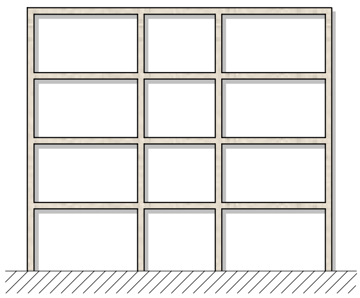
DESIGN OF HIGH-RISE BUILDINGS

ACCORDING TO EUROCODE



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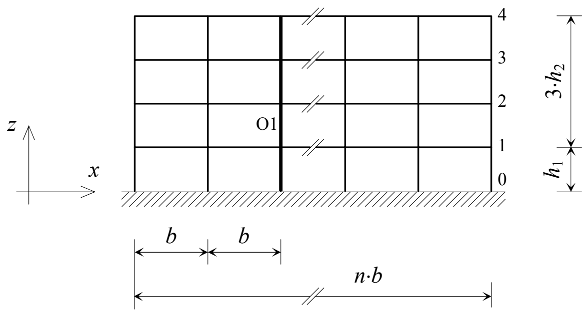
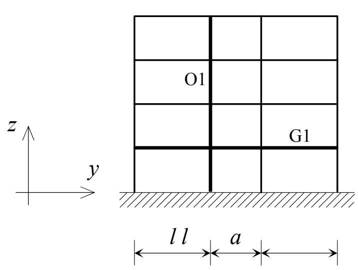
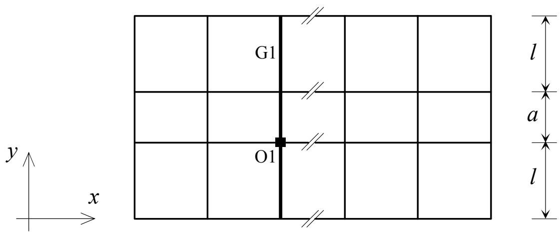
1. Initial data

1.1 Geometry

Geometry according to the data sheet.







1.2 Materials

Grade of materials:

C oncrete

Grade	C16/20	C 20/25	C 25/30
f_{ck} [N/mm²]	16	20	25
f_{cd} [N/mm²]	10,7	12	15
f_{ctm} [N/mm²]	1,9	2,21	2,56
f_{ctd} [N/mm²]	0,89	1,03	1,2
E_{cm} [kN/mm²]	27,4	28,8	30,5

Reinforcement

Grade	B 50.36	B 60.40	B 60.50
f_{yk} [N/mm²]	360	400	500
f_{yd} [N/mm²]	313	348	435
ε_{su} [%]	2,5	2,5	2,5
ξ_{c0}	0,55	0,53	0,49
ξ'_{c0}	1,45	1,59	2,11

1.3 Loads

1.3.1 Self weight and dead loads

The thickness of the floor slab $v_{slab} \approx l_{shorter}/35 = b/35$. The loads of the inner and roof floors will differ due to the different layers and live load. See an example of interior floor slab layers:

Material of layers	**Thickness (v) [mm]**	**Density (ρ) [kN/m³]**	**Weight (g_i) [kN/m²]**
floor tile	10	23	0,23
cement mortar	20	22	0,44
screed	40	22	0,88
techn. insulation	-	-	-
mineral wool	30	0,5	0,015
in situ r.c. slab	v_{slab}	25	$v_{slab}\cdot 25$
plaster	15	20	0,3
partition wall			2,5

The self weight and the dead load of layers together: $g_k = \Sigma g_i$

See an example of roof floor slab layers:

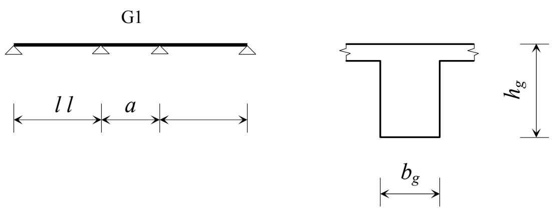
Material of layers	**Thickness (v) [mm]**	**Density (ρ) [kN/m³]**	**Weight (g_i) [kN/m²]**
bit. water proofing.	4	12	0,05
heat insulation	100	1,6	0,16
moisture barrier	2	12	0,025
breeze concrete	60	22	1,32
in situ r.c. slab	v_{slab}	25	$v_{slab}\cdot 25$
plaster	15	20	0,3

The self weight and the dead load of layers together: $g_{kf} = \Sigma g_i$

To calculate the self weight of beam G1 we assume its sizes as follows:

height: $h_g \approx \dfrac{l}{12}$

width: $b_g \approx \dfrac{h_g}{1{,}5 \div 2{,}0}$



Characteristic value of self weight: $g_{beam} = (h_g - v_{slab})\cdot b_g \cdot \rho_{rc}$

Safety factor for self weights and dead loads: $\gamma_G = 1{,}35$

1.3.2 Imposed loads

a.) **Imposed loads** on the interior floor slabs

See the characteristic value of the imposed load (q) on the data sheet. Safety factor: γ_Q = 1,5; the combination factor $\psi_0 = 0{,}7$ (for imposed loads in case of flats, dwelling houses, office buildings, areas for assembly and shops).

b.) **Meteorological loads**

Snow load

The design value of the snow load:

$s_d = \gamma_s \cdot s$

where: s projected snow load on the horizontal surface of roofs inclined by α
$\gamma_s = 1{,}5$ safety factor for snow load.

The projected snow load on a horizontal surface can be taken as:

$s = \mu_i \cdot C_e \cdot C_t \cdot s_k$

where: s_k characteristic snow load, in Hungary its value may be computed as follows:

$$s_k = 0{,}25 \cdot \left(1 + \frac{A}{100}\right) \qquad [\mathrm{kN/m^2}]$$

but: $s_k \geq 1{,}25$ kN/m^2 in zone I., (West-Hungary)
$s_k \geq 1{,}00$ kN/m^2 in zone II., (Hungary, except the Western part)

A – height of the area above Baltic Sea level [m].

C_e reduction factor due to the wind effect; in case of usual weather condition it is 1,0. This factor is less than 1,0 if the wind is generally very strong.

C_t reduction factor due to the heat effect, at normally isolated roofs it is 1,0. This factor is less than 1,0 if the loss of the heat through the roof is high, and it reduces the snow depth.

μ_i shape factor for the snow load, in case of inclination of the roof $\alpha = 0^0$ its value is $\mu_i = 0{,}8$.

