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VERIFICATION EXAMPLES
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TABLE OF CONTENTS

COMEIN

Calculation of the Load-bearing Capacity of the Column under Axial Compression

Objective: Check of the load-bearing capacity of the column under axial compression

Task: Check the capacity of masonry of the column under axial compression.

References: Nasonov S.B. Manual for Design and Analysis of Building Structures. M: ASV Publishing House, 2013, p. 181.

Initial data file:
Example 6.SAV
Example 6 mechnical.SAV
Example 6 fire.SAV

report – ComeIn 6 Nasonov.doc.
report – ComeIn 6 Nasonov-mechanical.doc.
report – ComeIn 6 Nasonov-fire.doc.

Compliance with the codes: SP 15.13330.2012

Initial data:

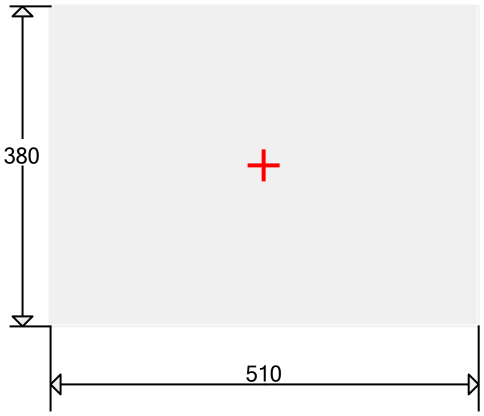
l_o = 4,5 m	Column height
h = 380 mm	Cross-sectional height
b = 510 mm	Cross-sectional width
N = 200 kN	Design load on the wall
Brick grade	M150
Mortar grade	M50

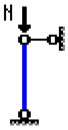
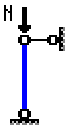
COMEIN initial data:

Importance factor $\gamma_n = 1$

Age of masonry - up to a year
Working life is 25 years
Stone/brick - Molded clay brick
Stone/brick grade - 150
Mortar - regular cement with mineral plasticizers
Mortar grade - 50

Structure:

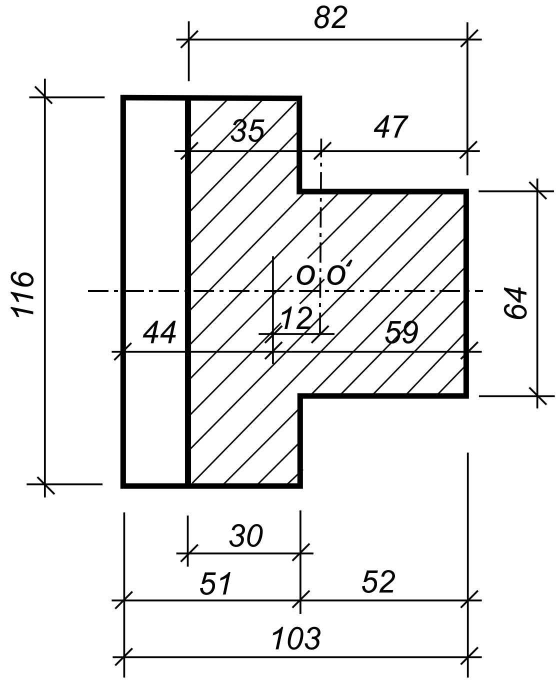
	Column height 4,5 m Longitudinal force 200 kN Factor for sustained load 1

Effective height in the XoY plane	**Effective height in the XoZ plane**
Bracing scheme  Effective height factor 1	Bracing scheme  Effective height factor 1

Comparison of solutions:

Check	Stability under axial compression
Manual	200/234,2 = 0,854
COMEIN	0,848
Deviation, %	0,7 %

Calculation of the T-section Wall Segment for Eccentric Compression (Eccentricity towards the Web)



Objective: Check of the calculation of eccentrically compressed columns.

Task: Verify the correctness of the stability analysis under eccentric compression.

References: Reference manual on design of masonry and reinforced masonry structures (supplement to SNiP II-22-81), 1989, p. 18-19.

Compliance with the codes: SNiP II-22-81, SP 15.13330.2012.

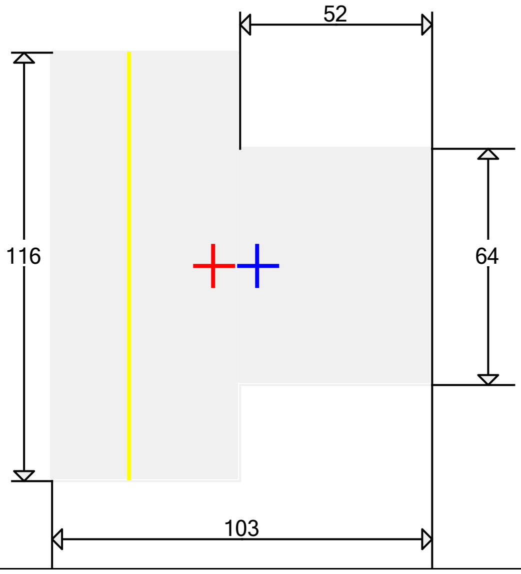
Initial data file:
Example 1.SAV;
ComeIn 1.doc — report

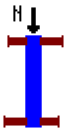
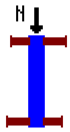
Initial data:

N = 850 kN	Longitudinal force;
M = 102 kN·m	Bending moment;
H = 5 m	Storey height;
Stone/brick	Molded clay brick, grade 100;
Mortar	Regular cement with mineral plasticizers, grade 50;
R = 1,5 MPa	Design resistance of masonry;
H_f = 0,22 m	Thickness of the precast reinforced concrete floor slab, embedded into wall masonry on the supports.

COMEIN initial data:
Importance factor $\gamma_n = 1$
Age of masonry - up to a year
Working life is 25 years
Stone/brick - Molded clay brick
Stone/brick grade - 100
Mortar - regular cement with mineral plasticizers
Mortar grade - 50

Structure:

	Eccentricity of longitudinal force - 12 cm along the Y axis Column height 4,78 m Longitudinal force 850 kN Factor for sustained load 1

Effective height in the XoY plane	Effective height in the XoZ plane
Bracing scheme  Precast floor slabs Distance between transverse rigid structures 5 m Effective height factor 0,9	Bracing scheme  Precast floor slabs Distance between transverse rigid structures 5 m Effective height factor 0,9

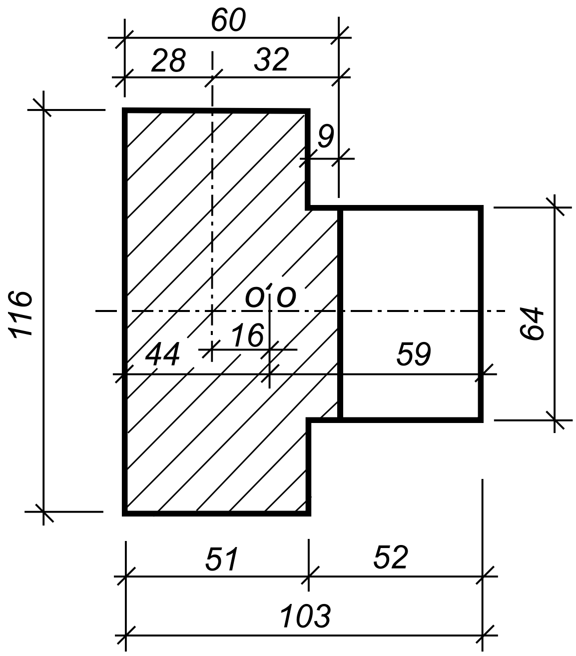
Comparison of solutions:

Check	stability in the eccentricity plane
Theory	850/1100 = 0,773
COMEIN	0,757
Deviation, %	2,07

Comments:

1. The eccentricity of the longitudinal force has to be specified in ComeIn instead of the value of the design moment. Its value is equal to $e_0 = \frac{M}{N} = \frac{102}{850} = 0{,}12$ m.
2. The column height in ComeIn is taken as the difference between the storey height and the floor slab thickness H - H_f = 5 – 0,22 = 4,78 m.
3. Distance between transverse rigid structures has to be specified in ComeIn. Since it is not determined in the problem, the value of 5 m is used.
4. Age of masonry and working life have to be specified in ComeIn. Since they are not determined in the problem, the following data are used: "up to a year" and 25 years respectively.
5. Deviation of the stability factor value in ComeIn from the result of the solution of the problem is caused by the fact that the requirements of Sec. 4.7 of SNiP II-22-81 (Sec. 7.7 of SP 15.13330.2012) for determining slenderness for the compressed part of the section for an alternating bending moment diagram are not taken into account when solving the problem.

Calculation of the T-section Wall Segment for Eccentric Compression (Eccentricity towards the Flange)



Objective: Check of the calculation of eccentrically compressed columns.

Task: Verify the correctness of the stability analysis under eccentric compression.

References: Reference manual on design of masonry and reinforced masonry structures (supplement to SNiP II-22-81), 1989, p. 19-20.

Initial data file:
Example 2.SAV;
ComeIn 2.doc — report

Initial data:

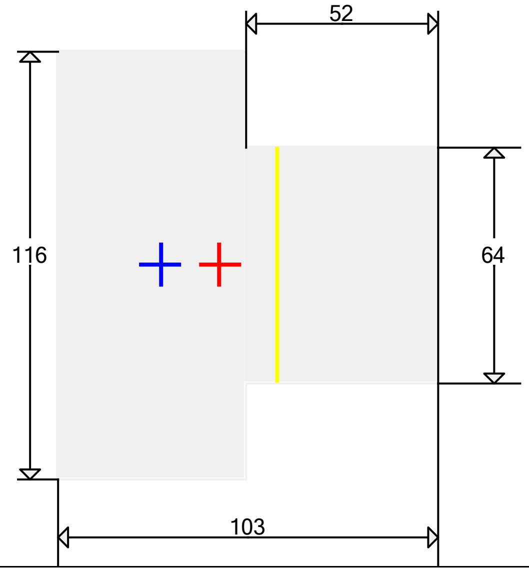
N = 850 kN	Longitudinal force;
e_0 = 0,16 m	Force eccentricity;
H = 5 m	Storey height;
Stone/brick	Molded clay brick, grade 100;
Mortar	Regular cement with mineral plasticizers, grade 50;
R = 1,5 MPa	Design resistance of masonry;
H_f = 0,22 m	Thickness of the precast reinforced concrete floor slab, embedded into wall masonry on the supports.

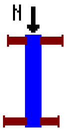
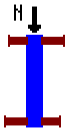
COMEIN initial data:

Importance factor $\gamma_n = 1$

Age of masonry - up to a year
Working life is 25 years
Stone/brick - Molded clay brick
Stone/brick grade - 100
Mortar - regular cement with mineral plasticizers
Mortar grade - 50

Structure:

	Eccentricity of longitudinal force - 16 cm along the Y axis Column height 4,78 m Longitudinal force 850 kN Factor for sustained load 1

Effective height in the XoY plane	**Effective height in the XoZ plane**
Bracing scheme  Precast floor slabs Distance between transverse rigid structures 5 m Effective height factor 0,9	Bracing scheme  Precast floor slabs Distance between transverse rigid structures 5 m Effective height factor 0,9

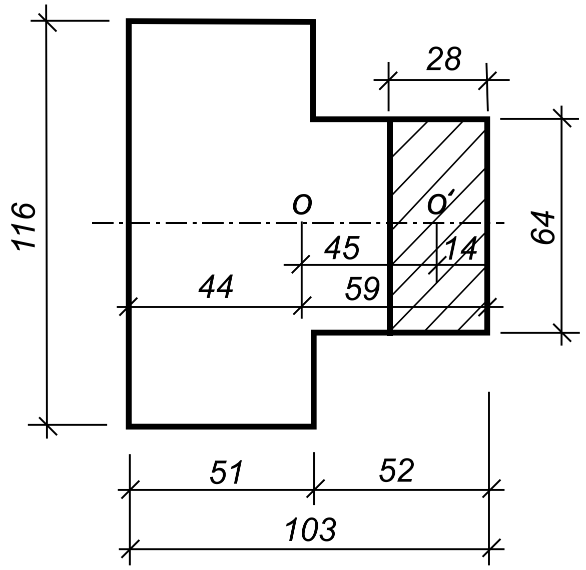
Comparison of solutions:

Check	Stability in the eccentricity plane under eccentric compression
Theory	850/1080 = 0,787
COMEIN	0,774
Deviation, %	1,652

Comments:

1. The column height in ComeIn is taken as the difference between the storey height and the floor slab thickness $H - H_f = 5 – 0{,}22 = 4{,}78$ m.
2. Distance between transverse rigid structures has to be specified in ComeIn. Since it is not determined in the problem, the value of 5 m is used.
3. Age of masonry and working life have to be specified in ComeIn. Since they are not determined in the problem, the following data are used: "up to a year" and 25 years respectively.
4. Deviation of the stability factor value in ComeIn from the result of the solution of the problem is caused by the fact that the requirements of Sec. 4.7 of SNiP II-22-81 (Sec. 7.7 of SP 15.13330.2012) for determining slenderness for the compressed part of the section for an alternating bending moment diagram are not taken into account when solving the problem.

Calculation of the T-section Wall Segment for Load-bearing Capacity and Crack Opening – Rigid Supports



Objective: Check of the calculation of eccentrically compressed columns.

Task: Verify the correctness of the analysis of stability and masonry seam opening under eccentric compression.

References: Reference manual on design of masonry and reinforced masonry structures (supplement to SNiP II-22-81), 1989, p. 20-21.

Initial data file:

1. when the longitudinal force is N = 326 kN:
   Example 3.1.SAV;
   ComeIn 3.1.doc — report
2. when the longitudinal force is N = 160 kN:
   Example 3.2.SAV;
   ComeIn 3.2.doc — report

Initial data:

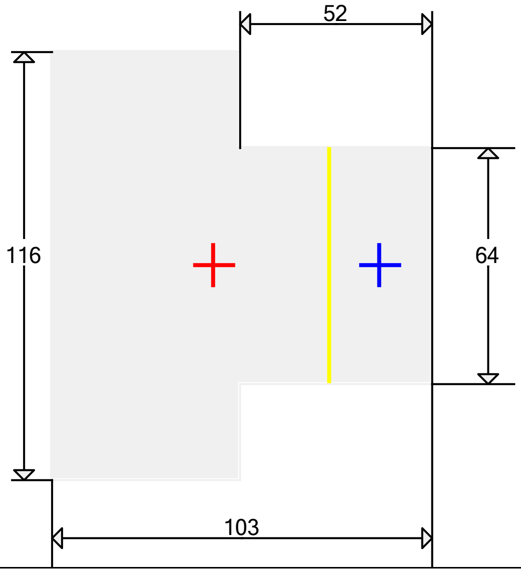
e_0 = -0,45 m	Force eccentricity
H = 5 m	Storey height
Stone/brick	Molded clay brick, grade 100
Mortar	Regular cement with mineral plasticizers, grade 50
R = 1,5 MPa	Design resistance of masonry
H_f = 0,22 m	Thickness of the precast reinforced concrete floor slab, embedded into wall masonry on the supports

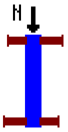
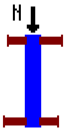
COMEIN initial data when the longitudinal force is N = 326 kN:

Importance factor γ_n = 1
Age of masonry - up to a year
Working life is 50 years
Stone/brick - Molded clay brick
Stone/brick grade - 100
Mortar - regular cement with mineral plasticizers

Mortar grade - 50

Structure:

 	Eccentricity of longitudinal force - 45 cm along the Y axis Column height 4,78 m Longitudinal force 326 kN Factor for sustained load 1

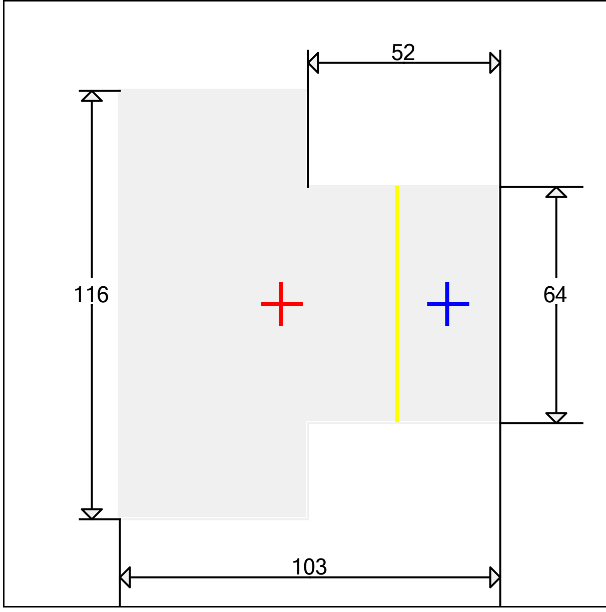
Effective height in the XoY plane	**Effective height in the XoZ plane**
Bracing scheme  Precast floor slabs Distance between transverse rigid structures 5 m Effective height factor 0,9	Bracing scheme  Precast floor slabs Distance between transverse rigid structures 5 m Effective height factor 0,9

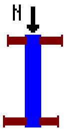
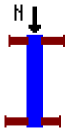
***COMEIN initial data when the longitudinal force is N = 160 kN*:**

Importance factor $\gamma_n = 1$

Age of masonry - up to a year
Working life is 50 years
Stone/brick - Molded clay brick
Stone/brick grade - 100
Mortar - regular cement with mineral plasticizers
Mortar grade - 50

Structure:

	Eccentricity of longitudinal force - 45 cm along the Y axis Column height 4,78 m Longitudinal force 160 kN Factor for sustained load 1

Effective height in the XoY plane	Effective height in the XoZ plane
Bracing scheme  Precast floor slabs Distance between transverse rigid structures 5 m Effective height factor 0,9	Bracing scheme  Precast floor slabs Distance between transverse rigid structures 5 m Effective height factor 0,9

Comparison of solutions:

Longitudinal force	*N* = 326 kN	*N* = 160 kN
Check	stability	masonry seam opening
Theory	326/326 = 1	160/160 = 1
COMEIN	0,958	0,999
Deviation, %	0,398	0,1

Comments:

1. Specific value of the applied longitudinal force is not determined in the problem, therefore ComeIn uses the calculated values of the load-bearing capacity N = 326 kN and N = 160 kN for stability and masonry seam opening respectively.
2. The column height in ComeIn is taken as the difference between the storey height and the floor slab thickness $H - H_f$ = 5 – 0,22 = 4,78 m.
3. Distance between transverse rigid structures has to be specified in ComeIn. Since it is not determined in the problem, the value of 5 m is used.
4. Age of masonry has to be specified in ComeIn. Since it is not determined in the problem, the value of "up to a year" is used.

Calculation of the Brick Column with Mesh Reinforcement for the Load-bearing Capacity

Objective: Check of the calculation of eccentrically compressed reinforced columns.

Task: It is necessary to verify the correctness of the analysis of stability in the eccentricity plane under eccentric compression and out of the eccentricity plane under axial compression.

References: Reference manual on design of masonry and reinforced masonry structures (supplement to SNiP II-22-81), 1989, p. 33-34.

Initial data file:
Example 8.SAV;
ComeIn 8.doc — report

Initial data:

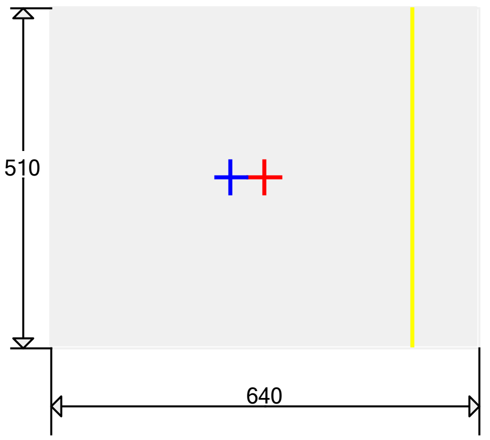
$b \times h$ = 0,51×0,64 m	Column dimensions in plan
l_0 = 3 m	Effective column height
N = 800 kN	Design longitudinal force
e_0 = 5 cm	Force eccentricity
Stone/brick	Molded clay brick, grade 100
Mortar	Regular cement with mineral plasticizers, grade 75

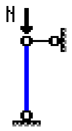
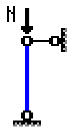
Selection of reinforcement is performed in the example given in the Manual, therefore the results of the selection are used for the verification of ComeIn.

Class of reinforcement Bp-I
Rebar diameter 4 mm
Spacing between meshes 2 masonry courses
Spacing dimensions in plan 3,2×3,2 cm

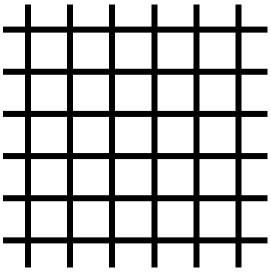
COMEIN initial data:
Importance factor γ_n = 1
Age of masonry - up to a year
Working life is 25 years
Stone/brick - Molded clay brick
Stone/brick grade - 100
Mortar - regular cement with mineral plasticizers
Mortar grade - 75

Structure:

	Eccentricity of longitudinal force - 50 mm along the Y axis Column height 3 m Longitudinal force 81,549 T Factor for sustained load 1

Effective height in the XoY plane	**Effective height in the XoZ plane**
Bracing scheme Effective height factor 1	Bracing scheme Effective height factor 1

Reinforcement:

Rectangular meshes 	Class of reinforcement Bp-I Rebar diameter 4 mm Rebar spacing in meshes 40 mm Number of masonry courses between meshes 2

Comparison of solutions:

Check	stability in the eccentricity plane under eccentric compression	stability out of the eccentricity plane under axial compression
Theory	800/828 = 0,966	800/1060 = 0,755
COMEIN	0,902	0,776
Deviation, %	6,62	1,44

Comments:

1. In order to ensure the value of the percentage of reinforcement μ, equal to the value obtained in the problem (μ = 0,4 %) the spacing dimension in plan is taken as 40 mm.
2. Age of masonry and working life have to be specified in ComeIn. Since they are not determined in the problem, the following data are used: "up to a year" and 50 years respectively.

3. The column height has to be specified in ComeIn. Since the effective column height determined in the problem is 3 m, this value is used for the column height in the pinned connection model, for which the effective height factor is equal to 1.
4. Deviation from the theory of the value of the position stability factor in the eccentricity plane under eccentric compression of 6,62 % is caused by the difference between the characteristic and design reinforcement strength R_{sn} and R_s. The following values $R_{sn} \cdot \gamma_{cs} = 405 \cdot 0{,}6 = 243$ MPa and $R_s \cdot \gamma_{cs} = 365 \cdot 0{,}6 = 219$ MPa in accordance with SNiP 2.03.01-84* are used in the problem; and the following values $R_{sn} \cdot \gamma_{cs} = 490 \cdot 0{,}6 = 294$ MPa and $R_s \cdot \gamma_{cs} = 410 \cdot 0{,}6 = 246$ MPa in accordance with the modification No. 2 of SNiP 2.03.01-84* (dated 12.11.1991) are used in ComeIn.

Calculation of the External Load-bearing Wall for Eccentric Compression

Objective: Check of the load-bearing capacity of an external wall.

Task: Check the capacity of masonry for eccentric compression.

References: Nasonov S.B. Manual for Design and Analysis of Building Structures. M: ASV Publishing House, 2013, p. 183-185.

Initial data file:
Example 7.SAV
report – ComeIn 7 Nasonov.doc.

Compliance with the codes: SP 15.13330.2012

Initial data:

l_o = 3,0 m	Wall height
h = 380 mm	Wall width
b = 1000 mm	Length of a wall section without openings
N = 350 kN	Design load on the wall
e_o = 0,03 m	Load eccentricity
Brick grade	M150
Mortar grade	M50

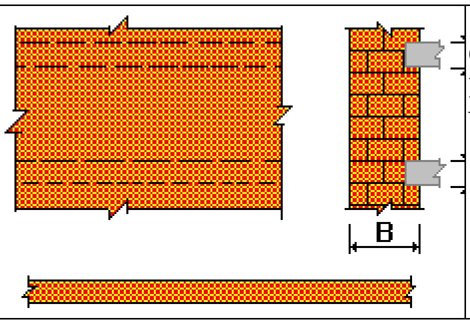
COMEIN initial data:

Importance factor γ_n = 1

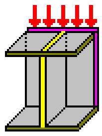
Importance factor (serviceability limit state) = 1

Age of masonry - up to a year
Working life is 25 years
Stone/brick - Molded clay brick
Stone/brick grade - 150
Mortar - regular cement with mineral plasticizers
Mortar grade - 50
Specific weight of masonry 1,8 T/m^3

Structure:

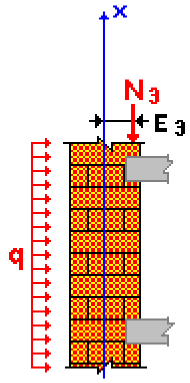
	Clear storey height H = 3 m Floor slab thickness t = 0,2 m Wall thickness B = 0,38 m

Effective height



Precast floor slabs.
Distance between transverse rigid structures 6 m.
Effective height factor 0,9.

Loads along the wall length

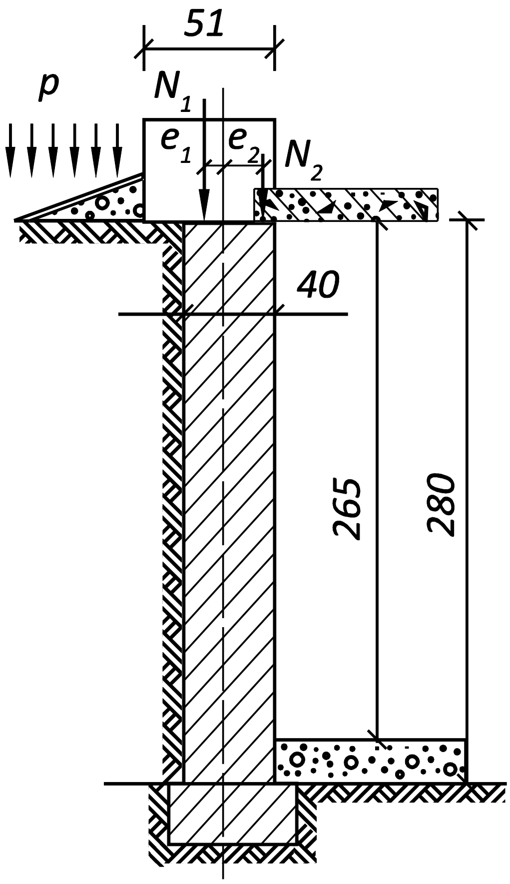


Wind load q = 0 T/m^2
Loads from the floor above the wall
N_3 = 350 kN/m
E_3 = 0,03 m
Factor for sustained load 1

Comparison of solutions:

Check	For eccentric compression in the upper section (under the floor slab)
Source	350/563,18 = 0,621
COMEIN	0,614
Deviation, %	1,1 %

Calculation of the Load-bearing Capacity of the Basement Wall of a Brick Building



Application of vertical loads

Objective: Check the calculation of the basement wall.

Task: Verify the correctness of the analysis of stability in the eccentricity plane under eccentric compression of the section with the maximum bending moment.

References: Reference manual on design of masonry and reinforced masonry structures (supplement to SNiP II-22-81), 1989, p. 81-82.

Initial data file:
Example 18.SAV;
ComeIn 18.doc — report

Initial data:

H = 2,8 m	Height of the basement wall;
$b \times h$ = 0,4×0,58 m	Dimensions of concrete blocks;
A_v = 25 %	Void percentage of blocks over the area of the middle horizontal cross-section;
V_v = 15 %	Void percentage of blocks over the volume;
l_0 = 2,65 m	Effective height of the basement wall;
b_1 = 0,51 m	Thickness of the first floor brick wall;
N_1 = 150 kN	Design load per 1 m of the basement wall from the first floor wall;
e_1 = 5,5 cm	Eccentricity of the load from the first floor wall;
N_2 = 22 kN	Design load per 1 m of the basement wall from the floor slab above the basement bearing on this wall;

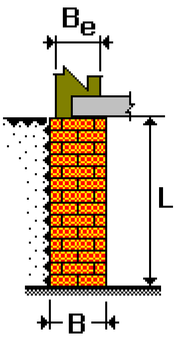
e_2 = 16 cm Eccentricity of the load from the floor slab above the basement bearing on the basement wall;
γ = 16 kN/m^3 Specific weight of fill-up soil;
φ = 38° Design internal friction angle of soil;
p = 10 kN/m^2 Characteristic value of the surface load from the fill-up soil;

Stone/brick Large hollow concrete blocks, grade 100;
Mortar Regular cement with mineral plasticizers, grade 50.

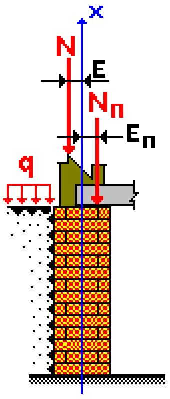
COMEIN initial data:

Importance factor γ_n = 1
Age of masonry - up to a year
Working life is 25 years
Stone/brick - Large concrete blocks, 500 mm ≤ H ≤ 1000 mm
Stone/brick grade - 100
Mortar - regular cement with mineral plasticizers
Mortar grade - 50
Reduction factor 0,5
Specific weight of masonry 22,44 kN/m^3

Structure:

	L = 2,65 m B = 0,4 m Be = 0,51 m

Loads per unit length

	Load on surface 12 kN/m^2 Specific weight of soil 19,2 kN/m^3 Angle of repose of soil 38 degrees Factor for sustained load 1 Nf = 22 kN/m Ef = 0,16 m Loads from the above floor slabs N = 150 kN/m E = 0,055 m Factor for sustained load 1

Comparison of solutions:

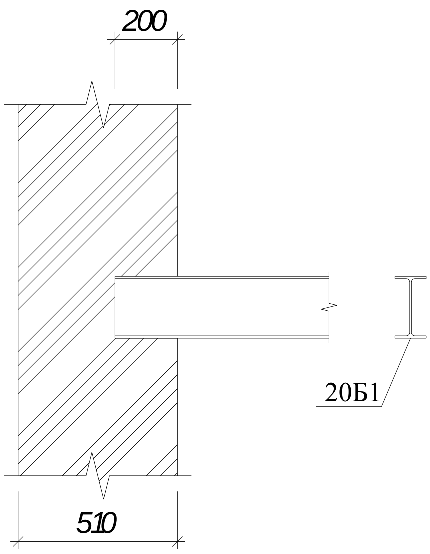
Check	stability under eccentric compression of the middle cross-section
Theory	181,5/380 = 0,478
COMEIN	0,481

Deviation, %	0,624

Comments:

1. The manual uses characteristic values of the load on surface and specific weight of soil, which are then multiplied by the corresponding overload factors n_1 = n_2 = 1,2. Their obtained design values are used in ComeIn: $p \cdot n_1 = 10 \cdot 1{,}2 = 12$ kN/m^2 and $\gamma \cdot n_2 = 16 \cdot 1{,}2 = 19{,}2$ N/m^3 respectively.
2. The value of the specific weight of soil is obtained by multiplying the specific weight of concrete 24 kN/m^3 by a factor of 0,85, taking into account the void percentage of blocks over the volume V_v=15 %, and the overload factor for masonry structures 1,1: $\gamma_m = 24 \cdot 0{,}85 \cdot 1{,}1 = 22{,}44$ kN/m^3.
3. Age of masonry and working life have to be specified in ComeIn. Since they are not determined in the problem, the following data are used: "up to a year" and 50 years respectively.
4. The column height has to be specified in ComeIn. Since the effective column height determined in the problem is 3 m, this value is used for the column height at the effective height factors equal to 1.

Check of the Support Joint of a Steel Floor Beam



Objective: Check of the local strength of masonry

Task: Check the load-bearing capacity of masonry under bearing stresses

References: Nasonov S.B. Manual for Design and Analysis of Building Structures. M: ASV Publishing House, 2013, p. 217-221.

Initial data file:
Example 13 Nasonov-2.SAV
report – ComeIn 13 Nasonov.doc.

Compliance with the codes: SP 15.13330.2012

Initial data:

l_o = 6,0 m	Beam span
a_1 = 200 mm	Length of the bearing part
b = 510 mm	Brick wall thickness
q = 5 kN/m	Uniformly distributed load on the beam
Q = 15 kN	Beam support reaction
Brick grade	M100
Mortar grade	M50

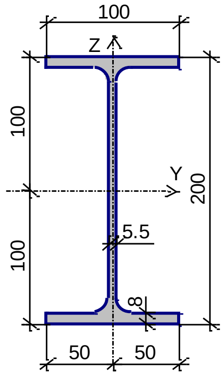
COMEIN initial data:

Importance factor $\gamma_n = 1$

Age of masonry - up to a year
Working life is 25 years
Stone/brick - Molded clay brick
Stone/brick grade - 100
Mortar - regular cement with mineral plasticizers
Mortar grade - 50

***Support design*:**

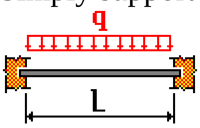
Steel beam



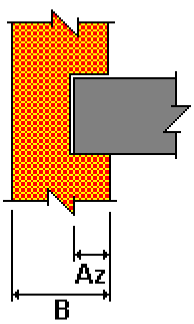
Regular I-beam STO ASChM 20-93 – 20B1

Support conditions:

Simply supported



L = 6 m
q = 0.5 T/m



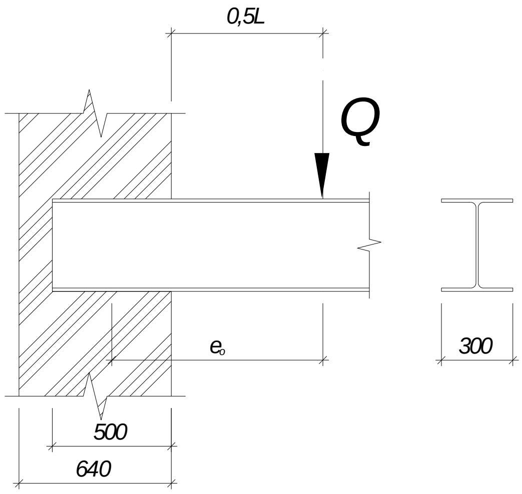
B = 510 mm
Az = 200 mm

Comparison of solutions:

Check	Bearing deformation of masonry under the beam
Manual	15/23,4 = 0,641
COMEIN	0,652
Deviation, %	1,7 %

Comment: The difference in the results of the calculation is related to the fact that the self-weight of beams is always taken into account in ComeIn.

Calculation of the Cantilever Beam Embedment in Masonry and Check of the Masonry for Local Bearing



Objective: Check of the local strength of masonry

Task: Check the load-bearing capacity of masonry under bearing stresses

References: Nasonov S.B. Manual for Design and Analysis of Building Structures. M: ASV Publishing House, 2013, p. 234-235.

***Initial data file*:**
Example 16 SP 2012.SAV
report – ComeIn 16 Nasonov.doc.

Compliance with the codes: SP 15.13330.2012

Initial data:

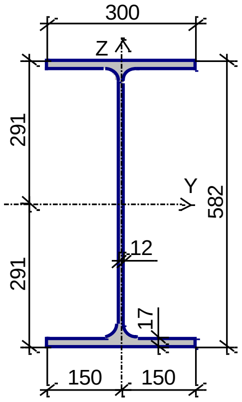
l_o = 1,5 m	Cantilever overhang length
a_1 = 500 mm	Length of the bearing part
b = 640 mm	Brick wall thickness
q = 5 kN/m	Uniformly distributed load on the beam
Q = 7,5 kN	Beam support reaction
Brick grade	M150
Mortar grade	M50

COMEIN initial data:

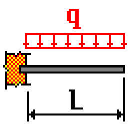
Importance factor $\gamma_n = 1$

Age of masonry - up to a year
Working life is 25 years
Stone/brick - Molded clay brick
Stone/brick grade - 150
Mortar - regular cement with mineral plasticizers
Mortar grade - 50

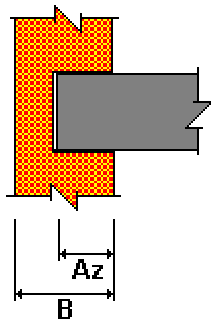
Support design:

<table>
<tr><td>Steel beam

</td><td>Wide flange I-beam STO ASChM 20-93 – 60W1</td></tr>
</table>

Support conditions:

<table>
<tr><td>Cantilever


without distribution packing plates

L = 1.5 m
q = 0.51 T/m</td><td>

B = 640 mm
Az = 500 mm</td></tr>
</table>

Comparison of solutions:

Check	Bearing deformation of masonry under the beam
Manual	7,5/39,0 = 0,192
COMEIN	0,185
Deviation, %	3,8 %

Comment: The difference in the results of the calculation is related to the fact that the self-weight of beams is always taken into account in ComeIn and only single beams are considered.

Calculation of the Hanging Wall

Objective: Check of the load-bearing capacity of the hanging wall.

Task: Check the bearing strength of masonry above the support of the foundation beam.

References: Bedov A. I., Schepetieva T. A., Design of masonry and reinforced masonry constructions, Moskow : ASV publisher, 2003, p. 228

Compliance with the codes: SNiP II-22-81, SP 15.13330.2012.

Initial data file:
Example 15.SAV;
ComeIn.doc — report

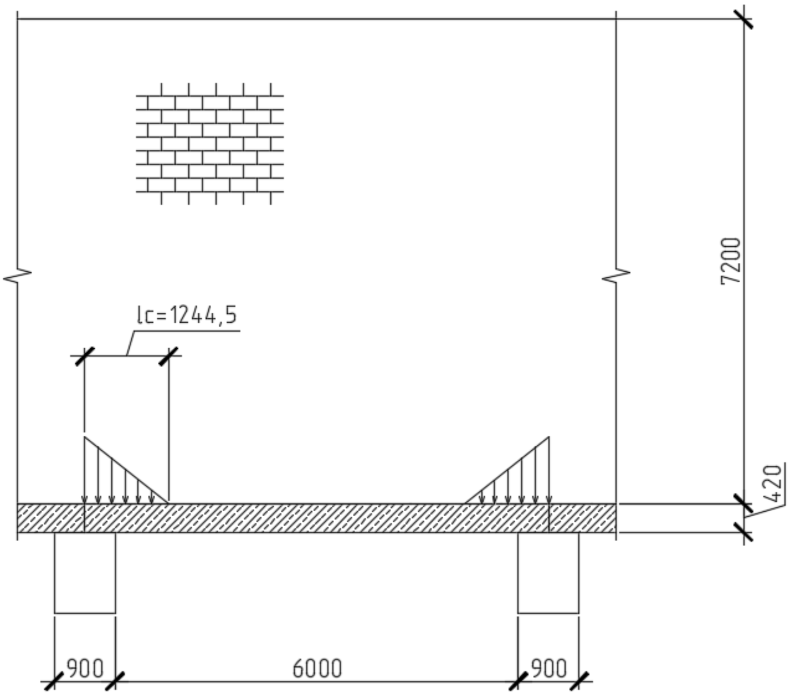
Initial data:

The hanging wall without openings 7,2 m high and 0,8 m thick is supported on the foundation reinforced concrete beam 0,38x0,42 (h) from heavy-weight natural hardening concrete of class B15 (E_b = 23 000 MPa). The distance between the supports of the beam is 6,0 m, the width of a support is 0,9 m.

The wall is made of molded clay brick of grade 75 with regular cement mortar of grade 25. The design resistance of masonry for compression is R = 1,3 MPa, the temporary resistance of masonry for compression is $R_u = kR = 2\times1{,}3 = 2{,}6$ MPa. The elastic characteristic of masonry is α = 1000. The deformation modulus of masonry is $E = 0{,}5E_0 = 0{,}5\alpha R_u = 0{,}5\times1000\times2{,}6 = 1300$ MPa.

Analytical solution:

The design beam support reaction without taking into account the load from its self-weight is equal to N = 0,38×7,2×18×1,1×(6+2×0,45)/2 = 186,89 кН



Moment of inertia of the beam

$$I = \frac{bh^3}{12} = \frac{0{,}38 \cdot 0{,}42^3}{12} = 0{,}002346 \text{ m}^4.$$

The height of the conventional chord with the equivalent rigidity is determined according to the following formula

$$H_0 = 2 \cdot \sqrt[3]{\frac{0{,}85 E_b I}{E \cdot b}} = 2 \cdot \sqrt[3]{\frac{0{,}85 \cdot 23000 \cdot 0{,}002346}{1300 \cdot 0{,}38}} = 0{,}905 \text{ m}$$

The length of the base of the pressure distribution diagram

$l_c = a_1 + 0,8H_0$ = 0,43+0,9×0,905 = 1,2445 m.

Bearing area

A_c = 1,2445×0,38 = 0,473 m^2.

Design bearing resistance of the masonry

$R_c = \xi R$ = 1,0×1,3 = 1,3 МПа

Bearing strength of the masonry

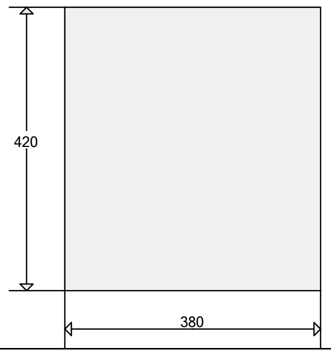
$N_c = \psi d R_c A_c = \psi(1,5 - 0,5\psi) R_c A_c$ =0,5x1,25x1,3x0,473 = 384,3 кН > N = 186,89 кН, i.e. the strength of the masonry is provided.

COMEIN initial data

Importance factor γ_n = 1
Age of masonry - up to a year
Working life is 25 years
Stone/brick - Molded clay brick
Stone/brick grade – 75
Mortar - regular cement with mineral plasticizers
Mortar grade - 50

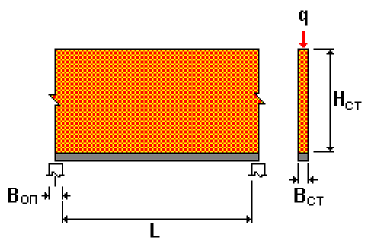
Supporting beam

Reinforced concrete beam
Single-span



Concrete
Concrete type: Heavy-weight
Concrete class: B15
Density of concrete 2,5 T/m^3
Hardening conditions: Natural
Hardening factor 1

Structure



Design load per running meter of the wall q = 0,001 T/m
Specific weight of masonry 1,8 T/m3
H_w = 7,2 m
B_w = 0,38 m
L = 6 m
B_{sup} = 0,9 m

Comparison of solutions:

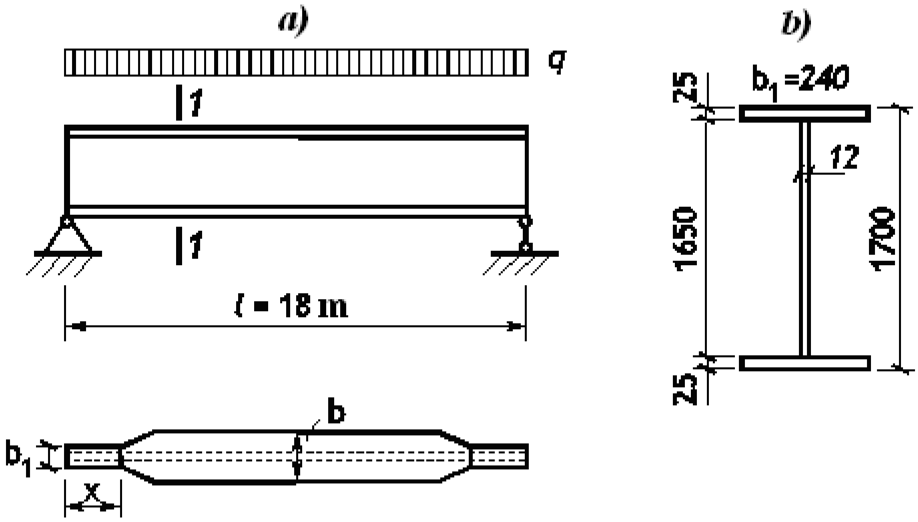
Check	Bearing strength of masonry above the support of the foundation beam
Manual	186,89/384,3 = 0,486
COMEIN	0,465
Deviation, %	3,6 %

KRISTALL

RESISTANCE OF SECTIONS

Strength and Stiffness Analysis of a Welded I-beam

Objective: Check of the **Resistance of Sections** mode



a – cross-section variation along the beam length; b – beam section and stress diagrams

Task: Check the design section of a welded I-beam for the main beams with a span of 18 m in a normal stub girder system. The top chord of the main beams is restrained by the stringers arranged with a spacing of 1,125 m.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p 195.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
4.1.sav; report — Kristall4.1.doc

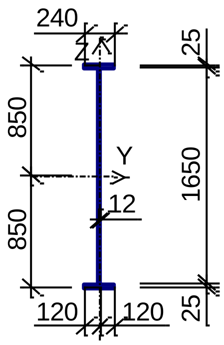
Initial data:

M_1 = 3469,28 kNm	Design bending moment;
Q_1 = 925 kN	Design shear force;
R_y = 23 kN/cm^2, R_s = 0,58*23=13,3 kN/cm^2	Steel grade C255 with thickness t>20 mm;
l = 18 m	Beam span;
$W_y = 15187,794$ cm^3	Geometric properties for a welded
$I_y = 1290962,5$ cm^4	I-section with flanges 240×25 mm and a web 1650×12 mm;
$S_y = 9108,75$ cm^3	
$i_y = 63,715$ cm, $i_z = 4,265$ cm	

KRISTALL parameters:
Steel: C255
Group of structures according to the table 50* of SNiP II-23-81* 3

Importance factor 1
Service factor 1
Limit slenderness for members in compression: 220
Limit slenderness for members in tension: 300

Section:



Manual calculation (SNiP II-23-81*):

1. Necessary beam section modulus:

$$W_{nes} = \frac{M_{\max}}{R_y \gamma_c} = \frac{3469,28 \cdot 100}{23} = 15083,826\ \text{cm}^3.$$

2. Maximum tangential stresses in support sections of the beam:

$$\tau_{\max} = \frac{Q_{\max} S_y}{I_y t_w} = \frac{925 \cdot 9108,75}{1290962,5 \cdot 1,2} = 5,4388\ \text{kN/cm}^2.$$

3. Reduced stresses in the considered beam section:

$$\sigma_y = \frac{M_y}{I_y}\frac{h_w}{2} = \frac{3469,28 \cdot 100 \cdot 165}{1290962,5 \cdot 2} = 22,1707\ \ \text{kN/cm}^2$$

$$\tau_{yz} = \frac{Q_z S_{yf}}{I_y t_w} = \frac{925 \cdot \left(24 \cdot 2,5 \cdot \left(0,5 \cdot 165 + 0,5 \cdot 2,5\right)\right)}{1290962,5 \cdot 1,2} = 3,00\ \ \text{kN/cm}^2$$

$$\sigma_{red} = \sqrt{\sigma_y^2 + 3\tau_{yz}^2} = \sqrt{22,1707^2 + 3 \cdot 3,00^2} = 22,7715\ \ \text{kN/cm}^2$$

4. Slenderness of the member in the moment plane:

$$\lambda_y = \frac{\mu l}{i_y} = \frac{18 \cdot 100}{63,715} = 28,2508.$$

5. Slenderness of the member out of the moment plane:

$$\lambda_y = \frac{\mu l}{i_y} = \frac{1,125 \cdot 100}{4,265} = 26,3775.$$

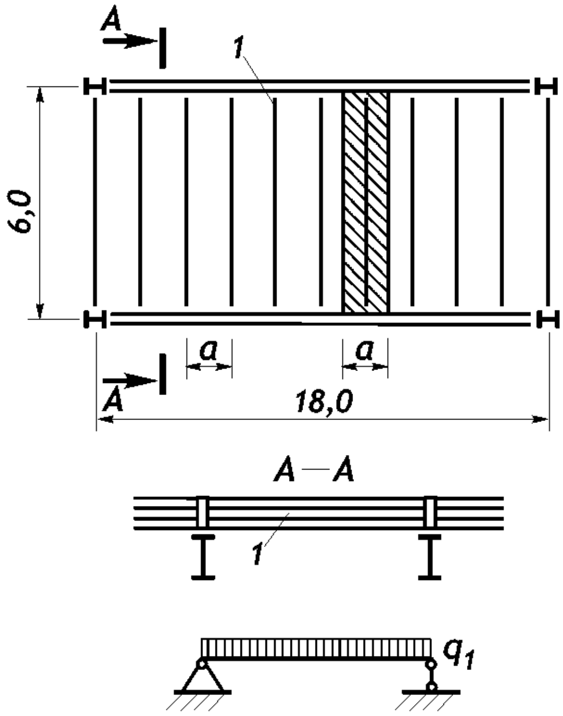
Comparison of solutions:

Factor	Source	Manual calculation	KRISTALL	Deviation from the manual calculation, %
Strength under action of bending moment *M*y	0,99	15083,826/15187,794 = = 0,993	0,993	0,0
Strength under action	–	5,4388/13,3 = 0,4089	0,408	0,0

Factor	Source	Manual calculation	KRISTALL	Deviation from the manual calculation, %
of lateral force *Q*z				
Strength for reduced stresses	–	22,7715/1,15/23 = 0,861	0,861	0,0
Strength under combined action of longitudinal force and bending moments, no plasticity	–	15083,826/15187,794 = = 0,993	0,993	0,0
Stability of in-plane bending	–	15083,826/1/15187,794 = = 0,993	0,993	0,0
Limit slenderness in XoY plane	–	26,3775/300 = 0,088	0,088	0,0
Limit slenderness in XoZ plane	–	28,2508/300 = 0,094	0,094	0,0

Strength and Stiffness Analysis of a Rolled I-beam

Objective: Check of the **Resistance of Sections** mode



1 – stringer

Task: Check the design section of a rolled I-beam for the stringers with a span of 6 m in a normal stub girder system. The top chord of the stringers is continuously restrained by the floor plate.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 183.

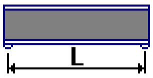
Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
4.2.sav; report — Kristall4.2.doc

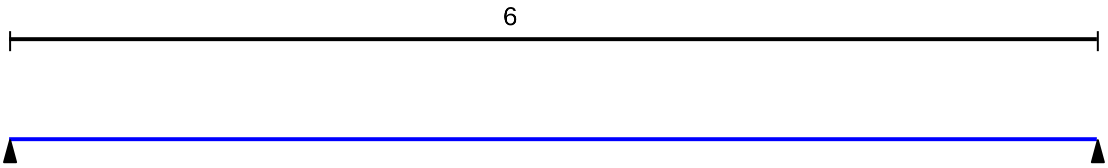
Initial data:

$a = 1{,}125$ m	Spacing of stringers;
$R_y = 23$ kN/cm^2,	Steel grade C235;
$M = 125{,}55$ kNm	Design bending moment;
$\gamma_c = 1$	Service factor;
$l = 6$ m	Beam span;
$c_x = 1{,}1$	Coefficient allowing for plastic deformations;
$W_x = 597$ cm^3	Selected I-beam No.33 GOST 8239-89;
$i_y = 13{,}524\,\text{cm},\ i_z = 2{,}791\,\text{cm}.$	

KRISTALL parameters:
Steel: C235
Group of structures according to the table 50* of SNiP II-23-81* 4
Importance factor $\gamma_n = 1$
Importance factor (serviceability limit state) = 1
Service factor 1



Structure:

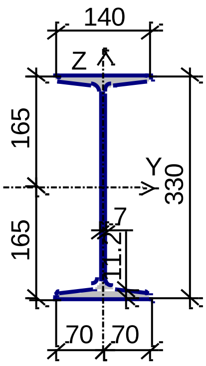


Restraints against lateral displacements and rotations:

	Left	Right
Displacement along Y	Restrained	Restrained
Displacement along Z	Restrained	Restrained
Rotation about Y		
Rotation about Z		

Continuous restraint of the compressed chord out of the bending plane

Section:



Profile: I-beam with sloped inner flange surfaces GOST 8239-89 33

Manual calculation (SNiP II-23-81*):

1. Necessary beam section modulus:

$$W_{nes} = \frac{M_{\max}}{R_y \gamma_c} = \frac{125,55 \cdot 100}{23} = 545,8696 \text{ cm}^3.$$

2. Slenderness of the member in the moment plane:

$$\lambda_y = \frac{\mu l}{i_y} = \frac{6 \cdot 100}{13,524} = 44,3656\,.$$

3. Slenderness of the member out of the moment plane:

$$\lambda_z = \frac{l_{ef,z}}{i_z} = \frac{6 \cdot 100}{2,791} = 214,9767\,.$$

Comparison of solutions:

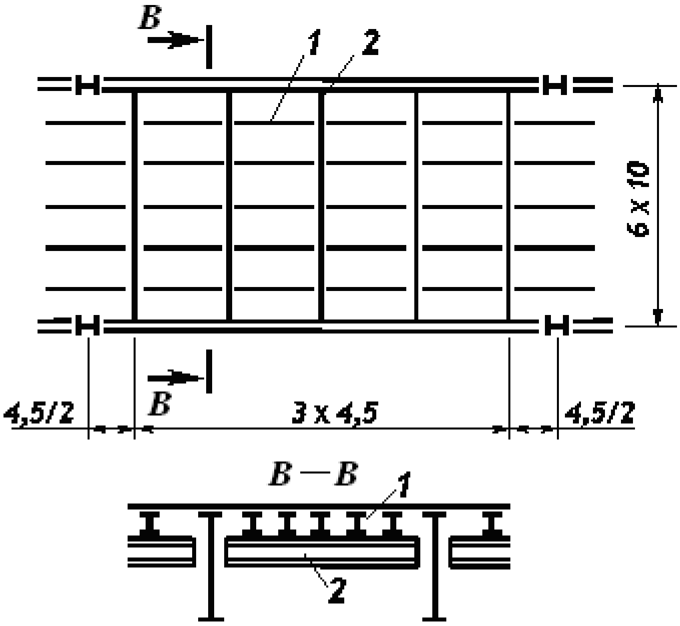
Factor	Source	Manual calculation	KRISTALL	Deviation from the manual calculation, %
Strength under action of bending moment M_y	0,83	545,8696 /597 = 0,914	0,915	0,0
Strength under combined action of longitudinal force and bending moments, no plasticity	–	545,8696 /597 = 0,914	0,915	0,0
Stability of in-plane bending	–	545,8696 /1/597 = 0,914	0,915	0,0
Limit slenderness in XoY plane	–	214,9767/250 = 0,86	0,86	0,0
Limit slenderness in XoZ plane	–	44,3656/250 = 0,177	0,177	0,0

Comments:

1. In the source the check of the beam strength was performed taking into account the development of the limited plastic deformations.
2. The check of the beam strength taking into account the development of the limited plastic deformations was not performed in the manual calculation, because according to the codes this calculation is possible only when the beam web has stiffeners. In the initial data of the example the stringer was specified without any intermediate stiffeners.

