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Department of Concrete and Masonry Structures

Bachelor Thesis

DESIGN METHODS OF FLAT SLABS FOR PUNCHING – EUROPEAN AND NORTH AMERICAN PRACTICES

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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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I hereby confirm that this bachelor's thesis was carried out by me as the result of my own original studies and research. I have acknowledged all the sources of information in the bibliography.

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Abstract

Up to this day, in different part of the world, many cases of collapses were established due to punching shear. This mechanism that causes a failure is considered as the main problem of a flat slab. So, the bachelor thesis will introduce the punching shear of a flat slab, possible retrofitting systems, and their appropriate design according to the European EN 1992 1-1 and the North American ACI-318. Additonally, many factors that cause the deterioration of a slab will be discussed. The main task of interest and case study will be the real failure of Piper's Row Car Park in Wolverhampton. It will be designed according to those provisions and the following part of the thesis will be dealt with analyzing the possible deterioration that will lead to the failure. A major part of the thesis will introduce and describe the reduction factors of mechanical properties of the structure based on the design punching shear equations according to those two codes by modification. The degradation of the slab will be shown by so-called reduction parameter and variables for that will be the effective depth, amount of shear, and the flexural reinforcement ratio. The final part of the thesis will indicate a possible extend of the European EN 1992 1-1 and the North American ACI-318 codes in regards with deterioration processes of both effective depth and corrosion of reinforcement, at the end will be the comparison and discussion of which of those provisions will show more conservative results and which one less. Thesis will include author's reasonings of obtained results and their applicability.

Keywords

Punching shear reinforcement, flat slab, deterioration, design analysis, application of codes, reduction parameters, discussion of results.

Content

List of figures

List of tables

1 Introduction

The purpose of the thesis is to provide a review for various types of punching shear reinforcement which are used in buildings, and the scope is to analyze and compare design procedures of punching shear within the scope of European standards EC2 and American code ACI-318.

This thesis presents state-of-the-art of punching shear in flat slabs, examination of the two practically different design approaches. The related sources cover research and technical literature, and includes practical knowledges of experienced university staff. The case study is a real car park (Piper's Row Car Park, Wolverhampton, U.K.), it was chosen for design procedures and related analysis. A 3D model of upmost floor of that car park was created in SCIA Engineer 2019 (ver. 1.1) software. It was analyzed and calculated by using finite element method (FEM), subsequently, the calculations have shown internal forces on a two-way slab. Then, corrosion effect in flexural and punching reinforcement was analyzed as one of the possible cases for punching failure and, most importantly, punching shear resistance of the slab was checked for possible punching failures, and the appropriate punching shear reinforcement was designed by both Eurocode 2 and ACI-318 design procedures.

1.1 Motivation

Flat slabs are being used commonly in large industrial structures, multi-story car park building structure, high-rise buildings and in cases where no beam is required because it's propped directly by columns. Mostly, structural engineers use flat slabs because of their fast realization and quick installation, it obviously leads to time-and labor-savings, so it is very efficient for construction companies. In addition, if we consider that from architectural point of view, it has good flexibility in the use of built areas and looks aesthetically pleasing. Despite to the fact that it has many advantages, the main problem of flat slabs is punching shear failure. Punching shear usually happens in a brittle manner when a high concentrated load acts on a slab which is transferred to a small local area at the slab-column connection. Punching is a local failure mechanism which generates formation of a truncated cone shape. In the past years, punching shear failure caused many accidents which had led to progressive collapses and catastrophic failures, but, nowadays, thanks to accumulated knowledges and many analyses from engineers, it became possible to prevent such failures. So, I was wondering about this topic, and I would like to go through it in detail. Later on, I would be glad to show some real failure due to punching and make designs according to European and American methods by using computer analysis, also, the analyses and assumptions of what caused the car parking to collapse are shown.

1.2 Situation

In this study, the real existing car park was chosen for the design and then it will be assessed to investigate the collapse due to punching. A more detailed examination of factors and analysis of degradation were raised.

The following situation is examined:

Piper's Row Multi-Storey Car Park was built in 1965 and collapsed in 1997 (Fig. 1.1). A 15x15 m section of the upmost floor of the car park crashed and the cause of that was the punching shear failure at one of the columns which then developed into a progressive collapse. The initial prerequisite of such a failure could potentially originate from crack formation of poorly casted/detailed concrete by freeze-thaw attack. Since concrete structures are very sensitive to propagation of cracks and, due to the fact that corrosion is the main reason of failures of such, the deterioration of reinforcements, both flexural and shear, could essentially decrease the designed resistance (Jonathan, 1997). Moreover, the swelled products of corrosion tend to create additional stresses leading to delamination processes of covering concrete.

Figure 1.1 Collapsed top floor of slab at Pipers Row Car Park (Jonathan , 1997)

Piper's Row Car Park accident

The actual reason of the collapse was that the bond between the steel reinforcement and the surrounding concrete became poor. Eight years before the collapse, the founder of Arup Group, Sir Ove Arup, was hired to make an inspection of the structure. He said that the structure will not collapse, but close attention should be made to prevent further deterioration. The slab has been repaired in 1991, but it was never mentioned on which part of the slab the repairs were performed. During the 1991, a quotation for a complete restoration was instructed to retrofit the columns with concrete capitals, however, this modification has never been performed (Fig.1.2).

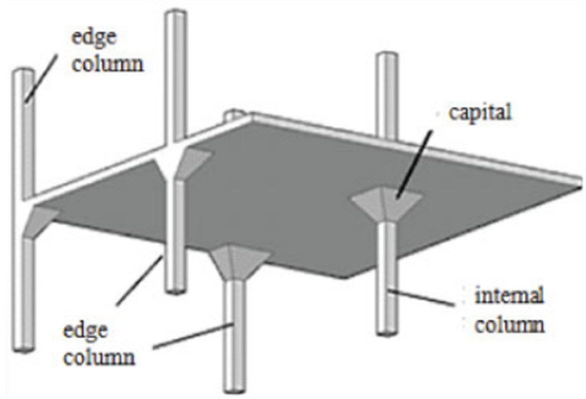


Figure 1.2 Retrofitting by column capitals

A year before the failure, a contractor inspector was hired to investigate waterproofing repairs. They found out that at some places the concrete was damaged up to 60 mm, and, a month before the collapse, the building owners ordered restorations without full investigation of damage development.