The combination factor for the snow load:

$\psi_0 = 0{,}6$

- Wind load:

  The horizontal wind load in longitudinal direction of the structure (in direction x) should be taken by shear walls. In transverse direction of the building (y) the frame itself will resist the wind load, because the frame is side-swaying. The frame should be analysed for the horizontal wind load, so the columns are unbraced. The wind load should be calculated for a b wide facade of a frame, where b is the distance between the frames.

Design wind pressure, perpendicular to the building facade

$w_d = \gamma_w \cdot w_e$

where: w_e wind pressure, perpendicular to the building facade
$\gamma_w = 1{,}5$ safety factor for the wind load.

The wind pressure, perpendicular to the building's facade:

$w_e = q_{ref} \cdot c_e(z_e) \cdot c_{pe}$

where: q_{ref} the average wind pressure, it is the characteristic value:

$$q_{ref} = \frac{\rho}{2} v_{ref}^2 \quad [\mathrm{N/m^2}]$$

where: ρ density of the air, depending on the height above sea level, the temperature and the air pressure, in general case it may be assumed as 1,25 kg/m^3.

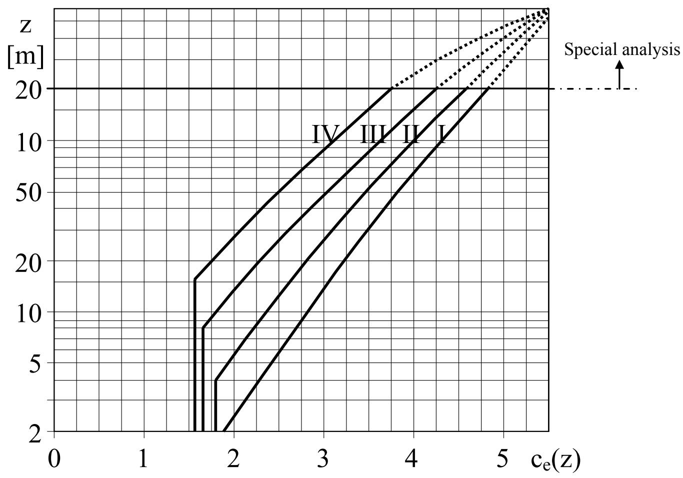
v_{ref} the reference value of the wind speed, in Hungary its value is 20 m/s.

Substituting the values above, in Hungary a value of $q_{ref} = 0{,}25$ kN/m^2 should be taken.

$c_e(z_e)$ topographical factor, its value depends on the topography of the surroundings in function of the reference height above sea level z_e. See the topographical categories in Table:

Topographical conditions	
I.	Open country, with no obstruction
II.	Open country with scattered wind break (rural area with small houses or trees
III.	Outskirts, or industrial zones, steady forest,
IV.	City zone, where at least the 15 % of the ground is covered with buildings higher than 15 meter

The topographical factor can be obtained from the diagram on the next page.



In case of wind load on the vertical surface of the building the EUROCODE classifies different zones with different wind pressures. If the height of the building does not exceed its width, it is enough to take only one zone into consideration. We use that case when the reference height is equal to the height of the structure:

$$z_e = H = m_f + m_t + h.$$

c_{pe} the <u>outer pressure coefficient</u> depends on the area of the surface on which the wind pressure should be calculated and may be obtained from equations below:

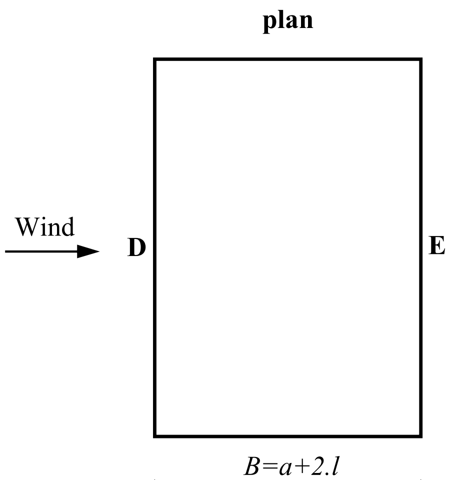
$c_{pe} = c_{pe,1}$ if $A \leq 1\ \text{m}^2$

$c_{pe} = c_{pe,1} + (c_{pe,10} - c_{pe,1})\cdot\log_{10}A$ if $1\ \text{m}^2 < A < 10\ \text{m}^2$

$c_{pe} = c_{pe,10}$ if $10\ \text{m}^2 \leq A$

where $c_{pe,1}$ and $c_{pe,10}$ are the values of c_{pe} to the loaded surface of $A = 1\ \text{m}^2$ and $A = 10\ \text{m}^2$, respectively. The values of the outer pressure coefficient are summarized in table.

The outer pressure coefficients in case of wind load on the vertical sidewall of the building:



B/H	Zones (surfaces)			
	D E			
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
≤ 1	+0,8	+1,0	-0,3	
≥ 4	+0,6	+1,0	-0,3	

Use linear interpolation for the interior values of b/H.

The safety factor for the wind load: $\gamma_w = 1{,}5$; and the combination factor: $\psi_0 = 0{,}6$.

1.4 Codes, manuals and computer programs

EUROCODE 1

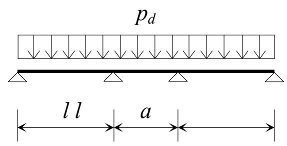
EUROCODE 2

AXIS-VM7

2. Sizing of load bearing elements

2.1 Bending moments and shear forces of beam G1

2.1.1 Bending moments due to vertical loads

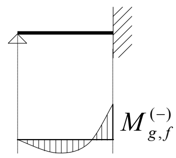


Design load acting on the interior floor:

$$p_d = b\cdot(\gamma_G\cdot g_k + \gamma_Q\cdot q) + \gamma_G\cdot g_{beam}$$

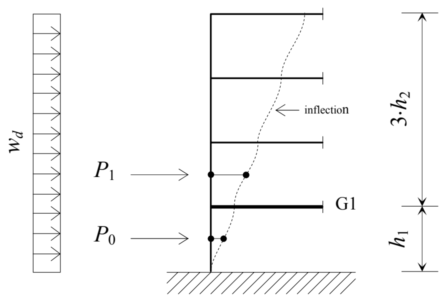
The ultimate bending moment due to the design load according to approximate calculation:

$$M_{g,f}^{(-)} \approx p_d \frac{l^2}{10{,}5}$$



2.1.2 Bending moments due to horizontal loads

Assuming that beams are much stiffer than columns we get the side swaying shape of the columns due to wind load:



The resultant of the wind load acting at height $h_1 + h_2/2$:

$$P_1 = 2{,}5\cdot b\cdot h_2\cdot w_d$$

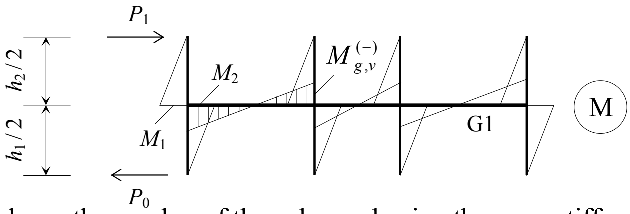
The resultant of wind load at height $h_1/2$:

$$P_0 = (3\cdot h_2 + 0{,}5\cdot h_1)\cdot b\cdot w_d$$

As the bending moment in the inflection point is zero and the stiffness of the columns against displacement are equal, the bending moment on the column due to the wind forces P_0 and P_1:

$$M_1 = \pm \frac{P_1}{4} \cdot \frac{h_2}{2}$$

$$M_2 = \pm \frac{P_0}{4} \cdot \frac{h_1}{2}$$



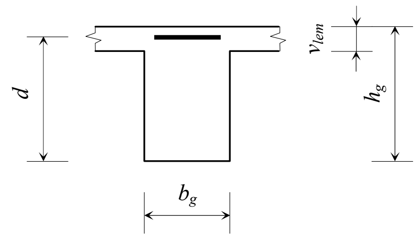
The number 4 in the denominator shows the number of the columns having the same stiffness.
The bending moment at the end of the left side of beam G1 due to wind load, approximately:

$M_{g,v}^{(-)} = M_1 + M_2$ (as a matter of fact, $M_{g,v}^{(-)} < M_1 + M_2$; the exact value depends on the relative stiffness of the members connecting into the joint. For example in case of $l=a$, $h_1=h_2$, I_{beam}=const. and I_{col}=const. $M_{g,v}^{(-)} = (M_1+M_2)/2$)

2.2 Checking the sizes of beam G1

The assumed sizes of the beam should be checked at the interior support. The actual ultimate bending moment is the sum of the moments due to horizontal and vertical loads, respectively:

$$M_{Sd} = M_{g,f}^{(-)} + M_{g,v}^{(-)}$$



Moment of resistance:

$$M_{Rd} = b_g \cdot \xi_c \cdot d \cdot \alpha \cdot f_{cd} \left(d - \frac{\xi_c \cdot d}{2} \right)$$

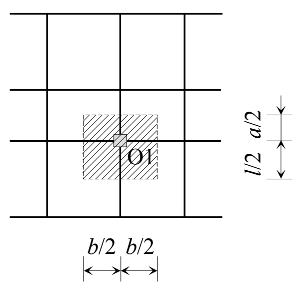
where $\xi_c = \xi_{c0} = \dfrac{560}{700 + f_{yd}}$ may be assumed.

The assumed sizes are appropriate, if

$M_{Sd} \leq M_{Rd}$

If the ultimate actual bending moment is greater than the moment of resistance by up to 20-30% ($M_{Sd} \leq 1{,}2 \div 1{,}3 * M_{Rd}$), the load bearing capacity can be even provided with compressed bars. Otherwise the sizes of sections should be increased.

2.3 Forces and moments, sizing of column O1



The sizes of the column section should be calculated with trial-and-error method. At first approximation, let the width of column section be equal to the width of the beam G1: $b_{col} = b_{beam}$. And the height of column section should be taken approximately $h_{col} = 1{,}0 \div 1{,}5 \cdot b_{col}$.

When sizing the column section we take into consideration only the axial force due to vertical loads. The bending moments should be taken into consideration only in exact method calculating reinforcement.

The characteristic value of the column's self weight:
$g_{col} = b_{col} \cdot h_{col} \cdot \rho_{rc}$

The sizes of the column due to upward decreasing load may be even decreased. As the length of steel bars may be about two levels long, the section of the column should be the same along two levels. Therefore it may be taken, that above section "2" we use a smaller section than at the lower levels. In this case two sections of the column should be analysed.

The axial forces on level "0" and "2" due to vertical loads:

$$N_{Sd}^{0} = \frac{l+a}{2} b \cdot \left[\gamma_G \cdot \left(3 \cdot g_k + g_{kf}\right) + \left(\gamma_Q \cdot 3 \cdot q + s_d\right)\right] + 4 \cdot \left(b + \frac{l+a}{2}\right) \cdot \gamma_G \cdot g_{beam} + \left(h_1 + 3 \cdot h_2\right) \cdot \gamma_G \cdot g_{col}$$

$$N_{Sd}^{2} = \frac{l+a}{2} b \cdot \left[\gamma_G \cdot \left(g_k + g_{kf}\right) + \left(\gamma_Q \cdot q + s_d\right)\right] + 2 \cdot \left(b + \frac{l+a}{2}\right) \cdot \gamma_G \cdot g_{beam} + 2 \cdot h_2 \cdot \gamma_G \cdot g_{col}$$

The required area of the concrete, neglecting the reinforcement:

$$A_{c,i} = \frac{N_{Sd}^{i}}{\alpha \cdot f_{cd}}$$

The assumed section of the column will be appropiate, if the assumed area is bigger than the required:

$$A_{c,i} \le A_{c,i,appl} = b_{col} \cdot h_{col}$$

Otherwise the sizes of the section should be increased, and the recalculation should be started with the recalculation of the selfweight. When sizing, please pay attention to the rule that the smallest size of the column section is 200 mm, and generally the height of section should not be bigger than four times their width.

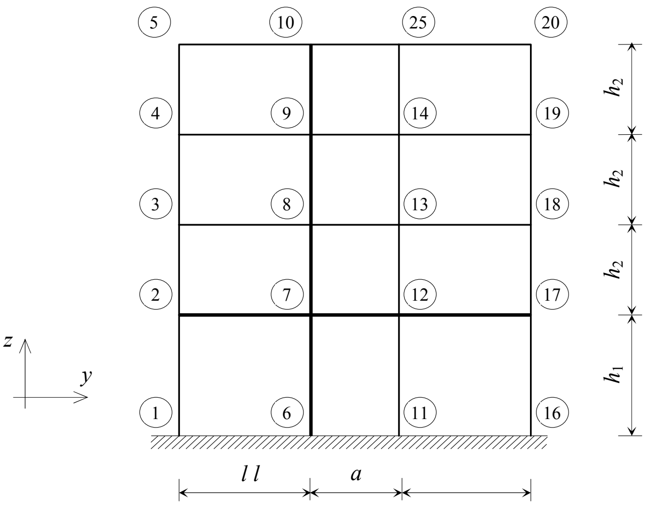
3. Exact analysis

3.1 Preparation for calculation

The exact calculation should be made with the computer program AXIS-VM7.

