Light-weight</td>
<td>0</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>0.91</td>
</tr>
<tr>
<td>Heavy-weight</td>
<td>0</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>0.92</td>
</tr>
<tr>
<td>Light-weight</td>
<td>0</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>0.92</td>
</tr>
</tbody>
</table>

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 $A_{t, tot}/3$

<table>
<thead>
<tr>
<th>Concrete</th>
<th>$N_l$</th>
<th>6</th>
<th>8</th>
<th>10</th>
<th>12</th>
<th>14</th>
<th>16</th>
<th>18</th>
<th>20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heavy-weight</td>
<td>0</td>
<td>0.93</td>
<td>0.92</td>
<td>0.91</td>
<td>0.90</td>
<td>0.89</td>
<td>0.86</td>
<td>0.83</td>
<td>0.79</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.92</td>
<td>0.91</td>
<td>0.90</td>
<td>0.89</td>
<td>0.87</td>
<td>0.84</td>
<td>0.79</td>
<td>0.74</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>0.91</td>
<td>0.90</td>
<td>0.88</td>
<td>0.84</td>
<td>0.81</td>
<td>0.78</td>
<td>0.74</td>
<td>0.68</td>
</tr>
<tr>
<td>Light-weight</td>
<td>0</td>
<td>0.92</td>
<td>0.92</td>
<td>0.91</td>
<td>0.89</td>
<td>0.88</td>
<td>0.85</td>
<td>0.82</td>
<td>0.77</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.92</td>
<td>0.91</td>
<td>0.90</td>
<td>0.88</td>
<td>0.86</td>
<td>0.83</td>
<td>0.77</td>
<td>0.71</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>0.92</td>
<td>0.91</td>
<td>0.90</td>
<td>0.88</td>
<td>0.85</td>
<td>0.82</td>
<td>0.74</td>
<td>0.67</td>
</tr>
</tbody>
</table>

B. By $0.25h > a = a' \geq 0.15h$ or by section area of no intermediate rods (see a sketch) is equal or more than $A_{t, tot}/3$ independently on value $a$

<table>
<thead>
<tr>
<th>Concrete</th>
<th>$N_l$</th>
<th>6</th>
<th>8</th>
<th>10</th>
<th>12</th>
<th>14</th>
<th>16</th>
<th>18</th>
<th>20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heavy-weight</td>
<td>0</td>
<td>0.92</td>
<td>0.92</td>
<td>0.91</td>
<td>0.89</td>
<td>0.87</td>
<td>0.85</td>
<td>0.82</td>
<td>0.79</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.92</td>
<td>0.91</td>
<td>0.90</td>
<td>0.88</td>
<td>0.86</td>
<td>0.84</td>
<td>0.81</td>
<td>0.76</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>0.92</td>
<td>0.91</td>
<td>0.89</td>
<td>0.86</td>
<td>0.82</td>
<td>0.77</td>
<td>0.70</td>
<td>0.63</td>
</tr>
<tr>
<td>Light-weight</td>
<td>0</td>
<td>0.92</td>
<td>0.92</td>
<td>0.91</td>
<td>0.89</td>
<td>0.88</td>
<td>0.85</td>
<td>0.81</td>
<td>0.76</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.92</td>
<td>0.91</td>
<td>0.90</td>
<td>0.88</td>
<td>0.86</td>
<td>0.81</td>
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<td>0.90</td>
<td>0.88</td>
<td>0.84</td>
<td>0.76</td>
<td>0.68</td>
<td>0.60</td>
<td>0.52</td>
</tr>
</tbody>
</table>

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 WITH 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:

\[ x = \frac{N + R_s A_{t} - R_s A_{tl}}{R_s b} ; \]  \hspace{1cm} (107a)

\[ \xi = \frac{\alpha_s (1 - \xi_R) + (\alpha_s + \alpha_{s'}) \xi_R + (\alpha_s - \alpha_{s'})}{1 - \xi_R + 2 \alpha_s} ; \] \hspace{1cm} (109a)

\[ \xi_s = \frac{\alpha_s' + \psi_s \alpha_s - \alpha_{s'}}{2 + \sqrt{\left(\alpha_s + \psi_s \alpha_s - \alpha_{s'}\right)^2 + \psi_s \alpha_s \omega}} ; \] \hspace{1cm} (110a)

Where $\alpha_{s'} = \frac{R_s A_{tl}}{R_s bh_0}$

#### 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:
\[ A_s' = \frac{Ne - 0.4R_ybh_0^2}{R_{sc}(h_0 - a')} \geq 0; \]  
\[ A_s = \frac{0.55R_ybh_0 - N}{R_s} + A_s'; \]  
(121)  
(122)

For elements of concrete more than B30:

\[ A_s' = \frac{Ne - \alpha_R R_ybh_0^2}{R_{sc}(h_0 - a')} \geq 0; \]  
\[ A_s = \frac{\xi_R R_ybh_0 - N}{R_s} + A_s'; \]  
(123)  
(124)

Where \( \alpha_R, \xi_R \) 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:

\[ A_{s,\text{min}} = \frac{N(h_0 - a' - e) - R_ybh(h/2 - a')}{R_{sc}(h_0 - a')}; \]  
(125)

And the section area of reinforcement \( S' \) is determined in the following manner:

- by negative value of \( A_{s,\text{min}} \) – by the following formula:

\[ A_s = \frac{(N - R_yba') - \sqrt{(N - R_yba')^2 - N(N - 2R_ybh_0 + 2R_ybe)}}{R_{sc}}; \]  
(126)

- by positive value of \( A_{s,\text{min}} \) – by the following formula:

\[ A_s = \frac{N - R_ybh}{R_{sc}} - A_{s,\text{min}}; \]  
(127)

If accepted section area of compressed reinforcement \( A_{s,\text{fact}} \) 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:

\[ A_s = \frac{\xi_R R_ybh_0 - N + R_{sc}A_{s,\text{fact}}}{R_s}; \]  
(128)

Where \( \xi \) is determined according to table 20 in compliance with the following value:

\[ \alpha_m = \frac{Ne - R_{sc}A_{s,\text{fact}}(h_0 - a')}{R_ybh_0^2}; \]  
(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 \( b'_j \) wide in compliance with Item 3.61:

\[ N \leq R_y b'_j h'_j \]  \hspace{1cm} (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 \( x = \frac{N - R_y A_{ov}}{R_y b} \):

a) by \( x \leq \xi_k h_0 \) section area is examined according to the following condition:

\[ N e \leq R_y b x (h_0 - x / 2) + R_y A_{ov} (h_0 - h'_j + x / 2) + R_y A_h (h_0 - a') \]  \hspace{1cm} (131)

b) by \( x > \xi_k h_0 \) section area is examined according to condition (131) determining the height of compressed zone by the following formula:

\[ x = h_0 \left[ -\frac{\alpha_z + \psi \alpha_s + \alpha_{ov} - \alpha_n}{2} + \sqrt{\frac{\alpha_z + \psi \alpha_s + \alpha_{ov} - \alpha_n}{2}^2 + \psi \alpha_n} \right] \]  \hspace{1cm} (132)

Where \( \alpha_z = \frac{R_y A_{ov}}{R_y b h_0} ; \alpha_n = \frac{N}{R_y b h_0} ; \alpha_{ov} = \frac{A_{ov}}{b h_0} \)

\( \psi, \xi_k, \alpha \) – See table 18 and 19

\( A_{ov} \) – Area of compressed flange overhangs equal to \( A_{ov} = (b'_j - b) h'_j \)

If value \( x \) determined by formula (132) is more than \( h - h'_j \) (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 \( b'_j = b_j \)) the calculation is made by formulas (131) and (132) replacing \( b_j \) by \( b'_j \) and \( h'_j \) by \( (h + h'_j - h_j) \) taking \( A_{ov} = -(b_j - b)(h-h'_j-h_j) \).

Note. By variable height of the flange overhangs values \( h_j \) and \( h'_j \) 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 \( b'_j \) wide according to Item 3.62.

If condition (130) is not met so reinforcement is chosen according to relative height of compressed zone \( \xi \):

a) by \( \xi \leq \xi_k \)

\[ A_s = A_s = \frac{R_y b h_0}{R_s} \frac{\alpha_{ml} - \xi (1 - \xi / 2) - \alpha_m}{1 - \delta} \]  \hspace{1cm} (134)

b) by \( \xi > \xi_k \)

\[ A_s = A_s = \frac{R_y b h_0}{R_s} \frac{\alpha_{ml} - \xi (1 - \xi / 2) - \alpha_m}{1 - \delta} \]  \hspace{1cm} (135)
Where relative height of compressed zone $\xi_1 = x / h_0$ is determined according to formula (132) by:

$$\alpha_1 = \frac{\alpha_{m1} - \xi(1 - \xi) - \alpha_{m,ov}}{1 - \delta} \quad (136)$$

In formulas (133)–(136):

$\alpha_{m,1} = \frac{Ne}{R_0 b h_0^2}$; $\delta = \alpha / h_0$;

$\alpha_{m,ov} = \alpha_{ov} \left( 1 - 0.5 h_i / h_0 \right)$.

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 $\xi_{cir}$:

$$\xi_{cir} = \frac{N + R_x A_{s,tot}}{R_x A + 2.7 R_x A_{s,tot}} \quad (137)$$

a) by $15 < \xi_{cir} < 0.6$ – according to the following condition:

$$Ne_0 \leq \left( R_x A_{m} + R_x A_{s,tot} r_s \right) \sin \frac{\pi \xi_{cir}}{2} + R_x A_{s,tot} r_s \left( 1 - 1.78 \xi_{cir} \right) \left( 0.2 + 1.3 \xi_{cir} \right) \quad (138)$$

b) by $\xi_{cir} \leq 0.15$ – according to the following condition:

$$Ne_0 \leq \left( R_x A_{m} + R_x A_{s,tot} r_s \right) \sin \frac{\pi \xi_{cir}}{2} + 0.29 R_x A_{s,tot} r_s \quad (139)$$

Where $\xi_{cir1} = \frac{N + 0.75 R_x A_{s,tot}}{R_x A + R_x A_{s,tot}} \quad (140)$

c) By $\xi_{cir} \geq 0.6$ – according to the following condition:

$$Ne_0 \leq \left( R_x A_{m} + R_x A_{s,tot} r_s \right) \sin \frac{\pi \xi_{cir2}}{2} \quad (141)$$

Where $\xi_{cir2} = \frac{N}{R_x A + R_x A_{s,tot}} \quad (142)$

In formulas (137)–(142):

$A_{s,tot}$ – Section area of total longitudinal reinforcement;

$r_m = \frac{r_1 + r_2}{2}$;

$r_s$ – 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.
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:

\[
N_{e_0} \leq \alpha_m R_b r_m A \\
A_{s,tot} = \frac{\alpha_s R_b A}{R_s}
\]

\[
\alpha_n = \frac{N}{R_b A}; \quad \alpha_m = \frac{N_{e_0}}{R_b A}; \quad \alpha_s = \frac{R_s A_{s,tot}}{R_b A}
\]

Where values \( \alpha_m \) and \( \alpha_s \) are determined in compliance with the diagram according to values \( \alpha_n = \frac{R_s A_{s,tot}}{R_b A} \) and \( \alpha_m = \frac{N_{e_0}}{R_b A} \) 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:

\[
N_{e_0} \leq \frac{2}{3} R_b A \frac{\sin^3 \pi \xi_{cir}}{\pi} + R_s A_{s,tot} \left( \frac{\sin \pi \xi_{cir}}{\pi} + \phi \right) r_s
\]

Where \( r \) – radius of the cross section;
\( \xi_{cir} \) – Relative area of the compressed concrete zone determined in the following manner:
If the following condition is met:

\[
N \leq 0.77 R_b A + 0.645 R_s A_{s,tot}
\]

According to the following equation:

\[
\xi_{cir} = \frac{N + R_s A_{s,tot} + R_b A \sin 2\pi \xi_{cir}}{R_b A + R_s A_{s,tot}} ;
\]

If equation (146) is not met so it’s determined according to the following equation:

\[
\xi_{cir} = \frac{N + R_b A \sin 2\pi \xi_{cir}}{R_b A + R_s A_{s,tot}} ;
\]

\( \phi \) – Coefficient considering the work of tensile reinforcement and taken equal to; if condition (146) is met \( \phi = 1.6(1 - 1.55 \xi_{cir})\xi_{cir} \) but no more than one; if condition (146) is not met so \( \phi = 0 \);

\( A_{s,tot} \) – 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:

\[ Ne_0 \leq \alpha_m R_b Ar; \]  \hspace{1cm} (149)

\[ A_{s,tot} = \alpha_s \frac{R_b A}{s}; \]  \hspace{1cm} (150)

Where values \( \alpha_m \) and \( \alpha_s \) are determined by Draft 41 according to values \( \alpha_s = \frac{R_b A_{s,tot}}{R_b A} \) and \( \alpha_m = \frac{Ne_0}{R_b Ar} \) 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 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

\[ a \text{ – by } \alpha_s = 0.2; \] \hspace{0.5cm} \( b \text{ – by } \alpha_s = 0.4; \) \hspace{0.5cm} \( c \text{ – by } \alpha_s = 0.6; \) \hspace{0.5cm} \( d \text{ – by } \alpha_s = 1.0 \)

(see \( \alpha_s = \frac{R_b A_{s,tot}}{R_b bh} \))

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 \( \alpha_s \) are located inside of the part bounded by a curve corresponding to parameter \( \alpha_{nl} \) 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 \( \eta_x \) and \( \eta_y \) 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.
\[ A_{sx} = A_{s1,x} \left( n_x + 1 \right) + \left( 2A_{s1,y} - A_{s1,x} - A_{s1,y} \right) \frac{\beta}{1 + \beta}; \]  
\[ A_{sy} = \frac{A_{s,\text{tot}}}{2} - A_{sx} \]  

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;  
\( A_{s10} \) – Angle rod area;  
\[ \beta = \frac{M_x}{h_x \cdot h_x}; \]  
\( h_x, h_y \) – Section height by eccentric compression in planes \( x \) and \( y \);  
\( A_{s,\text{tot}} \) – Section area of total longitudinal reinforcement.  

Parameters \( \alpha_s \) and \( \alpha_{ni} \) are determined by following formulas:  
\[ \alpha_s = \frac{R_s A_{s,\text{tot}}}{R_b bh} \; ; \; \alpha_{ni} = \frac{N}{R_b bh} \]  

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 \( b/f = 3–5 \) 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 \( \alpha_s = \frac{R_s A_{s,\text{tot}}}{R_b bh} \) ); \( e \) – by \( \alpha_s = 1.8 \); \( f \) – by \( \alpha_s = 2.8 \) (where \( \alpha_s = \frac{R_s A_{s,\text{tot}}}{R_b bh} \))  

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 \( N(h_0 - a^0)/2 \); limit moment \( M_x^0 \) 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:

\[ N \sigma \leq R_b S_b - \Sigma \sigma_{si} S_{si} \]  \hspace{1cm} (153)

Where \( \sigma \) – 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;

\( \sigma_{si} \) – Stress in the \( i \)-rod of longitudinal reinforcement determined according to the present item instructions.

The height of compressed zone \( x \) and stresses \( \sigma_{si} \) are determined according to the following equations:

\[ R_b A_b - \Sigma \sigma_{si} A_{si} - N = 0 \]  \hspace{1cm} (154)

\[ \sigma_{si} = \frac{\sigma_{sc,u}}{1 - \frac{\omega}{\xi_i}} \left( \frac{\omega}{\xi_i} - 1 \right) \]  \hspace{1cm} (155)

In formulas (154) and (155):

\( A_{si} \) – Section area of \( i \)-rod of longitudinal reinforcement;

\( \xi_i \) – 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);

\( \omega \) – 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; \( B \) – the same in tensile reinforcement; \( 1 \) – 8 – rods.

Stress \( \sigma_{si} \) 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 $\overline{\tau}$, $S_b$ and $S_s$ 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 $\overline{\tau}$, $S_b$ and $S_s$ 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 $\theta$ 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 $\theta$ and to make similar calculation one more time.

**EXAMPLES OF CALCULATION**

**RECTANGULAR SECTIONS WITH SYMMETRICAL REINFORCEMENT**

**Example 24. Given:** a column with the framework, 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 $A_x = A'_x = 1232 \text{mm}^2$ (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:** $h_b = 500 - 40 = 460$ 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):

$M_{II} = M_I = M + N \frac{h_b - a'}{2} = 213 + 700 \frac{0.46 - 0.04}{2} = 360$ kN·m

$M_I = M_{II} = M_I + N_l \frac{h_b - a'}{2} = 140 + 560 \frac{0.46 - 0.04}{2} = 276.5$ 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 $\gamma_b = 1.1$).

