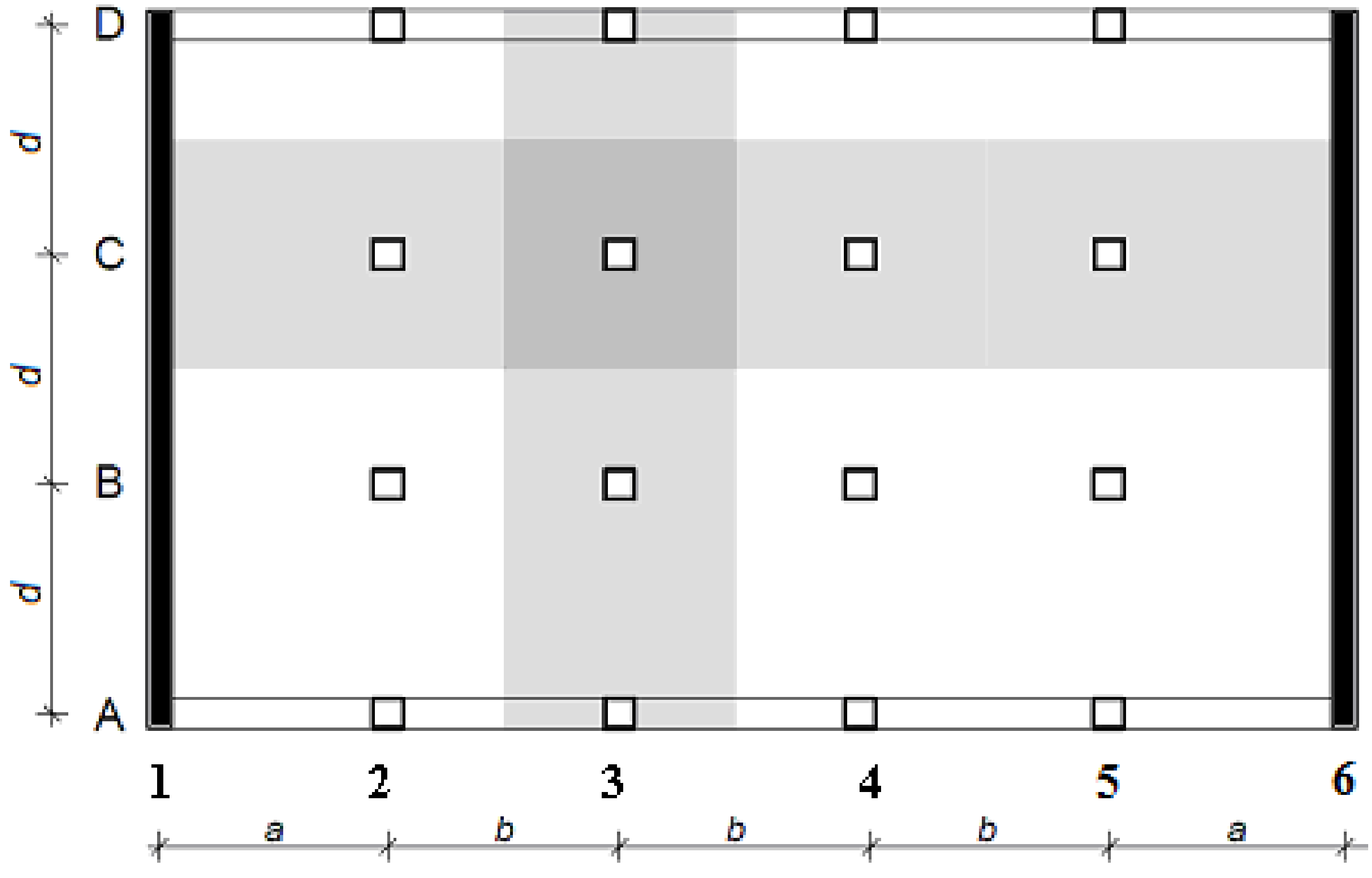
3rd task:
Two-way slab supported by columns (flat slab)



Our goal in 3rd task will be to:

- Design **dimensions** of all elements
- Perform detailed calculation of **bending moments** using „direct design method"

This week

Next week

- Design **bending reinforcement** of the slab
- Design **punching reinforcement** of the slab
- Draw **layout** of reinforcement

Design of dimensions – steps

- Depth of the **slab** – see following slides
- **Column** dimensions – see presentation from the 1st seminar, no differences
- Dimensions of **wall and edge beam** – see the assignment of your task
- Preliminary **check of punching**
- **Sketch of the structure** – redraw the plan from the assignment with **your** dimensions (given+designed ones) – as in the 1st task

Depth of the slab h_s

- Empirical estimation: $$h_s = \frac{1}{33} l_{n,max}$$

The longest clear span

- Effective depth d: $$d = h_s - c - \frac{\varnothing}{2}$$

Diameter of steel bars, estimate 10 mm

Cover depth, take the value from 1st task

- Span/depth ratio (deflection control):

$$\lambda = \frac{l}{d} \leq \lambda_{lim} = \kappa_{c1}\kappa_{c2}\kappa_{c3}\lambda_{d,tab}$$

Longer one of the axial spans l_x, l_y

Effect of shape 1.0

Effect of span 1.0

Effect of reinforcement 1.2

Depth of the slab h_s

$\lambda_{d,tab}$ for flat slabs

	Concrete class						
ρ %	12/15	16/20	20/25	25/30	30/37	40/50	50/60
0,5	17,5	19,0	20,4	22,2	24,6	30,9	38,4
1,5	14,6	15,1	15,6	16,2	16,8	18,0	19,2

- If span/depth ratio is not checked, increase the empirical h_s

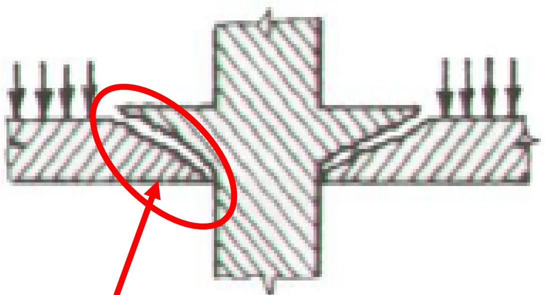
- Do not design h_s < 200 mm – you can´t use punching reinforcement for very thin slabs

Depth of the slab h_s

- After slab depth design, calculate the **total load** of the slab f_d (in a table). Self-weight is given by h_s, other loads are the same as in 1st task.