Strength and Stiffness Analysis of a Rolled I-beam

Objective: Check of the **Resistance of Sections** mode



1 – stringer; 2 – secondary beam

Task: Check the design section of a rolled I-beam for the stringers with a span of 4,5 m in a normal stub girder system. The top chord of the stringers is continuously restrained by the floor plate.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 183.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
4.3.sav; report — Kristall4.3.doc

Initial data:

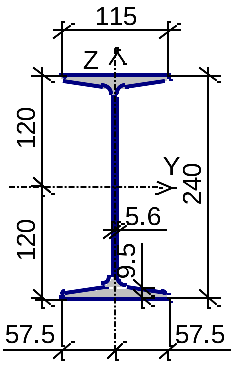
R_y = 23 kN/cm^2	Steel grade C235;
M = 62,78 kNm	Design bending moment;
γ_c = 1	Service factor;
l = 4,5 m	Beam span;
c_x = 1,1	Coefficient allowing for plastic deformations;
W_x = 288,33 cm^3	Selected I-beam No.24 GOST 8239-89;
$i_y = 9{,}971\,\text{cm},\ i_z = 2{,}385\,\text{cm}$	

KRISTALL parameters:
Steel: C235
Group of structures according to the table 50* of SNiP II-23-81* 4

Importance factor 1
Service factor 1

Limit slenderness for members in compression: 180 - 60α
Limit slenderness for members in tension: 250

Section:



Profile: I-beam with sloped inner flange surfaces GOST 8239-89 24

Manual calculation (SNiP II-23-81*):

1. Necessary beam section modulus:

$$W_{nes} = \frac{M_{\max}}{R_y \gamma_c} = \frac{62,78 \cdot 100}{23} = 272,9565\,\text{cm}^3.$$

2. Slenderness of the member in the moment plane:

$$\lambda_y = \frac{\mu l}{i_y} = \frac{4,5 \cdot 100}{9,971} = 45,131.$$

3. Slenderness of the member out of the moment plane:

$$\lambda_z = \frac{\mu l}{i_z} = \frac{4,5 \cdot 100}{2,385} = 188,679.$$

Comparison of solutions:

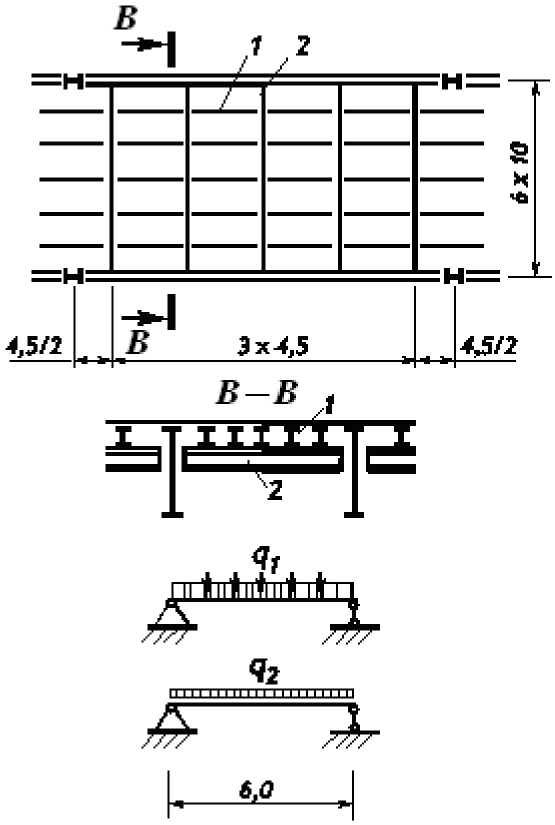
Factor	Source	Manual calculation	KRISTALL	Deviation from the manual calculation, %
Strength under action of bending moment M_y	0,86	272,9565/288,33 = 0,947	0,947	0,0
Strength under combined action of longitudinal force and bending moments, no plasticity	–	272,9565/288,33 = 0,947	0,947	0,0
Stability of in-plane bending	–	272,9565/1/288,33 = 0,947	0,947	0,0
Limit slenderness in XoY plane	–	188,679/250 = 0,755	0,755	0,0
Limit slenderness in XoZ plane	–	45,131/250 = 0,1805	0,181	0,0

Comments:

1. In the source the check of the beam strength was performed taking into account limited plastic deformations.
2. The check of the beam strength taking into account the development of the limited plastic deformations was not performed in the manual calculation, because according to the codes this calculation is possible only when the beam web has stiffeners. In the initial data of the example the stringer was specified without any intermediate stiffeners.

Strength and Stiffness Analysis of a Rolled I-beam

Objective: Check of the **Resistance of Sections** mode



1 – stringer; 2 – secondary beam

Task: Check the design section of a rolled I-beam for the secondary beams with a span of 6 m in a complex stub girder system. The top chord of the secondary beams is restrained by the stringers arranged with a spacing of 1 m.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 183.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
4.4.sav; report — Kristall4.4.doc

Initial data:

R_y = 23 kN/cm^2,	Steel grade C235;
M = 508,5 kNm	Design bending moment;
γ_c = 1	Service factor;
l = 6 m	Beam span;
c_x = 1,1	Coefficient allowing for plastic deformations;
W_x = 2034,982 cm^3	Selected I-beam No.55 GOST 8239-89;
$i_y = 21{,}777$ cm, $i_z = 3{,}39$ cm.	

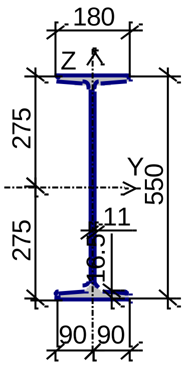
KRISTALL parameters:
Steel: C235
Group of structures according to the table 50* of SNiP II-23-81* 4

Importance factor 1

Service factor 1

Limit slenderness for members in compression: 180 - 60α
Limit slenderness for members in tension: 250

Section:



Profile: I-beam with sloped inner flange surfaces GOST 8239-89 55

Manual calculation (SNiP II-23-81*):

1. Necessary beam section modulus:

$$W_{nes} = \frac{M_{\max}}{R_y \gamma_c} = \frac{508,5 \cdot 100}{23} = 2210,8696\,\text{cm}^3.$$

2. Slenderness of the member in the moment plane and out of the moment plane:

$$\lambda_y = \frac{\mu l}{i_y} = \frac{6,0 \cdot 100}{21,777} = 27,552;$$

$$\lambda_z = \frac{\mu l}{i_z} = \frac{6,0 \cdot 100}{3,39} = 176,99.$$

Comparison of solutions:

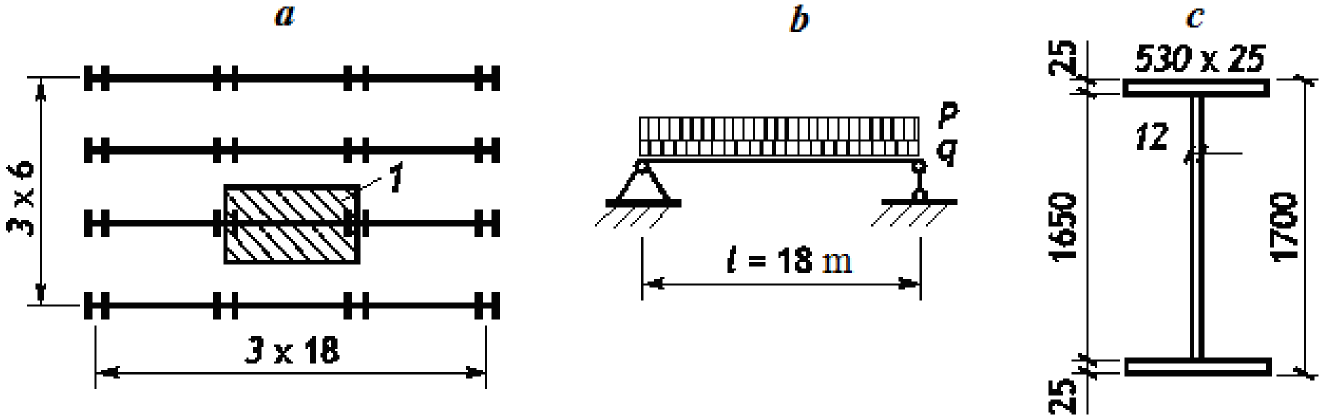
Factor	Source	Manual calculation	KRISTALL	Deviation from the manual calculation, %
Strength under action of bending moment M_y	0,99	2210,8696/2034,982 = 1,086	1,086	0,0
Strength under combined action of longitudinal force and bending moments, no plasticity	–	2210,8696/2034,982 = 1,086	1,086	0,0
Stability of in-plane bending	–	2210,8696/1/2034,982 = 1,086	1,086	0,0
Limit slenderness in XoY plane	–	176,99/250 = 0,708	0,708	0,0
Limit slenderness in XoZ plane	–	27,552/250 = 0,110	0,11	0,0

Comments:

1. In the source the check of the beam strength was performed taking into account the development of the limited plastic deformations.
2. The check of the beam strength taking into account the development of the limited plastic deformations was not performed in the manual calculation, because according to the codes this calculation is possible only when the beam web has stiffeners. In the initial data of the example the stringer was specified without any intermediate stiffeners.

Strength and Stiffness Analysis of a Welded I-beam

Objective: Check of the **Resistance of Sections** mode



a – floor plan; b – design model of the main beam; c – beam section;
1 – load area

Task: Check the design section of a welded I-beam for the main beams with a span of 18 m in a normal stub girder system. The top chord of the main beams is restrained by secondary beams arranged with a spacing of 1,0 m.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 192.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
4.5.sav; report — Kristall4.5.doc

Initial data:

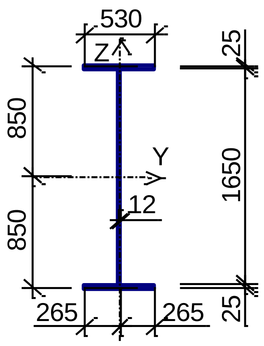
R_y = 23 kN/cm^2, R_s = 0,58*23=13,3 kN/cm^2	Steel grade C255 with thickness t>20 mm;
M = 6245 kNm	Design bending moment;
$\gamma_c = 1$	Service factor;
l = 18 m	Beam span;
$I_y = 2308077{,}083$ cm^4	Geometric properties for a welded
$W_y = 27153{,}848$ cm^3	I-section with flanges 1650×12 mm and a web
$i_y = 70{,}605$ cm, $i_z = 11{,}577$ cm.	530×25 mm;

KRISTALL parameters:
Steel: C255
Group of structures according to the table 50* of SNiP II-23-81* 3

Importance factor 1
Service factor 1

Limit slenderness for members in compression: 180 - 60α
Limit slenderness for members in tension: 250

Section:



Manual calculation (SNiP II-23-81*):

1. Necessary beam section modulus:

$$W_{nes} = \frac{M_{\max}}{R_y \gamma_c} = \frac{6245 \cdot 100}{23} = 27152{,}174 \text{ cm}^3.$$

2. Slenderness of the member in the moment plane and out of the moment plane:

$$\lambda_y = \frac{\mu l}{i_y} = \frac{18{,}0 \cdot 100}{70{,}605} = 25{,}4939\,;$$

$$\lambda_z = \frac{\mu l}{i_z} = \frac{18{,}0 \cdot 100}{11{,}577} = 155{,}481\,.$$

Comparison of solutions:

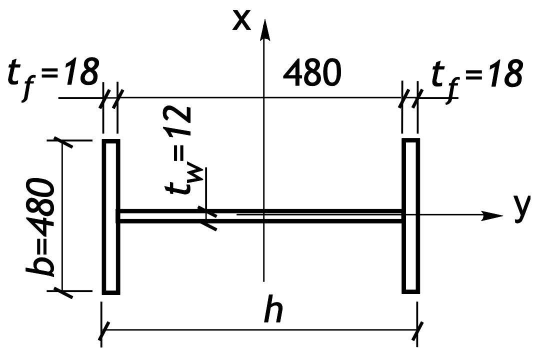
Factor	**Source**	**Manual calculation**	**KRISTALL**	**Deviation from the manual calculation, %**
Strength under action of bending moment M_y	1,0	27152,174/27153,848 = 1,0	1,0	0,0
Strength under combined action of longitudinal force and bending moments, no plasticity	–	27152,174/27153,848 = 1,0	1,0	0,0
Stability of in-plane bending	–	27152,174/1/27153,848 = 1,0	1,0	0,0
Limit slenderness in XoZ plane	–	25,4939/250 = 0,102	0,102	0,0
Limit slenderness in XoY plane	–	155,481/250 = 0,622	0,622	0,0

Comments:

The check of the beam strength taking into account the development of the limited plastic deformations was not performed, because according to the codes this calculation is possible only when the beam web has stiffeners. In the initial data of the example the stringer was specified without any intermediate stiffeners.

Analysis of an Axially Compressed Welded I-beam Column

Objective: Check of the **Resistance of Sections** mode



Task: Check the design section of a welded I-beam for the axially compressed column with a height of 6,5 m.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 256.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
4.6.sav; report — Kristall4.6.doc

Initial data:

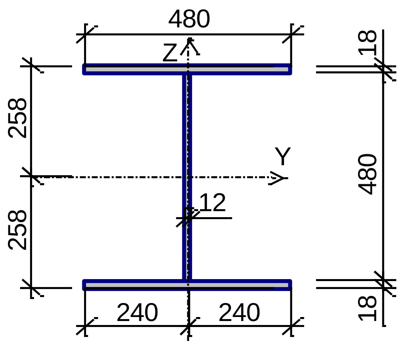
R_y = 24 kN/cm^2	Steel grade C245
l = 6,5 m	Column height
N = 5000 kN	Design longitudinal compressive force
μ = 0,7	The lower restraint is rigid and the upper one is pinned for both principal planes of inertia
γ_c = 1	Service factor
$A = 230,4\,\text{cm}^2$,	Geometric properties for a welded
$I_y = 118243,584\,\text{cm}^4$, $I_z = 33184,512\,\text{cm}^4$	I-section with a web 480×12 mm and flanges
$W_y = 4583,085\,\text{cm}^3$, $W_z = 1382,688\,\text{cm}^3$	480×18 mm
$i_y = 22,654$ cm, $i_z = 12,001$ cm	

KRISTALL parameters:
Steel: C245
Group of structures according to the table 50* of SNiP II-23-81* 3

Importance factor 1
Service factor 1

Limit slenderness for members in compression: 180 - 60□
Limit slenderness for members in tension: 250

Section:



Manual calculation (SNiP II-23-81*):

1. Load-bearing capacity of the element under axial compression/tension:

$$N = AR_y\gamma_c = 230,4 \cdot 24 \cdot 1 = 5529,6\,\text{kN}.$$

2. Slenderness of the element for both principal planes of inertia:

$$\lambda_y = \frac{l_{ef,y}}{i_y} = \frac{\mu l}{i_y} = \frac{0,7 \cdot 6,5 \cdot 100}{22,654} = 20,08475\,;$$

$$\bar{\lambda}_z = \frac{l_{ef,z}}{i_z} = \frac{\mu l}{i_z} = \frac{0,7 \cdot 6,5 \cdot 100}{12,001} = 37,9135\,.$$

3. Conditional slenderness of the element for both principal planes of inertia:

$$\bar{\lambda}_y = \frac{l_{ef,y}}{i_y}\sqrt{\frac{R_y}{E}} = \frac{\mu l}{i_y}\sqrt{\frac{R_y}{E}} = \frac{0,7 \cdot 6,5 \cdot 100}{22,654}\sqrt{\frac{240}{2,06 \cdot 10^5}} = 0,68555\,;$$

$$\bar{\lambda}_z = \frac{l_{ef,z}}{i_z}\sqrt{\frac{R_y}{E}} = \frac{\mu l}{i_z}\sqrt{\frac{R_y}{E}} = \frac{0,7 \cdot 6,5 \cdot 100}{12,001}\sqrt{\frac{240}{2,06 \cdot 10^5}} = 1,2941\,.$$

4. Buckling coefficients under axial compression:

$$\varphi_y = 1 - \left(0,073 - 5,53\frac{R_y}{E}\right)\bar{\lambda}_y\sqrt{\bar{\lambda}_y} = 1 - \left(0,073 - 5,53 \cdot \frac{240}{2,06 \cdot 10^5}\right) \cdot 0,68555\sqrt{0,68555} = 0,9622\,;$$

$$\varphi_z = 1 - \left(0,073 - 5,53\frac{R_y}{E}\right)\bar{\lambda}_z\sqrt{\bar{\lambda}_z} = 1 - \left(0,073 - 5,53 \cdot \frac{240}{2,06 \cdot 10^5}\right) \cdot 1,2941\sqrt{1,2941} = 0,902\,;$$

5. Load-bearing capacity of the element at its buckling:

$$N_{b,y} = \varphi_y AR_y\gamma_c = 0,9622 \cdot 230,4 \cdot 24 \cdot 1 = 5320,58\,\text{kN};$$

$$N_{b,z} = \varphi_z AR_y\gamma_c = 0,902 \cdot 230,4 \cdot 24 \cdot 1 = 4987,7\,\text{kN}.$$

6. Limit slenderness:

$$\lambda_{uy} = 180 - 60 \cdot \frac{N}{\varphi_y AR_y\gamma_c} = 180 - 60 \cdot \frac{5000}{0,9622 \cdot 230,4 \cdot 24 \cdot 1} = 123,615\,;$$

$$\lambda_{uz} = 180 - 60 \cdot \frac{N}{\varphi_z AR_y\gamma_c} = 180 - 60 \cdot \frac{5000}{0,902 \cdot 230,4 \cdot 24 \cdot 1} = 119,852\,.$$

Comparison of solutions:

Factor	Source	Manual calculation	KRISTALL	Deviation %
Strength under combined action of longitudinal force and bending moments, no plasticity	–	5000/5529,6 = 0,904	0,904	–
Stability under compression in XoY (XoU) plane	23,69/24 = = 0,987	5000/4987,7 = 1,002	1,002	–
Stability under compression in XoZ (XoV) plane	–	5000/5320,58 = 0,94	0,94	–
Strength under axial compression/tension	0,904	5000/5529,6 = 0,904	0,904	–
Limit slenderness in XoY plane	–	37,9135/119,852 = 0,316	0,316	–
Limit slenderness in XoZ plane	–	20,085/123,615= 0,162	0,162	–

Analysis of a Lattice Axially Compressed Column from Two Continuous Chords with a Channel Section on Battens

Objective: Check of the **Resistance of Sections** mode

Task: Check the design lattice section on battens with two continuous chords from rolled channels for the axially compressed column with a height of 6 m.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 257.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
4.7.sav; report — Kristall-4.7.doc

Initial data:

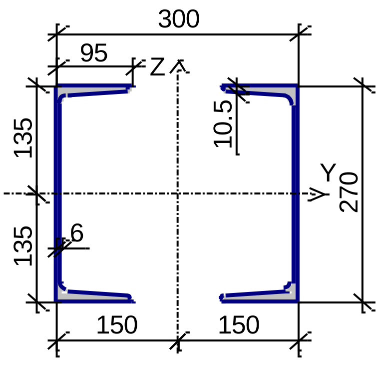
l = 6 m	Column height;
$\mu = 1$	Pinned restraint in both principal planes of inertia;
N = 1400 kN	Design compressive force;
$\gamma_c = 1$	Service factor ;
R_y = 24 kN/cm^2	Steel grade C245;
B = 300 mm	Distance between the chords (outer dimension);
b = 170 mm, s = 1120 mm	Batten height, distance between the batten axes;
t = 10 mm	Batten thickness;
$A = 70,4\,\text{cm}^2,\ I_y = 8320\ \text{cm}^4,\ I_z = 11576,86\,\text{cm}^4$	Geometric properties of the lattice section;
$i_y = 10,871\,\text{cm},\ i_z = 12,824\ \text{cm}$	
$A_b = 35,2\ \text{cm}^2,\ I_b = I_z = 262\ \ \text{cm}^4$	Geometric properties of the chord section;
$i_y = 10,871\,\text{cm},\ i_z = 2,728\,\text{cm}$	
$W_{b,z,\min} = 37,269\,\text{cm}^3.$	

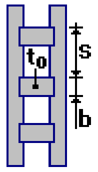
KRISTALL parameters:
Steel: C245
Group of structures according to the table 50* of SNiP II-23-81* 3

Importance factor 1
Service factor 1

Limit slenderness for members in compression: 180 - 60α
Limit slenderness for members in tension: 250

Section:





b = 170 mm
t_0 = 10 mm
s = 1120 mm

Profile: Channel with sloped inner flange surfaces
GOST 8240-89 27

Manual calculation (SNiP II-23-81*):

1. Moment of inertia and section modulus of one batten with respect to its own axis:

$$I_s = \frac{t_0 b^3}{12} = \frac{1 \cdot 17^3}{12} = 409,4167\ \text{cm}^4;$$

$$W_s = \frac{t_0 b^2}{6} = \frac{1 \cdot 17^2}{6} = 48,167\ \text{cm}^3.$$

2. Distance between chord axes:

$$b = B - 2z_0 = 30 - 2 \cdot 2,47 = 25,06\ \text{cm}.$$

3. Slenderness of one of the chords in the sections between the battens (in the clear):

$$\lambda_{1,y} = \frac{l_{1,y}}{i_{1,y}} = \frac{600}{10,871} = 55,193; \quad \bar{\lambda}_{1,y} = \lambda_{1,y}\sqrt{\frac{R_y}{E}} = 55,193\sqrt{\frac{240}{2,06 \cdot 10^5}} = 1,884 \le 5;$$

$$\lambda_{1,z} = \frac{l_{1,z}}{i_{1,z}} = \frac{s-b}{i_{1,z}} = \frac{112-17}{2,728} = 34,824 < 40; \quad \bar{\lambda}_{1,z} = \lambda_{1,z}\sqrt{\frac{R_y}{E}} = 34,824\sqrt{\frac{240}{2,06 \cdot 10^5}} = 1,1886 \le 5.$$

4. Column slenderness and respective conditional slenderness:

$$\lambda_y = \frac{l_{ef,y}}{i_y} = \frac{600}{10,871} = 55,193; \ \Rightarrow \ \bar{\lambda}_y = \lambda_y\sqrt{\frac{R_y}{E}} = 55,193\sqrt{\frac{240}{2,06 \cdot 10^5}} = 1,884 < 2,5;$$

$$\lambda_z = \frac{l_{ef,z}}{i_z} = \frac{600}{12,824} = 46,787.$$

5. Reduced and conditional reduced slenderness of the column with respect to the free axis:

$$\text{When } \frac{I_s s}{I_b b} = \frac{409,4167 \cdot 112}{262 \cdot 25,06} = 6,984 > 5:$$

$$\lambda_z = \lambda_{ef,z} = \sqrt{\lambda_z^2 + \lambda_{1,z}^2} = \sqrt{46,787^2 + 34,824^2} = 58,3244;$$

$$\bar{\lambda}_z = \lambda_z\sqrt{\frac{R_y}{E}} = 58,3244\sqrt{\frac{240}{2,06 \cdot 10^5}} = 1,991 < 2,5.$$

6. Buckling coefficients:

$$\varphi_y = 1 - \left(0,073 - 5,53\frac{R_y}{E}\right)\bar{\lambda}_y\sqrt{\bar{\lambda}_y} = 1 - \left(0,073 - 5,53\frac{240}{2,06 \cdot 10^5}\right) \cdot 1,884\sqrt{1,884} = 0,8279;$$

$$\varphi_z = 1 - \left(0,073 - 5,53\frac{R_y}{E}\right)\bar{\lambda}_z\sqrt{\bar{\lambda}_z} = 1 - \left(0,073 - 5,53\frac{240}{2,06 \cdot 10^5}\right) \cdot 1,991\sqrt{1,991} = 0,813.$$

7. Limit compressive forces causing the column buckling about the respective axes:

$$N_{b,y} = \varphi_y A R_y \gamma_c = 0,8279 \cdot 70,4 \cdot 24 \cdot 1 = 1398,82 \text{ kN};$$

$$N_{b,z} = \varphi_z A R_y \gamma_c = 0,813 \cdot 70,4 \cdot 24 \cdot 1 = 1373,645 \text{ kN}.$$

8. Conditional shear force Q_{fic}:

$$Q_{fic} = 7,15 \cdot 10^{-6} \left(2330 - \frac{E}{R_y} \right) \frac{N}{\varphi_z} = 7,15 \cdot 10^{-6} \left(2330 - \frac{2,06 \cdot 10^5}{240} \right) \frac{1400}{0,813} = 18,1198 \text{ kN}.$$

9. Force F, shearing the batten, and moment M_1, bending the batten in its plane:

$$F = \frac{Q_s s}{b} = \frac{Q_{fic} s}{2b} = \frac{18,1198 \cdot 112}{2 \cdot 25,06} = 40,4912 \text{ kN};$$

$$M_1 = \frac{Q_s s}{2} = \frac{Q_{fic} s}{4} = \frac{18,1198 \cdot 112}{4} = 507,3544 \text{ kNcm}.$$

10. Load-bearing capacity of the batten under bending:

$$W_s R_y \gamma_c = 48,167 \cdot 24 \cdot 1 = 1156,01 \text{ kNcm}.$$

11. Bending moment acting on the column chord and caused by the bending of the batten:

$$M_b = 2M_1 = 2 \cdot 507,3544 = 1014,7088 \text{ kNcm}.$$

12. Load-bearing capacity of the chord under bending in the batten plane:

$$W_{b,z,\min} R_y \gamma_c = 37,269 \cdot 24 \cdot 1 = 894,456 \text{ kNcm}.$$

13. Strength of the chord under the combined action of the longitudinal force and bending moment in the batten plane without taking the plasticity into account:

$$\frac{1}{R_y \gamma_c} \left(\frac{N}{A} + \frac{M_b}{W_{b,z,\min}} \right) = \frac{1}{24 \cdot 1} \left(\frac{1400}{70,4} + \frac{1014,7088}{37,269} \right) = 1,963.$$

14. Buckling coefficients for a chord:

$$\varphi_y = 1 - \left(0,073 - 5,53 \frac{R_y}{E} \right) \bar{\lambda}_{1,y} \sqrt{\bar{\lambda}_{1,y}} = 1 - \left(0,073 - 5,53 \frac{240}{2,06 \cdot 10^5} \right) 1,884 \sqrt{1,884} = 0,828;$$

$$\varphi_z = 1 - \left(0,073 - 5,53 \frac{R_y}{E} \right) \lambda_{1,z} \sqrt{\lambda_{1,z}} = 1 - \left(0,073 - 5,53 \frac{240}{2,06 \cdot 10^5} \right) \cdot 1,1886 \sqrt{1,1886} = 0,914.$$

15. Load-bearing capacity of the chord under compression:

$$\varphi_y A R_y \gamma_c = 0,828 \cdot 35,2 \cdot 24 \cdot 1 = 699,398 \text{ kN};$$

$$\varphi_z A R_y \gamma_c = 0,914 \cdot 35,2 \cdot 24 \cdot 1 = 772,1472 \text{ kN}.$$

16. Relative eccentricity, cross-section shape coefficient, reduced eccentricity, and buckling coefficient under eccentric compression of the chord:

$$m_z = \frac{M_z}{N} \cdot \frac{A_b}{W_{b,z,\min}} = \frac{1014,7088}{700} \cdot \frac{35,2}{37,269} = 1,36911 \le 5;$$

$$\frac{A_f}{A_w} = \frac{16,2}{19,95} = 0,812;$$

$$\eta = (1,25 - 0,05 m_z) - 0,01(5 - m_z) \bar{\lambda}_{1,z} = (1,25 - 0,05 \cdot 1,36911) - 0,01(5 - 1,36911) \cdot 1,1886 = 1,13838$$

(for the section type 9 according to the table 73 of SNiP II-23-81* when $\frac{A_f}{A_w} = 0,5$);

$$\eta = (1,5-0,1m_z)-0,02(5-m_z)\,\bar{\lambda}_{1,z} = (1,5-0,1\cdot 1,36911)-0,02(5-1,36911)\cdot 1,1886 = 1,27678$$

(for the section type 9 according to the table 73 of SNiP II-23-81* when $\frac{A_f}{A_w}=1,0$);

$\eta = 1,2247$ (for the section type 9 according to the table 73 of SNiP II-23-81* when $\frac{A_f}{A_w}=0,812$);

$\eta = 1,45+0,04m_z = 1,45+0,04\cdot 1,36911 = 1,50476$ (for the section type 11 according to the table 73 of SNiP II-23-81* when $\frac{A_f}{A_w}=0,5$);

$\eta = 1,8+0,12m_z = 1,8+0,12\cdot 1,36911 = 1,9643$ (for the section type 11 according to the table 73 of SNiP II-23-81* when $\frac{A_f}{A_w}=1,0$);

$\eta = 1,7915$ (for the section type 11 according to the table 73 of SNiP II-23-81* when $\frac{A_f}{A_w}=1,33811$);

$$m_{z,ef} = \eta m_z = 1,7915\cdot 1,36911 = 2,453;$$

$\varphi_e = 0,4174$ (according to the table 74 of SNiP II-23-81*).

17. Stability check of the chord in the batten bending plane:

$$\frac{N}{2\varphi_e A_b} = \frac{1400}{2\cdot 0,4174\cdot 35,2} = 47,6434\ \text{kN/cm}^2 > R_y\gamma_c = 24\cdot 1 = 24\ \text{kN/cm}^2.$$

18. Stability check of the chord out of the batten bending plane is performed as a stability check of an axially compressed bar in the respective plane according to Sec. 5.32 of SNiP II-23-81*.

Comparison of solutions:

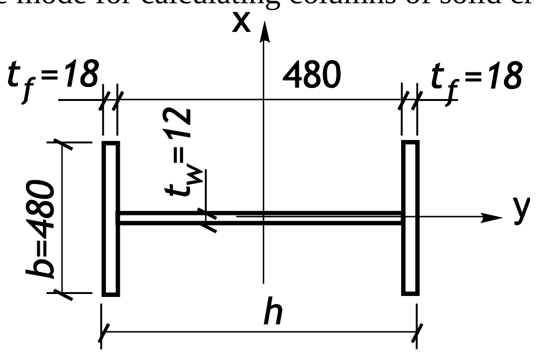
Factor	**Source**	**Manual calculation**	**KRISTALL**	**Deviation from the manual calculation, %**
General stability of a bar under axial compression in XoY plane	24/24=1	1400/1373,645 = 1,019	1,019	0,0
General stability of a bar under axial compression in XoZ plane	23,6/24=0,983	1400/1398,82 = 1,001	1,001	0,0
Resistance of a batten to bending	–	507,3544/1156,01 = 0,439	0,439	0,0
Strength under action of bending moment M_z	–	1014,7088/$894,456$ = 1,134	1,134	0,0
Strength under combined action of longitudinal force and bending moments, no plasticity	–	1,963	1,963	0,0
Stability of chord under compression in XoY plane	–	700/772,1472 = 0,9066	0,907	0,0
Stability of chord under	23,6/24=0,983	700/699,398 =	1,001	0,0

Factor	Source	Manual calculation	KRISTALL	Deviation from the manual calculation, %
compression in XoZ plane		1,001		
Stability of chord in the moment M_z plane under eccentric compression	–	47,6434/24 = 1,985	1,985	0,0
Stability of chord out of the moment M_z plane under eccentric compression	–	24,01735/24 = 1,001	1,001	0,0
Limit slenderness in XoY plane	–	58,3244/120 = 0,486	0,486	0,0
Limit slenderness in XoZ plane	–	55,193/120 = 0,46	0,46	0,0

COLUMNS

Analysis of an Axially Compressed Welded I-beam Column

Objective: Check the mode for calculating columns of solid cross-section



Task: Check the design section of a welded I-beam for the axially compressed column with a height of 6,5 m.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 256.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

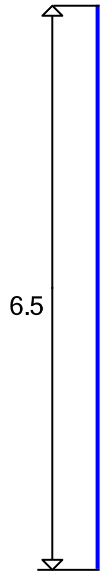
Initial data file:
5.1.sav; report — Kristall-5.1.doc

Initial data:

l = 6,5 m	Column height
μ = 0,7	The lower restraint is rigid and the upper one is pinned
N = 5000 kN	Design compressive force
γ_c = 1	Service factor
R_y = 24 kN/cm^2	Steel grade C245
$A = 230,4\,\text{cm}^2$	Geometric properties of
$I_y = 118243,584\,\text{cm}^4$, $I_z = 33184,512\,\text{cm}^4$	the selected section
$i_y = 22,654$ cm, $i_z = 12,001$ cm	

KRISTALL parameters:
Steel: C245
Group of structures according to the table 50* of SNiP II-23-81* 3

Importance factor 1
Service factor 1

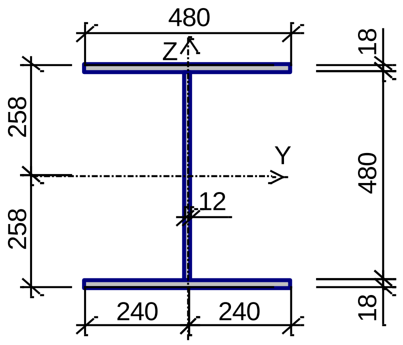


Member length – 6.5 m

Length between out-of-plane restraints – 6.5 m

Limit slenderness for members in compression: 180 - 60□
Limit slenderness for members in tension: 250

Section:



Manual calculation (SNiP II-23-81*):

1. Strength check of the selected column section:

$$\frac{N}{AR_y\gamma_c} = \frac{5000}{230,4 \cdot 24 \cdot 1} = 0,904 .$$

2. Slenderness of the column:

$$\lambda_y = \frac{l_{ef,y}}{i_y} = \frac{0,7 \cdot 6,5 \cdot 100}{22,654} = 20,08475 ;$$

$$\lambda_z = \frac{l_{ef,z}}{i_z} = \frac{0,7 \cdot 6,5 \cdot 100}{12,001} = 37,9135 .$$

3. Conditional slenderness of the column:

$$\bar{\lambda}_y = \frac{l_{ef,y}}{i_y}\sqrt{\frac{R_y}{E}} = \frac{0,7 \cdot 6,5 \cdot 100}{22,654}\sqrt{\frac{240}{2,06 \cdot 10^5}} = 0,68555 ;$$

$$\bar{\lambda}_z = \frac{l_{ef,z}}{i_z}\sqrt{\frac{R_y}{E}} = \frac{0,7\cdot 6,5\cdot 100}{12,001}\sqrt{\frac{240}{2,06\cdot 10^5}} = 1,2941.$$

4. Buckling coefficients:

$$\varphi_y = 1-\left(0,073-5,53\frac{R_y}{E}\right)\bar{\lambda}_y\sqrt{\bar{\lambda}_y} = 1-\left(0,073-\frac{5,53\cdot 240}{2,06\cdot 10^5}\right)\cdot 0,68555\sqrt{0,68555} = 0,9622;$$

$$\varphi_z = 1-\left(0,073-5,53\frac{R_y}{E}\right)\bar{\lambda}_z\sqrt{\bar{\lambda}_z} = 1-\left(0,073-\frac{5,53\cdot 240}{2,06\cdot 10^5}\right)\cdot 1,2941\sqrt{1,2941} = 0,902.$$

5. Strength of the column from the condition of providing the general stability under axial compression:

$$N_{b,y} = \varphi_y A R_y \gamma_c = 0,9622\cdot 230,4\cdot 24\cdot 1 = 5320,58\text{ kN};$$

$$N_{b,z} = \varphi_z A R_y \gamma_c = 0,902\cdot 230,4\cdot 24\cdot 1 = 4987,7\text{ kN}.$$

6. Limit slenderness of the column:

$$[\lambda]_y = 180-60\alpha_y = 180-60\cdot\frac{N}{\varphi_y A R_y \gamma_c} = 180-60\cdot\frac{5000}{5320,58} = 123,615;$$

$$[\lambda]_z = 180-60\alpha_z = 180-60\cdot\frac{N}{\varphi_z A R_y \gamma_c} = 180-60\cdot 1 = 120.$$

Comparison of solutions:

Factor	Source	Manual calculation	KRISTALL	Deviation, %
Strength under combined action of longitudinal force and bending moments, no plasticity	–	0,904	0,904	–
Stability under compression in XoY (XoU) plane	23,69/24=0,987	5000/4987,7 = 1,002	1,002	–
Stability under compression in XoZ (XoV) plane	–	5000/5320,58 = 0,940	0,94	–
Strength under axial compression/tension	5000/230,4/24= 0,904	0,904	0,904	–
Limit slenderness in XoY plane	–	37,9135/120 = 0,316	0,316	–
Limit slenderness in XoZ plane	–	20,08475/123,615 = 0,1625	0,1625	–

Analysis of a Lattice Axially Compressed Column from Two Rolled Channels

Objective: Check the mode for calculating columns of lattice cross-section

Task: Check the design lattice section from two channels on battens for the axially compressed column with a height of 6,5 m.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 257.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

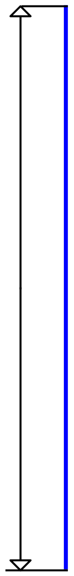
Initial data file:
5.2.sav; report — Kristall-5.2.doc

Initial data:

l = 6 m	Column height;
$\mu = 1$	Pinned restraint;
N = 1400 kN	Design compressive force;
$\gamma_c = 1$	Service factor ;
R_y = 24 kN/cm^2	Steel grade C245;
B = 300 mm	Distance between the outer faces of the chord;
b = 170 mm, s = 1120 mm	Batten height, distance between the batten axes;
t = 10 mm	Batten thickness;
$A = 70,4\,\text{cm}^2,\ I_y = 8320\ \text{cm}^4,\ I_z = 11576,86\,\text{cm}^4$	Geometric properties of the lattice section;
$i_y = 10,871\,\text{cm},\ i_z = 12,824\ \text{cm}$	
$A_b = 35,2\ \text{cm}^2,\ I_b = I_z = 262\ \ \text{cm}^4$	Geometric properties of the chord section;
$i_y = 10,871\,\text{cm},\ i_z = 2,728\,\text{cm}$	
$W_{b,z,\min} = 37,269\,\text{cm}^3.$	

KRISTALL parameters:
Steel: C245
Group of structures according to the table 50* of SNiP II-23-81* 3

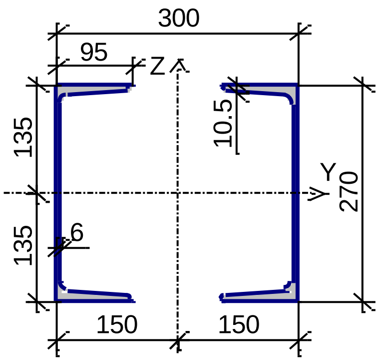
Importance factor 1
Service factor 1



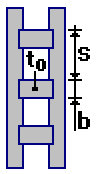
Member length – 6 m

Limit slenderness for members in compression: 180 - 60α
Limit slenderness for members in tension: 250

Section:



Lattice



b = 170 mm
t_0 = 10 mm
s = 1120 mm

Profile: Channel with sloped inner flange surfaces GOST 8240-89 27

Manual calculation (SNiP II-23-81*):

1. Moment of inertia and section modulus of one batten with respect to its own axis:

$$I_s = \frac{t_0 b^3}{12} = \frac{1 \cdot 17^3}{12} = 409,4167\ \text{cm}^4;$$

$$W_s = \frac{t_0 b^2}{6} = \frac{1 \cdot 17^2}{6} = 48,167\ \text{cm}^3.$$

2. Distance between chord axes:

$$b = B - 2z_0 = 30 - 2 \cdot 2,47 = 25,06\ \text{cm}.$$

3. Slenderness of one of the chords in the sections between the battens (in the clear):

$$\lambda_{1,y} = \frac{l_{1,y}}{i_{1,y}} = \frac{600}{10,871} = 55,193; \quad \bar{\lambda}_{1,y} = \lambda_{1,y}\sqrt{\frac{R_y}{E}} = 55,193\sqrt{\frac{240}{2,06 \cdot 10^5}} = 1,884 \le 5;$$

$$\lambda_{1,z} = \frac{l_{1,z}}{i_{1,z}} = \frac{s-b}{i_{1,z}} = \frac{112-17}{2,728} = 34,824 < 40; \quad \bar{\lambda}_{1,z} = \lambda_{1,z}\sqrt{\frac{R_y}{E}} = 34,824\sqrt{\frac{240}{2,06 \cdot 10^5}} = 1,1886 \le 5.$$

4. Column slenderness and respective conditional slenderness:

$$\lambda_y = \frac{l_{ef,y}}{i_y} = \frac{600}{10,871} = 55,193; \ \Rightarrow \ \bar{\lambda}_y = \lambda_y \sqrt{\frac{R_y}{E}} = 55,193\sqrt{\frac{240}{2,06 \cdot 10^5}} = 1,884 < 2,5;$$

$$\lambda_z = \frac{l_{ef,z}}{i_z} = \frac{600}{12,824} = 46,787.$$

5. Reduced and conditional reduced slenderness of the column with respect to the free axis:

When $\dfrac{I_s s}{I_b b} = \dfrac{409,4167 \cdot 112}{262 \cdot 25,06} = 6,984 > 5$:

$$\lambda_z = \lambda_{ef,z} = \sqrt{\lambda_z^2 + \lambda_{1,z}^2} = \sqrt{46,787^2 + 34,824^2} = 58,3244;$$

$$\bar{\lambda}_z = \lambda_z \sqrt{\frac{R_y}{E}} = 58,3244\sqrt{\frac{240}{2,06 \cdot 10^5}} = 1,991 < 2,5.$$

6. Buckling coefficients:

$$\varphi_y = 1 - \left(0,073 - 5,53\frac{R_y}{E}\right)\bar{\lambda}_y\sqrt{\bar{\lambda}_y} = 1 - \left(0,073 - 5,53\frac{240}{2,06 \cdot 10^5}\right) \cdot 1,884\sqrt{1,884} = 0,8279;$$

$$\varphi_z = 1 - \left(0,073 - 5,53\frac{R_y}{E}\right)\bar{\lambda}_z\sqrt{\bar{\lambda}_z} = 1 - \left(0,073 - 5,53\frac{240}{2,06 \cdot 10^5}\right) \cdot 1,991\sqrt{1,991} = 0,813.$$

7. Limit compressive forces causing the column buckling about the respective axes:

$$N_{b,y} = \varphi_y A R_y \gamma_c = 0,8279 \cdot 70,4 \cdot 24 \cdot 1 = 1398,82 \text{ kN};$$

$$N_{b,z} = \varphi_z A R_y \gamma_c = 0,813 \cdot 70,4 \cdot 24 \cdot 1 = 1373,645 \text{ kN}.$$

8. Conditional shear force Q_{fic}:

$$Q_{fic} = 7,15 \cdot 10^{-6}\left(2330 - \frac{E}{R_y}\right)\frac{N}{\varphi_z} = 7,15 \cdot 10^{-6}\left(2330 - \frac{2,06 \cdot 10^5}{240}\right)\frac{1400}{0,813} = 18,1198 \text{ kN}.$$

9. Force F, shearing the batten, and moment M_1, bending the batten in its plane:

$$F = \frac{Q_s s}{b} = \frac{Q_{fic} s}{2b} = \frac{18,1198 \cdot 112}{2 \cdot 25,06} = 40,4912 \text{ kN};$$

$$M_1 = \frac{Q_s s}{2} = \frac{Q_{fic} s}{4} = \frac{18,1198 \cdot 112}{4} = 507,3544 \text{ kNcm}.$$

10. Load-bearing capacity of the batten under bending:

$$W_s R_y \gamma_c = 48,167 \cdot 24 \cdot 1 = 1156,01 \text{ kNcm}.$$

11. Bending moment acting on the column chord and caused by the bending of the batten:

$$M_b = 2M_1 = 2 \cdot 507,3544 = 1014,7088 \text{ kNcm}.$$

12. Load-bearing capacity of the chord under bending in the batten plane:

$$W_{b,z,\min} R_y \gamma_c = 37,269 \cdot 24 \cdot 1 = 894,456 \text{ kNcm}.$$

13. Strength of the chord under the combined action of the longitudinal force and bending moment in the batten plane without taking the plasticity into account:

$$\frac{1}{R_y \gamma_c}\left(\frac{N}{A} + \frac{M_b}{W_{b,z,\min}}\right) = \frac{1}{24 \cdot 1}\left(\frac{1400}{70,4} + \frac{1014,7088}{37,269}\right) = 1,963.$$

14. Buckling coefficients for a chord:

$$\varphi_y = 1 - \left(0,073 - 5,53\frac{R_y}{E}\right)\bar{\lambda}_{1,y}\sqrt{\bar{\lambda}_{1,y}} = 1 - \left(0,073 - 5,53\frac{240}{2,06 \cdot 10^5}\right)1,884\sqrt{1,884} = 0,828;$$

$$\varphi_z = 1 - \left(0,073 - 5,53\frac{R_y}{E}\right)\lambda_{1,z}\sqrt{\lambda_{1,z}} = 1 - \left(0,073 - 5,53\frac{240}{2,06 \cdot 10^5}\right) \cdot 1,1886\sqrt{1,1886} = 0,914.$$

15. Load-bearing capacity of the chord under compression:

$$\varphi_y A R_y \gamma_c = 0,828 \cdot 35,2 \cdot 24 \cdot 1 = 699,398 \text{ kN};$$

$$\varphi_z A R_y \gamma_c = 0,914 \cdot 35,2 \cdot 24 \cdot 1 = 772,1472 \text{ kN}.$$

16. Relative eccentricity, cross-section shape coefficient, reduced eccentricity, and buckling coefficient under eccentric compression of the chord:

$$m_z = \frac{M_z}{N} \cdot \frac{A_b}{W_{b,z,\min}} = \frac{1014,7088}{700} \cdot \frac{35,2}{37,269} = 1,36911 \le 5;$$

$$\frac{A_f}{A_w} = \frac{16,2}{19,95} = 0,812;$$

$$\eta = (1,25 - 0,05 m_z) - 0,01(5 - m_z)\bar{\lambda}_{1,z} = (1,25 - 0,05 \cdot 1,36911) - 0,01(5 - 1,36911) \cdot 1,1886 = 1,13838$$

(for the section type 9 according to the table 73 of SNiP II-23-81* when $\frac{A_f}{A_w} = 0,5$);

$$\eta = (1,5 - 0,1 m_z) - 0,02(5 - m_z)\bar{\lambda}_{1,z} = (1,5 - 0,1 \cdot 1,36911) - 0,02(5 - 1,36911) \cdot 1,1886 = 1,27678$$

(for the section type 9 according to the table 73 of SNiP II-23-81* when $\frac{A_f}{A_w} = 1,0$);

$\eta = 1,2247$ (for the section type 9 according to the table 73 of SNiP II-23-81* when $\frac{A_f}{A_w} = 0,812$);

$\eta = 1,45 + 0,04 m_z = 1,45 + 0,04 \cdot 1,36911 = 1,50476$ (for the section type 11 according to the table 73 of SNiP II-23-81* when $\frac{A_f}{A_w} = 0,5$);

$\eta = 1,8 + 0,12 m_z = 1,8 + 0,12 \cdot 1,36911 = 1,9643$ (for the section type 11 according to the table 73 of SNiP II-23-81* when $\frac{A_f}{A_w} = 1,0$);

$\eta = 1,7915$ (for the section type 11 according to the table 73 of SNiP II-23-81* when $\frac{A_f}{A_w} = 1,33811$);

$$m_{z,ef} = \eta m_z = 1,7915 \cdot 1,36911 = 2,453;$$

$\varphi_e = 0,4174$ (according to the table 74 of SNiP II-23-81*).

17. Stability check of the chord in the batten bending plane:

$$\frac{N}{2\varphi_e A_b} = \frac{1400}{2 \cdot 0,4174 \cdot 35,2} = 47,6434 \text{ kN/cm}^2 > R_y \gamma_c = 24 \cdot 1 = 24 \text{ kN/cm}^2.$$

18. Stability check of the chord out of the batten bending plane is performed as a stability check of an axially compressed bar in the respective plane according to Sec. 5.32 of SNiP II-23-81*.

Comparison of solutions:

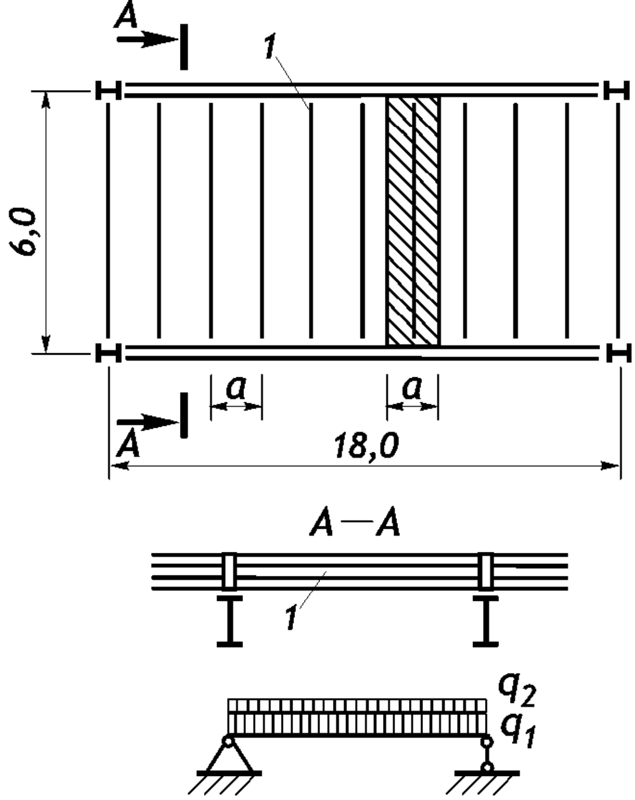
Factor	Source	Manual calculation	KRISTALL	Deviation from the manual calculation, %
General stability of a bar under axial compression in XoY plane	24/24=1	1400/1373,645 = 1,019	1,019	0,0

General stability of a bar under axial compression in XoZ plane	23,6/24=0,983	1400/1398,82 = 1,001	1,001	0,0
Resistance of a batten to bending	–	507,3544/1156,01 = 0,439	0,439	0,0
Strength under action of bending moment Mz	–	1014,7088/894,456 = 1,134	1,134	0,0
Strength under combined action of longitudinal force and bending moments, no plasticity	–	1,963	1,963	0,0
Stability of chord under compression in XoY plane	–	700/772,1472 = 0,9066	0,907	0,0
Stability of chord under compression in XoZ plane	23,6/24=0,983	700/699,398 = 1,001	1,001	0,0
Stability of chord in the moment M_z plane under eccentric compression	–	47,6434/24 = 1,985	1,985	0,0
Stability of chord out of the moment M_z plane under eccentric compression	–	24,01735/24 = 1,001	1,001	0,0
Limit slenderness in XoY plane	–	58,3244/120 = 0,486	0,486	0,0
Limit slenderness in XoZ plane	–	55,193/120 = 0,46	0,46	0,0

BEAMS

Strength and Stiffness Analysis of Stringers for a Normal Stub Girder System

Objective: Check the mode for calculating and selecting beams



1 – stringer

Task: Select a rolled I-beam for the stringers with a span of 6 m in a normal stub girder system. The top chord of the stringers is continuously restrained by the floor plate.