Figure 1.3 Piper's Row Car Park (Jonathan , 1997)

A week before the collapse, a structural engineering firm was instructed to prop the slab at the column, and at the same moment building owners started approaching contractors for the job. However, the structure collapsed at 03:20 a.m. on the 20th of March 1997.

2 State of the art

2.1 Two-way flat slabs

Flat slab is a reinforced concrete slab that usually does not have beams and instead of this, loads are passed directly to supported columns (Fig. 2.1).

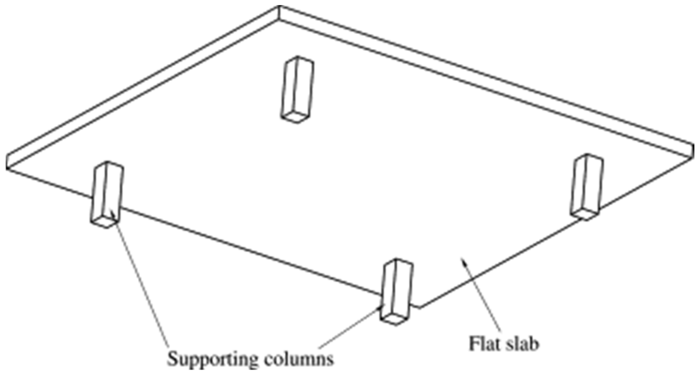


Figure 2.1 Flat slab system (Sahab, Ashour , & Toropov, 2004)

Speaking of advantages, the flat slab provides good ceiling superficies giving better diffusion of light, allowing minimum depth, easy and fast to construct as it requires less formwork, and it has good flexibility and pleasant appearance in the use of built areas. The flat slabs are usually retrofitted beside the supporting columns, and this ensures sufficient strength in shear and reduces the signs of negative bending in the support regions.

Types of construction of flat slab:

Simple flat slab

This type of flat slab doesn't have both column heads and drops (Mariyam & Sagar, 2019). It's also called as beam slab construction (Fig. 2.2). So, in many cases, when people construct car parks, storehouses, commercial spaces, and offices, they try to not use beams, hence, they try to use slabs, which are directly supported on columns.

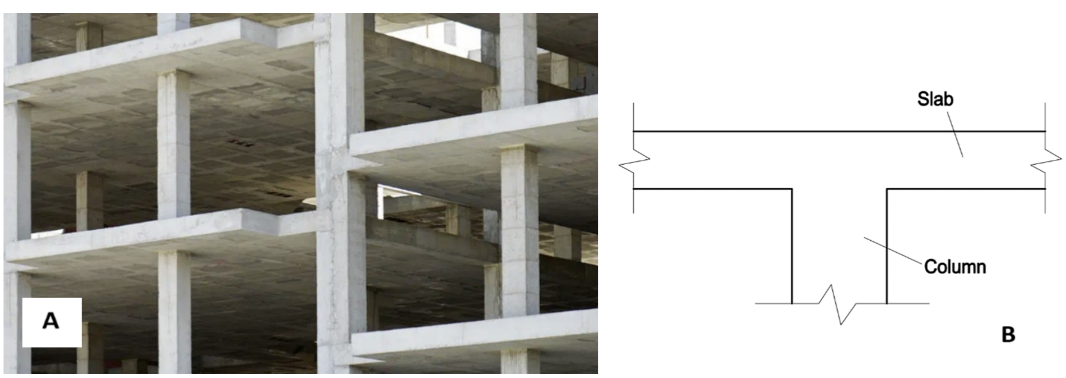


Figure 2.2 a) Typical flat slab (The Dlubal Structural Analysis Wiki , n.d.) and b) it's section view

Flat slab without column head and with drop

The drops are created in the area of the supporting column by increasing the thickness of the slab (Fig. 2.3). That's a very good retrofit to increase the shear strength of the slab and on the other hand, in reducing the required reinforcements for negative moments at column supports inasmuch as the drops enlarge the negative moment capacity of the slab. Additionally, would say that it stiffens the slab, hence decreasing deflection (Daily Civil, n.d.).

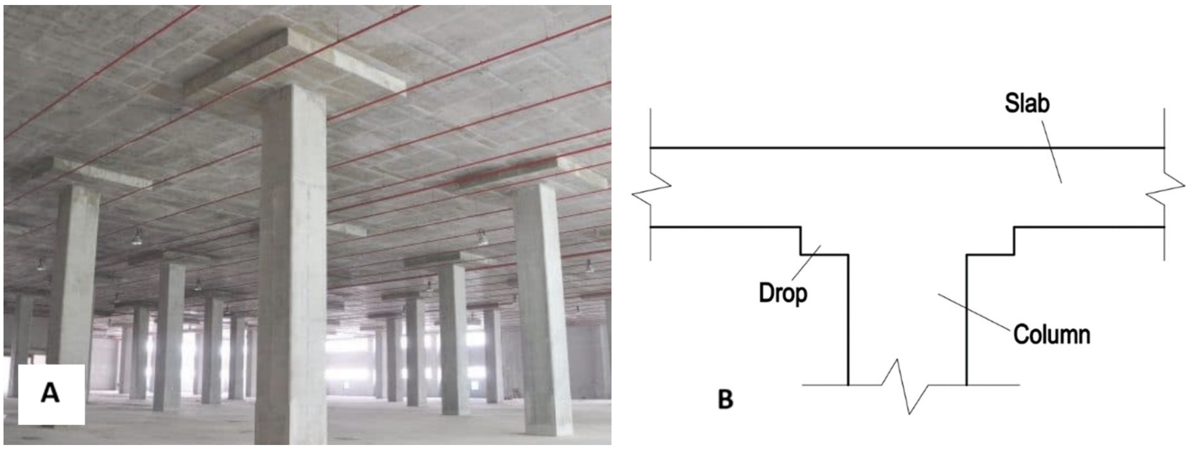


Figure 2.3 a) Flat slab with drop (Cornelius, 2018) and b) it's section view

Flat slab without drop panel and with column head

The column head or it's usually called "column capital", provided by forming a shape of reversed truncated cone and located at the top of column to increase the strength against punching shear (Fig 2.4). And it decreases the moments because the clear span is also decreased. Usually for the design, the angle of column head is done as 45º because in practice it's easy and more functional.

