CZECH TECHNICAL UNIVERSITY IN PRAGUE

FACULTY OF CIVIL ENGINEERING

Department of Concrete and Masonry Structures

Diploma Thesis

DESIGN METHODS FOR FLAT SLABS SUBJECTED TO PUNCHING SHEAR IN CONCRETE STRUCTURES

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BACHELOR´S THESIS ASSIGNMENT FORM

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Bachelor Thesis (BT) title: Metody navrhování lokálně podpřené desky na protlačení

Bachelor Thesis title in English: Design methods for flat slabs subjected to punching shear

Instructions for writing the thesis:

The thesis wil include the summary of fundamental design methods for flat slabs subjected to punching shear including the description of shear reinforcement used for this type of action. Then, the punching shear resistance of a reinforced concrete slab will be calculated by using selected methods and obtained results will be evaluated.

List of recommended literature:

EC 1992-1-1 , BS 8110-1, ACI-318, fib Model code 2010, Design of Concrete Structures (Arthur H. Nilson, David Darwin, Charles W. Dolan)

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III. ASSIGNMENT RECEIPT

I declare that I am obliged to write the Bachelor Thesis on my own, without anyone's assistance, except for provided consultations. The list of references, other sources and consultants' names must be stated in the Bachelor Thesis and in referencing I must abide by the CTU methodological manual "How to Write University Final Theses" and the CTU methodological instruction "On the Observation of Ethical Principles in the Preparation of University Final Theses".

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AUTHORSHIP STATEMENT

I hereby declare that this Bachelor thesis was written based on my own original research and that I conducted it independently by myself. To the best of my knowledge, this thesis does not contain material or sources previously published in the exact same form, other than referenced sources listed in the bibliography.

In……………...On……………… Signature…………………

ACKNOWLEDGEMENT

I am pleased to thank everyone who advised, guided, directed, or contributed with me in preparing this thesis by referring to the required references and resources at any stage. I especially thank my honorable supervisor Ing. Josef Novak, Ph.D. for his support and guidance for advice and correction.

ABSTRACT

The thesis present simplified summary of different principles for design methods of flat slabs subjected to punching shear mechanism in structural members and various types of shear reinforcement applied for that matter as well as punching shear resistance of reinforced concrete slabs with description of calculation performed will be encountered by using selected methods and obtained results will be evaluated and analyzed.

Keywords: flat slab,punching shear reinforcement,design analysis,comparison of results.

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THEORY

1 Introduction

The aim of the thesis is to review different types of punching shear reinforcement used in construction and to focus on design procedure for punching shear with EC2,ACI-318 and FIB MODEL 2010 standards.

Theoretical findings about punching shear in flat slabs were gathered from technical literature, studies and research, a model of selected building was created in SCIA Engineer software and a specified slab analyzed by using finite element method to obtained internal forces on a flat slab, the results were taken to design flexural reinforcements. Punching shear resistance of slab checked for possible punching failure and punching shear reinforcement designed with two different standards EC2,ACI-318 , both approaches were examined and compared.

1.1 Motivation

Flat slabs are commonly used nowadays as they are highly adaptable elements, its fast and cheap construction using simple formwork and particularly suitable for areas where for acoustic or fire reasons , supported directly by concrete columns without the use of beam as well as providing minimum depth allowing for greater roof heights and lighter floors, but this does not prevent some defects in this type of slabs such as punching shear failure which can be developed in a brittle manner nearly with no warning sign, furthermore the remaining strength following the punching is remarkably lower than the punching load, thus punching of a single column of a flat slab overcharge adjacent columns and potentially conduct to their failure on punching, historically in the past , a number of collapses have been reported in the world due to punching shear failure causes human casualties and huge damages, however with today's technologies and experience in construction and other fields it become possible to prevent painful accident and disasters and as being student in civil engineering university I would like to highlight more of this issue and introducing computer analysis and methods of modern technology uses in our time.

1.2 Goals

The wide spread of flat slab in construction induced me to investigate to the design and review the cons and pros of the structure based on several research and analysis. My goal was to gain in-depth understanding design procedure for punching shear, specifically design a model and check punching shear resistance for selected slab also sharing the information of design punching with overview of EC-2,ACI318,FIB MODEL 2010..

2 State of the art

2.1 Flat slabs

Flat slab is a two-way slab supported directly on columns or caps without beams where loads are transferred straight to columns (Fig. 1) [1], it is a device to reduce stress due to shear and negative bending around the column, the benefits of choosing flat slabs include a minimum depth solution, speed of construction and flexibility in the plan layout.

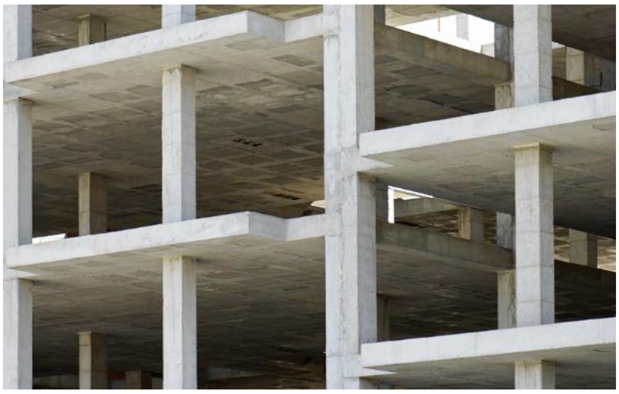



Figure 1: Flat slab without drop and column head (left) and flat slab with drops (right)

There are four types of flat slabs commonly used in construction [2]:

Typical flat slab

This type of construction is built without cap or drops and are mostly used in warehouses, offices and public halls, sometimes beams are avoided in the edge of slab, and slabs are directly supported by columns, (Fig. 2) [3].


Figure 2: Reinforced concrete flat plate system, Eugene, USA, (left) and a reinforced concrete building with flat plate system under construction, Kenya (right)

Slab with column head and without drop

The column head is an extra part which is placed on top of column and in the same times enlarged or widened in a way to enhance the punching shear resistance of the slab (Fig. 3) [4], It provides additional base area which serves as a base to other structural member, different angles of the column head can be presented from an architectural point of view, however for the design part of concrete structural engineers found out that the angle of 45° on all surrounding column head is reviewed as easier, effective and practical for the design.

Figure 3: Reinforced concrete flat slab with a column capital, London, UK (left) and concrete flat slab with head, Vancouver, Canada (right)

Slab with drop and column without column head

Drop panels enhance flat slabs negative moment capacity and decrease deflection by stiffening the slab as well as increase the shear strength, column drops are commonly either square or rectangular in profile. (Fig. 4) [5].

Figure 4: Reinforced concrete flat slab with drops

Slab with drop and column with column head

This is the combination of both drop and head column, for heavy loaded buildings where the load is excessively high therefore will result a significant increase of shear stress, , a sufficient depth around the column must be added in order to disallow the shear stress from creating punching and if the drop panel or column head depth considerably high , which is ostensibly unpleasant to see, it is fitter to design column head and drop together as well as for safety reasons (Fig. 5) [6].

Figure 5: Reinforced concrete flat slab with drop and column head

2.2 Punching shear design

In flat slabs punching can result from concentrated loads transmitted within a relatively small area, the zone surrounding the column region becomes extremely critical due to shear stress which may cause eventually punching failure for the slab (Fig. 6) [7], this failure threatens the hole stability of the structure ,where the slab is subjected to concentrated loads that exceeds its resistance capacity, in the end the slab will fail, therefore it's important to take into account all precautions while designing and to ensure to put the necessary materials and element during construction .

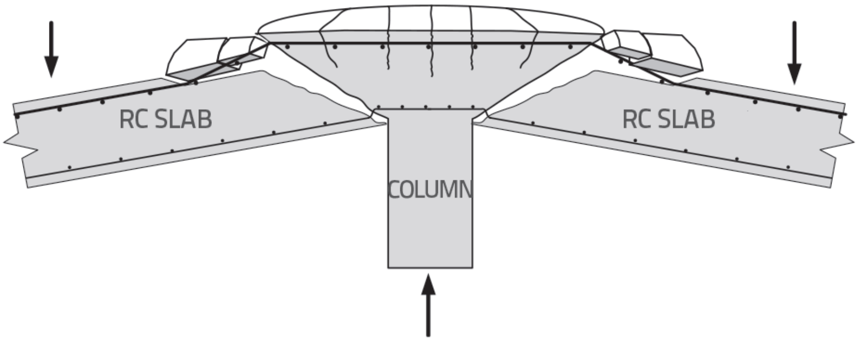


Figure 6: punching failure in the slab column connection area

It is very noticeable if the punching failure occurs, then the slab and column will have a physical disconnection from each other, this will result a significant disruption of the equilibrium for structure elements. Afterwards the fall of the hole structure may be induced because of load redistribution to other members that not designed to carry out such a new load, and since that concrete is brittle fracture material the punching failure happen all at once.

In order to prevent punching failure its necessary to reinforce the slab region close to column area by different reinforce arrangements, modern analysis declare that bent or straight reinforcement bars should be inserted in bottom part of the slab to avoid continuous construction breakdown producing from exceptional actions, even if the design computations demonstrate that punching shear reinforcement is not required, see (Fig. 7) [8].

Figure 7: Upper floor collapse of Wolverhampton parking (UK)

2.2.1 Design methods

Utilization of certain codes of practice is widespread in the world, studies and many researches provides a lots of information linked with the structural design of reinforced concrete flat slab which is based on different design methods and codes that provides different results in structural analysis, farther lead to variability in behavior of flat slabs. what differs one country code from another is the level of accuracy, safety, complexity, and details, so its obligation for structural engineers to provide accurate standards that result in better economy and performance.

Owing to the wide use of flat slabs in the structural engineering, punching shear behavior was analyzed by numerous researches over the years, as result of these experimental and practical investigation various approaches and methods were derived for the determination of punching shear resistance, Some of them are listed directly in standards (Eurocode 2, ACI 318, BS 8110-1) and other in technical literature (FIB Model code 2010). They provide recommendation for designing construction, assumptions, load safety factors, ductility, cross-sectional moment capacity and so on, furthermore they present various types of methods to carry out analysis and Adopt a suitable tool for checking the mechanical resistance of components, or checking the stability of structures.

The methodology for assessing the punching shear resistance calculations according to codes of practice of several countries involves examining a series of parameters, each located at fixed distances from the column face, at each individual perimeter the shearing stresses due to the punching is calculated and compared to an allowable shear stress, these perimeters are referred to as control perimeters denoted by letters (u or b) while this calculation can be performed for perimeters at any given distance from the column face, there are couple of parameters which are of critical importance and must be assessed, the following sections presents formulas available of the punching shear resistance design according three different codes based on various geometric and material parameters.

2.2.1.1 Design of punching shear resistance according to Eurocode [9]:

Based on Eurocode 2 provisions the calculations of punching shear resistance are evaluated and presented in this section; more detailed description can be taken from EC2 [9].

For punching shear design in flat slabs, it is important to check the so-called critical perimeter. The control perimeter is located 2.d (d = effective height of the plate) from the loaded region (Fig.8), the punching shear stress V_{Ed} is calculated along the defined control perimeter around the loaded area using the following expression:

$$v_{Ed} = \beta \,.\, \frac{V_{Ed}}{u_1 \,.\, d} \qquad (1)$$

u_1 - *circumference of column critical perimeter*

β - *increasing factor*

V_{Ed} - *design value of the punching load*

d - *mean effective depth of slab taken as (d_y + d_z)/2; d_y and d_z are effective depths in directions of the control sections*

Estimated values of factor β may be applied for fixed structures with span variations in the neighboring areas (Fig. 9) of less than 25%:

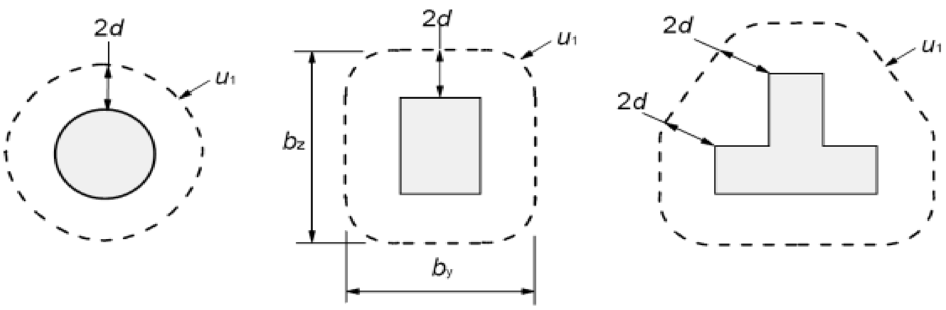


Figure 8: Typical basic control perimeters around loaded areas

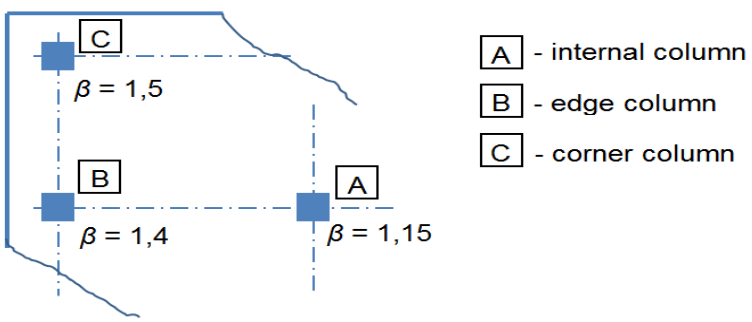


Figure 9: Recommended values for β

Mostly used method for calculating the load increase factor β is classified in Eurocode. It is determined due to unbalanced moment at a slab internal column connection in the critical perimeter.

$$\beta = 1 + k \,.\, \frac{M_{Ed}}{V_{Ed}} \,.\, \frac{u_1}{W_1} \tag{2}$$

k - *coefficient depending on the column dimensions*

M_{Ed} - *moment about the centroidal axis of the critical perimeter*

W_1 - *modulus of resistance of the critical perimeter*

Carrying out the punching shear design, first is necessary to check whether the design can be performed without shear reinforcement at the basic diameter u_1 if $V_{Ed} \leq V_{Rd,c}$, as follow:

V_{Ed} - *applied shear stress.*

$V_{Rd,c}$ - *the shear resistance without shear reinforcement.*

The design value for the shear resistance $V_{Rd,c}$ is given by:

$$V_{Rd,c} = C_{Rd,c} \,.\, k \,.\, \left(100.\rho_l \,.\, f_{ck}\right)^{1/3} + k_1.\sigma_{cp} \ \geq \ \left(V_{min} + k_1.\sigma_{cp}\right) \tag{3}$$

Where:

$C_{Rd,c} = \dfrac{0{,}18}{\gamma_c}$	- constant factor
γ_c	- partial safety factor for concrete
$k = 1 + \sqrt{\dfrac{200}{d}} \leq 2{,}0$	- factor accounting for size effects of the effective depth (d in mm)
$\rho_l = \sqrt{\rho_{ly}.\rho_{lx}} \leq 0{,}02$	- *the flexural reinforcement ratio (the values ρ_{ly} and ρ_{lx} and may be calculated for a strip width equal to the column width plus 3d each side).*
f_{ck}	- *characteristic compressive strength of cylinder concrete specimen*
$k_1 = 0{,}1$	- *recommended value*
$\sigma_{cp} = (\sigma_{cy} + \sigma_{cz})/2$	*where :* - $\sigma_{c,y} = \frac{N_{Ed,y}}{A_{cy}}$ and $\sigma_{c,z} = \frac{N_{Ed,z}}{A_{cz}}$ - *are the normal concrete stresses in the critical section in y and z directions.*
$N_{Ed,y}, N_{Ed,z}$	- *are the longitudinal forces across the full bay for internal columns and the control section for edge columns.*
A_c	- *area of concrete according to the definition of N_{Ed}*

$$V_{min} = 0{,}035.\,k^{3/2}.\,{f_{ck}}^{1/2} \tag{4}$$

To Ensure that maximum punching shear stress is not exceeded, the required shear reinforcement is determined in the following step $V_{Ed,\boldsymbol{u_0}} \leq V_{Rd,max}$ at the column perimeter, maximum punching shear resistance $V_{Rd,max}$ must be determined in the next step as follows:

$$V_{Ed,u_0} = \frac{\beta.V_{Ed}}{u_0.d} \leq V_{Rd,max} \tag{5}$$

Where:

$$V_{Rd,max} = 0{,}4.\,v.\,f_{cd}$$

$$v = 0{,}6.\,(1 - \frac{f_{ck}}{250})$$

After the design of $V_{Rd,max}$ has been successfully executed, the required punching shear reinforcement is determined in the following step where $V_{Ed} < V_{Rd,c}$,The punching shear resistance inside the shear-reinforced zone $V_{Rd.c+s}$ is calculated (Eq. 6) by means of a truss model with an inclination of the compression struts of 33°:

$$V_{Ed} < V_{Rd.c+s} = V_{Rd,cs} + V_{Rd,s} = 0{,}75\,.V_{Rd,c} + 1{,}5\,.\frac{d}{s_r}\,.A_{sw}\,.f_{ywd,ef}\cdot\frac{1}{u_1.d}\,.\sin\alpha \leq \mathrm{k_{max}}.\,\mathrm{v_{Rd,c}} \tag{6}$$

$V_{Rd,c}$ *- concrete contribution according to (Eq. 3)*

s_r *- radial spacing of perimeters of shear reinforcement*

A_{sw} *- area of shear reinforcement along one perimeter*

$f_{ywd,ef}$ *- effective design strength of the shear reinforcement*

$$f_{ywd,ed} = 250 + 0{,}25.\,d \leq f_{ywd}$$

f_{ywd} *- the design yield strength of shear reinforcement*

$$f_{ywd} = \frac{f_{yk}}{\gamma_s}$$

α *- angle between the shear reinforcement and the plane of the slab*

u_1 *- control perimeter located at* 2,0 d *from the column face*

The capacity of the shear reinforcement is limited to a multiple of the punching shear capacity without shear reinforcement. In this context, Eurocode 2 (incl. Amendment 1: EN 1992-1-1:2004/A1:2014) recommends an increase factor $\mathrm{k_{max}} = 1{,}5.\,d$ for stirrups and bent-up bars (Eq. 6). Analogous to slabs without shear reinforcement, the punching shear capacity of flat slabs and column bases is limited to the capacity of the compression struts at the face of the loaded area (perimeter u_0 , (Eq. 5)).The control perimeter at which shear reinforcement is not required u_{out} (or $u_{out,ef}$) is determined by:

$$u_{out} = \frac{\beta.V_{Ed}}{V_{Rd,c}.d} \tag{7}$$

The beyond perimeter of the shear reinforcement should be located at a distance of not more than 1,5 d within the perimeter u_{out} (or $u_{out,ef}$) see (Fig.10). The shear reinforcement should be provided in at least two rows and the radial spacing should not outpace $s_r = 0{,}75.d$. At a distance $0{,}30.d \leq s_0 \leq 0{,}50.d$ from the face of the loaded area the first row of shear reinforcement must be placed.

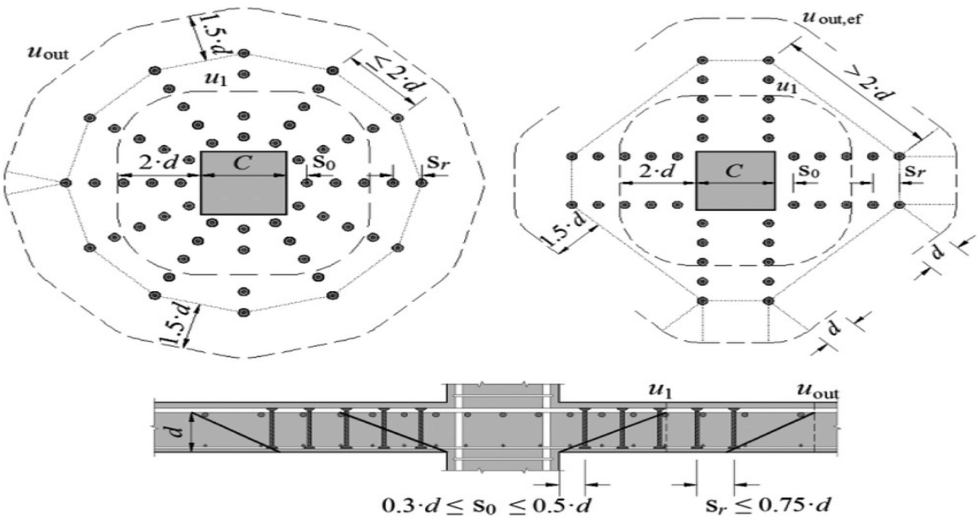


Figure 10: Control perimeters at internal columns

2.2.1.2 Design of punching shear resistance according to ACI 318 [10]:

The calculations according to ACI 318 code are planned to determine whether conditions used satisfies the minimum safety requirements against failure, the expressions presented in this section may vary from those stated in the above presented code. This because of the different safety ratios and materials properties.