References: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 183.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
3.1.sav; report — Kristall3.1.doc

Initial data:

$a = 1{,}125$ m	Spacing of stringers;
$q_н$ = (0,77 + 20) kN/m^2 ×1,125 m = 23,37 kN/m	Total characteristic load;
q_1 = 1,05×0,77 kN/m^2 ×1,125 m = 0,91 kN/m	Design permanent load;
q_2 = 1,2×20 kN/m^2 ×1,125 m = 27 kN/m	Design temporary load;
R_y = 23 kN/cm^2,	Steel grade C235;
l = 6 m	Beam span;
$[f]$ = 1/250×6,0 m = 24 mm	Limit deflection;
γ_c = 1	Service factor ;
W_x = 596,364 cm^3	Selected I-beam No.33 GOST 8239-89;

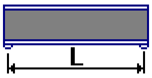
$I_x = 9840\ \text{cm}^4,\ S_x = 339\ \text{cm}^3,\ t_w = 7\ \text{mm}.$

KRISTALL parameters:
Steel: C235
Group of structures according to the table 50* of SNiP II-23-81* 4

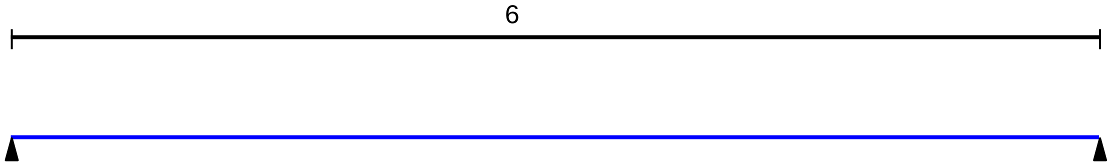
Importance factor $\gamma_n = 1$

Importance factor (serviceability limit state) = 1

Service factor 1



Structure:

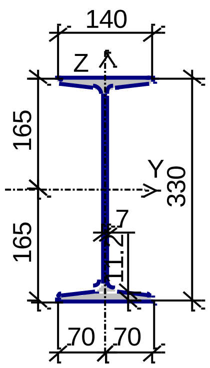


Restraints against lateral displacements and rotations:

	Left	Right
Displacement along Y	Restrained	Restrained
Displacement along Z	Restrained	Restrained
Rotation about Y		
Rotation about Z		

Continuous restraint of the compressed chord out of the bending plane

Section:



Profile: I-beam with sloped inner flange surfaces GOST 8239-89 33

Manual calculation:
1. Design bending moment and shear force:

$$M_{\max} = \frac{q_\Sigma l^2}{8} = \frac{(0.91+27)\cdot 6.0^2}{8} = 125.593\ \text{kNm};$$

$$Q_{\max} = \frac{q_{\Sigma} l}{2} = \frac{(0.91 + 27)\cdot 6.0}{2} = 83,73\,\text{kN}.$$

2. Necessary beam section modulus assuming that the deformations of steel are elastic:

$$W = \frac{M_{\max}}{R_y} = \frac{125.593\cdot 100}{23} = 546.057\ \text{cm}^3.$$

3. Maximum deflection occurring in the middle of the beam span:

$$f_{\max} = \frac{5}{384}\cdot\frac{q_{н} l^4}{EI_x} = \frac{5}{384}\cdot\frac{23,37\cdot 6^4}{2,06\cdot 10^5\cdot 10^3\cdot 9840\cdot 10^{-8}} = 19,46\,\text{mm}.$$

4. Check of the maximum shear stresses:

$$\tau_{\max} = \frac{Q_{\max} S_x}{I_x t_w} = \frac{83,73\cdot 339}{9840\cdot 0,7} = 4{,}12577\ \text{kN/cm}^2 < R_s\gamma_c = 0,58\cdot 23 = 13,34\ \text{kN/cm}^2.$$

Comparison of solutions:

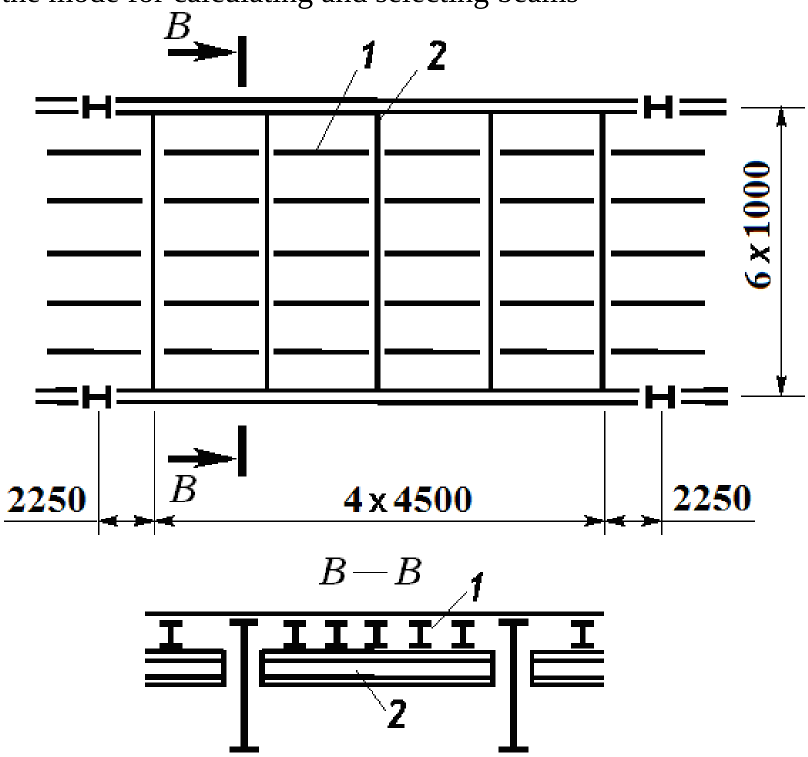
Factor	Strength under action of lateral force	Strength under action of bending moment	Stability of in-plane bending under moment	Maximum deflection
Manual calculation	4,126/13,34 = 0,309	546,06/596,36 = 0,916	–	19,46/24 = 0,81
KRISTALL	0,309	0,916	0,916	19,451/24 = 0,81
Deviation from the manual calculation, %	0,0	0,0	0,0	0,0
Source	–	0,83	–	0,81

Comments:

1. The check of the general stability of the beam was not performed in the manual calculation, because the compressed beam chord is restrained against lateral displacements out of the bending plane by a welded floor plate.
2. In the source the check of the beam strength was performed taking into account the development of the limited plastic deformations.
3. The check of the beam strength taking into account the development of the limited plastic deformations was not performed, because according to the codes this calculation is possible only when the beam web has stiffeners. In the initial data of the example the stringer was specified without any intermediate stiffeners.

Strength and Stiffness Analysis of Stringers for a Complex Stub Girder System

Objective: Check the mode for calculating and selecting beams



1 – stringer
2 – secondary beam

Task: Select a rolled I-beam for the stringers with a span of 4,5 m in a complex stub girder system. The top chord of the stringers is continuously restrained by the floor plate.

References: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 183.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
3.2.sav; report — Kristall3.2.doc

Initial data:

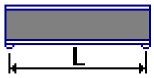
$a = 1{,}0$ m	Spacing of stringers;
$q_н = (0{,}77 + 20)$ kN/m^2 ×1 m = 20,77 kN/m	Total characteristic load;
$q_1 = 1{,}05{\times}0{,}77$ kN/m^2 ×1 m = 0,8085 kN/m	Design permanent load;
$q_2 = 1{,}2{\times}20$ kN/m^2 ×1 m = 24 kN/m	Design temporary load;
$R_y = 23$ kN/cm^2,	Steel grade C235;
$l = 4{,}5$ m	Beam span;
$[f] = 1/250{\times}4{,}5$ m = 18 mm	Limit deflection;
$\gamma_c = 1$	Service factor ;
$W_x = 288{,}33$ cm^3	Selected I-beam No.24 GOST 8239-89;
$I_x = 3460$ cm^4.	

KRISTALL parameters:
Steel: C235
Group of structures according to the table 50* of SNiP II-23-81* 4

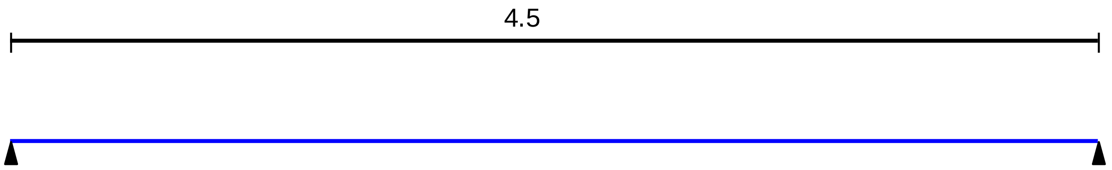
Importance factor $\gamma_n = 1$

Importance factor (serviceability limit state) = 1

Service factor 1



Structure:

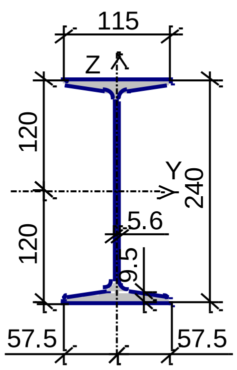


Restraints against lateral displacements and rotations:

	Left	**Right**
Displacement along Y	Restrained	Restrained
Displacement along Z	Restrained	Restrained
Rotation about Y		
Rotation about Z		

Continuous restraint of the compressed chord out of the bending plane

Section:



Profile: I-beam with sloped inner flange surfaces GOST 8239-89 24

Manual calculation:

1. Design bending moment acting in the beam span:

$$M_{\max} = \frac{q_\Sigma l^2}{8} = \frac{(0,8085 + 24)\cdot 4,5^2}{8} = 62,7965 \text{ kNm}.$$

2. Necessary beam section modulus assuming that the deformations of steel are elastic:

$$W = \frac{M_{\max}}{R_y} = \frac{62,7965 \cdot 100}{23} = 273,028 \text{ cm}^3.$$

3 Maximum deflection occurring in the middle of the beam span:

$$f_{\max} = \frac{5}{384} \cdot \frac{q_н l^4}{EI_x} = \frac{5}{384} \cdot \frac{20,77 \cdot 4,5^4}{2,06 \cdot 10^5 \cdot 10^3 \cdot 3460 \cdot 10^{-8}} = 15,56\,\text{mm}.$$

Comparison of solutions:

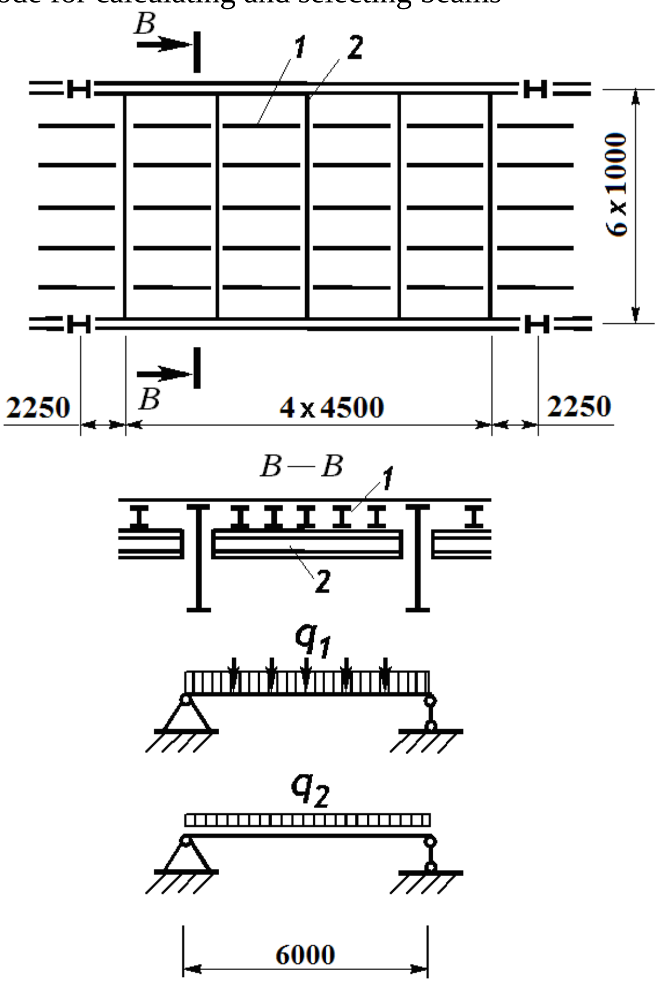
Factor	Strength under action of lateral force	Strength under action of bending moment	Stability of in-plane bending under moment	Maximum deflection
Manual calculation	–	273,028/288,33 = 0,947	–	15,56/18 = 0,864
KRISTALL	0,352	0,947	0,947	15,56/18 = 0,864
Deviation from the manual calculation, %	0,0	0,0	0,0	0,0
Source	–	0,858	–	0,87

Comments:

1. The check of tangential stresses was not performed in the manual calculation due to the absence of weakenings and a relatively large thickness of the beam webs.
2. The check of the general stability of the beam was not performed in the manual calculation, because the compressed beam chord is restrained against lateral displacements out of the bending plane by a welded floor plate.
3. In the source the check of the beam strength under the action of the bending moment was performed taking into account the development of the limited plastic deformations.
4. The check of the beam strength taking into account the development of the limited plastic deformations was not performed, because according to the codes this calculation is possible only when the beam web has stiffeners. In the initial data of the example the stringer was specified without any intermediate stiffeners.

Strength and Stiffness Analysis of Secondary Beams for a Complex Stub Girder System

Objective: Check the mode for calculating and selecting beams



1 – stringer
2 – secondary beam

Task: Select a rolled I-beam for the secondary beams with a span of 6 m in a complex stub girder system. The top chord of the secondary beams is restrained by the stringers arranged with a spacing of 1 m.

References: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 183.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
3.3.sav; report — Kristall3.3.doc

Initial data:

$a = 4{,}5$ m	Spacing of secondary beams;
$q_н$ = (0,77 + 27,3/102 + 20) kN/m^2 × 4,5 m = 94,67 kN/m	Total characteristic load;
q_1 = 1,05×(0,77 + 27,3/102) kN/m^2 × 4,5 m = 4,9 kN/m	Design permanent load;
q_2 = 1,2×20 kN/m^2 × 4,5 m = 108 kN/m	Design temporary load;
R_y = 23 kN/cm^2,	Steel grade C235;
l = 6,0 m	Beam span;

$[f] = 1/250 \times 6{,}0$ m = 24 mm — Limit deflection;

$\gamma_c = 1$ — Service factor ;

$W_y = 2034{,}98$ cm^3 — Selected I-beam No.55 GOST 8239-89;

$I_y = 55962$ cm^4.

KRISTALL parameters:

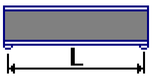
Steel: C235

Group of structures according to the table 50* of SNiP II-23-81* 4

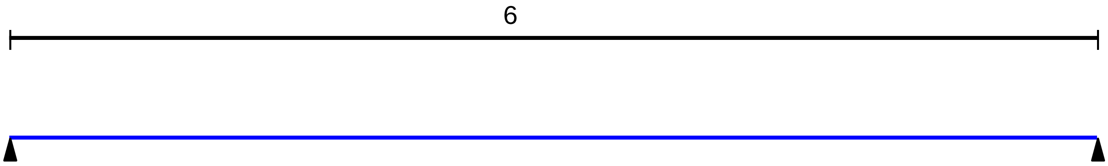
Importance factor $\gamma_n = 1$

Importance factor (serviceability limit state) = 1

Service factor 1

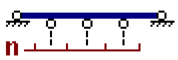


Structure:



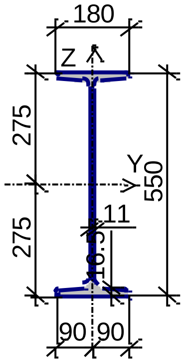
Restraints against lateral displacements and rotations:

	Left	Right
Displacement along Y	Restrained	Restrained
Displacement along Z	Restrained	Restrained
Rotation about Y		
Rotation about Z		

Restraints out of the bending plane 

n=6

Section:



Profile: I-beam with sloped inner flange surfaces GOST 8239-89 55

Manual calculation:

1. Design bending moment acting in the beam span:

$$M_{\max} = \frac{q_{\Sigma} l^2}{8} = \frac{(4,9+108)\cdot 6,0^2}{8} = 508,05\,\text{kNm}.$$

2. Necessary beam section modulus assuming that the deformations of steel are elastic:

$$W_{nes} = \frac{M_{\max}}{R_y} = \frac{508,05\cdot 100}{23} = 2208,913\,\text{cm}^3.$$

3. Maximum deflection occurring in the middle of the beam span:

$$f_{\max} = \frac{5}{384}\cdot\frac{q_{н} l^4}{EI_y} = \frac{5}{384}\cdot\frac{94,67\cdot 6,0^4}{2,06\cdot 10^5\cdot 10^3\cdot 55962\cdot 10^{-8}} = 13,858\,\text{mm}.$$

4. Conditional limit slenderness of the compressed beam chord:

$$\bar{\lambda}_{ub} = 0,35 + 0,0032\frac{b_f}{t_f} + \left(0,76 - 0,02\frac{b_f}{t_f}\right)\frac{b_f}{h_f} = 0,35 + 0,0032\frac{180}{16,5} + \left(0,76 - 0,02\frac{180}{16,5}\right)\frac{180}{533,5} = 0,5677$$

.

5. Conditional actual slenderness of the compressed beam chord:

$$\bar{\lambda}_b = \frac{l_{ef}}{b_f}\sqrt{\frac{R_y}{E}} = \frac{1000}{180}\sqrt{\frac{230}{2,06\cdot 10^5}} = 0,1856 < \bar{\lambda}_{ub} = 0,5677$$ – the stability check is not required.

Comparison of solutions:

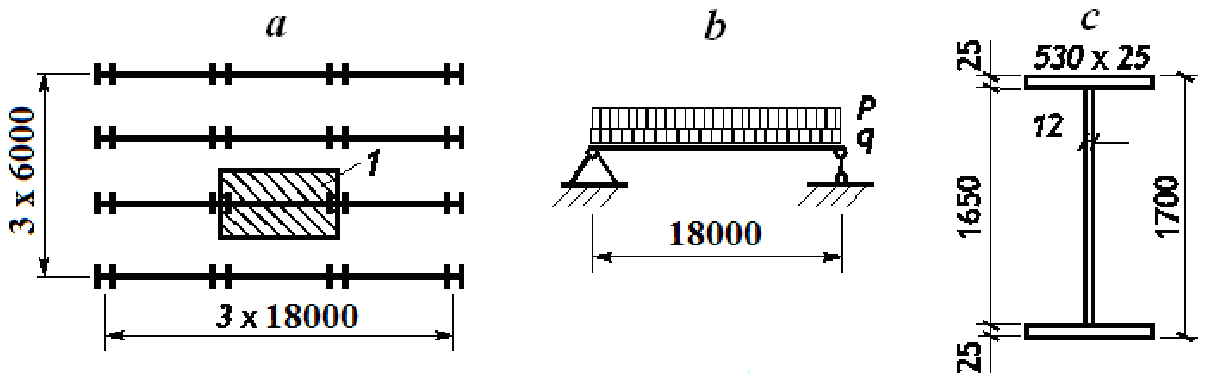
Factor	**Strength under action of lateral force**	**Strength under action of bending moment**	**Stability of in-plane bending under moment**	**Maximum deflection**
Manual calculation	–	2208,913/2034,98 =1,085	–	13,858/24 = 0,577
KRISTALL	0,488	1,085	1,085	13,856/24 = 0,577
Deviation from the manual calculation, %	0,0	0,0	0,0	0,0
Source	–	0,99	–	0,58

Comments:

1. The check of tangential stresses was not performed in the manual calculation due to the absence of weakenings and a relatively large thickness of the beam webs.
2. In the source the check of the beam strength under the action of the bending moment was performed taking into account the development of the limited plastic deformations.
3. The check of the beam strength taking into account the development of the limited plastic deformations was not performed, because according to the codes this calculation is possible only when the beam web has stiffeners. In the initial data of the example a rolled beam without intermediate stiffeners was selected for the secondary beam.

Strength and Stiffness Analysis of Main Beams of Complex Stub Girder Systems

Objective: Check the mode for calculating and selecting beams



a – floor plan; b – design model of the main beam; c – beam section;
1 – load area

Task: Select a welded I-beam for the main beams with a span of 18 m in a normal stub girder system. The top chord of the main beams is restrained by the stringers arranged with a spacing of 1 m.

References: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 192.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
3.4.sav; report — Kristall3.4.doc

Initial data:

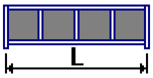
$a = 6$ m	Spacing of main beams;
$g_1 = 1{,}16$ kN/m^2	Weight of the floor plate and stringers;
$p = 20$ kN/m^2	Temporary (live) load;
$q_н = 127{,}099$ kN/m	Total characteristic load on the beam;
$q_1 = 1{,}05*1{,}16$ kN/m^2 $*$ 6 m$*1{,}02 = 7{,}454$ kN/m	Design permanent load; (coefficient 1,02 allows for the self-weight of the main beam);
$q_2 = 1{,}2*20$ kN/m^2 $*$ 6 m $= 144{,}0$ kN/m	Design live load;
$l = 18$ m	Main beam span;
$R_y = 23$ kN/cm^2	Steel grade C255 with thickness t>20 mm;
$R_s = 0{,}58*23 = 13{,}34$ kN/cm^2	
$[f] = l/400 = 45$ mm	Limit deflection;
$b_p \times t_p = 530 \times 20\,\text{mm}$	Section of the bearing stiffener;
$k_p = 6\,\text{mm}$	Fillet weld leg in a welded connection between a bearing stiffener and a beam;
$\gamma_c = 1$	Service factor ;
$W_y = 27153{,}85\,\text{cm}^3$	Geometric properties for a welded
$I_y = 2308077{,}083\,\text{cm}^4$	I-section with flanges 530×25 mm and a web 1650×12 mm;
$S_y = 15180{,}625\,\text{cm}^3$.	

KRISTALL parameters:
Steel: C255
Group of structures according to the table 50* of SNiP II-23-81* 3

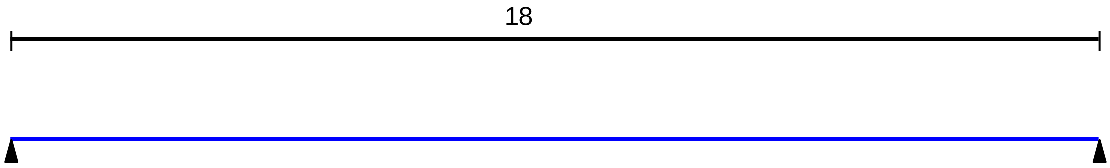
Importance factor γ_n = 1

Importance factor (serviceability limit state) = 1

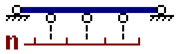
Service factor 1



Structure:

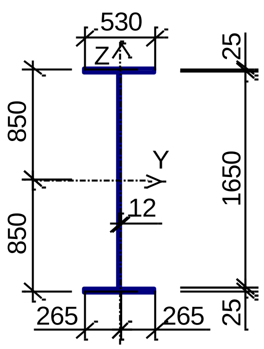


Restraints against lateral displacements and rotations:		
	Left	**Right**
Displacement along Y	Restrained	Restrained
Displacement along Z	Restrained	Restrained
Rotation about Y		
Rotation about Z		

Restraints out of the bending plane  n=17

Leg of girth welds – 8 mm
Leg of welds that attach the bearing stiffener – 6 mm

Section:



Manual calculation (SNiP II-23-81*):
1. Maximum bending moment and shear force acting in the design sections of the beam:

$$M_{\max}=\frac{q_{\Sigma}l^2}{8}=\frac{(7,454+144)\cdot 18,0^2}{8}=6133,887\text{ kNm.}$$

$$Q_{\max}=\frac{q_{\Sigma}l}{2}=\frac{(7,454+144)\cdot 18,0}{2}=1363,086\text{ kN.}$$

2. Necessary beam section modulus:

$$W_{nes}=\frac{M_{\max}}{R_y\gamma_c}=\frac{6133,887\cdot 100}{23}=26669,074\text{ cm}^3.$$

3. Maximum tangential stresses in the support section of the beam:

$$\tau_{\max}=\frac{Q_{\max}S_y}{I_yt_w}=\frac{1363,086\cdot 15180,625}{2308077,083\cdot 1,2}=7,471\text{ kN/cm}^2.$$

4. Maximum deflection occurring in the middle of the beam span:

$$f_{\max}=\frac{5}{384}\cdot\frac{q_н l^4}{EI_y}=\frac{5}{384}\cdot\frac{127,099\cdot 18,0^4}{2,06\cdot 10^5\cdot 10^3\cdot 2308077,083\cdot 10^{-8}}=36,539\text{ mm.}$$

5. Conditional limit slenderness of the compressed beam chord:

$$\bar{\lambda}_{ub}=0,35+0,0032\frac{b_f}{t_f}+\left(0,76-0,02\frac{b_f}{t_f}\right)\frac{b_f}{h_f}=0,35+0,0032\frac{530}{25}+\left(0,76-0,02\frac{530}{25}\right)\frac{530}{1675}=0,524$$

.

6. Conditional actual slenderness of the compressed beam chord:

$$\bar{\lambda}_b=\frac{l_{ef}}{b_f}\sqrt{\frac{R_y}{E}}=\frac{1000}{530}\sqrt{\frac{230}{2,06\cdot 10^5}}=0,063<\bar{\lambda}_{ub}=0,524$$ – the stability check is not required.

7. Conditional slenderness of the overhang of the compressed beam flange:

$$\bar{\lambda}_f=\frac{b_{ef}}{t_f}\sqrt{\frac{R_y}{E}}=\frac{b_f-t_w}{2t_f}\sqrt{\frac{R_y}{E}}=\frac{530-12}{2\cdot 25}\sqrt{\frac{230}{2,06\cdot 10^5}}=0,346<\bar{\lambda}_{uf}=0,5.$$

8. Strength of the bearing stiffener at the bearing of its end surface ($R_{un}=370$ MPa, $R_p=\frac{370}{1,025}=360,98$ MPa (see Table 1*)):

$$N_p=A_pR_p=53,0\cdot 2\cdot 36,098=3826,388\text{ kN.}$$

9. Reduced area, moment of inertia and slenderness of the bearing stiffener in the analysis of its stability:

$$A_{red}=b_pt_p+0,65t_w^2\sqrt{\frac{E}{R_y}}=53,0\cdot 2,0+0,65\cdot 1,2^2\sqrt{\frac{2,06\cdot 10^5}{230}}=134,012\text{ cm}^2;$$

$$I_p=\frac{1}{12}\left(t_pb_p^3+0,65t_w^4\sqrt{\frac{E}{R_y}}\right)=\frac{1}{12}\left(2,0\cdot 53,0^3+0,65\cdot 1,2^4\sqrt{\frac{2,06\cdot 10^5}{230}}\right)=24816,1948\text{ cm}^4.$$

$$\lambda_p=l_{ef}\sqrt{\frac{A_{red}}{I_p}}=(165+2,5)\cdot\sqrt{\frac{134,012}{24816,1948}}=12,309;$$

$$\bar{\lambda}_p=\lambda_p\sqrt{\frac{R_y}{E}}=12,309\cdot\sqrt{\frac{230}{2,06\cdot 10^5}}=0,411.$$

10. Buckling coefficient of the bearing stiffener of the beam:

$$\varphi=1-\left(0,073-5,53\frac{R_y}{E}\right)\bar{\lambda}_p\sqrt{\bar{\lambda}_p}=1-\left(0,073-5,53\cdot\frac{230}{2,06\cdot 10^5}\right)0,411\sqrt{0,411}=0,9824.$$

11. Load-bearing capacity of the bearing stiffener from the condition of providing its stability:

$$N_{p,b}=\varphi A_{red}R_y=0,9824\cdot 134,012\cdot 23,0=3028,028\text{ kN.}$$

12. Load-bearing capacity of the fillet welds attaching the bearing stiffener to the beam web:

$$N_f = 2\beta_f k_f l_f R_{wf}\gamma_{wf} = 2\beta_f k_f\left(85\beta_f k_f\right)R_{wf}\gamma_{wf} = 2\cdot 0,7\cdot 0,6\cdot\left(85\cdot 0,7\cdot 0,6\right)\cdot 18,0\cdot 1,0 = 539,784\,\text{kN}.$$

13. Load-bearing capacity per unit length of fillet welds attaching the beam flanges to the web:

$$N_f = 2\beta_f k_f R_{wf}\gamma_{wf} = 2\cdot 0,7\cdot 0,8\cdot 18,0\cdot 1,0 = 20,16\,\text{kN/cm}.$$

14. Shear force per unit length acting on the fillet welds attaching the beam flanges to the web:

$$T = \frac{Q_{\max} S_{yf}}{I_y} = \frac{1363,086\cdot 53,0\cdot 2,5\cdot 83,75}{2308077,083} = 6,5535\ \text{kN/cm}.$$

Comparison of solutions:

Factor	Source	Manual calculation	KRISTALL	Deviation, %
Stability of bearing stiffener	–	1363,086/3028,028 = 0,450	0,45	0,0
Bearing stiffener in bearing	–	1363,086/3826,388 = 0,356	0,357	0,0
Strength of girth weld	–	6,5535/20,16 = 0,325	0,315	1,23%
Strength of bearing stiffener weld	–	1363,086/539,784 = 2,525	2,525	0,0
Strength under action of lateral force	0,617	7,471/13,34 = 0,56	0,56	0,0
Strength under action of bending moment	1,0	26669,074/27153,85 =0,982	0,982	0,0
Stability of in-plane bending under moment	–	–	0,982	0,0
Local stability of web	–	–	0,6	0,0
Local stability of chord overhang	0,71	0,346/0,5 = 0,692	0,692	0,0
Maximum deflection	–	36,539/45 = 0,812	0,812	0,0

Comments:

1. In the source the check of the tangential stresses was performed according to the approximate formula.
2. The check of the local stability of the chord overhang performed in the source is incorrect.

TRUSS MEMBER

Analysis of a Top Truss Chord from Unequal Angles

Objective: Check the mode for calculating truss members

Task: Check the top truss chord section from two unequal angles L160x100x9. The truss panel length is 2,58 m. The top truss chord is restrained out of the plane through the panel.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 280.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
7.1.sav; report — Kristall-7.1.doc

Initial data:

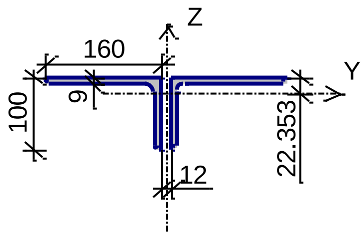
N = 535 kN	Design compressive force;
R_y = 24 kN/cm^2	Steel grade C245;
γ_c = 0,95	Service factor ;
g = 12 mm	Thickness of the gusset plate;
l_x = 2,58 m, l_y = 5,16 m	Effective lengths of the bar;
i_x = 2,851 cm, A = 45,74 cm^2	Geometric properties of
i_y = 7,745 cm	the top chord section from two angles
160x100x9.	

KRISTALL parameters:
Steel: C245
Group of structures according to the table 50* of SNiP II-23-81* 3

Importance factor 1
Type of element – Truss member
Panel length 2.58 m

Length between out-of-plane restraints – 5.16 m

Section:



Profile: Unequal angle GOST 8510-86* L160x100x9

Manual calculation (SNiP II-23-81*):
1. Strength check

$$\frac{N}{A} = \frac{535}{45,74} = 11,69655 \text{ kN/cm}^2 < R_y \gamma_c = 24 \cdot 0,95 = 22,8 \text{ kN/cm}^2.$$

2. Slenderness of the truss member:

$$\lambda_x = \frac{l_{ef,x}}{i_x} = \frac{2,58 \cdot 100}{2,851} = 90,49456;$$

$$\bar{\lambda}_y = \frac{l_{ef,y}}{i_y} = \frac{5,16 \cdot 100}{7,745} = 66,6236.$$

3. Conditional slenderness of the truss member:

$$\bar{\lambda}_x = \frac{l_{ef,x}}{i_x}\sqrt{\frac{R_y}{E}} = \frac{2,58 \cdot 100}{2,851}\sqrt{\frac{240}{2,06 \cdot 10^5}} = 3,0888;$$

$$\bar{\lambda}_y = \frac{l_{ef,y}}{i_y}\sqrt{\frac{R_y}{E}} = \frac{5,16 \cdot 100}{7,745}\sqrt{\frac{240}{2,06 \cdot 10^5}} = 2,274.$$

4. Buckling coefficients:

$$\varphi_x = 1,47 - 13,0\frac{R_y}{E} - \left(0,371 - 27,3\frac{R_y}{E}\right)\bar{\lambda}_x + \left(0,0275 - 5,53\frac{R_y}{E}\right)\bar{\lambda}_x^2 =$$

$$= 1,47 - \frac{13,0 \cdot 240}{2,06 \cdot 10^5} - \left(0,371 - \frac{27,3 \cdot 240}{2,06 \cdot 10^5}\right) \cdot 3,0888 + \left(0,0275 - \frac{5,53 \cdot 240}{2,06 \cdot 10^5}\right) \cdot 3,0888^2 = 0,60805$$

$$\varphi_y = 1 - \left(0,073 - 5,53\frac{R_y}{E}\right)\bar{\lambda}_y\sqrt{\bar{\lambda}_y} = 1 - \left(0,073 - \frac{5,53 \cdot 240}{2,06 \cdot 10^5}\right) \cdot 2,274\sqrt{2,274} = 0,77176.$$

5. Strength of the truss member from the condition of providing the general stability under axial compression:

$$N_{b,x} = \varphi_x A R_y \gamma_c = 0,60805 \cdot 45,74 \cdot 24 \cdot 0,95 = 634,118 \text{ kN};$$

$$N_{b,y} = \varphi_y A R_y \gamma_c = 0,77176 \cdot 45,74 \cdot 24 \cdot 0,95 = 804,847 \text{ kN}.$$

6. Limit slenderness of the truss member:

$$[\lambda]_x = 180 - 60\alpha_x = 180 - 60 \cdot \frac{N}{\varphi_x A R_y \gamma_c} = 180 - 60 \cdot \frac{535}{634,118} = 129,3785;$$

$$[\lambda]_y = 180 - 60\alpha_y = 180 - 60 \cdot \frac{N}{\varphi_y A R_y \gamma_c} = 180 - 60 \cdot \frac{535}{804,847} = 140,1166.$$

Comparison of solutions:

Factor	Source	Manual calculation	KRISTALL	Deviation from the manual calculation, %
Strength of member	535/45,8/22,8=0,512	11,6966/22,8 = 0,513	0,513	0,0
Stability of member in the truss plane	21,4/22,8=0,938	535/634,118 = 0,844	0,844	0,0
Stability of member out of the truss plane	not defined	535/804,847 = 0,665	0,665	0,0
Slenderness of member	not defined	90,4946/129,3785 = 0,7 66,6236/140,1166 = 0,4755	0,7	0,0

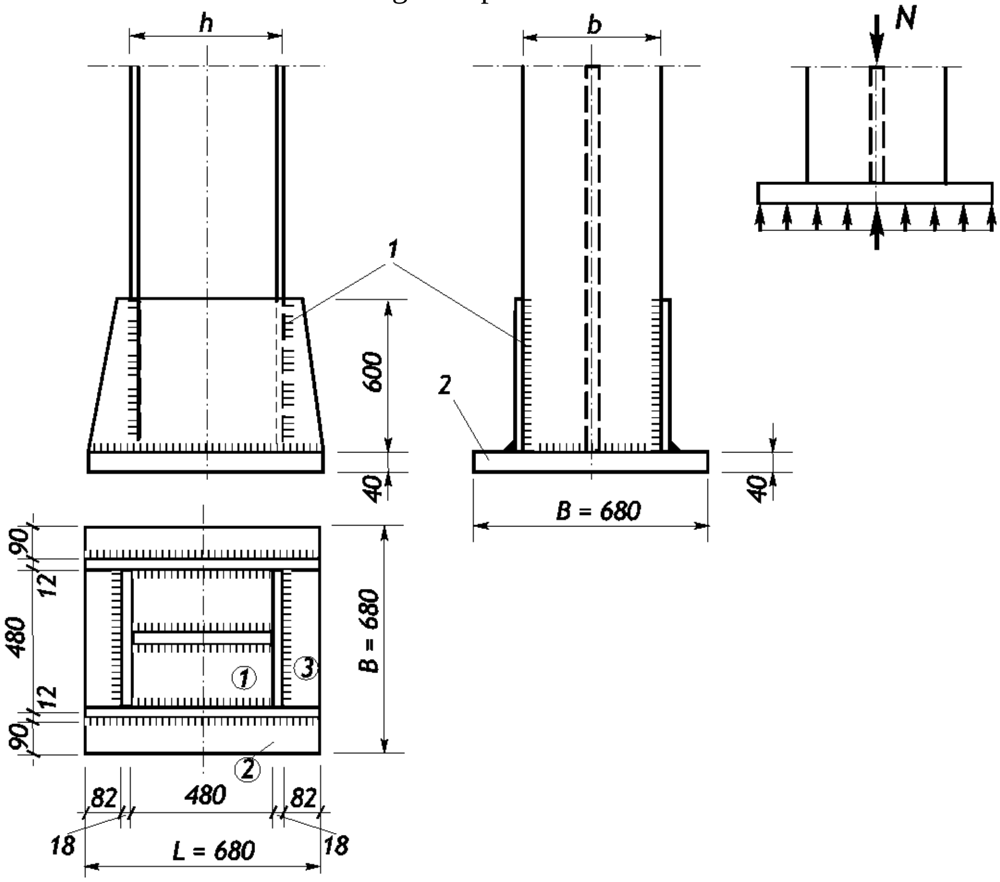
Comments:

1. In the source the buckling coefficient for the conditional slenderness of the bar of 3.09 was mistakenly taken as 0.546 instead of 0.6081, which caused the differences in the results of the stability analysis.
2. When checking the slenderness of the truss member the value of the factor was taken as the larger one calculated for the slenderness of the element in two principal planes of inertia of the section.

BASE PLATES

Analysis of the Base of a Solid I-beam Column

Objective: Check the mode for calculating base plates



Design models of the column base (figures in the circles indicate the numbers of the design sections of the base plate: 1 – wing plate; 2 – base plate)

Task: Check the load-bearing capacity of the base plate for the design section No.1.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 259.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

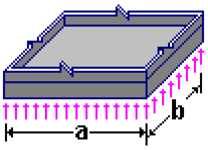
Initial data file:
6.1.sav; report — Kristall-6.1.doc

Initial data:

σ_f = 1,19 kN/cm^2 = 11,9 MPa	Stress under the base plate
R_y = 30 kN/cm^2	Steel grade C345
b/a = 480 mm / 234 mm	Dimensions of the design section of the base plate

KRISTALL parameters:
Steel: C345 category 1
Group of structures according to the table 50* of SNiP II-23-81* 3

Importance factor 1
Service factor 1,15



a = 0.48 m
b = 0.234 m
Plate thickness = 4 cm
Load 11.9 N/mm^2

Manual calculation (SNiP II-23-81*):

1. Design bending moment acting in the design section of the base plate:

$$M = \alpha \sigma_f a^2 = 0{,}125 \cdot 1{,}19 \cdot 23{,}4^2 = 81{,}45 \text{ kN/cm.}$$

2. Check the bending strength of the base plate ($\gamma_c = 1{,}15$ – according to the table 6* of SNiP II-23-81*):

$$\frac{6M}{t_p^2} = \frac{6 \cdot 81{,}45}{4^2} = 30{,}5436 \text{ kN/cm}^2 < R_y \gamma_c = 30 \cdot 1{,}15 = 34{,}5 \text{ kN/cm}^2.$$

Comparison of solutions:

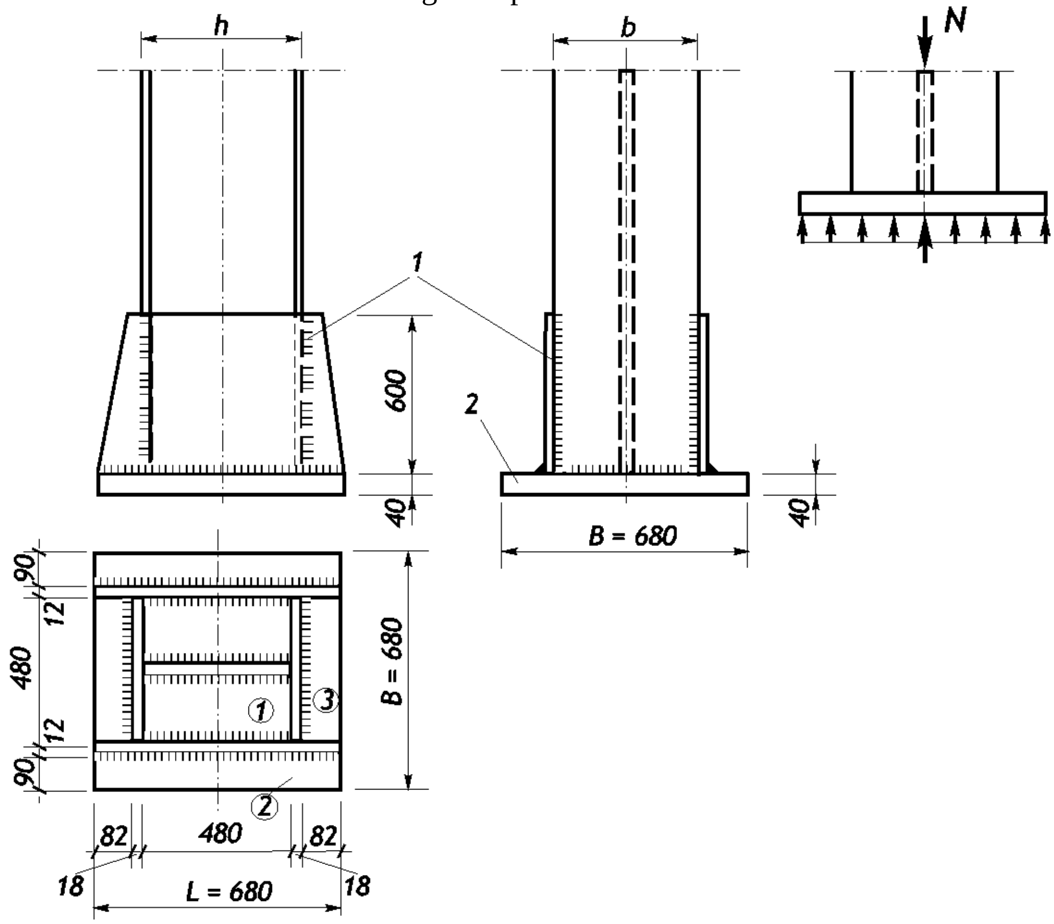
Factor	**Source**	**Manual calculation**	**KRISTALL**	**Deviation from the manual calculation, %**
for bending strength of the plate	4/4=1	30,5436/34,5 = 0,885	0,885	0,0

Comments:

The service factor of the base plate according to the table 6* of SNiP II-23-81* is not taken into account in the source.

Analysis of the Base of a Solid I-beam Column

Objective: Check the mode for calculating base plates



Design models of the column base (figures in the circles indicate the numbers of the design sections of the base plate: 1 – wing plate; 2 – base plate)

Task: Check the load-bearing capacity of the base plate for the design section No.2.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 259.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
6.2.sav; report — Kristall-6.2.doc

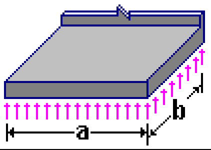
Initial data:

σ_f = 1,19 kN/cm^2 = 11,9 MPa	Stress under the base plate
R_y = 30 kN/cm^2	Steel grade C345
b/a = 90 mm / 680 mm	Dimensions of the design section of the base plate

KRISTALL parameters:
Steel: C345 category 1
Group of structures according to the table 50* of SNiP II-23-81* 3

Importance factor 1

Service factor 1,15

	a = 0.68 m b = 0.09 m Plate thickness = 4 cm Load 11.9 N/mm^2

Manual calculation (SNiP II-23-81*):

1. Design bending moment acting in the design section of the base plate:

$$M = 0,5\sigma_f c^2 = 0,5 \cdot 1,19 \cdot 9,0^2 = 48,195\,\text{kN/cm}.$$

2. Check the bending strength of the base plate ($\gamma_c = 1,15$ – according to the table 6* of SNiP II-23-81*):

$$\frac{6M}{t_p^2} = \frac{6 \cdot 48,195}{4^2} = 18,073125 \text{ kN/cm}^2 < R_y \gamma_c = 30 \cdot 1,15 = 34,5 \text{ kN/cm}^2.$$

Comparison of solutions:

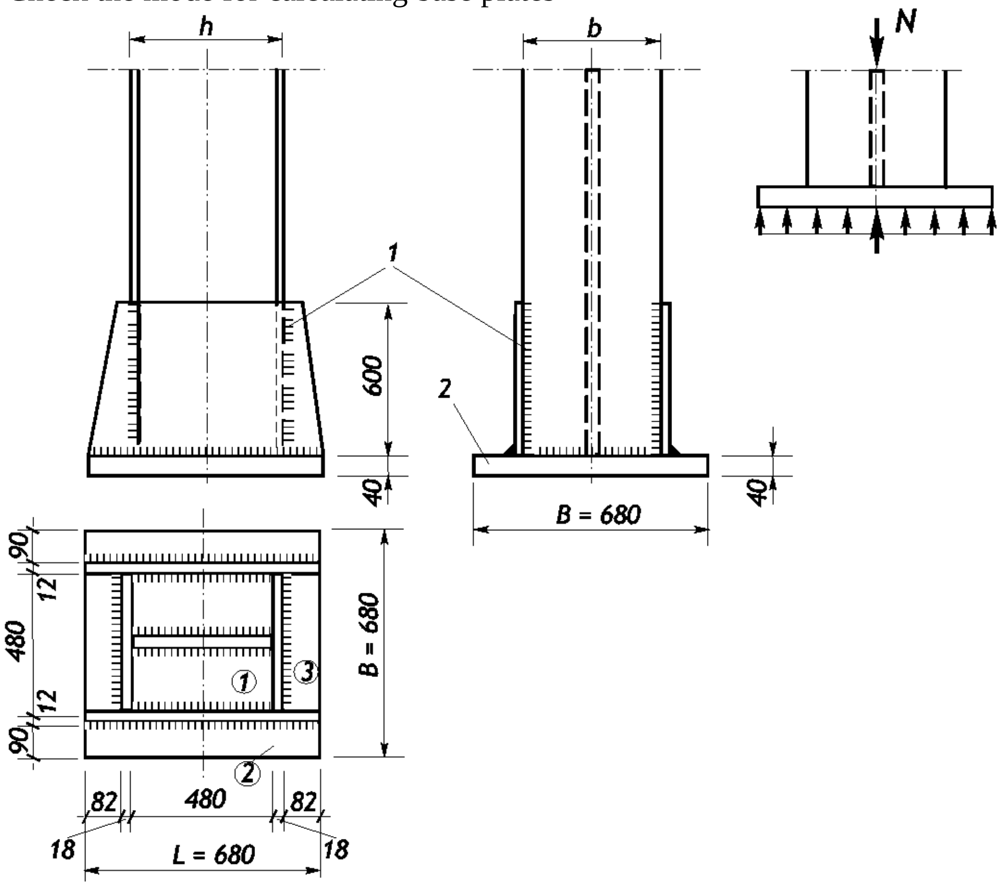
Factor	Source	Manual calculation	KRISTALL	Deviation from the manual calculation, %
for bending strength of the plate	48,2/81,45=0,592	18,073/34,5 = 0,524	0,524	0,0

Comments:

The service factor of the base plate according to the table 6* of SNiP II-23-81* is not taken into account in the source.

Analysis of the Base of a Solid I-beam Column

Objective: Check the mode for calculating base plates



Design models of the column base (figures in the circles indicate the numbers of the design sections of the base plate: 1 – wing plate; 2 – base plate)

Task: Check the load-bearing capacity of the base plate for the design section No.3.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 259.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

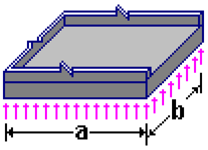
Initial data file:
6.3.sav; report — Kristall-6.3.doc

Initial data:

σ_f = 1,19 kN/cm^2 = 11,9 MPa	Stress under the base plate
R_y = 30 kN/cm^2	Steel grade C345
b/a = 480 mm / 82 mm plate	Dimensions of the design section of the base

KRISTALL parameters:
Steel: C345 category 1
Group of structures according to the table 50* of SNiP II-23-81* 3

Importance factor 1
Service factor 1,15



a = 0.082 m
b = 0.48 m
Plate thickness = 4 cm
Load 11.9 N/mm^2

Manual calculation (SNiP II-23-81*):

1. Design bending moment acting in the design section of the base plate:

$$M = 0,5\sigma_f c^2 = 0,5 \cdot 1,19 \cdot 8,2^2 = 40,0078 \text{ kN/cm.}$$

2. Check the bending strength of the base plate ($\gamma_c = 1,15$ – according to the table 6* of SNiP II-23-81*):

$$\frac{6M}{t_p^2} = \frac{6 \cdot 40,0078}{4^2} = 15,002925 \text{ kN/cm}^2 < R_y \gamma_c = 30 \cdot 1,15 = 34,5 \text{ kN/cm}^2.$$

Comparison of solutions:

Factor	Source	Manual calculation	KRISTALL	Deviation from the manual calculation, %
for bending strength of the plate	40/81,45=0,491	15,0029/34,5 = 0,43487	0,435	0,0

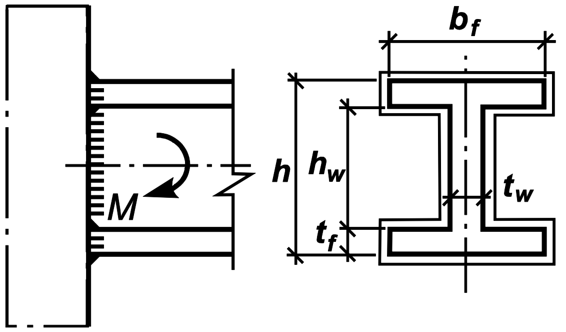
Comments:

The service factor of the base plate according to the table 6* of SNiP II-23-81* is not taken into account in the source.

WELDED CONNECTIONS

Analysis of a Welded Connection with Fillet Welds for a Bending Moment

Objective: Check the mode for calculating welded connections



b_f=18 cm; t_f=0,8 cm; t_w=0,6 cm; h_w=24 cm; h=25,6 cm;

Task: Check the welded connection with fillet welds for a bending moment

References: Manual to SNiP II-23-81. Welded Connections. 1984. p. 28-29.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, SP 16.13330.2017, DBN B.2.6-163:2010, DBN B.2.6-198:2014.

Initial data file:
1.1.sav; report — Kristall1.1.doc

Initial data:

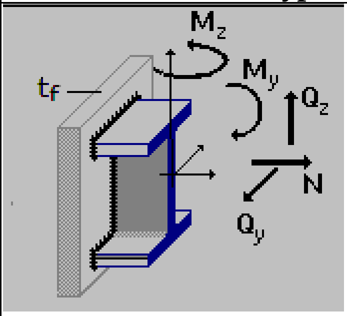
M = 75 kNm	Bending moment
R_{yn} = 345 MPa, R_{un} = 490 MPa	Steel 15HSND
R_{wf} = 215 MPa, β_f = 0,9	Flat CO2 semiautomatic welding with a 2 mm diameter Sv-08G2S wire
$\gamma_{wf} = \gamma_c = 1$	Service factors

KRISTALL parameters:
Steel: C345 category 3

Importance factor	1
Service factor	1
Group of structures according to the table 50* of SNiP II-23-81*	1

Properties of welding materials:	
Characteristic resistance of the weld metal based on the ultimate strength, R_{wun}	49949,032 T/m^2
Design resistance of the fillet welds for shear in the weld metal, R_{wf}	21916,412 T/m^2
Type of welding	Automatic and semiautomatic, diameter of the electrode wire not

Properties of welding materials:	
	less than 1.4-2.0 mm
Position of weld	Flat
Climatic region	with temperature t > -40°C

Type:	***Parameters:***
 Section - Full assortment of GOST profiles.. Wide flange I-beam GOST 26020-83 26W1	t_f = 6 mm Leg of weld near flange = 4 mm Leg of weld near web = 4 mm

Internal forces and moments:

N = 0 N

M_y = 75000 Nm

Q_z = 0 N

M_z = 0 Nm

Q_y = 0 N

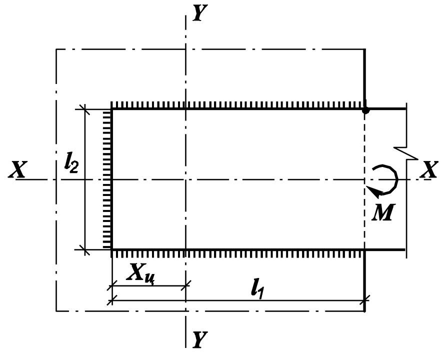
Comparison of solutions:

Factor	Strength of the weld metal
Source	208/215 = 0,967
KRISTALL	0,951
Deviation, %	1,655

Comments:

Deviations in the results of the calculation are due to the differences in the initial data. In this mode KRISTALL enables to select an I-section only from the assortment of rolled profiles. The I-section specified in theoretical solution has cross-sectional dimensions which do not have an exact assortment analog. When checking the connection with the help of KRISTALL, the closest I-section was selected – I-beam 26W1 GOST 26020-83.

Analysis of a Welded Connection with Fillet Welds for a Bending Moment Acting in the Weld Plane



l_1=30 cm; l_2=20 cm

Objective: Check the mode for calculating welded connections

Task: Check the welded connection with fillet welds. The connection is loaded with a bending moment acting in the weld plane.

References: Manual to SNiP II-23-81. Welded Connections. 1984. p. 29 – 30.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, SP 16.13330.2017, DBN B.2.6-163:2010, DBN B.2.6-198:2014.