What is punching?

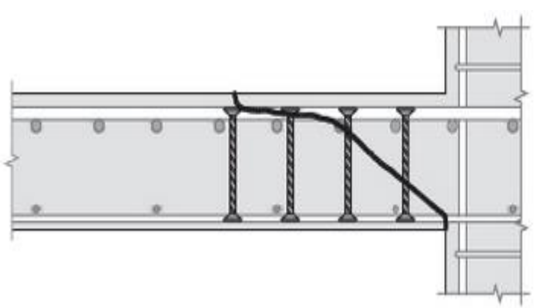
- A mode of shear failure of flat slabs
- No beams => load from large area of a slab is transferred directly to a column through small area of a joint => concentrated stresses => possible failure



Concentration of loads => failure

Piper's Row Car Park, Wolverhampton, UK, 1997 (built in 1965).

Reinforcement: Double-headed studs



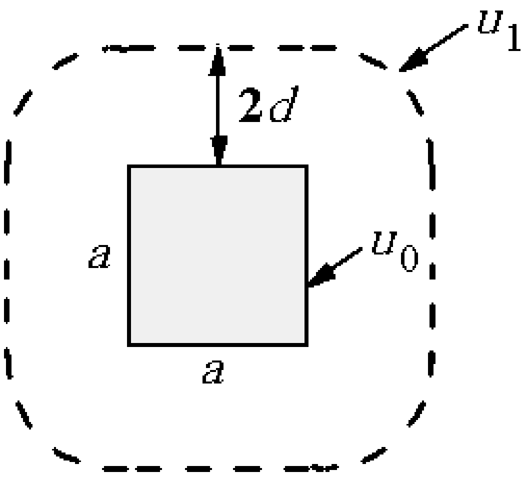
Preliminary check of punching

- Before designing the reinforcement, we have to check if the **structure is „suitable" for the design of punching shear reinforcement**

- „Non-suitable" structure will always fail, no matter how much reinforcement is provided !!!

Preliminary check of punching

- Control perimeters:



$$u_0 = 4a$$

$$u_1 = 4a + 2\pi \cdot 2d$$

Effective depth of your slab

Maximum punching shear resistance

- *Is the resistance of compressed concrete sufficient?*
- Check in **perimeter u_0**
- Stress values are in MPa

Maximum punching shear resistance

Stress in perimeter u_0

$$v_{\mathrm{Ed},0} = \frac{\beta V_{\mathrm{Ed}}}{u_0 d} \leq v_{\mathrm{Rd,max}} = 0.4 \nu f_{\mathrm{cd}}$$

Coefficient expressing position of the column, for inner columns $\beta = 1.15$

Coefficient expressing effect of shear on compressive strength

Coefficient expressing effect of additional stresses

$$\nu = 0,6\left(1 - \frac{f_{\mathrm{ck}}}{250}\right)$$

Shear force, equal to normal force in the column from **<u>ONE</u>** floor (do <u>not</u> sum the forces from all floors!!!)

Max. resistance with reinforcement

- *Is it possible to anchor the punching reinforcement in concrete sufficiently?*
- Check in **perimeter u_1**

$$v_{\mathrm{Ed,1}} = \frac{\beta V_{\mathrm{Ed}}}{u_1 d} \le k_{\max} \cdot v_{\mathrm{Rd,c}} = k_{\max} \cdot C_{\mathrm{Rd,c}} \cdot k \cdot \sqrt[3]{\left(100 \rho_l \cdot f_{\mathrm{ck}}\right)}$$

Coefficient of maximum resistance, see table

Reduction factor, 0.12

Effect of depth

$$k = 1 + \sqrt{\frac{200}{d}} \le 2{,}0$$

Reinforcement ratio of tensile reinforcement, estimation 0.005

S T I R R U P S	**effective depth of the slab**	**k_{max}**
	$d \le 200$ mm	1,45
	200 mm $\le d \le$ 700 mm	interpolation
	$d \ge 700$ mm	1,70
Double-headed studs connected to a spacer bar		1,80

Preliminary check of punching

- If any of the conditions is not met, it is not possible to design shear reinforcement
  - 1st condition (u0) not met –> the structure will fail due to **crushing of concrete**, no matter how much reinforcement you provide
  - 2nd condition (u01 not met –> the reinforcement will not be **anchored** in concrete sufficiently => it will be useless

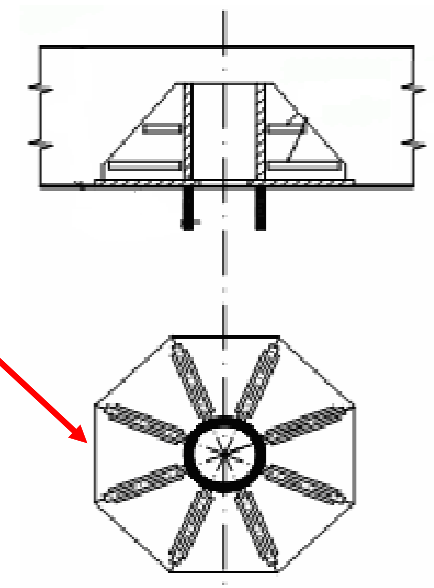
=> You have to **redesign the structure**

Redesigning – possibilities

- Increase **depth of the slab** – not effective, load is increased at the same time
- Increase **dimension of the column** – effective, but floor area is decreased
- Increase **concrete strength** – expensive
- Design a **slab with drops or flat beams** – complicated

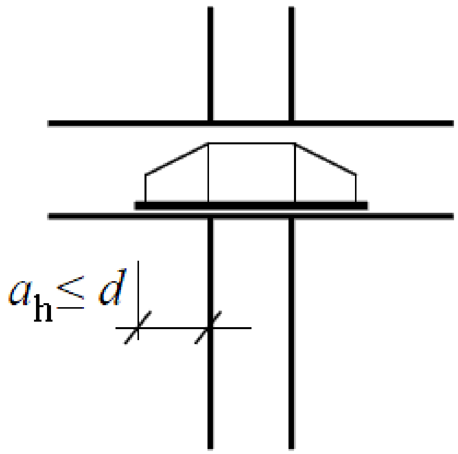
Redesigning – possibilities

- **Design columns with caps –**
  use steel flanged caps/collars (welded steel details that are put into the slab-column joint)