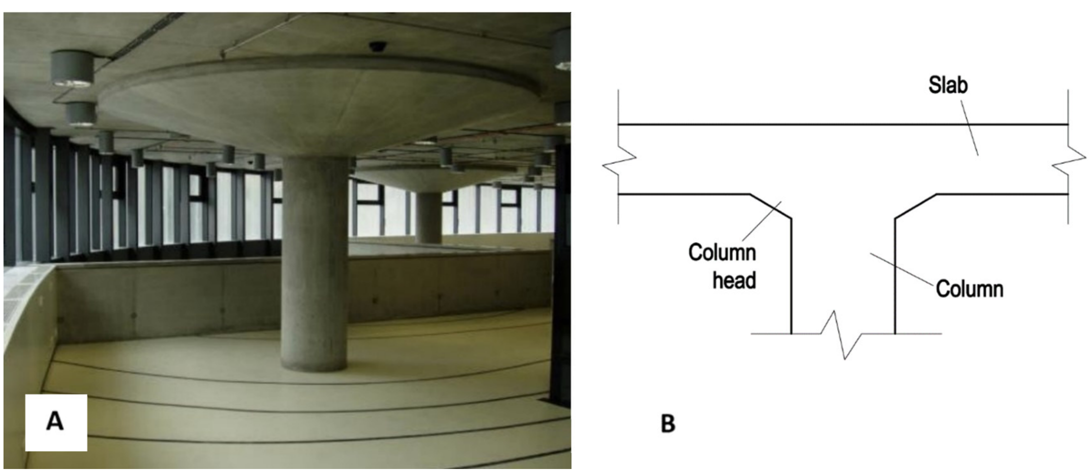


Figure 2.4 a) Flat slab with column head (NTK, n.d.) and b) its illustration

Flat slab with column head and with drop panel

This type of flat slab consists of both the column head and the drop (Fig 2.5). Usually, when the load is too high, as the solution, a combination of the column capital and the drop are used, where the drop panels are fixed to the slab, and the enlarged head of column is connected towards the panels (Anitha, 2007). That will increase the depth hence the shear strength also will be increased and at the same time deflection will be reduced.

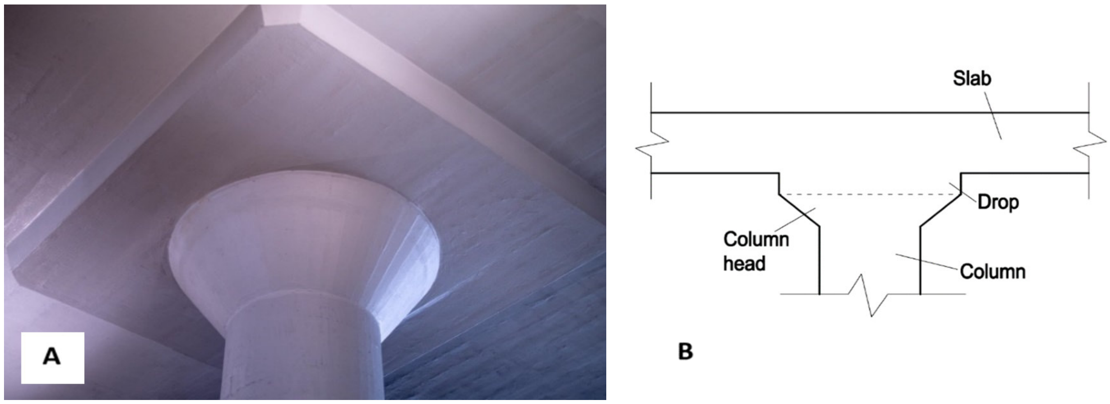


Figure 2.5 a) Flat slab with column head and drop (Toudai, 2014) and b) it's section view

2.2 Design procedure of punching shear reinforcement

Punching shear is a brittle failure mode of flat slabs that occurs in the slab-column connection because of high concentrated loads (Oliveira & Pereira Filho, 2013). It means that the local area surrounding the column is subjected to excessive loads due to shear stress which ultimately causes the punching shear failure for the flat slab (Fig. 2.6). Destruction in one place may attract to progressive collapse. Because loss of one member will affect the other members of the structure by loss of balance and the forces will be redistributed and spread to the other members that will not be able to hold it. That is in fact a progressive collapse when the initial failure causes the other failures by spreading in a manner of chain reaction (Bruce R. Ellingwood , Robert Smilowitz , & et al, 2007).

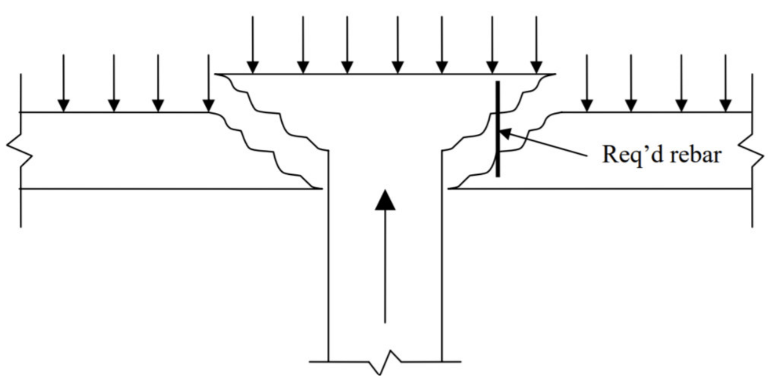


Figure 2.6 . Punching failure due to shear (CAPRANI)

To prevent this kind of disaster caused by punching failure, it's necessary to put so called shear reinforcements such as bent-up bars, stirrups or studs.