Initial data file:
1. when the weld leg is k_f = 10 mm: 1.2-1.sav; report — Kristall1.2-1.doc
2. when the weld leg is k_f = 6 mm: 1.2-2.sav; report — Kristall1.2-2.doc

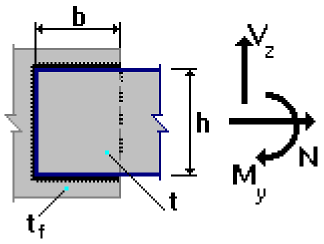
Initial data:

M = 55 kNm	Bending moment
R_{un} = 370 MPa	Steel VSt3
R_{wf} = 200 MPa, β_f = 0,7	Welding with coated E46 electrodes
$\gamma_{wf} = \gamma_c$ = 1	Service factors

KRISTALL initial data when the weld leg is 10 mm:
Steel: C255

Importance factor	1
Service factor	1
Group of structures according to the table 50* of SNiP II-23-81*	1

Properties of welding materials:	
Characteristic resistance of the weld metal based on the ultimate strength, R_{wun}	45871,56 T/m^2
Design resistance of the fillet welds for shear in the weld metal, R_{wf}	20387,36 T/m^2
Type of welding	Manual
Position of weld	Flat
Climatic region	with temperature t > -40°C

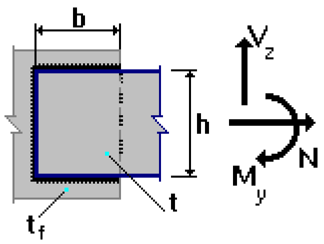
Type:	*Parameters:*
	Weld leg = 10 mm b = 300 mm h = 200 mm t = 10 mm t_f = 10 mm

Internal forces and moments:
N = 0 kN
M_y = 55 kNm
Q_z = 0 kN

KRISTALL initial data when the weld leg is 6 mm:
Steel: C255

Importance factor	1
Service factor	1
Group of structures according to the table 50* of SNiP II-23-81*	1

Properties of welding materials:	
Characteristic resistance of the weld metal based on the ultimate strength, R_{wun}	45871,56 T/m^2
Design resistance of the fillet welds for shear in the weld metal, R_{wf}	20387,36 T/m^2
Type of welding	Manual
Position of weld	Flat
Climatic region	with temperature t > -40°C

Type:	*Parameters:*
	Weld leg = 6 mm b = 300 mm h = 200 mm t = 10 mm t_f = 10 mm

Internal forces and moments:
N = 0 kN
M_y = 55 kNm
Q_z = 0 kN

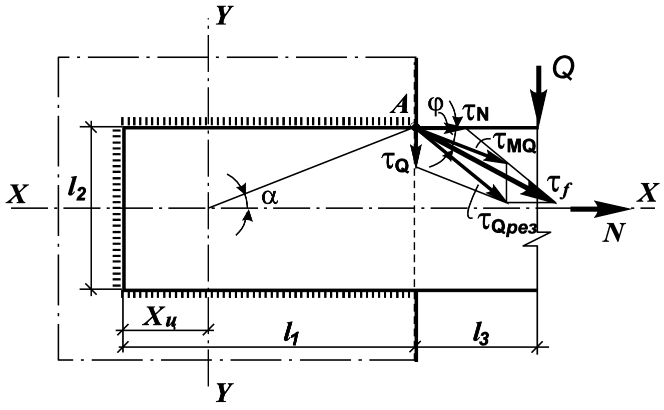
Comparison of solutions:

Weld leg, mm	10	6
Check	of the weld metal	of the weld metal

Source	117/200 = 0,58	199/200 = 0,995
KRISTALL	0,555	0,937
Deviation, %	4,31	5,8

Analysis of a Welded Connection with Fillet Welds at the Simultaneous Action of Longitudinal and Lateral Forces

Objective: Check the mode for calculating welded connections



l_1=30 cm; l_2=20 cm; l_3=81 cm

Task: Check the welded connection with fillet welds. The connection is loaded with longitudinal and lateral forces.

References: Manual to SNiP II-23-81. Welded Connections. 1984. p. 30 – 33.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, SP 16.13330.2017, DBN B.2.6-163:2010, DBN B.2.6-198:2014.

Initial data file:
1. when the weld leg is k_f = 10 mm: 1.3-1.sav; report — Kristall1.3-1.doc
2. when the weld leg is k_f = 5 mm: 1.3-1.sav; report — Kristall1.3-2.doc

Initial data:

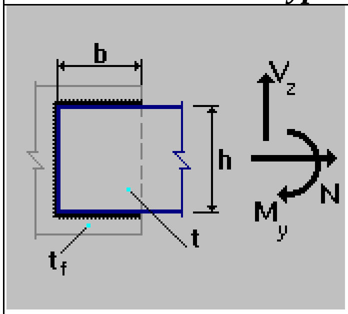
N = 100 kN	Longitudinal force
Q = 38 kN	Lateral force
R_{un} = 370 MPa	Steel VSt3
R_{wf} = 200 MPa, β_f = 0,7	Welding with coated E46 electrodes
$\gamma_{wf} = \gamma_c$ = 1	Service factors

KRISTALL initial data when the weld leg is 10 mm:
Steel: C255

Importance factor	1
Service factor	1
Group of structures according to the table 50* of SNiP II-23-81*	1

Properties of welding materials:	
Characteristic resistance of the weld metal based on the ultimate strength, R_{wun}	45871.56 T/m^2
Design resistance of the fillet welds for shear in the weld metal, R_{wf}	20387.36 T/m^2
Type of welding	Manual
Position of weld	Flat

Properties of welding materials:	
Climatic region	with temperature $t > -40°C$

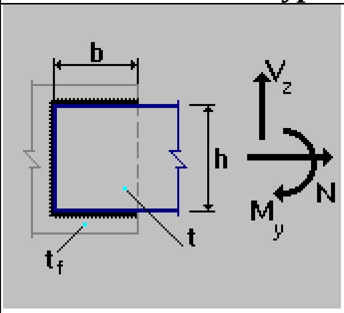
Type:	*Parameters:*
	Weld leg = 10 mm b = 300 mm h = 200 mm t = 10 mm t_f = 10 mm

Internal forces and moments:
N = 100 kN
M_y = 30.78 kNm
Q_z = -38 kN

KRISTALL initial data when the weld leg is 5 mm:
Steel: C255

Importance factor	1
Service factor	1
Group of structures according to the table 50* of SNiP II-23-81*	1

Properties of welding materials:	
Characteristic resistance of the weld metal based on the ultimate strength, R_{wun}	45871.56 T/m^2
Design resistance of the fillet welds for shear in the weld metal, R_{wf}	20387.36 T/m^2
Type of welding	Manual
Position of weld	Flat
Climatic region	with temperature $t > -40°C$

Type:	*Parameters:*
	Weld leg = 5 mm b = 300 mm h = 200 mm t = 10 mm t_f = 10 mm

Internal forces and moments:
N = 100 kN
M_y = 30.78 kNm
Q_z = -38 kN

Comparison of solutions:

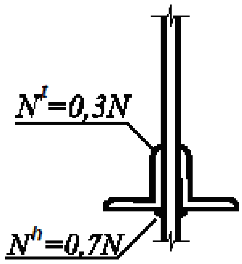
Weld leg, mm	10	5
Check	of the weld metal	of the weld metal
Source	96,2/200 = 0,48	198/200 = 0,99
KRISTALL	0,461	0,941
Deviation, %	3,95	4,9

Comments:

The difference in the results is due to the inaccuracy made by the authors of the example in the design section of the weld. It should be also noted, that in the problem the force Q is transferred through the cantilever of 81 cm, therefore the moment M = 38 kN *0,81 m = 30,78 kNm is additionally specified in KRISTALL.

Analysis of a Connection between a Bar in Tension from Two Angles and a Gusset Plate

Objective: Check the mode for calculating welded connections.



Task: Check the connection between a steel bar in tension from two equal angles and a gusset plate.

References: Moskalev N.S., Pronosin J.A. Steel Structures. Handbook / M.: ASV Publishing House, 2010. p. 83 – 84.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, SP 16.13330.2017, DBN B.2.6-163:2010, DBN B.2.6-198:2014.

Initial data:

R_{un} = 490 MPa	Steel C345
Section	Angle 80x7 mm
t = 12 mm	Thickness of the gusset plate
R_{wf} = 215 MPa	CO2 semiautomatic welding with a Sv-08G2S wire
N = 700 kN	Longitudinal force
k_{f1} = 6 mm	Weld at free leg
k_{f2} = 6 mm	Weld at connected leg
l_{w1} = 22 cm	weld length at free leg
l_{w2} = 10 cm	weld length at connected leg

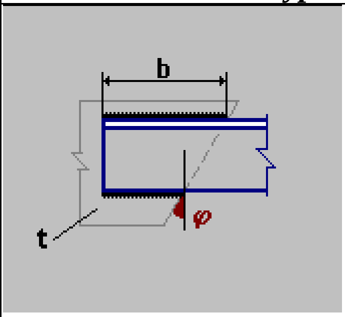
Initial data file:
1.4.sav; report — Kristall1.4.doc

KRISTALL initial data:
Steel: C345 category 1

Importance factor	1
Service factor	1
Group of structures according to the table 50* of SNiP II-23-81*	1

Properties of welding materials:	
Characteristic resistance of the weld metal based on the ultimate strength, R_{wun}	490 N/mm^2
Design resistance of the fillet welds for shear in the weld metal, R_{wf}	215 N/mm^2

Properties of welding materials:	
Type of welding	Automatic and semiautomatic, diameter of the electrode wire not less than 1.4-2.0 mm
Position of weld	Flat
Climatic region	with temperature t > -40°C

Type:	*Parameters:*
 Section - Full assortment of GOST profiles.. Equal angle GOST 8509-93 L80x7	Weld at free leg = 6 mm Weld at connected leg = 6 mm b = 220 mm φ = 56.31 degrees t = 12 mm

Internal forces:

N = 700 kN

Checked according to SNiP	**Check**	**Utilization factor**
Sec.11.2 Formula (120)	of the weld metal	**1.036**
Sec.11.2 Formula (121)	of the metal of the fusion border	0.902

Comparison of solutions

Check	of the weld metal
Source	9,04 cm / 9 cm = 1,0044 21,1 cm / 21 cm = 1,0048
KRISTALL	1,036
Deviation, %	3,01
Refined manual calculation (see comments)	0,72125×9,04 cm / 0,7×9 cm = 1,035 0,72125×21,1 cm / 0,7×21 cm = 1,035
Deviation, %	0,1

Comments:

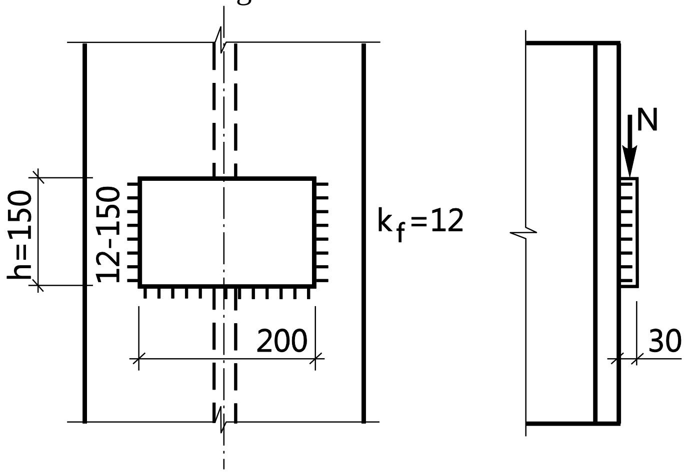
The distribution of the external longitudinal force between the welds along the toe and heel of the angle is not specified precisely in the verification example, i.e. the longitudinal force in the weld along the heel is given as 70% of the external longitudinal force, and that along the toe is given as 30% of the force. The exact value of the longitudinal force acting in the welds along the heel is calculated as:

$$(b_{angle} - y_0)/b_{angle} \times N = 0{,}72125 \times N,$$

where b_{angle} – angle leg width, y_0 – length of a perpendicular dropped from the center of mass of the angle to the respective external leg face.

Analysis of a Welded Connection between an Angle Cleat and a Column Flange

Objective: Check the mode for calculating welded connections.



Task: Check the welded connection with fillet welds between an angle cleat and a column flange.

References: Moskalev N.S., Pronosin J.A. Steel Structures. Handbook / M.: ASV Publishing House, 2010. p. 85.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, SP 16.13330.2017, DBN B.2.6-163:2010, DBN B.2.6-198:2014.

Initial data:

N = 1000 kN	Force
R_{un} = 370 MPa	Steel C245
R_{wf} = 215 MPa	CO2 semiautomatic welding with a Sv-08G2S wire
b = 200 mm	width of the cleat (transverse fillet weld)
k_f = 12 mm	Weld leg
h = 150 mm	height of the cleat (longitudinal fillet welds)

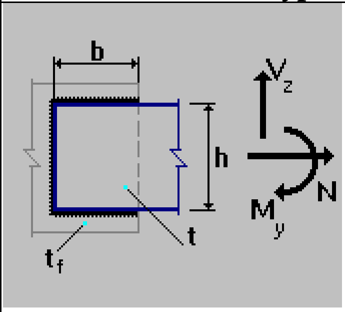
Initial data file:
1.5.sav; report — Kristall1.5.doc

KRISTALL initial data:
Steel: C245

Importance factor	1
Service factor	1
Group of structures according to the table 50* of SNiP II-23-81*	1

Properties of welding materials:	
Characteristic resistance of the weld metal based on the ultimate strength, R_{wun}	490 N/mm^2
Design resistance of the fillet welds for shear in the weld metal, R_{wf}	215 N/mm^2
Type of welding	Automatic and semiautomatic, diameter of the electrode wire not less than 1.4-2.0 mm
Position of weld	Flat

Properties of welding materials:	
Climatic region	with temperature t > -40°C

Type:	***Parameters:***
 Section - Full assortment of GOST profiles. Equal angle GOST 8509-93 L80x7	Weld leg = 12 mm b = 150 mm h = 200 mm t = 30 mm t_f = 30 mm

Internal forces and moments:
N = 1000 kN
M_y = 0 kNm
Q_z = 0 kN

Checked according to SNiP	Check	Utilization factor
Sec.11.2 Formula (120)	of the weld metal	**1.042**
Sec.11.2 Formula (121)	of the metal of the fusion border	1.001

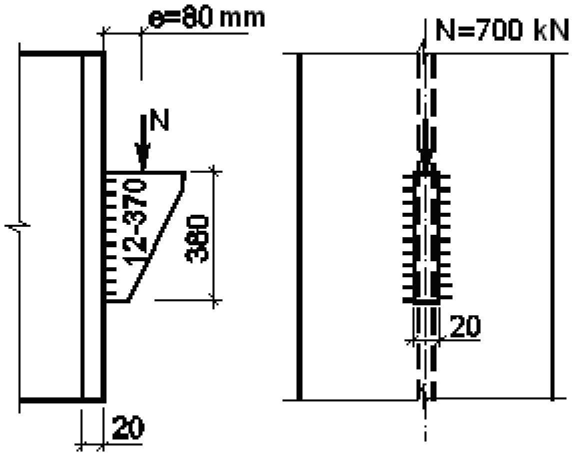
Comparison of solutions:

Check	of the weld metal
Source	48 cm / 47.64 cm = 1,0076
KRISTALL	1,042
Deviation, %	3,3

Comments:
The difference can be explained by the fact that in the source book the value of the design resistance is taken as R_{wf} = 215 MPa, while KRISTALL uses the value of R_{wf} = 200 MPa in full compliance with tables 55* and 56 SNiP II-23-81*.

Analysis of a Connection between an Angle Cleat and a Column Flange for an Eccentrically Applied Force

Objective: Check the mode for calculating welded connections



Task: Check the fillet welded connection between an angle cleat and a column flange for an eccentrically applied force.

References: Moskalev N.S., Pronosin J.A. Steel Structures. Handbook / M.: ASV Publishing House, 2010. p. 86-87.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, SP 16.13330.2017, DBN B.2.6-163:2010, DBN B.2.6-198:2014.

Initial data:

N = 700 kN	Force
e = 80 mm	Eccentricity
R_{un} = 370 MPa	Steel C245
R_{wf} = 185 MPa	Manual welding with E42 electrodes
h = 380 mm	height of the cleat
k_f = 12 mm	Weld leg
t = 20 mm	thickness of elements

Initial data file:

1.6.sav; report — Kristall1.6.doc

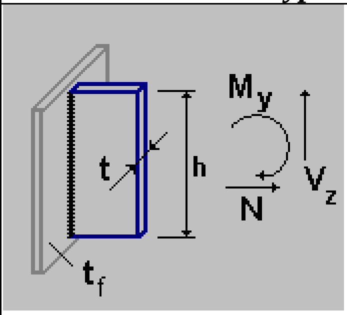
KRISTALL initial data:

Steel: C245

Importance factor	1
Service factor	1
Group of structures according to the table 50* of SNiP II-23-81*	4

Properties of welding materials:	
Characteristic resistance of the weld metal based on the ultimate strength, R_{wun}	410000 kN/m^2
Design resistance of the fillet welds for shear in the	180000 kN/m^2

Properties of welding materials:	
weld metal, R_{wf}	
Type of welding	Manual
Position of weld	Flat
Climatic region	with temperature t > -40°C

Type:	*Parameters:*
	Weld leg = 12 mm h = 380 mm t = 20 mm t_f = 20 mm

Internal forces and moments:
N = 0 N
M_y = 56000 Nm
Q_z = 700000 N

Checked according to SNiP	Check	Utilization factor
Sec.11.2 Formula (120)	of the weld metal	1.025
Sec.11.2 Formula (121)	of the metal of the fusion border	0.775

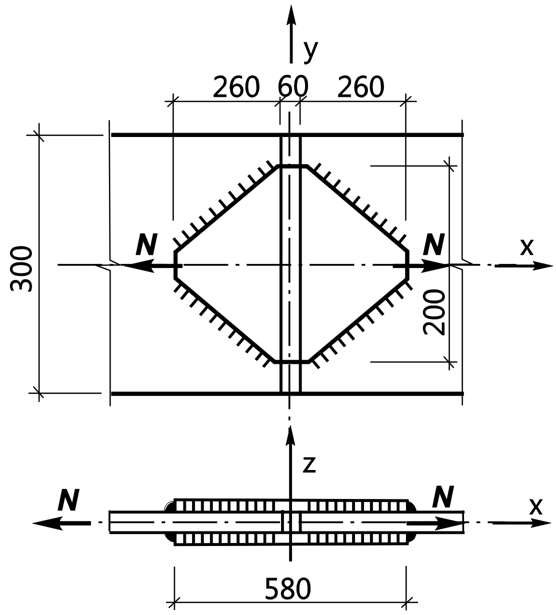
Comparison of solutions:

Check	of the weld metal
Source	18,44 kN/cm^2 / 18,5 kN/cm^2 = 0,997
KRISTALL	1,025
Deviation, %	2,81

Comments:
The difference can be explained by the fact that in the book the value of the design resistance is taken as R_{wf} = 185 MPa, while KRISTALL uses the value of R_{wf} = 180 MPa in full compliance with SP 16.13330.2011 and DBN B.2.6-163:2010.

Analysis of a Welded Connection of Elements with Packings

Objective: Check the mode for calculating welded connections.



Task: Check the fillet welded connection between two strips in tension with packings.

References: Moskalev N.S., Pronosin J.A. Steel Structures. Handbook / M.: ASV Publishing House, 2010. p. 87-88.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, SP 16.13330.2017, DBN B.2.6-163:2010, DBN B.2.6-198:2014.

Initial data:

300x20 mm	Strip section
250x12 mm	Packing section
R_{un} = 360 MPa	Steel C235
N = 1380 kN	Force
R_{wf} = 180 MPa	Manual welding with E42 electrodes
k_f = 10 mm	Weld leg

Initial data file:

1.7.sav; report — Kristall1.7.doc

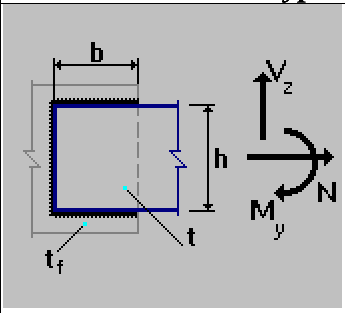
KRISTALL initial data:

Steel: C235

Importance factor	1
Service factor	1
Group of structures according to the table 50* of SNiP II-23-81*	4

Properties of welding materials:	
Characteristic resistance of the weld metal based on the ultimate strength, R_{wun}	410000 kN/m^2
Design resistance of the fillet welds for shear in the	180000 kN/m^2

Properties of welding materials:	
weld metal, R_{wf}	
Type of welding	Manual
Position of weld	Flat
Climatic region	with temperature t > -40°C

Type:	Parameters:
	Weld leg = 10 mm b = 260 mm h = 60 mm t = 12 mm t_f = 20 mm

Internal forces and moments:

N = 1380 kN

M_y = 0 kN

Q_z = 0 kN

Checked according to SNiP	Check	Utilization factor
Sec.11.2 Formula (120)	of the weld metal	0.944
Sec.11.2 Formula (121)	of the metal of the fusion border	0.734

Comparison of solutions:

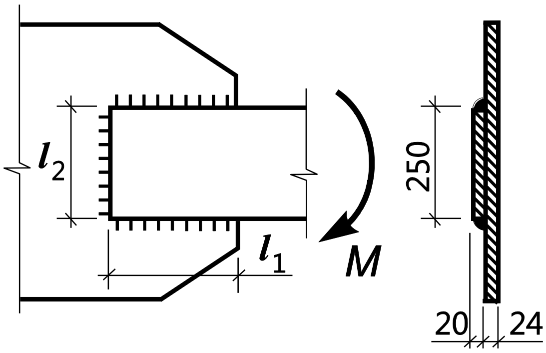
Check	of the weld metal
Source	27,4 cm / 28/cm = 0,979
KRISTALL	0,944
Deviation, %	3,58

Comments:

In the example, the plates are cut in the shape of a fish, the theoretical length of the weld along the diagonal is 290 mm, one of the legs has the size of 260 mm, and the other – 290 – 260 = 30 mm. Therefore, the width of the plate is specified in KRISTALL as 2*30 mm = 60 mm to maintain the same length of the welds.

Analysis of a Welded Connection for a Bending Moment Acting in the Fillet Weld Plane

Objective: Check the mode for calculating welded connections.



Task: Check the welded connection with fillet welds. The connection is loaded with a bending moment acting in the weld plane.

References: Moskalev N.S., Pronosin J.A. Steel Structures. Handbook / M.: ASV Publishing House, 2010. p. 88-89.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, SP 16.13330.2017, DBN B.2.6-163:2010, DBN B.2.6-198:2014.

Initial data:

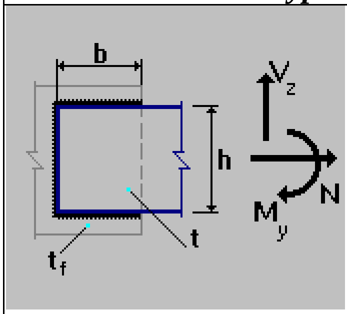
R_{un} = 370 MPa	Steel C245
M = 51 kNm	Force
l1 = 20 cm	Geometric length of longitudinal fillet welds
l2 = 25 cm	Geometric length of the transverse fillet weld
R_{wf} = 185 MPa	Manual welding with E46 electrodes
k_f = 8 mm	Weld leg

Initial data file:
1.8.sav; report — Kristall1.8.doc

KRISTALL initial data:
Steel: C245

Importance factor	1
Service factor	1
Group of structures according to the table 50* of SNiP II-23-81*	4

Properties of welding materials:	
Characteristic resistance of the weld metal based on the ultimate strength, R_{wun}	450000 kN/m^2
Design resistance of the fillet welds for shear in the weld metal, R_{wf}	200000 kN/m^2
Type of welding	Manual
Position of weld	Flat
Climatic region	with temperature t > -40°C

Type:	*Parameters:*
	Weld leg = 8 mm b = 200 mm h = 250 mm t = 20 mm t_f = 24 mm

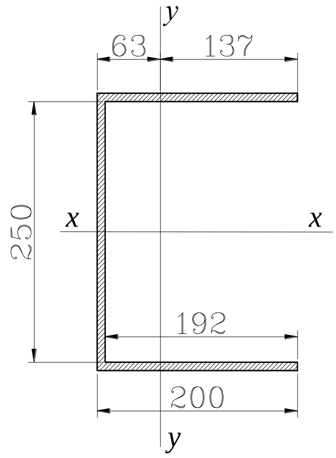
Internal forces and moments:
N = 0 kN
M_y = 51 kNm
Q_z = 0 kN

Checked according to SNiP	**Check**	**Utilization factor**
Sec.11.2 Formula (120)	of the weld metal	0.793
Sec.11.2 Formula (121)	of the metal of the fusion border	0.659

Comparison of solutions

Check	of the weld metal	of the metal of the fusion border
Source	1760 kN/cm^2 / 1850 kN/cm^2 = 0,951	1231,5 kN/cm^2 / 1665 kN/cm^2 = 0,740
KRISTALL	0,793	0,659
Deviation, %	16,6	10,95
Refined manual calculation (see comments)	1636,178 kN/cm^2 / 2000 kN/cm^2 = 0,818	1135,787 kN/cm^2 / 1665 kN/cm^2 = 0,682
Deviation, %	3,06	3,37

Comments:
The difference in the results is due to the inaccuracy made by the authors of the example in the design section of the weld. Moreover, the design resistance of the fillet welds for shear in the weld metal for the E46 electrodes was incorrectly taken in the example as R_{wf} = 185 MPa, while KRISTALL and design codes use the value of R_{wf} = 200 MPa.



Design section of the weld

Let's determine the moments of inertia of the weld with respect to the principal axes of inertia for the correct design section of the weld given in the figure:

$$I_{fx} = \frac{25^3 \cdot 0,7 \cdot 0,8}{12} + \frac{(0,7 \cdot 0,8)^3 \cdot 20}{6} + 2 \cdot 0,7 \cdot 0,8 \cdot 20 \cdot \left(\frac{25}{2} + \frac{0,8 \cdot 0,7}{2} \right.$$

$$= 4388,31\,\text{cm}^4$$

$$I_{fy} = \frac{25 \cdot (0,7 \cdot 0,8)^3}{12} + 25 \cdot 0,7 \cdot 0,8 \cdot \left(6,31 - \frac{0,7 \cdot 0,8}{2} \right)^2 + \frac{0,7 \cdot 0,8 \cdot 20}{6}$$

$$+2 \cdot 0,7 \cdot 0,8 \cdot 20 \cdot \left(\frac{20}{2} - 6,31 \right)^2 = 1561,086 \text{ см}^4$$

$$I_{zx} = \frac{25^3 \cdot 1,0 \cdot 0,8}{12} + \frac{(1,0 \cdot 0,8)^3 \cdot 20}{6} + 2 \cdot 1,0 \cdot 0,8 \cdot 20 \cdot \left(\frac{25}{2} + \frac{0,8 \cdot 1,0}{2} \right)$$

$$= 6368,5\,\text{cm}^4$$

$$I_{zy} = \frac{25 \cdot (1,0 \cdot 0,8)^3}{12} + 25 \cdot 1,0 \cdot 0,8 \cdot \left(6,31 - \frac{1,0 \cdot 0,8}{2} \right)^2 + \frac{1,0 \cdot 0,8 \cdot 20^3}{6}$$

$$+2 \cdot 1,0 \cdot 0,8 \cdot 20 \cdot \left(\frac{20}{2} - 6,31 \right)^2 = 2202,01 \text{ см}^4$$

Then the strength checks of the weld will be as follows:

– of the weld metal:

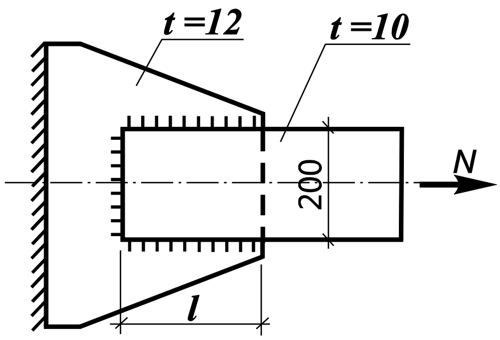
$$\sigma_f = \frac{510000}{4388,31 + 1561,086} \sqrt{13,3^2 + 13,69^2} = 1636,178 \text{ kN/cm}^2 < 2000 \text{ kN/cm}^2$$

– of the metal of the fusion border:

$$\sigma_f = \frac{510000}{6368,5 + 2202,01} \sqrt{13,3^2 + 13,69^2} = 1135,787 \text{ kN/cm}^2 < 1665 \text{ kN/cm}^2.$$

Analysis of an Overlapping Welded Connection of an Element in Tension

Objective: Check the mode for calculating welded connections.



Task: Check the overlapping fillet welded connection of an element in tension.

References: Steel Structures. In 3 v. — V. 1. Elements of Steel Structures/ Gorev V.V., Uvarov B.Yu., Filippov V.V. and others — M.: High school, 2004. p. 158-159.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, SP 16.13330.2017, DBN B.2.6-163:2010, DBN B.2.6-198:2014.

Initial data:

R_{un} = 400 MPa	Steel C285
t = 10 mm	Thickness of the plate
h = 200 mm	Height of the plate
N = 540 kN	Longitudinal force
k_f = 8 mm	Weld leg
R_{wf} = 205 MPa	Manual welding with E46 electrodes
l = 15 cm	Lap length

Initial data file:

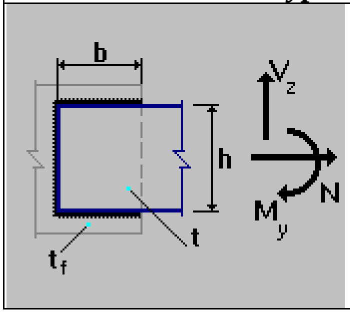
1.9.sav; report — Kristall1.9.doc

KRISTALL initial data:

Steel: C285

Importance factor	1
Service factor	1
Group of structures according to the table 50* of SNiP II-23-81*	1

Properties of welding materials:	
Characteristic resistance of the weld metal based on the ultimate strength, R_{wun}	450 N/mm^2
Design resistance of the fillet welds for shear in the weld metal, R_{wf}	200 N/mm^2
Type of welding	Manual
Position of weld	Flat
Climatic region	with temperature t > -40°C

Type:	*Parameters:*
	Weld leg = 8 mm b = 150 mm h = 200 mm t = 10 mm t_f = 12 mm

Internal forces and moments:

N = 540 kN

M_y = 0 kNm

Q_z = 0 kN

Checked according to SNiP	**Check**	**Utilization factor**
Sec.11.2 Formula (120)	of the weld metal	0.964
Sec.11.2 Formula (121)	of the metal of the fusion border	0.789

Comparison of solutions:

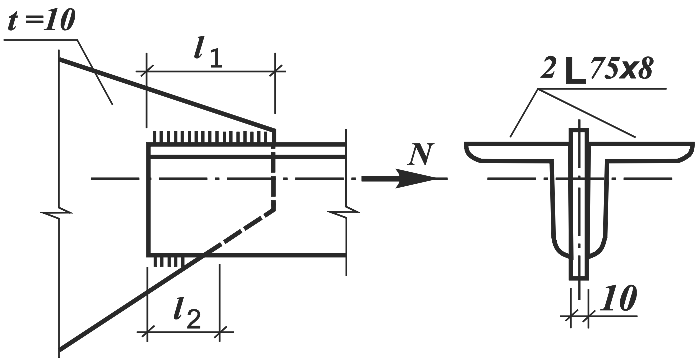
Check	of the weld metal
Source	161 / 11,5 / 14 = 1
KRISTALL	0,964
Deviation, %	3,6

Comments:

The slight difference in the results is explained by the difference in the value of the design resistances R_{wf} = 205 MPa (source) and R_{wf} = 200 MPa (code and KRISTALL).

Analysis of a Welded Connection between a Bar in Tension from Two Angles and a Gusset Plate

Objective: Check the mode for calculating welded connections.



Task: Check the fillet welded connection between a bar in tension from two angles 75x8 and a gusset plate.

References: Steel Structures. In 3 v. — V. 1. Elements of Steel Structures/ Gorev V.V., Uvarov B.Yu., Filippov V.V. and others — M.: High school, 2004. p. 159-160.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, SP 16.13330.2017, DBN B.2.6-163:2010, DBN B.2.6-198:2014.

Initial data:

t = 10 mm	thickness of the gusset plate
N = 425 kN	Longitudinal force
R_{un} = 380 MPa	Steel C245
R_{wf} = 220 MPa	CO2 semiautomatic welding with a Sv-08G2S wire, d = 1,2 mm
k_{f1} = 6 mm	Weld at connected leg
k_{f2} = 6 mm	Weld at free leg
Section	Angle 75x8 mm
l_{w1} = 175 mm	weld length along the free leg
l_{w2} = 80 mm	weld length along the connected leg

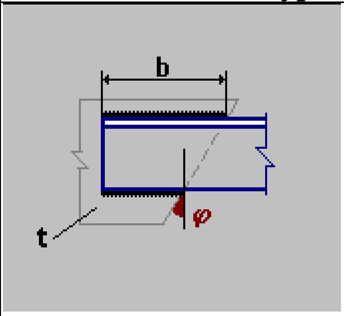
Initial data file:
1.10.sav; report — Kristall1.10.doc

KRISTALL initial data:
Steel: C245

Importance factor	1
Service factor	1
Group of structures according to the table 50* of SNiP II-23-81*	1

Properties of welding materials:	
Characteristic resistance of the weld metal based on the ultimate strength, R_{wun}	490 N/mm^2

Properties of welding materials:	
Design resistance of the fillet welds for shear in the weld metal, R_{wf}	215 N/mm^2
Type of welding	Automatic and semiautomatic, diameter of the electrode wire not less than 1.4-2.0 mm
Position of weld	Flat
Climatic region	with temperature t > -40°C

Type:	***Parameters:***
 Section - Full assortment of GOST profiles.. Equal angle GOST 8509-93 L75x8	Weld at free leg = 6 mm Weld at connected leg = 6 mm b = 175 mm φ = 51.71 degrees t = 10 mm

Internal forces:

N = 425 kN

Checked according to SNiP	**Check**	**Utilization factor**
Sec.11.2 Formula (120)	of the weld metal	**1.018**
Sec.11.2 Formula (121)	of the metal of the fusion border	0.921

Comparison of solutions:

Check	of the weld metal along the free leg
Source	7,9 cm / 8 cm = 0,9875 17,2 cm / 17,5 cm = 0,9829
KRISTALL	1,018
Deviation, %	3,0
Refined manual calculation (see comments)	0,7133×425 / (2×0,7×0,6×16,5cm×21,50)=1,017
Deviation, %	0,1

Comments

The difference in the results is due to the difference in the assumed values of the design resistance R_{wf} = 220 MPa (book) and R_{wf} = 215 MPa (design codes and KRISTALL). Moreover, the distribution of the external longitudinal force between the welds along the toe and heel of the angle is not specified precisely in the verification example, i.e. the longitudinal force in the weld along the heel is given as 70% of the external longitudinal force, and that along the toe is given as 30% of the force. The exact value of the longitudinal force acting in the welds along the heel is calculated as:

$$(b_{angle} - y_0)/b_{angle} \times N = 0{,}7133 \times N,$$

where b_{angle} – angle leg width, y_0 – length of a perpendicular dropped from the center of mass of the angle to the respective external leg face.

BOLTED CONNECTIONS

Analysis of an Overlapping Bolted Connection of Steel Sheets with Ordinary Bolts

Objective: Check the mode for calculating bolted connections

Task: Check an overlapping connection of 500x12 mm sheets with ordinary bolts from steel grade C245 for a shear force.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 165.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

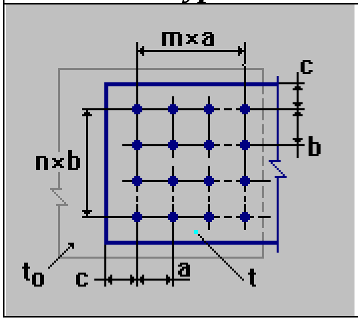
Initial data file:
2.1.sav; report — Kristall2.1.doc

Initial data from the source:

N = 1000 kN	Shear force;
R_{bp} = 450 MPa	Steel grade C245;
	Thickness of plates: two external – 8 mm, internal – 12 mm;
R_{bs} = 200 MPa	Bolts of 5.8 strength class and C accuracy class;
	Diameter of bolts 20 mm, diameter of holes 23 mm;
γ_c = 1	Service factor of the structure;
γ_b = 0,9	Service factor of the bolted connection.

KRISTALL initial data:
Steel: C245
Group of structures according to Annex C of SP 16.13330.2011 2

Importance factor	1
Service factor	1
Service factor of members to be joined	1
Design shear strength of bolts R_{bs}	20897,044 T/m^2
Design bearing strength of bolt elements R_{bp}	49541,284 T/m^2

Type:	*Bolts:*	*Parameters:*
	Diameter of bolts 20 mm Diameter of holes 23 mm Class of bolts 5.8 Accuracy class B or C	m = 5 n = 1 a = 60 mm b = 60 mm c = 50 mm t = 8 mm t_0 = 12 mm

Internal forces and moments:
N = 101,937 T
M_y = 0 T*m

$Q_z = 0$ T

Manual calculation (SP 16.13330.2011):

1. Design shear resistance of the bolts was calculated as follows
$R_{bs} = 0.41 R_{bun} = 0.41 \times 500 = 205$ MPa (see table 5).
2. Design bearing resistance of the bolts was calculated as follows
$R_{bp} = 1.35 R_u = 1.35 \times 360 = 486$ MPa (see table 5).
3. Shear strength of the bolts was calculated according to the following formula:
$$N_{bs} = R_{bs} A_b n_s \gamma_b \gamma_c = 205 \times 10^3 \times 3{,}14 \times 10^{-4} \times 2 \times 1{,}0 \times 0{,}9 = 115{,}866 \text{ kN.}$$
4. Bearing strength of the bolts was calculated according to the following formula:
$$N_{bp} = R_{bp} D \left(\sum_i t_i \right)_{\min} \gamma_b \gamma_c = 486 \times 10^3 \times 20 \times 12 \times 10^{-6} \times 1{,}0 \times 0.9 = 104.976 \text{ kN.}$$

Manual calculation (SNiP II-23-81*):

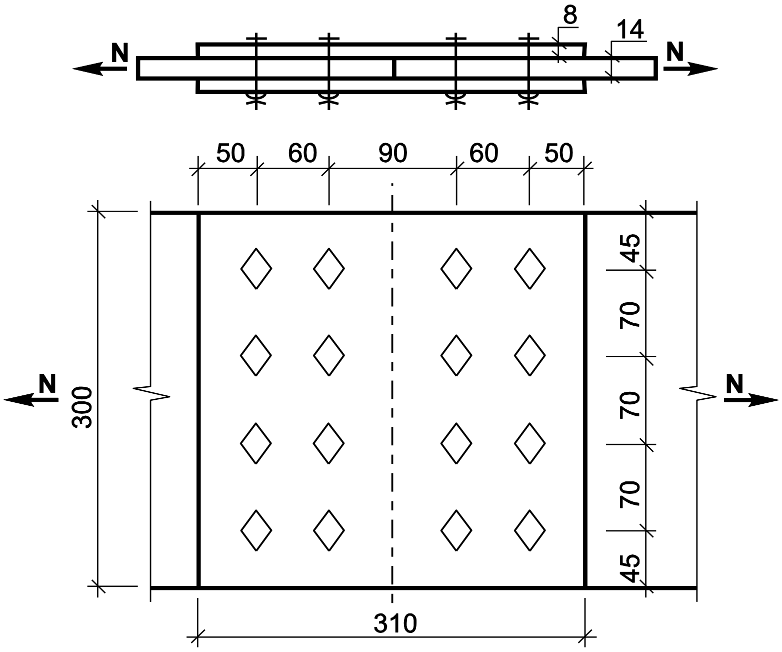
1. Design shear resistance of the bolts was calculated as follows
$R_{bs} = 0{,}4 R_{bun} = 0{,}4 \times 500 = 200$ MPa (see table 5*).
2. Design bearing resistance of the bolts was taken as (see table 5*):
$$R_{bp} = \left(0{,}6 + 340 \frac{R_{un}}{E} \right) R_{un} = \left(0{,}6 + 340 \cdot \frac{370}{2{,}06 \cdot 10^5} \right) \cdot 370 = 447{,}95 \text{ MPa.}$$
3. Shear strength of the bolts was calculated according to the following formula:
$$N_{bs} = R_{bs} A_b n_s \gamma_b \gamma_c = 200 \times 10^3 \times 3{,}14 \times 10^{-4} \times 2 \times 1{,}0 \times 0{,}9 = 113{,}04 \text{ kN.}$$
4. Bearing strength of the bolts was calculated according to the following formula:
$$N_{bp} = R_{bp} D \left(\sum_i t_i \right)_{\min} \gamma_b \gamma_c = 447{,}95 \times 10^3 \times 20 \times 12 \times 10^{-6} \times 1{,}0 \times 0{,}9 = 96{,}7575 \text{ kN.}$$

Comparison of solutions:

Factor	**Codes**	**Shear strength**	**Bearing strength**
Manual calculation	SP 16.13330.2011	1000/(12*115,866) = 0,719	1000/(12*104,976) = 0,794
KRISTALL	SP 16.13330.2011	0,719	0,794
Deviation from the manual calculation, %		0,0	0,0
Manual calculation	SNiP II-23-81*	1000/(12*113,04) = 0,737	1000/(12*96,7575) = 0,861
KRISTALL	SNiP II-23-81*	0,737	0,865
Deviation from the manual calculation, %		0,0	0,5
Source	SNiP II-23-81*	0,737	0,857

Analysis of an Overlapping Bolted Connection of Steel Sheets with Ordinary Bolts

Objective: Check the mode for calculating bolted connections



Task: Check an overlapping connection of 300x14 mm sheets with ordinary bolts from steel grade C275 for a shear force.

Source: Moskalev N.S., Pronosin J.A. Steel Structures. Handbook / M.: ASV Publishing House, 2010. p. 100.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
2.5.sav; report — Kristall2.5.doc

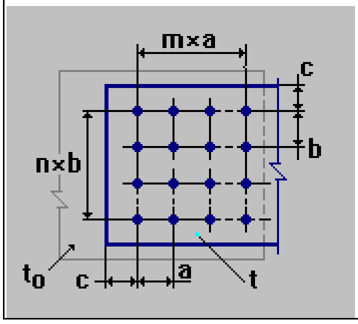
Initial data:

N = 800 kN	Shear force;
R_{bp} = 450 MPa,	Steel grade C275;
	Thickness of the gusset plate 8 mm, middle plate 14 mm;
R_{bs} = 190 MPa,	Bolts of 5.6 strength class and C accuracy class;
	Diameter of bolts 20 mm, diameter of holes 22 mm;
γ_c = 1,1	Service factor;
γ_b = 0,9	Service factor of the bolted connection.

KRISTALL parameters:
Steel: C275
Group of structures according to the table 50* of SNiP II-23-81* 3

Importance factor	1
Service factor	1
Service factor of members to be joined	1
Product of the joint service factor (y_b) and the service factor of members to be joined (y_c)	1
Design shear strength of bolts R_{bs}	190 N/mm^2

Design bearing strength of bolt elements R_{bp}	459.139 N/mm^2

Type:	***Bolts:***	***Parameters:***
	Diameter of bolts 20 mm Clearance 2 mm Diameter of holes 22 mm Class of bolts 5.6 Accuracy class B or C	m = 1 n = 3 a = 60 mm b = 70 mm c = 50 mm t = 8 mm t_0 = 14 mm

Internal forces and moments:
N = 800 kN
M_y = 0 kN*m
Q_z = 0 kN

Manual calculation (SNiP II-23-81*):
1. Design shear resistance of the bolts was calculated as follows
$R_{bs} = 0,38 R_{bun} = 0,38 \times 500 = 190$ MPa (see table 5*).
2. Design bearing resistance of the bolts was calculated as follows (see table 5*):

$$R_{bp} = \left(0,6 + 340\frac{R_{un}}{E}\right)R_{un} = \left(0,6 + 340 \cdot \frac{370}{2,06 \cdot 10^5}\right) \cdot 370 = 447,95 \text{ MPa.}$$

3. Shear strength of the bolts was calculated according to the following formula:

$$N_{bs} = R_{bs} A_b n_s \gamma_b \gamma_c = 190 \times 10^3 \times 3,14 \times 10^{-4} \times 2 \times 0,9 \times 1,1 = 118,127 \text{ kN.}$$

4. Bearing strength of the bolts was calculated according to the following formula:

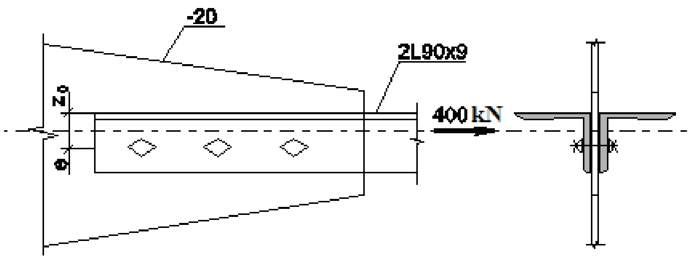
$$N_{bp} = R_{bp} D \left(\sum_i t_i\right)_{\min} \gamma_b \gamma_c = 447,95 \times 10^3 \times 20 \times 14 \times 10^{-6} \times 0,9 \times 1,1 = 124,172 \text{ kN.}$$

Comparison of solutions:

Factor	**Manual calculation**	**KRISTALL**	**Deviation from the manual calculation, %**
shearing of bolts	800/8/118,127 = 0,8465	0,846	0,06
bearing	800/8/124,172 = 0,805	0,809	0,5

Analysis of a Bolted Connection between an Angle and a Gusset Plate with Ordinary Bolts

Objective: Check the mode for calculating bolted connections



Task: Check a bolted connection between two 90x9 mm angles and a 20 mm thick gusset plate with bolts of 8.8 strength class for a shear force of 400 kN.

Source: Moskalev N.S., Pronosin J.A. Steel Structures. Handbook / M.: ASV Publishing House, 2010. p. 102.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

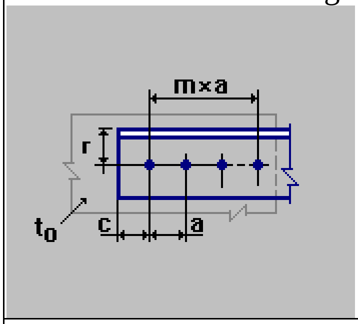
Initial data file:
2.7.sav; report — Kristall2.7.doc

Initial data:

N = 400 kN	Shear force;
a = 100 mm	Distance between bolts;
γ_b = 0,9	Service factor of the bolted connection;
	Diameter of bolts 20 mm, diameter of holes 22 mm.

KRISTALL parameters:
Steel: C255
Group of structures according to the table 50* of SNiP II-23-81* 3

Importance factor	1
Service factor	1
Service factor of members to be joined	1
Product of the joint service factor (y_b) and the service factor of members to be joined (y_c)	1
Design shear strength of bolts R_{bs}	320 N/mm^2
Design bearing strength of bolt elements R_{bp}	440.64 N/mm^2

Type:	*Bolts:*	*Parameters:*
Attachment of double angles  	Diameter of bolts 20 mm Clearance 2 mm Diameter of holes 22 mm Class of bolts 8.8 Accuracy class B or C	m = 2 a = 100 mm c = 50 mm r = 49 mm t_0 = 20 mm
Section - Full assortment of GOST profiles.. Equal angle GOST 8509-93 L90x9		

Internal forces:
N = 400 kN

Manual calculation (SNiP II-23-81*):
1. Design shear resistance of the bolts was calculated as follows
$R_{bs} = 0,40 R_{bun} = 0,40 \times 800 = 320$ MPa (see table 5*).
2. Design bearing resistance of the bolts was taken as (see table 5*):
– when a 20 mm thick gusset plate is in bearing, $R_{un} = 370$ MPa:

$$R_{bp} = \left(0,6 + 340\frac{R_{un}}{E}\right)R_{un} = \left(0,6 + 340 \cdot \frac{370}{2,06 \cdot 10^5}\right) \cdot 370 = 447,95 \text{ MPa;}$$

– when a 9 mm thick angle is in bearing, $R_{un} = 380$ MPa:

$$R_{bp} = \left(0,6 + 340\frac{R_{un}}{E}\right)R_{un} = \left(0,6 + 340 \cdot \frac{380}{2,06 \cdot 10^5}\right) \cdot 380 = 466,33 \text{ MPa.}$$

3. Shear strength of the bolts was calculated according to the following formula:

$$N_{bs} = R_{bs} A_b n_s \gamma_b \gamma_c = 320 \times 10^3 \times 3,14 \times 10^{-4} \times 2 \times 1,0 \times 0,9 = 180,864 \text{ kN.}$$

4. Bearing strength of the bolts was calculated according to the following formula:
– when a 20 mm thick gusset plate is in bearing, $R_{bp} = 447,95$ MPa:

$$N_{bp} = R_{bp} D \left(\sum_i t_i\right)_{\min} \gamma_b \gamma_c = 447,95 \times 10^3 \times 20 \times 20 \times 10^{-6} \times 1,0 \times 0,9 = 161,262 \text{ kN;}$$

– when a 9 mm thick angle is in bearing, $R_{bp} = 466,33$ MPa:

$$N_{bp} = R_{bp} D \left(\sum_i t_i\right)_{\min} \gamma_b \gamma_c = 466,33 \times 10^3 \times 20 \times 18 \times 10^{-6} \times 1,0 \times 0,9 = 151,091 \text{ kN.}$$

5. Design force per one bolt of the connection calculated taking into account the eccentricity $e = 2{,}35$ mm:

$$N_{red} = \sqrt{\left(\frac{N}{3}\right)^2 + \left(\frac{eN}{2a}\right)^2} = \sqrt{\left(\frac{400}{3}\right)^2 + \left(\frac{400 \cdot 2,35}{2 \cdot 100}\right)^2} = 133,416 \text{ kN,}$$

where a – bolt spacing in the connection.
6. Cross-sectional area of one angle weakened by the holes:

$$A_{net} = A - td_0 = 15,6 - 0,9 \cdot 2,2 = 13,62 \text{ cm}^2.$$

Comparison of solutions:

Factor	**Shear strength**	**Bearing strength**	**Strength of the weakened section**
Manual	133,416/180,864 = 0,737	133,416/151,091 = 0,883	400/2/13,62/25 = 0,587

calculation			
KRISTALL	0,737	0,885	0,587
Deviation from the manual calculation, %	0,0	0,2	0,0
Source	0,737	0,857	–

FRICTION CONNECTIONS

Analysis of an Overlapping Bolted Connection of Steel Sheets with High Strength Bolts

Objective: Check the mode for calculating friction connections

Task: Check an overlapping connection of 500x12 mm sheets with high strength bolts from steel grade C245 for a shear force.

References: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 165.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
2.2.sav; report — Kristall2.1.doc

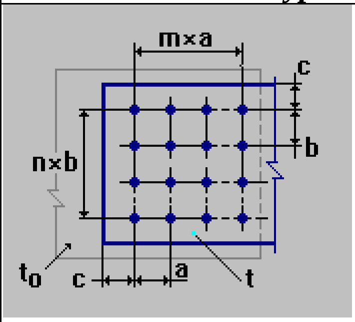
Initial data from the source:

N = 1000 kN	Shear force;
R_y = 240 MPa	Steel grade C245;
	Thickness of plates: two external – 8 mm, internal – 12 mm;
R_{bun} = 110 kN/cm^2	High strength bolts from 40H “select” steel;
	Diameter of bolts 20 mm, diameter of holes 23 mm;
γ_c = 1	Service factor ;
γ_b = 0,9	Service factor of the friction connection;
	Method of cleaning the surfaces – flame treatment, without preservation;
μ = 0,42	Coefficient of friction;
γ_h = 1,12	Tightening control – by the nut rotation angle.

***KRISTALL parameters*:**
Steel: C245

Importance factor	1
Service factor	1
Service factor of members to be joined	1

Diameter of bolts 20 mm
Steel: 40H "select"
Clearance 3 mm
Method of cleaning the surfaces to be joined: Flame treatment of two surfaces, without preservation

Type:	***Parameters:***
	m = 1 n = 3 a = 60 mm b = 60 mm c = 50 mm t = 8 mm t_0 = 12 mm

Internal forces and moments:
N = 1000 kN
M = 0 kNm
Q = 0 kN

Manual calculation:
1. Design tension resistance of high strength bolts was calculated according to the following formula:

$$R_{bh} = 0.7 R_{bun} = 0.7 \times 1100 = 770\,\text{N/mm}^2 = 77,0\,\text{kN/cm}^2.$$

2. Design force which can be resisted by each plane of friction:

$$Q_{bh} = R_{bh} A_{bn} \,{}^{\mu}\!/_{\gamma_h} = 77 \times 2.45 \times {}^{0.42}\!/_{1,02} = 77,68\,\text{kN},$$

where $\gamma_h = 1,02$ for flame treatment without preservation, when the difference between the nominal diameters of the holes and of the bolts is 3 mm, and the bolt tightening is controlled by the nut rotation angle.

3. Required number of bolts:

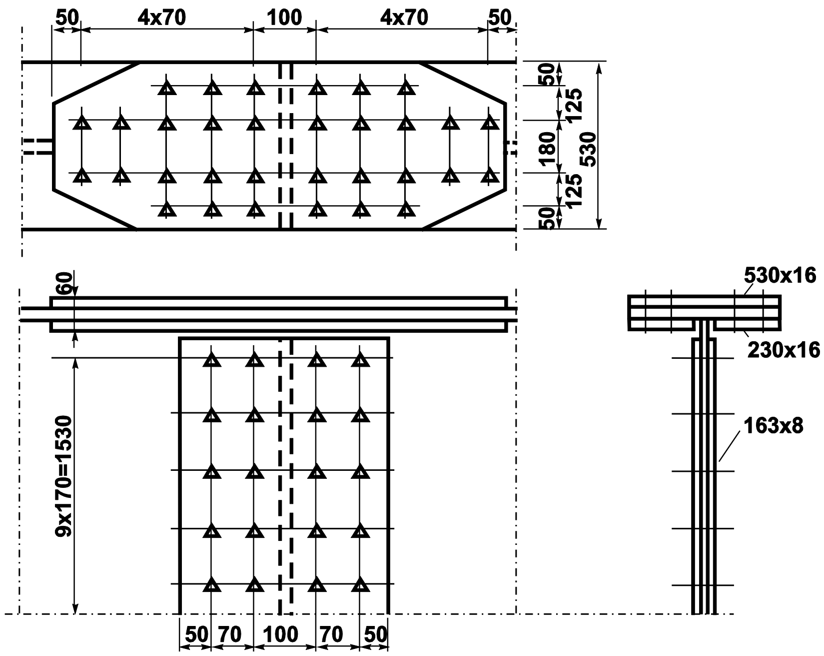
$$n \geq \frac{N}{Q_{bh} \kappa \gamma_b \gamma_c} = \frac{1000}{77.68 \times 2 \times 0.9 \times 1.0} = 7,152.$$

Comparison of solutions:

Factor	**Friction force limit**
Manual calculation	7,152/8 = 0,894
KRISTALL	0,894
Deviation from the manual calculation, %	0,0
Source	0,893

Analysis of an Erection Joint in the Beam Chord with High Strength Bolts

Objective: Check the mode for calculating friction bolted connections



Task: Check the erection joint of the chords of a compound I-beam with high strength bolts.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 216.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
2.3.sav; report — Kristall2.3.doc

Initial data from the source:

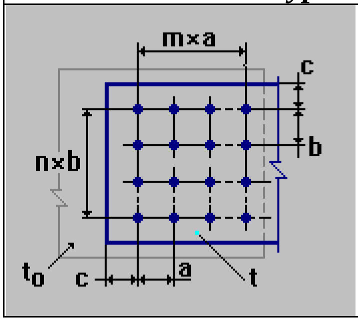
N = 3003 kN	Shear force;
R_y = 240 MPa	Steel grade C245;
	Beam chord section: 530×25 mm;
R_{bun} = 110 kN/cm2	High strength bolts from 40H “select” steel;
	Diameter of bolts 24 mm, diameter of holes 27 mm;
γ_c = 1	Service factor ;
γ_b = 1	Service factor of the friction connection;
	Method of cleaning the surfaces – flame treatment, without preservation;
μ = 0,42	Coefficient of friction;
γ_h = 1,12	Tightening control – by the torque.