Redesigning

- The **collar increases u_0 and u_1** => stresses in the control perimeters are decreased
- Resistances are not changed



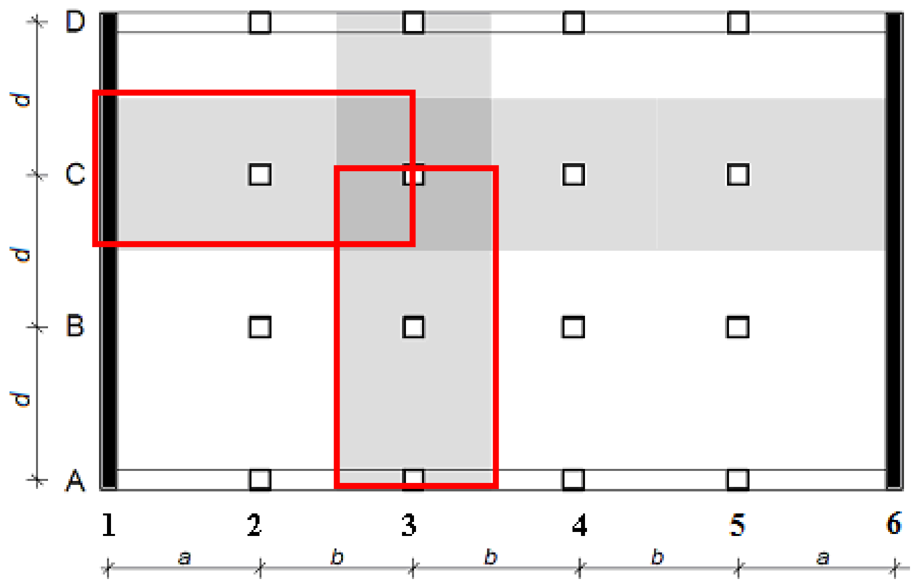
$$u_0 = 4a + 8a_h$$
$$u_1 = 4a + 8a_h + 2\pi \cdot 2d$$

$\Rightarrow$ Recalculate the two conditions with new values

$\Rightarrow$ If it does not help, either increase h_s or dimension of the column and recalculate the conditions

Calculation of bending moments

- Use **direct design method (DDM)**
- Analyze one belt in longitudinal direction, one belt in transverse direction (grey belts in the assignment)
- For each belt, analyze the **outer panel** and the adjacent **inner panel**