The verification of punching shear resistance according to ACI 318-08 should be done by checking shear stresses in a control perimeter $d/2$ away from the ends of loaded area as shown in (Fig. 11),In these cases, the shear stress (V_u) shall be less than the shear strength provided by concrete (V_c),

The equation for slabs without shear reinforcement is shown in Equation 8:

$$V_u \leq \emptyset.V_c \tag{8}$$

Where $\emptyset = 0.75$ (strength reduction factor). The punching shear strength of flat-plate slabs without shear reinforcement is given by the minimum of the estimates from the following three equations:

$$V_c = 0{,}17.\left(1+\frac{2}{\beta_c}\right).\sqrt{f'_c}, \qquad \text{[MPa]} \qquad (9.a)$$

$$V_c = 0{,}083.\left(2+\frac{\alpha_s\, d}{b_0}\right).\sqrt{f'_c}, \qquad \text{[Mpa]} \qquad (9.b)$$

$$V_c = 0{,}33.\sqrt{f'_c}, \qquad \text{[Mpa]} \qquad (9.c)$$

V_c - *punching shear strength*

f'_c - *specified concrete cylinder strength (Mpa)*

α_c - *equal to 40 for interior columns*

β_c - *ratio of longer to shorter dimension of the loaded area*

b_0 - *the shear perimeter and is equal to* $\pi.(c+d)$ *for interior circular column and equal to* $\sum c + 4.d$ *for interior rectangular columns (Fig. 11)*

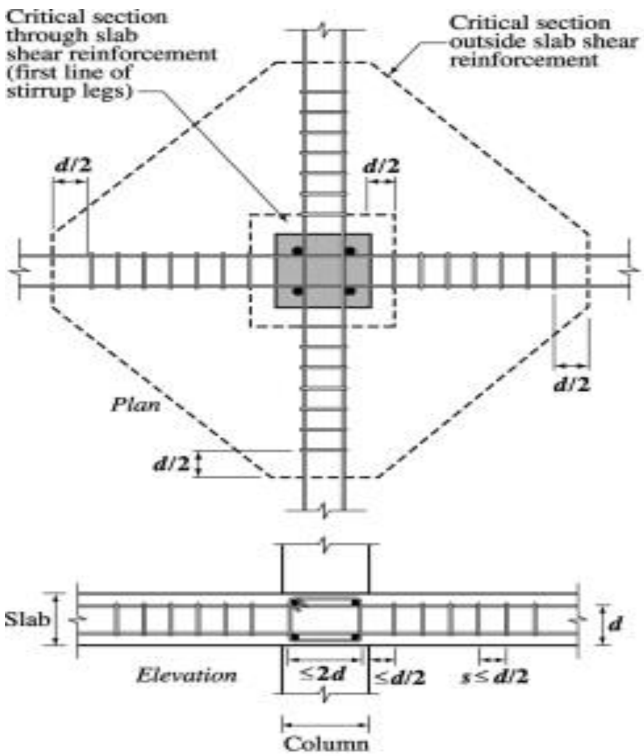


Figure 11: Control perimeter and arrangement of shear reinforcement according to ACI-318

If shear reinforcement in form of bars, wires or single- or multiple leg stirrups is used allowing effective depth for slabs, d, greater than or equal to (6 inch) 152.4 mm, then the punching shear capacity with stirrups of slab is calculated in accordance to Equation (10) :

$$V_u \leq \emptyset . V_n \qquad [\text{Mpa}] \qquad (10)$$

$$V_n = V_c + V_s \leq \frac{1}{2}\sqrt{f_c{}'} \qquad [\text{Mpa}] \qquad (11)$$

$\emptyset$ - *is the strength reduction factor for shear equal to 0,75*

V_n - *the nominal shear strength*

V_s - *the shear stress resistance provided by the shear reinforcement*

$$\mathrm{V_s} = \frac{\emptyset.\mathrm{A_v}.\mathrm{f_{yt}}}{\mathrm{s}.\mathrm{b_0}} \qquad [\text{Mpa}] \qquad (12)$$

V_c - *the punching shear strength in (Eq .11) should not exceed the limit of* $\frac{1}{6}.\sqrt{f_c{}'}$ *and the part of shear reinforcement provided is calculated from (Eq .12) :*

A_v - *area of shear reiforcement in a distance s in* mm^2*, s is the spacing of shear reinforcement in a parallel direction with the longitudinal reinforcement.*

$$A_v = \frac{(\mathrm{v_u} - \emptyset.\mathrm{v_c}).\mathrm{b_0}.\mathrm{s}}{\emptyset.\mathrm{f_y}} \qquad [mm^2] \qquad (13)$$

The ACI 318 code initiate the utilization of stud rails shear reinforcements in 2008, the Equation (11) used to determine the punching shear capacity of slabs reinforced with stud rails. Hence, the nominal shear strength, V_n, is limited to $0{,}67.\sqrt{f_c{}'}$ and V_c should not be taken greater than $0{,}25.\sqrt{f_c{}'}$.

2.2.1.3 Design of punching shear resistance according to FIB Model 2010 [11]:

MC 2010 recommendations are based on the so-called critical shear crack theory (CSCT) of Mutoni 2008[12], which consider punching shear resistance to the slab rotation outside the critical shear crack. Recent version of the code adopts a similar approach considering outer perimeter of shear reinforcement, the following regulations stated in this section were taken from FIB MC 2010.

The critical section lies at $0{,}5.d_v$ from supported area where d_v is the distance from the centroid of the reinforcement layers to supported area (Fig. 12):

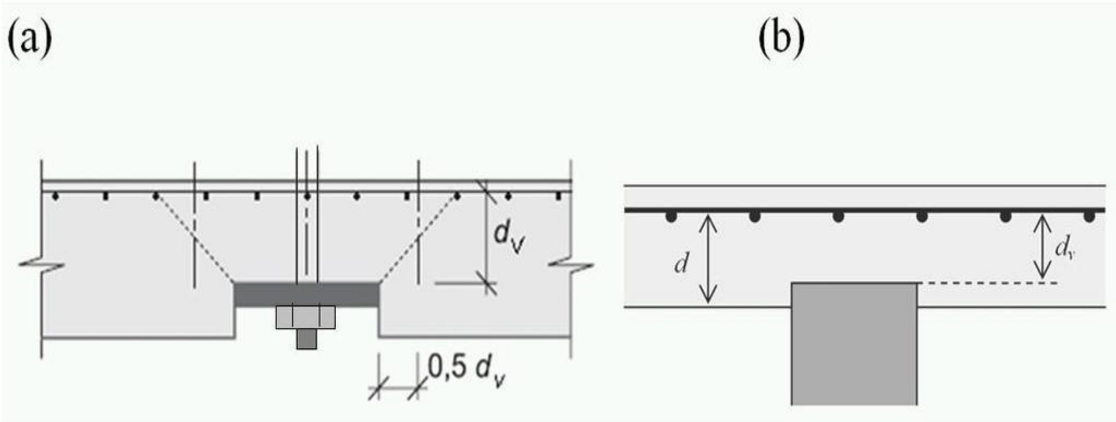


Figure 12: Effective depth of the slab considering support penetration (d_v) and effective depth for bending calculation(d)

In order to calculate the punching shear resistance, the control perimeter b_0 must be adopted, which accounts for the non-uniform arrangement of shear forces across the basic control perimeter, b_0 can be calculated according to equation (14):

$$b_0 = \frac{V_{Ed}}{V_{prep,d,max}} \tag{14}$$

V_{Ed} - *the design shear force with respect to punching*

$V_{prep,d,max}$ - *is maximum shear force per unit length perpendicular to the basic control perimeter (Fig 13)*

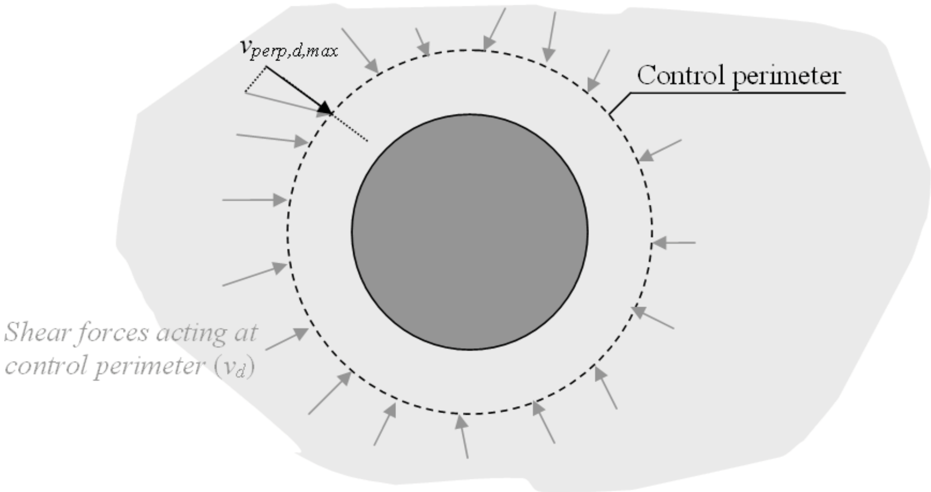


Figure 13: Shear force per unit length (V_d) and maximum value perpendicular to the basic control perimeter

Fib Model 2010 encounter four level of design, where levels I to III are considered for design and Level IV for the assessment. Level III is purposed for slabs with irregular geometry. The resistance of the shear is determined in terms of the slab geometry ψ (rotation), where Level II is calculated as follows:

$$\psi = 1{,}5.\frac{r_s}{d}.\frac{f_{yd}}{E_s}.(\frac{m_{Ed}}{m_{Rd}})^{1{,}5} \quad (15)$$

r_s *- indicate the position where the radial bending moment is zero with respect to the column axis*

d *- effective depth for bending calculation*

f_{yd} *- design yield strength of flexural reinforcement*

E_s *- modulus of elasticity of reinforcement*

m_{Ed} *- is the average bending moment per unit width in the support strip, which is approximated per unit width* $1{,}5.r_s$ *where* $r_s = 0{,}22.L$ *,for coaxial loaded internal columns,* $m_{Ed} = \frac{v_d}{8}$ *for level II*

m_{Rd} *- design average flexural strength per unit width of support strip*

The punching shear strength is based on calculation on the theory of critical shear crack, punching shear resistance is calculated as:

$$V_{Rd} = V_{Rd,c} + V_{Rd,s} \geq V_{Ed} \quad (16)$$

V_{Rd} *- design value of resistance to shear force*

$V_{Rd,c}$ *- design shear resistance connected to concrete*

$V_{Rd,s}$ *- design shear resistance provided by stirrups*

V_{Ed} *- design value of applied shear force*

The deign shear resistance attributed to the concrete may be taken as:

$$V_{Rd,c} = k_{\psi}.\frac{\sqrt{f_{ck}}}{\gamma_c}.b_0.d_v \quad (17)$$

f_{ck} *- characteristic compressive strength of concrete*

γ_c - *the concrete safety factor*

b_0 - *the control shear resistance perimeter*

d_v - *effective depth of the slab*

The perimeter k_ψ depend on the rotations of slab around the support region, calculated as follows:

$$k_\psi = \frac{1}{1{,}5+0{,}9.k_{dg}.\psi.\mathrm{d}} \leq 0{,}6 \tag{18}$$

$$k_{\mathrm{dg}} = \frac{32}{16+ d_g} \geq 0{,}75 \tag{19}$$

d_g - *maximum aggregate size*

The shear resistance provided by transverse reinforcement (stirrups) is calculated as:

$$V_{Rd,s} = \sum A_{sw}.k_e.\sigma_{swd} \tag{20}$$

$\sum A_{sw}$ - *is the all shear reinforcement cross-sectional area within the zone bounded by* $0{,}35\ d_v$ *and* d_v *from the border of the support region*

σ_{swd} - *is the stress can be activated in the shear reinforcement, and is taken as*

$$\sigma_{swd} = \frac{E_s.\psi}{6}.\left(1 + \frac{f_{bd}}{f_{ywd}}.\frac{d}{\emptyset_w}\right) \leq f_{ywd} \tag{21}$$

f_{ywd} - *the design yield strength of shear reinforcement*

$\emptyset_w$ - *the shear reinforcement diameter*

f_{bd} - *the bond strength, can be taken as 3MPa for corrugated bars may be used for design*

The maximum punching resistance is limited by crushing of the concrete struts near the support region, such that:

$$V_{Rd,max} = k_{sys}.k_\psi.\frac{\sqrt{f_{ck}}}{\gamma_c}.b_0.d_v \leq \frac{\sqrt{f_{ck}}}{\gamma_c}.b_0.d_v \tag{22}$$

k_{sys} coefficient represent the performance of punching shear reinforcing systems, for stirrups is taken as 2,4 and for studs 2,8, given the radial spacing to the first perimeter of shear reinforcement systems from the column face s_0 is $\leq 0{,}5\ d_v$ and the spacing of consecutive perimeters of shear reinforcement is less than $0{,}6\ d_v$. The spacing of vertical legs of shear reinforcement on all side of a

perimeter should not be more to 3 d_v, where that part of the perimeter is assumed to contribute to the shear capacity.
For slabs with insufficient amount shear reinforcement or deformations capacities additional reinforcement should be added (Fig. 14) to prevent continuous collapse.

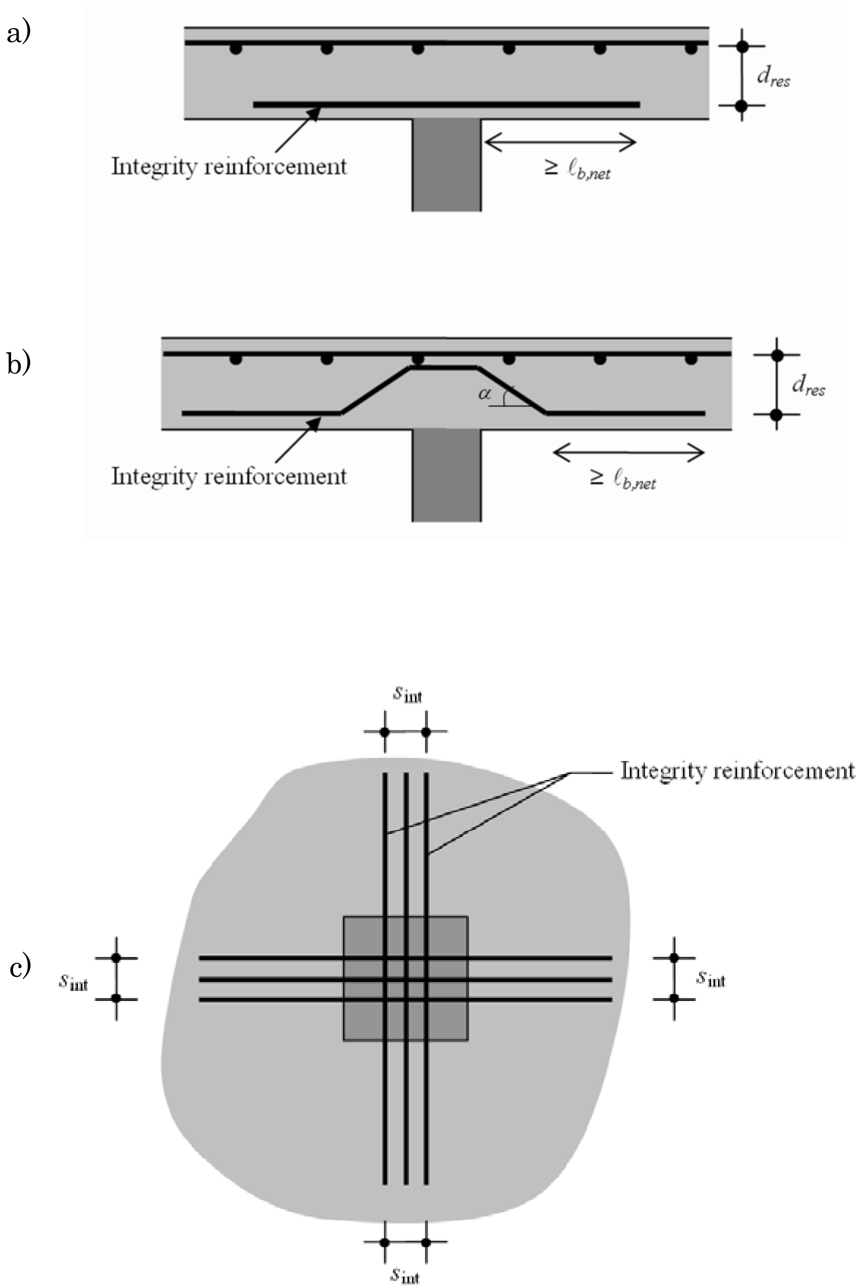


Figure 14: Integrity reinforcement: (a) straight bars; (b) bent-up bars: and (c) example of arrangement of integrity reinforcement

The required resistance provided after punching by the additional reinforcement can be calculated as follows:

$$V_{Rd,int} = \sum A_s . f_{yd} . \left(\frac{f_t}{f_y}\right)_k . \sin \alpha_{ult} \leq \frac{0{,}5.\sqrt{f_{ck}}}{\gamma_c} d_{res} . b_{int} \tag{23}$$

A_s - *represent sum of all reinforcement cross-sections developed on the compression side beyond the supported part of the slab or to well-anchored bent-up bars*

f_{yd} - *the design yield strength of bars*

$\left(\frac{f_t}{f_y}\right)_k, \varepsilon_{uk}$ - *parameter are defined in sub-clause (5.2.5.4) and they depend on the ductility class of reinforcement*

α_{ult} - *the angle of the integrity bar with respect to the slab plane at failure:*

α_{ult}	Type of integrity reinforcement
0°	Straight bars, class of ductility: A
20°	Straight bars, class of ductility: B
25°	Straight bars, class of ductility: C or D
$\alpha \leq 40°$	Inclined or bent-up bars, class of ductility: B, C or D

α - *the angle of the integrity bars with respect to the slab plane*

d_{res} - *the distance between the centroid of the flexural reinforcement ratio and the centroid of additional reinforcement, (Fig. 14 (a) and (b))*

b_{int} - *the control perimeter activated by the integrity reinforcement after punching. Calculated as follows:*

$$b_{int} = \sum \left(s_{int} + \frac{\pi}{2} . d_{res}\right) \tag{24}$$

The summation symbol refers to the groups of bars at the edge of the supported area and s_{int} is equal to the width of the group of bars (Fig. 14).

2.2.1.4 Observations:

The differential of punching shear capacity calculations according to Eurocode 2, ACI 318, Model code 2010, are based on a critical perimeter, that is situated between 0,5 to 2 d from the face of the column, essential variation of punching shear capacity without shear reinforcement are determined by the concrete compressive strength in ACI-318 and the parameter of flexural reinforcement ratio in EC2, rotation of slab in MC 2010, The classifications of shear reinforcement presented in European codes and ACI 318 is non-uniform, in Europe the distribution of shear reinforcement is uniform within the required area only, Eurocode consider that shear reinforcement should be distributed evenly around at least two perimeters , ACI 318 specifies concentrated arrangement along the direction of column axis and requires that the stress on the critical section at $d/2$ from the outermost shear reinforcement should be less than the one way shear resistance of concrete ($\frac{1}{6}.\sqrt{f'c}$), while Model code 2010 assesses both distributions, but in the case of the concentrated stirrups allocation, the length of the outer control perimeter is decreased, therefore the punching shear capacity is also reduced outside of the shear reinforcement zone.

2.2.1.5 Reinforcements types:

The brittle collapse of the slab is critical due to punching, therefore there are different methods can be used to avoid the punching failure such as:

- Increase slab depth, as an example to apply drop panels or column capitals.
- improve materials strength (concrete, steel, aggregate, etc.)
- inserting additional reinforcements

The first method not usually suitable in view of architectural or practical inspection.

The usage of high-performance materials in slab may not be reasonable from economically point of view, hence the shear reinforcement would be the conventional solution to avoid punching.

The aim of all shear reinforcement types is to increase shear capacity of slab-column connection, however different types can be used for new or existing building depending on design requirements.

1. . *Punching shear reinforcement for new structures:*

Headed shear studs (Fig. 15)[13], is preferred type option and widely used in flat slabs.

The good mechanical anchorage of heads increases punching resistance and load-bearing capacity.

At the construction site, the install of these reinforcements would be easy as they consist of studs welded to flat steel bars placed on formwork.

The stud shear reinforcement designed in a way to match the specifications and dimensions of structural steel standards. in Europe studs are often organized in a radial arrangement from the center of the column (Fig. 15) as a technical report [14] shows that this arrangement is better than the orthogonal one.