KRISTALL parameters:

Steel: C245

Importance factor	1
Service factor	1
Service factor of members to be joined	1

Diameter of bolts 24 mm
Steel: 40H "select"
Clearance 3 mm
Method of cleaning the surfaces to be joined: Flame treatment of two surfaces, without preservation

Type:	***Parameters:***
 	m = 3 n = 3 a = 70 mm b = 125 mm c = 50 mm t = 16 mm t_0 = 24 mm

Internal forces and moments:
N = 3003 kN
M = 0 kNm
Q = 0 kN

Manual calculation:
1. Design tension resistance of high strength bolts was calculated according to the following formula:

$$R_{bh} = 0.7R_{bun} = 0,7 \times 1100 = 770\,\text{N/mm}^2 = 77,0\,\text{kN/cm}^2.$$

2. Design force which can be resisted by each plane of friction:

$$Q_{bh} = R_{bh}A_{bn}\,{}^{\mu}\!/_{\gamma_h} = 77,0 \times 3.53 \times {}^{0.42}\!/_{1.12} = 101,93\,\text{kN}.$$

3. Required number of bolts:

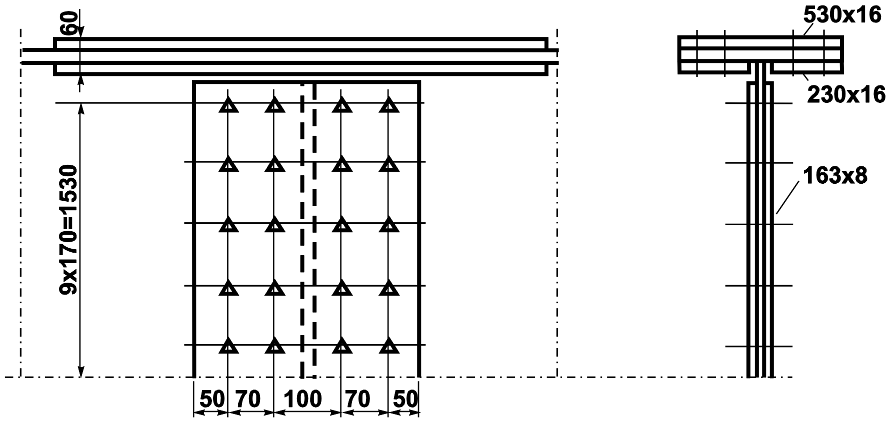
$$n \geq \frac{N}{Q_{bh}\kappa\gamma_b\gamma_c} = \frac{3003}{101,93 \times 2 \times 1,0 \times 1,0} = 14,731.$$

Comparison of solutions:

Factor	**Friction force limit**
Manual calculation	14,731/16 = 0,921
KRISTALL	0,923
Deviation from the manual calculation, %	0,2
Source	0,925

Analysis of an Erection Joint in the Beam Web with High Strength Bolts

Objective: Check the mode for calculating friction bolted connections



Task: Check the erection joint of the compound I-beam web with high strength bolts.

Source: Steel Structures: Student Handbook / [Kudishin U.I., Belenya E.I., Ignatieva V.S and others] - 13-th ed. rev. - M.: Publishing Center "Academy", 2011. p. 216.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
2.4.sav; report — Kristall2.4.doc

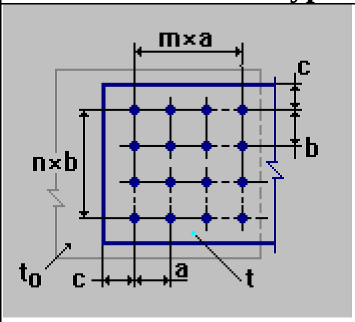
Initial data from the source:

M = 1216 kNm	Bending moment acting in the web plane;
R_y = 240 MPa,	Steel C245;
	Thickness of the beam web 8 mm;
R_{bun} = 110 kN/cm2,	High strength bolts from 40H “select” steel;
	Diameter of bolts 24 mm, diameter of holes 27 mm;
γ_c = 1	Service factor;
γ_b = 1	Service factor of the friction connection;
	Method of cleaning the surfaces – flame treatment, without preservation;
μ = 0,42	Coefficient of friction;
γ_h = 1,12	Tightening control – by the torque.

KRISTALL parameters:
Steel: C245

Importance factor	1
Service factor	1
Service factor of members to be joined	1

Diameter of bolts 24 mm
Steel: 40H "select"
Clearance 3 mm
Method of cleaning the surfaces to be joined: Flame treatment of two surfaces, without preservation

Type	Parameters
	m = 1 n = 9 a = 70 mm b = 170 mm c = 50 mm t = 16 mm t_0 = 16 mm

Internal forces and moments:
N = 0 kN
M = 1216 kNm
Q = 0 kN

Manual calculation:
1. Design tension resistance of high strength bolts was calculated according to the following formula:

$$R_{bh} = 0.7R_{bun} = 0.7 \times 1100 = 770\,\text{N/mm}^2 = 77,0\,\text{kN/cm}^2.$$

2. Design force which can be resisted by each plane of friction:

$$Q_{bh} = \kappa R_{bh} A_{bn} \mu / \gamma_h = 2 \times 77,0 \times 3,53 \times 0.42/1,12 = 203,8575\,\text{kN}.$$

3. Force on the end bolt:

$$N_{\max} = \frac{M \cdot y_{\max}}{2\sum_i y_i^2} = \frac{1216 \cdot 1.53}{2\left((1 \cdot 0.17)^2 + (3 \cdot 0.17)^2 + (5 \cdot 0.17)^2 + (7 \cdot 0.17)^2 + (9 \cdot 0.17)^2\right)} = 195.080\,\text{kN}.$$

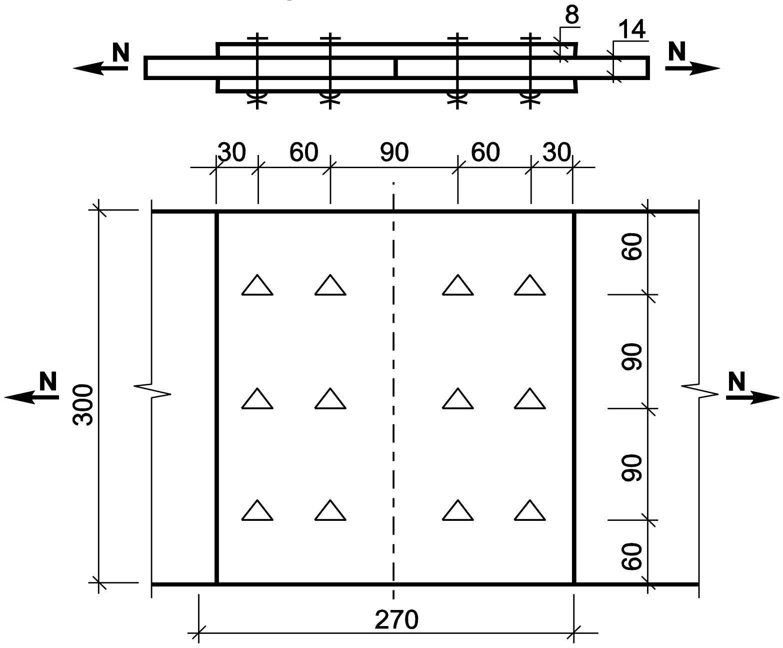
Comparison of solutions:

Factor	**Friction force limit**
Manual calculation	195,080/203,8575 = 0,957
KRISTALL	0,956
Deviation from the manual calculation, %	0,1
Source	0,96

Comments:
In the considered example the vertical distance between bolts is taken as 170 mm, which exceeds the limiting distance 12t = 12*8 mm = 96 mm, calculated according to the codes. To perform the computer-aided calculation in KRISTALL the thickness values of the web and the gusset plates were specified as $t = t_0$ = 16 mm, which does not affect the result of the calculation.

Analysis of an Overlapping Bolted Connection of Steel Sheets with High Strength Bolts

Objective: Check the mode for calculating bolted connections



Task: Check an overlapping connection of 300x14 mm sheets with high strength bolts from steel grade C275 for a shear force.

Source: Moskalev N.S., Pronosin J.A. Steel Structures. Handbook / M.: ASV Publishing House, 2010. p. 101.

Compliance with the codes: SNiP II-23-81*, SP 16.13330.2011, DBN B.2.6-163:2010.

Initial data file:
2.6.sav; report — Kristall2.6.doc

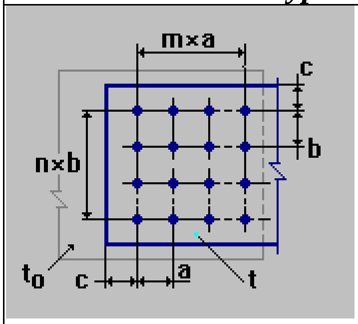
Initial data:

N = 800 kN	Shear force;
R_{bh} = 945 MPa	Bolts from steel grade 30H3MF;
	Diameter of bolts 20 mm, diameter of holes 23 mm;
γ_c = 1; γ_b = 0,9	Service factors;
μ = 0,35	Method of cleaning the surfaces – steel brush;
γ_h = 1,06	Tightening control – by the nut rotation angle.

KRISTALL parameters:
Steel: C275

Importance factor	1
Service factor	1
Service factor of members to be joined	1

Diameter of bolts 20 mm
Steel: 30H3MF
Clearance 3 mm
Method of cleaning the surfaces to be joined: Steel brush, two surfaces without preservation

Type:	Parameters:
	m = 1 n = 2 a = 60 mm b = 90 mm c = 30 mm t = 8 mm t_0 = 14 mm

Internal forces and moments:
N = 800 kN
M = 0 kNm
Q = 0 kN

Manual calculation:
1. Design tension resistance of high strength bolts was calculated according to the following formula:

$$R_{bh} = 0.7R_{bun} = 0,7 \times 1350 = 945\,\text{N/mm}^2 = 94,5\,\text{kN/cm}^2.$$

2. Design force which can be resisted by each plane of friction:

$$Q_{bh} = R_{bh}A_{bn}\,{}^{\mu}\!/_{\gamma_h} = 94,5 \times 2,45 \times {}^{0,35}\!/_{1,06} = 76,447\,\text{kN}.$$

3. Required number of bolts:

$$n \geq \frac{N}{Q_{bh}\kappa\gamma_b\gamma_c} = \frac{800}{76,447 \times 2 \times 0,9 \times 1,0} = 5,814\,.$$

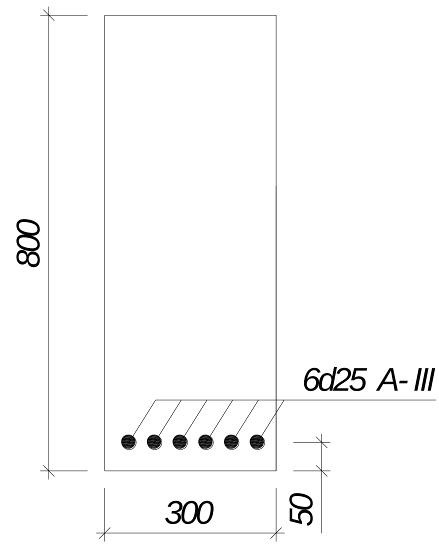
Comparison of solutions:

Factor	Manual calculation	KRISTALL	Deviation from the manual calculation, %
Friction force limit	5,814/6 = 0,969	0,969	0,0

ARBAT

CALCULATIONS ACCORDING TO SNIP 2.03.01-84*

Strength Analysis of a Rectangular Section



Objective: Check of the strength analysis of the section

Task: Verify the correctness of the strength analysis of the section

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight or lightweight concrete (no prestressing) (to SNiP 2.03.01-84), 1989, p. 26.

Initial data file:
Example 3 Guide to SNiP.SAV
report – Arbat 3 SNiP.doc.

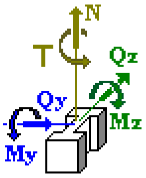
Compliance with the codes: SNiP 2.03.01-84*.

Initial data:

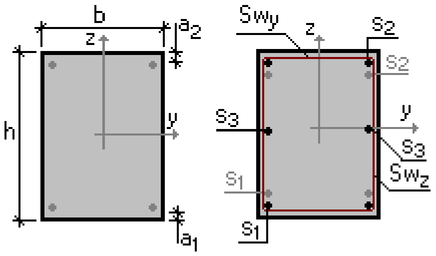
b = 200 mm	Beam section sizes
h = 800 mm	
a = 50 mm	Distance from the center of gravity of the reinforcement to the compressed edge of the section
A_s = 2945 mm^2 (6Ø25)	Cross-sectional area of reinforcement
Concrete class	B25
Class of reinforcement	A-III
M = 550 kNm	Bending moment in the section

ARBAT initial data:

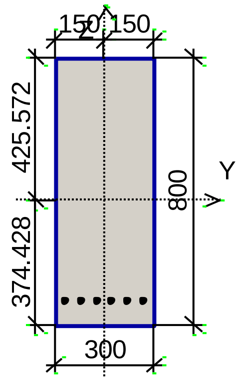
Importance factor $\gamma_n = 1$
Member length 1 m
Effective length factor in the XoY plane 1
Effective length factor in the XoZ plane 1
Random eccentricity along Z according to SNiP 2.03.01-84* (Russia and other CIS countries)
Random eccentricity along Y according to SNiP 2.03.01-84* (Russia and other CIS countries)
Structure is statically determinate
Limit slenderness - 200



Section:



b = 300 mm
h = 800 mm
a_1 = 58.5 mm
a_2 = 20 mm



S_1 - 6∅25

Reinforcement	Class	Service factor
Longitudinal	A-III	1
Transverse	A-I	1

Concrete:

Concrete type: Heavy-weight
Concrete class: B25
Hardening conditions: Natural
Hardening factor 1

Service factor for concrete		
γ_{b2}	allowance for the sustained loads	0.9
	resulting factor without γ_{b2}	1

Results for combinations of loadings

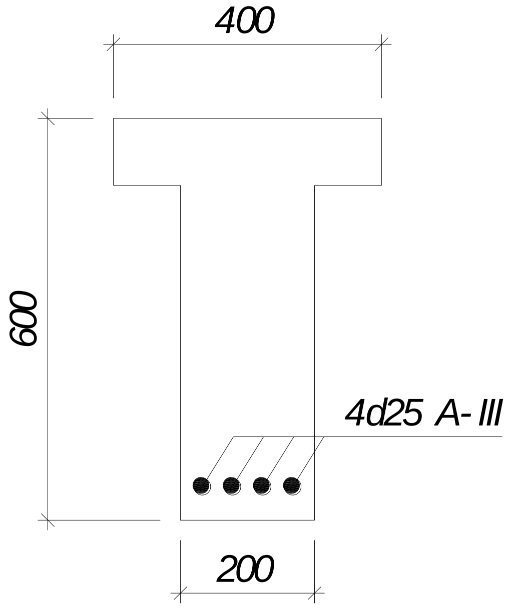
	N	M_y	Q_z	M_z	Q_y	T	Safety factor	Factor for	Short-term	Seismic
	kN	kN*m	kN	kN*m	kN	kN*m	for load	sustained load		
1	0	550	0	0	0	0	1	1		

Comparison of solutions:

Check	strength of the section
Guide	550/636,4 = 0,864
ARBAT	0,859

Deviation, %	0,6 %

Strength Analysis of a T-section



Objective: Check of the strength analysis of the section

Task: Verify the correctness of the strength analysis of the section

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight or lightweight concrete (no prestressing) (to SNiP 2.03.01-84), 1989, p. 27-28.

Initial data file:
Example 7.SAV
report – Arbat 7 SNiP.doc.

Compliance with the codes: SNiP 2.03.01-84*.

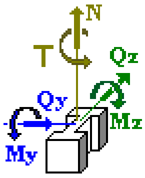
Initial data:

b = 200 mm	Beam section sizes
h = 600 mm	
b'_f = 400 mm	
h'_f = 100 mm	
a = 50 mm	Distance from the center of gravity of the reinforcement to the compressed edge of the section
A_s = 1964 mm^2 (4Ø25)	Cross-sectional area of reinforcement
Concrete class	B25
Class of reinforcement	A-III
M=300 kN*m	Bending moment in the section

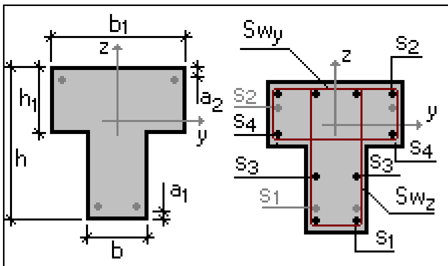
ARBAT initial data:

Importance factor $\gamma_n = 1$

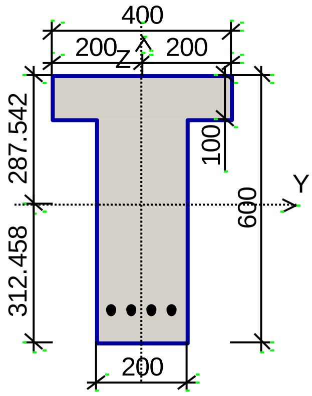
Member length 1 m
Effective length factor in the XoY plane 1
Effective length factor in the XoZ plane 1
Random eccentricity along Z according to SNiP 2.03.01-84* (Russia and other CIS countries)
Random eccentricity along Y according to SNiP 2.03.01-84* (Russia and other CIS countries)
Structure is statically determinate
Limit slenderness - 200



Section:



b = 200 mm
h = 600 mm
b_1 = 400 mm
h_1 = 100 mm
a_1 = 58.5 mm
a_2 = 20 mm



S_1 - 4∅25

Reinforcement	Class	Service factor
Longitudinal	A-III	1
Transverse	A-I	1

Concrete:

Concrete type: Heavy-weight
Concrete class: B25
Hardening conditions: Natural
Hardening factor 1

Service factor for concrete		
γ_{b2}	allowance for the sustained loads	0.9
	resulting factor without γ_{b2}	1

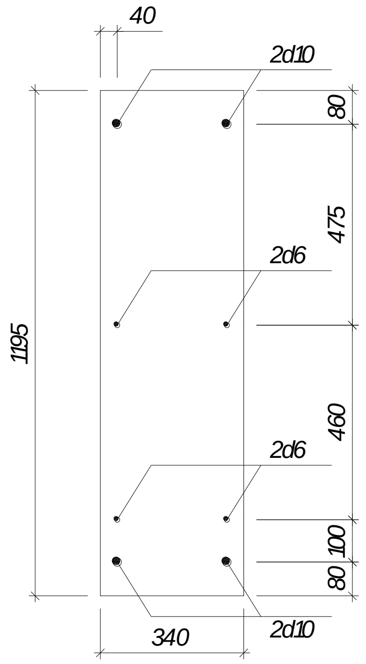
Results for combinations of loadings

	N kN	M_y kN*m	Q_z kN	M_z kN*m	Q_y kN	T kN*m	Safety factor for load	Factor for sustained load	Short-term	Seismic
1	0	300	0	0	0	0	1	1		

Comparison of solutions:

Check	strength of the section
Guide	300/327,1 = 0,917
ARBAT	0,914
Deviation, %	0,3 %

Strength Analysis of a Wall Panel



Objective: Check of the strength of the wall panel

Task: Check the strength of the section

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight or lightweight concrete (no prestressing) (to SNiP 2.03.01-84), 1989, p. 32-34.

Initial data file:
Example 12 SNiP.SAV
report – Arbat 12.doc.

Compliance with the codes: SNiP 2.03.01-84*.

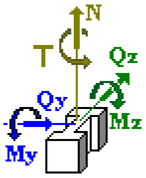
Initial data:

l = 5,8 m	Wall panel span
$b \times h$ = 340×1195 mm	Wall panel section sizes
q_{tot} = 3,93 kN/m^2	Total vertical uniformly distributed load
q_w = 0,912 kN/m^2	Wind load
Concrete class	B3,5; D1100
Class of reinforcement	A-III

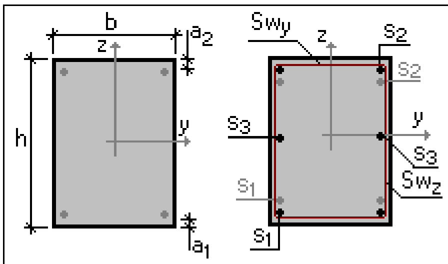
ARBAT initial data:

Importance factor $\gamma_n = 1$

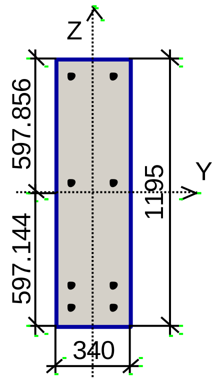
Member length 5.8 m
Effective length factor in the XoY plane 1
Effective length factor in the XoZ plane 1
Random eccentricity along Z according to SNiP 2.03.01-84* (Russia and other CIS countries)
Random eccentricity along Y according to SNiP 2.03.01-84* (Russia and other CIS countries)
Structure is statically determinate
Limit slenderness - 200



Section:



b = 340 mm
h = 1195 mm
a_1 = 75 mm
a_2 = 75 mm



S_1 - 2∅10, second row 2∅6
Clear distance between rows 92 mm)
S_2 - 2∅10, second row 2∅6
Clear distance between rows 467 mm)

Reinforcement	Class	Service factor
Longitudinal	A-III	1
Transverse	A-I	1

Concrete:

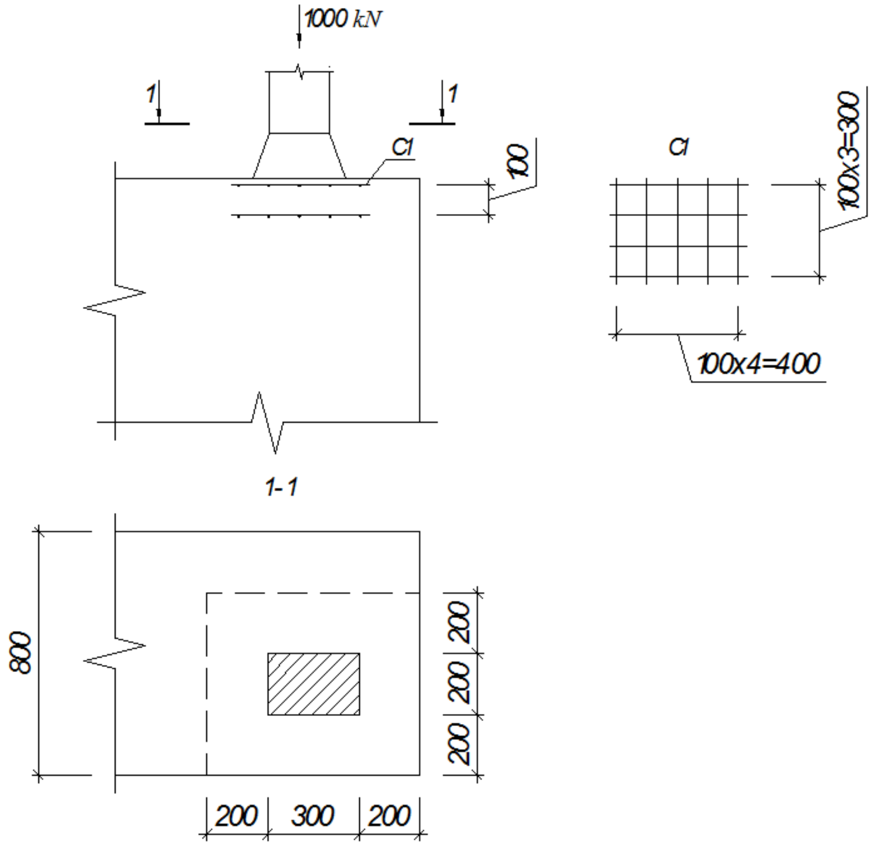
Concrete type: Lightweight
Concrete class: B3,5
Grade by average density: D1100
Aggregate: Artificial dense
Hardening conditions: Natural
Hardening factor 1

Service factor for concrete		
γ_{b2}	allowance for the sustained loads	1.1
	resulting factor without γ_{b2}	1

Comparison of solutions:

Check	strength of the section
Guide	74,5/78,4= 0,950
ARBAT	0,953
Deviation, %	0,3 %

Local Compression Analysis



Objective: Check the local compression analysis of the foundation

Task: Verify the correctness of the local compression analysis

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight or lightweight concrete (no prestressing) (to SNiP 2.03.01-84), 1989, p. 101-102.

Initial data file:
Example 48.SAV
report – Arbat 48.doc.

Compliance with the codes: SNiP 2.03.01-84*.

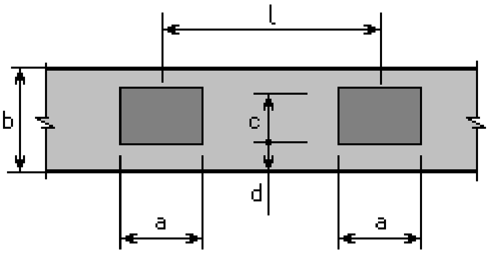
***Initial data*:**

b = 0,8 m	Width of the foundation
$a_1 \times a_2$ = 300×200 mm	Sizes of the load application area
$A_{sx} = A_{sy}$ = 7,1 mm^2 (1Ø3)	Cross-sectional area of reinforcement
N = 1000 kN	Vertical load
Concrete class	B12,5
Class of reinforcement	Bp-I

ARBAT initial data:

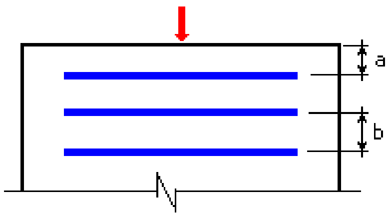
Importance factor $\gamma_n = 1$

Load arrangement:

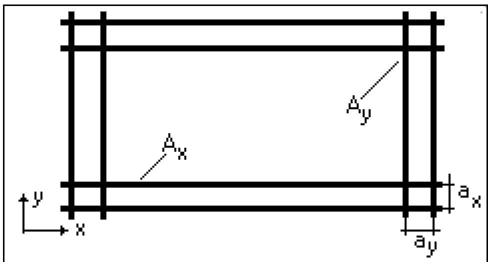
Local load applied to a part of length and width of an element Local edge load within a protruding part of a wall or a pier	a = 300 mm b = 800 mm c = 200 mm d = 200 mm Number of load application areas - one
	

Lateral reinforcement by flat meshes:

Class of reinforcement: B500
Arrangement of meshes

	Concrete cover a = 20 mm Spacing of meshes b = 100 mm Number of meshes - 2

Meshes:

	Meshes Rebars along X Diameter 3 mm Spacing a_x = 100 mm Number of rebars - 4 Rebars along Y Diameter 3 mm Spacing a_y = 100 mm Number of rebars - 5

Concrete:

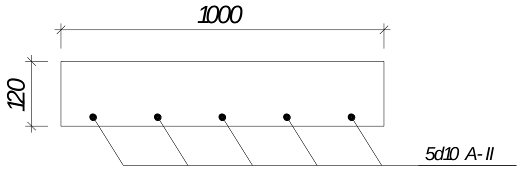
Concrete type: Heavy-weight
Concrete class: B12,5

Service factor for concrete		
γ_{b2}	allowance for the sustained loads	0.9
	resulting factor without γ_{b2}	1

Comparison of solutions:

Check	strength condition of local compression
Guide	1000/1182 = 0,846
ARBAT	0,84
Deviation, %	0,7 %

Slab Deflection Analysis



Objective: Check of the slab deflection analysis

Task: Verify the correctness of the deflection calculation

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight or lightweight concrete (no prestressing) (to SNiP 2.03.01-84), 1989, p. 139-140.

Initial data file:
Example 57.SAV
report – Arbat 57.doc.

Compliance with the codes: SNiP 2.03.01-84*.

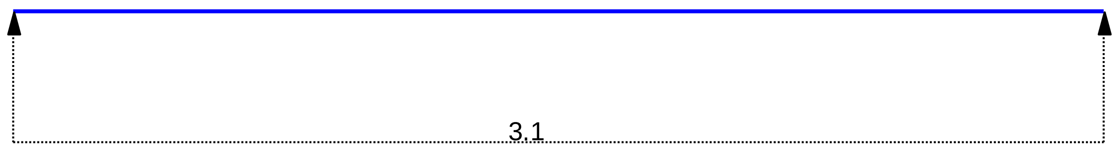
Initial data:

l = 3,1 m	Slab span
$b \times h$ = 1000×150 mm	Slab section sizes
a = 15 mm	Distance from the center of gravity of the reinforcement to the compressed edge of the section
A_s = 393 mm^2 (5Ø10)	Cross-sectional area of reinforcement
q_{tot} = 7 kN/m	Total vertical uniformly distributed load
q_l = 6 kN/m	Part of the total uniformly distributed load from permanent and long-term loads
Concrete class	B25
Class of reinforcement	A-II

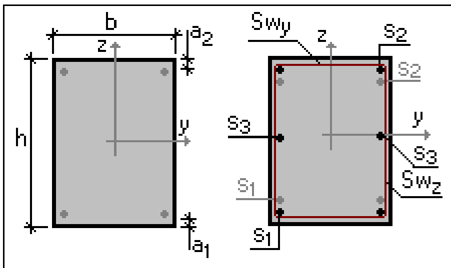
ARBAT initial data:
Importance factor γ_n = 1

Importance factor (serviceability limit state) = 1

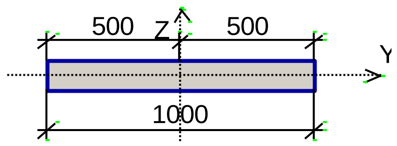
Structure:



Section:

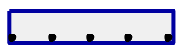


b = 1000 mm
h = 120 mm
a_1 = 10 mm
a_2 = 10 mm



Reinforcement	Class	Service factor
Longitudinal	A-II	1
Transverse	A-I	1

Specified reinforcement:

Span	Segment	Length (m)	Reinforcement	Section
span 1	1	3.1	S_1 - 5□10	

Concrete:
Concrete type: Heavy-weight
Concrete class: B25
Density of concrete 2,5 T/m^3

Service factor for concrete:		
γ_{b2}	allowance for the sustained loads	1
	resulting factor without γ_{b2}	1

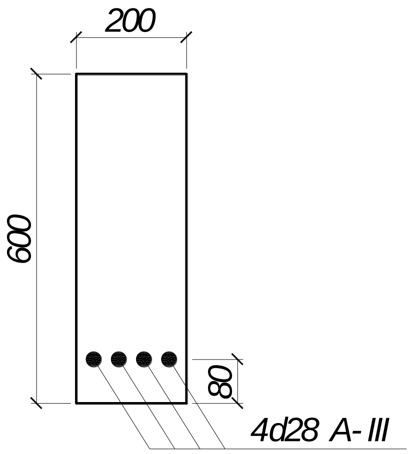
Conditions of operation:
Mode of concrete humidity - Natural humidity
Humidity of environmental air - 40-75%

Comparison of solutions:

Check	maximum deflection
Guide	12,1 mm
ARBAT	13,098 mm
Deviation, %	8,2 %

Comment: The difference in the results is due to the fact that approximate empirical formulas are used in the Guide.

Girder Deflection Analysis



Objective: Check of the girder deflection analysis

Task: Verify the correctness of the deflection calculation

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight or lightweight concrete (no prestressing) (to SNiP 2.03.01-84), 1989, p. 140.

Initial data file:
Example 58.SAV
report – Arbat 58.doc.

Compliance with the codes: SNiP 2.03.01-84*.

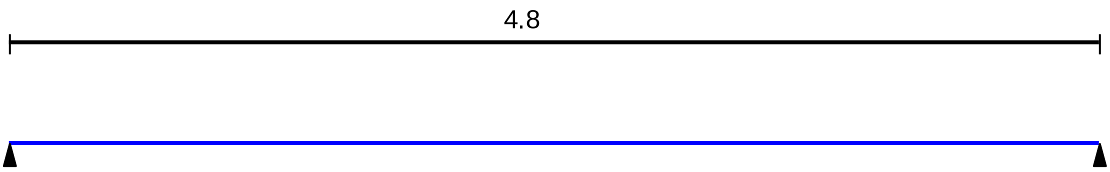
Initial data:

l = 4,8 m	Girder span
$b \times h$ = 200×600 mm	Girder section sizes
a = 80 mm	Distance from the center of gravity of the reinforcement to the compressed edge of the section
A_s = 2463 mm^2 (4Ø28)	Cross-sectional area of reinforcement
q_{tot} = 85,5 kN/m	Total uniformly distributed load
q_l = 64 kN/m	Part of the total uniformly distributed load from permanent and long-term loads
Concrete class	B25
Class of reinforcement	A-III

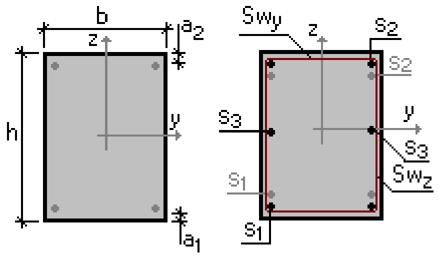
Deflection is limited by aesthetic requirements.

ARBAT initial data:
Importance factor $\gamma_n = 1$

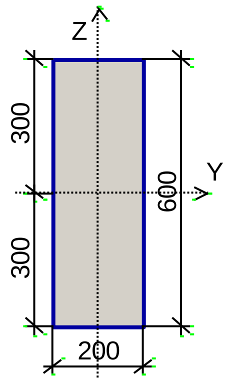
Structure:



Section:

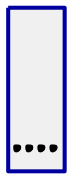


b = 200 mm
h = 600 mm
a_1 = 66 mm
a_2 = 20 mm



Reinforcement	Class	Service factor
Longitudinal	A-III	1
Transverse	A-I	1

Specified reinforcement:

Span	Segment	Length (m)	Reinforcement	Section
span 1	1	4.8	S_1 - 4∅28	

Concrete:

Concrete type: Heavy-weight
Concrete class: B25
Density of concrete 2,5 T/m^3

Hardening conditions: In steam-curing chambers
Hardening factor 1

Service factor for concrete		
γ_{b2}	allowance for the sustained loads	1
	resulting factor without γ_{b2}	1

Conditions of operation:

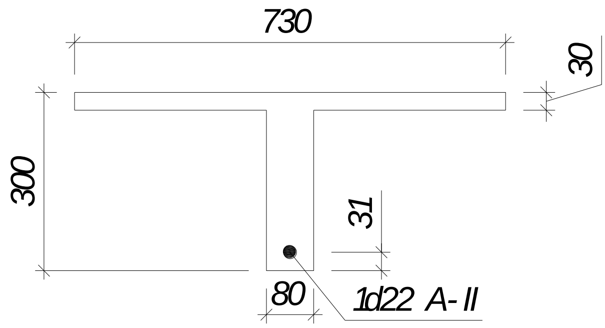
Mode of concrete humidity - Natural humidity
Humidity of environmental air - 40-75%

Comparison of solutions:

Check	maximum deflection
Guide	19,7 mm
ARBAT	20,298 mm
Deviation, %	3,4 %

Comment: Since the deflection is limited by aesthetic requirements, the load was taken as q_l=64 kN/m (see Sec. 1.17 of the Guide).

Tee Slab Deflection Analysis



Objective: Check of the slab deflection analysis

Task: Verify the correctness of the deflection calculation

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight or lightweight concrete (no prestressing) (to SNiP 2.03.01-84), 1989, p. 140-141.

Initial data file:
Example 59.SAV
report – Arbat 59.doc.

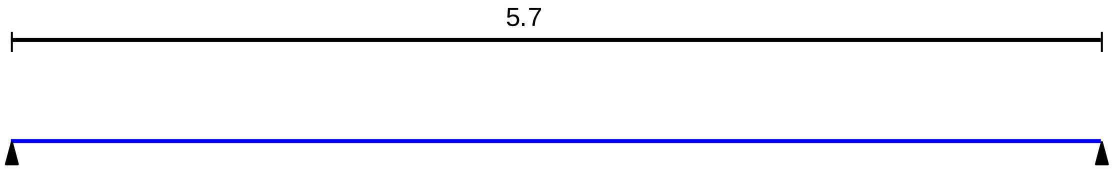
Compliance with the codes: SNiP 2.03.01-84*.

Initial data:

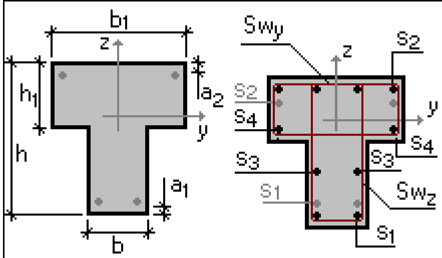
l = 5,7 m	Slab span
$b \times h$ = 80×300 mm	Slab section sizes
a = 31 mm	Distance from the center of gravity of the reinforcement to the compressed edge of the section
A_s = 380 mm^2 (1Ø22)	Cross-sectional area of reinforcement
q_l = 8,75 kN/m	Permanent and long-term distributed load
Concrete class	B25, D1600
Class of reinforcement	A-II

ARBAT initial data:
Importance factor γ_n = 1

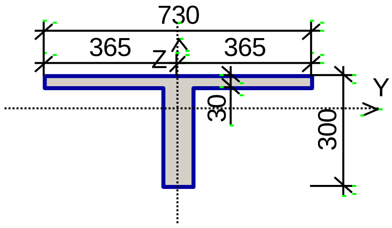
Structure:



Section:

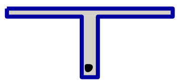


b = 80 mm
h = 300 mm
b_1 = 730 mm
h_1 = 30 mm
a_1 = 20 mm
a_2 = 20 mm



Reinforcement	Class	Service factor
Longitudinal	A-II	1
Transverse	A-I	1

Specified reinforcement:

Span	Segment	Length (m)	Reinforcement	Section
span 1	1	5.7	S_1 - 1∅22	

Concrete:
Concrete type: Lightweight
Concrete class: B25
Grade by average density: D1600
Aggregate: Artificial dense
Density of concrete 1.6 T/m^3

Hardening conditions: In steam-curing chambers
Hardening factor 1

Service factor for concrete		
γ_{b2}	allowance for the sustained loads	1
	resulting factor without γ_{b2}	1

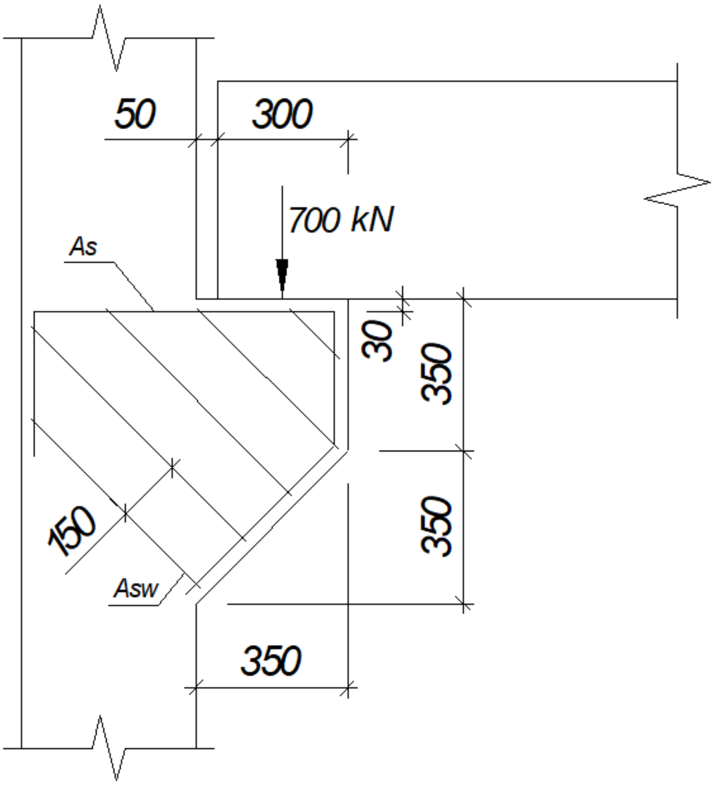
Conditions of operation:
Mode of concrete humidity - Natural humidity
Humidity of environmental air - 40-75%

Comparison of solutions:

Check	maximum deflection
Guide	23,2 mm
ARBAT	22,905 mm
Deviation, %	1,3 %

Comment. The difference in the results is due to the fact that approximate empirical formulas are used in the Guide.

Analysis of Short Cantilevers



Objective: Check of the analysis of short cantilevers

Task: Verify the correctness of the analysis of short cantilevers

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight or lightweight concrete (no prestressing) (to SNiP 2.03.01-84), 1989, p. 105-106.

Initial data file:
Example 49.SAV
report – Arbat 49.doc.

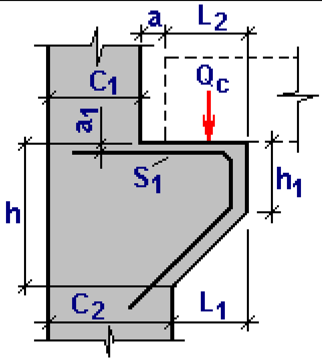
Compliance with the codes: SNiP 2.03.01-84*.

Initial data:

b = 400 mm	Width of the cantilever
h = 700 mm	Height of the cantilever
l_1 = 350 mm	Cantilever overhang length
$l_{sup,f}$ = 300 mm	Length of the bearing area along the cantilever
A_s = 1140 mm^2 (3Ø22)	Cross-sectional area of longitudinal reinforcement
A_{sw} = 157 mm^2 (1Ø10)	Cross-sectional area of transverse reinforcement
N = 700 kN	Vertical load on the cantilever
Concrete class	B25
Class of reinforcement	AIII

ARBAT initial data:

Importance factor $\gamma_n = 1$

 Hinge bearing of the girder on the column cantilever	h = 700 mm h_1 = 350 mm a = 50 mm L_1 = 350 mm C_1 = 400 mm C_2 = 400 mm

Width of the column (cantilever) b = 400 mm
Length of the girder bearing area L_2 = 300 mm
Concrete cover a_1 = 19 mm
Width of the girder b_1 = 400 mm
Load on the column cantilever Q_c = 700 kN
Longitudinal reinforcement of the cantilever A-III 3∅22
Transverse reinforcement of the cantilever A-III ∅10, spacing of stirrups 150 mm

Concrete:
Concrete type: Heavy-weight
Concrete class: B25

Service factor for concrete		
γ_{b2}	allowance for the sustained loads	0.9

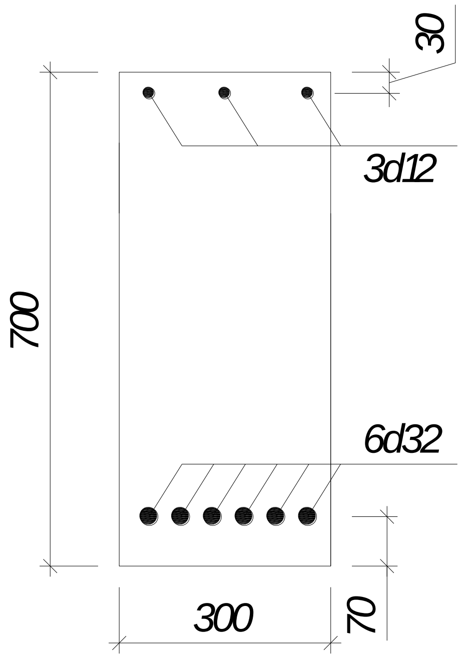
Comparison of solutions:

Check	Providing strength in an oblique compressed strip between a load and a support
Guide	700/717 = 0,976
ARBAT	0,973
Deviation, %	0,3 %

Check	Load-bearing capacity of the longitudinal reinforcement
Guide	1002/1140 = 0,879
ARBAT	0,879
Deviation, %	0 %

CALCULATIONS ACCORDING TO SNIP 52-01-2003

Strength Analysis of a Section



Objective: Check of the strength analysis of the section

Task: Verify the correctness of the strength analysis of the section

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight concrete (no prestressing) (to SP 52-101-2003), 2005, p. 28.

Initial data file:
Example 6 Guide to SP.SAV
report – Arbat 6 SP.doc.

Compliance with the codes: SP 52-101-2003.

Initial data:

$b \times h$ = 300×700 mm	Section sizes
a = 70 mm	Distance to the c.o.g. of tensile reinforcement
a'= 30 mm	Distance to the c.o.g. of compressed reinforcement
A_s = 4826 mm^2 (6∅32)	Cross-sectional area of tensile reinforcement
A'_s = 339 mm^2 (3∅12)	Cross-sectional area of compressed reinforcement
M = 630 kNm	Bending moment
Concrete class	B20
Class of reinforcement	A400

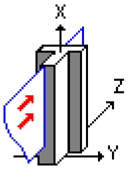
ARBAT initial data:

Importance factor γ_n = 1

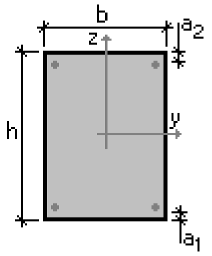
Importance factor (serviceability limit state) = 1

Member length 3 m
Effective length factor in the XoY plane 1
Effective length factor in the XoZ plane 1
Random eccentricity along Z according to SNiP 52-01-2003 (Russia)
Random eccentricity along Y according to SNiP 52-01-2003 (Russia)
Structure is statically determinate
Limit slenderness - 200

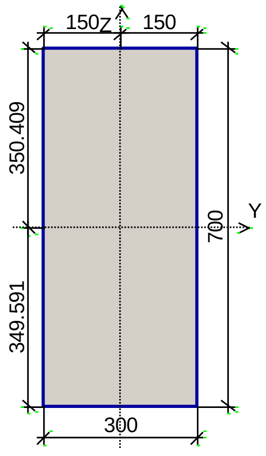
Plane of loading



Section:



b = 300 mm
h = 700 mm
a_1 = 70 mm
a_2 = 30 mm



Areas of reinforcement
$AS_1 = 48.258\ cm^2$
$AS_2 = 3.393\ cm^2$

Reinforcement	Class	Service factor
Longitudinal	A400	1
Transverse	A240	1

Concrete:

Concrete type: Heavy-weight
Concrete class: B20

Service factor for concrete		
γ_{b1}	allowance for the sustained loads	1
γ_{b2}	allowance for the failure behavior	1
γ_{b3}	allowance for the vertical position during concreting	1
γ_{b4}	allowance for the freezing/thawing and negative temperatures	1

Humidity of environmental air - 40-75%

Comparison of solutions:

Check	Strength of the section
Guide	630/606,2 = 1,039
ARBAT	1,039
Deviation, %	0 %

Calculation of a Rib of a TT-shaped Floor Slab for Load-bearing Capacity under Lateral Forces

Objective: Check of the calculation of the resistance of reinforced concrete sections.

Task: Verify the correctness of the strength analysis of oblique sections and a concrete strip between the oblique sections.

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight concrete (no prestressing) (to SP 52-101-2003), 2005, p. 56-57.

Initial data file:

when the lateral force is Q = 62 kN — Example 12.1.SAV
report – Arbat 12.1.doc.
when the lateral force is Q = 58,4 kN — Example 12.2.SAV
report – Arbat 12.2.doc.

Compliance with the codes: SP 52-101-2003, SP 63.13330.2012.

Initial data:

$b \times h$ = 85×350 mm	Slab rib section sizes
a = 35 mm	Distance from the center of gravity of the longitudinal reinforcement to the fiber of the section under the greatest tension
d = 8 mm	Diameter of transverse reinforcement
s_w = 100 mm	Spacing of transverse reinforcement
q = 21,9 kN/m	Load on the rib
q_v = 18 kN/m	Temporary equivalent load
Q_{max} = 62 kN	Lateral force on the support

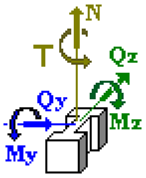
Concrete class B15
Class of reinforcement A400

ARBAT initial data when the lateral force is Q = 62 kN:

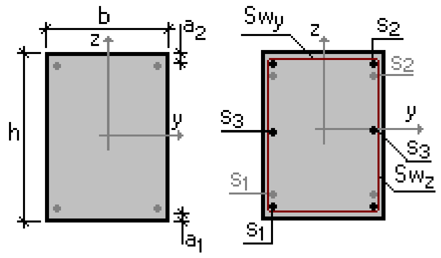
Importance factor γ_n = 1

Importance factor (serviceability limit state) = 1

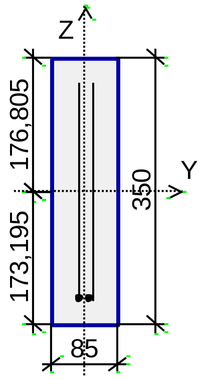
Member length 1 m
Effective length factor in the XoY plane 1
Effective length factor in the XoZ plane 1
Random eccentricity along Z according to SNiP 52-01-2003 (Russia)
Random eccentricity along Y according to SNiP 52-01-2003 (Russia)
Structure is statically indeterminate
Limit slenderness - 200



Section:



b = 85 mm
h = 350 mm
a_1 = 32 mm
a_2 = 32 mm



S_1 - 2∅6
Transverse reinforcement along the Z axis 1∅8, spacing of transverse reinforcement 100 mm

Reinforcement	Class	Service factor
Longitudinal	A400	1
Transverse	A400	1

Concrete:
Concrete type: Heavy-weight
Concrete class: B15

Service factor for concrete		
γ_{b1}	allowance for the sustained loads	1
γ_{b2}	allowance for the failure behavior	1
γ_{b3}	allowance for the vertical position during concreting	1
γ_{b4}	allowance for the freezing/thawing and negative temperatures	1

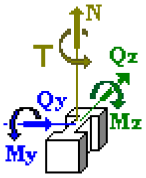
Humidity of environmental air - 40-75%

Crack resistance:
No cracks

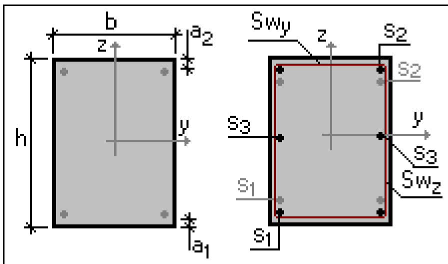
Forces:
N = 0 kN
M_y = 0 kN*m
Q_z = 62 kN
M_z = 0 kN*m
Q_y = 0 kN
T = 0 kN*m
Factor for sustained load 1

ARBAT initial data when the lateral force is Q = 58,4 kN:
Importance factor $\gamma_n = 1$
Importance factor (serviceability limit state) = 1
Member length 1 m
Effective length factor in the XoY plane 1
Effective length factor in the XoZ plane 1
Random eccentricity along Z according to SNiP 52-01-2003 (Russia)

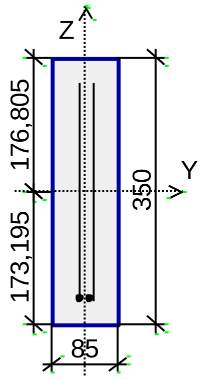
Random eccentricity along Y according to SNiP 52-01-2003 (Russia)
Structure is statically indeterminate
Limit slenderness - 200



Section:



b = 85 mm
h = 350 mm
a_1 = 32 mm
a_2 = 32 mm



S_1 - 2∅6
Transverse reinforcement along the Z axis 1∅8, spacing of transverse reinforcement 100 mm

Reinforcement	Class	Service factor
Longitudinal	A400	1
Transverse	A400	1

Concrete:
Concrete type: Heavy-weight
Concrete class: B15

Service factor for concrete		
γ_{b1}	allowance for the sustained loads	1
γ_{b2}	allowance for the failure behavior	1
γ_{b3}	allowance for the vertical position during concreting	1
γ_{b4}	allowance for the freezing/thawing and negative temperatures	1

Humidity of environmental air - 40-75%

Crack resistance:
No cracks

Forces:
N = 0 kN
M_y = 0 kN*m
Q_z = 58,4 kN
M_z = 0 kN*m
Q_y = 0 kN
T = 0 kN*m
Factor for sustained load 1

Comparison of solutions:

Check	strength in a concrete strip between oblique sections	strength for an oblique section
Guide	62/68,276 = 0,908	58,4/63,97 = 0,913
ARBAT	0,908	0,912
Deviation, %	–	0,11

Comments:

1. The strength check of oblique sections is performed by comparing a sum of lateral forces resisted by concrete and stirrups in the oblique section ($Q_b + Q_{sw}$), with a lateral force Q in the oblique section which is determined as a projection on the normal to the longitudinal axis of the element of the resultant of all external forces acting on the element on one side of the considered oblique section ($Q = Q_{max} – q_1 c$). In order to check the strength of the oblique section the value of the lateral force in the normal section is taken as Q = 58,4 kN according to the Guide.
2. The member length has to be specified in ARBAT. Since it is not defined in the problem, it is taken as 1 m.
3. The data on the longitudinal reinforcement has to be specified in ARBAT. Since it is not defined in the problem, the following reinforcement is used: class A400, rebars 2Ø6. Respectively, the value of the concrete cover is $a_1 = a_2 = a – d/2$ = 35 – 6/2 = 32 mm.