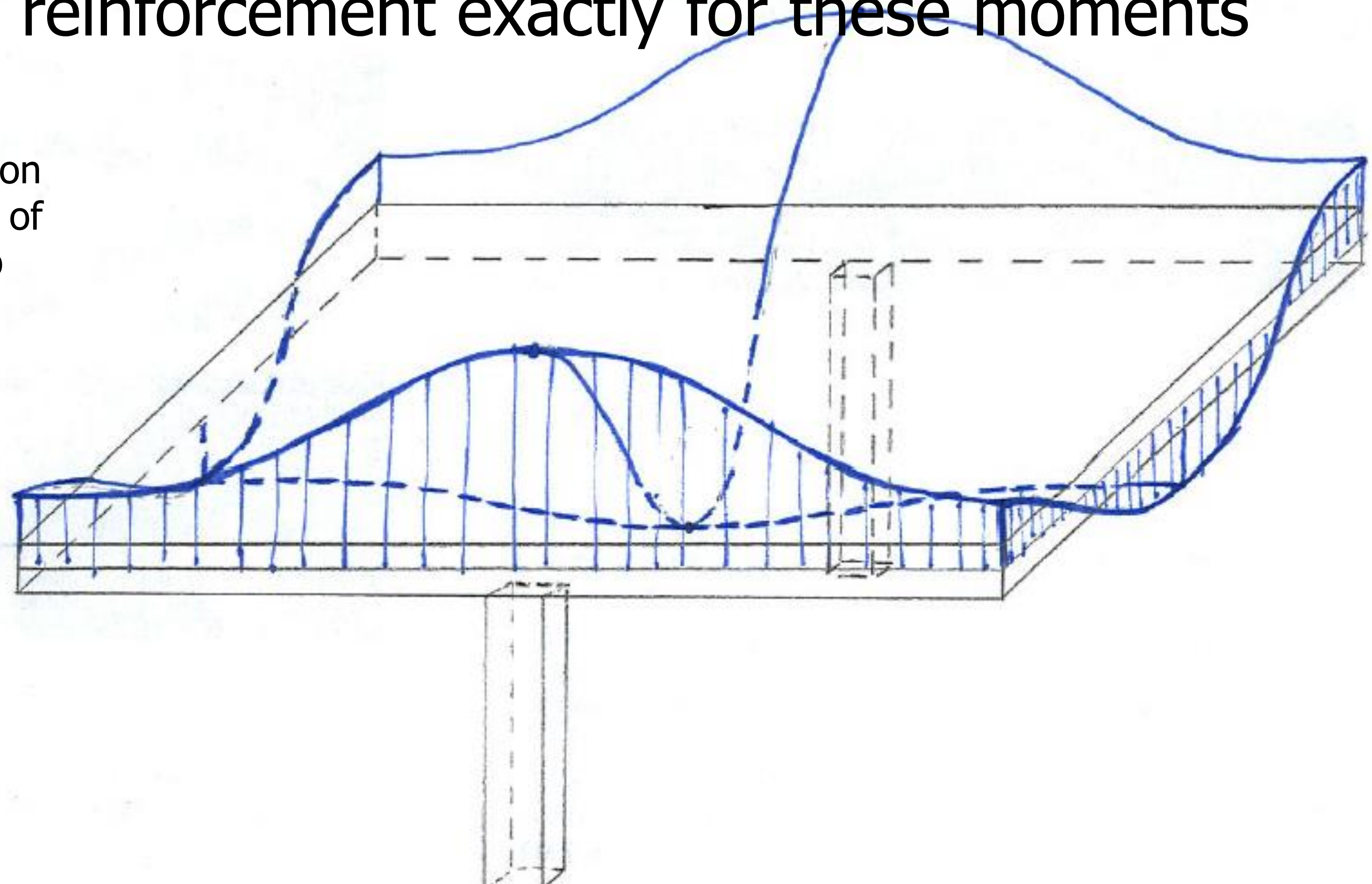


DDM - background

Real distribution of bending moments: 2D curve

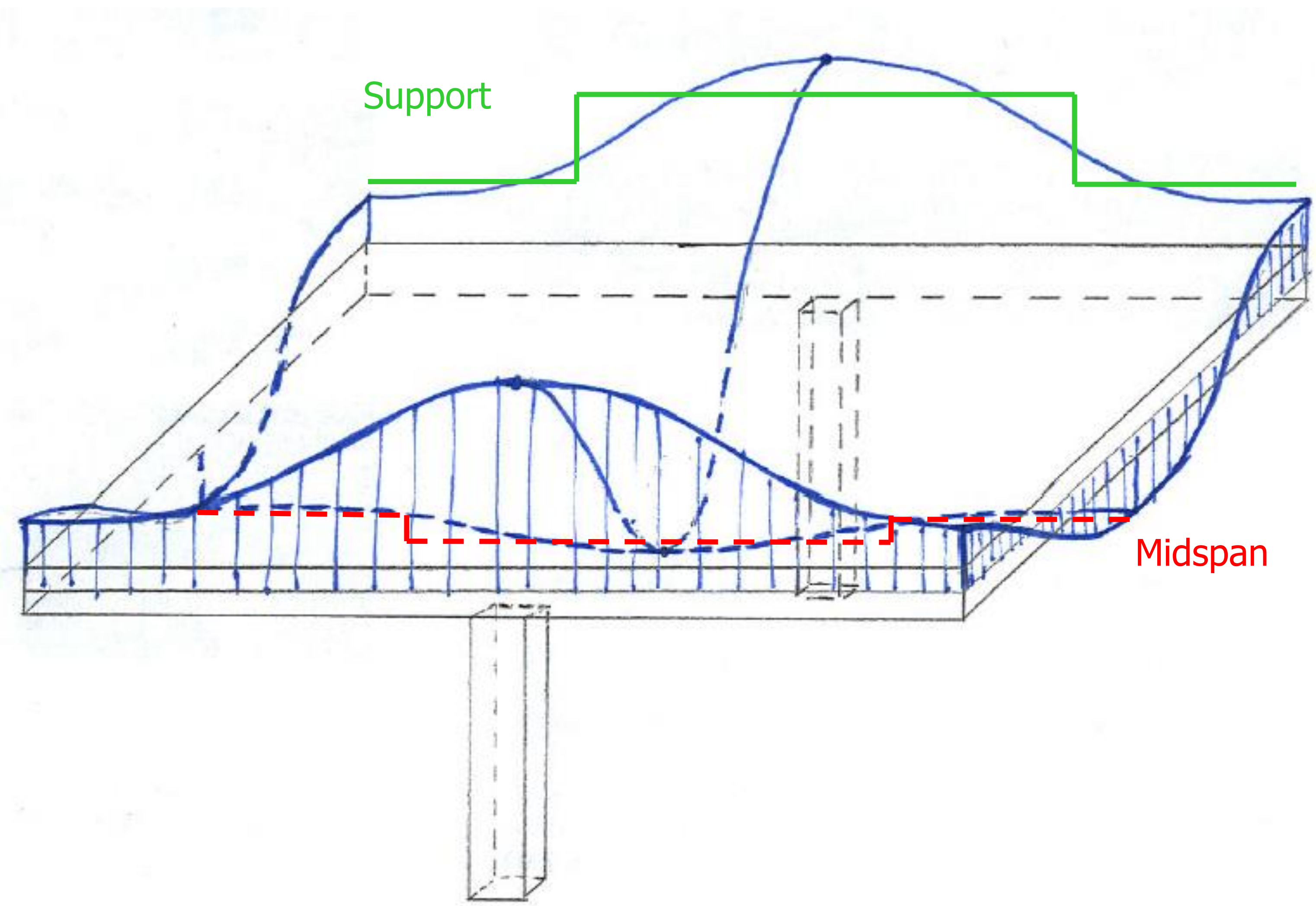
From practical point of view, it is impossible to provide reinforcement exactly for these moments

Moments on one panel of the slab



DDM - background

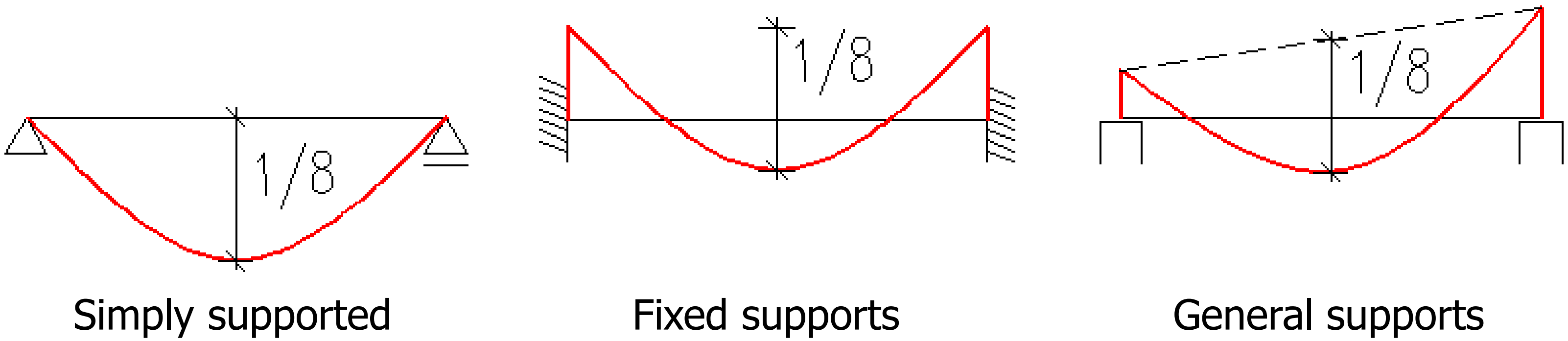
=> We need to calculate „**representative moments**“ for particlar areas of the slab