The structure, the coordinates

Prepare the data for the computer program as follows:



- coordinates of the joints (node),

- sizes of the cross-section of columns and beams, respectively (it may be calculated neglecting the reinforcement, that is gross section),

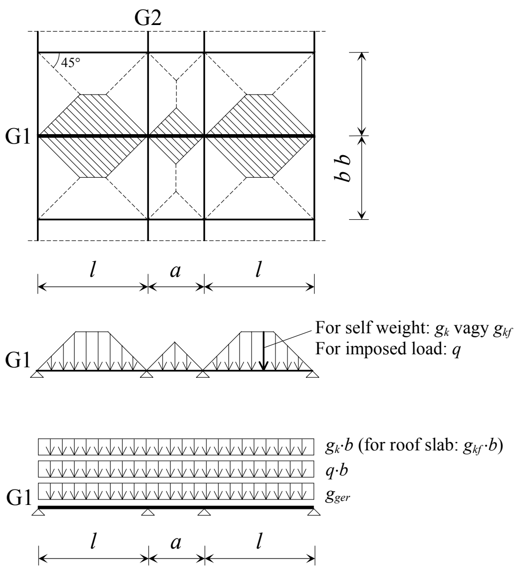
- Young modulus of concrete ($E_{c,eff}$).

3.2 Loads

The loading from the slabs onto the beam G1 may be taken exactly or approximately.

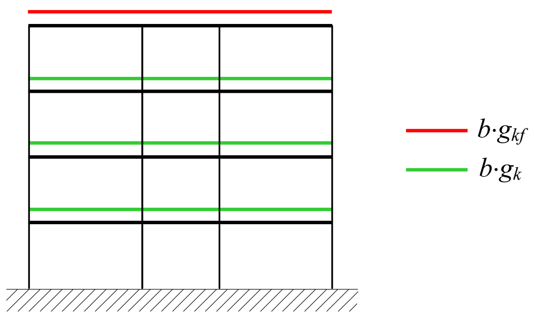
Using the exact method the loading area of beam G1 will be trapezoidal as seen in the figure. In that case the reactions of beam G2 should be taken into account on the column as a concentrated load. Even the self weight g_{beam} of beam G1 should be taken into calculation.

Using approximate method it may be assumed that the total load is transferred to the beam G1 with a rectangular shaped loading scheme, as it can be seen in figure. The frame should be partially loaded with the imposed load. All loads are UDL.

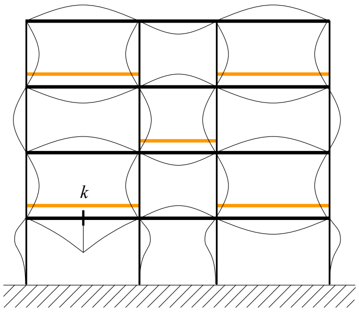


3.3 Load cases

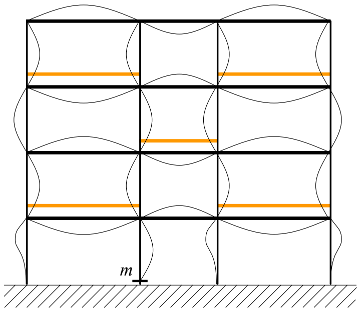
TE1: The total load is the dead loads of the interior and roof floors ($b \cdot g_k$, $b \cdot g_{kf}$). If the program does not calculate the self weight of beams and columns (g_{beam}, g_{col}) automatically, these loads should be added to this load case. More exact result will be produced with consideration of the self weight of beam G2 (only its part under the slab) in the joints as concentrated loads (except joints 1, 6, 11, 16).



TE2: Partial loading with imposed load ($b{\cdot}q$) on slabs as seen in figure. This load case produces the ultimate (maximum) positive midspan bending moment in beam G1 ($M^{(+)}_{Sd,k}$), and the bending moment in column O1 at level "0" ($M_{Sd,m}$).



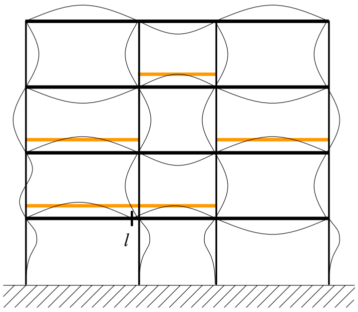
Influence line for bending moments for section "*k*" in beam G1



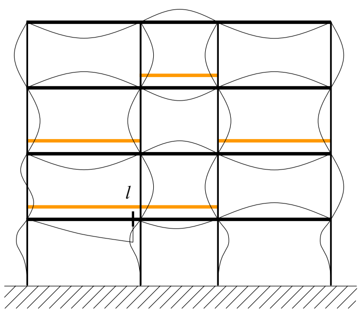
Influence line for bending moments for section "*m*" in column O1

$b{\cdot}q$

TE3: Partial loading with imposed load ($b{\cdot}q$) on slabs as seen in figure. This load case produces the ultimate (maximum) negative support bending moment in beam G1 ($M^{(-)}_{Sd,l}$), and its shear force ($V_{Sd,l}$),.



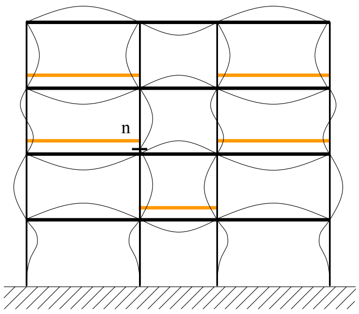
Influence line for bending moments for section "*l*" in beam G1



Influence line for shear forces for section "*l*" in beam G1

$b{\cdot}q$

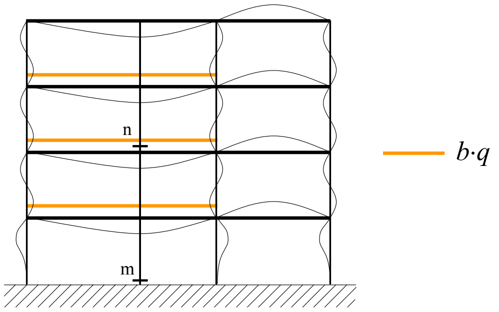
TE4: Partial loading with imposed load ($b{\cdot}q$) on slabs as seen in figure. This load case produces the ultimate (maximum) bending moment due to the vertical load at level "2" of column O1 ($M_{Sd,n}$).