Calculation of a Simply Supported Floor Beam for Load-bearing Capacity under Lateral Forces

Objective: Check of the calculation of the resistance of reinforced concrete sections.

Task: Verify the correctness of the strength analysis of oblique sections.

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight concrete (no prestressing) (to SP 52-101-2003), 2005, p. 57-58.

Initial data file:
Example 13.SAV
report – Arbat 13.doc.

Compliance with the codes: SP 52-101-2003, SP 63.13330.2012.

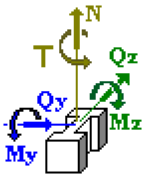
Initial data:

$b\times h$ = 200×400 mm	Beam section sizes
h_0 = 370 mm	Effective height of the beam section
d = 8 mm	Diameter of transverse reinforcement (two-leg stirrups)
s_w = 150 mm	Spacing of transverse reinforcement
q_v = 36 kN/m	Temporary equivalent load
q_g = 14 kN/m	Permanent load
$Q_{\max}$ = 137,5 kN	Lateral force on the support
Concrete class B25	
Class of reinforcement A240	

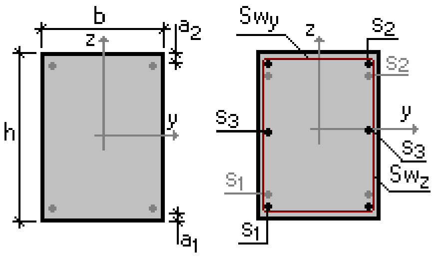
ARBAT initial data:
Importance factor γ_n = 1

Importance factor (serviceability limit state) = 1

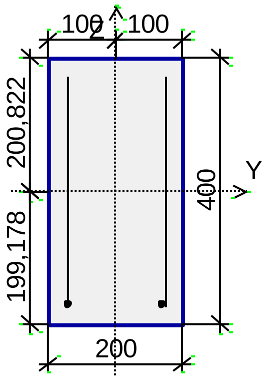
Member length 1 m
Effective length factor in the XoY plane 1
Effective length factor in the XoZ plane 1
Random eccentricity along Z according to SNiP 52-01-2003 (Russia)
Random eccentricity along Y according to SNiP 52-01-2003 (Russia)
Structure is statically indeterminate
Limit slenderness - 200



Section:



b = 200 mm
h = 400 mm
a_1 = 27 mm
a_2 = 27 mm



S_1 - 2∅6
Transverse reinforcement along the Z axis 2∅8,
spacing of transverse reinforcement 150 mm

Reinforcement	Class	Service factor
Longitudinal	A240	1
Transverse	A240	1

Concrete:
Concrete type: Heavy-weight
Concrete class: B25

Service factor for concrete		
γ_{b1}	allowance for the sustained loads	1
γ_{b2}	allowance for the failure behavior	1
γ_{b3}	allowance for the vertical position during concreting	1
γ_{b4}	allowance for the freezing/thawing and negative temperatures	1

Humidity of environmental air - 40-75%

Crack resistance:
No cracks

Forces:
N = 0 kN
M_y = 0 T*m
Q_z = 100,35 kN
M_z = 0 T*m
Q_y = 0 kN
T = 0 T*m
Factor for sustained load 1

Comparison of solutions:

Check	strength for an oblique section
Guide	100,35/100,69 = 0,997
ARBAT	0,982
Deviation, %	1,5

Comments:

1. The strength check of oblique sections is performed by comparing a sum of lateral forces resisted by concrete and stirrups in the oblique section ($Q_b + Q_{sw}$), with a lateral force Q in the oblique section which is determined as a projection on the normal to the longitudinal axis of the element of the resultant of all external forces acting on the element on one side of the considered oblique section ($Q = Q_{max} – q_1 c$). In order to check the strength of the oblique section the value of the lateral force in the normal section is taken as Q = 58,4 kN according to the Guide.
2. The member length has to be specified in ARBAT. Since it is not defined in the problem, it is taken as 1 m.
3. The data on the longitudinal reinforcement has to be specified in ARBAT. Since it is not defined in the problem, the following reinforcement is used: class A400, rebars 2Ø6. Respectively, the value of the concrete cover is $a_1 = a_2 = h – h_0 – d/2$ = 400 – 370 – 6/2 = 27 mm.

Calculation of a Column of a Multi-storey Frame for Load-bearing Capacity under a Lateral Force

Objective: Check of the calculation of the resistance of reinforced concrete sections.

Task: Verify the correctness of the strength analysis of oblique sections.

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight concrete (no prestressing) (to SNiP 52-101-2003), 2005, p. 104-105.

Initial data file:
Example 34.SAV;
report:
when the analysis is performed according to SNiP 52-01-2003 – Arbat 34.1.doc,
when the analysis is performed according to SP 63.13330.2012 – Arbat 34.2.doc.

Compliance with the codes: SNiP 52-101-2003, SP 63.13330.2012.

Initial data from the source:

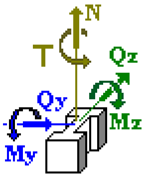
$b \times h$ = 400×600 mm	Column section sizes
l = 3,3 m	Column length (distance between support sections)
$a = a'$ = 50 mm	Distance from the center of gravity of the longitudinal reinforcement to the fiber of the section under the greatest tension
d = 12 mm	Diameter of transverse reinforcement
s_w = 400 mm	Spacing of transverse reinforcement
M_{sup} = 350 kN·m	Bending moment in the upper support section
M_{inf} = 250 kN·m	Bending moment in the lower support section
N = 572 kN	Longitudinal force
Concrete class	B25
Class of transverse reinforcement	A240

ARBAT initial data:

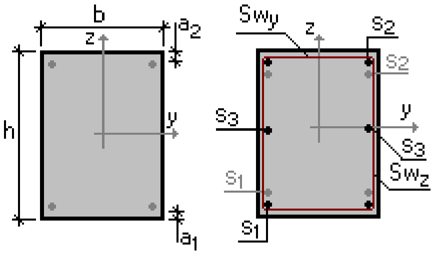
Importance factor $\square_n$ = 1

Importance factor (serviceability limit state) = 1

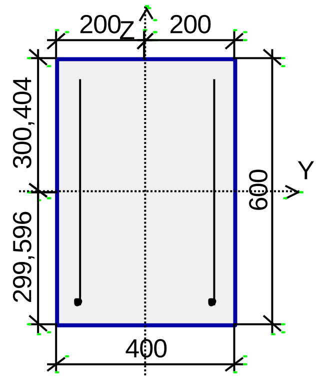
Member length 3,3 m
Effective length factor in the XoY plane 1
Effective length factor in the XoZ plane 1
Random eccentricity along Z according to SNiP 52-01-2003 (Russia)
Random eccentricity along Y according to SNiP 52-01-2003 (Russia)
Structure is statically indeterminate
Limit slenderness - 200



Section



b = 400 mm
h = 600 mm
a_1 = 47 mm
a_2 = 47 mm



S_1 - 2∅6
Transverse reinforcement along the Z axis 2∅12,
spacing of transverse reinforcement 400 mm

Reinforcement	Class	Service factor
Longitudinal	A400	1
Transverse	A240	1

Concrete
Concrete type: Heavy-weight
Concrete class: B25

Service factor for concrete		
γ_{b1}	allowance for the sustained loads	1
γ_{b2}	allowance for the failure behavior	1
γ_{b3}	allowance for the vertical position during concreting	1
γ_{b4}	allowance for the freezing/thawing and negative temperatures	1

Humidity of environmental air - 40-75%

Crack resistance:
No cracks

Forces

	N	M_y	Q_z	M_z	Q_y	T	Safety factor for load	Factor for sustained load	Short-term	Seismic
	kN	kN*m	kN	kN*m	kN	kN*m				
1	-572	600	181,8	0	0	0	1	1		

Comparison of solutions (according to SNiP 52-101-2003):

File	Example 34.SAV
Report file	Arbat 34.1.doc
Check	strength for an oblique section
Guide	181,8/184,8 = 0,984
ARBAT	0,982
Deviation, %	0,17 %

Comparison of solutions (according to SP 63.13330.2012):

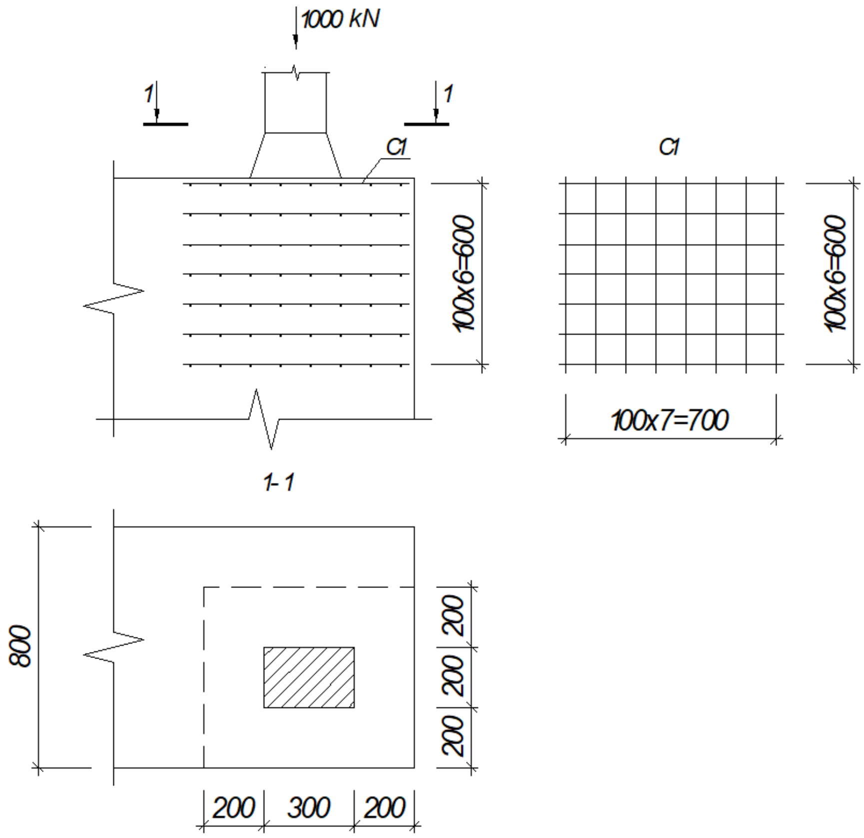
File	Example 34.SAV
Report file	Arbat 34.2.doc
Check	strength for an oblique section
Guide	181,8/184,8 = 0,984

ARBAT	0,843
Deviation, %	14,3%

Comments:

1. Bending moment M_y is determined as a sum of moments in the upper and lower support sections $M_y = M_{sup} + M_{inf}$ = 350 + 250 = 600 kN.
2. The lateral force in the column is determined as: $Q_z = M_y/l$ = 600/3,3 = 181,8 kN.
3. The data on the longitudinal reinforcement has to be specified in ARBAT. Since it is not defined in the problem, the following reinforcement is used: class A400, rebars 2Ø6. Respectively, the value of the concrete cover is $a_1 = a_2 = a - d/2$ = 50 – 6/2 = 47 mm.
4. The difference between the utilization factors of 14,3% in the results of the solution in the Guide and in ARBAT according to SP 63.13330.2012 is due to the fact that compressive stresses are taken into account in different ways according to the given codes (Sec. 8.1.34) and according to SNiP 52-101-2003.

Local Compression Analysis



Objective: Check the local compression analysis of the foundation

Task: Verify the correctness of the local compression analysis

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight concrete (no prestressing) (to SP 52-101-2003), 2005, p. 128-129.

Initial data file:
Example 39.SAV
report – Arbat 39.doc.

Compliance with the codes: SP 52-101-2003, SP 63.13330.2012.

Initial data:

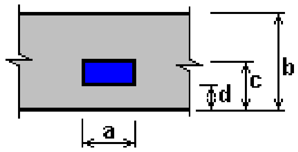
b = 0,8 m	Width of the foundation
$a_1 \times a_2$ = 300×200 mm	Sizes of the load application area
$A_{sx} = A_{sy}$ = 12,6 mm^2 (1Ø4)	Cross-sectional area of reinforcement
N = 1000 kN	Vertical load
Concrete class	B10
Class of reinforcement	B500

ARBAT initial data:

Importance factor $\gamma_n = 1$

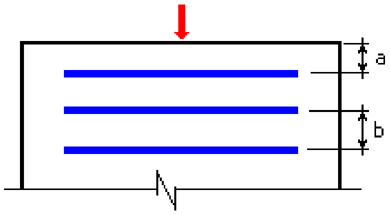
Load arrangement:

Local load near one edge of an element



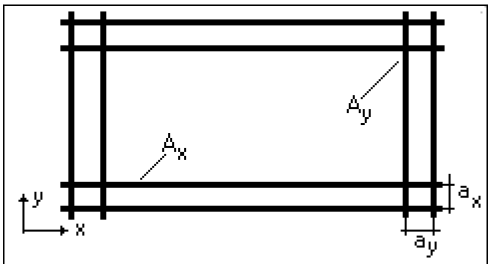
a = 300 mm
b = 800 mm
c = 400 mm
d = 200 mm

Lateral reinforcement by flat meshes:
Class of reinforcement: B500
Arrangement of meshes



Concrete cover a = 20 mm
Spacing of meshes b = 100 mm
Number of meshes - 4

Meshes:



Meshes

Rebars along X
Diameter 4 mm
Spacing a_x = 100 mm
Number of rebars - 7

Rebars along Y
Diameter 4 mm
Spacing a_y = 100 mm
Number of rebars - 8

Concrete:
Concrete type: Heavy-weight
Concrete class: B10

Service factor for concrete		
γ_{b1}	allowance for the sustained loads	1
γ_{b2}	allowance for the failure behavior	1
γ_{b3}	allowance for the vertical position during concreting	1
γ_{b4}	allowance for the freezing/thawing and negative temperatures	1

Comparison of solutions:

Check	strength condition of local compression
Guide	1000/1147,2 = 0,872
ARBAT	0,873
Deviation, %	0,1 %

Punching Analysis of a Reinforced Concrete Floor Slab

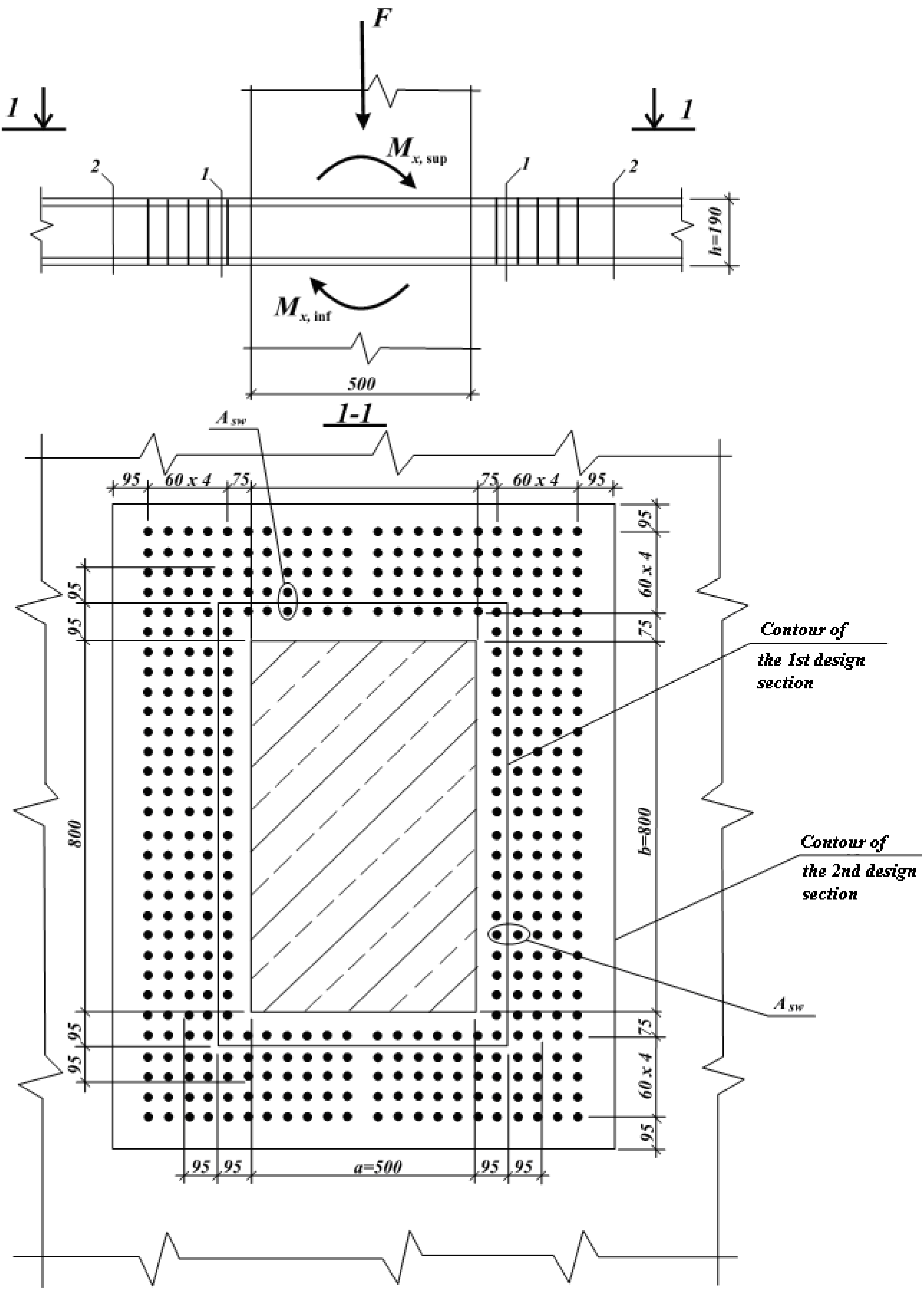


Figure 1. To the example of the calculation 40
1 - 1-st design section, 2 - 2-nd design section

Objective: Check the **Punching** mode.

Task: Verify the correctness of the punching strength analysis of a concrete element with transverse reinforcement under a concentrated force and bending moments and punching strength analysis beyond the boundary of transverse reinforcement.

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight concrete (no prestressing) (to SP 52-101-2003), 2005, p. 137-140.

Initial data file:
Example 40.SAV
report – Arbat 40.doc

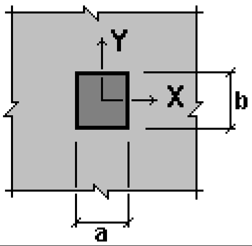
Compliance with the codes: SP 52-101-2003, SP 63.13330.2012.

Initial data from the source:

h = 220 mm	Slab thickness
$a \times b$ = 500×800 mm	Column section sizes
N = 800 kN	Load transferred from the floor slab to the column
$M_{x,sup}$ = 70 kN·m	Moment in the column section on the upper face of the slab in the direction of the X axis
$M_{y,sup}$ = 30 kN·m	The same in the direction of the Y axis
$M_{x,inf}$ = 60 kN·m	Moment in the column section on the lower face of the slab in the direction of the X axis
$M_{y,inf}$ = 27 kN·m	The same in the direction of the Y axis
d = 6 mm	Diameter of transverse reinforcement
Concrete class	B30
Class of reinforcement	A240

ARBAT initial data:
Importance factor $\gamma_n = 1$

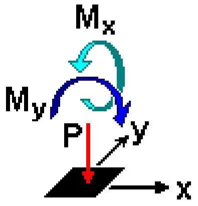
Load application area is inside the element

	a = 500 mm b = 800 mm Effective height of the section for longitudinal reinforcement along X-axis - 190 mm along Y-axis - 190 mm

Concrete:
Concrete type: Heavy-weight
Concrete class: B30

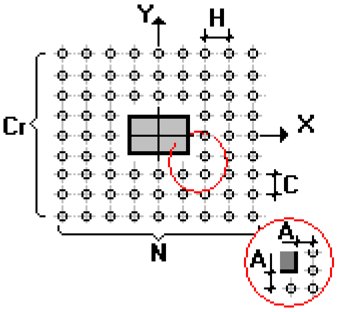
Service factor for concrete		
γ_{b1}	allowance for the sustained loads	1
γ_{b2}	allowance for the failure behavior	1
γ_{b3}	allowance for the vertical position during concreting	1
γ_{b4}	allowance for the freezing/thawing and negative temperatures	1

Loads:



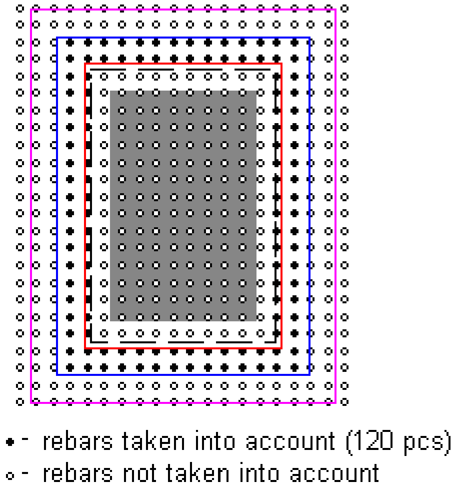
	P	M_x	M_y
	kN	kN*m	kN*m
1	800	57	130

Uniform reinforcement:



Class of reinforcement: A240
Diameter 6 mm

Distance to the load application area 75 mm
Spacing of rebars in a row 60 mm
Number of rebars in a row 20
Spacing of rows 60 mm
Number of rows of rebars 25



Forces:

P = 800 kN
M_x = 57 kN*m
M_y = 130 kN*m

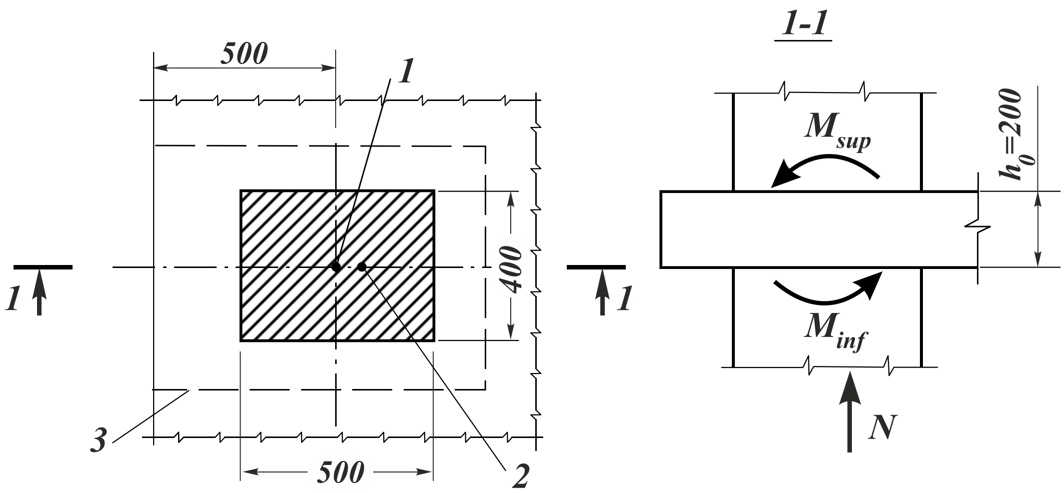
Comparison of solutions:

Check	punching strength of a concrete element with transverse reinforcement under a concentrated force and bending moments with their vectors along X and Y axes	punching strength of a concrete element with transverse reinforcement under a concentrated force beyond the boundary of transverse reinforcement
Guide	343,5/347,7 = 0,988	146,5/218,5 = 0,67
ARBAT	0,973	0,75
Deviation, %	1,518%	10,667%

Comments:

1. The average effective height of the slab is taken as h_0 = 190 mm in the calculation of the problem in the Guide. This value is used in ARBAT.
2. In the Guide moments M_x and M_y are moments in the directions of X and Y axes respectively. In ARBAT moments M_x and M_y are moments about X and Y axes respectively, therefore moments M_x and M_y in the example of the Guide correspond to the moments M_y and M_x in ARBAT. The values of the sum of moments M_{sup} and M_{inf} on the upper and lower faces of the slab are used in ARBAT. Thus, M_x = 30 + 27 = 57 kN·m, M_y = 70 + 60 = 130 kN·m.
3. The number of rebars in a row 20 and the number of rows of rebars 25 are taken in accordance with the sizes given in the drawing in the Guide.
4. The difference between the second factor and the solution from the Guide is due to the following reasons:
   - in the problem the boundaries of the second design contour are considered at the distance of 0,5h_0 from the boundary of the specified transverse reinforcement. Moreover, in the calculation of the geometric properties in the Guide the sizes of the contour were incorrectly taken as greater by 0,5h_0 than the sizes of the considered contour. In ARBAT the boundaries of the second design contour were taken at the distance of 0,5h_0 from the boundary of the transverse reinforcement considered in the calculation;
   - in the Guide this strength check is performed taking into account the bending moments. In ARBAT the check is performed according to Sec.6.2.48 of SP 52-101-2003 by the formula for the punching analysis under the action of a concentrated force.

Punching Analysis of a Flat Monolithic Floor Slab



1 – force application point *N*; 2 – center of gravity of the open contour; 3 – open contour of the design section

Objective: Check the **Punching** mode.

Task: Verify the correctness of the punching strength analysis of a concrete element under a concentrated force and a bending moment when the load application area is near the edge of the slab.

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight concrete (no prestressing) (to SP 52-101-2003), 2005, p. 140-142.

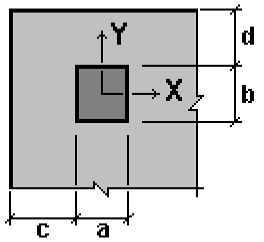
Initial data file:
Example 41.SAV;
report:
when the analysis is performed according to SNiP 52-01-2003 – Arbat 41.1.doc,
when the analysis is performed according to SP 63.13330.2012 – Arbat 41.2.doc.

Compliance with the codes: SNiP 52-101-2003, SP 63.13330.2012.

Initial data from the source:

h = 230 mm	Slab thickness
$a \times b$ = 500×400 mm	Column section sizes
N = 150 kN	Load transferred from the floor slab to the column
M_{sup} = 80 kN·m	Moment in the column section on the upper face of the slab
M_{inf} = 90 kN·m	Moment in the column section on the lower face of the slab
x_0 = 500 mm	Distance from the center of the column section to the free edge of the slab
Concrete class	B25

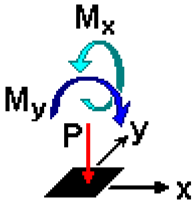
ARBAT initial data:
Importance factor γ_n = 1
Load application area is near the free edge of the element

	a = 0,5 m b = 0,4 m c = 0,25 m d = 4 m Effective height of the section for longitudinal reinforcement along X-axis - 0,2 m along Y-axis - 0,2 m

Concrete:
Concrete type: Heavy-weight
Concrete class: B25

Service factor for concrete		
γ_{b1}	allowance for the sustained loads	1
γ_{b2}	allowance for the failure behavior	1
γ_{b3}	allowance for the vertical position during concreting	1
γ_{b4}	allowance for the freezing/thawing and negative temperatures	1

Loads:



	P	M_x	M_y
	kN	kN*m	kN*m
1	150	0	170

Forces:
P = 150 kN
M_x = 0 kN*m
M_y = 170 kN*m

Comparison of solutions (according to SP 52-101-2003):

Report file	Arbat 41.1.doc	
Check	punching strength of a concrete element under a concentrated force and bending moments with their vectors along X and Y axes	punching strength of an unclosed concrete element under a concentrated force and bending moments (including additional ones caused by the eccentric application of a force with respect to the punched contour) with their vectors along X,Y-axes (load application area is near the edge of the slab)
Guide	203,4/210 = 0,969	202,2/210 = 0,963
ARBAT	0,549	0,621

Deviation, %	43,4%	35,5%
Analytical solution (see below)	0,550	0,622
Deviation, %	0,1 %	0,1 %

Comparison of solutions (according to SP 63.13330.2012):

Report file	Arbat 41.2.doc	
Check	punching strength of a concrete element under a concentrated force and bending moments with their vectors along X and Y axes	punching strength of an unclosed concrete element under a concentrated force and bending moments (including additional ones caused by the eccentric application of a force with respect to the punched contour) with their vectors along X,Y-axes (load application area is near the edge of the slab)
ARBAT	0,413	0,466
Analytical solution (see below)	0,412	0,466
Deviation, %	0,1 %	0 %

Comments:

1. The average effective height of the slab is taken as h_0 = 200 mm in the calculation of the problem in the Guide. This value is used in ARBAT.
2. The value of the sum of moments M_{sup} and M_{inf} on the upper and lower faces of the slab is used in ARBAT. Thus, M = 80 + 90 = 170 kN·m.
3. Distance from the edge of the load application area to the free edge of the slab c is equal to the difference between the distance from the center of the column section to the free edge of the slab and half the size of the column section in this direction: $c = x_0 - a/2$ = 0,5 – 0,5/2 = = 0,25 m.
4. In order to analyze the case when the (column) load transfer area is located near the edge of the flat element (floor slab), in ARBAT one of the values of the distance from the edge of the load application area to the free edge of the slab has to be greater than three times the effective height of the slab. Thus, d = 4 m > $3h_0$ = 0,6 m.
5. Such significant differences in the obtained factors with the solution from the Guide are due to the following reasons:
   - it is indicated in the codes that the calculations use the smallest values of the section moduli W_{bx}, determined from the following formulas:

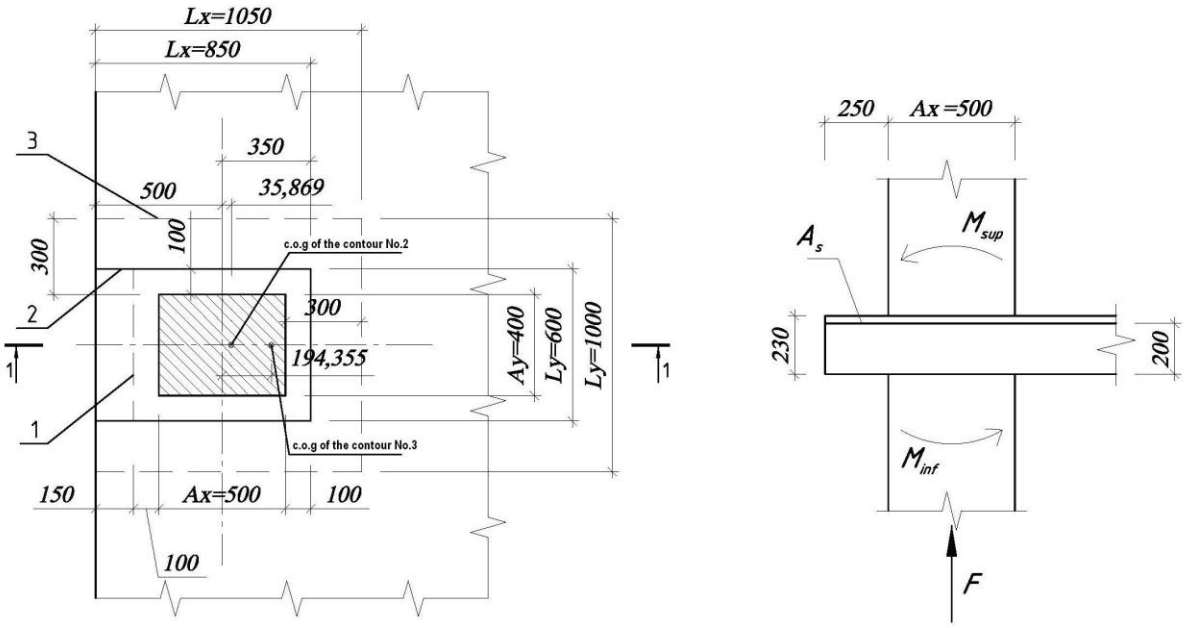
$$W_{\mathrm{bx}} = \frac{I_{\mathrm{bx}}}{x_0} \text{ and } W_{\mathrm{bx}} = \frac{I_{\mathrm{bx}}}{L_{\mathrm{x}} - x_0}.$$

   In this problem the smaller value is the one determined by the first formula, since x_0 = 0,5 + 0,0359 = 0,5359 m > $L_x - x_0$ = 0,85 – 0,5359 = 0,3141 m (where x_0 is the position of the center of gravity of the design open contour in the direction of the X axis). Thus, the value W_{bx} determined by the first formula is used in ARBAT. While the value determined by the second formula is used in the Guide;
   - the check of the strength requirements in the Guide does not take into account the recommendations of the codes according to which under the action of concentrated moments and a force the ratio between the acting concentrated moments M, taken into account at punching, and the ultimate ones M_{ult} should be taken not greater than the ratio between the acting concentrated force F and the ultimate one F_{ult} (Sec. 6.2.46 of

SNiP 52-101-2003) and not greater than half the ratio between the acting concentrated force F and the ultimate one F_{ult} (Sec. 8.1.46 of SP 63.13330.2012).

6. The analytical solution is given below.

Analytical solution



1 – closed design contour №1, 2 – open design contour №2, 3 – open design contour №3.

In this case it is necessary to check the strength of three contours of the design cross-section:

contour №1 – closed contour around the column section at a distance of $0,5h_0$ from the column contour;

contour №2 – open contour around the column section at a distance of $0,5h_0$ from the column contour with the extension of the contour to the free edge of the slab;

contour №3 – open contour around the column section at a distance of $1,5h_0$ from the column contour (contour of the verification analysis without the consideration of the reinforcement).

Closed contour №1:

$L_x = A_x + h_0$ = 500 + 200 = 700 mm = 0,7 m,

$L_y = A_y + h_0$ = 400 + 200 = 600 mm = 0,6 m,

Perimeter of the design contour of the cross-section:

$u = 2(L_x + L_y) = 2$ (0,7 + 0,6) = 2,6 m.

Area of the design contour of the cross-section:

$A_b = uh_0 =$ 2,6 × 0,2 = 0,52 m^2.

Ultimate force resisted by concrete:

$F_{b,ult} = R_{bt} A_b =$ 1,05 ×10^3 × 0,52 = 546 kN.

Moment of inertia of the design contour with respect to the X axis passing through its center of gravity:

$$I_{bx} = 2\frac{L_y^3}{12} + 2L_x\left(\frac{L_y}{2}\right)^2 = 2\frac{0,6^3}{12} + 2\cdot 0,7\left(\frac{0,6}{2}\right)^2 = 0{,}162 \text{ m}^3.$$

Section modulus of the design contour of concrete:

$$W_{bx} = \frac{I_{bx}}{y_{\max}} = \frac{0{,}162}{0{,}3} = 0{,}54 \text{ m}^2.$$

Moment of inertia of the design contour with respect to the Y axis passing through its center of gravity:

$$I_{by} = 2\frac{L_x^3}{12} + 2 \cdot L_y \left(\frac{L_x}{2}\right)^2 = 2\frac{0{,}7^3}{12} + 2 \cdot 0{,}6\left(\frac{0{,}7}{2}\right)^2 = 0{,}204 \text{ m}^3.$$

Section modulus of the design contour of concrete:

$$W_{by} = \frac{I_{by}}{x_{\max}} = \frac{0{,}204}{0{,}35} = 0{,}583 \text{ m}^2.$$

Bending moment which can be resisted by concrete in the design cross-section:

$$M_{bx,ult} = R_{bt} W_{bx} h_0 = 1{,}05 \times 10^3 \times 0{,}54 \times 0{,}2 = 113{,}4 \text{ kNm}.$$

$$M_{by,ult} = R_{bt} W_{by} h_0 = 1{,}05 \times 10^3 \times 0{,}583 \times 0{,}2 = 122{,}4 \text{ kNm}.$$

For SNiP 52-101-2003:

$$\frac{M_x}{M_{bx,ult}} \le \frac{F}{F_{b,ult}}; \qquad \frac{M_y}{M_{by,ult}} \le \frac{F}{F_{b,ult}}$$

$$\frac{M_y}{M_{by,ult}} = \frac{85}{122{,}4} = 0{,}694 \le \frac{F}{F_{b,ult}} = \frac{150}{546} = 0{,}275$$ – condition is not met.

Assume

$$\frac{M_y}{M_{by,ult}} = \frac{F}{F_{b,ult}} = 0{,}275$$

Punching strength of the slab:

$$K_1 = \left[\frac{F}{F_{b,ult}} + \frac{M_x}{M_{bx,ult}} + \frac{M_y}{M_{by,ult}}\right] \le 1{,}0$$

$$K_1 = 0{,}275 + 0 + 0{,}275 = 0{,}55$$

For SP 63.13330.2012:

$$\frac{M_x}{M_{bx,ult}} + \frac{M_y}{M_{by,ult}} \le 0{,}5\frac{F}{F_{b,ult}}$$

$$\frac{M_y}{M_{by,ult}} = \frac{85}{122{,}4} = 0{,}694 \le 0{,}5\frac{F}{F_{b,ult}} = \frac{150}{546} = 0{,}5 \cdot 0{,}275 = 0{,}1375$$ – condition is not met.

Assume

$$\frac{M_y}{M_{by,ult}} = \frac{F}{F_{b,ult}} = 0{,}1375$$

Punching strength of the slab:

$$K_1 = \left[\frac{F}{F_{b,ult}} + \frac{M_x}{M_{bx,ult}} + \frac{M_y}{M_{by,ult}}\right] \le 1{,}0$$

$$K_1 = 0{,}275 + 0 + 0{,}1375 = 0{,}413$$

Open contour №2:

$$L_x = A_x + h_0 + 150 = 500 + 200 + 150 = 850 \text{ mm} = 0{,}85 \text{ m},$$

$$L_y = A_y + h_0 = 400 + 200 = 600 \text{ mm} = 0{,}6 \text{ m},$$

Perimeter of the design contour of the cross-section:

$u = 2L_x + L_y$ = 2×0,85 + 0,6 = 2,3 m.

Area of the design contour of the cross-section:

$A_b = uh_0 =$ 2,3 × 0,2 = 0,46 m^2.

X coordinate of the center of gravity of the open contour with respect to the left edge of the slab:

$$X = \frac{425 \cdot 850 \cdot 2 + 850 \cdot 600}{850 \cdot 2 + 600} = 535{,}869 \text{ mm}$$

Ultimate force resisted by concrete:

$F_{b,ult} = R_{bt} A_b =$ 1,05 ×10^3 × 0,46 = 483 kN.

Moment of inertia of the design contour with respect to the X axis passing through its center of gravity:

$$I_{bx} = \frac{L_y^3}{12} + 2L_x \left(\frac{L_y}{2}\right)^2 = \frac{0{,}6^3}{12} + 2 \cdot 0{,}85 \left(\frac{0{,}6}{2}\right)^2 = 0{,}171 \text{ m}^3.$$

Section modulus of the design contour of concrete:

$$W_{bx} = \frac{I_{bx}}{y_{\max}} = \frac{0{,}171}{0{,}3} = 0{,}57 \text{ m}^2.$$

Moment of inertia of the design contour with respect to the Y axis passing through its center of gravity:

$$I_{by} = 2\frac{L_x^3}{12} + 2L_x(0{,}075 + 0{,}035869)^2 + L_y\left(0{,}35 - 0{,}035869\right)^2 = 2\frac{0{,}85^3}{12} + 2 \cdot 0{,}85(0{,}075 + 0{,}035869)^2 + 0{,}6\left(0{,}35 - 0{,}035869\right)^2 = 0{,}183 \text{ m}^3.$$

Section modulus of the design contour of concrete:

$$W_{by} = \frac{I_{by}}{x_{\max}} = \frac{0{,}183}{0{,}535869} = 0{,}341 \text{m}^2.$$

Bending moment which can be resisted by concrete in the design cross-section:

$M_{bx,ult} = R_{bt} W_{bx} h_0 =$ 1,05 ×10^3 × 0,57 × 0,2 = 119,7 kNm;

$M_{by,ult} = R_{bt} W_{by} h_0 =$ 1,05 ×10^3 × 0,341 × 0,2 = 71,6 kNm;

$M_y = M_y - Fe_0 =$ 85 – 150×0,035869 = 85 – 5,38 = 79,62 kNm.

For SNiP 52-101-2003:

$$\frac{M_x}{M_{bx,ult}} \le \frac{F}{F_{b,ult}}; \qquad \frac{M_y}{M_{by,ult}} \le \frac{F}{F_{b,ult}}$$

$\dfrac{M_y}{M_{by,ult}} = \dfrac{79{,}62}{71{,}6} = 1{,}112 \le \dfrac{F}{F_{b,ult}} = \dfrac{150}{483} = 0{,}311$ – condition is not met.

Assume

$$\frac{M_y}{M_{by,ult}} = \frac{F}{F_{b,ult}} = 0{,}311$$

Punching strength of the slab:

$$K_1 = \left[\frac{F}{F_{b,ult}} + \frac{M_x}{M_{bx,ult}} + \frac{M_y}{M_{by,ult}}\right] \le 1{,}0$$

K_1 = 0,311+0+0,311 = 0,622

For SP 63.13330.2012:

$$\frac{M_x}{M_{bx,ult}} + \frac{M_y}{M_{by,ult}} \le 0{,}5\frac{F}{F_{b,ult}}$$

$$\frac{M_y}{M_{by,ult}} = \frac{79,62}{71,6} = 1,112 \le 0,5\frac{F}{F_{b,ult}} = \frac{150}{483} = 0,5 \cdot 0,311 = 0,155$$ – condition is not met.

Assume

$$\frac{M_y}{M_{by,ult}} = \frac{F}{F_{b,ult}} = 0,155$$

Punching strength of the slab:

$$K_1 = \left[\frac{F}{F_{b,ult}} + \frac{M_x}{M_{bx,ult}} + \frac{M_y}{M_{by,ult}}\right] \le 1,0$$

K_1 = 0,311+0+0,155 = 0,466

Open contour №3:

$L_x = A_x + 1,5h_0 + 250$ = 500 +1,5×200 + 250 = 1050 mm = 1,05 m,

$L_y = A_y + 2 \cdot 1,5h_0$ = 400 + 2×1,5×200 = 1000 mm = 1,0 m,

Perimeter of the design contour of the cross-section:

$u = 2L_x + L_y$ = 2×1,05 + 1,0 = 3,1 m.

Area of the design contour of the cross-section:

$A_b = uh_0 = 3,1 \times 0,2 = 0,62$ m^2.

X coordinate of the center of gravity of the open contour with respect to the left edge of the slab:

$$X = \frac{525 \cdot 1050 \cdot 2 + 1050 \cdot 1000}{1050 \cdot 2 + 1000} = 694,355 \text{ mm}$$

Ultimate force resisted by concrete:

$F_{b,ult} = R_{bt}A_b = 1,05 \times 10^3 \times 0,62$ = 651 kN.

Moment of inertia of the design contour with respect to the X axis passing through its center of gravity:

$$I_{bx} = \frac{L_y^3}{12} + 2L_x\left(\frac{L_y}{2}\right)^2 = \frac{1,05^3}{12} + 2 \cdot 1,05\left(\frac{1,0}{2}\right)^2 = 0,608 \text{ m}^3.$$

Section modulus of the design contour of concrete:

$$W_{bx} = \frac{I_{bx}}{y_{\max}} = \frac{0,608}{0,5} = 1,217 \text{ m}^2.$$

Moment of inertia of the design contour with respect to the Y axis passing through its center of gravity:

$$I_{by} = 2\frac{L_x^3}{12} + 2L_x(0,194355 - 0,025)^2 + L_y\left(1,05 - 0,694355\right)^2 = 2\frac{1,05^3}{12} + 2 \cdot 1,05(0,194355 - 0,025)^2 + 1,0\left(1,05 - 0,694355\right)^2 = 0,38 \text{ m}^3.$$

Section modulus of the design contour of concrete:

$$W_{by} = \frac{I_{by}}{x_{\max}} = \frac{0,38}{0,694355} = 0,547 \text{ m}^2.$$

Bending moment which can be resisted by concrete in the design cross-section:

$M_{bx,ult} = R_{bt}W_{bx}h_0 = 1,05 \times 10^3 \times 1,217 \times 0,2$ = 255,57 kNm.

$M_{by,ult} = R_{bt}W_{by}h_0 = 1,05 \times 10^3 \times 0,547 \times 0,2$ = 114,87 kNm.

$M_y = M_y - Fe_0$ = 85 – 150×0,194355 = 85 – 29,15 = 55,85 kNm.

For SNiP 52-101-2003:

$$\frac{M_x}{M_{bx,ult}} \le \frac{F}{F_{b,ult}}; \qquad \frac{M_y}{M_{by,ult}} \le \frac{F}{F_{b,ult}}$$

$\frac{M_y}{M_{by,ult}} = \frac{55,85}{114,87} = 0,486 \le \frac{F}{F_{b,ult}} = \frac{150}{651} = 0,23$ – condition is not met.

Assume

$\frac{M_y}{M_{by,ult}} = \frac{F}{F_{b,ult}} = 0,23$

Punching strength of the slab:

$$K_1 = \left[\frac{F}{F_{b,ult}} + \frac{M_x}{M_{bx,ult}} + \frac{M_y}{M_{by,ult}}\right] \le 1,0$$

$$K_1 = 0,23+0+0,23 = 0,46$$

For SP 63.13330.2012:

$$\frac{M_x}{M_{bx,ult}} + \frac{M_y}{M_{by,ult}} \le 0,5\frac{F}{F_{b,ult}}$$

$\frac{M_y}{M_{by,ult}} = \frac{55,85}{114,87} = 0,486 \le 0,5\frac{F}{F_{b,ult}} = \frac{150}{651} = 0,5 \cdot 0,23 = 0,115$ – condition is not met.

Assume

$\frac{M_y}{M_{by,ult}} = \frac{F}{F_{b,ult}} = 0,155$

Punching strength of the slab:

$$K_1 = \left[\frac{F}{F_{b,ult}} + \frac{M_x}{M_{bx,ult}} + \frac{M_y}{M_{by,ult}}\right] \le 1,0$$

$$K_1 = 0,23+0+0,115 = 0,345$$

Analysis of a Reinforced Concrete Foundation Slab for Normal Crack Opening

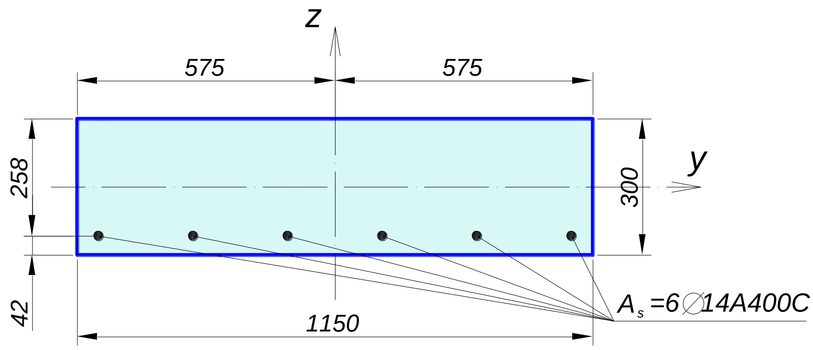


Figure 1. Design cross-section of the element

Objective: Check the calculation of the crack opening width.

Task: Verify the correctness of the analysis of normal crack opening.

References:
1. Guide on designing of concrete and reinforced concrete structures made of heavy-weight concrete (no prestressing) (to SP 52-101-2003), 2005, p. 155-157.
2. M.A. Perelmuter, K.V. Popok, L.N. Skoruk, *Calculation of the Normal Crack Opening Width for SP 63.13330.2012*, Concrete and Reinforced Concrete, 2014, №1, p.21,22

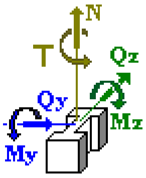
Initial data file:
Example 43.SAV
report – Arbat 43.doc

Compliance with the codes: SP 52-101-2003, SP 63.13330.2012.

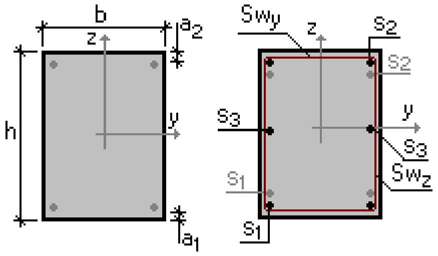
Initial data:

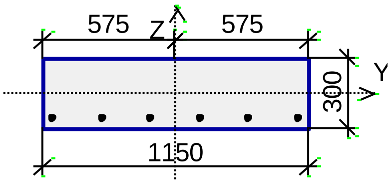
$b \times h$ = 1150×300 mm	Slab section sizes
a = 42 mm	Distance from the center of gravity of the reinforcement to the compressed edge of the section
A_s = 923 mm^2 (6Ø14)	Cross-sectional area of reinforcement
M_l = 50 kN·m	Moment in the design section from permanent and long-term loads
M_{sh} = 10 kN·m	Moment from short-term loads
Concrete class B15	
Class of reinforcement A400	

ARBAT initial data:
Importance factor γ_n = 1
Importance factor (serviceability limit state) = 1
Member length 1 m
Effective length factor in the XoY plane 1
Effective length factor in the XoZ plane 1
Random eccentricity along Z according to SNiP 52-01-2003 (Russia)
Random eccentricity along Y according to SNiP 52-01-2003 (Russia)
Structure is statically indeterminate
Limit slenderness - 200



Section:





b = 1150 mm
h = 300 mm
a_1 = 35 mm
a_2 = 35 mm

S_1 - 6∅14

Reinforcement	Class	Service factor
Longitudinal	A400	1
Transverse	A240	1

Concrete:
Concrete type: Heavy-weight
Concrete class: B15

Service factor for concrete		
γ_{b1}	allowance for the sustained loads	1
γ_{b2}	allowance for the failure behavior	1
γ_{b3}	allowance for the vertical position during concreting	1
γ_{b4}	allowance for the freezing/thawing and negative temperatures	1

Humidity of environmental air - 40-75%

Crack resistance:
Limited crack opening width
Requirements to crack opening width are based on the preservation of reinforcement
Allowable crack opening width:
  Short-term opening 0,4 mm
  Long-term opening 0,3 mm

Forces:
N = 0 kN
M_y = 60 kN*m
Q_z = 0 kN
M_z = 0 kN*m
Q_y = 0 kN
T = 0 kN*m
Factor for sustained load 0,83333

Theoretical solution:

Diagrams of the strain ε and stress σ distribution in concrete for the determination of the stress σ_s obtained in the theoretical calculation [2] based on the nonlinear deformation model are shown in Fig. 2. The following values of the internal longitudinal force N and bending moment M correspond to these diagrams

$$N = 0{,}00439 \text{ kN} \approx 0;$$

$$M = 50{,}096 \approx 50 \text{ kNm}.$$

There is a balance between internal and external forces. In this solution the stress in the tensile reinforcement is σ_s =236,692 MPa.

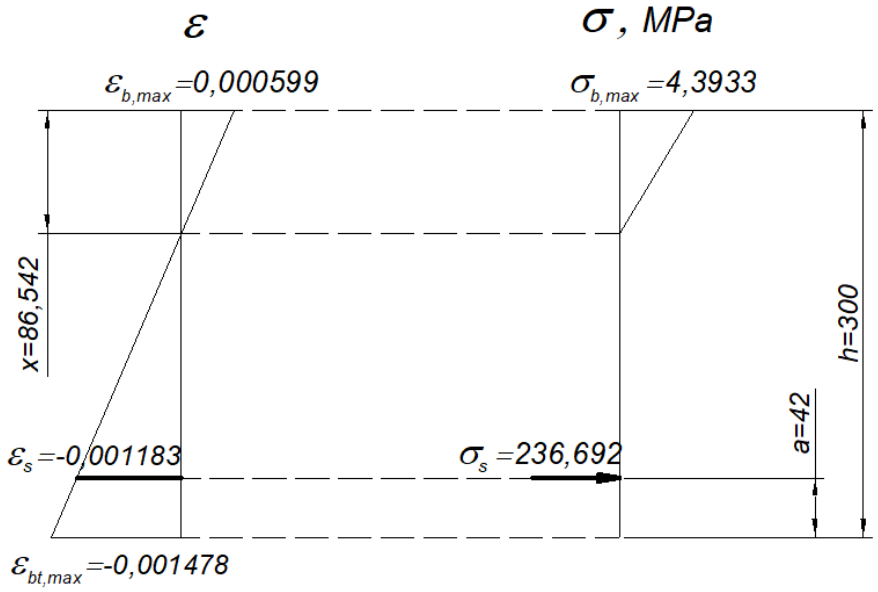


Figure 2. Strain ε and stress σ diagrams (for the determination of σ_s)

Similarly, solving the problem of determining the cracking moment, we obtain the following diagrams (Fig. 3), which satisfy the requirements of Sec. 8.2.14 of SP 63.13330.2012.

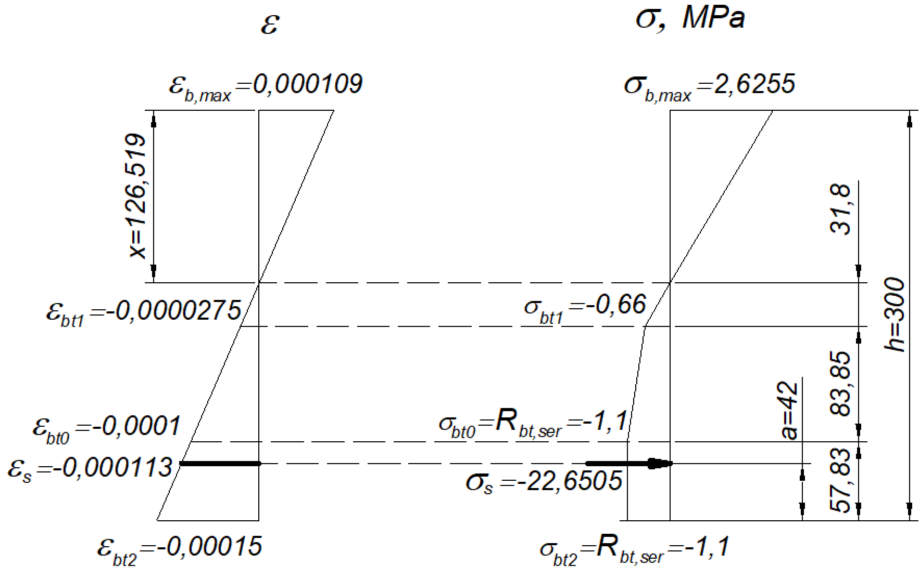


Figure 3. Strain ε and stress σ diagrams (for the determination of $\sigma_{s,crc}$)

In accordance with these diagrams M_{crc}= 36,244 kN·m, $\sigma_{s,crc}$ = 22,651 MPa.