As $l_0 / h = 6 / 0.5 = 12 > 10$ so the calculation is made considering the column deflection according to Item 3.54, calculating $N_{EI}$ by formula (93).

For that we determine:
\[ \varphi_i = 1 + \beta \frac{M_u}{M_1} = 1 + 1 \frac{276.5}{360} = 1.77 \]

[Here \( \beta = 1.0 \) for heavy-weight concrete (see Table 16)];

\[ \mu \alpha = \frac{A_t + A_e}{bh} \frac{E_s}{E_b} = \frac{2 \cdot 1232 \cdot 2 \cdot 10^5}{400 \cdot 500 \cdot 2.7 \cdot 10^4} = 0.0913; \]

\[ e_0 = \frac{N}{M} \frac{213 \cdot 10^6}{700 \cdot 10^3} = 304 \text{ mm} > e_w = h/30 \]

So that means accidental eccentricity is not to be taken into account.

As \( \frac{e_0}{h} = \frac{304}{500} = 0.608 > \delta_{e,\text{min}} = 0.5 - 0.01 \frac{l_0}{h} - 0.01R_b = 0.5 - 0.01 \cdot 12 \cdot 16 = 0.22 \) so we take \( \delta_e = \frac{e_0}{h} = 0.608 \).

Coefficient \( \eta \) is to be determined by formula (91):

Value \( e \) is:

\[ e = e_0 \eta + \frac{h_0 - a'}{2} = 304 \cdot 1.115 + \frac{460 - 40}{2} = 549 \text{ mm} \approx 0.55 \text{ m}. \]

Let’s determine the height of compressed zone \( x \) by formula (107):

\[ N = 600 \text{ kN}, M = 170 \text{ kN} \cdot \text{m}; \text{ from wind load} N_{sh} = 200 \text{ kN}, M_{sh} = 110 \text{ kN} \cdot \text{m}; \]

\[ l_0 = 8 \text{ m}. \]

It is required to determine section area of reinforcement.

Example 25. Given: section of the element with dimensions \( b = 400 \text{ mm}, h = 500 \text{ mm}; a = a' = 40 \text{ mm}; \) heavy-weight concrete B25 (\( E_b = 2.7 \cdot 10^4 \text{ MPa}; \) symmetrical reinforcement A-III (\( R_s = R_{sc} = 365 \text{ MPa}; \) \( E_s = 2 \cdot 10^5 \text{ MPa}; \) longitudinal forces and bending moments: from dead loads and long-term loads \( N_1 = 600 \text{ kN}, M_1 = 170 \text{ kN} \cdot \text{m}; \) from wind load \( N_{sh} = 200 \text{ kN}, M_{sh} = 110 \text{ kN} \cdot \text{m}; \) design length \( l_0 = 8 \text{ m}. \)

It is required to determine section area of reinforcement.

Calculation: \( h_0 = 500 - 40 = 460 \text{ mm}. \) As there are wind load forces so let’s check condition (1). For that we determine:

\[ \text{kN} \cdot \text{m}; \]

\[ \text{kN} \cdot \text{m}; \]

\[ \text{kN}; \]

\[ \text{kN} \cdot \text{m}. \]

As \( 0.82 M_\Pi = 0.82 \cdot 448 = 368 \text{ kN} \cdot \text{m} > M_1 = 296 \text{ kN} \cdot \text{m} \) so the calculation is made only according to case „b”, that is as regards all loads, taking \( R_b = 16 \text{ MPa} \) (by \( \gamma_b = 1.1 \)).

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:

\[ [\beta = 1.0 \text{ – See Table 16}]; \]

\[ e_0 = \frac{M}{N} = \frac{280 \cdot 10^6}{800 \cdot 10^3} = 350 \text{ mm} > e_w = h/30 \]

(See Item 3.50)
As $e_0/h = 350/500 = 0.7 > \delta_{e,\text{min}} = 0.5 - 0.01 - 0.01R_b$, so we take $\delta_e = 0.7$.

On the first approximation we take $\mu = 0.01$, $\mu' = 7.4$.

So

$$N_{cr} = \frac{1.6E_bh}{(l_0/h)^2} \left[ \frac{0.11}{0.1 + \delta_e} + \frac{0.1}{3\varphi_i} + \mu\alpha \left( \frac{h_0 - a'}{h} \right)^2 \right] = \frac{1.6 \cdot 2.7 \cdot 10^4 \cdot 400 \cdot 500}{16^2} \times$$

$$\times \left[ \frac{0.11}{0.1 + 0.7} + \frac{0.1}{3 \cdot 1.66} + 0.01 \cdot 7.4 \left( \frac{460 - 40}{500} \right)^2 \right] = 33.75 \cdot 10^6 \left( 0.0477 + 0.0522 \right) = 3372 \cdot 10^3 H = 3372 \text{ kN}$$

Coefficient $\eta$ is:

Considering the element deflection value $e$ is:

required reinforcement we determine according to Item 3.62.

Let’s determine the following values:

By Table 18 we find $\xi_R = 0.55$.

As $\alpha_n < \xi_R$, so value $A_s$ is to be determined by Formula (112):

$$A_s = A_s' = \frac{R_b h_0}{R_s} \frac{\alpha_n \alpha - (1 - \alpha_n/2)}{1 - \delta} = \frac{16 \cdot 400 \cdot 460}{365} \frac{0.395 - 0.272(1 - 0.272/2)}{1 - 0.087} = 1413 \text{ mm}^2,$$

From which

As the present reinforcement is more than reinforcement taken during determination of $N_{cr} (\mu = 0.01$) so value $A_s = 1413 \text{ mm}^2$ is determined „on the safe side” and it can be decreased if value $\mu$. is specified more exact

We take $\mu = (0.01 + 0.014)/2 = 0.012$ and determine value $A_s = :$

$$N_{cr} = 33.75 \cdot 10^6 \left[ 0.0477 + 0.012 \cdot 7.4 \times \right.$$

$$\left. \cdot \left( \frac{460 - 40}{500} \right)^2 \right] = 3724 \cdot 10^3 H = 3724 \text{ kN};$$

mm;

$\text{mm}^2$.

Finally we take $A_s = 1362 \text{ mm}^2 (2 \varnothing 25 + 1 \varnothing 22)$.

**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: $N = 800 \text{ kN}; M = 280 \text{ kN} \cdot \text{m}; = 16; = 0.66$.

Let’s determine values $\alpha_n$ и $\alpha_m$.

By diagram $\delta$ of Annex 3 by $\alpha_n = 0.272$, $\alpha_m = 0.207$ and $\lambda = 15$ we find $\alpha_s = 0.16$.

By diagram $\theta$ of Annex 3 by $\alpha_n = 0.272$, $\alpha_m = 0.207$ and $\lambda = 20$ we find $\alpha_s = 0.2$. 
Value $\alpha_s$ corresponding to $\lambda = 16$ is to be determined by means of linear interpolation:

So reinforcement area is:

$$A_s = 1362 \text{ mm}^2.$$ 

We take $A_s = 1362 \text{ mm}^2$.

**Example 27.** Given: a column with the multistory framework with the section dimensions $b = 400 \text{ mm}$, $h = 500 \text{ mm}$; $a = a' = 40 \text{ mm}$; heavy-weight concrete B25 ($E_b = 2.7 \cdot 10^4 \text{ MPa}$); symmetrical reinforcement A-III ($R_s = R_{sc} = 365 \text{ MPa}$; $E_s = 2 \cdot 10^5 \text{ MPa}$); longitudinal forces and bending moments in the support section of the column: from dead loads and long-term loads on the floors $N_l = 2200 \text{ kN}$, $M_l = 259 \text{ kN}$·$\text{m}$; from wind loads $N_{sh} = 0$, $M_{sh} = 53.4 \text{ kN}$·$\text{m}$; no short-term loads on the floor; design length of the column is $l_0 = 6 \text{ m}$.

It is required to determine reinforcement area.

Calculation. $h_0 = h - a = 500 - 40 = 460 \text{ mm}$. As there is the wind load force we determine condition (1). We determine:

- $kN$·$\text{m}$;
- $kN$;
- $kN$·$\text{m}$;
- $kN$·$\text{m}$.

As $0.82 M_{II} = 0.82 \cdot 784.4 = 643 \text{ kN}$·$\text{m} < M_l = 721 \text{ kN}$·$\text{m}$ 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 $R_b = 13 \text{ MPa}$ (that is by $\gamma_b = 0.9$) and according to case "b" — as regards all loads by $R_b = 16 \text{ MPa}$ (that is by $\gamma_b = 1.1$). 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 $\eta_v$ for columns and multistory 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 $M = M_v \eta_v = 259 \text{ kN}$·$\text{m}$.

Design longitudinal force is $N = N_l = 2200 \text{ kN}$, so $e_0 = 118 \text{ mm}$. We take $e_0 = 118 \text{ mm}$.

By formula (111) we determine $e = e_0 + (h_0 - a'/2 = 118 + (460 - 40)/2 = 328 \text{ mm}$.

Required reinforcement is to be determined according to Item 3.62. Let’s determine the following values:

By Table 18 we find $\xi_R = 0.604$.

As $\alpha_v = 0.92 > \xi_R = 0.604$ so value $A_s =$ is to be determined by formula (113). For that we calculate values $\alpha_v$ and $\xi$ by formulas (114) and (109):

$$A_s = A_s' = \frac{R_b h_0}{R_s} \frac{\alpha_v - \xi (1 - \xi / 2)}{1 - \delta} = \frac{13 \cdot 400 \cdot 460}{365} \frac{0.656 - 0.772 (1 - 0.772 / 2)}{1 - 0.087} = 1304 \text{ mm}^2.$$ 

Calculation by case "b". In compliance with Item 3.54 we determine coefficient $\eta_v$, taking reinforcement, calculated according to case "a", that is:

$$[\beta = 1.0 – \text{See Table 16}];$$ 

mm.

As $e_0/h = 0.293 > \delta_{v,min} = 0.5 - 0.01 l_0/h - 0.01 R_b = 0.5 -0.01 \cdot 12 - 0.01 \cdot 16 = 0.22$, we take $\delta_v = e_0/h = 0.293$;
By formula (93) we determine $N_{cr}$:

$$
N_{cr} = \frac{1.6 E_s b h}{(l_0/h)^2} \left[ \frac{0.11}{0.1 + \delta} + 0.1 \frac{h_0 - a'}{a} \right] = \frac{1.6 \cdot 2.7 \cdot 10^4 \cdot 400 \cdot 500}{12^2} \times \left[ \frac{0.11}{0.1 + 0.293} + 0.096 \left( \frac{460 - 40}{500} \right)^2 \right] = 8021 \cdot 10^3 N.
$$

So coefficient $\eta$ is:

According to Item 3.56 coefficient $\eta = \eta_h = 1.38$ is multiplied by the wind loads moment $M_{sh} = M$ and coefficient $\eta_v = 1.0$, that’s why considering the element deflection the moment is equal to:

\( \text{kNm} \).

Required reinforcement we determine by Item 3.62 similar to the calculation according to case "a" taking $R_0 = 16 \text{ MPa}$:

\( \text{mm} \);

According to Table 18 we find $\xi_R = 0.55$.
As $\alpha_n > \xi_R$ so we determine value $A_s = \text{by formula (113)}$:

So

$$
A_s = A_s' = \frac{R_0 b h_0}{R_s} \frac{\alpha_m - \xi (1 - \xi / 2)}{1 - \delta} = \frac{16 \cdot 400 \cdot 460}{365} \frac{0.586 - 0.675(1 - 0.675/2)}{1 - 0.087} = 1228 \text{ mm}^2 < 1304 \text{ mm}^2.
$$

Finally we take $A_s = 1362 \text{ mm}^2$ (2 $\varnothing 25 + 1 \varnothing 22 > 1304 \text{ mm}^2$).

**Example 28.** Given: the element section with dimensions $b = 400 \text{ mm}$, $h = 600 \text{ mm}$; heavy-weight concrete $B25$ ($R_0 = 16 \text{ MPa}$ by $\gamma_{c2} = 1.1$; $E_b = 2.7 \cdot 10^4 \text{ MPa}$); reinforcement $A$-III ($R_s = R_{sc} = 365 \text{ MPa}$; $E_s = 2 \cdot 10^5 \text{ MPa}$) located in the section as it’s shown on Draft 46; longitudinal forces and bending moments: from all loads $N = 500 \text{ kN}$, $M = 500 \text{ kNm}$; from dead loads and long-term loads $N_l = 350 \text{ kN}$, $M_l = 350 \text{ kNm}$; design length $l_0 = 10 \text{ m}$.

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 $A_{sl,l} = 491 \text{ mm}^2$ ($\varnothing 25$), $\eta_l = 2$ and $A_{sl,tot} = 6890 \text{ mm}^2$ ($8 \varnothing 28 + 4 \varnothing 25$) we find reinforcement area $A_{sl}$ and $A_{sl,tot}$:

\( \text{mm}^2 \);

According to Draft 46 we have $a_1 = 45 \text{ mm}$, 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:

$$
[\beta = 1.0 \text{ (See Table 16)}];
$$
As } v / h = 1.67 > δ_{e,min} = 0.5 - 0.01 l_0/h - 0.01 R_b, so we take } v / h = 1.67.

Value } \mu \alpha \text{ we determine as for the section with reinforcement located along the height of the section in compliance with Item 3.54:

So

\[ N_{cr} = \frac{1.6E_e bh}{(l_0 / h)^2} = \left[ \frac{0.11}{0.1 + \delta_e} + \frac{0.1}{3 \varphi_l} + \mu \alpha \left( \frac{h_0 - a'}{h} \right)^2 \right] \frac{1.6 \cdot 2.7 \cdot 10^4 \cdot 400 \cdot 600}{16.7^2} \times \frac{0.11}{0.1 + 1.67} \left( \frac{0.1}{3 \cdot 1.7} + 0.11 \right) = 5271 \cdot 10^3 \text{ H} = 5271 \text{ kN} \]

Coefficient } \eta \text{ is:

Let’s determine the following values:

According to Table 18 we find } \omega = 0.722 \text{ and } \xi_b = 0.55. As } 0.24 < \xi_b = 0.55 \text{ so section strength is to be determined by formula (117):

\[ R_b h^2 \left[ 0.5 \xi \left( 1 - \xi \right) + \alpha_s \left( \xi - \xi_1 \right) \left( 1 - \xi_1 - \delta_1 \right) - 0.05 \alpha_s \xi_1^2 + \delta_1 \left( 1 - 2 \xi_1 \right) \right] = 16 \cdot 400 \cdot 600^2 \times \left[ 0.5 \cdot 0.24 \left( 1 - 0.24 \right) + 0.329 \left( 0.332 - 0.075 \right) \times \left( 1 - 0.332 - 0.075 \right) - 0.05 \cdot 0.329 \cdot 0.332^2 + + 0.187 \left( 1 - 2 \cdot 0.075 \right) \right] = 688 \cdot 10^6 \text{ H} \cdot \text{mm} = 688 \text{ kH} \cdot \text{m} > N_{eq} \eta = 500 \cdot 1.105 = 552 \text{ kN} \cdot \text{m} \]

That is the section strength is provided.

Example 29. Given: column section with dimensions } b = 600 \text{ mm, } h = 1500 \text{ mm; heavy-weight concrete B30 (} R_b = 19 \text{ MPa by } \gamma_{b2} = 1.1); \text{ reinforcement A-III (} R_s = 365 \text{ 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 = 12000 \text{ kN, } M = 5000 \text{ kN} \cdot \text{m}; \text{ from dead loads and long-term loads } N_l = 8500 \text{ kN, } M_l = 2800 \text{ kN} \cdot \text{m}; \text{ design length of the column in the bending plane } l_0 = 18 \text{ m}, \text{ out of the bending plane } l_0 = 12 \text{ m}; \text{ actual column length } l = 12 \text{ m. It is required to examine the section strength.}

Черт. 47. For the calculation example 29

The calculation in the bending plane is made according to Item 3.63. Taking } A_{sl,1} = 615, 8 \text{ mm}^2 \left( \varnothing 28 \right), \eta = 5 \text{ and } A_{s, tot} = 17417 \text{ mm}^2 \left( 14 \varnothing 32 + 10 \varnothing 28 \right), \text{ we find reinforcement area } A_{sl} \text{ and } A_{st}: A_{sl} = A_{sl,1} (\eta + 1) = 615, 8 (5 + 1) = 3695 \text{ mm}^2, \text{ mm}^2. \text{ Center of gravity of reinforcement located at tensile surface (7 } \varnothing 32 \text{) 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:

\[
R_b h^2 \alpha_{nb} \frac{\alpha_{m} - \alpha_{nb}}{\alpha_{m} - \alpha_{ar}} = 19 \cdot 600 \cdot 1500^2 \cdot 0.24 \times \\
\frac{1.372 - 0.702}{1.372 - 0.61} = 5413 \cdot 10^6 \ \text{H} \cdot \text{mM} = 5413 \ \text{kH} \cdot \text{m} > M = 5000 \ \text{kN} \cdot \text{m},
\]

That is the section strength in the bending plane is provided.