2.2.1 Types of reinforcement

If the design is not satisfied in the slab-column connection regarding punching, there are several solutions that can be used for it. Strength of the slab can be increased by taking actions, such as:

- The increase and enhancement of the column size, of the slab depth, of the longitudinal reinforcement, of the compressive strength of concrete
- The increase by using column heads and drops
- The increase by adding shear reinforcements (studs, stirrups, bent-up bars etc.)

Mostly, the increase of the column size or using the column heads are not suitable because this will change the appearance and not recommended from the view of architectural point. The increase of the slab depth will accordingly lead, the structure will rise in the value and foundation cost. To prevent punching shear by improving the material strength of the slab also non-economical and would have poor efficiency. So, the appropriate solution may be the use of shear reinforcement bars. The strength of shear reinforcement depends on the type of reinforcement, the quantity, position and spacing between bars and so on.

Bent-up bars. Usually, 135° inclined bent-up bars were widely used to prevent punching shear failure in the past. The main advantage of the bent-up bar is that it acts as a flexural reinforcement and as a shear reinforcement at the same time. Notwithstanding the foregoing, the bent-up bars were replaced by other shear reinforcements: headed shear studs and stirrups because they are simple as a construction. Nevertheless, now bent up bars have started to use not as a full-fledged shear reinforcement but as a helper in combination with other shear reinforcements or as an enhancer if no punching shear reinforcement is used *(Luca Tassinari & et.al, 2011)* (Fig. 2.7).

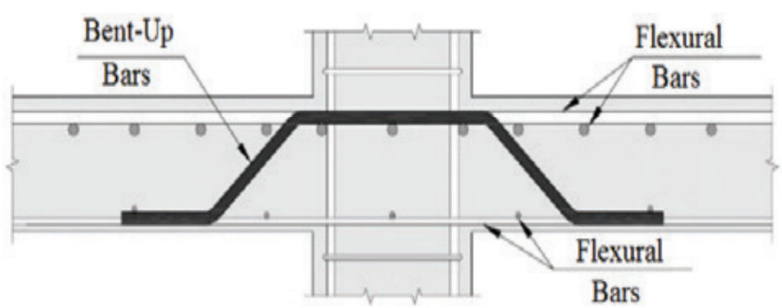


Figure 2.7 Bent up bars (Oliveira & Pereira Filho, 2013)

Headed shear studs consist of rails and shear studs (Fig. 2.8) are commonly used in flat slabs due to their good resistance since it has great mechanical anchorage and high quality.

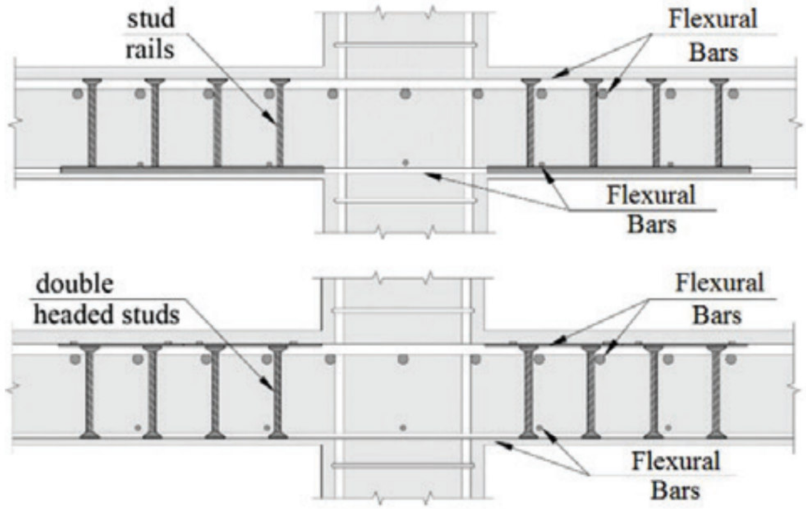


Figure 2.8 Headed shear studs (Oliveira & Pereira Filho, 2013)

Nowadays, headed shear studs are considered as most popular shear reinforcement in the construction industry. Allocation of the stud elements in rows allows significant areas to be strengthened fastly with less time and efficiently (Subramanian, 2014).

The arrangement of studs in Europe are radially and respectively cruciform arrangement in North America.

Shear reinforcement stirrups are mostly used to hold in position the flexural reinforcement and to prevent the shear in flat slabs. Generally, stirrups are made and designed with different shapes, such as rectangular and square or they can be provided as a circular (Oliveira & Pereira Filho, 2013).

The following types of shear reinforcement stirrups (Fig. 2.9) are commonly used in construction:

- Closed stirrups
- Single legged open stirrups
- U-shaped stirrups like "shear combs"
- Inclined stirrups

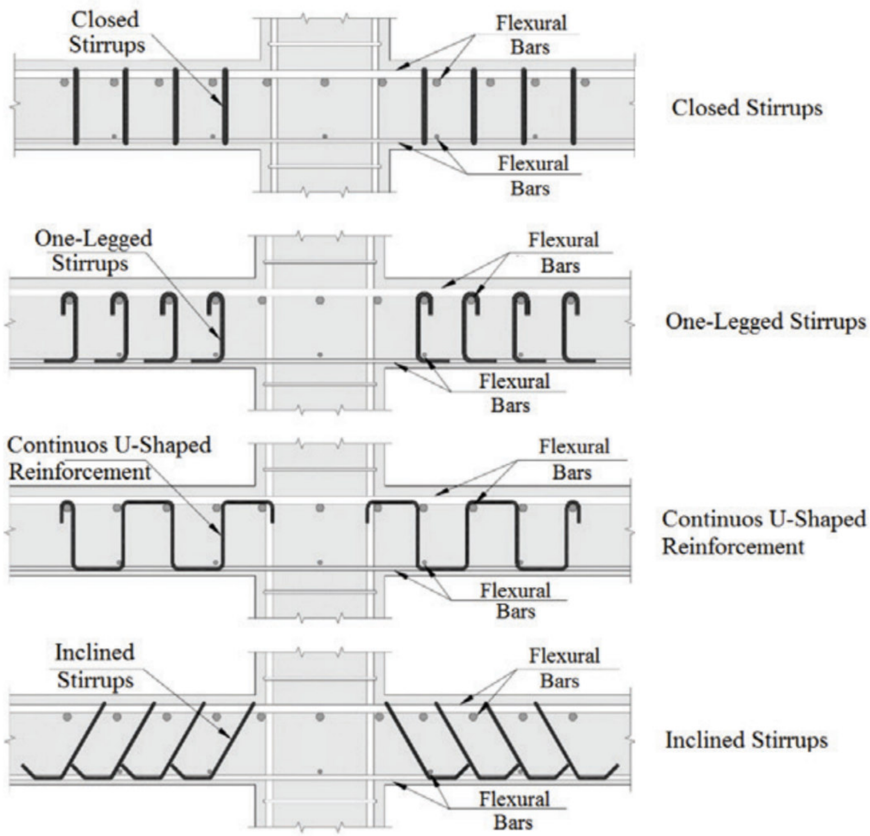


Figure 2.9 Different types of stirrups (Oliveira & Pereira Filho, 2013)