$b{\cdot}q$

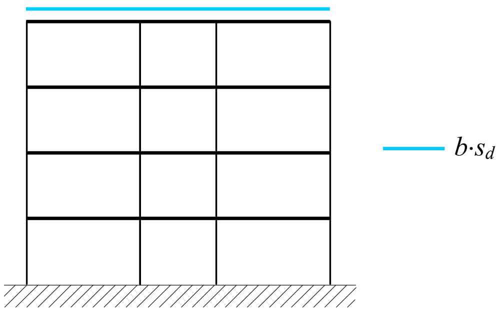
Influence line for bending moments for section "*n*" in column O1

TE5: Partial loading with imposed load ($b{\cdot}q$) on slabs as seen in figure. This load case produces the ultimate (maximum) axial force in column O1 $(N_{Sd,m}, N_{Sd,n})$.

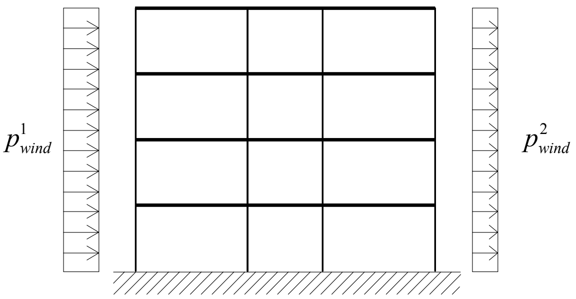


Influence line for axial force for sections "*m*" and "*n*" in column O1

TE6: Total snow load on a roof slab



TE7: The wind load (calculated in Chapter 1.3.2) for compression and suction respectively, should be placed onto the structure, as shown in figure.



$p^1_{wind} = w_d \cdot b$ (surface **D**)

$p^2_{wind} = w_d \cdot b$ (surface **E**)

(see Chapter 1.3.2)

3.4 Ultimate forces and bending moments

The ultimate bending moments, shear forces and axial forces should be compiled from the combinations of load cases.

3.4.1 Bending moments

The ultimate positive bending moment in section "*k*" of beam G1 will be obtained from the load combination as follows:

$$M_{Sd,k}^{(+)} = \gamma_G \cdot M_k(\mathrm{TE1}) + \gamma_Q \cdot M_k(\mathrm{TE2}) + \gamma_Q \cdot \psi_0 \cdot M_k(\mathrm{TE7})$$

where $M_k(\mathrm{TE}i)$ is the bending moment in section "*k*" from the load case "*i*". The value of combination factor ψ_0 in case of flats, dwelling houses, offices, assembly buildings is $\psi_0 = 0{,}7$, in case of snow and wind load it is $\psi_0 = 0{,}6$.

The ultimate negative bending moment in section "*l*" of beam G1 will be obtained from the load combination as follows:

$$M_{Sd,l}^{(-)} = \gamma_G \cdot M_l(\mathrm{TE1}) + \gamma_Q \cdot M_l(\mathrm{TE3}) + \gamma_Q \cdot \psi_0 \cdot M_l(\mathrm{TE6}) + \gamma_Q \cdot \psi_0 \cdot M_l(\mathrm{TE7})$$

When calculating the maximal bending moment in column O1 it can not be predicted, which load combination provides the bigger moment, either the imposed load on slab or the wind load should be beneficial. So the ultimate bending moment in section "*m*" of column O1 will be obtained from the load combinations as follows:

$$M_{Sd,m} = \max\begin{cases} \gamma_G \cdot M_m(\mathrm{TE1}) + \gamma_Q \cdot M_m(\mathrm{TE2}) \pm \gamma_Q \cdot \psi_0 \cdot M_m(\mathrm{TE7}) \\ \gamma_G \cdot M_m(\mathrm{TE1}) \pm \gamma_Q \cdot M_m(\mathrm{TE7}) + \gamma_Q \cdot \psi_0 \cdot M_m(\mathrm{TE2}) \end{cases}$$

The sign of the bending moment from wind load may be positive or negative, depending on the direction of wind. The ultimate bending moment in section "*n*" of column O1 will be similarly obtained from the load combinations as follows:

$$M_{Sd,n} = \max\begin{cases} \gamma_G \cdot M_n(\mathrm{TE1}) + \gamma_Q \cdot M_n(\mathrm{TE4}) \pm \gamma_Q \cdot \psi_0 \cdot M_n(\mathrm{TE7}) \\ \gamma_G \cdot M_n(\mathrm{TE1}) \pm \gamma_Q \cdot M_n(\mathrm{TE7}) + \gamma_Q \cdot \psi_0 \cdot M_n(\mathrm{TE4}) \end{cases}$$

3.4.2 Axial force on the column

Törölt: ¶

Formázott: Felsorolás és számozás

The ultimate axial force in sections "*n*" and "*m*" of column O1 arise from the load combination as follows:

$$N_{Sd,n} = \gamma_G \cdot N_n(\mathrm{TE1}) + \gamma_Q \cdot N_n(\mathrm{TE5}) + \gamma_Q \cdot \psi_0 \cdot N_n(\mathrm{TE6}) \pm \gamma_Q \cdot \psi_0 \cdot N_n(\mathrm{TE7})$$

and

$$N_{Sd,m} = \gamma_G \cdot N_m(\mathrm{TE1}) + \gamma_Q \cdot N_m(\mathrm{TE5}) + \gamma_Q \cdot \psi_0 \cdot N_m(\mathrm{TE6}) \pm \gamma_Q \cdot \psi_0 \cdot N_m(\mathrm{TE7})$$

3.4.3 Shear force in the beam

The ultimate shear force in sections "l" of beam G1 arise from the load combination as follows:

$$V_{Sd,l} = \gamma_G \cdot V_l(\text{TE1}) + \gamma_Q \cdot V_l(\text{TE3}) + \gamma_Q \cdot \psi_0 \cdot V_l(\text{TE6}) + \gamma_Q \cdot \psi_0 \cdot V_l(\text{TE7})$$

3.4.4 Appropriate axial loads and bending moments

Analysing the column loads and bending moments we need the pair of loads and moments associated such:

$M_{Sd,m}^{appr}$ - and $N_{Sd,m}$ maximum axial force in section "m" and appropriate bending moment,
$N_{Sd,m}^{appr}$ - and $M_{Sd,m}$ maximum bending moment in section "m" and appropriate axial force,
$M_{Sd,n}^{appr}$ - and $N_{Sd,n}$ maximum axial force in section "n" and appropriate bending moment,
$N_{Sd,n}^{appr}$ - and $M_{Sd,n}$ maximum bending moment in section "n" and appropriate axial force.