The calculation out of the bending plane. As design length out of the bending plane \( l_0 = 12 \) m and ratio \( l_0/h = 12/0.6 = 20 \) is more than ratio \( l_0/h = 18/1.5 = 12 \), 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 \( h = 600 \) mm instead of the section height.

As according to Item 3.50 the accidental eccentricity is equal to \( e_a = \frac{h}{30} = 600/30 = 20 \) mm \( \geq \frac{l}{600} = \frac{12000}{600} \) and \( l_0 = 12 \) m \( \leq 20h \), so the calculation is made according to Item 3.64.

Section area of intermediate rods located along the short sides is equal to \( A_{s, int} = 4826 \) mm\(^2\) (6 \( \varnothing \) 32). As \( 5800 \) mm\(^2\) > \( A_{s, int} = 4876 \) mm\(^2\) \( \Rightarrow a = 50 \) mm < 0.15\( h = 0.15 \cdot 600 = 90 \) mm so we use Table 27 in the calculation (Part A). According to Tables 26 and 27 by and we find \( \phi_b = 0.674 \) and \( \phi_{sb} = 0.77 \).

Value

By formula (120) we determine coefficient \( \varphi \):

\[
\varphi (R_b + R_{sc}) = 0.746 \cdot (19 \cdot 600 \cdot 1500 + 365 \cdot 17 \cdot 417) = 17 \cdot 500 \ \text{kH} > N = 12 \ 000 \ \text{kN},
\]

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 \( \gamma_{b2} = 0.9 \)); 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 3.50 the calculation is made considering occasional eccentricity \( e_a \).

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 \( N/N_l = 1800/2000 = 0.9 \), \( l_0/h = 6000/400 = 15 \), supposing that there are no intermediate rods by \( a = a' < 0.15 \ h \), we find \( \varphi_b = 0.8 \) and \( \varphi_{sb} = 0.858 \).

Taking in the first approximation \( \varphi = \varphi_{sb} = 0.858 \) according to condition (119) we find:
\[
R_s A_{s,\text{tot}} = \frac{N}{\varphi} - R_s A = \frac{2000 \cdot 10^3}{0.858} - 13 \cdot 400 \cdot 400 = 2331 \cdot 10^3 - 2080 \cdot 10^3 = 251 \cdot 10^3 \text{ N}.
\]

So

As \(\alpha_s < 0.5\) we specify value \(\varphi\) more exact calculating it by formula (120):

In a similar manner we determine

\[
R_s A_{s,\text{tot}} = \frac{2000 \cdot 10^3}{0.814} - 2080 \cdot 10^3 = 377 \cdot 10^3 \text{ N}.
\]

Calculated value \(R_s A_{s,\text{tot}}\) is more than the accepted in the first approximation value so we determine this value one more time:

\[
R_s A_{s,\text{tot}} = \frac{2000 \cdot 10^3}{0.821} - 2080 \cdot 10^3 = 360 \cdot 10^3 \text{ N}.
\]

As determined value \(R_s A_{s,\text{tot}}\) is close to the value accepted in the second approximation so total area of reinforcement section is taken equal to:

\[\text{mm}^2.\]

Finally we take \(A_{s,\text{tot}} = 1018 \text{ mm}^2 (4 \varnothing 18).\)

RECTANGULAR SECTIONS WITH ASYMETRICAL REINFORCEMENT

**Example 31.** Given: element section with dimensions \(b = 400 \text{ mm}, h = 500 \text{ mm} ; a = a' = 40 \text{ mm};\) heavy-weight concrete B25 \((R_b = 13 \text{ MPa by } \gamma_b = 0.9; E_b = 2.7 \cdot 10^4 \text{ MPa});\) reinforcement A-III \((R_s = R_{sc} = 365 \text{ MPa});\) longitudinal force \(N = 800 \text{ kN};\) its eccentricity relating to the center of gravity of concrete section \(e_0 = 500 \text{ mm};\) design length \(l_0 = 4.8 \text{ m}.\)

*It is required* to determine areas of reinforcement section \(S\) and \(S'.\)

Calculation. \(h_0 = 500 - 40 = 460 \text{ mm.}\) As \(4 < l_0/h = 4.8/0.5 = 9.6 < 10\) so the calculation 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:

\[
N_{cr} = 0.15 \frac{E_s A}{(l_0/h)^2} = 0.15 \frac{2.7 \cdot 10^4 \cdot 400 \cdot 500}{9.6^2} = 9110 \cdot 10^3 \text{ N} = 9110 \text{ kN}.
\]

Coefficient \(\eta\) is to be determined by formula (91):

Value \(e\) considering the element deflection:

\[\text{mm.}\]

Required section area of reinforcement \(S'\) and \(S\) we determine by formulas (121) and (122):

\[
A'_{s} = \frac{Ne - 0.4 R_s b h_0^2}{R_{a,\varphi} (h_0 - a')} = \frac{800 \cdot 10^3 \cdot 758 - 0.4 \cdot 13 \cdot 400 \cdot 460^2}{365(460 - 40)} = 1085 \text{ mm}^2 > 0;
\]

\[
A_s = \frac{0.55 R_s b h_0 - N}{R_s} + A'_{s} = \frac{0.55 \cdot 3.25 \cdot 10^4 \cdot 460 - 800 \cdot 10^3}{365} + 1085 = 2498 \text{ mm}^2.
\]

As \(0.018 < 0.025\) so values \(A_s\) and \(A'_{s}\) are not to be specified more exact.

We take \(A_{s,\text{tot}} = 1232 \text{ mm}^2 (2 \varnothing 28), A_s = 2627 \text{ mm}^2 (2 \varnothing 32 + 1 \varnothing 36).\)

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 \((R_b = 20 \text{ MPa by } \gamma_b = 0.9; R_{b,\text{ser}} = 29 \text{ MPa}; E_b = 3.25 \cdot 10^4 \text{ MPa});\) longitudinal reinforcement A-VI; confinement reinforcement meshes of rods A-III, diameter 10 mm \((R_{s,xy} = 365 \text{ MPa}),\) located with spacing \(s = 130 \text{ mm along the whole length of the column; longitudinal force by } \gamma_f > 1.0: \) from all loads
$N = 6600 \text{ kN}$, from dead loads and long-term loads $N_l = 4620 \text{ kN}$; the same by $\gamma_f = 1.0$:

$N = 5500 \text{ kN}$ and $N_l = 3850 \text{ kN}$; primary eccentricity of longitudinal force $e_0 = e_a = 13.3 \text{ mm}$; design length of the column $l_0 = 3.6 \text{ m}$.

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 $h_{ef} = b_{ef} = 350 \text{ mm}$. As $l_0/l_{ef} = 3600/350 = 10.3 < 16$ 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 $h = h_{ef} = 350 \text{ mm}$ 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 $h - 2a_1 = 350 - 2 \cdot 22 = 306 \text{ mm}$ [$58 \text{ mm} < = 76.5 \text{ mm}$ (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 $\varphi$ is to be determined by formula (94) taking $\beta = 1.0$ (see Table 16) and

Critical force $N_{cr}$ is to be determined by formula (93), taking

$\text{mm}^2 (6 \varnothing 25)$,

and multiplying the calculated value by coefficient $\varphi_1 = 0.25 + 0.05 = 0.25 + 0.05 \cdot 10.3 = 0.764$:

$$N_{cr} = \frac{1.6 E_s b h}{(l_0/h)^2} \left[ \frac{0.11}{0.1 + \delta_e} + \frac{0.1}{3 \phi_1} + \mu \alpha \left( \frac{h_0 - a'}{h} \right)^2 \right] \varphi_1 = \frac{1.6 \cdot 3.25 \cdot 10^4 \cdot 350 \cdot 350}{10.3^2} \times$$

$$\times \left[ \frac{0.11}{0.1 + 0.297} + \frac{0.1}{3 \cdot 1.7} + 0.281 \left( \frac{309 - 41}{350} \right)^2 \right] \times 0.764 = 10900 \cdot 10^3 \text{ H} = 10900 \text{ kN}.$$ Coefficient $\eta$ is equal to:

So in compliance with formula (111),

mm.

Let’s determine prism strength $R_{b,\text{red}}$ according to Item 3.57.

Taking $A_{sx} = A_{sy} = 78.5 \text{ mm}^2 (\varnothing 10)$, $n_x = n_y = 5$, $l_x = l_y = 350 \text{ mm}$ and $A_{cf} = h_{cf}b_{cf} = 350 \cdot 350 = 122500 \text{ mm}^2$ (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

$$R_{sc,red} = R_s \frac{1 + \delta_1 \lambda_1}{1 + \delta_2 \lambda_2} = \frac{500 \cdot 1 + 0.54 \cdot 2.04}{1 + 0.54 \cdot 0.77} = 742 \text{ MPa} < R_s = 815 \text{ MPa}$$

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 $\omega$ by formula (104). As $10\mu_{sy} = 10 \cdot 0.0173 = 0.173 > 0.15$, we take $\delta_2 = 0.15$, then $\omega = 0.85 - 0.008 R_b + \delta_2 = 0.85 - 0.008 \cdot 20 + 0.15 = 0.84 \leq 0.9$.

According to Items 3.61 and 3.65 we determine required coefficients $\alpha_n$, $\alpha_s$, and $\psi_c$, 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 and $R_{sc} = R_{sc,red} = 742$ MPa:

$$R_{j,bh_0} = 34.3 \cdot 350 \cdot 309 = 3710 \cdot 10^3 \text{ N}$$

So

Value $\xi_R$ with replacing $R_s$ by $0.8R_s$ is:

that means it was necessary to use formula (110a);

$$R_b, bx(h_0 - 0.5x) + R_s A'(h_0 - a') = 34.3 \cdot 350 \cdot 284(309 - 0.5 \cdot 284) + 742 \cdot 2945(309 - 41) = 1154.5 \cdot 10^6 \text{ H \cdot m} > Ne = 6600 \cdot 0.168 = 1109 \text{ kN \cdot m},$$

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 $\gamma = 1.0$) taking $R_b = R_{b,ser} = 29$ MPa, $R_s = R_{s,ser} = 980$ MPa, $\sigma_{sc,u} = 400$ MPa, $\omega = 0.85 - 0.006 R_{b,ser} = 0.85 - 0.006 \cdot 29 = 0.679$ 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 $l_0/h = 3600/400 = 9$, $e_0/h = 13.3/400 = 0.033$, $\delta_{min} = 0.5 - 0.01 - 0.008 R_{b,ser} = 0.5 - 0.01 \cdot 9 - 0.008 \cdot 29 = 0.178 > e_0/h$, that means $\delta = \delta_{min} = 0.178$.

During determination of coefficient $\varphi$ we consider longitudinal forces $N$ and $N_l$ by $\gamma = 1.0$, that means

so $\varphi = 1 + 0.7 = 1.7$;
Coefficient $\eta$ is:

\[
\text{mm.}
\]

Let’s make a calculation similar to the calculation as regards the strength:

\[
R_b h_0 = 29 \cdot 400 \cdot 3 = 3880 \cdot 10^3 \text{ N};
\]

\[
= 19900 \cdot 10^3 \cdot H = 19900 \eta u.
\]

I-SECTIONS

**Example 33.** Given: section dimensions and location of reinforcement — according to Draft 49; heavy-weight concrete B30 ($E_b = 2.9 \cdot 10^4$ MPa; $R_b = 19$ MPa by $\gamma_b = 1.1$); reinforcement A-III ($R_s = R_{sc} = 365$ MPa); its cross-section area $A_s = A_s' = 5630 \text{ mm}^2 (7 \Omega 32)$; longitudinal forces and bending moments: from dead loads and long-term loads $N_l = 2000 \text{ kN}, M_l = 2460 \text{ kN.m};$ from all loads $N = 2500 \text{ kN}, M = 3700 \text{ kN.m};$ design length of the element: in the bending plane $l_0 = 16.2 \text{ m},$ out of the bending plane $l_0 = 10.8 \text{ m};$ actual length of the element $l = 10.8 \text{ m}.$

It is required to check the section strength.

**Draft 49. For the calculation examples 33, 34 and 39**

The calculation in the bending plane. We take design thickness of the flange equal to the average height of overhangs $h_f' = h_f = 200 + 30/2 = 215 \text{ mm}.$

Let’s determine the area and inertial moment of concrete section:

\[
I = \frac{200 \cdot 1500^3}{12} + \frac{2 \cdot 400 \cdot 215^3}{12} + 2 \cdot 400 \cdot 215 \times
\]

\[
\times \left( \frac{1500}{2} + \frac{215}{2} \right)^2 = 1279 \cdot 10^3 \text{ mm}^4.
\]

Radius of inertia of the section is $\text{mm}.$
As \( l_0/i = 16 \times 200/520 = 31.1 < 35 \) 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:

\[
N_{cr} = \frac{2E_s l^3}{l_0^3} = \frac{2 \times 2.9 \times 10^4 \times 1279 \times 10^8}{16200^2} = 28270 \times 10^3 \text{ kN}
\]

Coefficient \( \eta \) is to be determined by formula (91):

Let’s check condition (130):

\[
R_{b, h'} h' = 19 \times 600 \times 215 = 2451 \times 10^3 \text{ kN} < N = 2500 \text{ kN},
\]

That means that the calculation is made as for the I-section.

Area of compressed flange overhangs is equal to:

\[
\text{mm}^2.
\]

Let’s determine the area of compressed zone:

\[
\text{mm}.
\]

From Table 18 we find \( \xi_R = 0.523 \). As \( x = 228 \text{ mm} < \xi_R h_0 = 0.523 \times 1421 = 743 \text{ mm} \), section strength is to be checked according to condition (131):

\[
R_{b, bx} \left( h_0 - \frac{x}{2} \right) + R_{b, A'} \left( h_0 - \frac{h'}{2} \right) + R_{b, c} A' (h_0 - a') = 19 \times 200 - 228 \times \frac{1421 - 228}{2} + 19 \times 6000 \left( 1421 - \frac{215}{2} \right) + 365 \times 5630 (1421 - 79) = 5847 \times 10^6 N \times \text{mm} = 5847 \text{ kN} \times \text{m} > Ne = 2500 \times 2.293 = 5725 \text{ kN} \times \text{m},
\]

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:

\[
\text{mm};
\]

\[
\text{mm}.
\]

As elasticity out of the bending plane \( l_0/i = 10 \times 800/134 = 80 \) 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 \( h = 600 \text{ mm} \).

As in compliance with Item 3.50 occasional eccentricity is \( e_a = \text{mm} > \text{mm} \), we take \( e_a = \text{that} \) 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 \( b = 2 \times 215 = 430 \text{ mm} \).

Section area of intermediate rods located along both flanges is \( A_{s, int} = 4826 \text{ mm}^2 \) and section area of all rods is \( A_{s, tot} = 11 \times 260 \text{ mm}^2 \). As \( A_{s, tot}/3 = 11 \times 260/3 = 3750 \text{ mm}^2 < A_{s, int} = 4826 \text{ mm}^2 \) so we use Table 27 (Part B) in the calculation. From Table 27 for heavy-weight concrete by \( N/\text{N} = 2000/2500 = 0.8 \) and \( l_0/h = 10.8/0.6 = 18 \) we find \( \varphi_{hb} = 0.724 \).

Value So, \( \varphi = \varphi_{hb} = 0.724 \).

Let’s check condition (119):

\[
\varphi \left( R_b A + R_{sc} A_{s, tot} \right) = 0.724(19 \times 430 \times 600 + 365 \times 11260) = 6525 \times 10^3 \text{N} > N = 2500 \text{ kN}
\]

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 \((R_b = 19 \text{ MPa})\) by \(\gamma_b = 1,1; E_b = 2,9 \cdot 10^4 \text{ MPa}\); symmetrical reinforcement A-III \((R_s = R_{sc} = 365 \text{ MPa})\); longitudinal force \(N = 6000 \text{ kN}\); bending moment \(M = 3100 \text{ kN} \cdot \text{m}\); design length of the element: in the bending plane \(l_0 = 16,2 \text{ m}\), out of the bending plane \(l_0 = 10,8 \text{ m}\).

It is required to determine the section strength of the element.

The calculation in the bending plane. From example 33 we have: \(h' = 15 \text{ mm}\); \(h_0 = 1421 \text{ mm}\); \(a' = 79 \text{ mm}\); \(N_{cr} = 28270 \text{ kN}\).