Closed stirrups are effective to prevent diagonal cracks and holding the shear stresses. However, the closed stirrups and continuous U-shaped cages are hard to use because of their assembly.

The use of one-legged stirrups is not often because they bind only two rods and as results have shown it has a bad anchorage. Inclined stirrups are usually made with a 45° inclination to resist and prevent diagonal stresses. Additionally, from the research and experiments are given by (Oliveira & Pereira Filho, 2013), they say that only stirrups with 60° inclination have shown more efficiency in enhancing the punching shear resistance.

Shear-head reinforcement is a resistance against punching by using I-sections (Fig. 2.10). These I-sections are done by welding the steel with equaled length in two directions (Subramanian, 2014). Despite it being easy to fabricate, it's heavy and high-priced compared with other shear reinforcements as stirrups, etc. that's why it's not common and it's not used widely. The difficulty of shear-head reinforcement is the integration with regular reinforcement, and it can lead to obstructing the transition of the column bars across the connection.

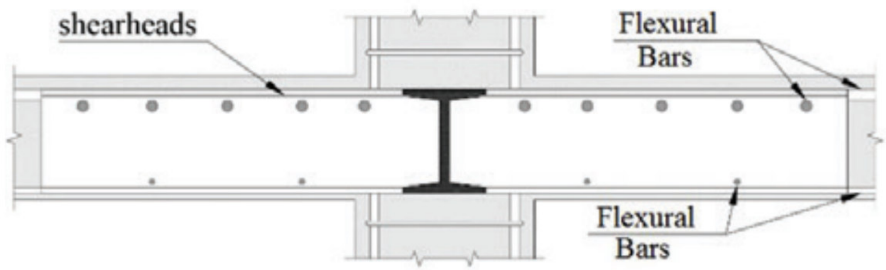


Figure 2.10 Sher-head reinforcement (Oliveira & Pereira Filho, 2013)

Shear-band reinforcement is manufactured by high-strength slim steel strips of high ductility (Saadoon, 2019). The advantages of this partly new system are that it can be bent into various shapes, and it anchors from the top after all flexural reinforcements are installed, with minimum loss of cover. This type of reinforcement (Fig. 2.11) is not only improving the ductility of the flat slabs but also enhances the shear strength of the slabs without increasing the flexural capacity. Additionally, it's very easy and efficient in the installation.

Figure 2.11 Shear-band reinforcement (Saadoon, 2019)

Lattice girders as punching shear reinforcement are well known and widely approval in the European community. Lattice reinforcement is arranged parallel to each other and the struts of it act as punching shear reinforcement (Fig. 2.12). The location of the upper longitudinal reinforcement is that it's placed on top of the upper chord. This system shows good effectiveness as a shear reinforcement with a stiff anchorage (Subramanian, 2014).

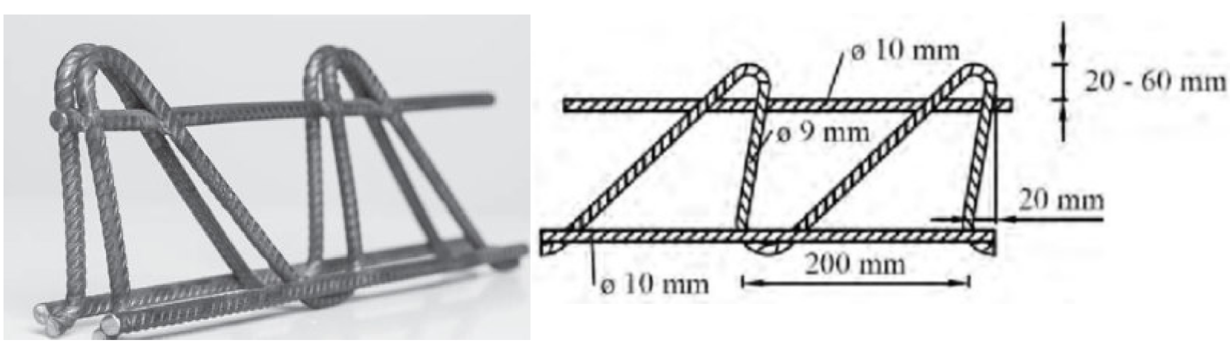


Figure 2.12 Lattice shear reinforcement

UFO punching preventers are made of steel plate material proposed by Alander (2000). It consists of a cone-shaped steel device and locates in the slab-column connection that increases a punching capacity (Fig. 2.13). Those holes of the UFO are made for passaging the vertical column reinforcements. The lower part of the system operates as a support for the slab in the outer area whereas inside of the slab-column connection the UFO congregates the reactions from the slab and passes them by membrane action to the region above the column (Subramanian, 2014).