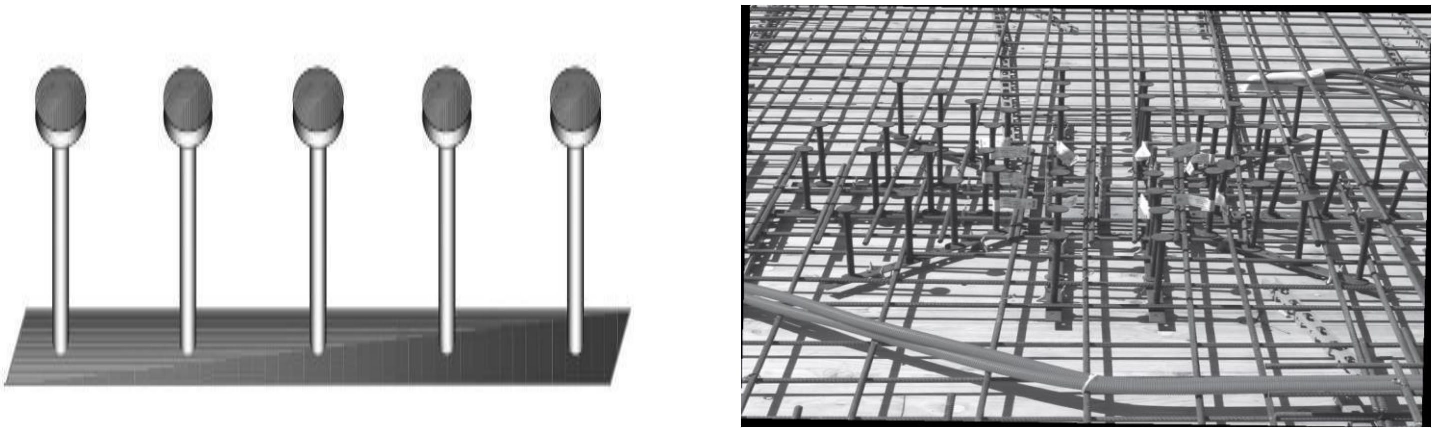


Figure 15: Shear reinforcement rail studs

Shear reinforcement stirrups made of cold-worked steel, the design of this type of reinforcement is similar to stirrups in beams.

shear stirrups can be formed to circular or rectangular shapes with different types (Fig. 16) such as:

- bent up bars
- closed stirrups
- one-legged open stirrups
- continuous U-shaped stirrups
- inclined stirrups

The u-shaped and closed stirrups reinforcement is difficult to use due to issues related to the assembly in the slab.

Based on study results [15], one-legged stirrups have a poor anchorage.

only inclined stirrups with a 60° inclination have shown efficiency to resist punching shear.

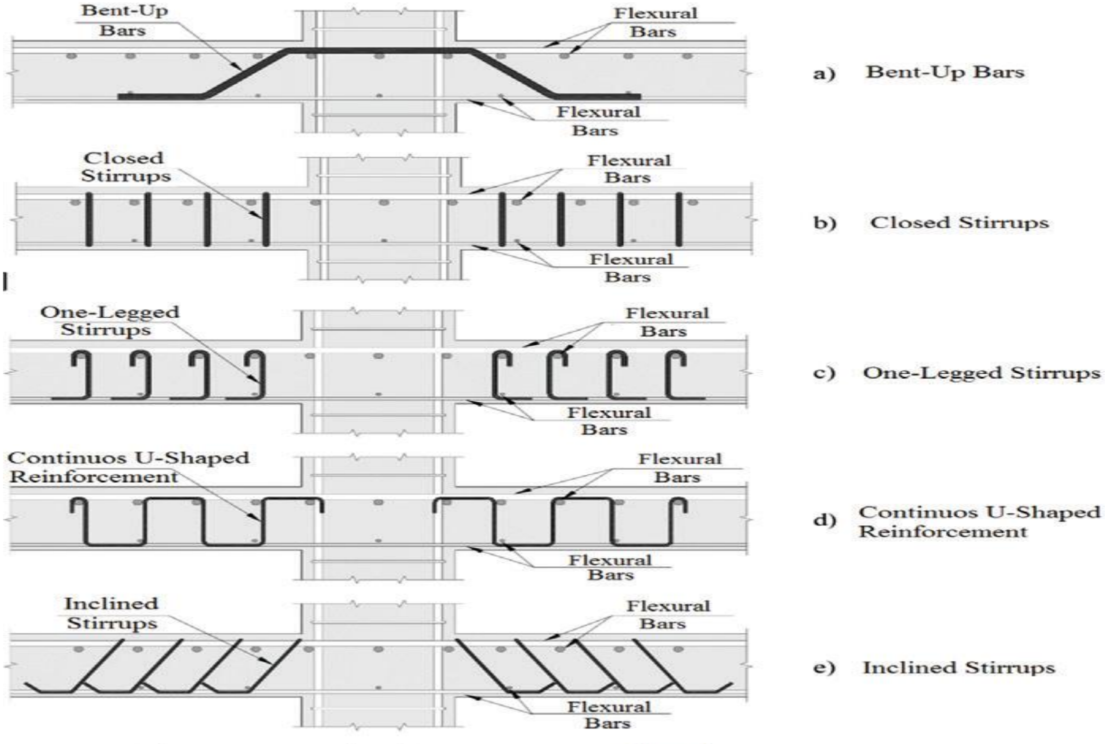


Figure 16: Types of shear reinforcement for slab-column connections [16]

Shear-band reinforcement [18]are made of thin steel strips of high ductility and strength.

The thin plate strip can be bent to different shapes (Fig. 17)[19] and placed from the top after the placement of all flexural reinforcement with minimum loss of cover.

This system is easy to fabricate, and install of reinforcement is efficient and soft

Shear-band reinforcement can improve the ductility of the slab and prevent brittle failure as well as increases the capacity of slab punching shear even if the flexural capacity not increased.

Figure 17: Shear-bands specimens

Lattice shear reinforcement [20] is a new type of reinforcement that consist of different layers linked together with inclined or vertical struts by welding.

At the vertices flexural reinforcement joined on the top of the upper chord and girders arranged parallel to each other (Fig. 18).

The lattice girders behave as a punching shear reinforcement close to the column area, improving the shear strength of the connection between slab and column and enhancing constructability with effective savings coasts.

Based on Eurocode 2 design and the significant stiffness of the struts anchorage, the highly functional girder system obtained a European authorization, many European countries and others using this system.

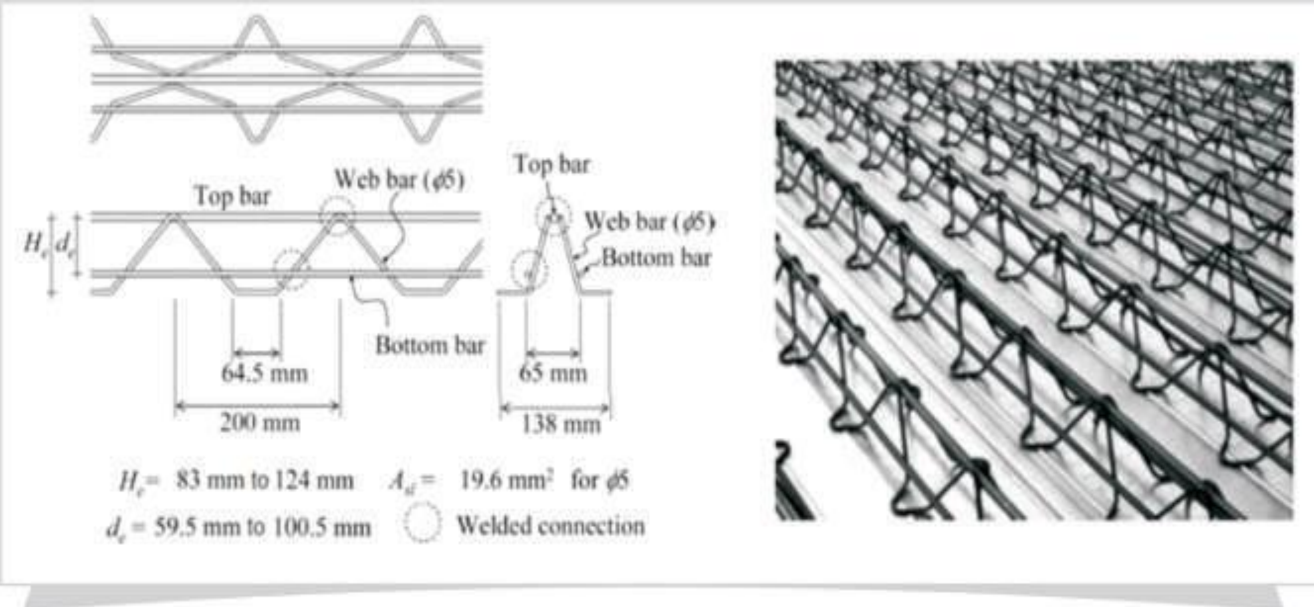


Figure 18: Lattice shear reinforcement]

UFO punching preventers [21] is a modern type of steel plate developed in Europe, this system consist of a coned shape used to strengthen slab-column connection see (Fig. 21), the holes on the top of the plate are made to fit the column reinforcement, the UFO's bottom part act as a support for the slab in outer area while inner region transfers reactions from the slab by membrane to area above column, this system can produce higher shear strength and the control perimeter can be determined by the size of UFO preventers.

Figure 21: Flat slab with UFO punching preventer [22]

2. . Punching shear reinforcement for existing structures:

Post-installed punching shear [23], is used for strengthening existing structures and can be installed in two ways: first is by drilling holes through the slab, then steel bars are inserted inside these holes and be prestressed against the plate by locking nuts on both upper and lower side (Fig. 19) [24].

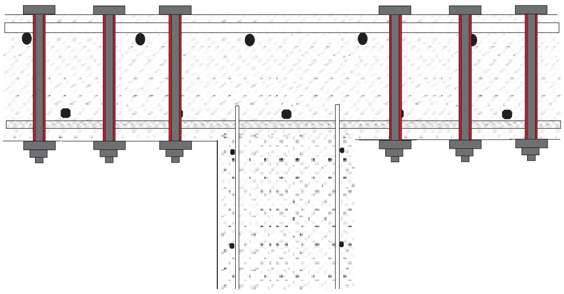


Figure 19: penetrating post-installed punching shear reinforcement

This way is only possible if both sides are accessible for work at once, the second way is when the upper side of the slab is not reachable, then the Hilti tension anchors known as HZA-P (Fig. 20) are joined into the drill holes in inclined position pointing towards the column and sealed by adhesive mortar.

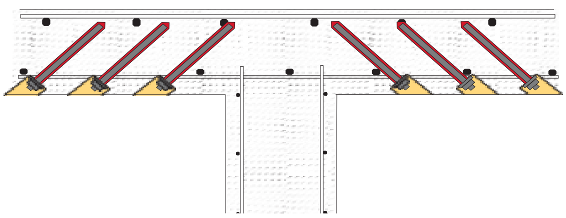


Figure 20: penetrating post-installed punching shear reinforcement(Right)
Hilti tension anchor HZA-P(Left)

3 Summary:

Regarding the advantages of flat slabs and their use in different countries, it is punctuated with faults and flaws, major problem of using such a plate is the deficiency of resisting punching between slab-column connection. The previous experiences from reported accident failures of slabs, improved the understanding of punching failures. in the corresponding critical zones, increasing concrete strength or slab depth will enhance punching shear capacity., the use of shear walls along with flat slab in critical zones resist both vertical and horizontal forces, the use of such methods wouldn't be the best option as it will rise the coast of the structure. shear reinforcement system could be a good manner in this case, there are punching shear reinforcement which is used for new structures such as , headed shear studs, shear reinforcement stirrups, shear band reinforcement, lattice shear reinforcement etc., However, it is even possible to enhance the punching shear resistance of existing slabs by adopting various layouts of post-installed punching shear.

Based on the information obtained from technical literature and reports, headed shear studs might be a personal recommendation for design of punching shear, the benefits with this system are exceptional ,increase space with slimmer and less obstructive structure, material savings, reduction of building height and excavation depth.; however, each type reveals different performance, a brief assessment of shear reinforcement use and equipment was introduced and based on the requirements a necessary approach is selected.

Structural design

1 General Information:

The objective of a practical part is to evaluate two procedures from different standards for the punching shear reinforcement design. First, the preliminary design was carried out to develop a structural system and to determine the geometry of fundamental structural elements. Subsequently, the flexural reinforcement of a selected reinforced concrete slab was design in accordance with ČSN EN 1992-1-1 [25]. Moreover, punching shear design was conducted in accordance with EC2 and ACI 318-08 ,The obtained results were compared and evaluated. As an outcome, the formwork drawing and reinforcement drawing of the selected slab was produced.

1.1 Layout:

The designed building is inspired from existing residential property (Fig. 22) located in Prague 13 Rotavska street in residential area with three-storey and one underground floor appears optically lower, it is oriented longitudinally with the street in the north-south direction (Fig. 23).

The designed solution is simplified to have a total height of 12,8 m from underground floor with 3,2 m each, length is 50,45 ,width 16,85 m, and three above floors with flat roof .

Underground floor sets on depth of 2,2 m from existing ground floor, it has an area of $827{,}20\,m^2$ fits for 31 cars and 21 storage rooms including technical and gas boiler room equipped with handicap car parking and a lift which leads to all above floors, the floor ventilated naturally by openings with height of 0,5 m and various lengths.

First floor Fig.(25) has an area of $827{,}07\,m^2$ with seven balconies and nine flats, the layout is with a hallway of 2,2 m width in the middle and flats oriented to the east and west. The entrance to the building is in the middle of the east facade directly from the street and is wheelchair accessible, as well as the entire building. The entrance is accentuated by both the facade and the wooden facade cladding in color to the entrance hall next to elevator. The typical floors are the same with few exceptions.

The roof is flat walkable, so it is accessible for peoples.

Figure 22: Existing residential building[26]

Figure 23: top view of the residential building

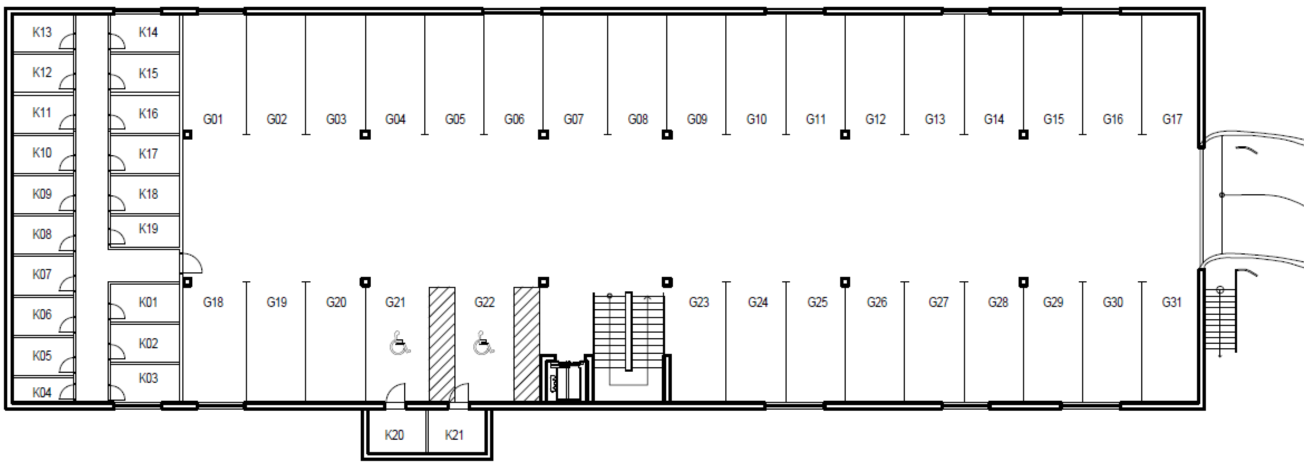


Figure 24: Architectural plan of underground floor

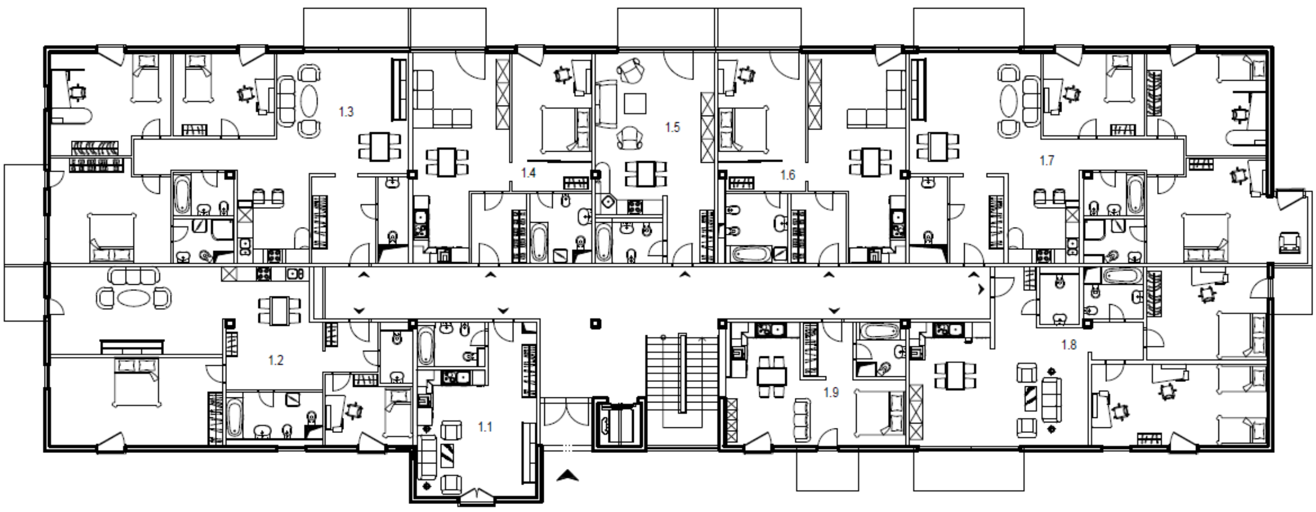


Figure 25: Architectural plan of first floor

1.2 Description of structural system:

The building founded on a foundation slab 200 *mm* with reinforced concrete columns, so the slab floor is monolithic connected with footings, the two-way flat slab on all floors supported by load bearing reinforced concrete perimeter walls 250 *mm* and columns 350 *mm x* 350 *mm* all other perimeter walls above underground have a thickness of 200 *mm*, balconies panels are cantilevered of 1,5 m made of reinforced concrete 250 *mm* connected to slab using Schöck Isokorb [27] load bearing and thermal insulation element, the staircase are monolithic supported on both side walls shown in (Fig.26) at inclination $28{,}1^{\circ}$ with step height of 160 *mm* , width 300 *mm* and length 1400 *mm* , the landing and flight made of reinforced concrete with 250 mm of thickness, ISI UNIT HALFEN-HIT and HALFEN TRAPEZ BOX type HBB-O [28] are used in staircase as impact sound insulation.

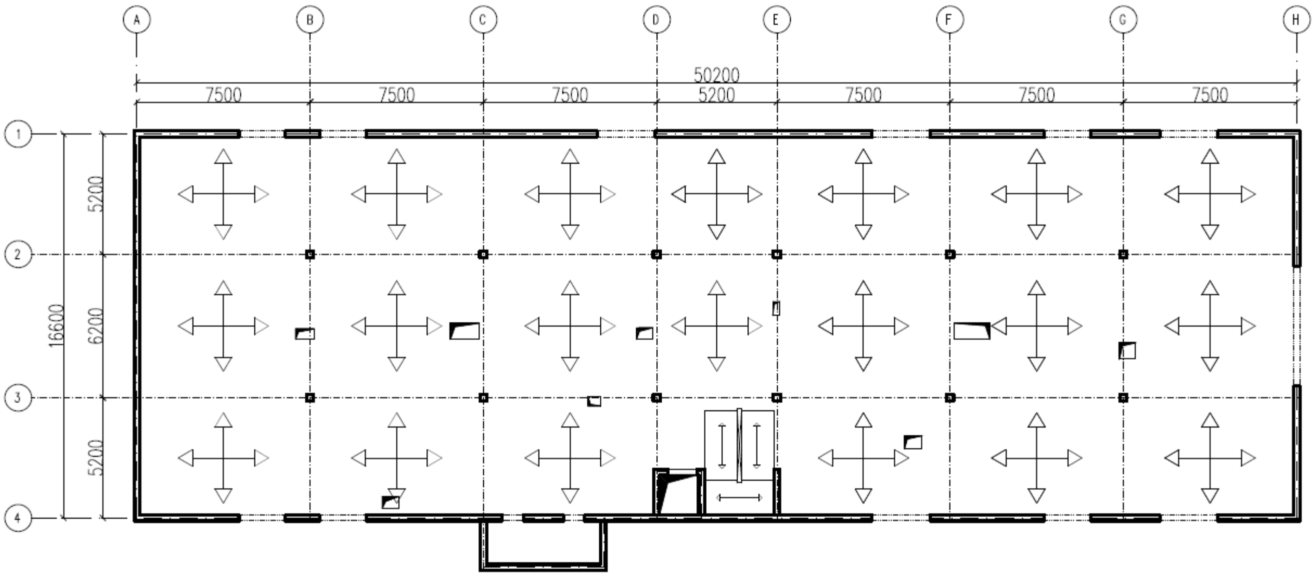


Figure 26: structural scheme of underground floor

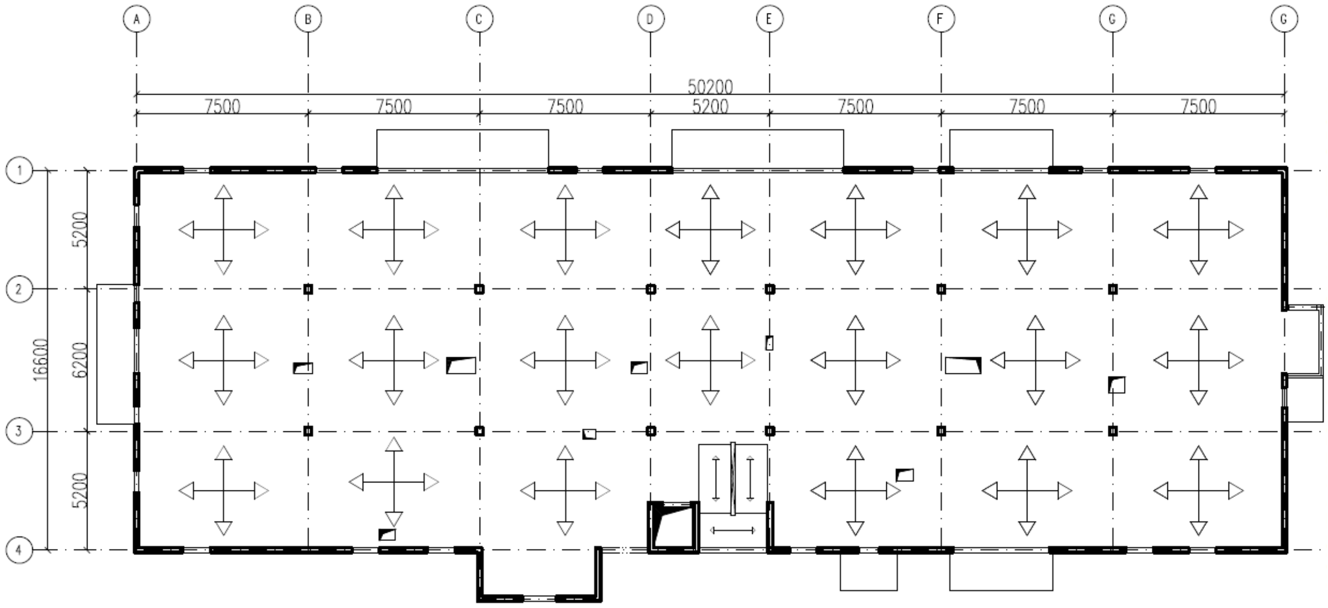


Figure 27.: structural scheme of first floor

1.3 Materials:

The materials were selected with accordance to ČSN EN 206-1:

Basement slab – waterproof concrete	C30/37-XC2, XD1-Dmax=22-Cl 0,20-S3
Underground floor – perimeter wall	C30/37-XC2,XD1-Dmax=22-Cl 0,20-S4
Underground floor – columns	C30/37-XC2-Dmax=22-Cl 0,40-S4
underground floor – slab	C30/37-XC1-Dmax=22-Cl 0,40-S4
1^{st} to 3^{rd} floor – load-bearing walls	C30/37-XC1-Dmax=22-Cl 0,40-S4
1^{st} to 3^{rd} floor – slab	C30/37-XC1-Dmax=22-Cl 0,40-S4
Balcony	C30/37-XC4, XF3-Dmax=16-Cl 0,40-S4
Monolithic staircase flight	C30/37-XC1-Dmax=22-Cl 0,40-S4
Monolithic landing	C30/37-XC1-Dmax=22-Cl 0,40-S4
1^{st} to 3^{rd} floor – non load bearing structures	Ytong blocks P2-500 + XELLA M

Steel B500A

1.4 Preliminary design:

In this section the design will be performed with the accordance to Eurocode .