On the basis of formula (1) (formula (8.128) SP 63.13330.2012) we obtain a_{crc} =**0,306** mm.

$$a_{crc} = \varphi_1 \cdot \varphi_2 \cdot \varphi_3 \cdot \psi_s \cdot \frac{\sigma_s}{E_s} \cdot l_s\,. \tag{1}$$

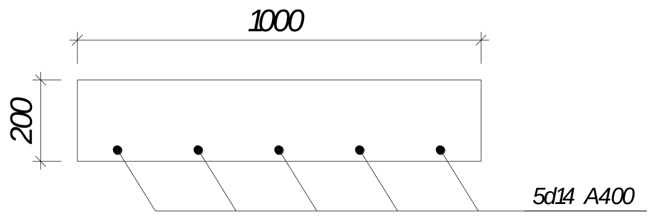
Comparison of solutions:

Check	crack opening width (long-term)
Theory	0,306/0,3 = 1,02
ARBAT	0,974
Deviation, %	4,51%

Comments:

1. The member length and the class of transverse reinforcement have to be specified in ARBAT. Since they are not determined in the problem, the following data are used 1 m and A240, respectively.
2. The value of the concrete cover is equal to $a - d/2$ = 42 – 14/2 = 35 mm.
3. The value of the total moment acting in the section, $M = M_l + M_{sh}$ = 50 + 10 = 60 kN·m, factor for sustained load is equal to M_l/M = 50/60 = 0,833.
4. The crack opening width obtained in the Guide [1] is equal to 0.227 mm. Such a significant discrepancy with the above theoretical solution is due to the use of the approach based on the ultimate forces, instead of the nonlinear deformation model (see [2]).
5. The deviation of the results of ARBAT from the theoretical solution is due to the fact that in order to provide computational stability, diagrams in which the horizontal part of the graph $\sigma(\varepsilon)$ has a small slope are used in ARBAT instead of the perfect diagrams of the material behavior.

Slab Deflection Analysis



Objective: Check of the slab deflection analysis

Task: Verify the correctness of the deflection calculation

References: Guide on designing of concrete and reinforced concrete structures made of heavy-weight concrete (no prestressing) (to SP 52-101-2003), 2005, p. 173-174.

Initial data file:
Example 45.SAV
report – Arbat 45.doc.

Compliance with the codes: SP 52-101-2003, SP 63.13330.2012.

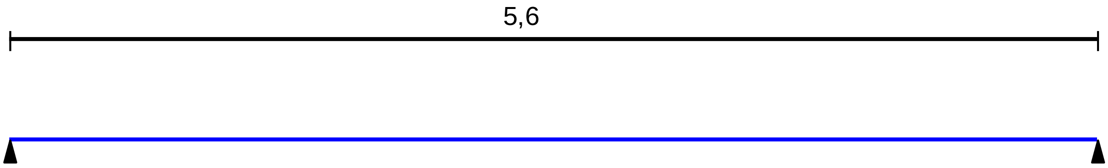
Initial data:

l=5,6 m	Slab span
$b\times h$ = 1000×200 mm	Slab section sizes
a = 27 mm	Distance from the center of gravity of the reinforcement to the compressed edge of the section
A_s = 769 mm^2 (5Ø14)	Cross-sectional area of reinforcement
q = 7 kN/m	Total uniformly distributed load
q_l = 6,5 kN/m	Part of the total uniformly distributed load from permanent and long-term loads
Concrete class	B15
Class of reinforcement	A400

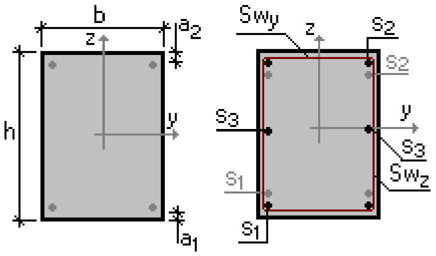
ARBAT initial data:
Importance factor γ_n = 1

Importance factor (serviceability limit state) = 1

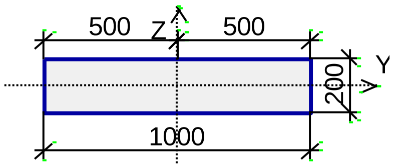
Structure:



Section:

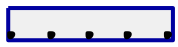


b = 1000 mm
h = 200 mm
a_1 = 20 mm
a_2 = 20 mm



Reinforcement	Class	Service factor
Longitudinal	A400	1
Transverse	A240	1

Specified reinforcement:

Span	Segment	Length (m)	Reinforcement	Section
span 1	1	5,6	S_1 - 5∅14	

Concrete:
Concrete type: Heavy-weight
Concrete class: B15
Density of concrete 2,5 T/m^3

Service factors for concrete		
γ_{b1}	allowance for the sustained loads	1
γ_{b2}	allowance for the failure behavior	1
γ_{b3}	allowance for the vertical position during concreting	1
γ_{b5}	allowance for the freezing/thawing and negative temperatures	1

Humidity of environmental air - 40-75%

Conditions of operation:
Mode of concrete humidity - Natural humidity
Humidity of environmental air - 40-75%

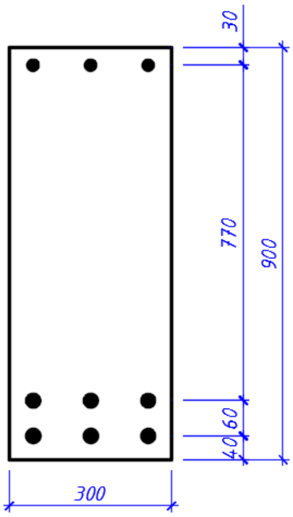
Comparison of solutions:

Check	maximum deflection
Guide	31,5 mm
ARBAT	32,847 mm
Deviation, %	4,2 %

Comment: The difference in the results is due to the fact that approximate empirical formulas are used in the Guide.

CALCULATIONS ACCORDING TO DBN V 2.6-98:2009

Section bearing capacity



Objective: Section bearing capacity

Task: Calculate ultimate moment strength of the section

References: Bliharsky Z.Y. Calculation and construction of bending reinforced concrete elements: Educational Guide / Z.Y. Bliharsky, I.I. Karhut. – Lviv: Lviv Polytechnic Publisher, 2017. – 188 p. (Example 5.10, pp. 73-75)

Initial data file:

ARBAT program section – Check, mode – Strength of RC Sections

Example-5.10-DBN.SAV
report – Arbat 5.10-DBN.doc.

Compliance with the codes: DBN V 2.6-98:2009, DSTU B.V 2.6-156:2010

Initial data:

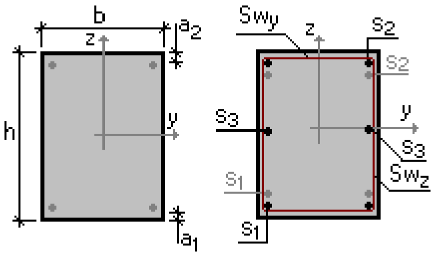
$b \times h$ = 300×900 mm	Beam section sizes
a_1= 27,5 mm	Distance from the edge of the lower reinforcement to the lower edge of the cross-section (protective layer)
A_2= 20 mm	Distance from the edge of the upper reinforcement to the upper edge of the section (protective layer)
A_{s1} = 2945 mm^2 (6Ø25)	Area of the lower reinforcement Cross-sectional area of reinforcement
A_{s2} = 942 mm^2 (3Ø20)	Area of the u reinforcement
M= 810,7 kNm	Bending moment
Concrete class	C20/25
Class of reinforcement	A400C

Initial data in ARBAT:

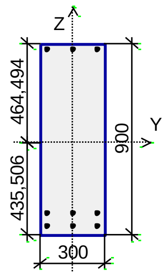
Importance factor γ_n = 1
Effective length factor in the XoY plane 1
Effective length factor in the XoZ plane 1
Random eccentricity along Z according to DBN B.2.6-98:2009 with modification No.1
Random eccentricity along Y according to DBN B.2.6-98:2009 with modification No.1

The structure is statically determinate

Section



b = 300 mm
h = 900 ммmm
a_1 = 27,5 mm
a_2 = 20 mm



S_1 - 3∅25, second line 3∅25
(Clear distance between rows 35 mm)
S_2 - 3∅20

Reinforcement	Class	Additional service factor
Longitudinal	A400C	1
Transverse	A240C	1

Concrete
Concrete type: Heavy-weight
Concrete class: C20/25
Aggregate: Quartzite

Additional parameters		
Additional service factor	1	
Age of concrete (days)	28	
Cement strength class	Class R	
Creep development time	36500	days
Temperature T(Δt)	20	°C
Number of days when the temperature T prevails Δt	28	days
Relative humidity	40	%

Results of analysis by load case combinations

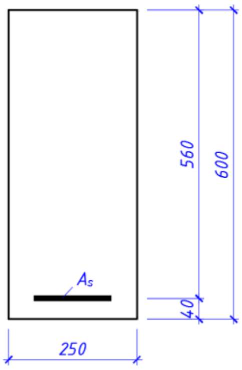
	N kN	M_y kN*m	Q_z kN	M_z kN*m	Q_y kN	T kN*m	Factor for sustained load	Short-term	Seismicity	Special
1	0	810,7	0	0	0	0	1			

Checked according to DBN	Check	Utilization Factor
	Ultimate moment strength of the section	0,991

Comparison of solutions

Check	Ultimate moment strength of the section
Guide	810,7 kNm
ARBAT	810,7/0,991 = 818,1 kNm
Deviation, %	0,9 %

Selection of beam reinforcement, Example 1



Objective: Selection of beam reinforcement

Task: Select area of the longitudinal reinforcement

References: Bliharsky Z.Y. Calculation and construction of bending reinforced concrete elements: Educational Guide / Z.Y. Bliharsky, I.I. Karhut. – Lviv: Lviv Polytechnic Publisher, 2017. – 188 p. (Example 4.1, p. 47-48)

Initial data file:

ARBAT mode – Selection of section reinforcement

Example-4.1-DBN.SAV
report – Arbat 4.1-DBN.doc.

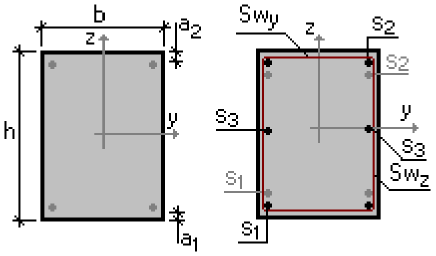
Compliance with the codes: DBN V 2.6-98:2009, DSTU B.V 2.6-156:2010

Initial data:

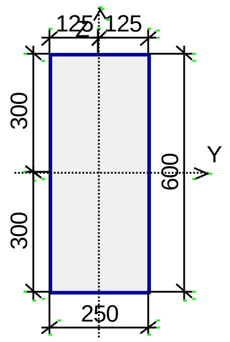
$b\times h$ = 250×600 mm	Cross-section sizes
a = 40 mm	Distance to the center of gravity of reinforcement
M= 200 кНм	Bending moment
Concrete class	C12/15
Class of reinforcement	A240C

Initial data in ARBAT:
Importance factor γ_n = 1
Member length 1 m
Effective length factor in the XoY plane 1
Effective length factor in the XoZ plane 1
Random eccentricity along Z according to DBN B.2.6-98:2009 with modification No.1
Random eccentricity along Y according to DBN B.2.6-98:2009 with modification No.1
The structure is statically determinate

Section



b = 250 mm
h = 600 mm
a_1 = 40 mm
a_2 = 40 mm



Reinforcement	Class	Additional service factor
Longitudinal	A240C	1
Transverse	A240C	1

Concrete
Concrete type: Heavy-weight
Concrete class: C12/15
Aggregate: Quartzite

Additional parameters		
Additional service factor	1	
Age of concrete (days)	28	
Cement strength class	Class R	
Creep development time	36500	days
Temperature T(Δt)	20	°C
Number of days when the temperature T prevails Δt	28	days
Relative humidity	40	%

	N	M_y	Q_z	M_z	Q_y	T	Factor for sustained load	Short-term	Seismicity	Special
	kN	kN*m	kN	kN*m	kN	kN*m				
1	0	200	0	0	0	0	1			

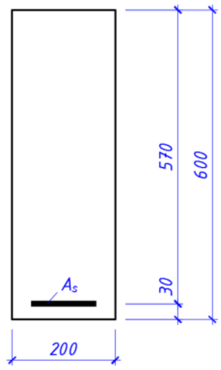
Results of the reinforcement selection

Type	Asymmetric reinforcement		Symmetric reinforcement	
	AS_1 cm^2	%	AS_1 cm^2	%
total	19,328	1,381	16,989	2,427

Comparison of solutions

Check	Selected reinforcement
Guide	1938,6 mm^2
ARBAT	1932,8 mm^2
Deviation, %	0,3 %

Selection of beam reinforcement, Example 2



Objective: Selection of beam reinforcement

Task: Select area of the longitudinal reinforcement

References: Bliharsky Z.Y. Calculation and construction of bending reinforced concrete elements: Educational Guide / Z.Y. Bliharsky, I.I. Karhut. – Lviv: Lviv Polytechnic Publisher, 2017. – 188 p. (Example 4.2, p. 48-49)

Initial data file:

ARBAT mode – Selection of section reinforcement

Example-4.2-DBN.SAV
report – Arbat 4.2-DBN.doc.

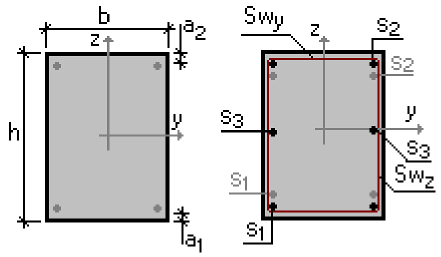
Compliance with the codes: DBN V 2.6-98:2009, DSTU B.V 2.6-156:2010

Initial data:

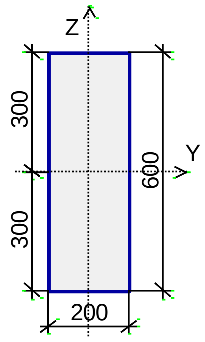
$b \times h$ = 200×600 mm	Cross-section sizes
a = 30 mm	Distance to the center of gravity of reinforcement
M= 135 кНм	Bending moment
Concrete class	C20/25
Class of reinforcement	A400C

Initial data in ARBAT:
Importance factor $\gamma_n = 1$
Member length 1 m
Effective length factor in the XoY plane 1
Effective length factor in the XoZ plane 1
Random eccentricity along Z according to DBN B.2.6-98:2009 with modification No.1
Random eccentricity along Y according to DBN B.2.6-98:2009 with modification No.1
The structure is statically determinate

Section



b = 200 mm
h = 600 mm
a_1 = 30 mm
a_2 = 30 mm



Reinforcement	Class	Additional service factor
Longitudinal	A400C	1
Transverse	A240C	1

Concrete
Concrete type: Heavy-weight
Concrete class: C20/25
Aggregate: Quartzite

Additional parameters		
Additional service factor	1	
Age of concrete (days)	28	
Cement strength class	Class R	
Creep development time	36500	days
Temperature T(Δt)	20	°C
Number of days when the temperature T prevails Δt	28	days
Relative humidity	40	%

	N	M_y	Q_z	M_z	Q_y	T	Factor for sustained load	Short-term	Seismicity	Special
	kN	kN*m	kN	kN*m	kN	kN*m				
1	0	135	0	0	0	0	1			

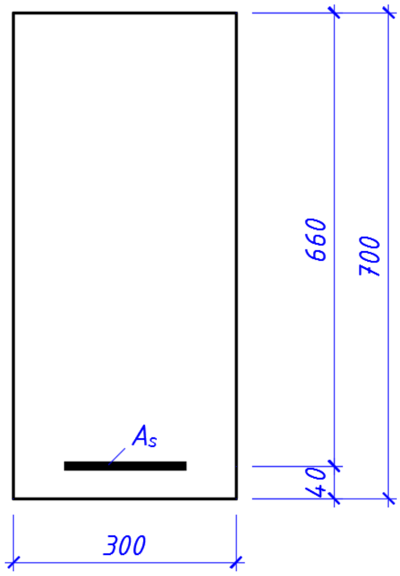
Results of the reinforcement selection

Type	Asymmetric reinforcement		Symmetric reinforcement	
	AS_1	%	AS_1	%
	cm^2		cm^2	
total	7,13	0,625	6,941	1,218

Comparison of solutions

Check	Selected reinforcement
Guide	707,6 mm^2
ARBAT	713,0 mm^2
Deviation, %	0,8 %

Selection of beam reinforcement, Example 3



Objective: Selection of beam reinforcement

Task: Select area of the longitudinal reinforcement

References: Bliharsky Z.Y. Calculation and construction of bending reinforced concrete elements: Educational Guide / Z.Y. Bliharsky, I.I. Karhut. – Lviv: Lviv Polytechnic Publisher, 2017. – 188 p. (Example 4.3, p. 50-51)

Initial data file:

ARBAT mode – Selection of section reinforcement

Example-4.3-DBN.SAV
report – Arbat 4.3-DBN.doc.

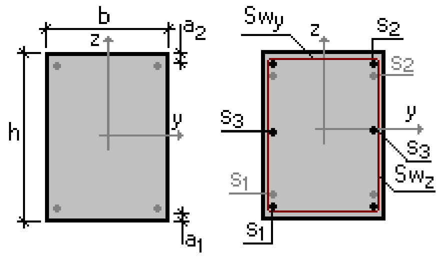
Compliance with the codes: DBN V 2.6-98:2009, DSTU B.V 2.6-156:2010

Initial data:

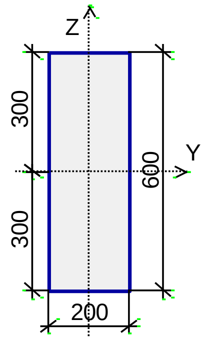
$b \times h$ = 200×700 mm	Cross-section sizes
a = 30 mm	Distance to the center of gravity of reinforcement
M= 475 кНм	Bending moment
Concrete class	C30/35
Class of reinforcement	A500C

Initial data in ARBAT:
Importance factor γ_n = 1
Member length 1 m
Effective length factor in the XoY plane 1
Effective length factor in the XoZ plane 1
Random eccentricity along Z according to DBN B.2.6-98:2009 with modification No.1
Random eccentricity along Y according to DBN B.2.6-98:2009 with modification No.1
The structure is statically determinate

Section



b = 200 mm
h = 600 mm
a_1 = 30 mm
a_2 = 30 mm



Reinforcement	Class	Additional service factor
Longitudinal	A500C	1
Transverse	A240C	1

Concrete
Concrete type: Heavy-weight
Concrete class: C20/25
Aggregate: Quartzite

Additional parameters		
Additional service factor	1	
Age of concrete (days)	28	
Cement strength class	Class R	
Creep development time	36500	days
Temperature T(Δt)	20	°C
Number of days when the temperature T prevails Δt	28	days
Relative humidity	40	%

	N	M_y	Q_z	M_z	Q_y	T	Factor for sustained load	Short-term	Seismicity	Special
	kN	kN*m	kN	kN*m	kN	kN*m				
1	0	475	0	0	0	0	1			

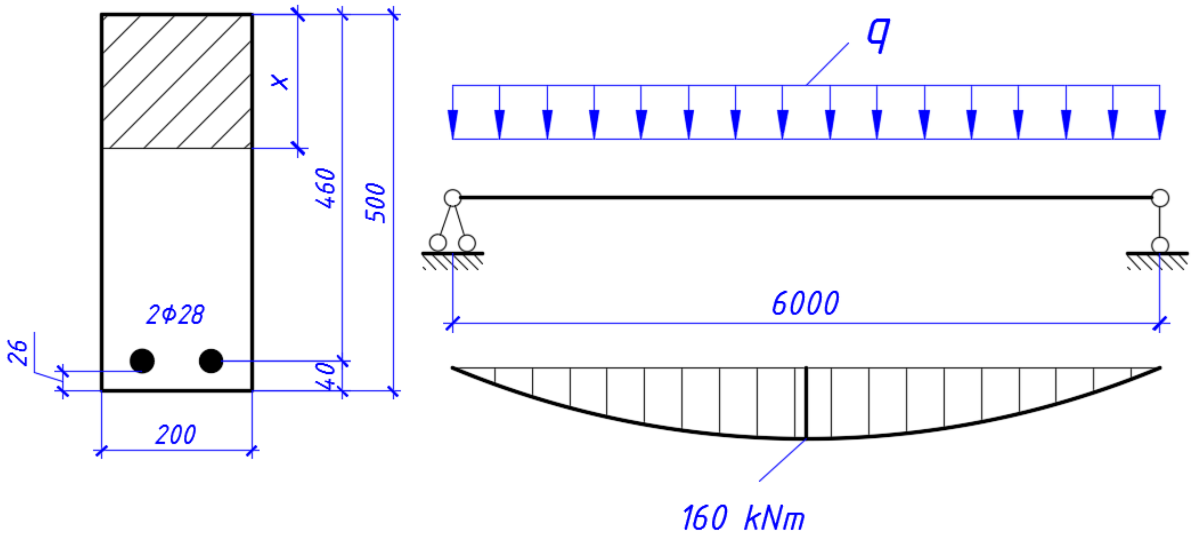
Results of the reinforcement selection

Type	Asymmetric reinforcement		Symmetric reinforcement	
	AS_1	%	AS_1	%
	cm^2		cm^2	
total	18,652	0,942	17,791	1,797

Comparison of solutions

Check	Selected reinforcement
Guide	1941,7 mm^2
ARBAT	1865,2 mm^2
Deviation, %	4,1 %

Beam Deflection Analysis



Objective: Calculate beam deflection

Task: Calculate the value of beam deflection

References: Practical calculation of elements of reinforced concrete structures according to DBN V. 2.6-98:2009 in comparison with calculations according to SNiP 2.03.01-84* and EN 1992-1-1 (Eurocode 2) / V.M. Babaev, A.M. Bambura, O.M. Pustovoitova et.al., edited by V.S. Shmukler, – Kharkiv: Golden Pages, 2015. – p. 280. (Example 15, p. 92.)
Initial data file:

ARBAT mode – Single-Span Beam Deflection

Example-15.1-DBN.sav
report – Arbat 15.1-DBN.doc

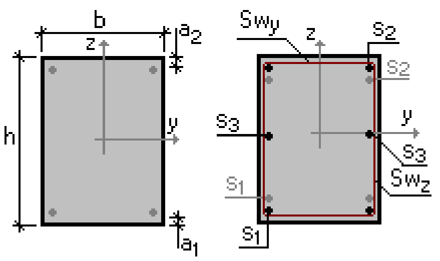
Compliance with the codes: DBN V 2.6-98:2009, DSTU B.V 2.6-156:2010

Initial data:

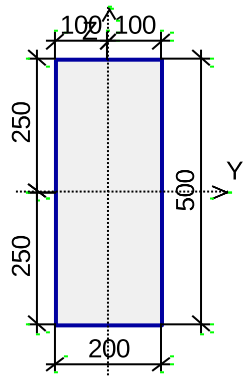
l=6,0 m	Beam span
b×h = 200×500 mm	Beam section sizes
a = 26 mm	Distance from the edge of the reinforcement to the edge of the section
A_s = 1232 mm^2 (2Ø28)	Cross-sectional area of reinforcement
q= 35,555 kN/m	Uniformly distributed load
Concrete class	C20/25
Class of reinforcement	A400C

Initial data in ARBAT:
Importance factor γ_n = 1
Member length 6 m

Section

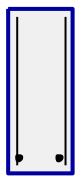


b = 200 mm
h = 500 mm
a_1 = 26 mm
a_2 = 26 mm



Reinforcement	Class	Additional service factor
Longitudinal	A400C	1
Transverse	A240C	1

Specified reinforcement

Span	Segment	Length (m)	Reinforcement	Section
span 1	1	6	S_1 - 2∅28 Transverse reinforcement along axis Z 2∅16, spacing of transverce reinforcement 100 mm	

Concrete

Concrete type: Heavy-weight
Concrete class: C20/25
Aggregate: Quartzite

Additional parameters		
Additional service factor	1	
Age of concrete (days)	28	
Cement strength class	Class R	
Creep development time	36500	days
Temperature T(Δt)	20	°C
Number of days when the temperature T prevails Δt	28	days
Relative humidity	40	%

Calculated deflections

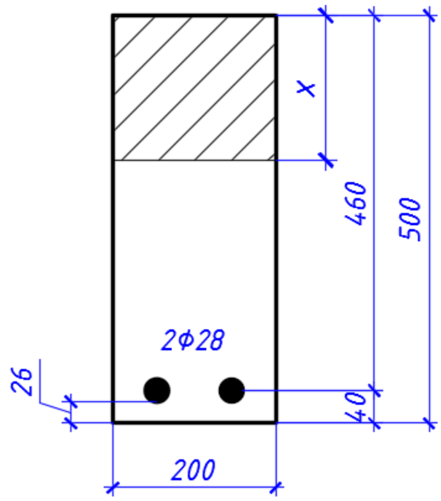
Span	Maximum deflection		Minimum deflection	
	Value	Snap	Value	Snap
	mm	m	mm	m
span 1	19,107	3,012	0	6

Comparison of solutions

Check	Deflection value
Guide	22,2 mm
ARBAT (M ≠ const)	19,107 mm
Deviation, %	16,2 %

Comment. The difference in the results is due to the fact that the Manual uses approximate empirical formulas for determining deflection, based on the use of curvature in the section with the maximum bending moment. In the ARBAT program, deflection is determined at the corresponding curvatures in different sections along the length of the beam.

Calculation of the crack opening width



Objective: Determine the crack resistance of a beam

Task: Determine the crack opening width

References: Practical calculation of elements of reinforced concrete structures according to DBN V. 2.6-98:2009 in comparison with calculations according to SNiP 2.03.01-84* and EN 1992-1-1 (Eurocode 2) / V.M. Babaev, A.M. Bambura, O.M. Pustovoitova et.al., edited by V.S. Shmukler, – Kharkiv: Golden Pages, 2015. – pp. 88-90.

Initial data file:

Example-14DBN.SAV
report – Arbat 14DBN.doc.

Compliance with the codes: DBN V 2.6-98:2009, DSTU B.V 2.6-156:2010

Initial data:

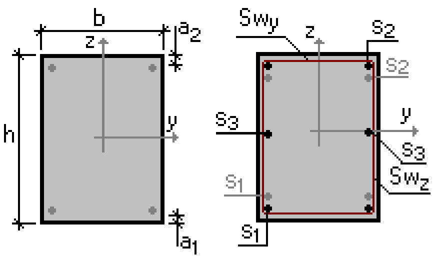
l=6,0 m	Beam span
$b \times h$ = 200×500 mm	Beam section sizes
a = 26 mm	Distance from the edge of the reinforcement to the edge of the section
A_s = 1232 mm^2 (2Ø28)	Cross-sectional area of reinforcement
M= 160 kNm	Bending moment
Concrete class	C20/25
Class of reinforcement	A400C

Initial data in ARBAT:

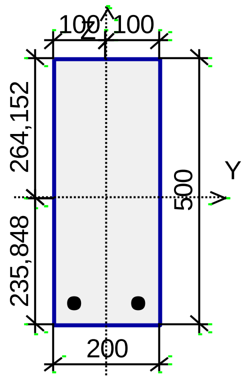
Importance factor γ_n = 1

Member length 6 m
Effective length factor in the XoY plane 1
Effective length factor in the XoZ plane 1
Random eccentricity along Z according to DBN B.2.6-98:2009 with modification No.1
Random eccentricity along Y according to DBN B.2.6-98:2009 with modification No.1
The structure is statically determinate

Section



b = 200 mm
h = 500 mm
a_1 = 26 mm
a_2 = 26 mm



S_1 - 2∅28

Reinforcement	Class	Additional service factor
Longitudinal	A400C	1
Transverse	A240C	1

Concrete

Concrete type: Heavy-weight
Concrete class: C20/25
Aggregate: Quartzite

Additional parameters		
Additional service factor	1	
Age of concrete (days)	28	
Cement strength class	Class R	
Creep development time	36500	days
Temperature T(Δt)	20	°C
Number of days when the temperature T prevails Δt	28	days
Relative humidity	40	%

Crack resistance

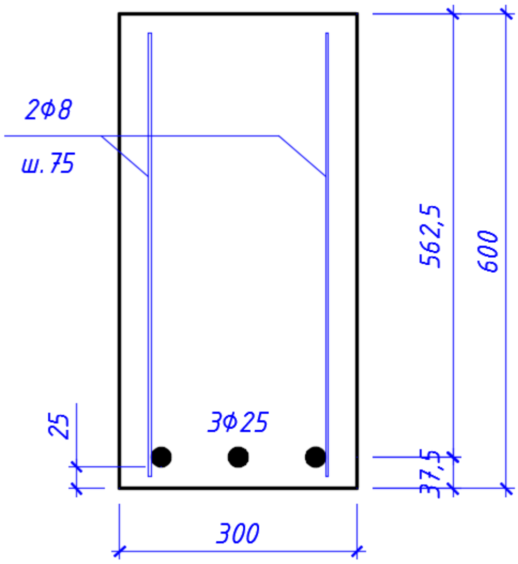
Parameters		
Maximum crack opening width w_{max}	0,3	mm

Checked according to DBN	Check	Utilization Factor
Sec. 7.3.4.2, Sec. 5.3.1.4 DSTU B V.2.6-156-2010	Crack opening width	0,869

Comparison of solutions

Check	Crack opening width
Guide	0,26 mm
ARBAT	0,869x0,3=0,261 mm
Deviation, %	0,4 %

Bearing capacity of inclined section



Objective: Determine the load-bearing capacity of an inclined section

Task: Check the shear resistance of an element with transverse reinforcement

References: Design of reinforced concrete structures. Guide / A.M. Bambura, I.R. Sazonova, O.V. Dorohova, O.V. Wojciechovsky; Under the editorship A.M. Bambura - Kyiv: Master of Books, 2018. - 240 p. (Example 5.2, p. 151-153)

Initial data file:

ARBAT mode – Strength of RC Section

Example-5.2-DBN.SAV
report – Arbat 5.2-DBN.doc

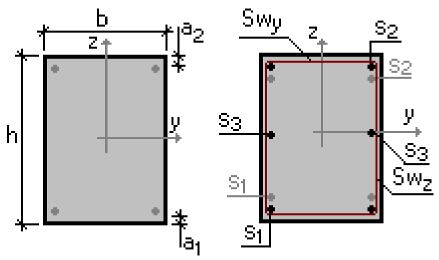
Compliance with the codes: DBN V 2.6-98:2009, DSTU B.V 2.6-156:2010

Initial data:

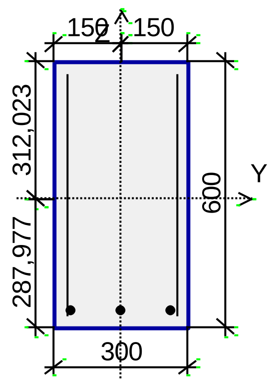
$b \times h$ = 300×600 мм	Cross-section sizes
a = 25 мм	The size of the protective layer of concrete
V_{Ed} = 91 кН	Design shear force
3∅25A500C	Lower tension reinforcement
Concrete class	C25/30
Класс продольной арматуры	A500C
Класс поперечной арматуры	A400C

Initial data in ARBAT:
Importance factor $\gamma_n = 1$
Importance factor (emergency state) 1
Member length 6 m
Effective length factor in the XoY plane 1
Effective length factor in the XoZ plane 1
Random eccentricity along Z according to DBN B.2.6-98:2009 with Change No 1
Random eccentricity along Y according to DBN B.2.6-98:2009 with Change No 1
Structure is statically determinate

Section



b = 300 mm
h = 600 mm
a_1 = 25 mm
a_2 = 25 mm



S_1 - 3∅25
Transverse reinforcement along axis Z 2∅8,
spacing of transverce reinforcement 75 mm

Reinforcement	Class	Additional service factor
Longitudinal	A500C	1
Transverse	A400C	1

Concrete
Concrete type: Heavy-weight
Concrete class: C25/30
Aggregate: Quartzite

Additional parameters		
additional service factor	1	
Concrete age (days)	28	
Cement strength class	Class R	
Creep progress time	36500	days
Temperature T(□t)	20	°C
The number of days where a temperature T prevails □t	28	days
Relative humidity	40	%

Results of analysis by load case combinations

	N	M_y	Q_z	M_z	Q_y	T	Factor for sustained load	Short-term	Seismicity	Special
	kN	T*m	kN	T*m	kN	T*m				
1	1	0	191	0	0	0	1			

Checked according to DBN	Check	Utilization Factor
Sec. 6.2.1.7	Shear resistance to Vz without transverse reinforcement	**1,915**
Sec. 6.2.1.6	Shear resistance to Vz with transverse reinforcement	0,987

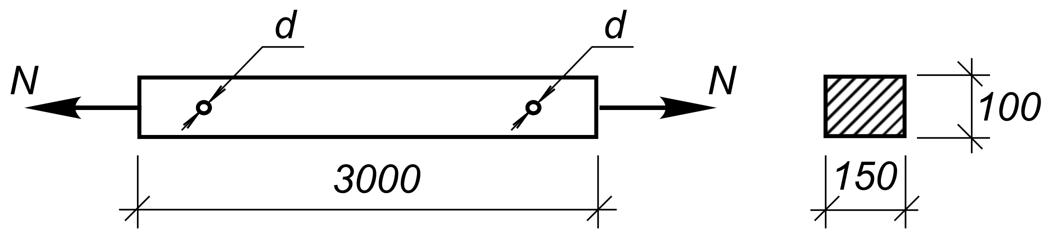
Comparison of solutions

Check	Shear resistance without transverse reinforcement
Guide	191/100=1,91
ARBAT	1,915
Deviation, %	0,3 %
Check	Shear resistance with transverse reinforcement

Guide	191/194=0,985
ARBAT	0,987
Deviation, %	0,2 %

DECOR

Check of the Load-bearing Capacity of a Bottom Truss Chord Section under Central Tension



Objective: Check of the calculation of the resistance of sections.

Task: Verify the correctness of the strength analysis of normal sections.

References: Nasonov S.B. Manual on design and analysis of building structures. – M: ASV Publishing House, 2013. – p. 62-63.

Initial data file: Example 3.SAV; report – Decor 3.doc.

Software version: DECOR 21.1.1.1, 17.02.2016.

Compliance with the codes: SNiP II-25-80, SP 64.13330.2011.

Initial data from the source:

$b \times h$ = 15×10 cm	Section sizes of the element
l = 3 m	Member length
d = 1,6 cm	Diameter of the hole
N = 60 kN	Tensile force

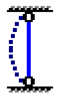
Material of the element: pine.
Grade of wood: 2.
Operating conditions class: 1 (A2 according to SNiP II-25-80).

DECOR initial data:

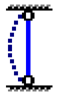
Importance factor $\gamma_n = 1$

Service factors	
Service factor for temperature and humidity operating conditions m_B	1
Allowance for the temperature conditions of operation m_T	1
Allowance for the duration of loading m_d	1
Service factor under short-term loads m_n	1
Factor that allows for the effect of impregnation with protective substances m_a	1

Wood species - Pine
Grade of wood - 2
Limit slenderness of members in tension - 200
Limit slenderness of members in compression - 120
Member length 3 m

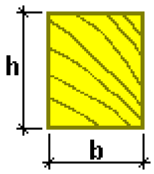


Effective length factor in the XoY plane - 1



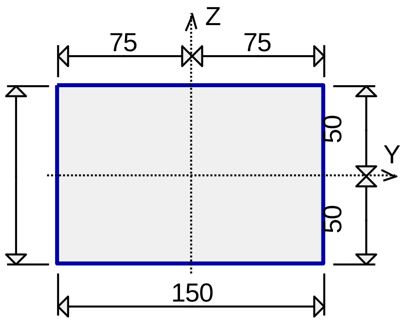
Effective length factor in the XoZ plane - 1

Section:



b = 150 mm
h = 100 mm

Non-glued timber section



Weakening not reaching the edge
Area of the weakening - 24 cm^2

Forces:

N = 60 kN
M_y = 0 kN*m
Q_z = 0 kN
M_z = 0 kN*m
Q_y = 0 kN

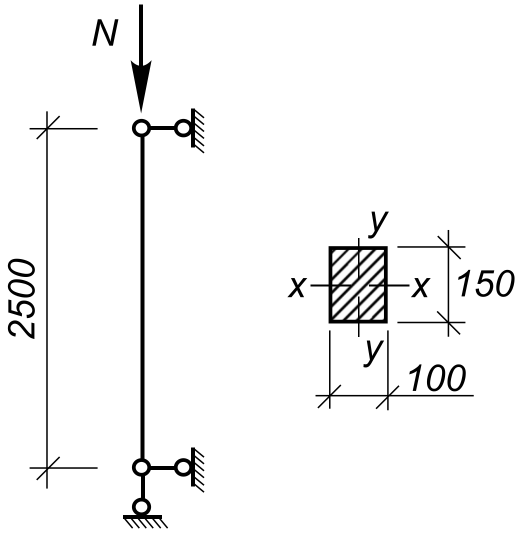
Comparison of solutions:

File	Example 3.SAV
Report file	Décor 3.doc
Check	Strength of the member under a longitudinal tensile force
Theory	0,47/0,56 = 0,839
DECOR	0,85
Deviation, %	1,3 %

Comments:

1. The area of the weakening in the section is determined as the product of the width of the cross-section by the diameter of the hole 15×1,6 = 24 cm^2.
2. Service factor for 1 (A2) class $m_в$ = 1 (table 5 of SNiP II-25-80, table 7 of SP 64.13330.2011).
3. Limit slenderness of the truss chord in tension (not in the vertical plane) is equal to λ_{max} = 200 (table 14 of SNiP II-25-80, table 17 of SP 64.13330.2011).
4. Boundary conditions of the element have to be specified in DECOR. Since they are not determined in the problem, it is assumed that the element is simply supported ($\mu_y = \mu_z = 1$).

Check of the Load-bearing Capacity of an Axially Compressed Column



Objective: Check of the calculation of the resistance of sections.

Task: Verify the correctness of the stability analysis of normal sections.

References: Nasonov S.B. Manual on design and analysis of building structures. – M: ASV Publishing House, 2013. – p. 66-67.

Initial data file: Example 4.SAV; report – Decor 4.doc.

Software version: DECOR 21.1.1.1, 17.02.2016.

Compliance with the codes: SNiP II-25-80, SP 64.13330.2011.

Initial data from the source:

$b \times h$ = 10×15 cm	Section sizes of the element
l = 2,5 m	Column height
$\mu_x = \mu_y = 1$	Effective length factors
N = 60 kN	Compressive force

Material of the element: pine.
Grade of wood: 2.
Operating conditions class: 1 (A2 according to SNiP II-25-80).

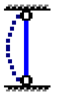
DECOR initial data:

Importance factor $\gamma_n = 1$

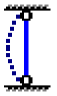
Service factors	
Service factor for temperature and humidity operating conditions m_B	1
Allowance for the temperature conditions of operation m_T	1
Allowance for the duration of loading m_d	1
Service factor under short-term loads m_n	1
Factor that allows for the effect of impregnation with protective substances m_a	1

Wood species - Pine
Grade of wood - 2

Limit slenderness of members in tension - 200
Limit slenderness of members in compression - 120
Member length 2,5 m

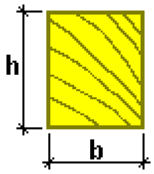


Effective length factor in the XoY plane - 1



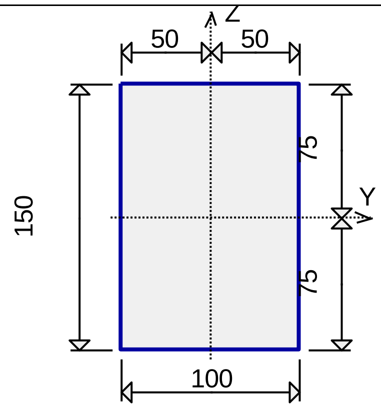
Effective length factor in the XoZ plane – 1

Section:



b = 100 mm
h = 150 mm

Non-glued timber section



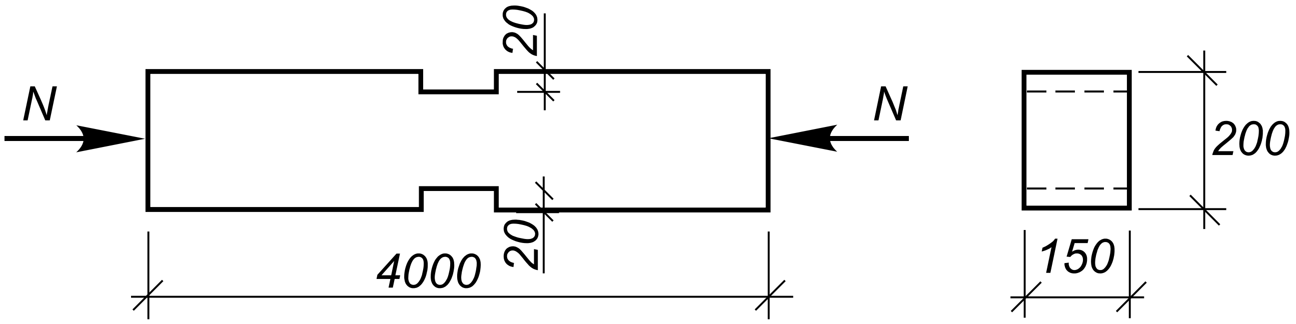
Forces:
N = -60 kN
M_y = 0 kN*m
Q_z = 0 kN
M_z = 0 kN*m
Q_y = 0 kN

Comparison of solutions:

File	Example 4.SAV	
Report file	Décor 4.doc	
Check	Stability in the XOZ plane under a longitudinal force	Stability in the XOY plane under a longitudinal force
Theory	0,55/1,3 = 0,423	1/1,3 = 0,769
DECOR	0,42	0,769
Deviation, %	0,83 %	–

Comments:
1. Service factor for 1 (A2) class $m_{в}$ = 1 (table 5 of SNiP II-25-80, table 7 of SP 64.13330.2011).
2. Limit slenderness of the compressed column is equal to λ_{max} = 120 (table 14 of SNiP II-25-80, table 17 of SP 64.13330.2011).

Check of the Load-bearing Capacity of a Section of an Axially Compressed Weakened Element with a Symmetric Weakening Reaching the Edge



Objective: Check of the calculation of the resistance of sections.

Task: Verify the correctness of the stability analysis of normal sections.

References: Nasonov S.B. Manual on design and analysis of building structures. – M: ASV Publishing House, 2013. – p. 67-68.

Initial data file: Example 5.SAV; report – Decor 5.doc.

Software version: DECOR 21.1.1.1, 17.02.2016.

Compliance with the codes: SNiP II-25-80, SP 64.13330.2011.

Initial data from the source:

$b \times h$ = 15×20 cm	Section sizes of the element
a = 20 mm	Height of the weakened section (Fig. 1)
l = 4 m	Member length
$\mu_x = \mu_y = 1$	Effective length factors
N = 100 kN	Compressive force

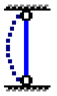
Material of the element: pine.
Grade of wood: 2.
Operating conditions class: 1 (A2 according to SNiP II-25-80).

DECOR initial data:

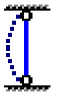
Importance factor $\gamma_n = 1$

Service factors	
Service factor for temperature and humidity operating conditions m_B	1
Allowance for the temperature conditions of operation m_T	1
Allowance for the duration of loading m_d	1
Service factor under short-term loads m_n	1
Factor that allows for the effect of impregnation with protective substances m_a	1

Wood species - Pine
Grade of wood - 2
Limit slenderness of members in tension - 200
Limit slenderness of members in compression - 120
Member length 4 m

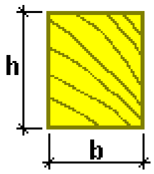


Effective length factor in the XoY plane - 1



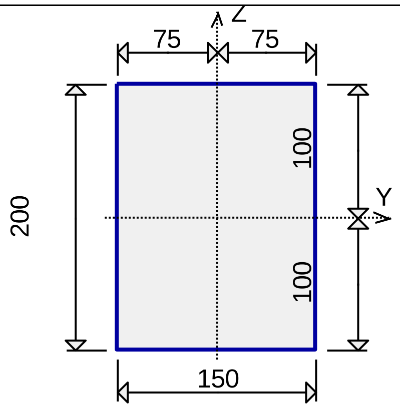
Effective length factor in the XoZ plane - 1

Section:



b = 150 mm
h = 200 mm

Non-glued timber section



Weakening reaching the edge
Area of the weakening - 60 cm^2

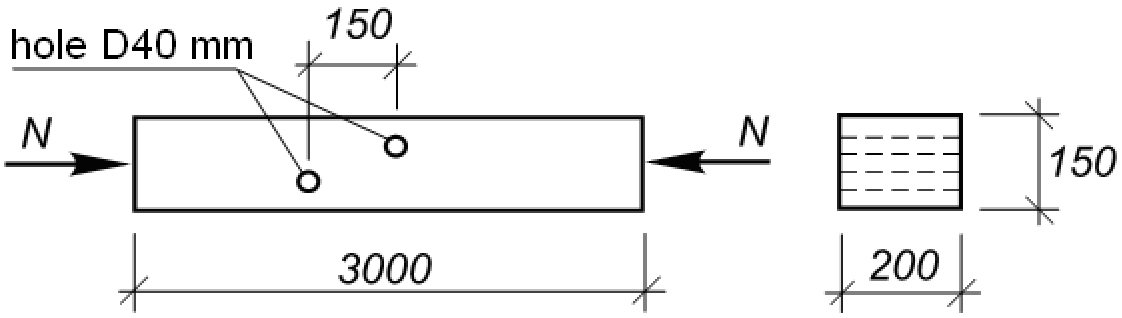
Forces:
N = -100 kN
M_y = 0 kN*m
Q_z = 0 kN
M_z = 0 kN*m
Q_y = 0 kN

Comparison of solutions:

File	Example 5.SAV
Report file	Décor 5.doc
Check	Stability in the XOY plane under a longitudinal force
Theory	1,19/1,5 = 0,793
DECOR	0,79
Deviation, %	0,4 %

Comments:
1. The area of the weakening in the section is determined as $2(a \times b) = 2 \cdot (2 \times 15) = 60$ cm^2.
2. Service factor for 1 (A2) class $m_B = 1$ (table 5 of SNiP II-25-80, table 7 of SP 64.13330.2011).
3. Limit slenderness of the compressed element is equal to $\lambda_{max} = 120$ (table 14 of SNiP II-25-80, table 17 of SP 64.13330.2011).

Check of the Load-bearing Capacity of a Section of an Axially Compressed Element Weakened by Holes in a Section of 150 mm



Objective: Check of the calculation of the resistance of sections.

Task: Verify the correctness of the stability analysis of normal sections.

References: Nasonov S.B. Manual on design and analysis of building structures. – M: ASV Publishing House, 2013. – p. 68-69.

Initial data file: Example 6.SAV; report – Decor 6.doc.

Software version: DECOR 21.1.1.1, 28.03.2016.

Compliance with the codes: SNiP II-25-80, SP 64.13330.2011.

Initial data from the source:

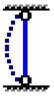
$b \times h$ = 20×15 cm	Section sizes of the element
d = 40 mm	Diameter of the hole
a = 150 mm	Distance between the centers of the holes
l = 3 m	Member length
$\mu_x = \mu_y = 1$	Effective length factors
N = 100 kN	Compressive force

Material of the element: pine.
Grade of wood: 2.
Operating conditions class: 1 (A2 according to SNiP II-25-80).

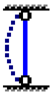
DECOR initial data:
Importance factor $\gamma_n = 1$

Service factors	
Service factor for temperature and humidity operating conditions m_B	1
Allowance for the temperature conditions of operation m_T	1
Allowance for the duration of loading m_d	1
Service factor under short-term loads m_n	1
Factor that allows for the effect of impregnation with protective substances m_a	1

Wood species - Pine
Grade of wood - 2
Limit slenderness of members in tension - 200
Limit slenderness of members in compression - 120
Member length 3 m

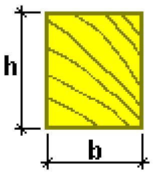


Effective length factor in the XoY plane - 1



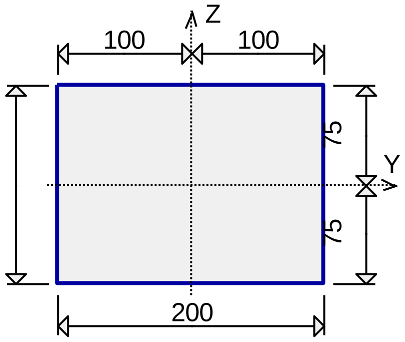
Effective length factor in the XoZ plane – 1

Section:



b = 200 mm
h = 150 mm

Non-glued timber section



Weakening not reaching the edge
Area of the weakening - 80 cm^2

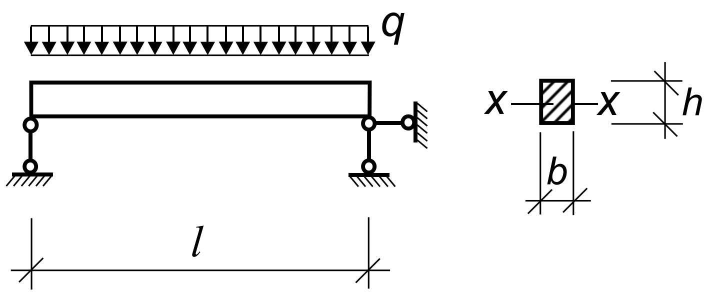
Forces:
N = -100 kN
M_y = 0 kN*m
Q_z = 0 kN
M_z = 0 kN*m
Q_y = 0 kN

Comparison of solutions:

File	Example 6.SAV
Report file	Décor 6.doc
Check	Stability in the XOZ plane under a longitudinal force
Theory	0,55/1,5 = 0,367
DECOR	0,369
Deviation, %	0,62 %

Comments:
1. The area of the weakening in the section is determined as the product of the width of the cross-section by the diameter of the hole 20×4 = 80 cm^2.
2. Service factor for 1 (A2) class $m_\text{в}$ = 1 (table 5 of SNiP II-25-80, table 7 of SP 64.13330.2011).
3. Limit slenderness of the compressed element is equal to λ_{max} = 120 (table 14 of SNiP II-25-80, table 17 of SP 64.13330.2011).

Check of a Section of a Flexural Member



Objective: Check of the beam analysis.

Task: Verify the correctness of the strength analysis and the calculation of the deflection of the element.

References: Nasonov S.B. Manual on design and analysis of building structures. – M: ASV Publishing House, 2013. – p. 80-82.

Initial data file: Example 9.SAV; report – Decor 9.doc.

Compliance with the codes: SNiP II-25-80, SP 64.13330.2011.

Initial data from the source:

$b \times h$ = 10×15 cm	Section sizes of the element
l = 3 m	Member length
$\mu_x = \mu_y = 1$	Effective length factors (Fig. 1)
q_s^{ch} = 3,1 kN/m	Uniformly distributed characteristic serviceability load
q_s^{d} = 3,7 kN/m	Uniformly distributed design serviceability load

Material of the element: pine.
Grade of wood: 2.
Operating conditions class: 1 (A2 according to SNiP II-25-80).
The beam is restrained out of the bending plane along the whole length of the compressed chord.