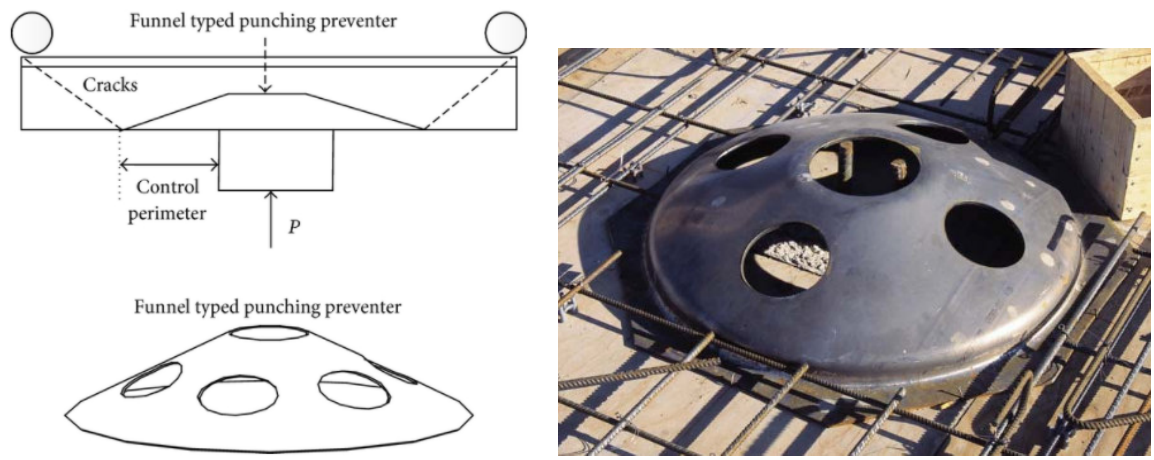
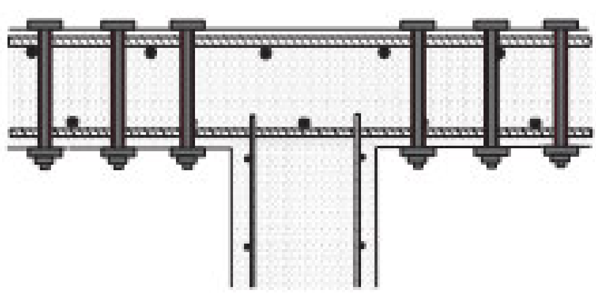


Figure 2.13 The UFO punching preventer (Subramanian, 2014)

Post-installed punching reinforcement is a retrofitting system for existing buildings to make them resistant against punching shear by bonded concrete screw anchors (Fig. 2.14: c, d). It has been done with combination of a self-tapping concrete screw anchor and an adhesive mortar (Muttoni, 2019). The installation depends on the accessibility of the slab for anchoring and can be done in two ways:

- First, if the bottom and upper parts of the slab are available for work, then for inserting the steel bars, holes need to be drilled across the slab. Then it will be prestressed towards the slab as it tights nuts on both sides (Fig. 2.14: a).
- Second, if the upper part of the slab is not available for work, then for inserting the steel bars into the slab, inclined holes need to be drilled from the bottom part of the slab with an angle of 45° and in the direction towards the column (Fig. 2.14: b). Those drilled holes should be at least at the level of the lowest layer of the upper reinforcement because working capacity against punching shear primarily depends on the quality of its anchorage.

A



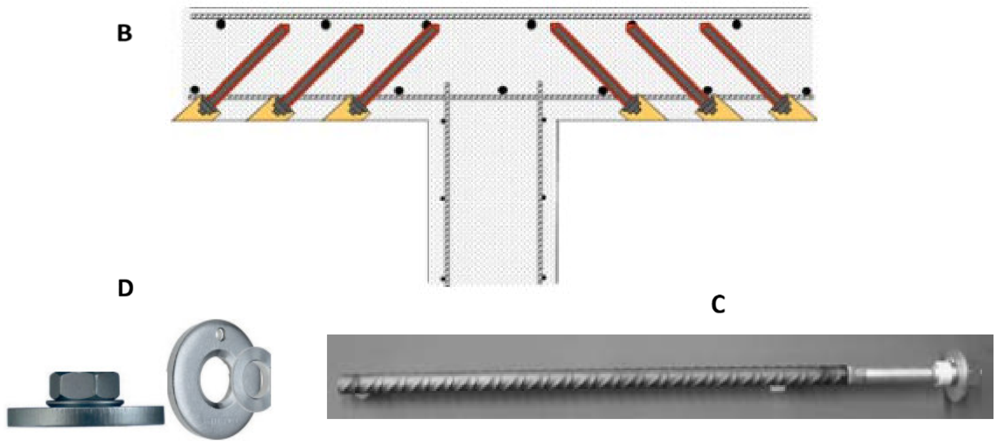


Figure 2.14 . a) Penetrating post installed punchin shear reinforcement; b) Post-installed punching shear reinforcement applied only from bottom part of the slab; c) Strengthening anchor Hilti HZA-P; d) Hilti filling set (Muttoni, 2019)

2.2.2 Design methods by European and American codes

Due to the fact that flat slabs are commonly used in construction, it was thoroughly researched and analyzed, especially for punching shear behavior. After all these investigations were proposed maximally accurate standards and methods that save time and provide good performance, but each country has its own standards related to their investigations. The difference between the Eurocode 2 and ACI-318 depends on the precision, complication, limitation, and safeness levels.

This section as a matter of fact presents the code provisions of the Eurocode 2 and ACI-318 for punching with respect to the resistance without shear reinforcement and with. Need to mark that those codes related to punching shear have absolutely different parameters of calculation, for instance for critical perimeters and so on. This indicates that the approaches are totally different, but the results are analogous. Further will be introduced the formulas in detail and described according to those codes.

2.2.2.1 Design procedure of punching shear capacity according to EC 2

This subsection presents the design rules of punching shear by Eurocode 2. All equations and figures were taken from section 6.4 in EN 1992-1-1(2004).