1.4.1 Procedure of design dimensions:

a) Slab depth:

As in the beginning it is required to design the slab depth, the simplified method obtained as per span/depth ratio is calculated to avoid slab deflection in normal circumstances.

Assuming that concrete is exposed to dry or permanently wet environmental conditions (XC1) during its service life of 50 years together with steel B500A using $\emptyset\, 10\, mm$ and concrete grade C30/37.

According to Eurocode the structural class is S3

Effective slab depth can be calculated as:

$$d_s \geq \frac{l}{k_{c1}.k_{c2}.k_{c3}.\lambda_{d,tab}} \tag{25}$$

Where:

d_s - *effective depth of cross-section*

l - *element span* $l = 7{,}5\ m$

k_{c1} - *coefficient of cross-section (rectangular cross-section 1; T-shape cross-section 0.8 used;* $k_{c1} = 1$ *)*

k_{c2} - *coefficient of span (for* $l \leq 7\ m$ *,* $k_{c2} = 1{,}0$*, other cases* $k_{c2} = 7/l$ *used;* $k_{c2} = \frac{7}{7{,}5} = 0{,}93$ *)*

k_{c3} - *coefficient of stress in tensile reinforcement (assumed ,*$k_{c3} = 1{,}1 - 1{,}3$ *used;* $k_{c3} = 1{,}3$*)*

$\lambda_{d,tab}$ - *design span to depth ratio obtained from table (used the reinforcement ratio ,*0,5% *)*

$\lambda_{d,tab} = 24{,}6$ *(from table)*

Substituting all values to eq.(25):

$$d_s \geq \frac{7{,}5}{1.0{,}93.1{,}3.24{,}6} = 0{,}252\ m$$

Design : $d_s = 255\ mm$

b) Concrete cover:

As effective depth is calculated,concrete cover shall be computed in the next step as follows:

$$c_{nom} = c_{min} + c_{dev} \tag{26}$$

c_{nom} - *required nominal concrete cover*

c_{min} - *minimum concrete cover*

c_{dev} - *allowance for deviation (for monolithic structures recommended: 10.0 mm)*

$$c_{min} = \max\left\{c_{min,b}, c_{min,dur} + \Delta c_{dur,y} - \Delta c_{dur,st} - \Delta c_{dur,add}, 10\ mm\right\} \tag{27}$$

$c_{min,b}$ - *minimum cover for bond*

$$c_{min,b} = 1{,}0.\emptyset = 1{,}0\ .\ 10 = 10\ \text{mm} \tag{28}$$

$c_{min,dur}$ - *minimum cover for durability calculated depending on structural and exposure class (taken as* $c_{min,dur} = 10\ mm$ *)*

$\Delta c_{dur,y}$ - *additive safety element (recommended* $\Delta c_{dur,y} = 0\ mm.$*)*

$\Delta c_{dur,st}$ - *Reduction of minimum cover for use of stainless steel (recommended* $\Delta c_{dur,st} = 0\ mm.$ *)*

$\Delta c_{dur,add}$ - *Reduction of minimum cover for use of additional protection (recommended* $\Delta c_{dur,add} = 0\ mm.$ *)*

$\emptyset$ - *the diameter of the reinforcement bar (* $\emptyset = 10\ mm$ *)*

Substituting to eq(27), the minimum concrete cover is:

$$c_{min} = \max\{10\,, 10\ +\ 0{,}0\ mm - 0\ mm - 0\ mm, 10\ mm\}$$

$$c_{min} = 10\ mm$$

From equation (26):

$$c_{nom} = c_{min} + c_{dev} = 10 + 10 = 20\ mm$$

Final slab depth is :

$$h_s = d_s + \frac{\emptyset}{2} + c = 255 + \frac{10}{2} + 20 = 280\ mm \qquad (29)$$

Design of slab depth can be reduced to $h_s = 250\ mm$ as the preliminary method does not consider the effect of loads on slab and assuming that live load on residential building is not significantly high, nevertheless the deflection of slab should be checked and design assumption has to be verified.

Design dimension of column will be evaluated after all loads on slab are calculated.

1.4.2 Calculation of loads:

a) Permanent loads:

Underground floor slab:

Permanent Loads	$f_k \left[\frac{kN}{m^2}\right]$	$\gamma[-]$	$f_d \left[\frac{kN}{m^2}\right]$
Porcelain tiles	$23{,}57.0{,}01 = 0{,}24$	1,35	0,324
Concrete leveling layer	$24.0{,}05 = 1{,}2$	1,35	1,63
EPS	$0{,}30.0{,}05 = 0{,}015$	1,35	0,020
Reinforced concrete slab	$25.0{,}250 = 6{,}25$	1,35	8.44
Mineral wool	$0{,}68.0{,}05 = 0{,}034$	1,35	0,046
Plaster	$18.0{,}005 = 0{,}09$	1,35	0,12
			$\sum = 10{,}58$

Typical floor slab:

Permanent Loads	$f_k\left[\frac{kN}{m^2}\right]$	$\gamma[-]$	$f_d\left[\frac{kN}{m^2}\right]$
Porcelain tiles	$23{,}57.0{,}01 = 0{,}24$	1,35	0,324
Concrete leveling layer	$24.0{,}05 = 1{,}2$	1,35	1,63
EPS	$0{,}30.0{,}05 = 0{,}015$	1,35	0,020
Reinforced concrete slab	$25.0{,}250 = 6{,}25$	1,35	8,44
Plaster	$18.0{,}005 = 0{,}09$	1,35	0,12
			$\sum = 10{,}53$

Roof slab:

Permanent Loads	$f_k\left[\frac{kN}{m^2}\right]$	$\gamma[-]$	$f_d\left[\frac{kN}{m^2}\right]$
Gravel 16/32	$15.0{,}05 = 0{,}75$	1,35	1,01
Geotextile	0,005	1,35	$6{,}75.10^{-3}$
EPS	$0{,}35.0{,}30 = 0{,}11$	1,35	0,15
2xBitumens	$2.(11.0{,}004) = 0{,}088$	1,35	0,12
Reinforced concrete slab	$25.0{,}250 = 6{,}25$	1,35	8,44
Concrete leveling layer	$24.0{,}05 = 1{,}2$	1,35	1,63
Plaster	$18.0{,}005 = 0{,}09$	1,35	0,12
			$\sum = 11{,}48$

Landing:

Permanent Loads	$f_k\left[\frac{kN}{m^2}\right]$	$\gamma[-]$	$f_d\left[\frac{kN}{m^2}\right]$
Ceramic tiles	$20.0{,}01 = 0{,}2$	1,35	0,27
Concrete leveling	$24.0{,}05 = 1{,}2$	1,35	1,62
Ployethylene foam sheet	$0{,}21.0{,}02 = 4{,}2.10^{-3}$	1,35	$5{,}67.10^{-3}$
Reinforced concrete slab	$25.0.2 = 5$	1,35	6,75
Gypsum board	$5{,}2.0{,}01 = 0{,}052$	1,35	0,07
			$\sum = 8{,}72$

Flight:

Permanent Loads	$f_k \left[\frac{kN}{m^2}\right]$	$\gamma[-]$	$f_d \left[\frac{kN}{m^2}\right]$
Slab	$\frac{0{,}2.25}{\cos(28{,}1)} = 5{,}67$	1,35	7,66
Cladding	$0{,}5.\frac{160+300}{300} = 0{,}77$	1,35	1,04
Steps	$\frac{0{,}160}{2}.25 = 2$	1,35	2,7
Gypsum board	$5{,}2.0{,}01 = 0{,}052$	1,35	0,07
			$\sum = 11{,}47$

Balcony:

Permanent Loads	$f_k \left[\frac{kN}{m^2}\right]$	$\gamma[-]$	$f_d \left[\frac{kN}{m^2}\right]$
Ceramic tiles	$20.0{,}01 = 0{,}2$	1,35	0,27
Waterproof	$11.0{,}002 = 0{,}022$	1,35	0,023
Reinforced concrete slab	$25.0.25 = 6{,}25$	1,35	8,44
Plaster	$18.0{,}005 = 0{,}09$	1,35	0,12
			$\sum = 8{,}85$

b) <u>Imposed loads:</u>

The imposed load actions values were taken from Eurocode of the category A for residential buildings according to the given tables. the specified values of snow loads were taken from the national annex ČSN [25]

Snow load:

For the persistent design situation snow load on roof should be calculated as follows:

$$s = \mu_i \, . \, C_e \, . C_t \, . s_k \tag{31}$$

μ_i - *the snow shape coefficient*

C_e - *the exposure coefficient*

C_t - *thermal coefficient*

s_k - *characteristic value of the snow load on the ground (according to snow area)*

The snow shape coefficient determined depending on the angle of roof inclination α as the roof assumed to be flat, $0^\circ \leq \alpha \leq 30^\circ$ which correspond to $\mu_1 = 0{,}8$

The exposure coefficient C_e considered to be equal to 1,0 as the topography assessed to be normal where area of the building has no remarkable removal of snow by wind actions on construction work.

Thermal coefficient C_t accounts for the reduction of snow loads on roof with height thermal transmittance ($u > 1 \frac{W}{m^2.k}$) for other cases $C_t = 1{,}0$
characteristic value of snow load s_k on the ground is calculated from the snow load zone map [29], and Prague is in the first zone with $s_k = 0{,}70 \frac{kn}{m^2}$

Substituting all values to equation (31):

$$s = 0{,}8\,.\ 1{,}0\,.\,1{,}0\,.\,0{,}7 = 0{,}56\ \frac{kN}{m^2}$$

Partition loads:

The calculation of movable wall partitions is computed as follows:

- Thickness of Ytong blocks P2-500 [30] $t = 150\ mm$
- Height of the wall $l = 3{,}2\ m$
- Density of masonry blocks $\rho_{,m} = 500\ \left[\frac{kg}{m^3}\right]$

Self-weight of blocks:

$$\gamma_b = \frac{\rho_{,m} \cdot g}{1000}.\,t. = \frac{500.9{,}81}{1000}.\,0{,}15.3{,}2 = 2{,}4\ \left[\frac{kN}{m}\right] \tag{30}$$

The uniformly distributed load is defined relating to the self-weight of the wall where $2{,}4 > 2 < 3\ \left[\frac{kN}{m}\right]$ then according to EC2 wall length correspond to $q_k = 1{,}2\ \left[\frac{kN}{m^2}\right]$

Loads on floor slab:

Live Loads	$f_k\left[\frac{kN}{m^2}\right]$	$\gamma[-]$	$f_d\left[\frac{kN}{m^2}\right]$
Floors Category A	1,5	1,5	2,25
Stairs	2,0	1,5	3
Balconies	3	1,5	3,75
Partition walls	1,2	1,5	1,8

Loads on roof slab:

Live Loads	$f_k \left[\frac{kN}{m^2}\right]$	$\gamma[-]$	$f_d \left[\frac{kN}{m^2}\right]$
Snow	0,56	1,5	0,84

c) Column dimension:

Design dimension of column requires the point load from all above floor on the selected center of column, the underground columns are the most loaded ones, for the design selected column (B3) Fig (26) according to longest spans in both directions (X-direction, and Y-direction).

The tributary area of the column is the middle of both spans in two directions :

$$A = 7{,}5 \,.\, 5{,}7 = 42{,}75\, m^2 \qquad (32)$$

Pont load calculation:

Structure	$f_k[KN/m^2]$	$A[m^2]$	$Quantity$	$F_K[KN]$	$\gamma[-]$	$F_d[KN]$
Underground floor						
Reinforced concrete slab	6,25	42,75	1	267,19	1,35	360,71
Flooring	1,6	42,75	1	68,4	1,35	92,34
Live load	1,5	42,75	1	64,13	1,5	96,20
Partition	1,2	42,75	1	51,3	1,5	76,95
Typical floors						
Reinforced concrete slab	6,25	42,75	2	534,38	1,35	721,41
Flooring	1,6	42,75	2	136,8	1,35	184,68
Live load	1,5	42,75	2	128,25	1,5	192,38
Partition	1,2	42,75	2	128,25	1,5	192,38
Roof						
Reinforced concrete slab	6,25	42,75	1	267,19	1,35	360,71
Flooring	1,5	42,75	1	64,13	1,35	96,20
Snow load	0,56	42,75	1	23,94	1,5	32,43

The sum of point load from all above floors $N_{Ed} = 2405{,}91\, KN$ shall be used to determine the cross-section size of centrally loaded column as follows:

$$\frac{N_{Ed}}{(0{,}8.f_{cd} + \rho.\sigma_s)} \leq A_c \qquad (33)$$

Where:

N_{Ed} - *maximum point load acting on the selected column*

f_{cd} - *design strength of concrete* $f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{30}{1,5} = 20\ mpa$

ρ - *reinforcement ratio (use* $1\% - 3\%$*) used* $\rho = 2\%$

f_{yk} - *tensile strength of steel*

σ_s - *stress in reinforcement (* $\sigma_s = 400\ Mpa$ *)*

γ_s - *partial safety factor of steel (recommended* $\gamma_s = 1,15$ *)*

A_c - *cross section area*

Substituting all value to equation (33):

$$\frac{2405,91\ .\ 1000}{(0,8\ .\ 20 + 0,02\ .\ 400)} \leq A_c$$

$$100246,25\ mm^2 \leq A_c$$

Design column : $350\ mm\ x\ 350\ mm$

1.4.3 Balcony structural system:

The balcony will be supported by Schöck Isokorb elements (fig.28) as it is a load bearing and thermal insulation that can minimize the thermal bridge and reduces the sound effects.

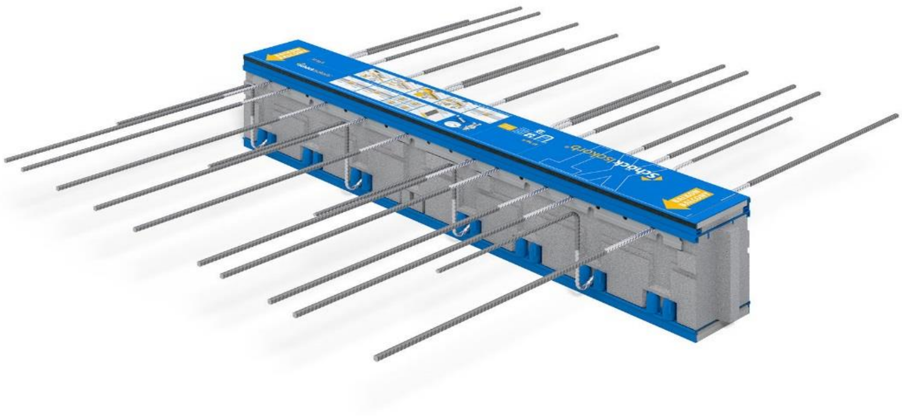


Figure 28: Schöck Isokorb element for balcony

The selection of types depending on the resistance for bending moment and shear forces therefore the internal forces on balconies should be calculated as follows :

The calculations used is for the longest balcony cantilever span shown in (Fig.29)

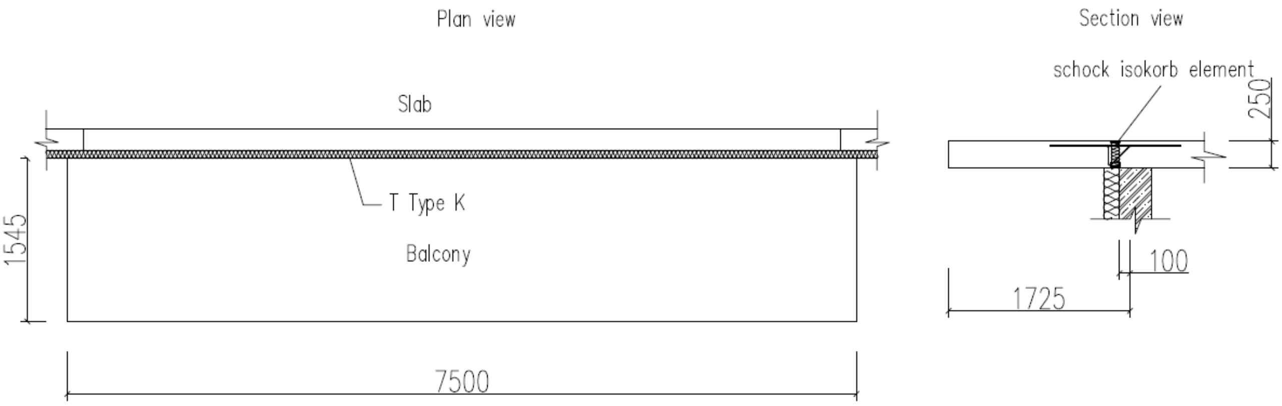


Figure 29.: plan view(left) and section view(right) of the balcony

<u>Load and static system of balcony:</u>

Geometry: length of cantilever $l_k = 1{,}725\ m$

Balcony slab thickness $h = 250\ mm$

Three-sided wraparound balustrade $h_R = 1{,}0\ m$

Loads on balcony: balcony slab and flooring $g = 6{,}6\ \frac{KN}{m^2}$

Live load $q = 3\ \frac{KN}{m^2}$

Edge load (balustrade) $g_R = 1{,}5\ \frac{KN}{m}$

Exposure classes: exterior $XC\ 4$

Interior $XC\ 1$

Selected materials: concrete grade C30/37 for both balcony and floor

concrete cover $c_{nom} = 35$ mm for Isokorb tension bars

(reduction Δc_{def} by 5mm, concerning quality measure *Schöck Isokorb* Production)

Connection geometry: No height offset, no floor edge down stand beam, no balcony upstand

Floor support: Floor edge supported directly

Balcony support: Restraint of cantilever slab using type k

Calculation of internal forces at the ultimate limit state:

$$m_{Ed} = -\left[(\gamma_G . g + \gamma_Q . q) . \frac{l_k^{\,2}}{2} + \gamma_G . g_R . l_k\right] \tag{34}$$

$$= -\left[(1{,}35.6{,}6 + 1{,}5.3) . \frac{1{,}725^2}{2} + 1{,}35.1{,}5.1{,}725\right]$$

$$m_{Ed} = -23.45\ \frac{\mathrm{KN.m}}{\mathrm{m}}$$

$$V_{Ed,z} = +[(\gamma_G . g + \gamma_Q . q) . l_k + \gamma_G . g_R] \tag{35}$$

$$V_{Ed,z} = +[(1{,}35.6{,}6 + 1{,}5.3) . 1{,}725 + 1{,}35.1{,}5]$$

$$V_{Ed,z} = +25{,}16 \frac{KN}{m}$$

Depending on the obtained results the selected type from catalog is:

$$T\ TYPE\ K - M2 - V1 - REI120 - CV35 - X80 - H250 - 6.0.$$

Where:

T	*- Schöck Isokorb mode*
K	*- Type*
$M2$	*- Main load-bearing level*
$V1$	*- Secondary load-bearing level*
$REI\ 120$	*- Fire protection*

$CV35$	*- Concrete cover*
$X80$	*- insulation element thickness*
$H250$	*- Isokorb height*
6,0	*- Generation*

The resistance of bending moment (catalog page 31) $M_{Rd} = 32{,}2\ \frac{KN.m}{m} > m_{Ed} = 23{,}45\frac{KN.m}{m}$

design is satisfied.