DDM - background

- For all types of panels, the total moment is:

$$1/8 * load * width * span^2$$

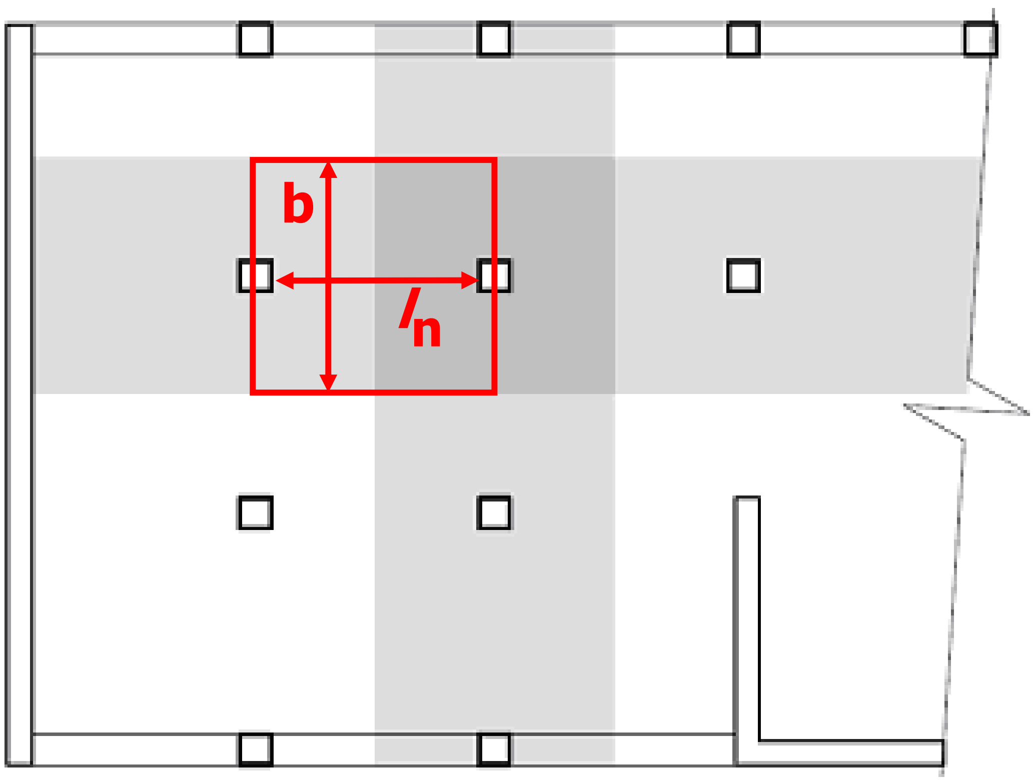


- For regular slabs, we are able to **divide the total moment into support/mid-span** moments using precalculated coefficients

Step 1: Total moment

- The total moment of a panel is:

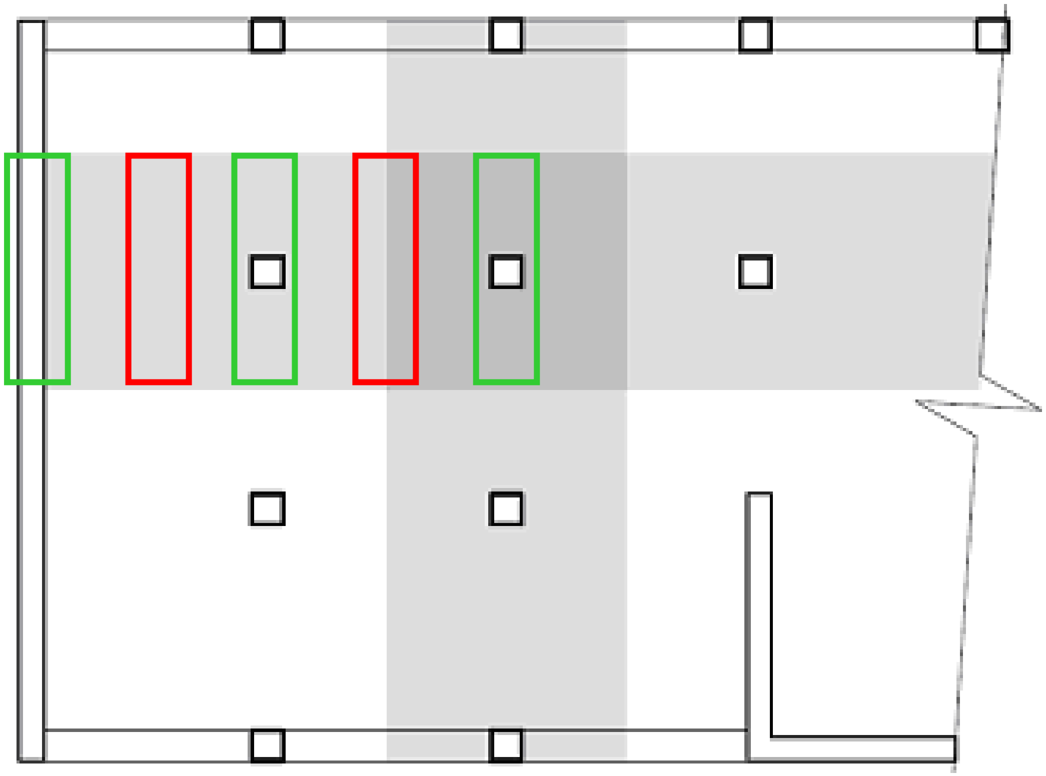
$$M_{tot} = \frac{1}{8} f_d b l_n^2$$



(The same applies to other panels as well)