In the beginning, the punching shear capacity needs to be checked at the face of the column and at the basic critical perimeter u_1 which is normally at a distance 2d (Fig. 2.15) from the loaded area. If the shear reinforcement is needed, so, in this case, outer perimeter u_{out} should be found where shear reinforcement is no more needed. The punching shear stress should be calculated regarding the control perimeter around the loaded region, and it is taken as:

$$\upsilon_{Ed} = \frac{\beta \cdot V_{Ed}}{u_1 \cdot d} \tag{1}$$

$u_1 - is\ the\ length\ of\ the\ basic\ control\ perimeter$

$\beta - increasing\ factor$

$V_{Ed} - the\ design\ shear\ force$

$d - the\ mean\ effective\ depth\ of\ the\ slab, which\ may\ be\ taken\ as\ \frac{d_y + d_z}{2}$

$d_y\ and\ d_z\ are\ the\ eff. depths\ of\ the\ reinforcement\ in\ two\ orthogonal\ directions$

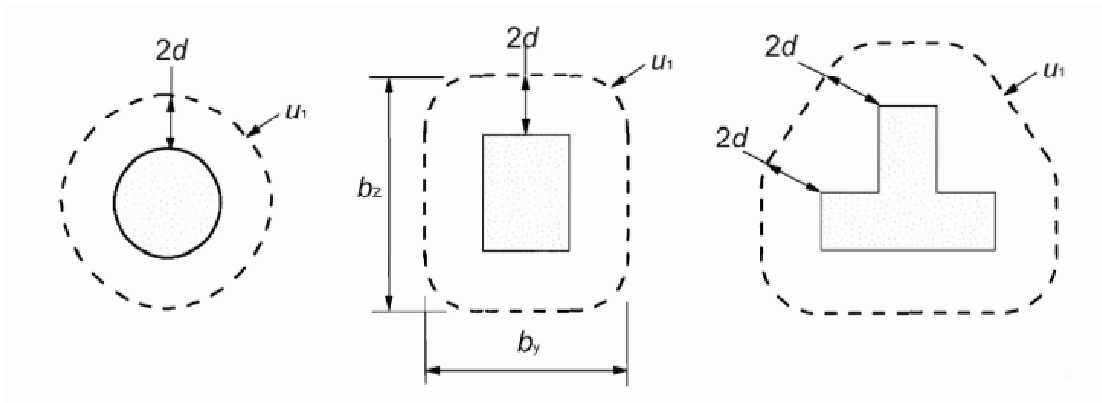


Figure 2.15 Basic control parameters

If the structure has an opening near to the loaded area, in that case, the shortest distance between the opening edge and the loaded region's perimeter needs to be checked. If it is less than 6d, so it says that one part (see Fig. 2.16) of the control perimeter is effectless.

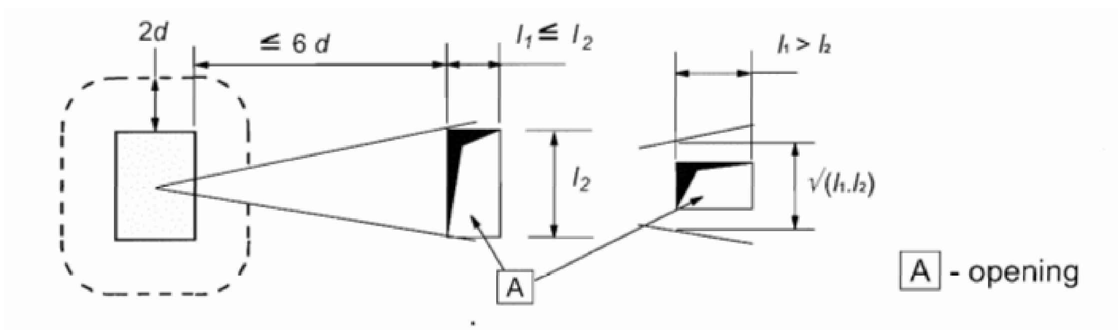


Figure 2.16 Control perimeter near an opening

If the loaded area is near to the edge or corner, in that case, the control perimeter should be taken as follows (Fig. 2.17) and it is only used if it is smaller than the control perimeter obtained from (Fig. 2.15). Additionally, if the loaded area near the edge or corner is smaller than d, so special reinforcement needs to be provided at the edge, see section 9.3.1.4. in EN 1992-1-1 (2004).

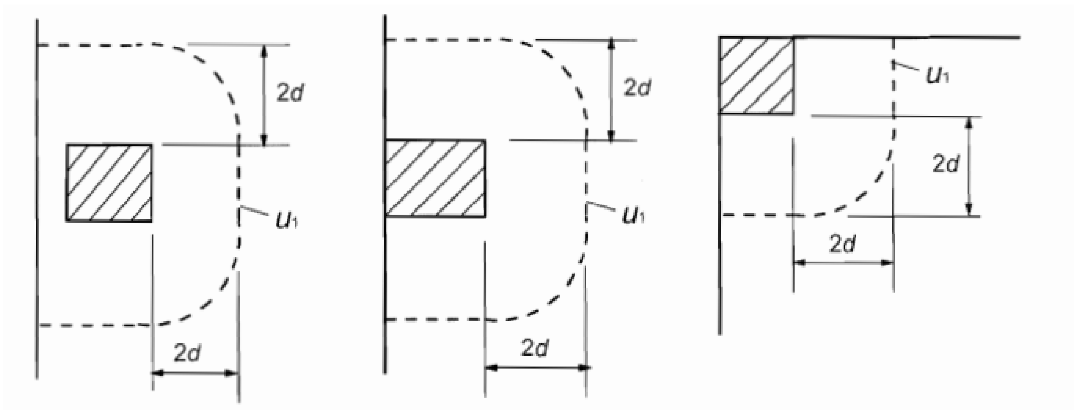


Figure 2.17 Basic control perimeters for edge and corners

Talking about the increasing factor β, it is estimated for rigidly fixed structures with the lengths of the spans changes do not exceed 25% (Fig. 2.18).