Shear resistance (catalog page 31) $V_{Rd,z} = 34{,}8\frac{KN}{m} > V_{Ed,z} = 25{,}16\frac{KN}{m}$

design is satisfied.

Deflection:

Generally, the deflection should be calculated in accordance to EC2 and BS plus the deflection from Schöck Isokorb taking to account the floor rotation angle and drainage direction.

For simplicity we evaluate only deflection p as result of Schöck Isokorb as follows:

Serviceability limit state (deflection):

$$p = \tan\alpha\,.\,l_k.\left(\frac{m_{pd}}{m_{Rd}}\right).10\ [mm] \tag{36}$$

$\tan\alpha$ - deflection factor from the Schöck Isokorb given under 100% steel utilization used for the required deflection estimation. ($\tan\alpha = 0{,}5$ page 34)

l_k - cantilever length [m]

m_{pd} - relevant bending moment $\left[\frac{KN.m}{m}\right]$ in the ultimate limit state for the determination of the deflection p [mm] from Schöck Isokorb

$g + {}^{q}/_{2}$ - selected load combination (recommended determination of deflection from Schöck Isokorb)

m_{Rd} - maximum design moment $\left[\frac{KN.m}{m}\right]$ of the Schöck Isokorb

$$m_{pd} = -\left[\left(\gamma_G.g + \gamma_Q.\frac{q}{2}\right).\frac{l_k^{2}}{2} + \gamma_G.g_R.l_k\right] \tag{37}$$

$$m_{pd} = -\left[\left(1{,}35.6{,}6 + 1{,}5.\frac{3}{2}\right).\frac{1{,}725^2}{2} + 1{,}35.1{,}5.1{,}725\right]$$

$$m_{pd} = -20{,}10\frac{KN.m}{m}$$

Substitution all values to equation (36):

$$p = 0{,}5.1{,}725.\left(\frac{-20{,}10}{-32{,}2}\right).10 = 5{,}38\,mm$$

Expansion joints: Length of balcony :$7{,}5\ m < e = 13{,}5\ m\ (given\ catalog\ page\ 36)$

No expansion joints are required.

<u>Number of schock isokorb elements :</u>

Standard product dimensions are shown in (fig. 30) for type k, the isokorb elements connected all together and placed along the required width of balcony, on site it is possible to cut at the shown position but that will result a reduction of load-bearing capacity.

in case the connection layout is not implemented in cataloge a speacial design can be made by the company for the requested layout.

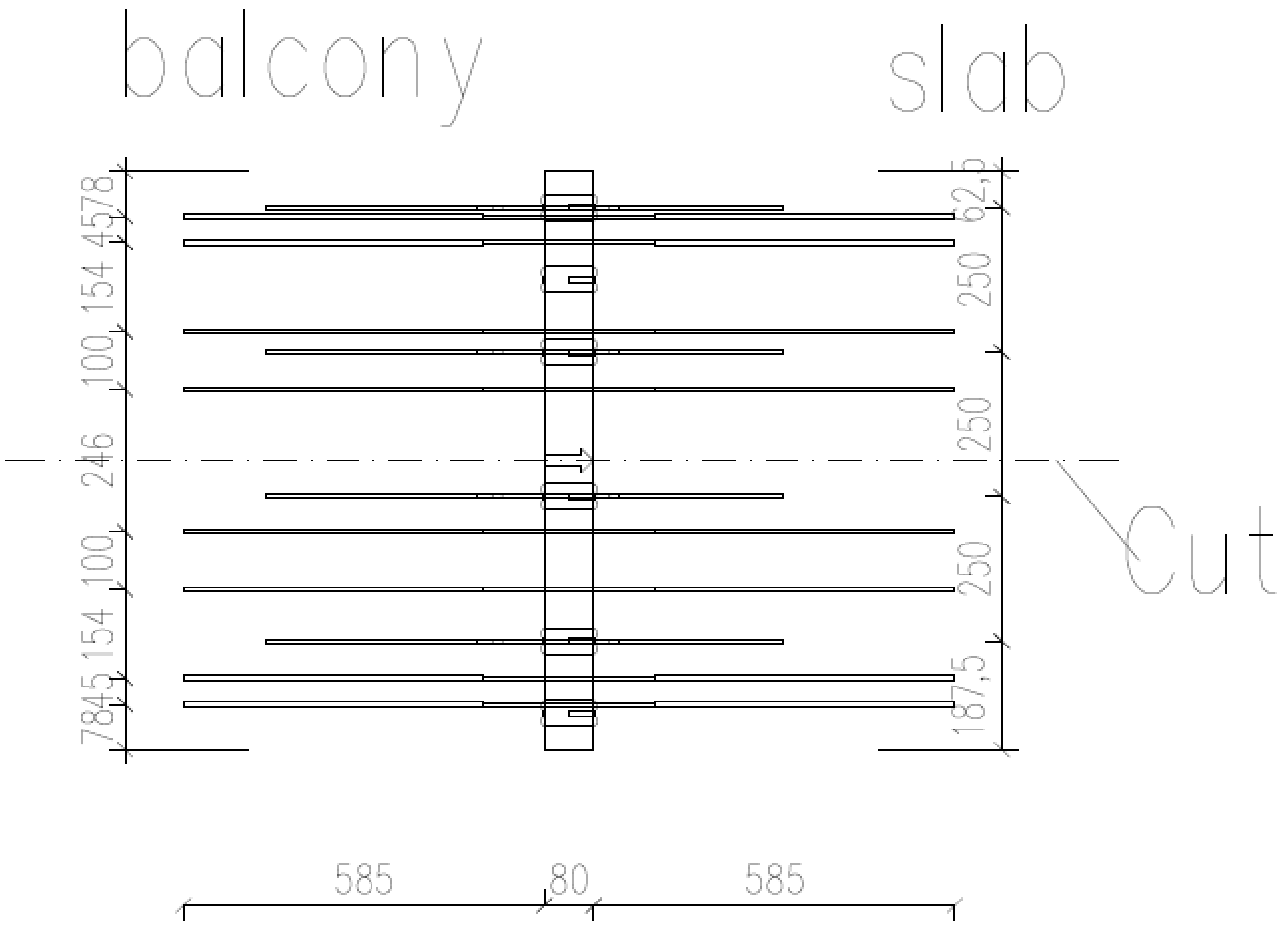


Figure 30.: Schöck Isokorb T TYPE K – M2 – V1 *: product layout*

2 Detailed analysis of flat slab:

In this section the design analysis will be performed for the first-floor slab ,punching resistance will be checked according to Eurocode and ACI standards.

The structural system of the building was modelized (Fig.31) using SCIA Engineering software version 19.1 [31], the software provides an advanced analysis of almost all structures and designing.

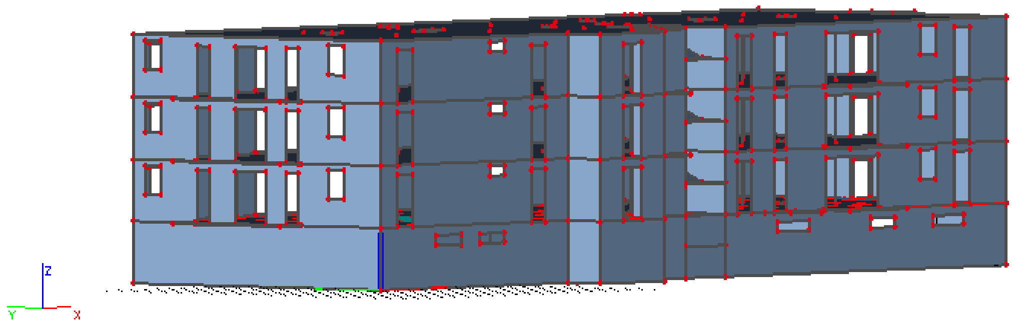


Figure 31.: structural model of the building with SCIA

2.1 Design check of deflection:

SCIA engineering software allows to check the long-term deflections of reinforced concrete flexural members through the option of code dependent deflection (CDD) based on technical standards for analysis and designing.

Check of deflection is performed on first floor slab with quasi-permanent combination set automatically after loading the slab with permanent and variable loads together with user reinforcements.

The software takes three main effects for the deformation analysis :

- Effect of loads : the deflection is computed based on long-term effects of applied load after a period of time.

- Effect of cracking : the process of permanent cracking is analyzed during the lifetime of structure taking the worst cases for calculation of long-term deflection using effective tensile concrete strength.

- Effect of creep: the effect of ongoing deformation under constant load encountered along with effective modulus of elasticity that is calculated using creep coefficient.

These effects are used in calculation process to evaluate two required checks :

a) Total deflection $\delta_{tot,z}$ where the slab subjected on an appropriate limit should not exceed $\delta_{tot,lim,z} = \frac{Span}{250}$.

$$\delta_{tot} = \delta_s + \delta_{l,creep} \tag{38}$$

Where :

δ_s	-Short term deflection
$\delta_{l,creep}$	-Long term deflection
δ_{imm}	-immediate deflection

b) Additional deflection $\delta_{add,z}$ the deflection that could damage adjacent construction parts of the structure, is the difference between sum of short-term and long-term with creep toward immediate deflection Eq.(39) , the deflection should be limited to $\delta_{add,lim,z} = \frac{Span}{500}$.

$$\delta_{add} = \delta_s + \delta_{l,creep} - \delta_{imm} \tag{39}$$

The component calculated by the software presented graphically in Fig.32

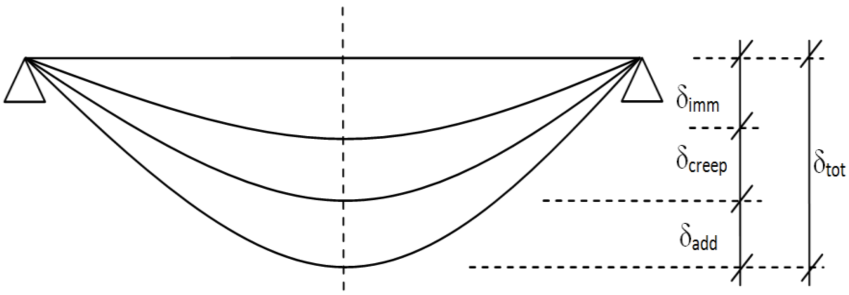


Fig. 32: Deflection components [32]

The results analysis of the slab as it shown in (Fig.33) and (Fig.34) validate both conditions where:

$$\delta_{tot,z} = 14{,}4\ mm\ <\ \delta_{tot,lim,z} = \frac{6200}{250} = 25\ mm \tag{40}$$

$$\delta_{add,z} = 9{,}7\ mm\ <\ \delta_{add,lim,z} = \frac{7500}{500} = 15\ mm \tag{41}$$

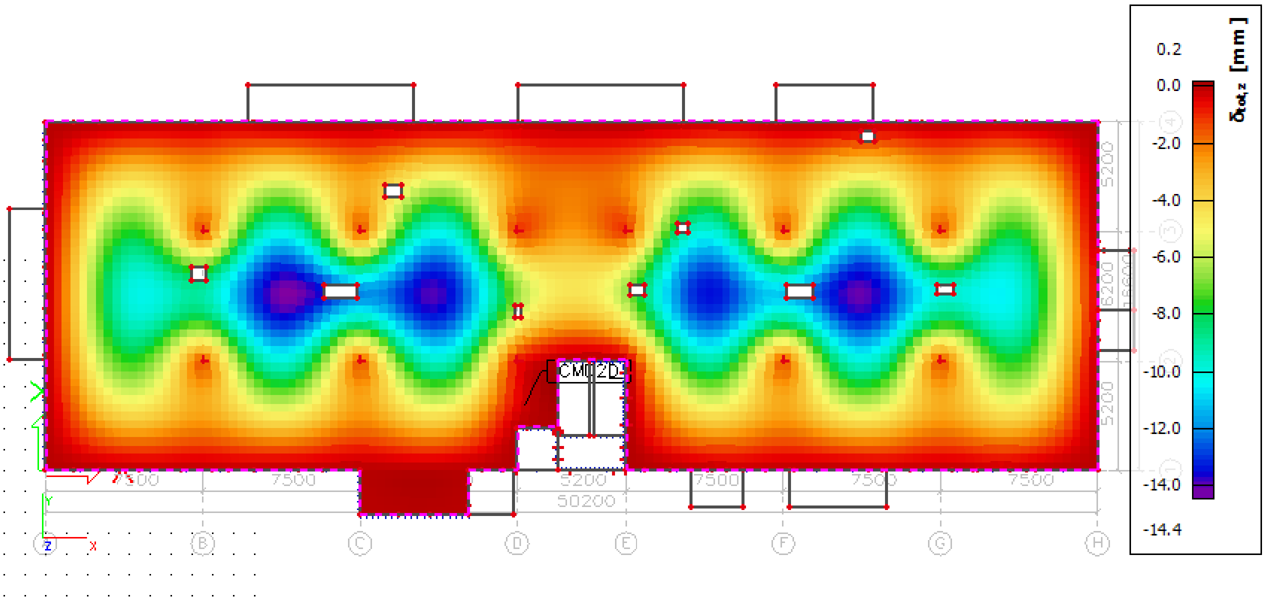


Fig. 33: SCIA results of total deflection

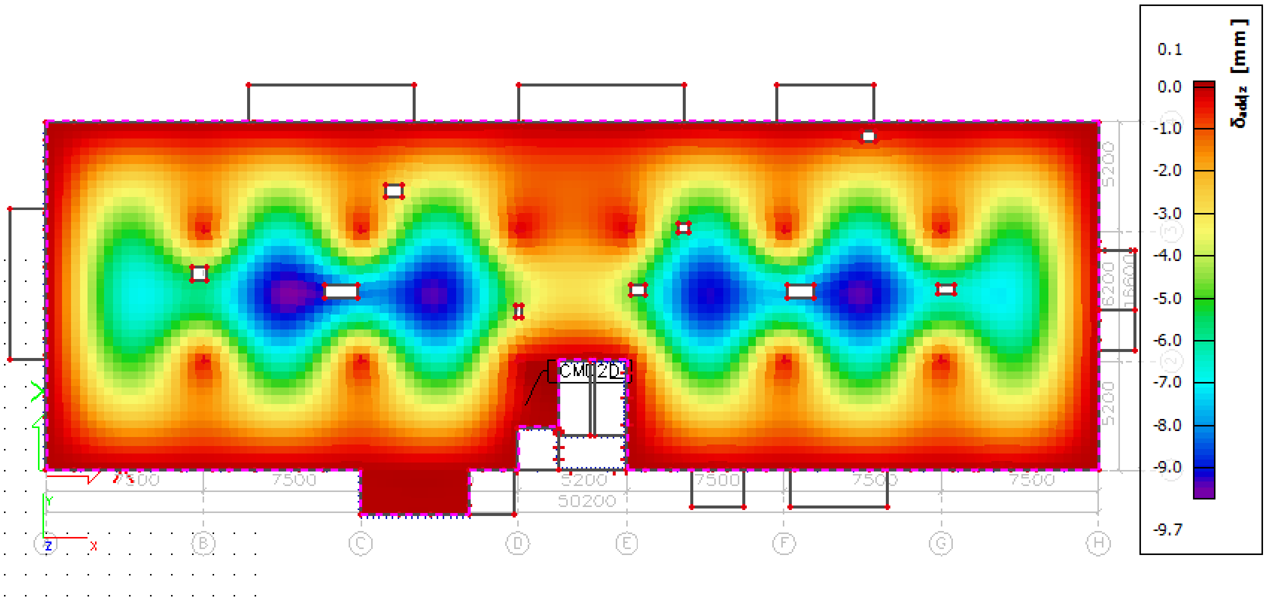


Fig. 34: SCIA results of additional deflection

In consequence, the design assumption of reduced slab depth can be released as the deflection does not exceeds the limitations and the slab depth can be kept as $h_s = 250mm$

2.2 Flexural reinforcement design:

To design bending reinforcement of the selected slab it is necessary to calculate the bending moments, therefore a set of combination of loads should be inserted to the software after loading the slab. the calculations are based on linear analysis,

The type of combinations is set as EN-ULS (STR/GEO) at ultimate limit state and with this type SCIA Engineer will generate combination automatically in accordance with Eurocode rules.

The internal forces obtained using result command and selecting the elementary design magnitude option permit to read 2D design bending moments on the selected slab, this method uses the local X and Y coordinate system of the current plate .

The values displayed in (Figs. 35.a ;35.b; 35.c ; 35.d) described as follows :

- m_xD +: Design bending moment in X axis direction of local coordinate system (LCS) for upper surface (+).

- m_xD –: Design bending moment in X axis direction of local coordinate system (LCS) for lower surface (-).

- m_yD +: Design bending moment in Y axis direction of local coordinate system (LCS) for upper surface (+).

- m_yD –: Design bending moment in Y axis direction of local coordinate system (LCS) for lower surface (-).