3.5 Analysing of beam

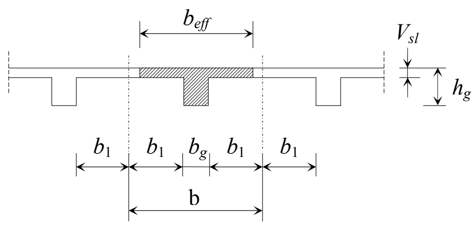
3.5.1 The bending moment

The reinforcement of beam G1 should be calculated in their midspan section ("k") and at the middle support section ("l").
The section of the beam may be assumed as rectangular at the support and T shaped in the midfield. In case of T shaped sections the width of the compression zone depends on the sizes of the web, the slab thickness, the type of loading, the span of the beam, the support conditions and the lateral reinforcement. The width of compression zone along the whole beam in a symmetrical case:

$$b_{eff} = b_g + \frac{1}{5} \cdot l_0 < b$$

at end beam:

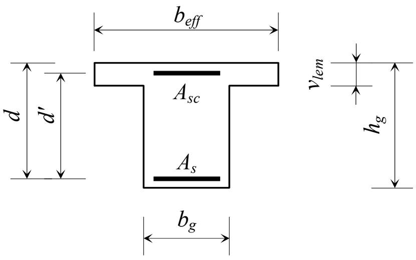
$$b_{eff} = b_g + \frac{1}{10} \cdot l_0 < b_1$$



In the expressions above, the length l_0 is the distance between the zero moment points of beams and l_0 has a value as follows:

$l_0 = 2 \cdot l$	in case of a cantilever beam,
$l_0 = 0{,}75 \cdot l$	fixed end beam (used in this project),
$l_0 = 0{,}85 \cdot l$	fixed end at one support and simply supported at the other one.

The compression zone x_c in case of T shaped sections (in the midspan) can be calculated using the equivalent equations for bending moments. Assuming that the compression zone is smaller than the slab thickness and no need for compression steel bars:



$$b_{eff} \cdot x_c \cdot \alpha \cdot f_{cd}\left(d - \frac{x_c}{2}\right) = M_{Sd,k}^{(+)}$$

The effective height of the section may be taken as $d \approx h_g$ - 50 mm (assuming 20 mm concrete cover, ∅10 stirrup, ∅20 main bars and 10 mm disadvantageous positioning of steel bars).

If the condition $\xi_c = \frac{x_c}{d} \leq \xi_{c0}$ is true, the tension bars will yield and there is no need for compression bars. In this case the area of steel required can be obtained from the horizontal force equilibrium:

$$A_{s,req} = \frac{b_{eff} \cdot x_c \cdot \alpha \cdot f_{cd}}{f_{yd}}$$

In case of $\xi_c > \xi_{c0}$ the tension bars will not yield, and we should apply compression bars, too. The maximum bending moment of the section without a compression bar in case of just yielding the tension bars:

$$M_0 = \alpha \cdot f_{cd} \cdot b_{eff} \cdot d^2 \cdot \xi_{c0}\left(1 - \frac{\xi_{c0}}{2}\right)$$

The area of compression bars to equalize the difference between bending moments M_0 and $M_{Sd,k}^{(+)}$:

$$A_{sc,req} = \frac{M_{Sd,k}^{(+)} - M_0}{f_{yd} \cdot d'}$$ where d' is the distance between the compression and tension bars.

And the area of tension reinforcement in this case:

$$A_{s,req} = A_{sc,req} + \frac{b_{eff} \cdot d \cdot \xi_{c0} \cdot \alpha \cdot f_{cd}}{f_{yd}}.$$

In case of a higher compression zone than the slab width, the equations of the area of steel required should be modified. The main reinforcement of the support section of beam G1 should be calculated for bending moment $M_{Sd,l}^{(-)}$ in the same way as for the midfield moment.

Reinforcement details

Take care of the most important rules of reinforcement listed below:

- there must be bars in every corners of the section,
- the main bars should be fixed with stirrups,
- the minimal area of tension bars:

$$A_{s,\min} = \max\begin{cases} 0{,}6 \cdot b_t \cdot d / f_{yk} \\ 0{,}0015 \cdot b_t \cdot d \end{cases}$$ where b_t is the general width of the tensioned concrete ,

- the maximal area of bars (the tension and compression bars together): $A_{s,\max} = 0{,}04 \cdot A_c$ where A_c is the area of the concrete section,
- the minimum distance between the main bars is the bigger of 20 mm and the diameter of bar,
- the diameter of the main bars should be at least 8 mm; in case of fixing or distributor bars at least 6 mm,
- bars in two or more lines should be placed above each other,
- the minimal concrete cover is the bigger of 15 mm and the diameter of the main bar,
- the one third of the midfield reinforcement should be provided above the support.

3.5.2 Analysis of shear

According to EC-2 the checking of the shear resistance of beams is based on the three resistance equations of the shear given with the following equations:

a.) Shear resistance of a section without shear reinforcement:

$$V_{Rd1} = \left[\tau_{Rd} \cdot k \cdot \left(1{,}2 + 40 \cdot \rho_l\right)\right] \cdot b_g \cdot d$$

in this equation above:

$\tau_{Rd} = (0.25 \cdot f_{ctk0.05}) / \gamma_c$ - design value of shear strength,
$k = \left|1{,}6 - d\right| \geq 1$ - where d is in [m],
$\rho_l = A_{sl} / \left(b_w \cdot d\right) \leq 0{,}02$ - proportional area of the tension bar,
A_{sl} – area of steel bars overlapping the checked section at least with a length of $d + l_{b,net}$,

b.) The maximal shear force taken by the pure concrete semi-members of the section without collapse of theses members:

$$V_{Rd2} = \frac{1}{2} \cdot \nu \cdot f_{cd} \cdot b_g \cdot 0{,}9 \cdot d \quad \geq \quad V_{Sd,l}$$

where the factor of the effectiveness: $\nu = 0{,}7 - \frac{f_{ck}}{200} \geq 0{,}5$

If the beam is not appropriate according to this analysis in *b.)*, the sizes of the concrete section should be modified!

c.) The shear resistance of the section with shear reinforcement:

$$V_{Rd3} = V_{cd} + V_{wd} \quad \geq \quad V_{Sd,l}^{red}$$

in the equation above:

$V_{Sd,l}^{red}$ - reduced shear force,
$V_{cd} = V_{Rd1}$ - shear force taken by the concrete,

$V_{wd} = \frac{A_{sw} \cdot f_{ywd}}{s} \cdot 0{,}9 \cdot d$ - shear force taken by the reinforcement, where

A_{sw} - area of shear reinforcement,
s – spacing of shear reinforcement,
f_{ywd} – design strength of shear reinforcement.