By formula (91) we determine coefficient \(\eta\):

\[
\eta = 1,127 + \frac{279}{10} \frac{3100 - 79}{6000 - 10^2} = 1327 \text{ mm}.
\]

Let’s check condition (130):

\[
R_{sc}h' = 19 \cdot 600 \cdot 215 = 2451 \cdot 10^3 < H = 2451 \text{ kN} < N = 6000 \text{ kN}.
\]

That means the calculation is to be made as for the I-section.

Area of compressed flange overhangs is equal to:

\[
\text{mm}^2.
\]

Let’s determine values \(\alpha_n, \alpha_{o1}, \alpha_{ov}, \alpha_{n,ov}, \delta\):

From Table 18 we find \(\xi_R = 0,523\).

As \(\xi = \alpha_n - \alpha_{ov} = 1,111 - 0,302 = 0,809 > \xi_R = 0,523\) so reinforcement area is to be determined by formula (135). For that we determine values \(\alpha_n\) and \(\xi_1 = \frac{x}{h_0}\) by formulas (136) and (132)

\[
\alpha_n = \frac{\alpha_{m1} - \xi(1 - \xi / 2) - \alpha_{ov}}{1 - \delta} = \frac{1,037 - 0,809(1 - 0,809 / 2) - 0,279}{1 - 0,55} = 0,292.
\]

From Table 18 we find \(\psi_{sc} = 3,0\) and \(\omega = 0,698\).

\[
\alpha_s + \psi_s \alpha_s + \alpha_{m1} - \alpha_n = \frac{0,292 + 3 \cdot 0,292 + 0,302 - 1,111}{2} = 0,18;
\]

\[
\xi_1 = \frac{-\alpha_n + \psi_s \alpha_s + \alpha_{m1} - \alpha_n}{2} + \sqrt{\left(\frac{\alpha_s + \psi_s \alpha_s + \alpha_{m1} - \alpha_n}{2}\right)^2 + \alpha_n \psi_s \omega} = -0,18 + \sqrt{0,18^2 + 0,292 \cdot 3 \cdot 0,698} = 0,602,
\]

So

\[
A_s = A'_s = \frac{R_s h_0 \alpha_{m1} - \xi_1 (1 - \xi_1 / 2) - \alpha_{m,ov}}{R_s} = \frac{19 \cdot 200 \cdot 1421 \cdot 1,037 - 0,602(1 - 0,602 / 2) - 0,279}{365} = 5278 \text{ mm}^2.
\]

We take \(A_s = A'_s = 5630 \text{ mm}^2\) (7 \(\geq\) 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 \( r_1 = 150 \text{ mm} \), external radius \( r_2 = 250 \text{ mm} \); heavy-weight concrete B25 (\( R_b = 16 \text{ MPa} \) by \( \gamma_b = 1,1 \)); longitudinal reinforcement A-III (\( R_s = R_{sc} = 365 \text{ MPa} \) ); its section area \( A_{s, tot} = 1470 \text{ mm}^2 \) (13 \( \varnothing 12 \)); longitudinal force from total load \( N = 1200 \text{ kN} \), its eccentricity relating to the center of gravity of the section considering the element deflection is \( e_0 = 120 \text{ mm} \).

It is required to check the section strength.

Calculation. Let’s calculate the area of the ring section:

\[
\pi (r_2^2 - r_1^2) = \pi (250^2 - 150^2) = \pi (62500 - 22500) = 39250 \pi \approx 125600 \text{ mm}^2
\]

Relative area of concrete compressed zone:

\[
\xi_{cr} = \frac{N + R_s A_{s, tot}}{R_b A + 2.7 R_s A_{s, tot}} = \frac{1200 \cdot 10^3 + 365 \cdot 1470}{16 \cdot 125600 + 2.7 \cdot 365 \cdot 1470} = 0.502;
\]

mm.

As \( 0.15 < \xi_{cr} = 0.502 < 0.6 \) so section strength is to be checked according to condition (138):

\[
(R_b A_{r, m} + R_s A_{s, tot} r_s) \sin \frac{\pi \xi_{cr}}{3} + R_s A_{s, tot} r_s (1 - 1.7 \xi_{cr}) (0.2 + 1.3 \xi_{cr}) =
\]

\[
= (16 \cdot 125600 \cdot 200 + 365 \cdot 1470 \cdot 200) \sin \frac{(180 \cdot 0.502)}{3.14} + 365 \cdot 1470 \cdot 200 (1 - 1.7 \cdot 0.502) (0.2 + 1.3 \cdot 0.502) =
\]

\[
= 175,4 \cdot 10^6 N \cdot \text{mm} = 175,4 \text{kN} \cdot \text{m} > N_{cr} = 1200 \cdot 0.12 = 144 \text{kN} \cdot \text{m};
\]

that is section strength is provided.

ROUND SECTIONS

Example 36. Given: a section with diameter \( D = 400 \text{ mm} \); \( a = 35 \text{ mm} \); heavy-weight concrete B25 (\( R_b = 13 \text{ MPa} \) by \( \gamma_b = 0.9 \); \( E_b = 2.7 \cdot 10^4 \text{ MPa} \)); longitudinal reinforcement A-III (\( R_s = R_{sc} = 365 \text{ MPa} \); \( E_s = 2 \cdot 10^5 \text{ MPa} \)); its section area \( A_{s, tot} = 3140 \text{ mm}^2 \) (10 \( \varnothing 20 \)); longitudinal forces and bending moments: from dead loads and long-term loads \( N_l = 400 \text{ kN} \cdot \text{m} \); from all loads \( N = 600 \text{ kN} \), \( M = 140 \text{ kN} \cdot \text{m} \); design length of the element \( l_0 = 4 \text{ m} \).

It is required to examine the section strength.

Calculation. Let’s calculate:

Area of the round section \( A = \frac{\pi D^2}{4} = \frac{3.14 \cdot 400^2}{4} = 125600 \text{ mm}^2 \); \n
Radius of the section inertia \( i = \frac{D}{4} = \frac{400}{4} = 100 \text{ mm} \); \n
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:

\[
r_s = \frac{D}{2} - a = \frac{400}{2} - 35 = 165 \text{ mm};
\]

\[
\varphi_l = 1 + \beta \frac{M_l}{M_s} = 1 + \beta \frac{N_l r_s}{M + N r_s} = 1 + \frac{100 + 400 \cdot 0.165}{140 + 600 \cdot 0.165} = 1.695
\]

[здесь \( \beta = 1.0 \) (see Table 16)];

\[
e_0 = M / N = 140 / 600 = 0.233 = 233 \text{ mm}
\]

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 \( \delta_l = e_0 / D = 0.583 \).

Inertia moments of the concrete section and all reinforcement are:

\[
I = \frac{\pi D^4}{64} = \frac{3.14 \cdot 400^4}{64} = 1256 \cdot 10^6 \text{ mm}^4
\]

\[
I_s = \frac{A_{s, tot} r_s^2}{2} = \frac{3140 \cdot 165^2}{2} = 42.74 \cdot 10^6 \text{ mm}^4
\]
\[
\alpha = \frac{E_s}{E_b} = \frac{2 \cdot 10^5}{2.7 \cdot 10^5} = 7.4.
\]

Then
\[
N_{cr} = \frac{6.4E_s}{I_s} \left[ L \left( \frac{0.11}{0.1 + \delta_e} + 0.1 \right) + \alpha I_s \right] = 6.4 \cdot 2.7 \cdot 10^4 \left[ 1256 \cdot 10^6 \left( \frac{0.11}{0.1 + 0.583} + 0.1 \right) + 7.4 \cdot 42.74 \cdot 10^6 \right] = 5505 \cdot 10^3 H = 5505 kN.
\]

Coefficient \( \eta \) we determine by formula (91):
\[
\eta = \frac{1}{1 - \frac{N}{N_{cr}}} = \frac{1}{1 - \frac{600}{5505}} = 1.12.
\]

Section strength is to be examined by means of the diagram of Draft 41.

According to values \( \alpha_n = \frac{N}{R_s A} = \frac{600 \cdot 10^3}{13 \cdot 125600} = 0.367, \)
\[
\alpha_s = \frac{R_s A_{tot}}{R_s A} = \frac{365 \cdot 3140}{13 \cdot 125 \cdot 600} = 0.702 \quad \text{and} \quad \frac{a}{D} = \frac{35}{400} = 0.0875 \quad \text{we find at the diagram} \quad \alpha_n = 0.51.
\]

As \( \alpha_n R_s A_r = 0.51 \cdot 13 \cdot 125 \cdot 600 \cdot 200 = 167 \cdot 10^6 \text{ N.mm} > N e \eta = 600 \cdot 0.233 \times 1.12 = 156.6 \text{ kN.m}, \) 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 \( i = 100 \text{ mm}, A = 125 \cdot 600 \text{ mm}^2, r_s = 165 \text{ mm}. \) 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 \( A_{s,tot} = 0.01 A = 1256 \text{ mm}^2, \) 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 \( \varphi = 1.695, \delta_e = 0.583, I = 1256 \cdot 10^6 \text{ mm}^4. \)

Then
\[
N_{cr} = \frac{6.4E_s}{I_s} \left[ L \left( \frac{0.11}{0.1 + \delta_e} + 0.1 \right) + \alpha I_s \right] = 6.4 \cdot 2.7 \cdot 10^4 \left[ 1256 \cdot 10^6 \left( \frac{0.11}{0.1 + 0.583} + 0.1 \right) + 7.4 \cdot 17.1 \cdot 10^6 \right] = 0.0108 \left(93.4 \cdot 10^6 + 126.5 \cdot 10^6 \right) = 3455 \cdot 10^3 H = 3455 kN.
\]

Coefficient is
\[
\eta = \frac{1}{1 - \frac{N}{N_{cr}}} = \frac{1}{1 - \frac{600}{3455}} = 1.21.
\]

Due to values \( \alpha_n = \frac{N}{R_s A} = \frac{600 \cdot 10^3}{13 \cdot 125600} = 0.367, \)
\[
\alpha_n = \frac{N e \eta}{R_s A_r} = \frac{600 \cdot 10^3 \cdot 233 \cdot 1.21}{13 \cdot 125 \cdot 600 \cdot 200} = 0.518 \quad \text{we find} \quad \alpha_n = 0.74, \text{ thereafter}
\]
\[
A_{s,tot} = \alpha_n R_s A_r = 0.74 \frac{13 \cdot 125 \cdot 600}{365} = 3310 \text{ mm}^2.
\]

As determined reinforcement is more than the one accepted in the first approximation (\( A_{s,tot} = 1256 \text{ mm}^2 \)) so value \( A_{s,tot} = 3310 \text{ mm}^2 \) is determined with „reserve“ and it can be a bit decreased after value \( N_{cr} \) is specified more exact.

We take \( A_{s,tot} = \frac{1256 + 3310}{2} = 2283 \text{ mm}^2 \) and make similar calculation:
\[
I_s = \frac{2283 \cdot 165^2}{2} = 31.08 \cdot 10^6 \text{ mm}^4;
\]
\[ N_c = 0.0108(193.4 \times 10^6 + 7.4 \times 31.08 \times 10^6) = 4573 \text{ kN}; \]
\[ \eta = \frac{1}{\frac{600}{4573}} = 1.151. \]

Due to values \( \alpha_m = 0.518 \frac{1151}{1.21} = 0.493 \), \( \alpha_n = 0.367 \) and \( a/D = 0.1 \) at the diagram of Draft 41 we find \( \alpha_s = 0.68 \).
\[ A_{s, tot} = 68 \times 0.13 \times 125 \times 4573 = 3042 \text{ mm}^2. \]

We take \( A_{s, tot} = 3142 \text{ mm}^2 (10\ Ø 20) \).

ELEMENTS WORKING IN SKEW BENDING

**Example 38.** Given: rectangular section of the column with dimensions \( b = 400 \text{ mm}, h = 600 \text{ mm} \); heavy-weight concrete \( \text{B25} (R_b = 16 \text{ MPa by } \gamma_b = 1.1) \); longitudinal reinforcement \( \text{A-III} (R = R_{sc} = 365 \text{ MPa}) \) located in the section due to Draft 50; in the section act both longitudinal force \( N = 2600 \text{ kN} \) and bending moments at the same time: in the plane parallel to the dimension \( h \), \( -M_x = 240 \text{ kN} \cdot \text{m} \); in the plane parallel to dimension \( b \), \( -M_y = 182.5 \text{ kN} \cdot \text{m} \);

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 \( A_{sx} \) and \( A_{sy} \). Due to Draft 50 \( A_{s1,x} = 0, n_x = 0, A_{s0} = 804.3 \text{ mm}^2 (Ø 32), A_{s1,y} = 314.2 \text{ mm}^2 (Ø 20), \)

\[ \beta = \frac{M_x}{M_y} = \frac{240}{400} = \frac{480}{800} = 0.877. \]

\[ A_{s,x} = A_{s1,x} (n_x + 1) + (2A_{s0} - A_{s1,x} - A_{s1,y}) \frac{\beta}{1 + \beta} = 0 + (2 \times 804.3 - 324.2) \frac{0.877}{1 + 0.877} = 605 \text{ mm}^2; \]

\[ A_{s, tot} = 3845 \text{ mm}^2 (4 Ø 32 + 2 Ø 20); \]

\[ A_{sy} = A_{s,y} / 2 = \frac{3845}{2} = 605 = 1318 \text{ mm}^2. \]

When determining moment \( M_x^0 \) which acts in the plane of axis \( x \) due to Item 3.63 we take:

\[ A_{sl} = A_{s,y} = 1318 \text{ mm}^2; A_{st} = A_{s,x} = 605 \text{ mm}^2; h = 600 \text{ mm}; b = 400 \text{ mm}. \]

\[ \delta_s = a_s / h = 50 / 600 = 0.083; \]

\[ R_{sh} = 16 \times 400 \times 600 = 3840 \times 10^3 \text{ N}; \]

\[ \alpha_{sl} = \frac{R_s A_{sl} / R_{sh}(0.5 - \delta_s)}{3840 \times 10^3 (0.5 - 0.083)} = 0.3; \]

\[ \alpha_{st} = \frac{R_s A_{st} / R_{sh}}{3840 \times 10^3} = 0.06; \]

\[ \alpha = \frac{N / R_{sh}}{3840 \times 10^3} = 0.677. \]

From Table 18 we find \( \omega = 0.722, \xi_R = 0.55 \).
As $\xi = \frac{\alpha_{a1} + \alpha_{sl}}{1 + 2 \alpha_{sl} / \omega} = \frac{0.677 + 0.3}{1 + 2 \cdot 0.3 / 0.722} = 0.534 < \xi_R = 0.55$ so value $M^0_y$ is to be determined by formula (117) due to $\xi_i = \frac{\xi}{\omega} = \frac{0.534}{0.722} = 0.74$:

$$M^0_y = R_b h^2 \left[ 0.5\xi(1 - 0.5\xi) + \alpha_{sl} (\xi_1 - \delta_1) \right] \times (1 - \xi_1 - \delta_1) = 0.05 \alpha_{sl} \xi_1^2 + \alpha_{sl} (1 - 2\delta_1)$$

$$= 3840 \cdot 10^{-3} \cdot 600 \cdot 0.5 \cdot 0.534 (1 - 0.5 \cdot 0.534) + 0.3 (1 - 0.74 - 0.083) \cdot 0.125 = 0.05 \cdot 0.3 \cdot 0.74^2 + 0.06 (1 - 2 - 0.083) \cdot 0.125 = 464.7 \cdot 10^6 N \cdot mm = 464.7 kN \cdot m.$$  

When determining moment $M^0_y$ which acts in the plane of axis $y$, we take: $A_{sl} = A_{sx} = 605 mm^2$; $A_{st} = A_{sy} = 1318 mm^2$; $h = 400 mm$; $b = 600 mm$; $\delta_1 = \frac{a_1}{h} = \frac{50}{400} = 0.125$.

$$\alpha_{sl} = \frac{R_b A_{sl}}{R_b bh(0.5 - \delta_1)} = \frac{365 - 605}{3840 \cdot 10^{-3} (0.5 - 0.125)} = 0.153;$$

$$\alpha_{st} = \frac{R_b A_{st}}{R_b bh} = \frac{365 - 1318}{3840 \cdot 10^{-3}} = 0.125.$$

As $\xi = \frac{\alpha_{a1} + \alpha_{sl}}{1 + 2 \alpha_{sl} / \omega} = \frac{0.677 + 0.153}{1 + 2 \cdot 0.153 / 0.722} = 0.583 > \xi_R = 0.55$ so value $M^0_y$ is to be determined by formula (118) calculating:

$$\alpha_{na} = 1 + \frac{R_b A_{tot}}{R_b bh} = 1 + \frac{365 - 3845}{3840 \cdot 10^{-3}} = 1.365;$$

$$\xi_R = \frac{\xi}{\omega} = \frac{0.55}{0.722} = 0.762;$$

$$\alpha_{nb} = 0.5 \xi_R (1 - \xi_R) + \alpha_{sl} (\xi_{1R} - \delta_1) \times (1 - \xi_{1R} - \delta_1) = 0.05 \alpha_{sl} \xi_{1R}^2 + \alpha_{sl} (1 - 2\delta_1)$$

$$= 0.5 \cdot 0.55(1 - 0.55) + 0.153(0.762 - 0.125) \times (1 - 0.762 - 0.125) - 0.05 \cdot 0.153 - 0.762^2 + 0.125 (1 - 2 - 0.125) = 0.224;$$

$$\alpha_{nh} = \xi_R + \alpha_{sl} (2 \xi_{1R} - 1) = 0.55 + 0.153 (2 \cdot 0.762 - 1) = 0.63.$$  

$$M^0_y = R_b h^2 \alpha_{nh} \left( \frac{\alpha_{na} - \alpha_{nl}}{\alpha_{na} - \alpha_{nl}} \right) = 3840 \cdot 10^{-3} \cdot 400 \cdot 0.224 \cdot 1.365 - 0.677 = 322 \cdot 10^6 N \cdot mm = 322 kN \cdot m.$$  

As $\alpha_{a} = \frac{R_b A_{tot}}{R_b bh} = 0.365$ so section strength is to be checked according to the diagram of Draft 42, $a$, $\delta$ corresponding to $\alpha_1 = 0.2$ and $\alpha = 0.4$. At both diagrams the point with coordinates $M_x / M^0_y = 240 / 464.7 = 0.516$ and $M_y / M^0_y = 182.5 / 322 = 0.566$ lies inside of the area bounded by means of the curve corresponding to parameter $\alpha_{a1} = 0.677$ 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 $h$, $- M_x = 3330 kN \cdot m$ and in the plane parallel to dimension $b$, $- M_y = 396 kN \cdot m$; 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 $M^0_y$ 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

$$M^0_y = \frac{5847 - \frac{N(h_o - a')}{2}}{2} = \frac{5847 - 2500 \cdot 1.421 - 0.079}{2} = 4170 kN \cdot m.$$
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; $b = 2 \cdot 215 = 430$ mm.