Step 2: Total Positive/Negative M

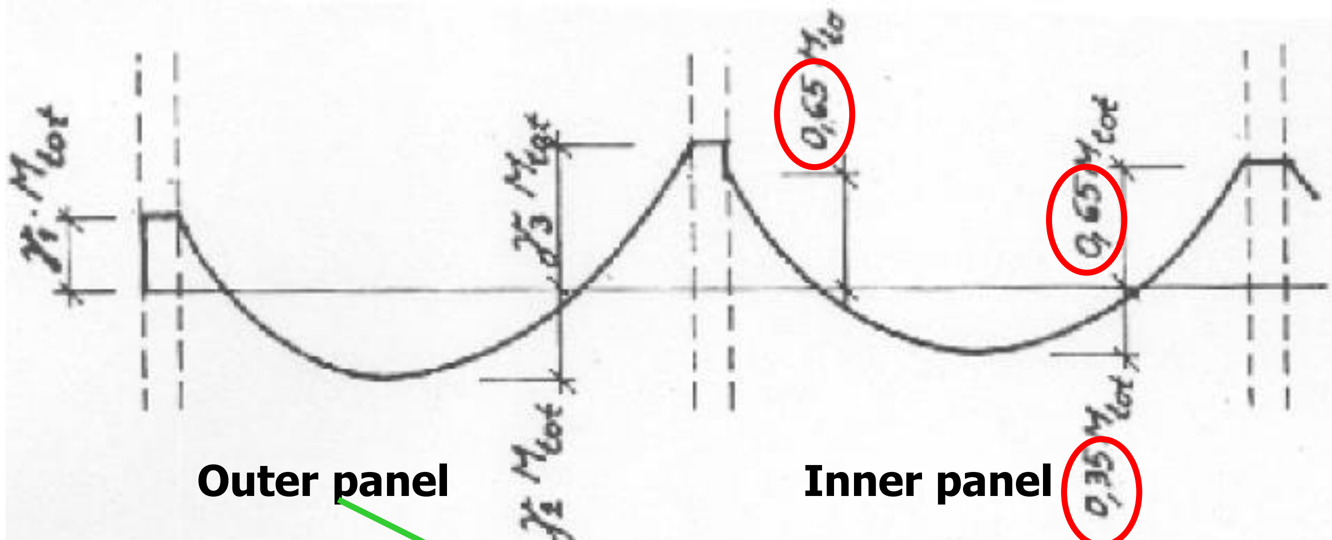
- Using γ coefficients, we divide total moment into total positive (midspan)/negative (support) moments in each panel



(The same applies to the second direction as well)

Step 2: Total Positive/Negative M

- γ coefficients:



	Edge of the slab is freely supported on wall	Slab has stiffening beams in all column strips	Slab has not stiffening beams in column strip and		Edge of the slab is fixed
			has not an edge beam	has an edge beam	
γ_1	0,00	0,16	0,26	0,30	0,65
γ_2	0,63	0,57	0,52	0,50	0,35
γ_3	0,75	0,70	0,70	0,70	0,65

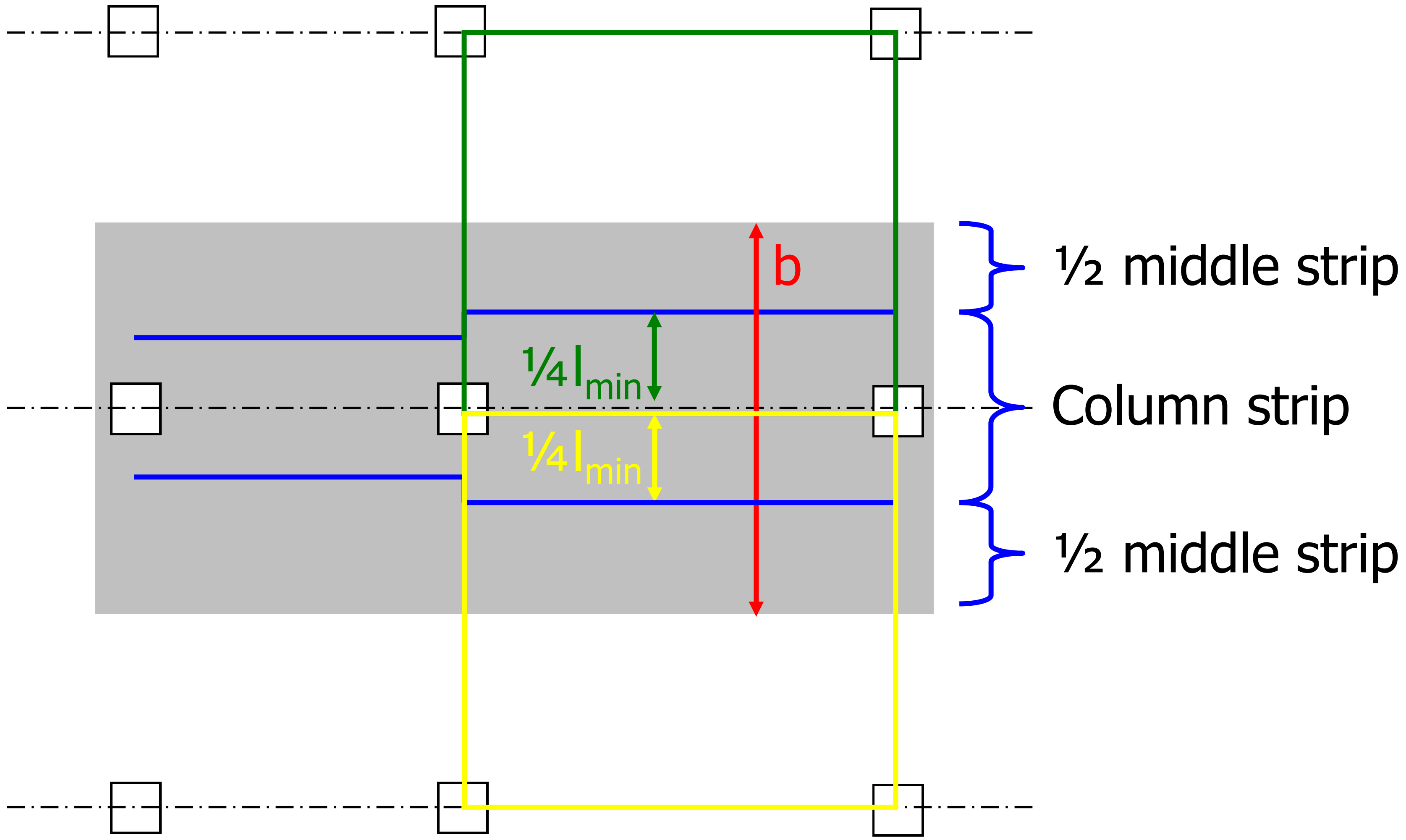
Step 3: Column and middle strips

- We have to **divide the belts into column strips** (more loaded) **and middle strips** (less loaded)

- The width of the **column strip is ¼ of shorter span** of adjacent panel to each side from the axis
- The **width of the middle strip is the rest** of the width of the belt

- The width of column strip does not have to be the same in all the panels!!!