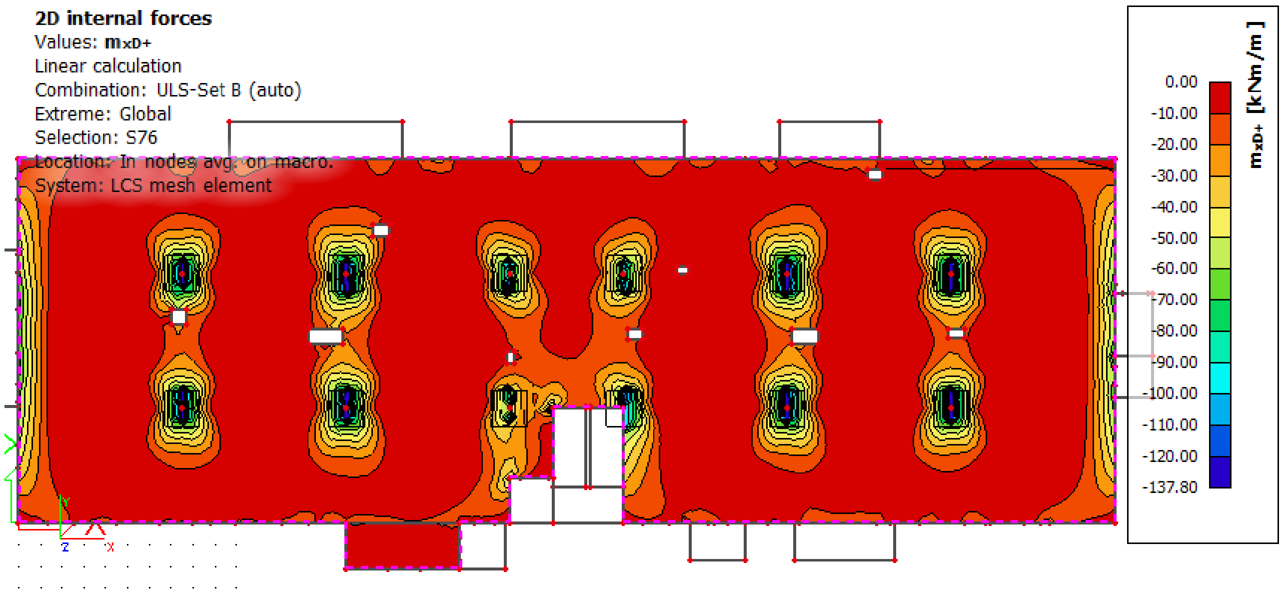


Fig. 35.a: Bending moments results of upper slab in x-direction

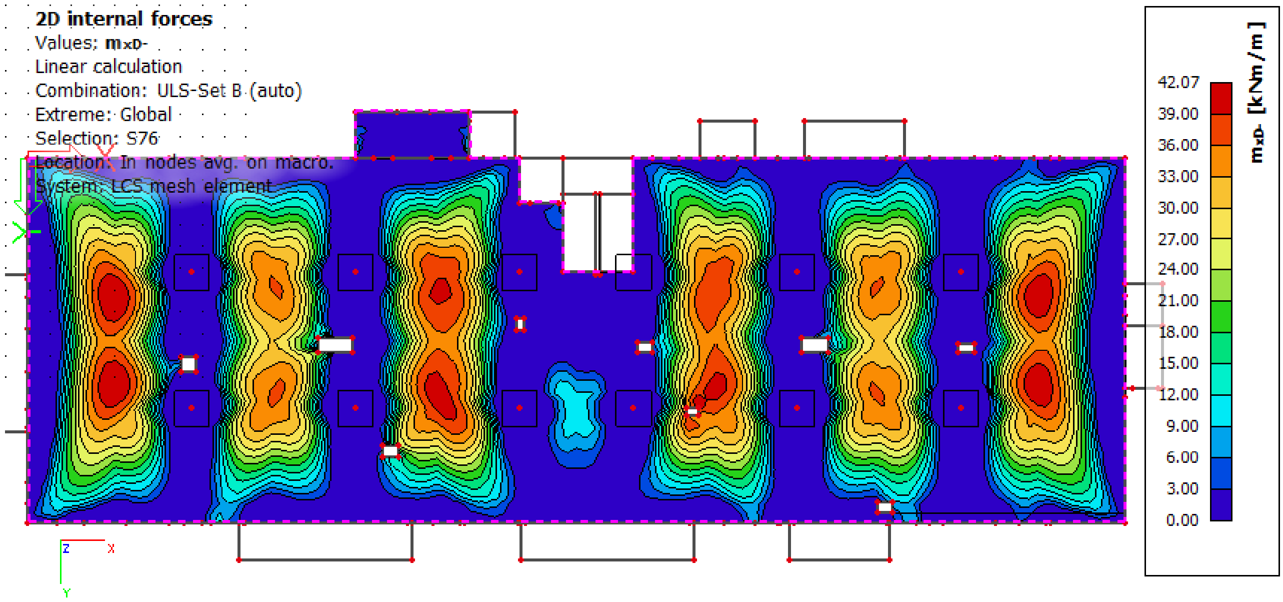


Fig. 35.b: Bending moments results of bottom slab in x-direction

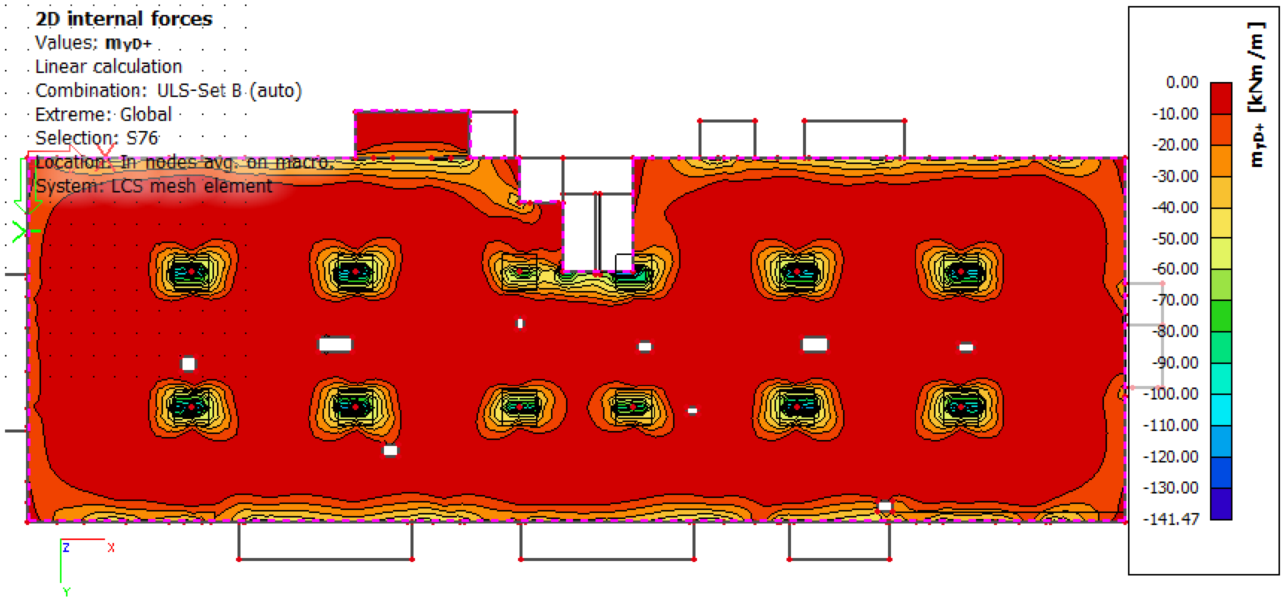


Fig. 35.c: Bending moments results of upper slab in y-direction

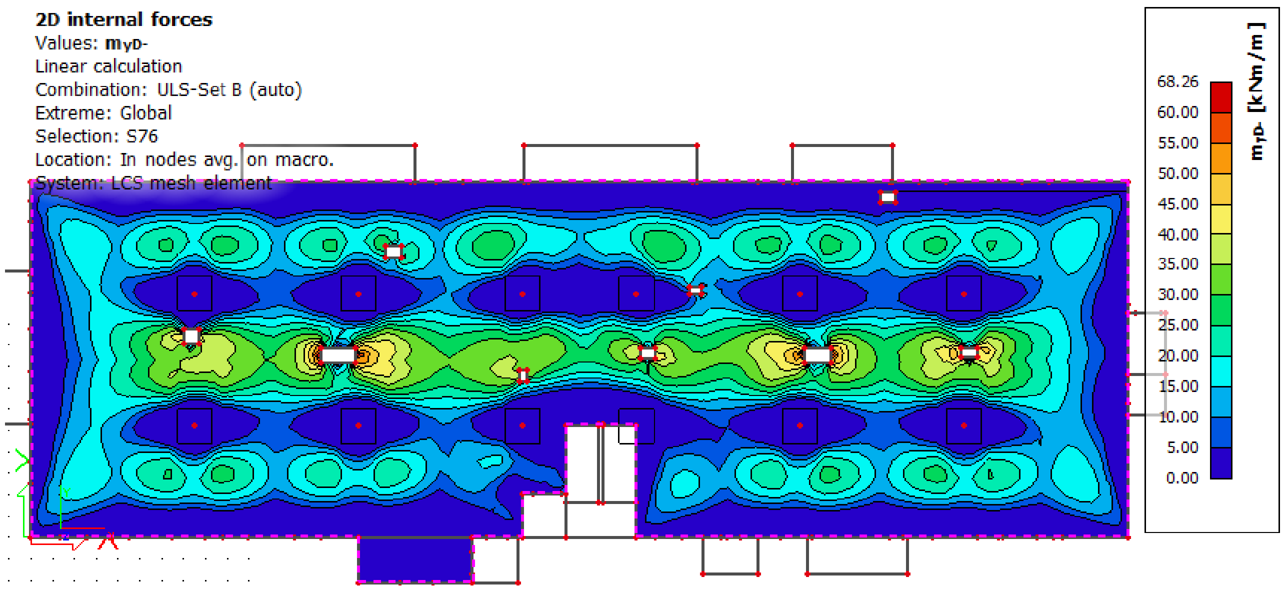


Fig. 35.d: Bending moments results of bottom slab in y-direction

2.2.1 Design of required reinforcement

Absolute extremes obtained from results were used for the design of required area of flexural reinforcements, the results are directly determined by the software through command *concret* > *Reinforcemnt design*

The results below obtained from SCIA evaluates the required reinforcement area in different position in the slab.

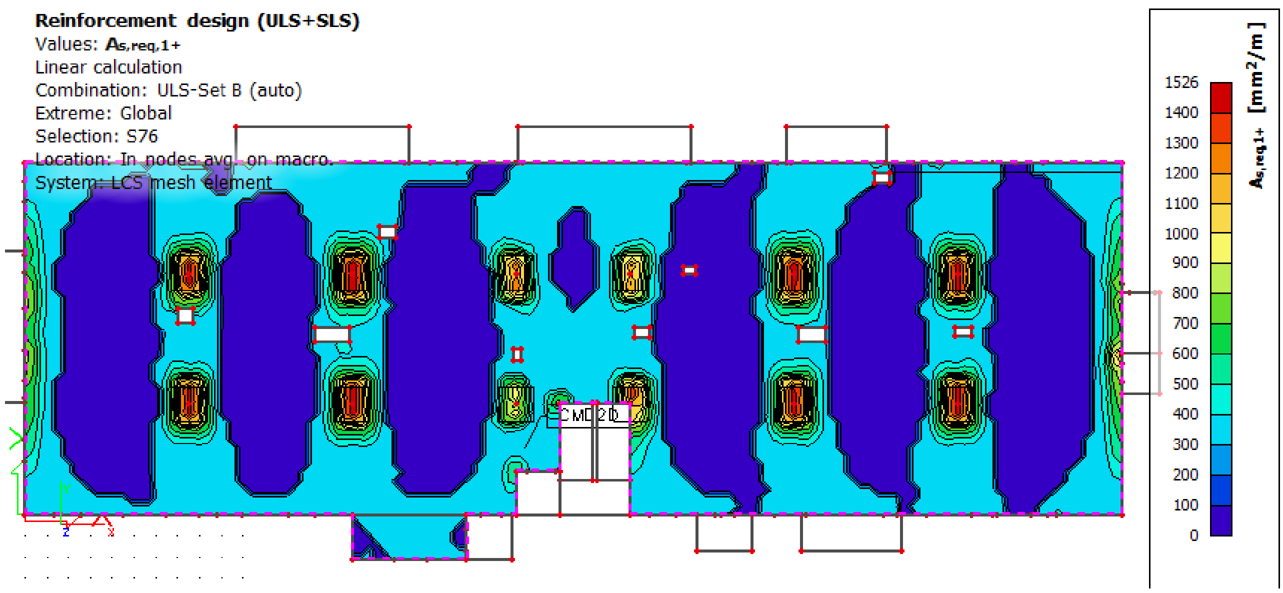


Fig. 36.a: required reinforcement area in upper slab x-direction

Fig.(36.a) represents the required reinforcements area $A_{s,req,1+}$ in x direction in upper part with highest value of $A_{s,req,1+} = 1526\ mm^2/m$ located in upper head of column.

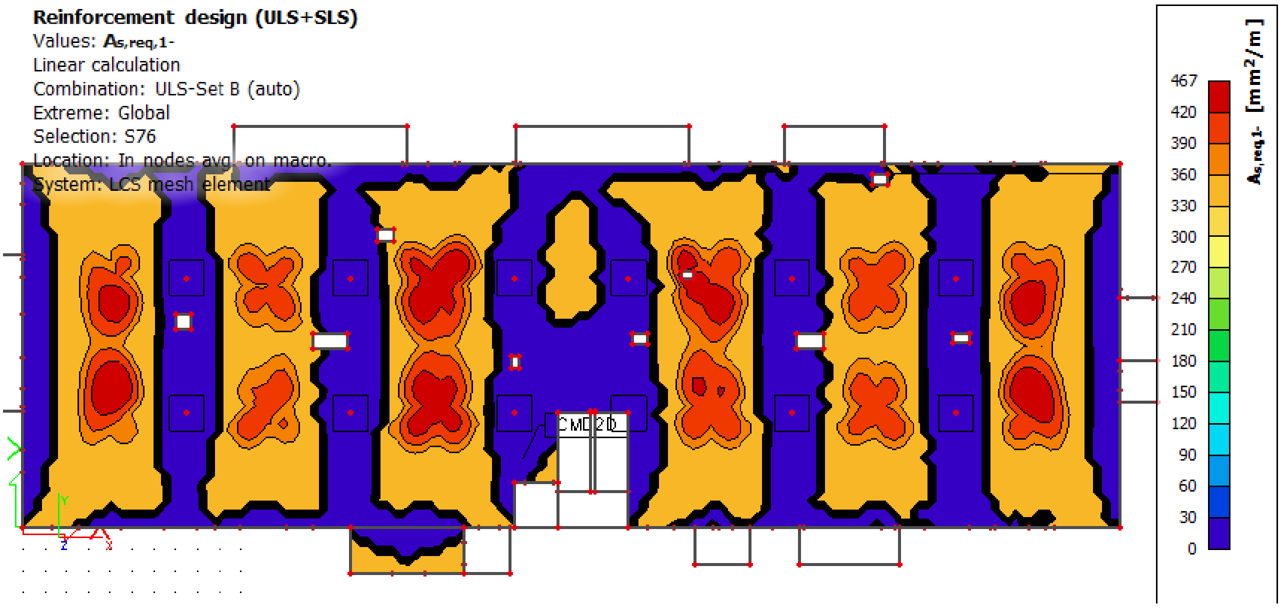


Fig. 36.b: required reinforcement area in bottom slab x-direction

Fig.(36.b) represents the required reinforcements area $A_{s,req,1-}$ in x direction in bottom part with highest value of $A_{s,req,1-} = 467\ mm^2/m$ located in mid span.

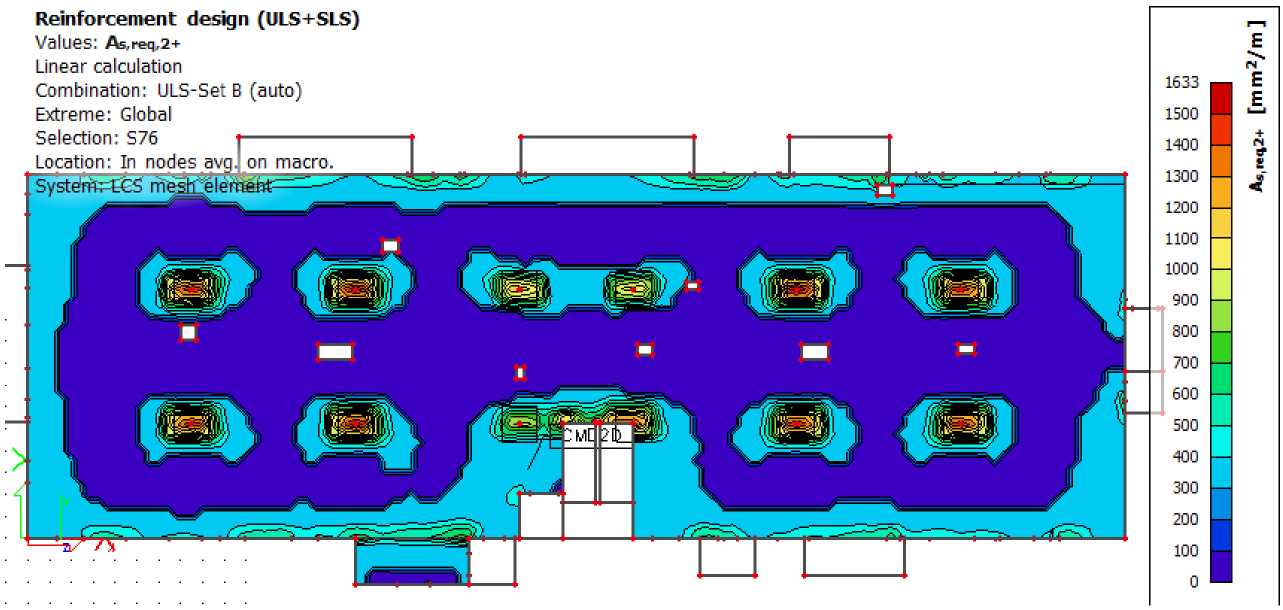


Fig. 36.c: required reinforcement area in upper slab y-direction

Fig.(36.c) represents the required reinforcements area $A_{s,req,2+}$ in y direction in upper part with highest value of $A_{s,req,1+} = 1633\ \mathrm{mm^2/m}$ located in the head of colum.

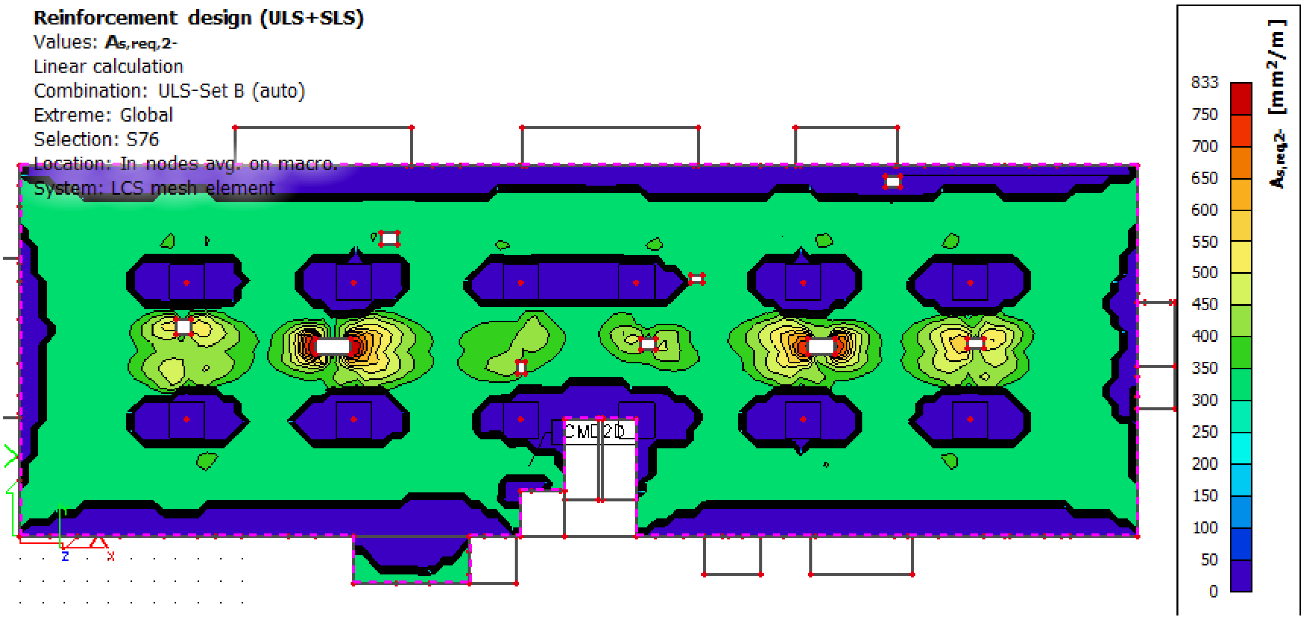


Fig. 36.d: required reinforcement area in bottom slab in y-direction

Fig.(36.d) represents the required reinforcements area $A_{s,req,2-}$ in y direction in lower part of the slab with highest value of $A_{s,req,2-} = 833\ \mathrm{mm^2/m}$ located in the span close to the opening between two columns.

In the design of reinforcement, it should hold that the area of provided reinforcement must be equal or greater than the required reinforcement and fulfill the condition for the minimum and maximum reinforcement area eq.(42) and (43)

$$A_{s,min} = \max\begin{Bmatrix} 0{,}26\,.\ \dfrac{f_{ctm}\,.b\,.\,d}{f_{yk}} \\ 1{,}3.\,10^{-3}.\,b.\,d \end{Bmatrix} \tag{42}$$

$$A_{s,max} = 0{,}04\,.\,b.\,h \tag{43}$$

f_{ctm} - Mean tensile strength

h - Cross section depth

Taking the design of reinforcement of bottom slab as an example:

For Bottom slab in x-direction all slab provided with basic reinforcement $\emptyset 10\ mm$ bars and spacing $s = 150\ mm$ given area $A_{s,prov,b} = 524\ \mathrm{mm^2/m}$

$$A_{s,prov,b} = 524\ \mathrm{mm^2/m} > \mathrm{A_{s,req,1-}} = 467\ \mathrm{mm^2/m}$$

$$\mathrm{A_{s,min}} = \max\begin{Bmatrix} 0{,}26\ .\ \frac{2{,}9\ .1000\ .225}{500} \\ 1{,}3.\ 10^{-3}.\ 1000.225 \end{Bmatrix} = 339\ \mathrm{mm^2/m}$$

$$\mathrm{A_{s,max}} = 0{,}04\ .\ \mathrm{b.h} = 0{,}0{,}4.1000.250 = 10000\ \mathrm{mm^2/m}$$

$$\mathrm{A_{s,min}} = 467\ \mathrm{mm^2/m} \leq A_{s,prov,b} = 524\ \mathrm{mm^2/m} \leq \mathrm{A_{s,max}} = 10000\ \mathrm{mm^2/m}$$

Therefore, the provided area of reinforcement is sufficient.

For Bottom slab in y-direction all slab provided with basic reinforcement Ø10/150 mm bars given area $A_{s,prov,b} = 524\ \mathrm{mm^2/m}$ and another additional reinforcement from the same type Ø10/150 mm $A_{s,prov,add} = 524\ \mathrm{mm^2/m}$ placed only to areas with high bending moment

$$A_{s,prov,tot} = A_{s,prov,b} + A_{s,prov,add} = 524 + 524 = 1048\ \mathrm{mm^2/m}$$

$$A_{s,prov,tot} = 1048\frac{mm^2}{m} > \mathrm{A_{s,req,2-}} = 833\ \mathrm{mm^2/m}$$

$$\mathrm{A_{s,min}} = 467\ \mathrm{mm^2/m} \leq A_{s,prov,b} = 1048\ \mathrm{mm^2/m} \leq \mathrm{A_{s,max}} = 10000\ \mathrm{mm^2/m}$$

As results the provided area of reinforcement is sufficient.

With the same principle, the slab designed for upper reinforcement in both directions and reinforcement drawings were performed.