Let’s determine distributed reinforcement $A_{sl}$ and $A_{st}$:

- $A_{sl} = 804.3 \text{ mm}^2$ ($\emptyset$ 32); $\eta_l = 3$.
- $A_{sl} = A_{sl,l} (n_l + 1) = 804.3 (3 + 1) = 3220 \text{ mm}^2$.
- $A_{sl} = A_{sl, tot} / 2 - A_{sl} = 1260 / 2 - 3220 = 2410 \text{ mm}^2$.

From Table 18 we find $\omega = 0.698$ and $\xi_R = 0.523$.

- $R_{bh} = 19 \cdot 430 \cdot 600 = 4902 \cdot 10^3 \text{ N};$
- $\delta_1 = a_{1l} / h = 0.083.$

\[
\begin{align*}
\alpha_{sl} &= \frac{R_{bh} A_{sl}}{R_{bh} (0.5 - \delta_1)} = \frac{365 \cdot 3220}{4902 \cdot 10^3 (0.5 - 0.083)} = 0.576; \\
\alpha_{st} &= \frac{N}{R_{bh}} = \frac{2500 \cdot 10^3}{4902 \cdot 10^3} = 0.51; \\
\alpha_{sl,s} &= \frac{R_{bh} A_{sl}}{R_{bh} (0.5 - \delta_1)} = \frac{365 \cdot 2410}{4902 \cdot 10^3} = 0.179; \\
\xi &= \frac{\alpha_{sl} + \alpha_{st}}{1 + 2 \delta / \omega} = \frac{0.51 + 0.576}{1 + 2 \cdot 0.576 / 0.698} = 0.41 < \xi_R = 0.523.
\end{align*}
\]

Value $M_y^0$ is to be determine by formula (117) after calculating $\xi = 0.41 / 0.698 = 0.59: \]

\[
\begin{align*}
M_y^0 &= R_{bh} h^2 \left[0.5 \xi (1 - \xi) + \alpha_{sl} (\xi_1 - \delta_1) \times (1 - \xi_1 - \delta_1) - 0.05 \alpha_{sl} \xi_1^2 + \alpha_{sl} (1 - 2 \delta_1) \right] = \\
&= 19 \cdot 430 \cdot 600^2 \left[0.5 \cdot 0.41 (1 - 0.41) + 0.576 (0.59 - 0.083) (1 - 0.59 - 0.083) - \\
&- 0.05 \cdot 0.576 \cdot 0.59^2 + 0.179 (1 - 2 \cdot 0.083) \right] = 1029 \cdot 10^3 H \cdot \text{mm} = 1029 kN \cdot m.
\end{align*}
\]

Let’s check the section strength taking $b = 200$ mm, $h = 1500$ mm.

As $\alpha_s = \frac{R_{bh} A_{sl, tot}}{R_{bh} (0.5 - \delta_1)} = \frac{365 \cdot 11260}{19 \cdot 200 \cdot 1500} = 0.721$, so section strength is to be checked according to diagrams of Draft 44, $\delta$, $\delta$, corresponding to $\alpha_s = 0.6$ and $\alpha_s = 1.0$.

At both diagrams the point with coordinates $M_x / M_y = 3300 / 4170 = 0.8$ and $M_x / M_y = 396 / 1029 = 0.385$ lies within the area which is bounded by a curve corresponding to the parameter $\alpha_{sl} = N / (R_{bh} h) = 2500 \cdot 10^3 / (19 \cdot 200 \cdot 1500) = 0.44$ 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 $B25$ ($R_c = 16$ MPa by $\gamma_b = 1.1$); 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 $\theta$ between axis $y$ and the line bounding the compressed zone we take as by the calculation of elastic body as regards the; that means:
\[ \text{tg} \theta = \frac{M_y}{M_x} = \frac{l}{L} = \frac{M_y}{M_x} \left( \frac{h}{b} \right)^2 = \frac{2000 \times 600}{250 \times 400} = 1.8. \]

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 \( \xi_i = x_i/h_0 \) by formula
\[ \xi_i = \frac{x_i}{a_i \text{tg} \theta + a_y} \]
where \( a_x \) and \( a_y \)
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 \( \xi_i \) we determine stress \( \sigma_{si} \), taking \( \sigma_{sc,u} = 400 \text{ MPa} \), \( \omega = 0.722 \) (see Table 18):
\[ \sigma_{si} = \sigma_{sc,u} \left( \frac{\omega}{\xi_i} - 1 \right) = \frac{400}{1 - \frac{0.722}{\xi_i} - 1} \times \left( \frac{0.722}{\xi_i} - 1 \right) = 1160 \left( \frac{0.722}{\xi_i} - 1 \right) \text{ (MPa)} \]

At the same time if \( \sigma_{si} > R_s = 365 \text{ MPa} \), what is equivalent to condition \( \xi_i < \xi_R = 0.55 \) (see Table 18) so we take \( \sigma_{si} < R_s = 365 \text{ MPa} \).

If \( \sigma_{si} < -R_{sc} = -365 \text{ MPa} \) we take \( \sigma_{si} = -365 \text{ MPa} \).

The last condition after we insert the equations for \( \sigma_{si} \) into it looks like:
\[ \xi_i > \frac{0.722}{365} = 1.054. \]

Then we determine the sum of forces in all rods \( \sum A_i \sigma_{si} \).

We take equation \( x_1 = h = 600 \text{ mm} \) in the first approximation and make mentioned calculations; the results of the calculations are given in the following table:

<table>
<thead>
<tr>
<th>Number of the rod</th>
<th>( A_{si} ) (mm²)</th>
<th>( a_{si} ) (mm)</th>
<th>( a_{si} \text{tg} \theta + a_{sy} ) (mm)</th>
<th>( x_i = 600 \text{ mm} )</th>
<th>( x_i = 660 \text{ mm} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \xi )</td>
<td>( \sigma_{si} ) (MPa)</td>
<td>( A_{si} \sigma_{si} ) H</td>
<td>( \xi )</td>
<td>( \sigma_{si} ) (MPa)</td>
</tr>
<tr>
<td>1</td>
<td>804,3</td>
<td>350</td>
<td>50</td>
<td>680</td>
<td>0.882</td>
</tr>
<tr>
<td>2</td>
<td>804,3</td>
<td>50</td>
<td>50</td>
<td>140</td>
<td>4.29</td>
</tr>
<tr>
<td>3</td>
<td>314,2</td>
<td>350</td>
<td>300</td>
<td>930</td>
<td>0.645</td>
</tr>
<tr>
<td>4</td>
<td>314,2</td>
<td>50</td>
<td>300</td>
<td>390</td>
<td>1.54</td>
</tr>
<tr>
<td>5</td>
<td>804,3</td>
<td>350</td>
<td>550</td>
<td>1180</td>
<td>0.508</td>
</tr>
<tr>
<td>6</td>
<td>804,3</td>
<td>50</td>
<td>550</td>
<td>640</td>
<td>0.937</td>
</tr>
</tbody>
</table>

As \( \frac{x_1}{\text{tg} \theta} = \frac{600}{1.8} = 333 \text{ mm} < b = 400 \text{ mm} \) so compressed zone has triangle form and its area is:
\[ A_b = \frac{x_1^2}{2 \text{tg} \theta} = \frac{600^2}{2 \times 1.8} = 100 000 \text{ mm}^2. \]

Let’s check condition (154):
\[ R_s A_b - \sum A_i \sigma_{si} = 16 \cdot 100 000 + 26 280 = 1626 \cdot 10^3 N = 1626 kN < N = 2600 kN, \]
That means area of compressed zone is decreased.

Let’s increase value \( x_1 \) up to 660 mm and determine \( \sum A_i \sigma_{si} \) (see the table of the present example).
By \( x_1 > h \) and \( x_j / \tan \theta = 660/1.8 = 367 \text{ mm} < b = 400 \text{ mm} \) the compressed zone has a trapezoid form and its area is:

\[
A_h = \frac{x_j^2 - (x_j - h)^2}{2 \tan \theta} = \frac{660 - 367^2}{2 \cdot 1.8} = 121100 - 1000 = 120100 \text{ mm}^2.
\]

As \( R_{A0} - \sum A_{si} \sigma_{si} = 16 \cdot 120 100 + 648 080 = 2570 \cdot 10^3 \text{ N} = 2570 \text{ kN} \approx N = 2600 \text{ kN} \), 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:

\[
S_{bx} = \frac{x_j^2}{2 \tan \theta} (a_{s5} - \frac{x_j}{3}) + \frac{(x_j - h)^2}{2 \tan \theta} (-a_{s5} + \frac{x_j - h}{3} + h) = 121100 \left( \frac{550 - 660}{3} \right) + 1000 \left( \frac{550 + 660 - 600}{3} + 600 \right) = 40036000 \text{ mm}^3;
\]

\[
S_{by} = \frac{x_j^2}{2 \tan \theta} (a_{s5} - \frac{x_j}{3} \tan \theta) + \frac{(x_j - h)^2}{2 \tan \theta} (a_{s5} - \frac{x_j - h}{3 \tan \theta}) = 121100 \left( \frac{350 - 367}{3} \right) - 100 \left( \frac{350 - 660}{3} \right) = 27912000 \text{ mm}^3.
\]

Then \( M_{sx} = R_b S_{bx} - \sum A_{si} \sigma_{si} (a_{s5} - a_{s}) = 16 \cdot 40036000 - \left[ -238877(550 - 50) - 293570 \times (550 - 50) + 6284(550 - 300) - 114683(550 - 300) \right] = 933.9 \cdot 10^6 \text{ N} \cdot \text{mm} = 934 \text{ kN} \cdot \text{m};
\]

\[
M_{sy} = R_b S_{by} - \sum A_{si} \sigma_{si} (a_{s5} + a_{s}) = 16 \cdot 27912000 - \left[ -293570(350 - 50) - 114683(350 - 50) - 279896(350 - 50) \right] = 653 \cdot 10^6 \text{ N} \cdot \text{mm} = 653 \text{ kN} \cdot \text{m}.
\]

Moments of external forces relating to axes \( y \) and \( x \) are:

\[
M_{sx} = M_{sx} + N \left( \frac{h}{2} - 50 \right) = 250 \cdot 10^6 + 2600 \cdot 10^4 \left( \frac{600}{2} - 50 \right) = 900 \cdot 10^6 \text{ N} \cdot \text{mm};
\]

\[
M_{sy} = M_{sy} + N \left( \frac{b}{2} - 50 \right) = 200 \cdot 10^6 + 2600 \cdot 10^4 \left( \frac{400}{2} - 50 \right) = 590 \cdot 10^6 \text{ N} \cdot \text{mm}.
\]

As \( M_{sx} > M_{x1} \), \( M_{sy} > M_{y1} \) so section strength is provided.

**CACLULATION OF INCLINED SECTIONS**

**Example 41.** Given: a column of a multistory frame work with the section dimensions \( b = 400 \text{ mm}, h = 600 \text{ mm} \); \( a = a' = 50 \text{ mm} \); heavy-weight concrete \( \text{B25} \) \( (R_{bt} = 0.95 \text{ MPa} \text{ by } \gamma_b = 0.9) \); stirrups located along the column surfaces made of reinforcement \( \text{A-III}, \text{diameter 10 mm} \) \( (R_{sw} = 255 \text{ MPa}; A_{sw} = 157 \text{ mm}^2) \), spacing \( s = 400 \text{ mm} \); bending moments in the top and the bottom support sections are \( M_{sup} = 350 \text{ kN} \cdot \text{m}, M_{inf} = 250 \text{ kN} \cdot \text{m} \) and stretch the left and the right surface of the column; longitudinal force \( N = 572 \text{ kN} \); column length (distance between support sections) \( l = 2.8 \text{ m} \).

It is required the strength of inclined sections of the column as regards the shear force.

Calculation. \( h_0 = h - a = 600 - 50 = 550 \text{ mm} \). 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; that is equal to

\[
c_{max} = \frac{\phi_{b2}}{\phi_{b3}} h_0 = \frac{2}{0.6} = 1.833 < l = 2.8 \text{ m}.
\]
Let's determine coefficient $\phi$:

$$\phi_n = 0.1 \frac{N}{R_u b h_0} = 0.1 \frac{572,000}{0.95 \cdot 400 \cdot 550} = 0.27 < 0.5; \quad \phi_f = 0.$$ 

As $c = c_{max}$, $Q_b = Q_{b,min} = \phi_b (1 + \phi_f) R_u b h_0 = 0.6 (1 + 0.27) 0.95 \cdot 400 \cdot 550 = 159.2 \cdot 10^3 \text{ H} < Q = 214 \text{ kN}$ so the stirrups are required due to the calculation.

Value $q_{sw}$ is to be determined by formula (55):

$$q_{sw} = \frac{R_u A_{sw}}{s} = \frac{255 \cdot 157}{400} = 100.1 \text{ N/mm}.$$ 

Let’s check condition (57):

$$\frac{Q_{b,min}}{2h_0} = \frac{159.2 \cdot 10^3}{2 \cdot 550} = 144.7 \text{ N/m} > q_{sw} = 100.1 \text{ N-mm}$$

As condition (57) is not met so we determine value by the following formula

$$M_h q_{b,h} = 0.1 \cdot 100.1 \cdot 10^3.$$ 

So

$$Q_b = \frac{M_h}{c} = \frac{2 \cdot 550 \cdot 100.1}{2h_0} = 2 \cdot 550 \cdot 100.1 = 110.1 \cdot 10^3 \text{ N}.$$ 

$c_0$ is taken equal to $c_0 = 2h_0 = 2 \cdot 550 = 1100 \text{ mm}$, and $Q_{sw} = q_{sw} c_0 = 100.1 \cdot 1100 = 110.1 \cdot 10^3 \text{ N}.$

Let’s check condition (50):

$$Q_b + Q_{sw} = 110.1 \cdot 10^3 + 110.1 \cdot 10^3 = 220.2 \cdot 10^3 \text{ H} > Q = 214 \text{ kN}$$

That means the section strength as regards the shear force is provided.

### Центрально- и внешнецентрированные элементы

**CENTRALLY TENSILE ELEMENTS**

- (3.26). During calculation of eccentric tensile reinforced concrete elements the following condition must be met:

$$N \leq R_s A_{s, tot},$$

where $A_{s, tot}$ — section area of total longitudinal reinforcement.