In those sections where the reduced shear force $V_{Sd,l}^{red}$ is bigger than the shear resistance V_{Rd1}, the computed shear reinforcement should be provided to fulfil the requirement of $V_{Sd,l}^{red} \le V_{Rd3}$. The area of steel required for the shear can be computed from the inequality above. From the two unknown values either the area of steel bars (A_{sw}) or the spacing (s) should be assumed. Remember the rules of reinforcement! The computed reinforcement must not be less than the minimal area of steel given in the Code. Bent up bars for shear in beams should be used with stirrups. In case bent up bars are used, at least the half of the ultimate shear force $V_{Sd,l}$ should be taken by stirrups.

The rules for shear reinforcement:

- The relative area of steel for shear: $\rho_w = \frac{A_{sw}}{b_g \cdot s} \ge \rho_{w,\min}$ where A_{sw} is the area of shear reinforcement along a distance of s, and s is the distance between bars provided for shear. The minimal values of $\rho_{w,min}$ given in EC2 can be seen in Table below:

Grade of concrete	Grade of reinforcement		
	S220	S400	S500
C12/15 and C20/25	0,0016	0,0009	0,0007
C25/30 and C35/45	0,0024	0,0013	0,0011
C40/50 and C50/60	0,0030	0,0016	0,0013

- The limitation of the cracks due to tangential forces may be sufficient if the spacing of stirrups fulfil the requirements of the code. For this limitation the maximal spacing of stirrups are given in Table below:

$\frac{(V_{Sd,l} - 3V_{cd})}{\rho_w b_g d}$ [N/mm^2]	spacing [mm]
≤ 50	300
75	200
100	150
150	100
200	50

- The inclination of shear reinforcement to the middle plane of the structural element should be 45º-90º.
- The shear reinforcement should be anchored properly.
- The maximum spacing (s_{max}) of stirrups along the axis of the beam may be determined based on the conditions below:

if $V_{Sd,l} \leq \frac{1}{5} V_{Rd2}$, then $s_{max} = 0{,}8 \cdot d \leq 300$ mm,

if $\frac{1}{5} V_{Rd2} \leq V_{Sd,l} \leq \frac{2}{3} V_{Rd2}$, then $s_{max} = 0{,}6 \cdot d \leq 300$ mm,

if $\frac{2}{3} V_{Rd2} \leq V_{Sd,l}$, then $s_{max} = 0{,}3 \cdot d \leq 200$ mm.

3.6 Analysis of column O1

3.6.1 Initial loads and bending moments

To analyse the column O1 we need the results of the computer analysis as follows:

On level "0": $N_{Sd,m}$; $M_{Sd,m}^{appr}$

$M_{Sd,m}$; $N_{Sd,m}^{appr}$

On level "2": $N_{Sd,n}$; $M_{Sd,n}^{appr}$

$M_{Sd,n}$; $N_{Sd,n}^{appr}$

The column should be analysed in these sections to prove that they are sufficient. As it can not be decided in advance, which pair of load and bending moment will result the bigger effect, all cases should be calculated.

3.6.2 Buckling length of columns

The buckling length of reinforced concrete columns can be calculated with:

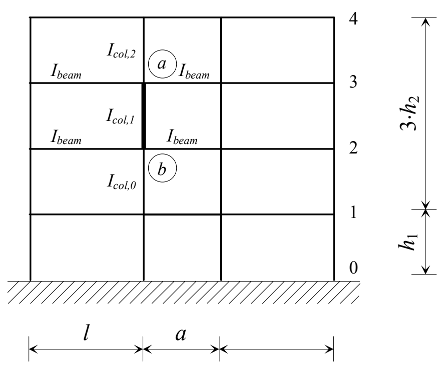
$l_0 = \beta \cdot l_{col}$

where l_{col} is the effective height of the column, and β is a factor in the function of the end conditions as follows:

in case of unbraced columns: $$\beta = \min \begin{cases} 1{,}0 + 0{,}15 \cdot (k_a + k_b) \\ 2{,}0 + 0{,}3 \cdot k_{min} \end{cases}$$

in case of braced columns: $$\beta = \min \begin{cases} 0{,}7 + 0{,}05 \cdot (k_a + k_b) \\ 0{,}85 + 0{,}05 \cdot k_{min} \\ 1 \end{cases}$$

In our project the frame is assumed to be unbraced in plane *z-y*, and braced in plane *x-z*. In the equations above the restraint coefficients k_a and k_b depend on the relative stiffness of the beams joining to the end of column, coefficient k_{min} is the lowest of these two values.



$$k_a(\text{or } k_b) = \frac{\sum \frac{E_c \cdot I_{col}}{l_{col}}}{\sum \frac{E_c \cdot \alpha \cdot I_{beam}}{l_{beam}}}$$

where:

E_c - the Young-modulus of concrete,
I_{col} - inertia if columns are connected to the joint,
l_{col} - effective height of the column connected to the joint,
I_{beam} - inertia if beams are connected to the joint,
L_{beam} - effective length of the beam connected to the joint,
α - factor taking into consideration the end condition of the other end of the beam, its value is as follows:
$\alpha = 1$ if the other end is fixed (use this case in the project),
$\alpha = 0{,}5$ if the other end is hinged,
$\alpha = 0$ if the other end is free (cantilever).

At the end of a column where there is a pinned joint the value of k may be assumed as $k = \infty$. If the end of the column is fixed (see in our project at level "0"), then $k = 0$. In the project the buckling length of the column above the levels "0" and "2" should be determined in plane x-z and y-z, respectively. For the inertia of beam G2 running in direction x it may be assumed that $I_{g,x} \geq I_{beam}$, where $I_{g,y} = I_g$ is the inertia of the beam G1.

To go on with the analysis we need the slenderness of the column:

$$\lambda = \frac{l_0}{\sqrt{\frac{I_{col}}{A_{c,col}}}}$$

where I_{col} and $A_{c,col}$ are the inertia and the area of the cross-section of the column (assuming non-cracked, concrete section).
The column may be assumed as stub, and the effect of lateral deflections may be neglected if the slenderness (in plane y-z) fulfils the requirements:

$$\lambda < \min\begin{cases} 1{,}5\sqrt{\frac{A_{c,col} \cdot f_{cd}}{N_{Sd}}} \\ 25 \end{cases}$$

If the column is a stub and the frame is braced and the slenderness of the column is smaller than the critical slenderness, the column can be analysed for the axial force N_{Sd} and the bending moment M_{Sd}^{appr} (the bending moment M_{Sd}^{appr} must not be smaller than $N_{Sd} \cdot h_{col}/20$). The critical slenderness:

$$\lambda_{krit} = 25\left(2 - \frac{e_{o1}}{e_{o2}}\right)$$

where e_{o1} and e_{o2} are the eccentricities of the axial force at the two ends of the column ($|e_{o1}| \leq |e_{o2}|$).