2.3 Design of anchorage and lap length:

2.3.1 Anchorage length:

a) Bond stress

The reinforcing bars should be anchored to concrete safely transmitting the bond stress to the surface, therfore ultimate bond strengh shall be sufficient to prevent bond failure.the design value of the ultimate bond stress $\mathrm{f_{bd}}$ for ribbed bars may be taken as :

$$\mathrm{f_{bd}} = 2{,}25.\eta_1.\eta_2.\mathrm{f_{ctd}} \tag{44}$$

Where:

η_1 - *quality of the bond condition (taken as $\eta_1 = 1$ for good bond condition)*

η_2 - *effect of large bar diameters for $\emptyset \leq 32\ mm$ $\eta_2 = 1$*

f_{ctd} - *design tensile strength of concrete*

$$f_{ctd} = \alpha_{ct}.f_{ctk,0,05/\gamma_c} \tag{45}$$

α_{ct} - *Coefficient taking account of long-term effects and loading effects on the tensile strength of concrete (Rec $\alpha_{ct} = 1$)*

$f_{ctk,0,05}$ - 5% *fractile tensile strength*

$$f_{ctk,0,05} = 0,7\ .f_{ctm} \tag{46}$$

f_{ctm} - *mean tensile strength ($f_{ctm} = 2,9\ Mpa$)*

$$F_{ctk,0,05} = 0,7\ .f_{ctm} = 0,7\ .2,9 = 2,03\ Mpa$$

$$f_{ctd} = \alpha_{ct}.f_{ctk,0,05/\gamma_c} = 1,0.\frac{2,03}{1,5} = 1,35\ Mpa$$

Substituting all value to eq.(44), the bond stress is equal to :

$$f_{bd} = 2,25.\eta_1.\eta_2.f_{ctd} = 2,25.1,0.1,0.1,35 = 3,04\ \text{Mpa}$$

b) Basic anchorage lenght :

Assume maximum design stress in the bar $\sigma_{sd} = \frac{f_{yk}}{\gamma_s} = \frac{500}{1,15} = 435\ Mpa$

Bar diameter $\emptyset = 10\ mm$

For anchoring steel bars with diameter $\emptyset$ in concrete under design stress σ_{sd} the basic required anchorage lenght $l_{b,rqd}$ is defined as :

$$l_{b,rqd} = \left(\frac{\emptyset}{4}\right).\left(\frac{\sigma_{sd}}{f_{bd}}\right) \tag{47}$$

$$l_{b,rqd} = \left(\frac{\emptyset}{4}\right).\left(\frac{\sigma_{sd}}{f_{bd}}\right) = \left(\frac{10}{4}\right).\left(\frac{435}{3,04}\right) = 357,73\ m$$

Minimum anchorage lenght:

Anchorage lengh has to be equal at least to the minimum value $l_{b,min}$ where :

- <u>For anchorage in tension:</u>

$$l_{b,min} \geq \max[0{,}3.l_{b,rqd}, 10.\emptyset, 200\ mm] \quad (48)$$

$$l_{b,min} \geq \max[0{,}3.l_{b,rqd}, 10.\emptyset, 200\ mm] = \max[107{,}31; 100; 200\ mm] = 200\ mm$$

$$l_{b,min} = 200\ mm$$

<u>- For anchorage in compression :</u>

$$l_{b,min} \geq \max[0{,}6.l_{b,rqd}, 10.\emptyset, 200\ mm] \quad (49)$$

$$l_{b,min} \geq \max[0{,}6.l_{b,rqd}, 10.\emptyset, 200\ mm] = \max[214{,}64; 100; 200\ mm] = 214{,}64\ mm$$

$$\mathrm{l_{b,min}} = 220\ \mathrm{mm}$$

c) Design anchorage lenght:

The equation for design anchorage length taken as follows:

$$l_{bd} = \alpha_1.\alpha_2.\alpha_3.\alpha_4.\alpha_5.l_{b,rqd} \geq l_{b,min} \quad (50)$$

α_1 - *effect of shape of bar, in tension is taken as:*

- $\alpha_1 = 1$ *for straight bars*

- $\alpha_1 = 1$ *for bend bars where* $c_d \leq 3.\emptyset \rightarrow c_d = 20 < 3.10 = 30\ mm$

(The condition of adequate cover $c_d > 3.\emptyset$ *for other-than-straight bars in tension is defined in EN1992-1-1 Figure 8.3 [9])*

α_2 - *effect of minimum concrete cover in tension is taken as*

- $\alpha_2 = 1 - 0{,}15.\left(\frac{c_d-\emptyset}{\emptyset}\right) = 1 - 0{,}15.\left(\frac{20-10}{10}\right) = 0{,}85$ *for straight bars*

- $\alpha_2 = 1 - 0{,}15.\left(\frac{c_d-3.\emptyset}{\emptyset}\right) \leq 1$ *for bend bars*$\rightarrow \alpha_2 = 1 - 0{,}15.\left(\frac{20-3.10}{10}\right) = 1{,}15 \leq 1$ *then* $\alpha_2 = 1$

α_3 - *effect of confinement by transverse reinforcement not welded to main reinforcement (taken as* $\alpha_3 = 1$ *(conservative value with* $k = 0$ *)*

α_4 - *effect of confinement by welded transverse reinforcement (N/A* $\alpha_4 = 1{,}0$ *)*

α_5 - *effect of confinement by transverse pressure (taken as* $\alpha_5 = 1{,}0$ *conservative value)*

Substituting to eq.(50) anchorage length is :

<u>For straight bars in tension:</u>

$$l_{bd} = \alpha_1 . \alpha_2 . \alpha_3 . \alpha_4 . \alpha_5 . l_{b,rqd} \geq l_{b,min} = 1 . 0.85 . 1 . 1 . 1 . 357{,}73 = 304{,}07\ mm \geq 200\ mm$$

Design: $l_{bd} = 310\ mm$

<u>For bend bars in tension:</u>

$$l_{bd} = 1 . 1 . 1 . 1 . 1 . 357{,}73 = 357{,}73\ mm \geq 200\ mm$$

Design: $l_{bd} = 360\ mm$

For anchorage in compression, the quality of bond condition still considered to be good as it satisfies the case (b) in fig. 8.2 EN 1992-1-1 [9].

- $\alpha_1 = \alpha_2 = \alpha_3 = \alpha_4 = \alpha_5 = 1$

<u>Anchorage length in compression:</u>

$$l_{bd} = \alpha_1 . \alpha_2 . \alpha_3 . \alpha_4 . \alpha_5 . l_{b,rqd} \geq l_{b,min} = 1 . 1 . 1 . 1 . 1 . 357{,}73 = 357{,}73\ mm \geq 220\ mm$$

Design: $l_{bd} = 360\ mm$

2.3.2 Lapping length:

Lapping lenght is the amount of overlapping between two bars and calculations are very similiar to anchorage.

As the design procedure of lapping is almost the same as the anchorage with few exceptions, design of lapping can be evaluated as follows:

a) Minimum lapping length:

The provided lap length should be equal at least to $l_{0,min}$ when there are no other applicable limitations:

$$l_{0,min} \geq \max[0{,}3 . \alpha_6 . l_{b,rqd}, 15 . \emptyset, 200\ mm] \tag{51}$$

$$l_{0,min} \geq \max[160{,}1; 150; 200\ mm] = 200$$

b) Design lap length:

$$l_0 = \alpha_1 . \alpha_2 . \alpha_3 . \alpha_5 . \alpha_6 . l_{b,rqd} \geq l_{0,min} \tag{52}$$

α_6 - coefficient expressing amount of lapped reinforcement (taken as $\alpha_6 = 1{,}5$)

Substituting to eq.(52) lap length is :

For straight bars in tension:

$$l_0 = l_{bd} . \alpha_6 \geq l_{0,min} = 304{,}07.1{,}5 = 456{,}12\ mm > 200\ mm$$

Design: $l_0 = 500\ mm$

For bend bars in tension:

$$l_0 = l_{bd} . \alpha_6 \geq l_{0,min} = 357{,}73\ .1{,}5 = 536{,}60\ mm > 200\ mm$$

Design: $l_0 = 550\ mm$

Lapping length in compression:

$$l_0 = l_{bd} . \alpha_6 \geq l_{0,min} = 357{,}73\ .1{,}5 = 536{,}60\ mm > 200\ mm$$

Design: $l_0 = 550\ mm$

As result, taking the extreme obtained values for the design as follows:

-Anchorage length design : $l_{bd} = 360\ mm$

-Lapping length design : $l_0 = 550\ mm$

2.4 Punching design according to Eurocode:

2.4.1 check of punching resistance:

The total shear force acting on the investigated column (B3) see (fig.26) is:

$$V_{Ed} = 625{,}43\ KN$$

in the first step its necessary to check if the capacity of concrete is sufficient to transfer shear forces from the slab to the column,therefore the procedure should be carried as follows.

The effective depth is taken as the average value from both directions (longitudinal and lateral) which is obtained from :

$$d_x = h - c - 1{,}5.\emptyset$$

$$d_x = 250 - 20 - 1{,}5.10 = 215\ mm$$

$$d_y = h - c - 0{,}5.\emptyset$$

$$d_y = 250 - 20 - 0{,}5.10 = 225\ mm$$

$$d = (d_x + d_y).0{,}5$$

$$d = (215 + 225).0{,}5 = 220\ mm$$

The maximum punching shear resistance is :

$$V_{Rd,max} = 0{,}5.v.f_{cd} = 0{,}5.0{,}53.20 = 5{,}3\ Mpa$$

Where:

$$v = 0{,}6.\left(1 - \frac{f_{ck}}{250}\right) = 0{,}6.\left(1 - \frac{30}{250}\right) = 0{,}53$$

Perimeter of column is

$$u_0 = 4.350 = 1400\ mm$$

The factor $\beta = 1{,}15$ since the location of the column is considered to be internal. The value of shear force related to the perimeter of column is then equal to :

$$V_{Ed,u0} = \frac{\beta.V_{Ed}}{u_0.d} = \frac{1{,}15.625{,}43}{1400.220} = 2{,}34\ Mpa \qquad (5)$$

Then the following condition is satisfied:

$$V_{Ed,u0} = 2{,}34\ Mpa < V_{Rd,max} = 5{,}3\ Mpa$$

With this result the slab is able to transfer the shear stress.

The second step was to check whether the concrete mass of the slab is capable of resisting shear stress at the critical perimeter located $2.d$ from the loaded area, calculated as;

$$u_1 = 4.a + 2.\pi.2.d = 4.350 + 2.\pi.2.220 = 4165\ mm$$

The shear resistance value of concrete is given by:

$$V_{Rd,c} = C_{Rd,c}\,.\,k\,.\left(100.\,\rho_l\,.\,f_{ck}\right)^{1/3} \geq V_{min} \tag{3}$$

Where:

$$C_{Rd,c} = \frac{0,18}{\gamma_c} = \frac{0,18}{1,5} = 0,12$$

$$k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{220}} = 1,95 < 2,0; k = 1,95$$

$$\rho = \sqrt{\rho_{lx}.\,\rho_{ly}} \leq 0,02\,,$$

$$\rho_{ly} = \frac{1655}{1000.220} = 0,00784$$

$$\rho_{lx} = \frac{1526}{1000.220} = 0,00693$$

$$\rho = \sqrt{0,00784.0,00693} = 0,00737 < 0,02$$

Then the design value of shear resistence:

$$V_{Rd,c} = 0,12\,.1,95\,.\,(100.\,0,00737\,.\,30)^{1/3} = 0,656\ Mpa$$

Further its necessary to check design value within the limitation:

$$\mathrm{V_{min}} = 0,035.\,\mathrm{k}^{3/2}.\,\mathrm{f_{ck}}^{1/2} \tag{4}$$

$$V_{min} = 0,035.\,1,95^{3/2}.\,30^{1/2} = 0,522\ Mpa$$

As result:

$$V_{Rd,c} = 0,656\ Mpa > V_{min} = 0,522\ Mpa$$

The value of shear force at critical perimeter u_1 is calculated as

$$v_{Ed} = \beta\,.\,\frac{V_{Ed}}{u_1\,.\,d} \tag{1}$$

$$v_{Ed} = 1,15\,.\,\frac{625,43}{4165\,.220} = 0,785\ Mpa$$

Since the following condition is not satisfied:

$$v_{Ed} = 0,785\ Mpa < v_{Rd,c} = 0,656 Mpa$$

Then the punching shear reinforcements is required.

2.4.2 Design of punching reinforcement:

Stud Rail were selected for the design as they show efficiency for punching shear and easy to install at the construction site.

For check of the shear reinforcement, the length of perimeter, where the condition $v_{Ed} = V_{Rd,c}$ is satisfied, u_{out} must be calculated as :

$$u_{out} = \frac{\beta . V_{Ed}}{V_{Rd,c} . d} \tag{7}$$

$$u_{out} = \frac{1{,}15.625{,}43}{0{,}656 .220} = 4{,}983\ m$$

The length of perimeter shows that shear reinforcement should be provided at the distance from the face of column of $u_{out} = 4{,}983\ m$

Simultaneously, radius of outer perimeter can be determined:

$$r_{out} = \frac{u_{out}}{2.\pi} = \frac{4{,}983}{2.\pi} = 793\ mm$$

Estimated number of rails can be taken as 8 rails with diameter $\emptyset = 10\ mm$ and spacing

$s = 100\ mm$

Cross-sectional area of studs in one perimeter:

$$A_{sw} = n. A_{sw,1} \tag{7.1}$$

Where :

$A_{sw,1}$ - *Cross-sectional area of one stud*

$$A_{sw} = 12. \frac{\pi . 10^2}{4} = 942\ mm^2$$

The distances between headed shear studs shown in fig. (37) are calculated as follows:

Radius of last perimeter stud must be more than $(r_{out} - 1{,}5d)$

$$r_{out} - 1{,}5. d = 793 - 1{,}5.220 = 463\ mm$$

First stud is located at a distance $(0{,}3 - 0{,}5). d$ behind the face of column

$$(0{,}3 - 0{,}5). d = (0{,}3.220 - 0{,}5.220) = (66\ mm - 110\ mm) \rightarrow s_1 = 80\ mm$$

Last stud not more than $(1{,}5. d)$ from u_{out}

$$1{,}5.d = 1{,}5.220 = 330\ mm$$

Spacing of intermediate studs $s_r \leq 0{,}75.d$

$$s_r \leq 0{,}75.220 = 165 \rightarrow s_r = 150\ mm$$

Spacing of rails: $s_t \leq 2.d$

$$s_t \leq 2.220 = 440\ mm \rightarrow s_t = 400\ mm$$

Spacing of rails in perimeter u_1 should be less or equal to $1{,}5.d$

$$1{,}5.d = 330\ mm$$

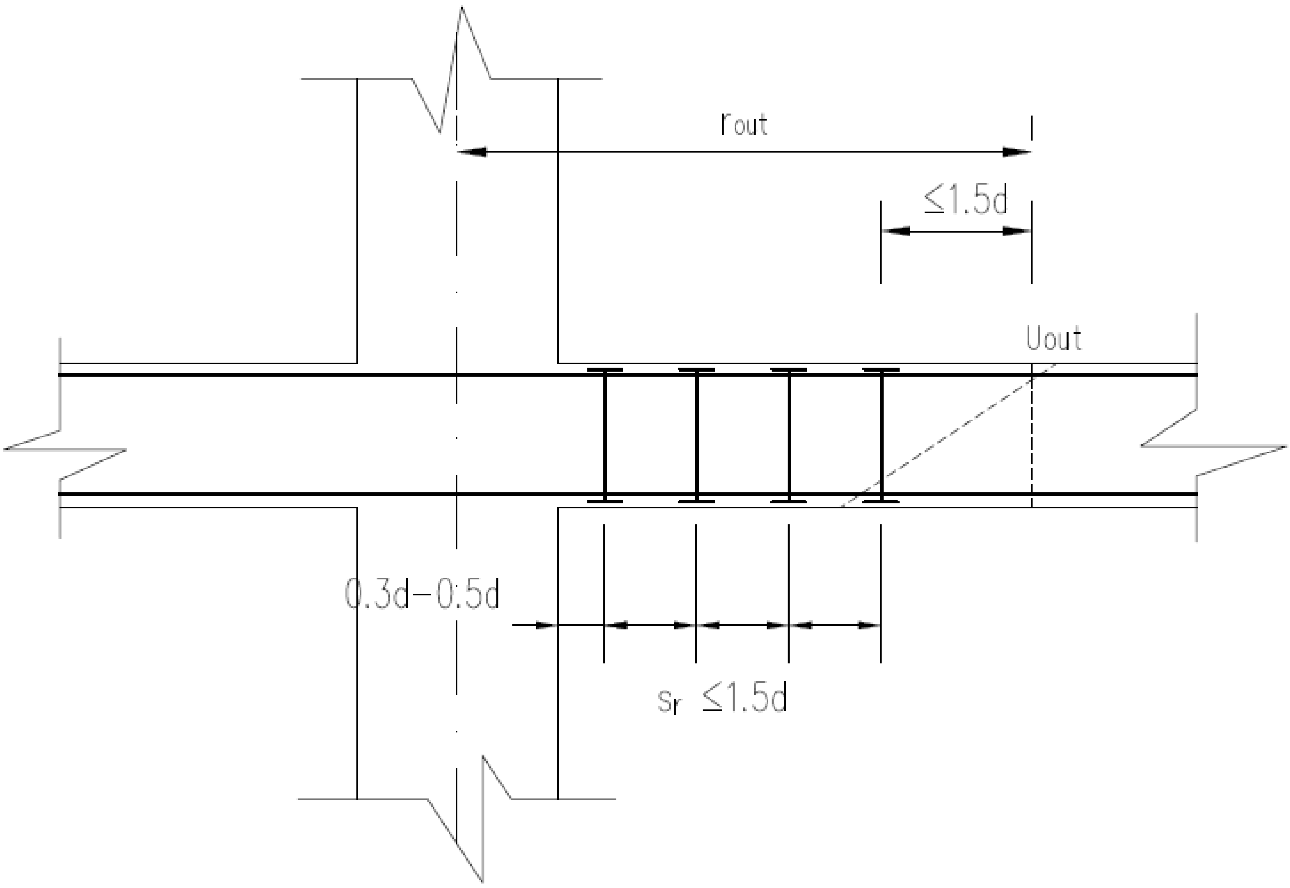


Fig. 37: section view of spacing between studs

The maximum resistance of concrete with shear head reinforcement can be checked as follows:

$$\mathrm{V_{Ed}} \leq \mathrm{k_{max}}.\mathrm{v_{Rd,c}} \qquad (7.2)$$

Where k_{max} is the coefficient of maximum resistance (taken as 1,80 for studs)

$$\mathrm{V_{Ed}} = 0{,}785\ \mathrm{Mpa} \leq \mathrm{k_{max}}.\mathrm{v_{Rd,c}} = 1{,}80.0{,}656 = 1{,}18\ \mathrm{Mpa}$$

Therefore, shear punching reinforcement can be anchored in concrete sufficiently.

Further the design shear capacity including shear reinforcement for punching is calculating from the following:

$$V_{Ed} < V_{Rd.c+s} = V_{Rd,cs} + V_{Rd,s} = 0{,}75 \,.\, V_{Rd,c} + 1{,}5 \,.\, \frac{d}{s_r} \,.\, A_{sw} \,.\, f_{ywd,ef} \cdot \frac{1}{u_1 . d} \,.\, sin\,\alpha \leq \mathrm{k_{max}} . \mathrm{v_{Rd,c}} \quad (6)$$

$$f_{ywd,ef} = 250 + 0{,}25.\,d \leq f_{ywd} = \frac{f_{yk}}{\gamma_s}$$

$$f_{ywd,ef} = 250 + 0{,}25.220 = 305\ Mpa \leq f_{ywd} = \frac{f_{yk}}{\gamma_s} = \frac{500}{1.15} = 435\ Mpa$$

$$V_{Rd,c+s} = 0{,}75.0{,}656 + 1{,}5.\frac{220}{150}.\,942.305.\frac{1}{4165.220}.\sin 90 = 1{,}18\ Mpa > V_{Ed} = 0{,}785\ Mpa$$

$$V_{Rd,c+s} = 1{,}18\ \mathrm{Mpa} \leq \mathrm{k_{max}} . \mathrm{v_{Rd,c}} = 1{,}80.0{,}656 = 1{,}18\ \mathrm{Mpa}$$

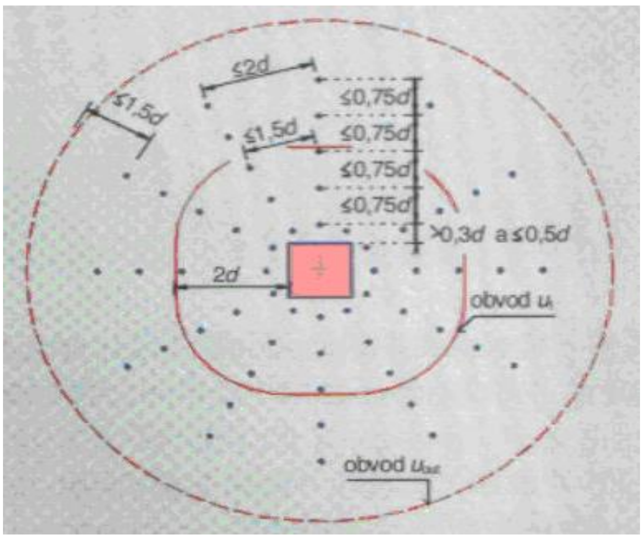
In consequence the shear resistance of shear reinforcement arrangement (Fig.38) is sufficient to transfer effects of loads from the slab to the column.