**ECCENTRICALLY TENSILE 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 least 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 $e' \leq h_0 - a'$, — so the calculation is made due to the following conditions:

$$Ne' \leq R_s A_s (h_0 - a');$$

$$Ne \leq R_s A_s' (h_0 - a');$$
b) if longitudinal force $N$ is applied beyond the area between resultant forces in reinforcement $S$ and $S'$ (Draft 51, $\delta$), that means by $e' > h_0 - a'$, so the calculation is made due to the following condition

$$Ne \leq R_y b x (h_0 - 0.5x) + R_s A'_s (h_0 - a'), \quad (159)$$

at the same time the height of compressed zone $x$ is determined by formula

$$x = \frac{R_s A_s - R_s A'_s - N}{R_y b}. \quad (160)$$

**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 $S'$;

$\delta$ — 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 $\xi_R$ 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 $x = \xi_R h_0$, 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 $e' \leq h_0 - a'$ it’s determined section area of reinforcement $S$ and $S'$ by the following formulas:

$$A = \frac{Ne'}{R_s (h_0 - a')}; \quad (161)$$

$$A'_s = \frac{Ne}{R_s (h_0 - a')}; \quad (162)$$

b) by $e' > h_0 - a'$ it’s determined section area of stretched reinforcement $A_s$ by formula

$$A_s = \frac{\xi b h_0 R_s + N}{R_s} + A'_s \frac{R_s}{R_s}, \quad (163)$$

where $\xi$ is taken by Table 20 due to value

$$\alpha_m = \frac{Ne - R_s A'_s (h_0 - a')}{R_y b h_0^2}. \quad (164)$$

At the same time condition $\alpha_m \leq \alpha_R$ 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 $\xi$, determined due to Table 20 without considering compressed reinforcement, that means due to
value \( \alpha_n = \frac{N_{e}}{R_{b}h_{0}} \), is less than \( 2a'/h_{0} \). In that case section area of stretched reinforcement \( A_{s} \) is determined by the following formula

\[
A_{s} = \frac{N(e+\zeta h_{0})}{R_{b}\zeta h_{0}},
\]

where \( \zeta \) is determined by formula 20 according to value \( \alpha_n = \frac{N_{e}}{R_{b}h_{0}} \).

**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

\[
N\tilde{e}' \leq \Sigma\sigma_{i}S_{b} - R_{b}S_{b},
\]

where \( \tilde{e}' \) — 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;

\( \sigma_{si} \) — stress in the \( i \)-rod of longitudinal reinforcement.

Compressed zone height \( x \) and stresses \( \sigma_{si} \) 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

\[
M_{b} = \varphi_{k2}(1 + \varphi_{f} - \varphi_{n})R_{b}bh_{0},
\]

where \( \varphi_{n} = 0.2\frac{N}{R_{b}bh_{0}} \), but no more than 0.8;

value \( Q_{b,min} \) is taken equal to \( \varphi_{b3}(1 + \varphi_{f} - \varphi_{n})R_{b}bh_{0} \). Besides in all formulas of Items 3.29, 3.40 and 3.41 coefficient \( \varphi_{b4} \) is replaced by \( \varphi_{b4}(1 - \varphi_{n}) \).

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 \( b = 500 \text{ mm}, h = 200 \text{ mm}; a = a' = 40 \text{ mm} \); longitudinal reinforcement A-III \((R_s = R_{sc} = 365 \text{ MPa})\); its section area \( A_s = A' s = 982 \text{ mm}^2 (2 \bigcirc 25) \); heavy-weight concrete B25 \((R_b = 16 \text{ MPa by } \gamma_b = 1,1)\); longitudinal force \( N = 44 \text{ kN} \); maximum bending moment \( M = 43 \text{ kN} \cdot \text{m} \).

*It is required* to examine the strength of the normal section.

Calculation. \( h_0 = 200 - 40 = 160 \text{ mm} \);
\[
e_0 = \frac{M}{N} = \frac{43 \cdot 10^6}{44 \cdot 10^3} = 977 \text{ mm};
\]
\[
e' = e_0 + \frac{h}{2} - a' = 977 + \frac{200}{2} - 40 = 1037 \text{ mm};
\]
\[
e = e_0 - \frac{h}{2} + a = 977 - \frac{200}{2} + 40 = 917 \text{ mm}.
\]

As there is symmetrical reinforcement so the strength is examined due to condition (157):
\[
R_s A_s (h_0 - a') = 365 \cdot 982(160 - 40) = 40 \cdot 10^6 N \cdot \text{mm} < Ne' = 44 \cdot 10^3 \cdot 1037 = 45.6 \cdot 10^6 N \cdot \text{mm}
\]
That means condition (157) is not met. As \( e' = 1037 \text{ mm} > h_0 - a' = 120 \text{ mm} \), and the height of compressed zone \( x \), determined by formula (160) without considering compressed reinforcement is
\[
x = \frac{R_s A_s - N}{R_b b} = \frac{365 \cdot 982 - 44 \cdot 10^3}{16 \cdot 500} = 40'' < 2a' = 2 \cdot 40 = 80 \text{ mm}
\]
according to the note to Item 3.78 we check the strength according to Formula (159), taking \( x = 40 \text{ mm} \) and \( A' s = 0 \):
\[
R_b b(x(h_0 - 0.5x) = 16 \cdot 500 \cdot 40(160 - 0.5 \cdot 40) = 40.6 \cdot 10^6 N \cdot \text{mm} > Ne = 44 \cdot 10^3 \cdot 917 = 40.4 \cdot 10^6 N \cdot \text{mm}
\]
that is the strength the normal section is provided.

**Example 43.** Given: a rectangular section with dimensions \( b = 1000 \text{ mm}, h = 200 \text{ mm}; a = a' = 35 \text{ mm} \); heavy-weight concrete B15 \((R_b = 7.7 \text{ MPa by } \gamma_b = 0.9)\); longitudinal reinforcement A-III \((R_s = R_{sc} = 365 \text{ MPa})\); reinforcement section area \( S' A' s = 1005 \text{ mm}^2 \); tension force \( N = 160 \text{ kN} \); bending moment \( M = 116 \text{ kN} \cdot \text{m} \).

*It is required* to determine section area of reinforcement \( S \).

Calculation. \( h_0 = 200 - 35 = 165 \text{ mm} \);
\[
e_0 = \frac{M}{N} = \frac{116 \cdot 10^6}{160 \cdot 10^3} = 725 \text{ mm};
\]
\[
e = e_0 - \frac{h}{2} + a = 725 - \frac{200}{2} + 35 = 660 \text{ mm};
\]
\[
e' = e_0 + \frac{h}{2} - a' = 725 + \frac{200}{2} - 35 = 790 \text{ mm}.
\]

As \( e' = 790 \text{ mm} > h_0 - a' = 165 - 35 = 130 \text{ mm} \) so let’s determine required section area of stretched reinforcement according to Item 3.796.

Let’s determine the following value:
\[
\alpha_m = \frac{Ne - R_s A_s (h_0 - a')}{R_b b h_0^2} = \frac{160 \cdot 10^3 \cdot 660 - 365 \cdot 1005(165 - 35)}{7.7 \cdot 1000 \cdot 165^2} = 0.276
\]
As \( 0 < \alpha_m < \alpha_0 = 0.44 \) (see Table 18) so value \( A_s \) is to be determined by formula (163). For that we find \( \zeta = 0.33 \) by \( \alpha_m = 0.276 \) according to Table 20.
Example 44. Given: a rectangular section with dimensions $b = 1000$ mm, $h = 200$ mm; $a = a' = 40$ mm; heavy-weight concrete B15 ($R_b = 7.7$ MPa by $γ_b = 0.9$); longitudinal reinforcement A-III ($R_s = R_{sw} = 365$ 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):

\[ A_s = A'_s = \frac{N e'}{R_s (h_0 - a')} = \frac{532 \cdot 10^3 \cdot 199}{365 \cdot 160} = 2417 \text{ mm}^2. \]

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 $ζ$ without considering compressed reinforcement. For that we determine value $α_m$:

\[ α_m = \frac{N e}{R_s bh_0^2} = \frac{532 \cdot 10^3 \cdot 79}{7.7 \cdot 1000 \cdot 160^2} = 0.213. \]

Due to Table 20 by $α_m = 0.213$ we find $ζ = 0.24$ and $ζ = 0.88$. As $ζ = 0.24 < \frac{2a'}{h_0} = \frac{2 \cdot 40}{160} = 0.5$, we determine value $A_s$ by formula (165):

\[ A_s = A'_s = \frac{N (e + ζ h_0)}{R_s ζ h_0^2} = \frac{532 \cdot 10^3 (79 + 0.88 \cdot 160)}{365 \cdot 0.88 \cdot 160} = 2275 \text{ mm}^2. \]

We take $A_s = A'_s = 2281 \text{ mm}^2$ (6 $∅ 28$).

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 ($R_b = 1.15$ MPa by $γ_b = 1.1$); 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), $φ_3 = 0$ and $φ_s = 0.2$,

\[ φ_s = 0.2 \frac{N}{R_s bh_0^2} = 0.2 \frac{44000}{1.15 \cdot 500 \cdot 160^2} = 0.096 < 0.8; \]

\[ M_b = φ_{b2} (1 - φ_3) R_b h_0^2 = 2 (1 - 0.096) 1.15 \cdot 500 \cdot 160^2 = 26.6 \cdot 10^4 \text{ N} \cdot \text{mm} \]

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:

\[ c = c_{max} = \frac{φ_{b2}}{φ_{b3}} h_0 = \frac{2}{0.6} = 160 = 533 \text{ mm} < l = 600 \text{ mm}. \]

Then
\[ Q_c = \frac{M_c}{c} = \frac{26.6 \cdot 10^6}{533} = 49.9 \cdot 10^3 \text{H} = Q_{b,\min} \]

As \( 2h_0 = 2 \cdot 160 = 320 \text{ mm} < c = 533 \text{ mm} \) so we take \( c_0 = 2h_0 = 320 \text{ mm} \).

Определим коэффициент \( \alpha \) :

\[ \alpha = \frac{Q - Q_{b,\min}}{Q_{b,\min}} = \frac{143 - 49.9}{49.9} = 1.866. \]

As \( \frac{c}{c_0} = \frac{533}{320} = 1.667 < \alpha = 1.866 < \frac{c}{h_0} = \frac{533}{160} = 3.33 \) so stirrups quantity is determined by formula (63):

\[ q_{sw} = \frac{(Q - Q_{b,\min})^2}{M_b} = \frac{(143 - 49.9)^2}{26.6} = 325.9 \text{ kN/m}. \]

Maximum allowable stirrups quantity according to Item 3.30 is:

\[ s_{\text{max}} = \frac{\varphi_{sw}(1 - \varphi_{sw})R_{sw}b_h}{Q} = \frac{1.5(1 - 0.096)(1.5 \cdot 500 \cdot 160)^2}{143 \cdot 10^3} = 139.6 \text{ mm}. \]

Besides in compliance with Item 5.58 the stirrups quantity must be no more than \( 2h = 2 \cdot 200 = 400 \text{ mm} \).

We take stirrups quantity \( s = 100 \text{ mm} < s_{\text{max}} \), so

\[ A_{sw} = \frac{q_{sw}s}{R_{sw}} = \frac{325.9 \cdot 100}{290} = 112.4 \text{ mm}^2. \]

We take two stirrups with diameter 10 mm (\( A_{sw} = 157 \text{ mm}^2 \)).

**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:

\[ T \leq 0.1 R_{b,\min} b^2 h, \]  

where \( b, h \) — are the larger and the less dimensions of the element surfaces.

At the same time value \( R_{b,\min} \) for concrete more than \( \text{B}30 \) 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

\[ T + M b \leq R_{b,\min} \left( \frac{b}{c_1} + q_{sw}s \delta \right) (h_0 - 0.5 \delta), \]  

\[ 169 \]
At the same time value $R_s A_s$ is taken no more than $2q_{sw1}b + \frac{M}{h_0 - 0.5x_1}$, and value $q_{sw1}$ — no more than
\[
\frac{1.5}{b} \left( R_s A_s - \frac{M}{h_0 - 0.5x_1} \right).
\]
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 $T$;

$A_s$ — 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:

\[
q_{sw1} = \frac{R_s A_{sw1}}{s_1},
\]

where $A_{sw1}$ — is section area of one cross rod;

$s_1$ — distance between cross rods;

\[
\delta_1 = \frac{b}{2h + b}.
\]

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

\[
x_1 = \frac{R_s A_s - R_s A'_s}{R_s b},
\]

where $A'_s$ — 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 $\xi_R$ — 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 $A_s$ 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 $R_s A_s$ 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; *b* — 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

\[ \frac{R_i A_{i1} (h_0 - 0.5 x_i)}{M_{\text{max}}} \geq M_{\text{max}} + \frac{(T - 0.5 Q b)^2}{4 \delta q_{sw1} b (h_0 - 0.5 x_i)}, \]  

where \( M_{\text{max}} \) — 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 \( q_{sw1} b (h_0 - 0.5 x_1) \) is taken no more than \( \frac{0.6 T}{\sqrt{\delta_1}} \);

**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

6) for elements loaded only by distributed load \( q \) if in the span section with maximum bending moment \( M_{\text{max}} \) there is a bending moment \( T_0 \), — due to the following condition

\[ \frac{R_i A_{i1} (h_0 - 0.5 x_i)}{M_{\text{max}}} \geq M_{\text{max}} + \frac{T_0^2}{4 \delta q_{sw1} b (h_0 - 0.5 x_i) - 2 t^2 / q}, \]  

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 \( M_{\text{max}} = 0 \).

If on the considered parts the following condition is not met

\[ T < 0.5 Q b, \]  

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

\[ q_{sw1} b (h_0 - 0.5 x_i) \geq \frac{T}{2 \sqrt{2 \delta_1}}. \]  

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:

\[ c_1 = 2 b \frac{R_i A_{i1} (h_0 - 0.5 x_i) - M_{\text{max}}}{T - 0.5 Q b}, \]  

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

\[ T + 0.5 Q b \leq \left( \frac{R_i A_{i2}}{c_2} + q_{sw2} c_2 \delta_2 \right) \left( b - 2 a_2 \right), \]  

At the same time \( R_i A_{i2} \) value is taken no more than \( 2 q_{sw2} h \) and value \( q_{sw2} \) — no more than \( 1.5 \frac{R_i A_{i2}}{h} \).

In condition (178):
$A_{s2}$ — section area of all tensile longitudinal rods located at the surface with 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

$$c_{2} = 2h \frac{R_{a}s_{2}(b-2a_{2})}{T+0.5Qb}$$  \hspace{1cm} (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 $T$;

$$q_{sw2} = \frac{R_{sw}A_{sw2}}{s_{2}};$$  \hspace{1cm} (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 $h$;

$$\delta_{2} = \frac{h}{2b+h};$$  \hspace{1cm} (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, $\delta$).

In case if condition (175) is met calculation of the spatial section due to the 2$^{nd}$ 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 2$^{nd}$ scheme density of stirrups $\frac{A_{sw2}}{s_{2}}$ can be determined by the following formulas:

by $\varphi_{t} = \frac{T+0.5Qb}{R_{a}s_{2}(b-2a_{2})\sqrt{28_{2}}} \leq 1$

$$\frac{A_{sw2}}{s_{2}} = 0.5\frac{R_{x}s_{2}}{R_{sw}h}\varphi_{t};$$  \hspace{1cm} (182)

by $1.75 \geq \varphi_{t} > 1$

$$\frac{A_{sw2}}{s_{2}} = 0.5\frac{R_{x}s_{2}}{R_{sw}h}\varphi_{t}^{2};$$  \hspace{1cm} (183)

where $T, Q$ — is maximum value of corresponding torsion moment and shear force on the part under review.

By $\varphi_{t} > 1675$ it is necessary to increase the section area of reinforcement $A_{s2}$ or dimension of the section $b$ so that condition $\varphi_{t} \leq 1.75$ 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

\[
T \leq 0.1 R_s \Sigma b_i^2 \cdot h_i, \quad (184)
\]

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

\[
T + M \frac{b_j'}{c_1} \leq R_s A_{s1} \frac{b_j'}{c_1} (h_0 - 0.5 x_1) + q_{sw} b_j (h_{0w} - 0.5 x_1) \quad (185)
\]

at the same time value \(R_s A_{s1}\) is taken no more than \(2 q_{sw} b_j + \frac{M}{h_0 - 0.5 x_1}\).