Step 3: Column and middle strips



(The same applies to the second direction as well)

Step 4: Moments in col./mid. strips

- Using ω coefficients, we **divide total positive/negative moments** into **moments in column/middle strips**

- The moment in **column** strip is:

  ω * (total positive or negative moment)

- The moment in **middle** strip is:

  (1-ω) * (total positive or negative moment)

Step 4: Moments in col./mid. strips

- ω coefficients:

Moment		$\alpha_1 l_2 / l_1$		ω for l_2/l_1 0,5	1,00	2,00
Negative	Outer sup-port	$\alpha_1 l_2 / l_1 = 0$	$\beta_t = 0$	1,00	1,00	1,00
			$\beta_t \geq 2,5$	0,75	0,75	0,75
		$\alpha_1 l_2 / l_1 \geq 1,0$	$\beta_t = 0$	1,00	1,00	1,00
			$\beta_t \geq 2,5$	0,90	0,75	0,45
	Inner sup-port	$\alpha_1 l_2 / l_1 = 0$		0,75	0,75	0,75
		$\alpha_1 l_2 / l_1 \geq 1,0$		0,90	0,75	0,45
Positive		$\alpha_1 l_2 / l_1 = 0$		0,60	0,60	0,60
		$\alpha_1 l_2 / l_1 \geq 1,0$		0,90	0,75	0,45

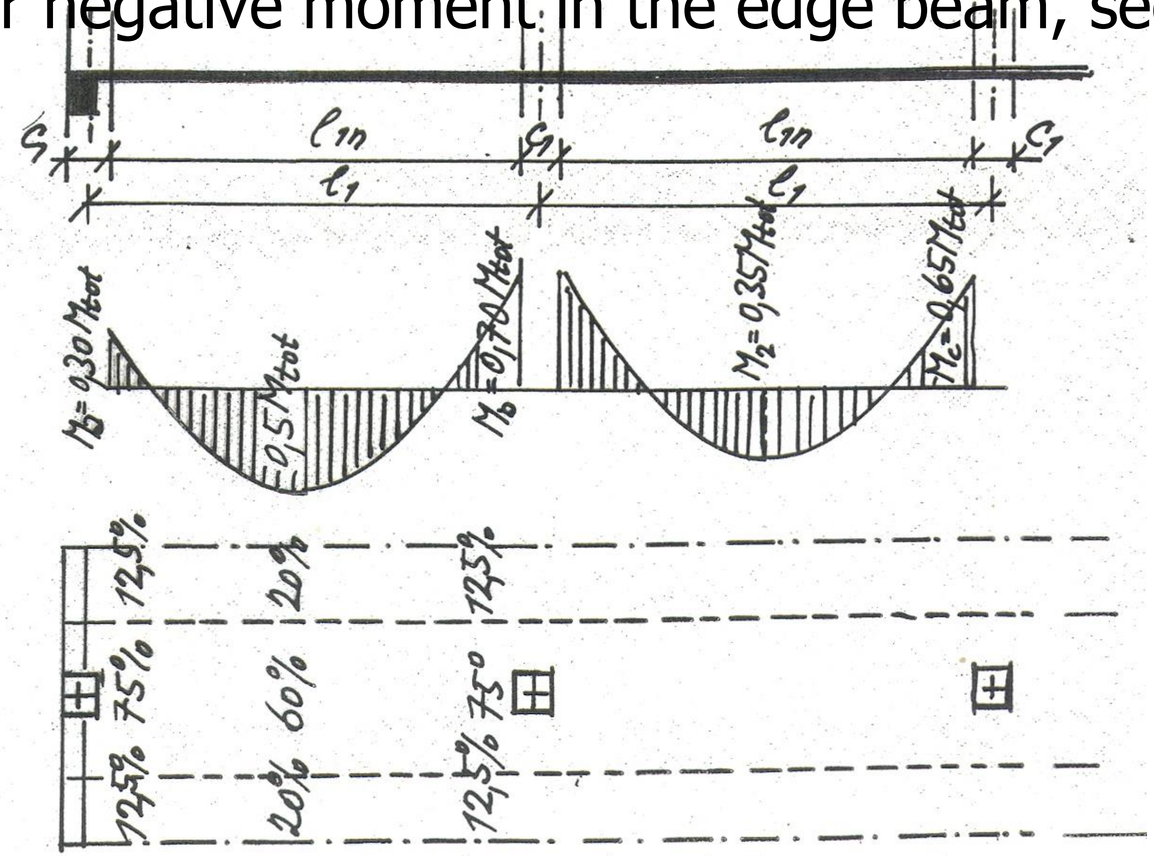
$\alpha = 0$

α = effect of longitudinal stiffening, we don´t have any stiffening

β – refers to rigidity of edge beam

Step 4: Moments in col./mid. strips

- ω coefficients in our case:
  - For all positive moments (**midspans**), ω = **0,6**
  - For all negative moments **above columns**, ω = **0,75**
  - For negative moment **above the wall**, ω = **1,00**
  - For negative moment in the edge beam, see next page



Step 5: Rigidity of edge beam

- Rigidity coefficient of edge beam is:

$$\beta_t = \frac{I_t}{2I_s}$$

- I_s is moment of inertia of the slab in belt 3:

$$I_s = \frac{1}{12} b h_s^3$$

Width of belt 3

Depth of the slab

- I_t is torsion moment of inertia of edge beam:

$$I_t = \sum_{i=1}^{n} \left(1 - 0,63 \frac{t_i}{a_i}\right) \cdot \frac{t_i^3 a_i}{3}$$

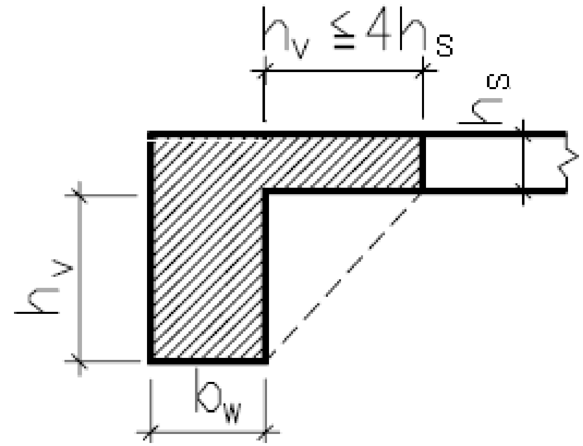
You have to sum torsion moments of all parts of the cross-section