The amount of shear reinforcement can be verified using punching reinforcement ratio where:

$$\rho_{sw} = 1{,}5.\frac{A_{sw,1}}{s_r . s_t} \geq \rho_{sw,min} = 0{,}08.\frac{\sqrt{f_{ck}}}{f_{yk}} \quad (7.4)$$

ρ_{sw} - punching reinforcement ratio of slab

$\rho_{sw,min}$ - minimum punching reinforcement ratio



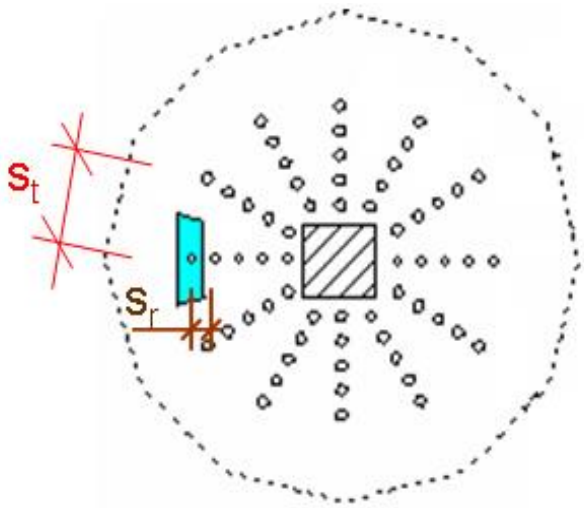


Fig. 38:plan view distribution of shear headed studs [33]

$$\rho_{sw} = 1{,}5.\frac{78{,}54}{150.400} = 0{,}00196 \geq \rho_{sw,min} = 0{,}08.\frac{\sqrt{30}}{500} = 0{,}00087$$

Then the quantity of reinforcement used is safficent for the design .

The design of punching shear reinforcement illustrated in Fig.(39)

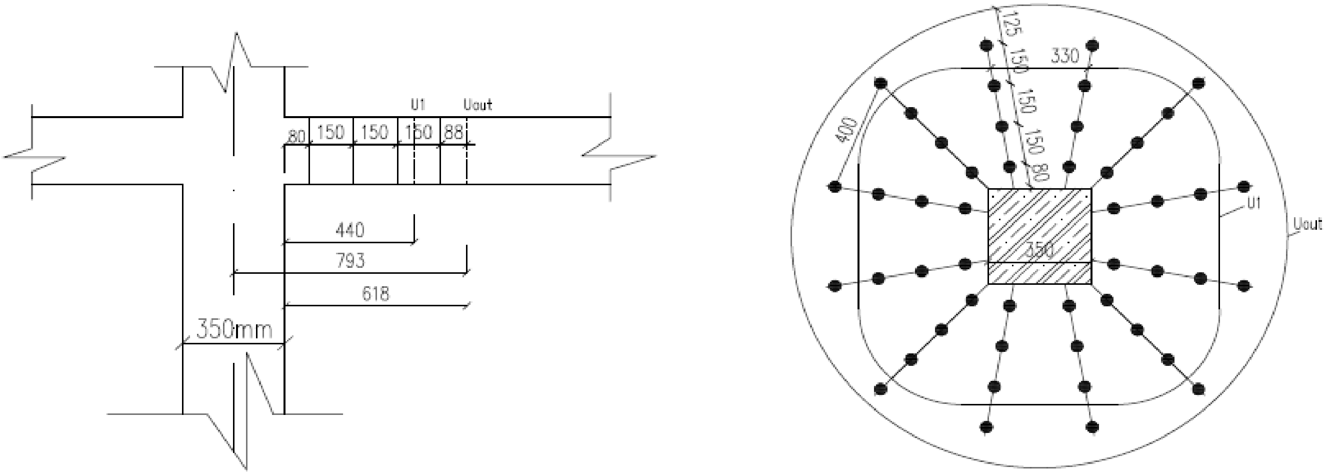


Fig. 39:Distribution of shear studs by EC2

2.5 Punching design according to ACI:

2.5.1 check of punching resistance:

For the design analogy between EC 2 and ACI-318 the same materials were used for design punching shear reinforcement with ACI, concrete C30/37 with design strength $f'_c = 30\ Mpa$ and steel B500A with yield strength $f_y = 500\ Mpa$

The shear stress V_u should be calculated at the critical perimeter b_0 and compared with shear strenght of concrete:

$$v_u \leq \emptyset.v_c \tag{8}$$

At the critical shear perimeter b_0 shear stress v_u is computed with contribution of unbalanced moment as follows:

$$v_u = \frac{v_u}{b_0.d} + \frac{\gamma_{v1}.M_{u1}}{J_{c1}} + \frac{\gamma_{v2}.M_{u2}}{J_{c2}} \tag{8.1}$$

Where:

$c_1 c_{2,}$ - dimensions of column $c_1 = c_2 = 350\ mm$,

$b_{1,} b_{2,}$ - width and depth of critical sections $b_1 = b_2 = c_1 + d = 350 + 220 = 570\ mm,$

γ_{v1}, γ_{v2} - fraction of the moment transferred by shear

$$\gamma_{v1} = \gamma_{v2} = (1 - \gamma_f) = \left(1 - \frac{1}{1 + (2/3).\sqrt{\frac{b_1}{b_2}}}\right) = \left(1 - \frac{1}{1 + (2/3).\sqrt{\frac{570}{750}}}\right) = 0{,}4$$

J_c - property of assumed critical section analogous to polar moment of inertia

$$J_{c1} = J_{c2} = \frac{b_1.d.(b_1 + 3.b_2) + d^3}{3} = \frac{570.220.(570 + 3.570) + 220^3}{3} = 9{,}885.10^7 mm^3$$

M_u - unbalanced column moment, $M_{u,1} = 11{,}31\ KN.m$, $M_{u,2} = 30{,}48\ KN.m$

(The values of $M_{u,1}$ and $M_{u,2}$ were taken from analysis of slab)

b_0 -Critical shear perimeter $b_0 = \sum c + 4.d = 4.350 + 4.220 = 2280\ mm$

Substituting the values to equation (8):

$$v_u = \frac{625{,}43.10^3}{2280.220} + \frac{0{,}4.11{,}31.10^6}{9{,}885.10^7} + \frac{0{,}4.30{,}48.10^6}{9{,}885.10^7} = 1{,}416\ Mpa$$

The minimum value of shear strength of concrete should be taken from these equations:

$$v_c = 0{,}17.\left(1 + \frac{2}{\beta_c}\right).\sqrt{f'_c} \quad \text{[Mpa]} \tag{9.a}$$

$$v_c = 0{,}083.\left(2 + \frac{\alpha_s\, d}{b_0}\right).\sqrt{f'_c} \quad \text{[Mpa]} \tag{9.b}$$

$$v_c = 0{,}33.\sqrt{f'_c} \quad \text{[Mpa]} \tag{9.c}$$

$$v_c = min \left\{ \begin{array}{c} v_c = 0{,}17.\left(1 + \frac{2}{1}\right).\sqrt{30} = 2{,}793\ Mpa \\ v_c = 0{,}83.\left(2 + \frac{40.220}{2280}\right).\sqrt{30}.2280.220 = 2{,}664\ Mpa \\ v_c = 0{,}33.\sqrt{30} = 1{,}807\ Mpa \end{array} \right\}$$

$$v_u = 1{,}416\, Mpa \;\leq\; \emptyset . v_c = 0{,}75.1{,}807 = 1{,}356\, Mpa$$

As the condition not satisfied, at a perimeter b_0 the slab has an inadequate concrete shear strength therefore punching shear reinforcement should be provided.

2.5.2 Design of punching reinforcement:

In principle the shear resistance shall be checked based on equation (10):

$$V_u \;\leq\; \emptyset . V_n \qquad \text{[Mpa]} \tag{10}$$

The maximum shear resistance provided by concrete and bars, wires, or single-multiple leg stirrup Should be checked where:

$$v_u \;= 1{,}416\, Mpa \leq\; \emptyset . v_{n,max} = 0{,}75.0{,}5.\sqrt{f_c{'}} = 0{,}375.\sqrt{30} = 2{,}05\, Mpa \tag{10.1}$$

As result punching shear reinforcement mentioned above might be sufficient for the design hence, to match the similarity with EC2 shear studs can be introduced and checked for its maximum resistance as follows:

$$v_u \;= 1{,}416\, Mpa \;\leq\; \emptyset . v_{n,max} = 0{,}75.0{,}67.\sqrt{f_c{'}} = 0{,}375.\sqrt{30} = 2{,}71\, Mpa \tag{10.2}$$

In consequence studs punching shear reinforcement can be used for the design.

In case of using shear studs reinforcement the maximum shear resistance provided by concrete only is written as:

$$v_c = 0{,}25.\sqrt{f_c{'}} = 1{,}369\, Mpa \tag{10.3}$$

For one peripheral line use the assumed shear studs can be taken for 8 rails, diameter $\emptyset\ 10\ mm$ spacing 100 mm

The required area of shear studs can be determined as:

$$A_v = \frac{(V_u - \emptyset . V_c).b_0 . s_{stud}}{\emptyset . f_{y,stud}} = \frac{(1{,}416 - 0{,}75.1{,}369).2280.100}{0{,}75.500} = 237\, mm^2 \tag{13}$$

Design check : $\qquad A_v = 237\, mm^2 < A_{v,prov} = 8.\frac{\pi . d^2}{4} = 628\, mm^2$

Further, the contribution provided by shear studs reinforcement should be greater than:

$$v_s = \frac{A_{v,prov} . \emptyset . f_{y,stud}}{b_0 . s_{stud}} = \frac{628.0{,}75.500}{2280.100} = 1{,}033\, Mpa \tag{12}$$

$$v_s = 1{,}033\, Mpa \geq \emptyset . 0{,}17.\sqrt{f_c{'}} = 0{,}75.0{,}17.\sqrt{30} = 0{,}698\, Mpa \tag{12.1}$$

Since the condition is satisfied it is necessary to check shear strength resistance from both shear reinforcement and concrete as follows:

$$V_u \leq \emptyset . V_n$$

$$V_n = V_c + V_s \leq 0{,}67.\sqrt{f_c{'}} \tag{11.1}$$

$$v_u = 1{,}416\ Mpa \ \leq \emptyset . v_n = \emptyset .(v_c + v_s) = 0{,}75.(1{,}369 + 1{,}033) = 1{,}801\ Mpa$$

And shear stress V_n is limited to:

$$v_n = 2{,}402 \leq v_{n,max} = 0{,}67.\sqrt{f_c{'}} = 0{,}67.\sqrt{30} = 3{,}669\ Mpa$$

In results the provided type of punching shear reinforcement is sufficient for resisting punching shear.

The length of critical section outside slab shear reinforcement from column face can be evaluated as:

$$a = \frac{\frac{V_u . b_0}{0{,}17.\emptyset.\sqrt{f_c{'}}} - 2.(c_1 + c_2)}{4.\sqrt{2}} = \frac{\frac{1{,}416.2280}{0{,}17.0{,}75.\sqrt{30}} - (2.(350+350))}{4.\sqrt{2}} = 570\ mm \tag{13.1}$$

Arrangement of shear studs is illustrated in (Fig.40), spacing between rails and studs is set as follows:

Spacing of intermediate studs : $s_0 \leq 0{,}5.d = 110\ mm \rightarrow s_0 = 80\ mm$

Spacing between rails: $s \leq 2.d = 440\ mm \rightarrow s = 350\ m$

Spacing of last stud $s_r \leq 0{,}5.d = 110\ mm \rightarrow s_r = 60\ m$

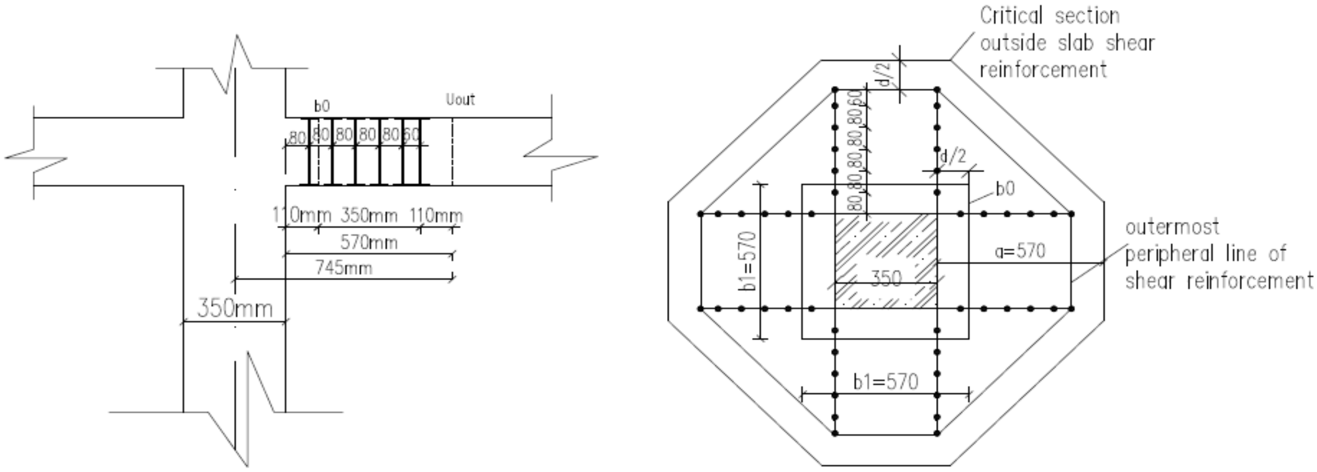


Fig.40:Distribution of shear studs by ACI

2.6 Comparison of results:

At first glance EC2 and ACI 318 appears to be different for design punching shear reinforcement, however, the principle is the same, an obvious dissimilarity from designing punching shear reinforcement essentially is the location of critical perimeter, where EC2 takes the control perimeter 2.d away from the face of the column of loaded area, while ACI 318 consider only critical perimeter at 0.5.d away from column area, in the design procedures ACI 318 uses principle of taking the minimum shear resistance provided by concrete from three different equations, while in EC2 the shear resistance of concrete is set to the minimum value $V_{Rd,c} = C_{Rd,c}\,.\,k\,.\left(100.\,\rho_l\,.\,f_{ck}\right)^{1/3} \geq V_{min}$ taking the contribution of flexural reinforcement ratio and size effect of effective depth d. The stress in shear reinforcement in EC2 is conservative and limited than ACI 318.

From the computed equations and conditions, the slab shows an inadequate concrete resistance for punching with both standards, with the use of headed shear studs reinforcement in both methods, the results shows that the length of the outer shear perimeter where reinforcements are no more required is relatively close $r_{out} = 793\,mm$ for EC2 and $745\,mm$ for ACI-318. Regardless to the spacing that is applied between studs for each approach, the amount of punching shear reinforcement is various within the outer perimeter yet the extension of reinforced shear zone fairly the same

3 Conclusion:

The objective of the thesis was to provide a brief review of flat slab and its functionality and to go through the mechanism and the fundamentals cause of punching shear failure. A review of punching shear design was carried out with different code provisions EC2 ACI 318 and FIB MODEL 2010, similarly shear reinforcement types for new and existing building were introduced

Among the things that were conducted in this part; flat slabs have a wide range of use in construction industry owing to ease and speed of implementation , the system considered to be efficient giving architectural flexibility due to the disappearance of beams, however, from structural point of view punching shear failure can occur when the value of shear stress exceeds the maximum resistance of concrete, punching shear reinforcement such as headed shear studs, stirrups, shear band reinforcement etc. can be provided to avoid the collapse of structure.

There are different standards can be used to design punching shear reinforcement, these provisions are based on same approach, typically to evaluate shear stress in the critical perimeter.

In the structural part the aim was to asses two different procedures for design punching shear reinforcement using EC2 and ACI 318, in the beginning the preliminary design of a selected slab was computed to determine the structural element dimensions .afterward the slab model was taken for the analysis with the use of SCIA software,. Consequently, the deflection of slab was checked, and flexural reinforcement were designed based on the obtained results from the software, in line with the EC2 and ACI 318-08 standards punching shear design was compared.

From a personal point of view the design with ACI-318 seemed to be easier as the approach sets various limitations and computation takes less time, nevertheless there is no agreement can be taken as to which of these codes is better since they have very similar approach.

4 Reference:

[1] International Journal of Advance Engineering and Research Development Volume 5, Issue 02, February -2018

[2] International Research Journal of Engineering and Technology (IRJET) e-ISSN: 2395-0056 Volume: 06 Issue: 05 May 2019 www.irjet.net p-ISSN: 2395-0072 © 2019, IRJET | Impact Factor value: 7.211 | ISO 9001:2008 Certified Journal | Page 7629 A Technical Approach to Flat Slab

[3] web source: https://taxonomy.openquake.org/index.php/terms?start=120

[4] L. Allen, A.W. Charleson, S. Brzev, C. Scawthorn (2013), Glossary for the GEM Building Taxonomy, GEM Technical Report 2013-03 V1.0.0, 536 pp., GEM Foundation, Pavia, Italy, doi: 10.13117/GEM.EXPMOD.TR2013.03.

[5] Web source :Ultracon Structural System :http://www.utraconindia.com/advantages_of_post-tensioning.html

[6] Mashhour Ghoneim and Mahmoud El-Mihilmy, "Design of reinforced concrete structures", 2nd Edition, Vol.2, 2008.

[7] Marko Bartolac, Domagoj Damjanović, Ivan Duvnjak, "Punching strength of flat slabs with and without shear reinforcement", GRAĐEVINAR 67 - 8, 771-786, 2015

[8] Wood, J.G.M.: Pipers Row Car Park Wolverhampton: Quantitative Study of the Causes of the Partial Collapse on 20th March 1997.

[9] EN 1992-1-1 (2004) (English): Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings [Authority: The European Union Per Regulation 305/2011,Directive 98/34/EC, Directive 2004/18/EC].

[10] Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary an ACI Standard Reported by ACI Committee 318

[11] Fédération Internationale du Béton (fib): Shear and punching shear in RC and FRC elements, fib Bulletin 57, 2010.

[12] Article MC2010: The Critical Shear Crack Theory as a mechanical model for punching shear design and its application to code provisions

[13] HDB 17-E Halfen punching shear reinforcement and shear reinforcement

[14] Punching shear of structural concrete slabs: Technical report Honoring Neil M. Hawkins American concrete institute

[15] Punching resistance of internal slab-column connections with double-headed shear studs volume 6, Number 5 (October 2013) p. 681-714 • ISSN 1983-4195

[16] OLIVEIRA, D. R., MELO, G. S., REGAN, P. E., Punching Strengths of Flat Plates with Vertical or Inclined Stirrups. ACI Structural Journal, V. 97, No. 3, May-June 2000, pp. 485-491.

[17] REGAN, P. E., SAMADIAN, F., Shear Reinforcement against Punching in Reinforced Concrete Flat Slabs, The Structural Engineer, V. 79, No. 10, May 2001, pp. 24-31.

[18] Pilakoutas, K., Li, X. (2003) Alternative shear reinforcement for reinforced concrete flat slabs, Journal of Structural Engineering, 129 (9), pp. 1164-1172

[19] Nasr Z. Hassan, Mostafa A. Osman, Awad M. El-Hashimy, and Heba K. Tantawy, “Enhancement of punching shear strength of flat slabs using shear-band reinforcement”, Housing and Building National Research Center HBRC Journal, 26 November 2017.

[20] Alternative punching shear reinforcement for RC flat slabs, January 2014, Indian Concrete Journal 88(1):33-44

[21] Strength of Footing with Punching Shear Preventers Sang-Sup Lee,1 Jiho Moon,2 Keum-Sung Park,1 and Kyu-Woong Bae1

[22] UFO-Punching shear preventers Literature Review

[23] Enhanced Safety with post-installed shear reinforcement Jakob Kunz*, Miguel Fernández Ruiz*, Aurelio Muttoni* Hilti Corporation, FL-9494 Schaan, Liechtenstein Ecole Polytechnique Fédérale de Lausanne, CH-1015 Lausanne, Switzerland

[24] Fastening Technology Manual Post-Installed Shear Reinforcement Hilti HZA-P

[25] ČSN EN 1992-1-1 (731201) Eurokód 2: Navrhování betonových konstrukcí - Část 1-1: Obecná pravidla a pravidla pro pozemní stavby

[26] Web source: https://www.archiweb.cz/b/vila-nad-udolim--1

[27] CATALOG SCHOCK ISOKORB® T TYPE K

[28] CATALOG HALFEN SOUND INSULATION PRODUCTS

[29] Web source:https://www.dlubal.com/en/load-zones-for-snow-wind-earthquake

[30] Autoclaved aerated concrete masonry units YTONG P1.8-300, P2-350, P2-400, P2-500, P4-500, P4-550, P6-650

[31] SCIA. Handbooks. www.scia.net [online]

Available online: https://www.scia.net/cs/support/downloads

[32] Web source: https://help.scia.net/15.1/en/dlo/concrete15/deflection/deflection_03_tb.htm

[33] KATEDRA BETONOVÝCH A ZDĚNÝCH KONSTRUKCÍ (K133)Concrete and Masonry Structures 1 (133CM01)

[34] Prochazka,J., Stemberk, P; Concrete structure 1. CVUT Praha,2007.

[35] Prochazka,J., Stemberk, P; Design Procedures For Reinforced Concrete Structures