In condition (185):

- \(b_j', b_j\) — 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

\[
c_1 = 2h + b + (b_j' - b) + (b_j' - b) = 2h + 2b_j + b_j' - 2b.
\]

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);

\[
q_{sw} = \frac{R_s A_{sw1}}{s_1}, \quad (186)
\]

- \(A_{sw1}, s_1\) — 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

$$T + 0.5Qb_{f,min} \leq R_sA_{s2} \frac{h}{c_2}(b_0 - 0.5x_2) + q_{sw2}h(b_{0w} - 0.5x_2)$$

(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:

$$c_2 = 2b_{f,min} + h + 2b_{0w},$$

where $b_{0w}$ — 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;

$$q_{sw2} = \frac{R_swA_{sw2}}{s_2};$$

(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
  $$ T \leq 0.08 R_s \pi (r_2^3 - r_1^3), $$
  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
  $$ T + M \frac{b}{c} \leq M_u \frac{b}{c} + q_{sr} r_c \beta, $$
  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
  $$ b = 2 \sqrt{r_2^3 - (r_1 \cos \pi \xi_{cir})^3}, $$
  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**

- $\xi_{cir}$ - 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);
- $q_{sr} = \frac{R_s A_{sr}}{s}$; (192)
  $A_{sr,s}$ – section area of the rod of spiral (ring) reinforcement and spacing of the spiral lapping (rings spacing);
- $\beta$ - coefficient determined by formula
  $$ \beta = 1 - \frac{b}{2\pi \xi_{cir}} \times \left[ \frac{\sin \pi \xi_{cir}}{\pi (1 - \xi_{cir})} + \cos \pi \xi_{cir} \right], $$
  or according to Draft 60.

**Draft 60. Diagram for determination of coefficient $\beta$ 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
  $$ \frac{1.5 R_s A_{s,tot}}{2\pi \xi_{cir}} \left( 1 - \frac{M}{M_u} \right). $$
- 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 \( c_{\text{max}} = 2\pi r_2 \left(1 - \xi_{\text{me}} \right) \).

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 \( M = M_{\text{max}} \). In the last case the most disadvantageous value \( c \) is:

\[
c = 2b \left( M_u - M_{\text{max}} \right) / T - Qb / 2.
\]

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:

\[
c = 2b \left( M_u - M \right) / T.
\]

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 \( q = 154,4 \text{ kN/m} \) and distributed torsion moments \( t = 34,28 \text{ kN·m/m} \); 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, \( \delta \); 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, \( \sigma \); heavy-weight concrete \( \mathbf{B} \mathbf{2}5 \); longitudinal and cross reinforcement \( \mathbf{А} \mathbf{-III} \left( R_s = R_{sc} = 365 \text{ MPa}; R_{sw} = 290 \text{ MPa} \right) \).

**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 \( R_b = 13 \text{ MPa} \) (that is by \( \gamma_{b2} = 0,9 \));

\[
0.1 R_b \sum b_i^2 h_i = 0.1 \cdot 13 \left(320^2 \cdot 800 + 155^2 \cdot 250 \right) = 114,3 \cdot 10^6 \text{Н·мм} > T = 84 \text{ кН·м},
\]

that means condition (184) is met.

The calculation of spatial sections is made as for a rectangle sections with dimensions \( b = 300 \text{ mm} \) и \( h = 800 \text{ mm} \), as bottom surface of the collar-beam and the flange form an angle.

As for the support section \( 0,5Qb = 0,5 \cdot 460 \cdot 0,3 = 69 \text{ kN·м} < T = 84 \text{ kN·м} \) 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 \( \delta_2 \) и \( \varphi_i \):

\[
\delta_2 = \frac{h}{2b + h} = \frac{800}{2 \cdot 300 + 800} = 0,571;
\]
\[
\varphi_i = \frac{T + 0.5Qb}{R_i A_{st}^2 (b - 2a_i) \sqrt{2\delta}} = \frac{84 \cdot 10^6 + 0.5 \cdot 460 \cdot 10^3 \cdot 300}{365 \cdot 2304 (300 - 2 \cdot 50) \sqrt{2 \cdot 0.571}} = 0.851,
\]
where \(A_{st} = 1609 + 314 + 380 = 2304 \text{ mm}^2\) (2\(\varnothing\)32 + \(\varnothing\)20 + \(\varnothing\)22).

As \(\varphi_i < 1\) so stirrups quantity is to be determined by formula (182):

\[
\frac{A_{sw,2}}{s_2} = 0.5 \frac{R_i A_{st}}{R_{sw} h} \varphi_i = 0.5 \frac{365 \cdot 2304}{290 \cdot 800} 0.851 = 1.54 \text{ mm}.
\]

Taking the spacing of vertical stirrups \(s_2 = 100 \text{ mm}\), we find the area of one stirrup:

\(A_{sw,2} = 1.54 \cdot 100 = 154 \text{ mm}^2\).

We take stirrups with diameter 14 mm (\(A_{sw} = 154 \text{ mm}^2\)).

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 3.85 (the 1st scheme).

Due to Draft 61, \(a\) we find \(A_{sl} = 3217 \text{ mm}^2\) (4\(\varnothing\)32) and \(A_{sl}' = 1388 \text{ mm}^2\) (2\(\varnothing\)20 + 2\(\varnothing\)22), \(a' = 68 \text{ mm}\).

By formula (172) we determine the height of compressed zone \(x_f\) taking \(R_b = 16 \text{ MPa}\) (that is by \(\gamma_{b,2} = 1.1\) as wind load is taken into account):

\[
x_f = \frac{R_i A_{st} A_{sl} (2a - x_f)}{R_b b} = 139 \text{ mm} > 2a' = 2 \cdot 68 = 136 \text{ mm}.
\]

Spacing and diameter of horizontal cross rods of the support part we take the same as for vertical stirrups, that means \(s_1=100 \text{ mm}\), \(A_{sw,1} = 154 \text{ mm}^2\) so

\[
q_{sw,1} = \frac{R_{sw} A_{sw,1}}{s_1} = \frac{290 \cdot 154}{100} = 446.6 \text{ N/m};
\]

\[
\delta_1 = \frac{b}{2h + b} = \frac{300}{2 \cdot 800 + 300} = 0.158;
\]

\[
h_0 = 800 - 80 = 720 \text{ mm}.
\]

Let’s check the equation \(q_{sw,1}b(h_0 - 0.5x_f) = 446.6 \cdot 300(720 - 0.5 \cdot 139) = 87.2 \cdot 10^6 \text{ N-mm} < \frac{0.6T}{\sqrt{\delta_1}} = \frac{0.6 \cdot 84 \cdot 10^6}{\sqrt{0.158}} = 126.8 \cdot 10^6 \text{ N-mm} \) So \(q_{sw,1}\) is not to be changed.

Let’s check condition (173):

\[
M_{\text{max}} = \frac{(T - 0.5Qb)^2}{4\delta_1 q_{sw,1} b(h_0 - 0.5x_f)} = 490 \cdot 10^6 + \frac{(84 \cdot 10^6 - 0.5 \cdot 460 \cdot 10^3 \cdot 300)^2}{4 \cdot 0.158 \cdot 87.2 \cdot 10^6}
\]

\(494 \cdot 10^6 \text{ N-mm} < R_s A_s (h_0 - 0.5x_f) = 365 \cdot 3217(720 - 0.5 \cdot 139) = 763.8 \cdot 10^6 \text{ N-mm} \)

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:

\[
q_{sw,1}b(h_0 - 0.5x_f) = 446.6 \cdot 300(720 - 0.5 \cdot 139) = 87.2 \cdot 10^6 \text{ N-mm} > \frac{T}{2\sqrt{2\delta}} = \frac{84}{2\sqrt{2 \cdot 0.158}} = 74.7 \text{ kN-m}
\]

That means there is installed enough horizontal cross reinforcement on the support part.

As it’s shown on Draft 61, \(\delta, \phi\), 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 \(\varnothing\)32 are broken that’s why due to Draft 61, \(a\), we have \(A_{sw}' = 1609 \text{ mm}^2\) (2\(\varnothing\)32); \(a' = 62 \text{ mm}\); \(A_{sl} = 1388 \text{ mm}^2\) (2\(\varnothing\)20 + 2\(\varnothing\)22); \(a = 68 \text{ mm}\).
Let’s determine the height of compressed zone $x_1$, taking $R_b = 13\,\text{MPa}$ (that is by $\gamma_b = 0.9$, as wind load is not taken into account):

$$x_1 = \frac{RA_{sl}}{RB} = \frac{365(1388 - 1609)}{13 \cdot 300} < 0.$$

We take $x_1 = 2a'$, so $h_0 - 0.5x_1 = h - a - a' = 800 - 68 - 62 = 670\,\text{mm}$.

Horizontal cross rods in the middle part of the span are taken with diameter 14 mm ($A_{sw1} = 154\,\text{mm}^2$) and spacing $s_1 = 200\,\text{mm}$, so

$$q_{sw1} = \frac{RA_{sw1}}{s_1} = \frac{290 \cdot 154}{200} = 223.3\,\text{N/mm}.$$

From Draft 61, $\delta$, $\vartheta$ we have:

$$T_0 = \frac{2.71 - 2.45}{2.45} \cdot 84 = 8.9\,\text{kN \cdot m};$$

$$M_{\text{max}} = 321\,\text{kN \cdot m}.$$

Let’s check condition (174):

$$M_{\text{max}} + \frac{T_0^2}{4\delta q_{sw1}b(h_0 - 0.5x_1)} = \frac{321 \cdot 10^6 + (8.9 \cdot 10^6)^2}{4 \cdot 0.158 \cdot 223.3 \cdot 300 \cdot 670 - 2 \cdot 34.28^2 \cdot 10^6} = 326.9 \cdot 10^6\,\text{N \cdot mm} < R_Ax_1(h_0 - 0.5x_1) = 365 \cdot 1388 - 670 = 339.4 \cdot 10^6\,\text{N \cdot mm}.$$

That means that according to the strength conditions there is installed enough bottom longitudinal reinforcement.

Let’s determine distance $l_0$ from the zero point of the diagram $T$ possible for spacing of horizontal cross rods 200 mm using condition (176). Taking $T = tl_0$ we have $q_{sw1}b(h_0 - 0.5x_1) = \frac{tl_0}{2\sqrt{2\delta}}$, so

$$l_0 = \frac{2\sqrt{2\delta} q_{sw1}b(h_0 - 0.5x_1)}{t} = \frac{2\sqrt{2 \cdot 0.158 \cdot 223.3 \cdot 0.67}}{34.28} = 1.47\,\text{m}.$$

Therefore spacing of horizontal rods 100 mm on support parts can be 2.45-1.47 ≈ 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, $\delta$; heavy-weight concrete $V_25$ ($R_b = 13\,\text{MPa}$ by $\gamma_b = 0.9$); longitudinal and cross reinforcement A-III ($R_s = R_{sc} = 365\,\text{MPa}$; $R_{sw} = 290\,\text{MPa}$).

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):

$$0.1R_s \sum b_i^2h_i = 0.1 \cdot 13(200^2 \cdot 400 + 350^2 \cdot 400) = 84.5 \cdot 10^6\,\text{H \cdot MM} > T = 40\,\text{kN \cdot m}.$$

**Черт. 62.** For the calculation example 47

Due to Draft 62, $a$ we have $h_0 = 800 - 50 = 750\,\text{mm}$.

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 \(h_s = 400 - 50 = 350\text{ mm}\)).
The force per a unit length of the beam in vertical stirrups located at the right surface caused by tearing force is to be determined by distributing the tearing force on two branches of stirrups and taking the width of the support platform of force \(F_b = 300\text{ mm}\), then
\[ a = 2h_s + b = 2 \cdot 350 + 300 = 1000\text{ mm} = 1\text{ m}, \]
that means
\[ q_{sw} = \frac{F/2}{a} = \frac{149.3/2}{1} = 74.6\text{kN/m} = 74.6\text{kN/mm}. \]

So considered by the calculation of the spatial section value \(q_{sw}\) by \(A_{sw} = 154\text{ mm}^2\) (1∅14) and \(s_2 = 100\text{ mm}\) (see Draft 62, \(a\)) will be:
\[ q_{sw} = \frac{R_{sw} A_{sw}}{s_2} - q_{sw} = \frac{290 \cdot 154}{100} - 74.6 = 372\text{N/mm} \]

Due to Draft 58, \(b\) and 62, \(a\), we take \(b_{f,\text{min}} = 200\text{ mm}\), \(h = 800\text{ mm}\), \(b_{ov} = 0\), \(A_{s2} = 1071\text{ mm}^2\) (1∅32 + 1∅12 + 1∅14).

Then value \(c_2\) will be:
\[ c_2 = 2b_{f,\text{min}} + h + 2b_{ov} = 2 \cdot 200 + 800 = 1200\text{ mm} \]

Spatial section is located at the support of the beam. As \(c_2 < 1.94\text{ m}\) that means spatial section doesn’t go beyond the borders of a part with the zero values \(T\) so we live \(c_2 = 1.2\text{ m}\).

Design values \(Q\) and \(T\) are taken at the distance \(c_2/2\) from the support that means
\[ Q = Q_{\max} - q \frac{c_2}{2} = 297 - 5.7 \frac{1.2}{2} = 293.5\text{kH}; T = 40\text{kN-m} \]

As \(R_{s2} A_{s2} = 365 \cdot 1071 = 391 \cdot 10^3\text{ N} < 2q_{sw}h = 2 \cdot 372 \cdot 800 = 595 \cdot 10^3\text{ N}\) so we live \(R_{s2} A_{s2} = 391\text{ kN}\).

The height of compressed zone \(x_2\) is determined as for the rectangular section according to Item 3.20, taking for the present scheme \(h_0 = b_0 = 200 - 50 = 150\text{ mm}\) and \(b = h = 800\text{ mm}\) (compressed overhang of a flange is not taken into account).

As \(a' = 50\text{ mm}\) is a great part of \(h_0 = 150\text{ mm}\) so value \(x_2\) is determined without considering compressed zone:
\[ x_2 = \frac{R_{s2} A_{s2}}{R_{a2}} = \frac{391 \cdot 1000}{13 \cdot 800} = 37.6\text{ mm} < a' = 50\text{ mm}. \]

Let’s check condition (187) taking \(b_{sw} = b_o = 150\text{ mm}\):
\[ R_{s2} \frac{h}{c_2} (b_0 - 0.5x_2) + q_{sw} h (b_{sw} - 0.5x_2) = 391 \cdot 1000 \frac{800}{(150 - 0.5 \cdot 37.6) + 372 \cdot 800 \times (150 - 0.5 \cdot 37.6) = 73.2 \cdot 10^4\text{ N} \cdot \text{mm} > T + 0.5 Q_b f, \text{min} = 40 + 0.5 \cdot 293.5 \cdot 0.2 = 69.35\text{ kN-m}, \]

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 no 3.89. Let’s take \(b_f' = b = 200\text{ mm}; b_f = 350\text{ mm}; A_{s1} = 2526\text{ mm}^2\) (3∅32 + 1∅12); \(A_{s1} = 308\text{ mm}^2\) (2∅14); \(A_{sw1} = 154\text{ mm}^2\) (1∅14); \(s_1 = 200\text{ mm}\).

Let’s determine the projection length \(c_1\):
\[ c_1 = 2h + 2b_f + b_f' \cdot 2b = 2 \cdot 800 + 2 \cdot 350 + 200 = 2100\text{ mm}. \]

Spatial section is located on the part between the support and the first load near the point of application of this load. As \(c_1 > 1.94\text{ m}\) 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\( \cdot \)m; \( T = 40 \) kN\( \cdot \)m.

The height of compressed zone is determined as for the rectangular section:

\[
x_1 = \frac{R_s A_{s1} - R_{sw} A'_{s1}}{R_b b} = \frac{365(2526 - 308)}{13 \cdot 200} = 311 \text{ mm}
\]

At the same time \( x_1 = 311 \text{ mm} < \xi_R h_0 = 0.604 \cdot 750 = 453 \text{ mm} \) (where \( \xi_R \) — see Table. 18);

\[
q_{sw1} = \frac{R_{sw} A_{sw1}}{s_1} = \frac{290 \cdot 154}{200} = 223 \text{ N/mm}.
\]

As \( 2q_{sw1} b_f + \frac{M}{h_0 - 0.5x_1} = 2 \cdot 223 \cdot 350 \cdot \frac{285.4 \cdot 10^3}{750 - 0.5 \cdot 311} = 636.2 \cdot 10^3 \text{ N} < R_s A_{s1} = 365 \cdot 2526 = 922 \cdot 10^3 \text{ N} \), we take \( R_s A_{s1} = 636.9 \cdot 10^3 \text{ N} \).