Longer side of a part of the cross-section

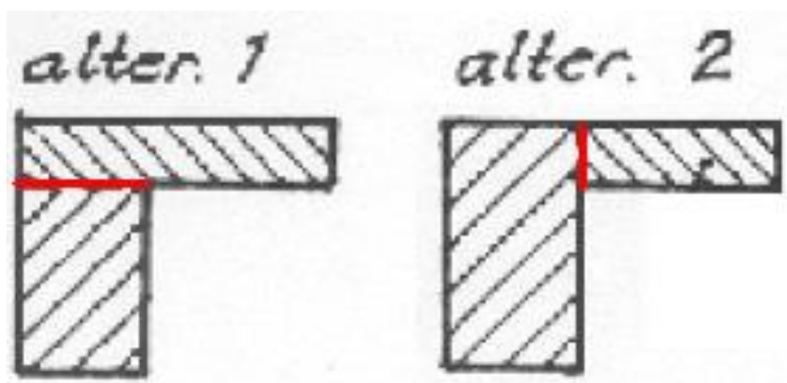
Shorter side of a part of the cross-section

Step 5: Rigidity of edge beam

- Cross-section of edge beam



- Dividing into parts – 2 alternatives:



- Calculate I_t for both alternatives and use the higher value to calculate β_t

Step 5: Rigidity of edge beam

- Interpolate ω according to the value of β_t
  - $\beta_t = 0$ => edge beam has no influence, the edge behaves as free edge without beam
  - $\beta_t \geq 2,5$ => edge beam is so rigid that the edge behaves as a fixed edge
  - **$0 < \beta_t < 2,5$ => something between the two boundary cases**

$\beta_t = 0$ 1,00

$\beta_t = 0,131$ ..... ω

$\beta_t = 2,5$ 0,75

$$\frac{2,5 - 0,131}{2,5 - 0} = \frac{0,75 - \omega}{0,75 - 1,00}$$

$$\underline{\omega = 0,986}$$

Step 6: Moments per 1 meter

- Up to this step, all the moments were in kNm.

- Divide the calculated moments in column/middle strips [kNm] **by the width of column/middle strip** to receive moments per 1 meter of the slab [knm/m].

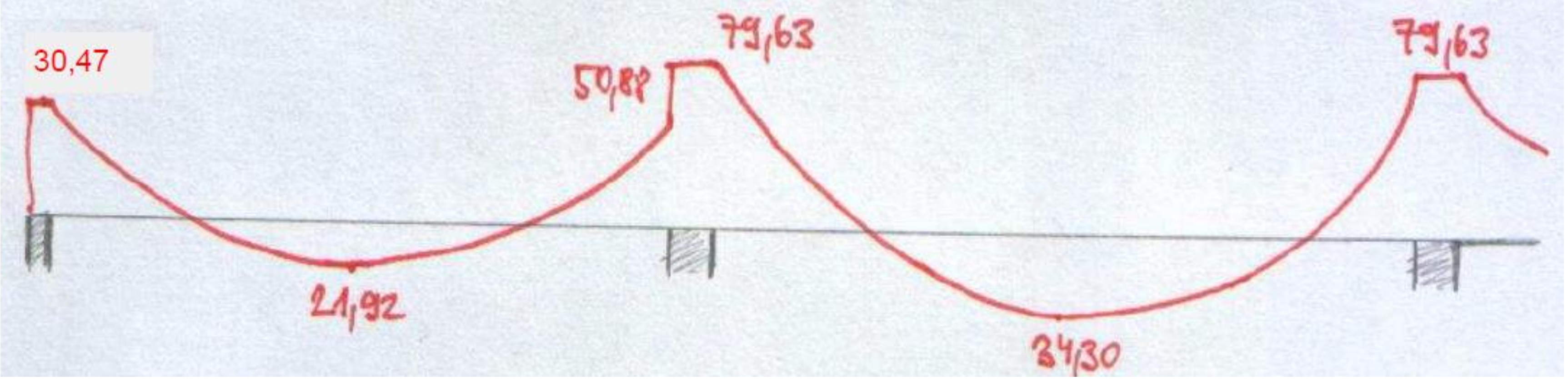
- The new moments are in kNm/m!

Step 6: Moments per 1 meter

- **For the wall**, we don´t distinguish between column/middle strip – total negative moment is divided by total width of the belt

Step 7: Moment curves

- Draw moment curves for calculated moments per 1 meter
- There will be 4 curves
  - Belt C – column strip
  - Belt C – middle strip
  - Belt 3 – column strip
  - Belt 3 – middle strip



Conclusions

- Steps 1,2,3,5,7 will be done manually
- Steps 4,6 will be calculated in a table

Moments in column and middle strips							
Panel	Cross-section	Positive/negative moment M_i [kNm]	Strip	ω	Moment in column/middle strip M_j [kNm]	Width of the strip s_j [m]	Moment per 1 m of the slab m_j [kNm/m]
C_o	1 (left support)	179,76	no division	1,00	179,76	5,90	30,47
	2 (midspan)	96,80	Column	0,60	58,08	2,65	21,92
			Middle		38,72	3,25	11,91
	3 (right support)	179,76	Column	0,75	134,82	2,65	50,88
			Middle		44,94	3,25	13,83
C_{in}	1 (left support)	313,20	Column	0,75	234,90	2,95	79,63
			Middle		78,30	2,95	26,54
	2 (midspan)	168,66	Column	0,60	101,20	2,95	34,30
			Middle		67,46	2,95	22,87
	3 (right support)	313,20	Column	0,75	234,90	2,95	79,63
			Middle		78,30	2,95	26,54
3_o	1 (left support)	119,11	Column	0,99	117,44	2,95	39,81
			Middle		1,67	4,05	0,41
	2 (midspan)	198,50	Column	0,60	119,10	2,95	40,37
			Middle		79,40	4,05	19,60
	3 (right support)	277,92	Column	0,75	208,44	2,95	70,66
			Middle		69,48	4,05	17,16
3_{in}	1 (left support)	258,07	Column	0,75	193,55	2,95	65,61
			Middle		64,52	4,05	15,93
	2 (midspan)	138,96	Column	0,60	83,38	2,95	28,26
			Middle		55,58	4,05	13,72
	3 (right support)	258,07	Column	0,75	193,55	2,95	65,61
			Middle		64,52	4,05	15,93

- **See an example on my webpage!**