If the column slenderness is lower than 140 ($\lambda \leq 140$), the columns should be analysed for the pair of axial force and bending moment as follows:

N_{Sd}; $M_{Sd} = N_{Sd} \cdot e_{tot}$

where e_{tot} is the eccentricity of the column in the middle section.

3.6.3 Increments in lateral displacement

The eccentricity of the column in the middle section:

$e_{tot} = e_o + e_a + e_2$

In the equation above the e_o is the eccentricity due to the initial axial force and bending moment:

$$e_o = \max\begin{cases} 0{,}6 \cdot e_{o2} + 0{,}4 \cdot e_{o1} \\ 0{,}4 \cdot e_{o2} \end{cases}$$

where e_{o1} and e_{o2} are the eccentricities of the axial force on the two ends of the column ($|e_{o1}| \leq |e_{o2}|$).

e_a is the eccentricity due to construction inaccuracy:

$$e_a = \nu \frac{l_0}{2}$$

where l_0 is the buckling length of the column, ν is the slanting of the building:

$$\nu = \max\begin{cases} \dfrac{1}{100 \cdot \sqrt{l_{col}}} \\ \dfrac{1}{200} \end{cases}$$

e_2 is the second order eccentricity:

$$e_2 = k_1 \frac{l_0^2}{10} \cdot \kappa$$

where: l_0 is the buckling length of the column,

$$k_1 = \begin{cases} \dfrac{\lambda}{20} - 0{,}75 & if\ 15 \le \lambda \le 35 \\ 1 & if\ \lambda > 35 \end{cases}$$

$\kappa = \dfrac{2 \cdot k_2 \dfrac{f_{yd}}{E_s}}{0{,}9 \cdot d}$ the curvature of the r.c. section loaded with axial force N_{Sd} and bending moment $M_{Sd} = N_{Sd} \cdot e_{tot}$,

$$k_2 \approx \frac{\alpha \cdot f_{cd} \cdot A_{c,col} + f_{yd} \cdot A_s - N_{Sd}}{\alpha \cdot f_{cd} \cdot A_{c,col} + f_{yd} \cdot A_s - 0{,}4 \cdot f_{cd} \cdot A_{c,col}} \le 1.$$

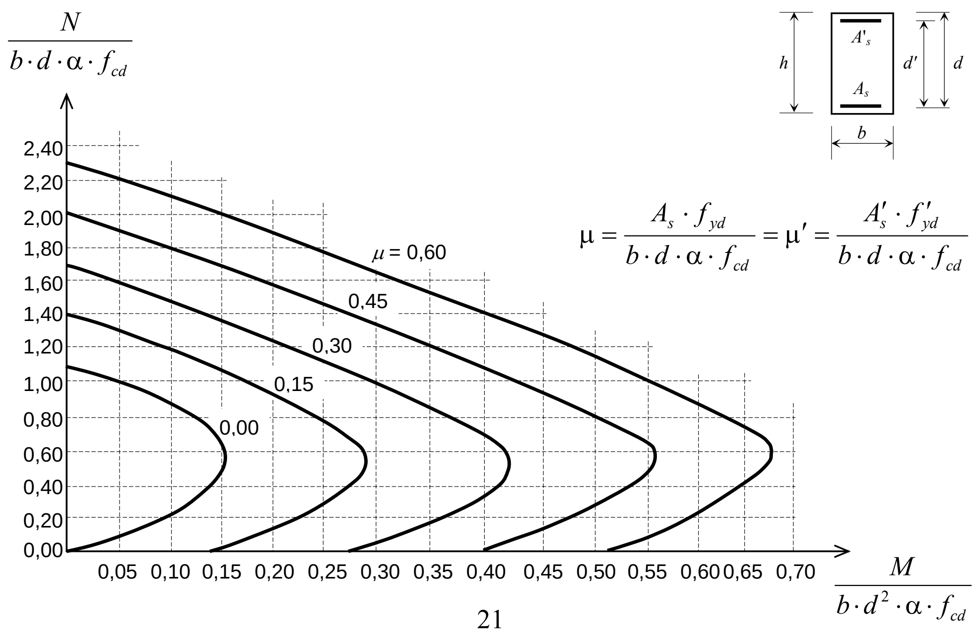
3.6.4 Control of column section

The total eccentricity in the section in plane *x-z* and *y-z* should be determined and the section should be analysed for biaxial eccentric compression with the eccentricities of $e_{tot,x}$ és $e_{tot,y}$. The initial eccentricity of the column in direction x should be zero as the building is braced in that direction. But in that case the eccentricity due to inaccuracy and the second order eccentricity in that direction should be calculated. The control of the section may be performed with the simplified spherical interaction diagram of M-N. Using the interaction diagram (see below) the ultimate moment of resistant in direction *x* and *y* ($M_{Rd,x}$ and $M_{Rd,y}$) may be obtained, respectively.

The section is sufficient if:

$$\frac{M_{Sd,x}}{M_{Rd,x}} + \frac{M_{Sd,y}}{M_{Rd,y}} \le 1$$

where $M_{Sd,x} = N_{Sd} \cdot e_{tot,x}$ and $M_{Sd,y} = N_{Sd} \cdot e_{tot,y}$.



Interaction diagram for rectangular section with symmetrical reinforcement

Designing the main bars of the reinforcement details should be taken into consideration. Here are the most important rules:

- the smallest dimension of the column is 200 mm, and the longer side can not be bigger than four times the smallest,
- bars should be placed in the corners of the column,
- the smallest diameter of the main bars is 12 mm, the distances between the bars are the same as in case of beams,
- the minimal area of steel required:

  $$A_{s,\min} = \max\begin{cases} 0{,}15 \cdot N_{Sd} / f_{yd} \\ 0{,}003 \cdot A_c \end{cases}$$ where A_c is the area of concrete section,
- at the overlapping the area of reinforcement must not be bigger than the 8 % of the section area .

The rules of shear reinforcement are the same as in case of beams. But take into consideration the rules listed below:

- The minimal diameter of the links is the bigger of either 6mm or the one quarter of the main bar diameter.
- The spacing between the stirrups may not be bigger than:

  $$s_{\min} = \min\begin{cases} 12 \cdot \varnothing \\ \text{smallest dimension of the column} \\ 300\text{ mm} \end{cases}$$ where $\varnothing$ is diam. of the smallest main bar

  The spacing above should be multiplied by 0,6 in case of:
  - overlapping, if the main bar diameter is at least 14 mm,
  - where a beam or slab is connected to the column along a length of the biggest dimension of the column.