Let’s check condition (185) taking \( h_{ow} = h_0 = 750 \) mm:

\[
R_s A_{s1} \left( \frac{h_0 - 0.5x_1}{c_1} \right) + q_{sw1} b_f \left( h_{ow} - 0.5x_1 \right) = 636.2 \cdot 10^3 \frac{200}{1940}(750 - 0.5 \cdot 311) + 223 \cdot 350 \times (750 - 0.5 \cdot 311) = \]

\[
= 85.4 \cdot 10^6 \text{ N} \cdot \text{mm} > T + M \frac{b_f'}{c_1} = 40 + 285.4 \frac{0.2}{1.94} = 69.4 \text{ kN} \cdot \text{m},
\]

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:

\[
N \leq \psi R_{b,loc} A_{loc1},
\]

where \( N \) — longitudinal compression force caused by local;

\( A_{loc1} \) — compression area (see Draft 63);

\( \psi \) — coefficient equal 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

\[
R_{b,loc} = \alpha \varphi_b R_b,
\]

here \( \alpha \varphi_b \geq 1.0 \);

\( \alpha = 1.0 \) for concrete grade less than B25;

\( \alpha = 13.5 R_{bt} / R_b \) 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, \( a, e, x, e, \) 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, \( \delta, \delta, \varphi \) independently on concrete class — 1,0;

\( R_{bt} \), \( R_b \) — 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, \( a \));

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, \( b \));

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, \( c \));

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, \( e \));

By local edge load on the element angle (Draft 63, \( d \)) design area \( A_{loc2} \) is equal to the compression area \( A_{loc1} \);

- \( \text{Note.} \) 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 \( A_{loc1} \) and \( A_{loc2} \) 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:

\[
N \leq R_{b,loc}^* A_{loc1},
\]

(196)

where \( A_{loc1} \) — compression area;

\( R_{b,loc}^* \) — prism strength of concrete to local compression determined by the following formula

\[
R_{b,loc}^* = R_b \varphi_b + \varphi \mu_{xy} R_{s,xy} \varphi_s
\]

(197)

here \( R_{s,xy} \), \( \varphi \), \( \mu_{xy} \) — are the same symbols like in Item 3.57;

\( \varphi_b = \sqrt[3]{\frac{A_{loc2}}{A_{loc1}}} \), but no more than 3.5;

\( \varphi_s \) — coefficient considering the influence of confinement reinforcement in the local compression zone; for diagrams of Drafts 63, \( b \), \( d \), \( e \) it is taken...
\[ \phi_s = 1.0, \] 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, \( a, b, c, e, u \) coefficient \( \phi_s \) is determined by formula

\[ \phi_s = 4.5 - 3.5 \frac{A_{loc1}}{A_{ef}}, \]

\( A_{ef} \) — 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, \( a - \partial, \chi, u \)) 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, \( a - e \)
  \[ h_d = \phi_d \left( \sqrt{\frac{N}{R_b}} - \sqrt{A_{loc1}} \right); \] (198)

- by loading schemes due to Draft 63, \( a, \partial, \chi, u \)
  \[ h_d = \phi_d \left( \frac{N}{R_b} - A_{loc1} \right). \] (199)

In formulas (198) and (199):
- \( \phi_d = 0.5 \) — by loading schemes due to Draft 63, \( a, e, u \);
- \( \phi_d = 0.75 \) — by loading schemes due to Draft 63, \( a, \chi \);
- \( \phi_d = 1.0 \) — by loading schemes due to Draft 63, \( \partial, \partial, \chi \).

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 \( \mu_{ky} \).

**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 \( \gamma_{b2} = 0.9 \)).

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, \( e \).

Due to Draft 64, we have \( c_1 = 200 \text{ mm} < b = 800 \text{ mm} \); \( a_j = 200 \cdot 2 + 300 = 700 \text{ mm} \);

\[ b_1 = 200 \cdot 2 + 200 = 600 \text{ mm}; \] \( A_{loc2} = a_1 b_1 = 700 \cdot 600 = 420000 \text{ mm}^2 \).

Compression area is \( A_{loc1} = 300 \cdot 200 = 60000 \text{ mm}^2 \). As concrete class is less than B25, \( \alpha = 1.0 \).

Coefficient \( \phi_b \) is:

\[ \phi_b = \sqrt[3]{\frac{A_{loc2}}{A_{loc1}}} = \sqrt[3]{\frac{420000}{60000}} = 1.9 < 2.5. \]
Let’s determine design resistance of concrete against compression by formula (195), taking $R_b$ considering $\gamma_b = 0.9$ (see Table 9) as for the concrete structure: $R_b = 6.7 \times 0.9 = 6.03$ MPa:

$$R_{b,loc} = \alpha \varphi_b R_b = 1 \times 0.9 \times 6.03 = 11.5 \text{ MPa}$$

(where $\alpha \varphi_b = 1 \times 1.9 = 1.9 > 1.0$).

Let’s check condition (194), taking $\psi = 1.0$ as by even distribution of the local load

$$\psi R_{bt,loc} = 1 \times 11.5 \times 60000 = 690000 \text{ N} = 690 \text{ kN} < N = 1000 \text{ kN},$$

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 $B_p - 1$, diameter 3 mm, dimensions of a cell $100 \times 100$ 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 $\varphi_b = 1.9 < 3.5$, so $\varphi_b = 1.9$ is inserted into the calculation.

Coefficient of confinement reinforcement by meshes $\mu_{xy}$ 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$^2$ (Ω3); $A_{ef} = l_x l_y = 300 \times 400 = 120000$ mm$^2 > A_{loc1} = 60000$ mm$^2$, so

$$\mu_{xy} = \frac{n_x A_{sx} l_x + n_y A_{sy} l_y}{A_{ef} s} = \frac{5 \times 7.1 \times 300 + 4 \times 7.1 \times 400}{120000 \times 100} = 0.00183,$$

By formulas (101) and (100) we determine $\psi$ and $\varphi$:

$$\psi = \frac{\mu_{xy} R_{s,xy}}{R_b + 10} = \frac{0.00183 \times 375}{6.7 + 10} = 0.041;$$

$$\varphi = \frac{1}{0.23 + \psi} = \frac{1}{0.23 + 0.041} = 3.69.$$

Coefficient $\varphi$ is:

$$\varphi_b = 4.5 - 3.5 \frac{A_{loc1}}{A_{ef}} = 4.5 - 3.5 \times 60000/120000 = 2.75.$$

Specified concrete strength $R_{b,loc}^*$ is determined by formula (197):

$$R_{b,loc}^* = R_b \varphi_b + \varphi \mu_{xy} R_{s,xy} \varphi_b = 6.7 \times 1.9 + 3.69 \times 0.00183 \times 375 \times 2.75 = 19.7 \text{ MPa}.$$

Let’s check condition (196):

$$R_{b,loc}^* A_{loc1} = 19.7 \times 60000 = 1182 \times 10^3 H > N = 1000 \text{ kN},$$

That means concrete strength is provided.

Let’s determine the least depth of meshes setting by formula (198), taking $\varphi_d = 0.5$:

$$h_d = \varphi_d \left( \frac{N}{R_b} - \sqrt{A_{loc1}} \right) = 0.5 \left( \frac{1000 \times 10^3}{6.7} - \sqrt{60000} \right) = 70.7 \text{ mm} < s = 100 \text{ mm}$$

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

$$F \leq \alpha R_{bt} u_m h_d,$$

where $F$ — pressing through force;

$\alpha$ — 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, \( b \))] so the right part of condition (200) is determined for actual punching pyramid and is multiplied by \( h_s/c \) (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 \( c = 0.4h_s \).

Draft 65. Scheme of the pressing pyramid by angle of inclination of its lateral surfaces to the horizontal line 
\( a \) — 45°; \( b \) — 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

\[ F \leq F_b + 0.8F_{sw}, \quad (201) \]

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

\[ F \left( 1 - \frac{h_s}{h_b} \right) \leq \Sigma R_{sw} A_{sw}, \quad (202) \]

where \( F \) — break force;

\( h_s \) — distance from level of transferring breaking force to the center of gravity of the section of the longitudinal reinforcement \( S \); prior to the force through the
монолитно связанные балки или консоли принимается, что нагрузка передается на уровне центра тяжести сжатой зоны элемента, вызывающего отрыв;

\[ \Sigma R_{sw} A_{sw} \] — сумма сил, передаваемых арматурой, устанавливаемой в дополнение к арматуре, предусмотренной расчетом для наклонных или пространственных сечений по поз. 3.31—3.39, 3.86, 3.87 и 3.90; эти арматуры расположены вдоль длины зоны разрыва, равной:

\[ a = 2h_s + b \]  \hspace{1cm} (203)

here \( b \) – ширина зоны разрыва силы.

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:

\[ F_1 = 2R_A A_{s1} \cos \frac{\beta}{2} \]  \hspace{1cm} (204)

б) 35 percent of resultant forces in all longitudinal stretched rods equal to:

\[ F_2 = 0.7R_A A_{s1} \cos \frac{\beta}{2} \]  \hspace{1cm} (205)

Required by these calculations stretched reinforcement must be located along the length

\[ s = h \cdot \tan \frac{3}{8} \beta \]

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 \( \Sigma R_{sw} A_{sw} \cos \theta \geq F_1 + F_2 \).

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;

\( \beta \) — re-entrant angle in the stretched zone of the element;

\( \Sigma A_{sw} \) — cross section of longitudinal reinforcement within the length \( s \);

\( \theta \) — angle of slope of cross rods onto the bisectrix of angle \( \beta \).

**Draft 66. Scheme for determination of the break zone length**

\( a \) — by adjoining of beams; \( b \) — 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 \([l_1 \leq 0.9h_0; \text{ (Draft 68)}]\) 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

\[ Q \leq 0.8R_0 b l_{sup} \sin^2 \theta (1 + 5a \mu_w), \]  \hspace{1cm} (207)

Where right part is taken no less than \( 3.5R_0 bh_0 \) and no less than \( 2.5R_0 bh_0 \).

In condition (207):
\( l_{sup} \) — the length of the area of the load supporting along the console overhanging length;

\( \theta \) — angle of slope of design compressed strip onto the horizontal line расчетной

\[
\sin^2 \theta = \frac{h_0^2}{h_c^2 + l_1^2}
\]

\( \mu_w = \frac{A_w}{b_s w} \) — 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 \( 5R_{b,loc}h_0 \).

**Draft 68. Design 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

\[
Q \frac{l_1}{h_0} \leq R_c A_s , \tag{208}
\]

where \( l_1, h_0 \) — 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:

\[
Q \frac{l_1}{h_0} - N_s \leq R_c A_s , \tag{209}
\]

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:

\[
N_s = \frac{M + Q l_{sup}}{h_{ob}} / 2 \tag{210}
\]
and taken no more than 1.4 \( kd_a R_{wf} \) + 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 \( \varnothing 42 R_{wf} = 180 \text{ MPa} \); 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):

\[
M, Q \quad \text{— bending moment and cross force in the normal section of the collar beam along the edge of the console; if moment } M \text{ stretches lower surface of the collar-beam so value } M \text{ is considered in formula (210) with the sign "minus";}
\]

\[
l_{sup} \quad \text{— actual length of the load support area along the console overhanging length;}
\]

\[
h_{wb} \quad \text{— working height of the collar beam.}
\]

**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 \( l_{sup,f} = 300 \text{ mm} \); console length \( b = 400 \text{ mm} \); height of the column and overhanging length of the column \( h = 700 \text{ mm} \), \( l_1 = 350 \text{ mm} \); heavy weight concrete of the column \( B_{25} (R_b = 13 \text{ MPa, } R_{bt} = 0.95 \text{ MPa by } \gamma_b = 0.9; \ E_b = 27 \cdot 10^3 \text{ MPa}); \) longitudinal reinforcement A-III \( (R_s = 365 \text{ MPa}); \) load on the console \( Q = 700 \text{ kN} \).

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 \text{ mm. As } 3.5 R_{sw} b h_0 = 3.5 \cdot 0.95 \cdot 400 \cdot 670 = 891.1 \cdot 10^3 \text{ N} = 891.1 \text{ kN} > Q = 700 \text{ kN and at the same time } 2.5 R_{sw} b h_0 = 2.5 \cdot 0.95 \cdot 400 \cdot 670 = 636.5 \text{ kH} < Q = 700 \text{ kH 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:

\[
l_{sup} = 2/3 \cdot l_{sup,f} = 2/3 \cdot 300 = 200 \text{ mm}
\]

Due to Item 5.77 we take the stirrups spacing equal to

\[
s_w = 150 \text{ mm} < \frac{h}{4} = \frac{700}{4} = 175 \text{ mm}
\]

By two-leg stirrups with diameter 10 mm we have \( A_{sw} = 157 \text{ mm}^2 \), so

\[
\mu_w = \frac{A_{sw}}{b s_w} = \frac{157}{400 \cdot 150} = 2.62 \cdot 10^{-3};
\]

\[
\alpha = \frac{E_s}{E_b} = \frac{20 \cdot 10^6}{27 \cdot 10^3} = 7.4;
\]

\[
\sin^2 \theta = \frac{h_0^2}{h_0^2 + l_1^2} = \frac{670^2}{670^2 + 350^2} = 0.786;
\]

\[
0.8 R_b b l_{sup} \sin^2 \theta (1 + 5 \alpha \mu_w) = 0.8 \cdot 13 \cdot 400 \cdot 200 \cdot 0.786 (1+5\cdot7.4 \cdot 2.62 \cdot 10^{-3}) = 717 \cdot 10^3 \text{ H} > Q = 700 \text{ kN},
\]

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:

\[ A_s = \frac{Q_l}{h_o R_s} = \frac{700 \cdot 10^3 \cdot 350}{670 \cdot 365} = 1002 \text{ mm}^2. \]

We take 3 Ø 22 \((A_s = 1140 \text{ mm}^2)\).

**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

\[ A_{an} = \frac{11 \sqrt{N_{an}^2 + \left(\frac{Q_{an}}{\lambda \delta}\right)^2}}{R_s}, \]

(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:

\[ N_{an} = \frac{M}{z} + \frac{N}{n_{an}}; \]

(212)

\(Q_{an}\) — shearing force on one anchors row equal to:

\[ Q_{an} = \frac{Q - 0.3N_{an}'}{n_{an}}; \]

(213)

\(N_{an}'\) — maximum compression force in one row of anchors determined by formula

\[ N_{an}' = \frac{M}{z} - \frac{N}{n_{an}}; \]

(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;

\(\lambda\) — coefficient determined for anchor rods with diameter 8 – 25 mm for heavy-weight and fine concrete \(V_{12.5} – V_{50}\) and light-weight concrete \(V_{12.5} – V_{30}\) by the following formula

\[ \lambda = \frac{4.75 \sqrt{R_b}}{(1 + 0.15A_{an})R_s \beta}, \]

(215)

but taken no more than 0.7; for heavy-weight and fine concrete of class more than \(V_{50}\) coefficient \(\lambda\) is taken as for class \(V_{50}\), and for light-weight concrete of class more than \(V_{30}\) — as for class \(V_{30}\). For heavy-weight concrete coefficient \(\lambda\) can be determined by Table. 28.

In formula (215):

\(R_b, R_s\) — in MPa;

By determination of \(R_b\) coefficient \(\gamma_{b2}\) (see Item 3.1) is taken equal to 1.0;
\(A_{an1}\) – section area of the anchor rod of the most stressed row наилболее, \(\text{cm}^2\);
\(\beta\) – 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 ....................... \(\rho_m/2300\) (\(\rho_m\) – average concrete density, \(\text{kg/m}^3\));
\(\delta\) – coefficient determined by formula
\[
\delta = \frac{1}{\sqrt{1+\omega}}, \quad (216)
\]
but taken no less than 0,15;
here \(\omega = 0,3 \frac{N_{an}}{Q_{an}}\) by \(N_{an} > 0\) (with pressure);
\[
\omega = 0,6 \frac{N}{Q} \quad \text{by } N_{an} \leq 0 \quad (\text{no pressure});
\]
if there are no tension forces in anchors so coefficient \(\delta\) 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 \(N_{an}^\prime\) 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 \(N_{an}^\prime = N\).

By location of the embedded element on the top surface of the detail (during concreting) coefficient \(\lambda\) is decreased by 20 percent and value \(N_{an}^\prime\) 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
\[
A_{an,inc} = \frac{Q - 0.3N_{an}^\prime}{R_s}, \quad (217)
\]
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 \(\delta = 1.0\) 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 \(Q - 0.9R_sA_{an,inc}\) on normal anchors. In that case \(\delta\) is determined by formula (216).
Anchor diameter | Coefficient $\lambda$ for calculation of normal anchors of embedded elements according to class of heavy-weight concrete and reinforcement
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Notes: 1. For concrete class В12.5 coefficient $\lambda$ is to be decreased by 0.02 in comparison with coefficient $\lambda$ for concrete class В15.
2. Values of coefficient $\lambda$ are given by $\gamma_{bi} = 1.00$.

- 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 $R_s$ 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. During 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 $n_uA_{an}/R_y$ (where $n_u$ — number of anchors in the present row).

Besides the thickness of plate $t$ of design embedded element must be examined due to the condition

$$t \geq 0.25 d_{an} \frac{R_s}{R_{sq}},$$

(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

$$N_{an}' \leq 0,$$

(219)