DECOR initial data:

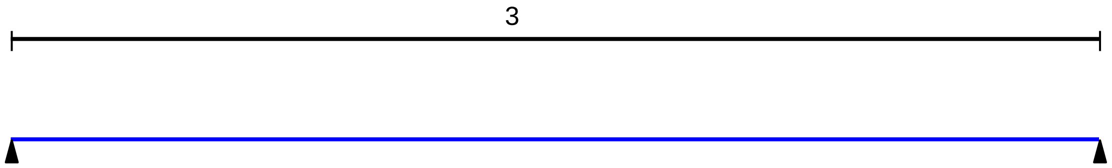
Importance factor γ_n = 1

Importance factor (serviceability limit state) = 1

Service factors	
Service factor for temperature and humidity operating conditions m_B	1
Allowance for the temperature conditions of operation m_T	1
Allowance for the duration of loading m_d	1
Service factor under short-term loads m_n	1
Factor that allows for the effect of impregnation with protective substances m_a	1

Wood species - Pine
Grade of wood - 2
Density of wood 5 kN/m^3

Structure:

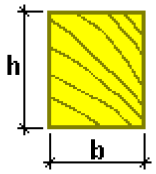


Restraints against lateral displacements and rotations:

	Left	Right
Displacement along Y	Restrained	Restrained
Displacement along Z	Restrained	Restrained
Rotation about Y		
Rotation about Z		

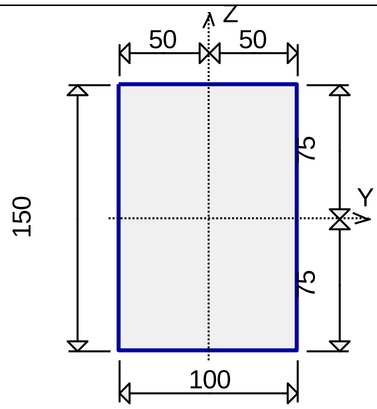
Continuous restraint of the compressed section elements out of the bending plane

Section:



b = 100 mm
h = 150 mm

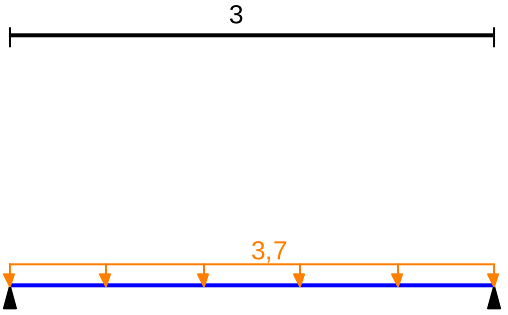
Non-glued timber section

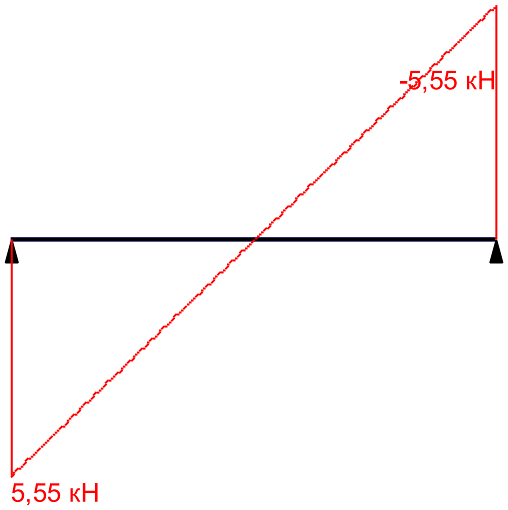


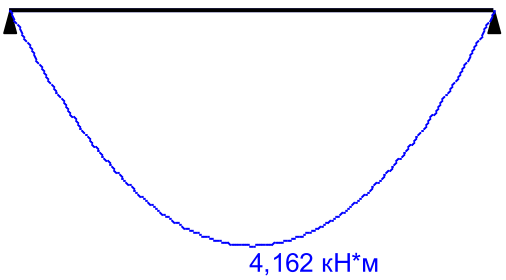
Load case 1 - permanent

	Load type	Value		Dead weight factor
	length = 3 m			
		3,7	kN/m	

Load case 1 - permanent
Safety factor for load: 1,19355



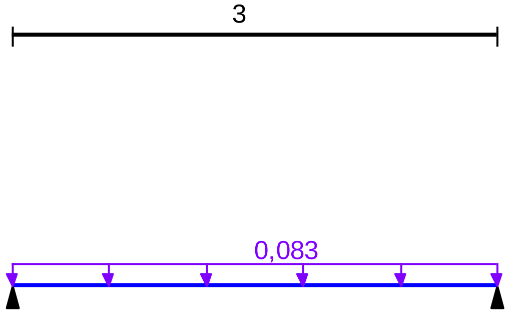


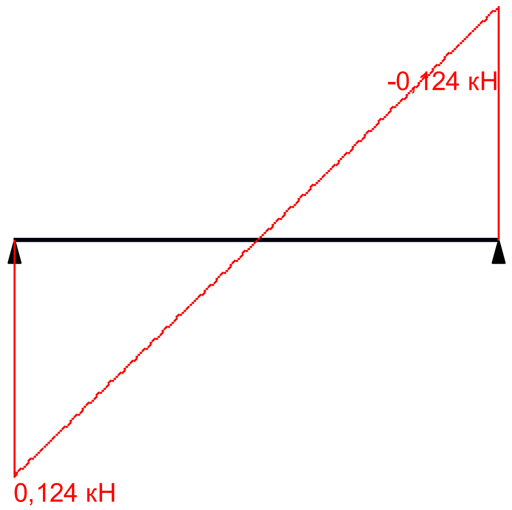


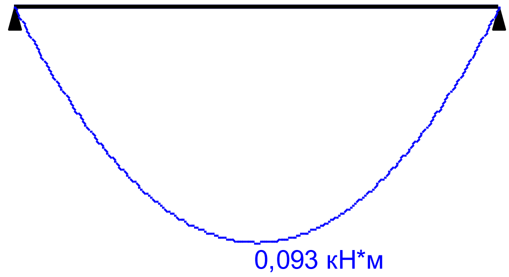
Load case 2 - permanent

	Load type	Value		Dead weight factor
		0,075	kN/m	1,1

Load case 2 - permanent
Safety factor for load: 1,1







	Support reactions	
	Force in support 1	Force in support 2
	kN	kN
by criterion M_{max}	5,674	5,674
by criterion M_{min}	5,674	5,674
by criterion Q_{max}	5,674	5,674
by criterion Q_{min}	5,674	5,674

Comparison of solutions:

File	Example 9SAV		
Report file	Décor 9.doc		
Check	Strength of the member under the bending moment	Strength under the lateral force	Deflection
Theory	1,14/1,3 = 0,877	0,057/0,16 = 0,356	1,19/1,2 = 0,992
DECOR	0,873	0,355	1,04
Deviation, %	0,463%	0,46%	4,62%

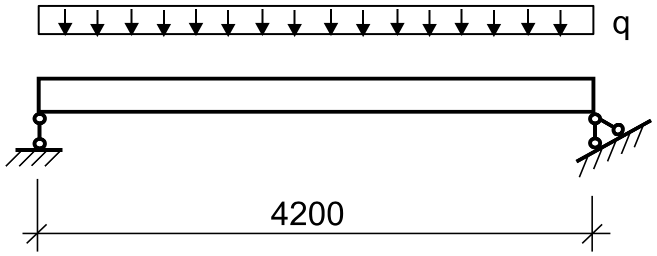
Comments:

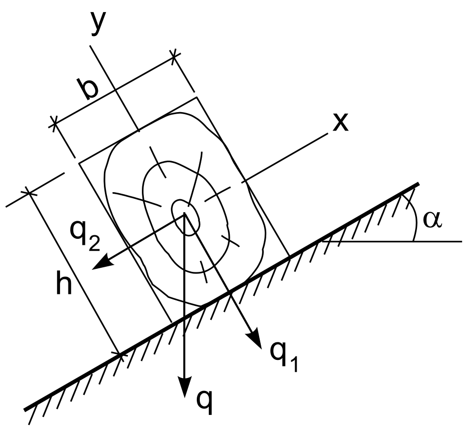
1. Maximum allowable deflection for interstorey floor beams is determined as (1/250)l = 0,004l (table 16 of SNiP II-25-80, table 19 of SP 64.13330.2011).
2. The loads in DECOR are specified in the following way:

- Load case 1 – external load (design value) 3,7 kN/m with a safety factor for load equal to q_s^d / q_s^{ch} = 3,7/3,1 = 1,19355;
- Load case 2 – load from the self-weight with a dead weight factor of 1,1 and a safety factor for load of 1,1.

3. The difference in the deflection of 4,62% was obtained due to the fact that the effect of shear is not taken into account in the calculation in the source.

Analysis of a Purlin for Biaxial Bending





Objective: Check the calculation of continuous purlins.

Task: Verify the correctness of the strength analysis and the calculation of the deflection of the element.

References: Nasonov S.B. Manual on design and analysis of building structures. – M: ASV Publishing House, 2013. – p. 86-88.

Initial data file: Example 11.SAV; report – Decor 11.doc.

Software version: DECOR 21.1.1.1, 27.05.2016.

Compliance with the codes: SNiP II-25-80, SP 64.13330.2011.

Initial data from the source:

$b \times h$ = 15×20 cm	Section sizes of the element
l = 4,2 m	Purlin span
$\mu_x = \mu_y = 1$	Effective length factors
q_s^{ch} = 3,0 kN/m	Uniformly distributed serviceability load (characteristic value)
q_s^{d} = 3,5 kN/m	Uniformly distributed serviceability load (design value)
α = 30°	Purlin inclination
Material of the element:	pine
Grade of wood:	2

Operating conditions class: 1 (A2 according to SNiP II-25-80).

DECOR initial data:

Importance factor γ_n = 1

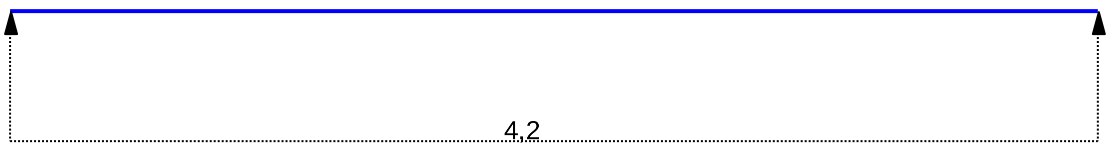
Importance factor (serviceability limit state) = 1

Service factors	
Service factor for temperature and humidity operating conditions m_B	1
Allowance for the temperature conditions of operation m_T	1
Allowance for the duration of loading m_d	1

Service factor under short-term loads m_n	1
Factor that allows for the effect of impregnation with protective substances m_a	1

Wood species - Pine
Grade of wood - 2
Density of wood 5 kN/m^3

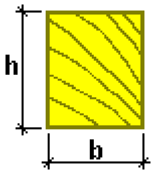
Structure



Spacing of bracing in the roof plane 0,6 m

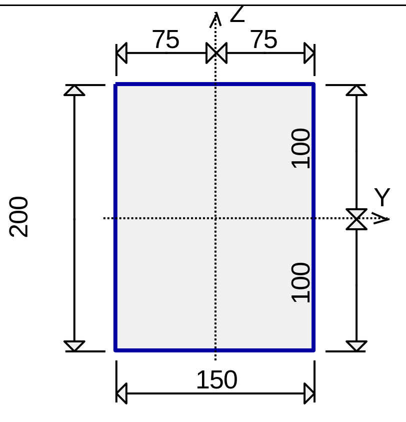
Roof inclination 30 degrees

Section



b = 150 mm
h = 200 mm

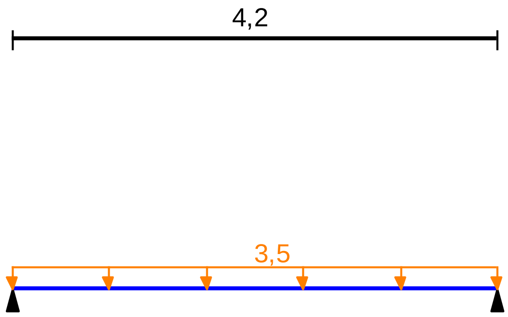
Non-glued timber section

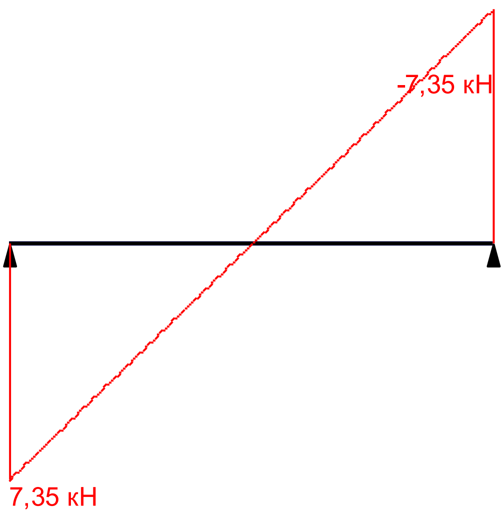


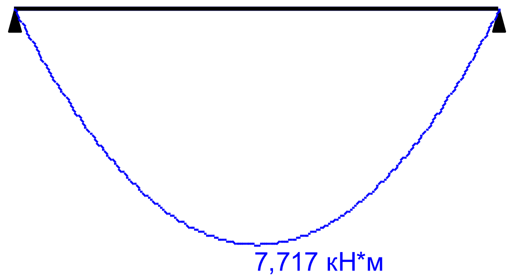
Load case 1 - permanent

	Load type	Value		Dead weight factor
	span 1, length = 4,2 m			
		3,5	kN/m	

Load case 1 - permanent
Safety factor for load: 1,16667



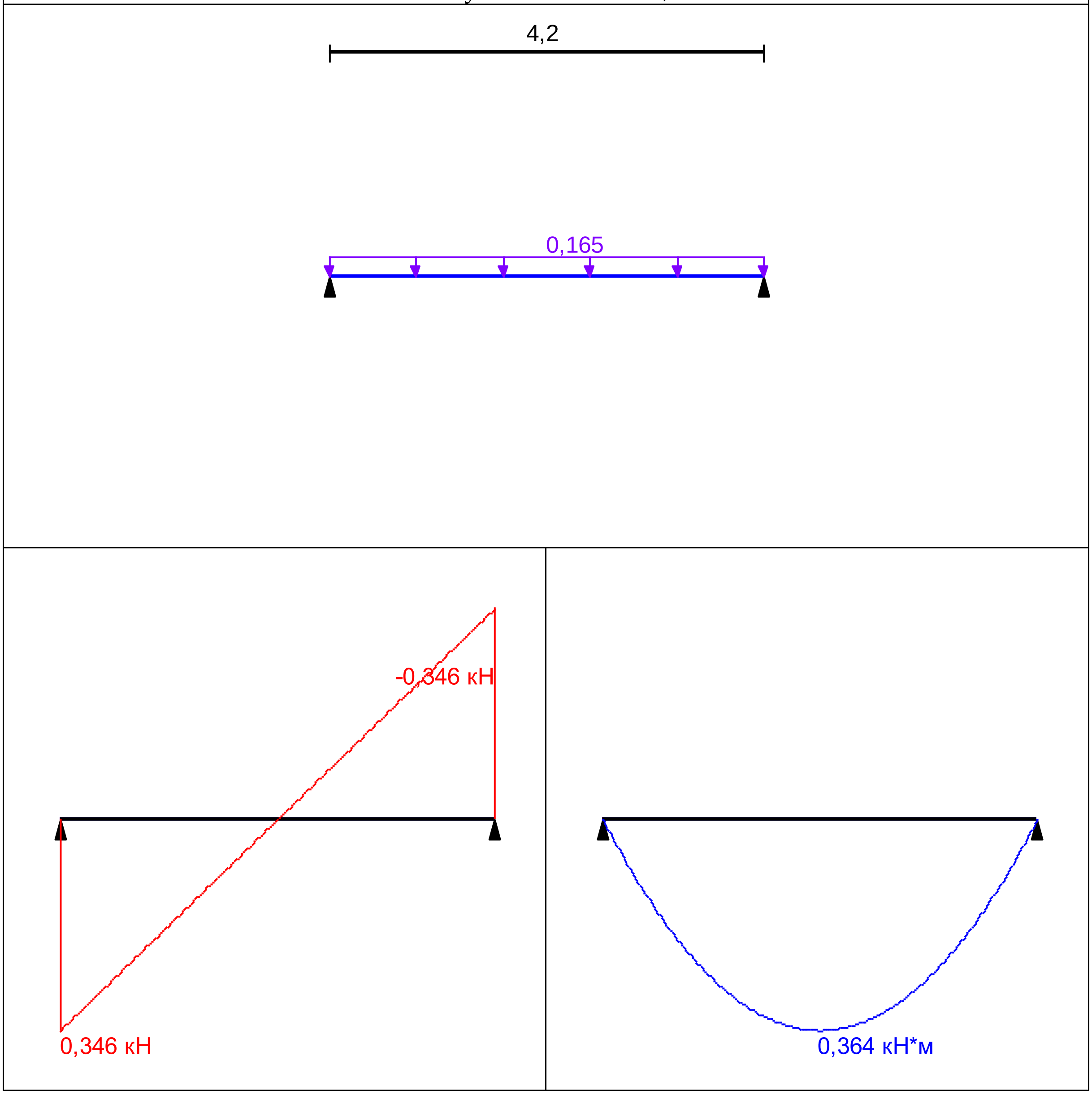




Load case 2 - permanent

	Load type	Value		Dead weight factor
		0,15	kN/m	1,1

Load case 2 - permanent
Safety factor for load: 1,1



	Support reactions	
	Force in support 1	Force in support 2
	kN	kN
by criterion M_{max}	7,697	7,697
by criterion M_{min}	7,697	7,697
by criterion Q_{max}	7,697	7,697
by criterion Q_{min}	7,697	7,697

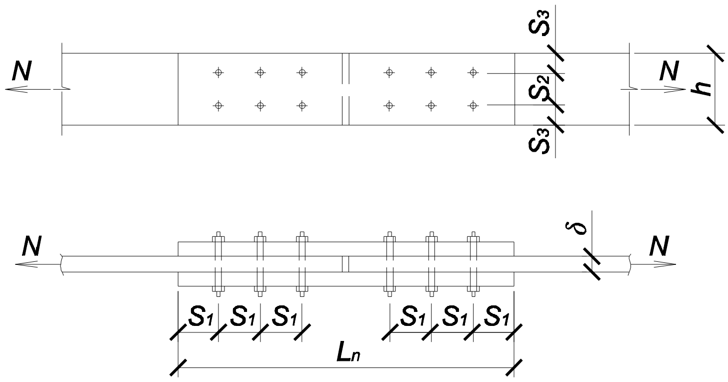
Comparison of solutions:

File	Example 11.SAV	
Report file	Decor 11.doc	
Check	Strength of the member under the bending moment M_y	Deflection
Theory	1,24/1,5 = 0,827	1,59/2,1 = 0,757
DECOR	0,826	0,775
Deviation, %	0,115%	2,36%

Comments:

1. Maximum allowable deflection for purlins is determined as $(1/200)l = 0{,}005l$ (table 16 of SNiP II-25-80, table 19 of SP 64.13330.2011).
2. Spacing of bracing in the roof plane has to be specified in DECOR. Since it is not determined in the problem, the value of 0,6 m is used.
3. The density of pine at the operating conditions class 1 (A2) is equal to ρ = 500 kg/m^3 = 5 kN/m^3 (Annex 3 of SNiP II-25-80, Annex E of SP 64.13330.2011).
4. The loads in DECOR are specified in the following way:
   - Load case 1 – external load (design value) 3,5 kN/m with a safety factor for load equal to q_s^d / q_s^{ch} = 3,5/3,0 = 1,16667;
   - Load case 2 – load from the self-weight with a dead weight factor of 1,1 and a safety factor for load of 1,1.
5. The difference in the deflection of 2,36% was obtained due to the fact that shear was not taken into account in the calculation in the theoretical source.

Check the Load-bearing Capacity of a Connection with Dowels



Objective: Check the calculation of the resistance of connections.

Task: Verify the correctness of the load-bearing capacity analysis of the bottom truss chord joint for bearing of the side and middle members and bending of the dowel.

References: Nasonov S.B. Manual on design and analysis of building structures. – M: ASV Publishing House, 2013. – p. 114-116.

Initial data file: Example 18.SAV; report – Decor 18.doc.

Compliance with the codes: SNiP II-25-80, SP 64.13330.2011.

Initial data from the source:

$\delta \times h$ = 5×15 cm	Section sizes of the chord boards and gusset plates
d = 12 mm	Diameter of bolts
N = 30 kN	Tensile force in the chord
Material of the element:	pine
Grade of wood:	2
Operating conditions class:	1 (A2 according to SNiP II-25-80).

DECOR initial data:

Importance factor γ_n = 1

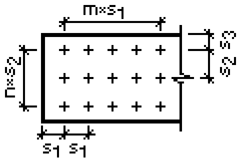
Service factors	
Service factor for temperature and humidity operating conditions m_B	1
Allowance for the temperature conditions of operation m_T	1
Allowance for the duration of loading m_d	1
Service factor under short-term loads m_n	1
Factor that allows for the effect of impregnation with protective substances m_a	1

Wood species - Pine
Grade of wood - 2
Non-glued timber section

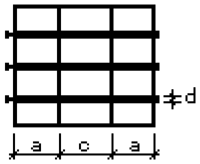
Connection with cylindrical dowels

Dowel type - Steel

Arrangement of dowels – in-line



Symmetric joint



Number of design-glue-lines for one dowel - 2
Diameter of dowel 12 mm
m = 2
n = 1
s_1 = 84 mm
s_2 = 42 mm
s_3 = 36 mm
a = 50 mm
c = 50 mm

Forces
N = 30 kN

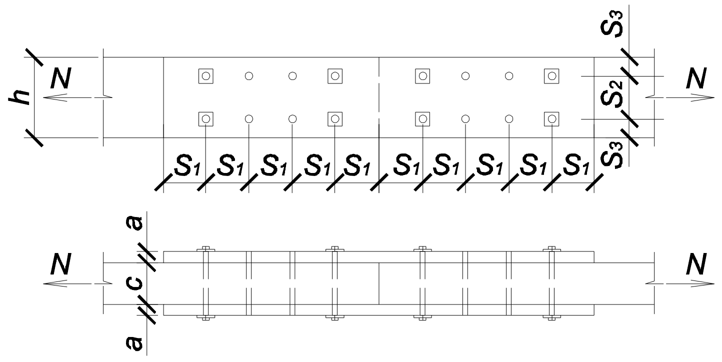
Comparison of solutions:

Check	Load-bearing capacity for bearing of the side member	Load-bearing capacity for bearing of the middle member	Bending of the steel dowel
Theory	30/57,6 = 0,521	30/36 = 0,833	30/37,1 = 0,809
DECOR	0,521	0,833	0,809
Deviation, %	0%	0%	0%

Comment:

The necessary number of dowels in the given connection was determined in the theoretical solution of the problem in the source, and the joint was designed. In the result it was decided to put 6 dowels on each side of the joint and arrange them in two rows, and the distances between the dowel axes were determined: S_1 = 8,4 cm, S_2 = 4,2 cm and S_3 = 3,6 cm. These parameters are used in DECOR for checking the connection.

Check the Load-bearing Capacity of the Bottom (Tensile) Truss Chord



Objective: Check the calculation of the resistance of connections.

Task: Verify the correctness of the load-bearing capacity analysis of the bottom truss chord joint for bearing of the side and middle members and bending of the dowel.

References: Nasonov S.B. Manual on design and analysis of building structures. – M: ASV Publishing House, 2013. – p. 116-118.

Initial data file: Example 19.SAV; report – Decor 19.doc.

Compliance with the codes: SNiP II-25-80, SP 64.13330.2011.

Initial data from the source:

$c \times h$ = 10×20 cm	Section sizes of the timber chord
$a \times h$ = 5×20 cm	Section sizes of steel gusset plates
d = 16 mm	Diameter of bolts
N = 80 kN	Tensile force in the chord
Material of the element:	pine
Grade of wood:	2
Steel grade of the gusset plates:	C235
Operating conditions class:	1 (A2 according to SNiP II-25-80).

DECOR initial data:

Importance factor γ_n = 1

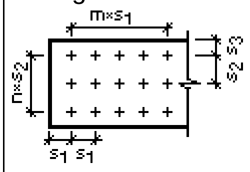
Service factors	
Service factor for temperature and humidity operating conditions m_B	1
Allowance for the temperature conditions of operation m_T	1
Allowance for the duration of loading m_d	1
Service factor under short-term loads m_n	1
Factor that allows for the effect of impregnation with protective substances m_a	1

Wood species - Pine
Grade of wood - 2
Non-glued timber section

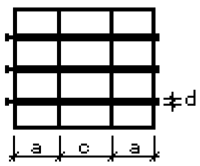
Connection with cylindrical dowels

Dowel type - Steel

Arrangement of dowels – in-line



Symmetric joint



Number of design-glue-lines for one dowel - 2
Diameter of dowel 16 mm
m = 3
n = 1
s_1 = 112 mm
s_2 = 56 mm
s_3 = 48 mm
a = 50 mm
c = 100 mm

Forces
N = 80 kN

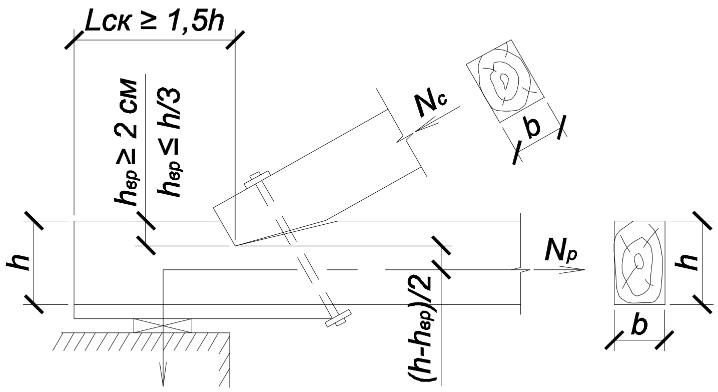
Comparison of solutions:

Check	Load-bearing capacity for bearing of the side member	Load-bearing capacity for bearing of the middle member	Bending of the steel dowel
Theory	80/102,4 = 0,781	80/128 = 0,625	80/81,73 = 0,979
DECOR	0,781	0,625	0,979
Deviation, %	0%	0%	0%

Comment:

The necessary number of dowels in the given connection was determined in the theoretical solution of the problem in the source, and the joint was designed. In the result it was decided to put 6 dowels on each side of the joint and arrange them in two rows, and the distances between the dowel axes were determined: S_1 = 11,2 cm, S_2 = 5,6 cm and S_3 = 4,8 cm. These parameters are used in DECOR for checking the connection.

Check of the Load-bearing Capacity of a Truss Support Joint



Objective: Check the calculation of the resistance of connections.

Task: Verify the correctness of the strength analysis of the truss support joint for bearing and shearing.

References: Nasonov S.B. Manual on design and analysis of building structures. – M: ASV Publishing House, 2013. – p. 107-108.

Initial data file: Example 16.SAV; report – Decor 16.doc.

Software version: DECOR 21.1.1.1, 27.05.2016.

Compliance with the codes: SNiP II-25-80, SP 64.13330.2011.

Initial data from the source:

$b \times h$ = 15×20 cm	Section sizes
h_{bp} = 5,5 cm	Depth of the notch
$L_{ck} = 10h_{bp}$ = 55 cm	Length of the shearing area
α = 21°48′	Angle between the chords
N = 89 kN	Compressive force in the top chord
Material of the element:	pine
Grade of wood:	2
Operating conditions class:	1 (A2 according to SNiP II-25-80).

DECOR initial data:

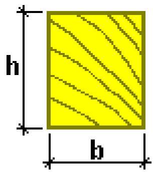
Importance factor $\gamma_n = 1$

Service factors	
Service factor for temperature and humidity operating conditions m_B	1
Allowance for the temperature conditions of operation m_T	1
Allowance for the duration of loading m_d	1
Service factor under short-term loads m_n	1
Factor that allows for the effect of impregnation with protective substances m_a	1

Wood species - Pine
Grade of wood - 2

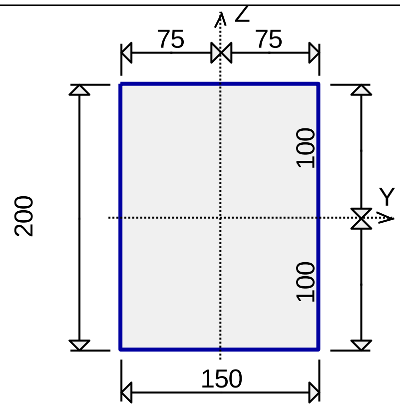
Notched connection

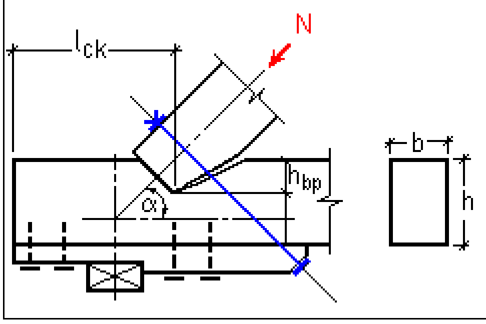
Section



b = 150 mm
h = 200 mm

Non-glued timber section





Asymmetric joint
l_{ck} = 550 mm
h_{bp} = 55 mm
α = 21,8 degrees

Forces
N = 89 kN

Comparison of solutions:

Check	Strength based on the bearing conditions	Strength based on the shearing conditions
Theory	89/110,11 = 0,808	82,6/83,32 = 0,991
DECOR	0,805	0,805
Deviation, %	0,46 %	18,8%

Comment:

The difference between the strength factor based on the shearing condition and the result of the theoretical solution of 18,8% was obtained due to the different determination of the average design shearing resistance of timber over the shearing area R_{ck}^{cp}: in DECOR this factor is determined by the formula (59) according to Sec. 7.3 of SP 64.13330.2011 (formula (54) according to Sec. 5.3 of SNiP II-25-80); and in the theoretical source R_{ck}^{cp} is determined from the table given in it, the origin of which is not explained by the author.

MAGNUM

Calculation of the Gross Cross-Sectional Properties of a Load-Bearing Member from a C-shaped Cold-Formed Profile (Rotation of the Principal Axes of Inertia)

Task: Verify the correctness of the calculation of the gross cross-sectional properties for load-bearing members from cold-formed profiles.

Source: [1] Worked examples according to EN 1993-1-3, Eurocode 3, Part 1-3 // ECCS TC7 TWG 7.5 Practical Improvement of Design Procedure. – 1st Ed., ECCS CECM, EKS, 2008. – 235 p.

Compliance with the codes: EN 1993-1-3.

Initial data file:
Task 1.1.sav; report – Report 1.1.doc

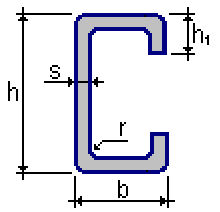
Program version: MAGNUM 23.1.1.3, 07.02.2024

Initial data:

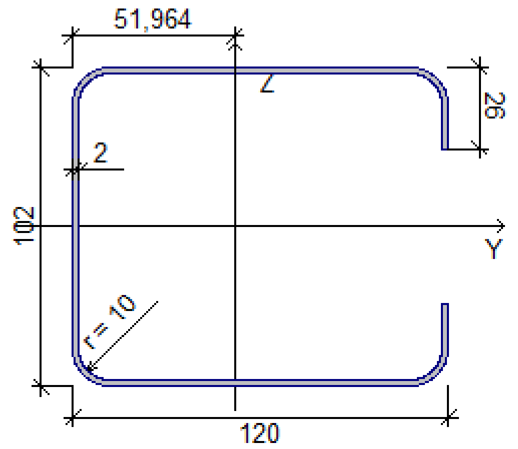
h = 102 mm	Section height (along the outer edge)
b = 120 mm	Flange width (along the outer edge)
c = 26 mm	Flange bend length (along the outer edge)
t = 2 mm	Profile thickness (minus the coating thickness)
r = 10 mm	Fillet radius (inner)

Results in MAGNUM:

Section



h = 102 mm
b = 120 mm
s = 2 mm
h1 = 26 mm
r = 10 mm



Comparison of solutions

Geometric property	[1], p. 147	MAGNUM, EN1993-1-1	%
Gross cross-sectional area, cm^2	7,34	7,342	0,027
Moment of inertia about the y-y axis, cm^4	139,10	139,185	0,06
Section modulus about the y-y axis, cm^3	140,45	140,562	0,08

Comments

The calculation of the gross cross-sectional properties in MAGNUM is performed accurately, taking into account the fillets, due to which the calculation results correspond to the exact analytical solution.

Calculation of the Gross Cross-Sectional Properties of a Load-Bearing Member from a C-shaped Cold-Formed Profile

Task: Verify the correctness of the calculation of the gross cross-sectional properties for load-bearing members from cold-formed profiles.

Source: [2] Leroy Gardner and David A. Nethercot. Designer’s guide to EuroCode 3: Design of steel buildings EN 1993-1-1, -1-3 and -1-8. Second edition. ISBN 978-0-7277-4172-1. doi: 10.1680/dsb.41721.001

Compliance with the codes: EN 1993-1-3.

Initial data file:
Task 1.2.sav; report – Report 1.2.doc

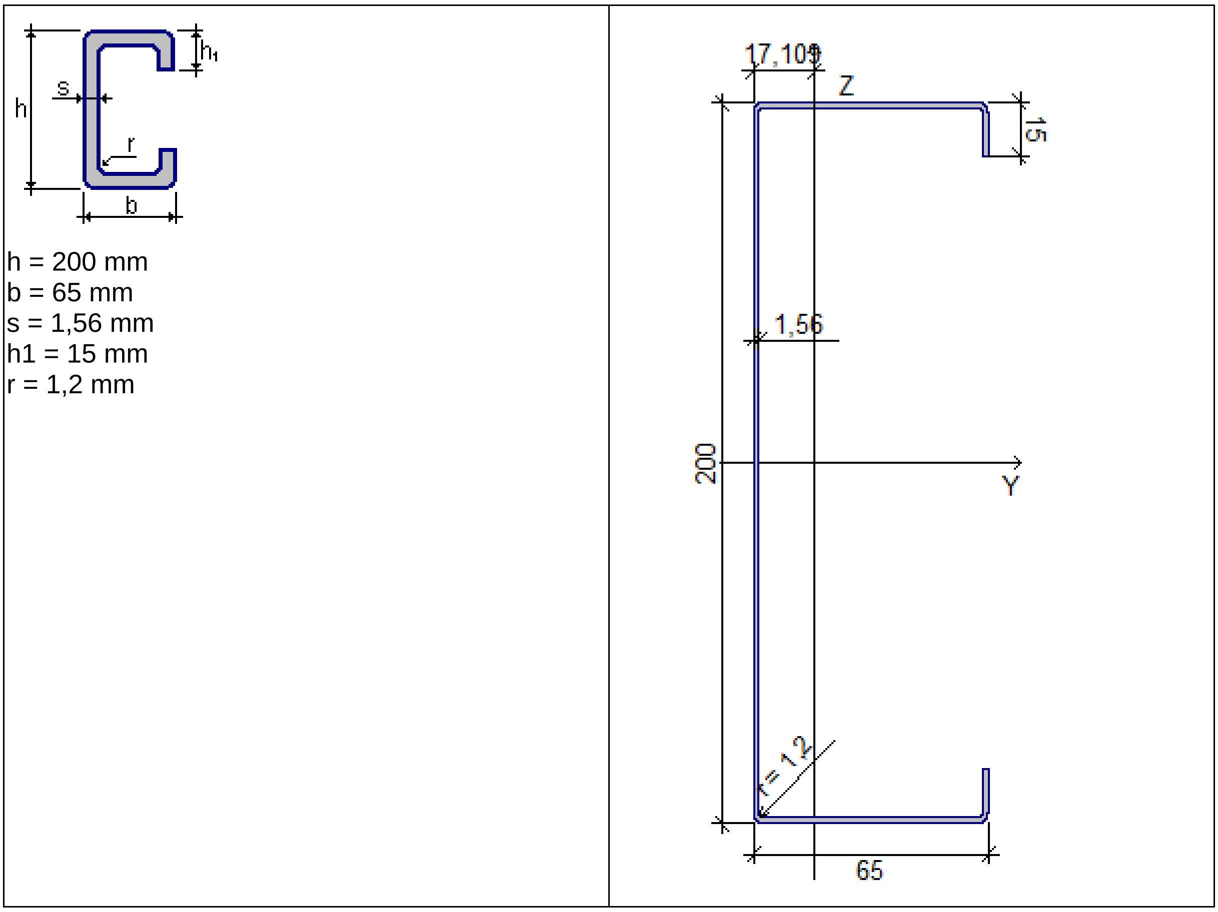
Program version: MAGNUM 23.1.1.3, 07.02.2024

Initial data:

h = 200 mm	Section height (along the outer edge)
b = 65 mm	Flange width (along the outer edge)
c = 15 mm	Flange bend length (along the outer edge)
t = 1.56 mm	Profile thickness (minus the coating thickness)
r = 1.2 mm	Fillet radius (inner)

Results in MAGNUM:

Section



Comparison of solutions

Geometric property	[2], p. 139	MAGNUM, EN1993-1-1	%
Gross cross-sectional area, mm^2, $мм^2$	551.6	546.6	0,9

Comments

The calculation of the gross cross-sectional properties in MAGNUM is performed accurately, taking into account the fillets, due to which the calculation results correspond to the exact analytical solution.

Calculation of the Gross Cross-Sectional Properties of a Load-Bearing Member from a Cold-Formed Hat Channel

Task: Verify the correctness of the calculation of the gross cross-sectional properties for load-bearing members from cold-formed profiles.

Source: [1] Worked examples according to EN 1993-1-3, Eurocode 3, Part 1-3 // ECCS TC7 TWG 7.5 Practical Improvement of Design Procedure. – 1st Ed., ECCS CECM, EKS, 2008. – 235 p.

Compliance with the codes: EN 1993-1-3.

Initial data file:
Task 1.3.sav; report – Report 1.3.doc

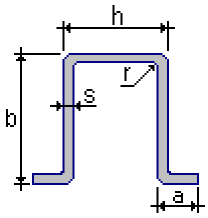
Program version: MAGNUM 23.1.1.3, 07.02.2024

Initial data:

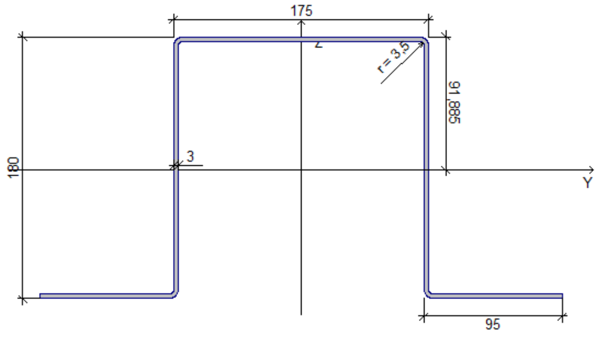
h = 180 mm	Section height (along the outer edge)
b = 175 mm	Flange width (along the outer edge)
c = 95 mm	Flange bend length (along the outer edge)
t = 3 mm	Profile thickness (minus the coating thickness)
r = 3.5 mm	Fillet radius (inner)

Results in MAGNUM:

Section



h = 175 mm
b = 180 mm
s = 3 mm
a = 95 mm
r = 3,5 mm



Comparison of solutions

Geometric property	[1], p. 170	MAGNUM, EN1993-1-1	%
Gross cross-sectional area, mm^2	2113,2	2113,2	0
Moment of inertia about the y-y axis, mm^4	$10{,}994\times10^6$	10997360	0,03
Moment of inertia about the z-z axis, mm^4	$19{,}231\times10^6$	19231920	0.005

Comments

The calculation of the gross cross-sectional properties in MAGNUM is performed accurately, taking into account the fillets, due to which the calculation results correspond to the exact analytical solution.

Calculation of the Effective Cross-Sectional Properties of a Load-Bearing Member from a C-shaped Cold-Formed Profile under Axial Compression

Task: Verify the correctness of the calculation of the effective cross-sectional properties for load-bearing members from cold-formed profiles under axial compression.

Source: [1] Worked examples according to EN 1993-1-3, Eurocode 3, Part 1-3 // ECCS TC7 TWG 7.5 Practical Improvement of Design Procedure. – 1st Ed., ECCS CECM, EKS, 2008. – 235 p.

Compliance with the codes: EN 1993-1-3.

Initial data file:
Task 2.1.sav; report – Report 2.1.doc

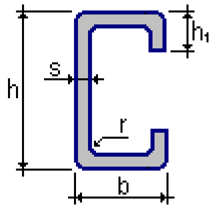
Program version: MAGNUM 23.1.1.3, 07.02.2024

Initial data:

f_y = 355 N/mm^2	Yield strength
h = 102 mm	Section height (along the outer edge)
b = 120 mm	Flange width (along the outer edge)
c = 26 mm	Flange bend length (along the outer edge)
t = 2 mm	Profile thickness (minus the coating thickness)
r = 10 mm	Fillet radius (inner)

Results in MAGNUM:
Steel: S355

Section



h = 102 mm
b = 120 mm
s = 2 mm
h1 = 26 mm
r = 10 mm

Comparison of solutions

Geometric property	[1], p. 154	MAGNUM, EN1993-1-1	%
Gross cross-sectional area, cm^2	4,67	4,609	1.3
Moment of inertia about the y-y axis, cm^4	87,24	86,266	1.12
Moment of inertia about the z-z axis, cm^4	94,80	94,18	0.65

Calculation of the Effective Cross-Sectional Properties of a Load-Bearing Member from a Cold-Formed Hat Channel under Axial Compression

Task: Verify the correctness of the calculation of the effective cross-sectional properties for load-bearing members from cold-formed profiles under axial compression.

Source: [1] Worked examples according to EN 1993-1-3, Eurocode 3, Part 1-3 // ECCS TC7 TWG 7.5 Practical Improvement of Design Procedure. – 1st Ed., ECCS CECM, EKS, 2008. – 235 p.

Compliance with the codes: EN 1993-1-3.

Initial data file:
Task 2.2.sav; report – Report 2.2.doc

Program version: MAGNUM 23.1.1.3, 07.02.2024

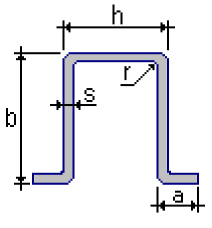
Initial data:

f_y = 355 N/mm^2	Yield strength
h = 180 mm	Section height (along the outer edge)
b = 175 mm	Flange width (along the outer edge)
c = 95 mm	Flange bend length (along the outer edge)
t = 3 mm	Profile thickness (minus the coating thickness)
r = 3.5 mm	Fillet radius (inner)

Results in MAGNUM:

Steel: S355

Section



h = 175 mm
b = 180 mm
s = 3 mm
a = 95 mm
r = 3,5 mm

Comparison of solutions

Geometric property	[1], p. 175	MAGNUM, EN1993-1-1	%
Gross cross-sectional area, mm^2	1188.26	1229,7	3.37

Calculation of the Effective Cross-Sectional Properties of a Load-Bearing Member from a C-shaped Cold-Formed Profile under Bending

Task: Verify the correctness of the calculation of the effective cross-sectional properties for load-bearing members from cold-formed profiles under bending.

Source: [1] Worked examples according to EN 1993-1-3, Eurocode 3, Part 1-3 // ECCS TC7 TWG 7.5 Practical Improvement of Design Procedure. – 1st Ed., ECCS CECM, EKS, 2008. – 235 p.

Compliance with the codes: EN 1993-1-3.

Initial data file:
Task 3.1.sav; report – Report 3.1.doc

Program version: MAGNUM 23.1.1.3, 07.02.2024

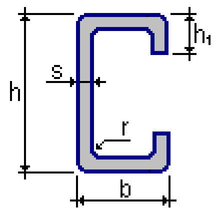
Initial data:

f_y = 355 N/mm^2	Yield strength
h = 102 mm	Section height (along the outer edge)
b = 120 mm	Flange width (along the outer edge)
c = 26 mm	Flange bend length (along the outer edge)
t = 2 mm	Profile thickness (minus the coating thickness)
r = 10 mm	Fillet radius (inner)

Results in MAGNUM:

Steel: S355

Section



h = 102 mm
b = 120 mm
s = 2 mm
h1 = 26 mm
r = 10 mm

Comparison of solutions

	[1], p. 160	MAGNUM, EN1993-1-1	%
Gross cross-sectional area, cm^2	6,86	6,602	3,76
Moment of inertia about the y-y axis, cm^4	129,73	124,976	3,66
Section modulus about the y-y axis, cm^3	122,49	113,842	7,06

Comments

In [1] the calculation of the effective cross-sectional properties is performed taking into account the fillets. In MAGNUM they are calculated without taking the fillets into account, which is permitted by the codes for the considered fillet radius (see Sec. 5.1(3) EN1993-1-3).

Calculation of the Effective Cross-Sectional Properties of a Load-Bearing Member from a Cold-Formed Hat Channel under Bending (the Flange is in Compression)

Task: Verify the correctness of the calculation of the effective cross-sectional properties for load-bearing members from cold-formed profiles under bending (the flange is in compression).

Source: [1] Worked examples according to EN 1993-1-3, Eurocode 3, Part 1-3 // ECCS TC7 TWG 7.5 Practical Improvement of Design Procedure. – 1st Ed., ECCS CECM, EKS, 2008. – 235 p.

Compliance with the codes: EN 1993-1-3.

Initial data file:
Task 3.2.sav; report – Report 3.2.doc

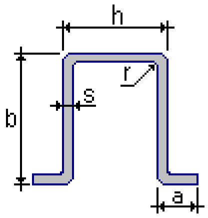
Program version: MAGNUM 23.1.1.3, 07.02.2024

Initial data:

f_y = 355 N/mm^2	Yield strength
h = 180 mm	Section height (along the outer edge)
b = 175 mm	Flange width (along the outer edge)
c = 95 mm	Flange bend length (along the outer edge)
t = 3 mm	Profile thickness (minus the coating thickness)
r = 3.5 mm	Fillet radius (inner)

Results in MAGNUM:
Steel: S355

Section



h = 175 mm
b = 180 mm
s = 3 mm
a = 95 mm
r = 3,5 mm

Comparison of solutions

Geometric property	[1], p. 177	MAGNUM, EN1993-1-1	%
Cross-sectional area, cm^2	–	19,375	–
Moment of inertia about the y-y axis, mm^4	9,73×10^6	9,45217×10^6	2,86
Section modulus about the y-y axis, mm^3 (maximum)	123,11×10^3	117,644×103	4,44
Section modulus about the y-y axis, mm^3 (minimum)	99,28×10^3	94,85×103	4,46

Comments

In [1] the calculation of the effective cross-sectional properties is performed taking into account the fillets. In MAGNUM they are calculated without taking the fillets into account, which is permitted by the codes for the considered fillet radius (see Sec. 5.1(3) EN1993-1-3).

Calculation of the Effective Cross-Sectional Properties of a Load-Bearing Member from a Cold-Formed Hat Channel under Bending (the Flange is in Tension)

Task: Verify the correctness of the calculation of the effective cross-sectional properties for load-bearing members from cold-formed profiles under bending (the flange is in tension).

Source: [1] Worked examples according to EN 1993-1-3, Eurocode 3, Part 1-3 // ECCS TC7 TWG 7.5 Practical Improvement of Design Procedure. – 1st Ed., ECCS CECM, EKS, 2008. – 235 p.

Compliance with the codes: EN 1993-1-3.

Initial data file:
Task 3.3.sav; report – Report 3.3.doc

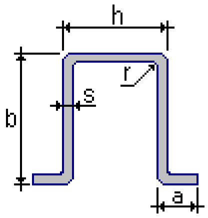
Program version: MAGNUM 23.1.1.3, 07.02.2024

Initial data:

f_y = 355 N/mm^2	Yield strength
h = 180 mm	Section height (along the outer edge)
b = 175 mm	Flange width (along the outer edge)
c = 95 mm	Flange bend length (along the outer edge)
t = 3 mm	Profile thickness (minus the coating thickness)
r = 3.5 mm	Fillet radius (inner)

Results in MAGNUM:
Steel: S355

Section



h = 175 mm
b = 180 mm
s = 3 mm
a = 95 mm
r = 3,5 mm

Comparison of solutions

Geometric property	[1], p. 180	MAGNUM, EN1993-1-1	%
Cross-sectional area, mm^2	1762,29	1773,5	0,63
Moment of inertia about the y-y axis, mm^4	$7{,}98\times10^6$	$8{,}15496\times10^6$	2,15
Section modulus about the y-y axis, mm^3 (maximum)	$110{,}09\times10^3$	$106{,}908\times10^3$	2,89
Section modulus about the y-y axis, mm^3 (minimum)	$76{,}37\times10^3$	$78{,}901\times10^3$	3,21

Comments

In [1] the calculation of the effective cross-sectional properties is performed taking into account the fillets. In MAGNUM they are calculated without taking the fillets into account, which is permitted by the codes for the considered fillet radius (see Sec. 5.1(3) EN1993-1-3).

Calculation of the Load-Bearing Capacity of a Bar Structural Member from a C-shaped Cold-Formed Profile under Axial Compression

Task: Verify the correctness of the calculation of the load-bearing capacity of bar structural members from cold-formed profiles under axial compression.

Source: [1] Worked examples according to EN 1993-1-3, Eurocode 3, Part 1-3 // ECCS TC7 TWG 7.5 Practical Improvement of Design Procedure. – 1st Ed., ECCS CECM, EKS, 2008. – 235 p.

Compliance with the codes: EN 1993-1-3.

Initial data file:
Task 4.1.sav; report – Report 4.1.doc

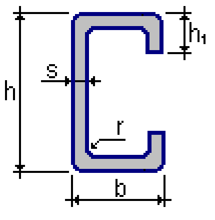
Program version: MAGNUM 23.1.1.3, 07.02.2024

Initial data:

E = 210000 N/mm^2	Elastic modulus
v = 0.3	Poisson's ratio
f_y = 355 N/mm^2	Yield strength
γ_{M0} = 1	Partial safety factor
γ_{M1} = 1	Partial safety factor
h = 102 mm	Section height (along the outer edge)
b = 120 mm	Flange width (along the outer edge)
c = 26 mm	Flange bend length (along the outer edge)
t = 2 mm	Profile thickness (minus the coating thickness)
r = 10 mm	Fillet radius (inner)
N = 85.7 kN	Design axial force
ℓ = 150 cm	Effective length of the bar member

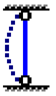
Results in MAGNUM:
Steel: S355
Importance factor 1
Effective length factor for torsional buckling:
coefficient to the geometric length = 1

Section

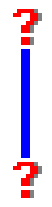


h = 102 mm
b = 120 mm
s = 2 mm
h1 = 26 mm
r = 10 mm

Member length 1,5 m

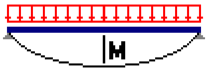


Effective length factor in the XOY plane - 1

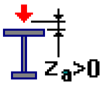


Effective length factor in the XOZ plane - 1

Type of moment diagram



Load position



Height of the load application point = 0 mm

Length between restraints out of the bending plane:
geometric length factor = 1
Effective length factors depending on the boundary conditions of the support sections:
rotation out of the bending plane = 1
warping = 1

	N	M_y	V_z	M_z	V_y	T	B	T_w
	kN	t*m	kN	t*m	kN	t*m	kN*m^2	t*m
1	-85,7	0	0	0	0	0	0	0

Comparison of solutions

Factor	[1], p.143…165	MAGNUM, EN1993-1-1	%
Strength of member	0,634	0.643	1,42
Stability under axial compression (flexural buckling about the y-y axis)	85,7/156,2 = 0,549	0.556	1,26
Stability under axial compression (torsional-flexural buckling)	85,7/109,7 = 0,781	0.791	1,26
Stability under eccentric compression	1,0	0.686	36,9

Comments

When assessing the overall stability of a bar member under eccentric compression in [1] its load-bearing capacity was calculated using a simplified approach based on the formula according to 6.2.5(2), (6.36) EN1993-1-3. In MAGNUM the overall stability of a cold-formed bar member is determined more accurately according to 6.2.5(1) EN1993-1-3.

Calculation of the Load-Bearing Capacity of a Bar Structural Member from a Cold-Formed Hat Channel under Axial Compression

Task: Verify the correctness of the calculation of the load-bearing capacity of bar structural members from cold-formed profiles under axial compression.

Source: [1] Worked examples according to EN 1993-1-3, Eurocode 3, Part 1-3 // ECCS TC7 TWG 7.5 Practical Improvement of Design Procedure. – 1st Ed., ECCS CECM, EKS, 2008. – 235 p.

Compliance with the codes: EN 1993-1-3.

Initial data file:
Task 4.2.sav; report – Report 4.2.doc

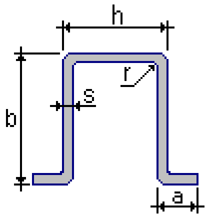
Program version: MAGNUM 23.1.1.3, 07.02.2024

Initial data:

E = 210000 N/mm^2	Elastic modulus
v = 0.3	Poisson's ratio
f_y = 355 N/mm^2	Yield strength
γ_{M0} = 1	Partial safety factor
γ_{M1} = 1	Partial safety factor
h = 180 mm	Section height (along the outer edge)
b = 175 mm	Flange width (along the outer edge)
c = 95 mm	Flange bend length (along the outer edge)
t = 3 mm	Profile thickness (minus the coating thickness)
r = 3.5 mm	Fillet radius (inner)
N = 214.29 kN	Design axial force
ℓ_y = 316 cm	Effective length of the bar member with respect to the y-y axis
ℓ_z = 158 cm	Effective length of the bar member with respect to the z-z axis

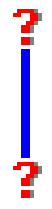
Results in MAGNUM:
Steel: S355
Importance factor 1
Effective length factor for torsional buckling:
coefficient to the geometric length = 1

Section

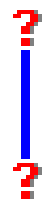


h = 175 mm
b = 180 mm
s = 3 mm
a = 95 mm
r = 3,5 mm

Member length 3,16 м

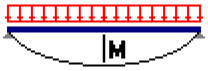


Effective length factor in the XOY plane - 0,5

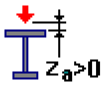


Effective length factor in the XOZ plane - 1

Type of moment diagram



Load position



Height of the load application point = 0 mm

Length between restraints out of the bending plane:
geometric length factor = 1
Effective length factors depending on the boundary conditions of the support sections:
rotation out of the bending plane = 1
warping = 1

	N	M_y	V_z	M_z	V_y	T	B	T_w
	kN	t*m	kN	t*m	kN	t*m	kN*m^2	t*m
1	-214,29	0	0	0	0	0	0	0

Comparison of solutions

Factor	[1], p.181…184	MAGNUM, EN1993-1-1	%
Strength of member	–	0,554	–
Stability under axial compression (flexural buckling about the y-y axis)	214,29/385,56 = 0,556	0,56	0,714
Stability under axial compression (flexural buckling about the z-z axis)	214,29/421,83 = 0,508	0,491	3,34
Stability under axial compression (torsional and torsional-flexural buckling)	214,29/214,29 = 1	1,113	11,3

Comments

When assessing the overall stability of a bar member in [1] the cross-sectional properties were calculated taking into account the fillets. In MAGNUM they are calculated without taking the fillets into account, which is permitted by the codes for the considered fillet radius (see Sec. 5.1(3) EN1993-1-3).