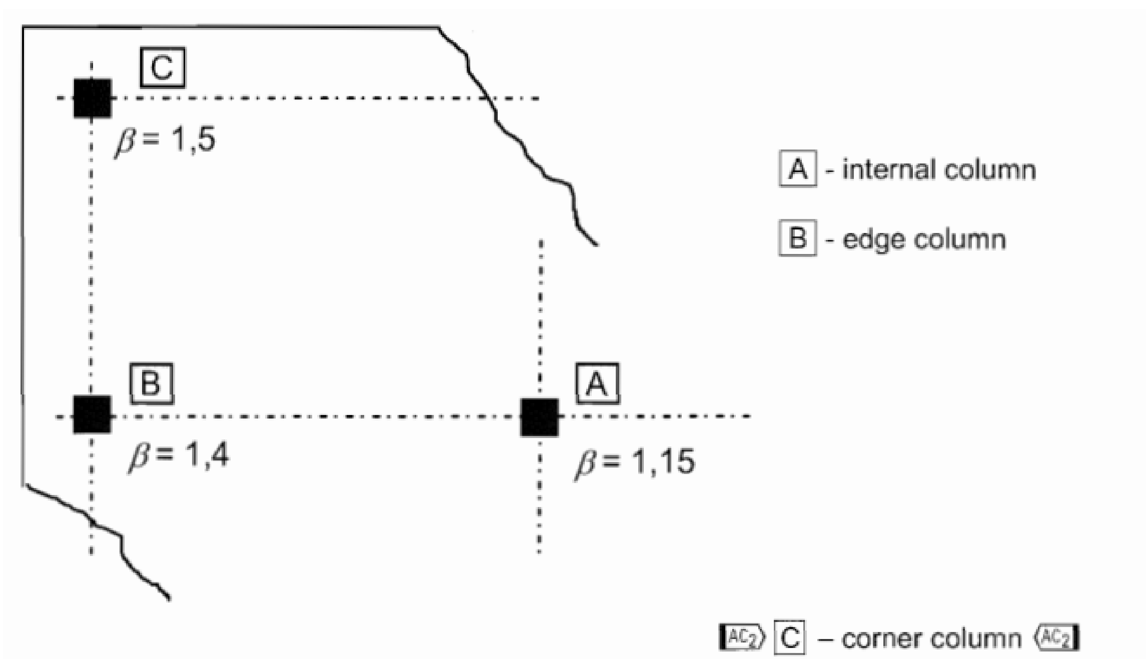


Figure 2.18 Recommended values for β

In other cases, the parameter β needs to be determined by the following calculation:

$$\beta = 1 + k \cdot \frac{M_{Ed}}{V_{Ed}} \cdot \frac{u_1}{W_1} \quad (2)$$

$u_1 - the\ length\ of\ the\ basic\ control\ perimeter$

$k - coeff.dependent\ on\ the\ ratio\ between\ the\ column\ dimensions$

$M_{Ed} - moment\ about\ the\ centroidal\ axis\ of\ the\ control\ perimeter$

$W_1 - modulus\ of\ resistance\ of\ the\ control\ perimeter$

It is defined as an eccentricity of the moment at a slab-internal column connection in the control perimeter. And it varies depending on different cases, see more in EN 1992-1-1 (2004).

When designing a flat slab, some checks should be considered regarding the punching. It's necessary to check if the punching shear reinforcement at the basic perimeter u_1 is needed or not, as follow: $V_{Ed} \leq V_{Rd,c}$

$V_{Ed} - applied\ shear\ stress$

$V_{Rd,c} - punching\ shear\ resistance\ withou\ shear\ reinforcement$

The punching shear resistance is calculated as:

$$v_{Rd,c} = C_{Rd,c} \cdot k \cdot \sqrt[3]{100 \cdot \rho_l \cdot f_{ck}} + k_1 \cdot \sigma_{cp} \geq v_{min} + k_1 \cdot \sigma_{cp} \tag{3}$$

$C_{Rd,c} = \dfrac{0{,}18}{\gamma_c} - constant\ factor;\ \gamma_c - partial\ safety\ factor\ of\ concrete$

$k = 1 + \sqrt{\dfrac{200}{d}} \leq 2 - size\ effect\ factor$

$\rho_l = \sqrt{\rho_{ly} \cdot \rho_{lz}} \leq 0.02 - the\ flexural\ reinforcement\ ratio$

$\rho_{ly}, \rho_{lz} -\ calculated\ for\ a\ strip\ width\ equal\ to\ the\ column\ width\ plus\ 3d\ each\ side$

$f_{ck} - characteristic\ compressive\ strength\ of\ cylinder\ concrete$

$k_1 = 0.1\ - recommended\ value$

$\sigma_{cp} = \frac{\sigma_{cy}+\sigma_{cz}}{2};\ \sigma_{cy(cz)} = \frac{N_{Ed,y(Ed,z)}}{A_{cy(cz)}}\ are\ the\ normal\ concrete\ stresses$

$N_{Ed,y}, N_{Ed,z}\ are\ the\ longitudinal\ forces\ across\ the\ full\ bay\ for\ internal\ columns$

$A_c -\ area\ of\ concrete\ according\ to\ the\ definition\ of\ N_{Ed}$

$v_{min} = 0.035 \cdot k^{\frac{3}{2}} \cdot f_{ck}^{\frac{1}{2}}\ -\ where\ v_{min}\ is\ a\ minimum\ shear\ stress$

For maximum punching shear stress the condition needs to be checked at the column perimeter u_0 as follow:

$$\upsilon_{Ed,u_0} = \frac{\beta \cdot V_{Ed}}{u_0 \cdot d} \leq \upsilon_{Rd,max} = 0.4 \cdot \nu \cdot f_{cd} \tag{4}$$

$$where: \nu = 0.6 \cdot \left(1 - \frac{f_{ck}}{250}\right)$$

$$u_0 - enclosing\ minimum\ periphery$$

When the check requirement of $V_{Rd,max}$ has been controlled and fulfilled, and the condition of shear reinforcement necessity says that we need to provide punching shear reinforcement, so, punching shear resistance with shear reinforcement needs to be checked as follows:

$$\upsilon_{Ed,u_1} \leq \upsilon_{Rd,c+s} = 0.75 \cdot \upsilon_{Rd,c} + 1.5 \cdot \frac{d}{s_r} \cdot A_{sw} \cdot f_{ywd,ef} \cdot \frac{1}{u_1 \cdot d} \cdot sin\alpha \leq k_{max} \cdot \upsilon_{Rd,c} \tag{5}$$

$$s_r - \ the\ radial\ spacing\ between\ perimeters\ of\ shear\ reinforcement$$

$$A_{sw} - the\ section\ area\ of\ shear\ reinforcement\ along\ one\ perimeter$$

$$f_{ywd,ef} = 250 + 0.25 \cdot d \leq f_{ywd} = \frac{f_{yk}}{\gamma_s}$$

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

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

$$\alpha - the\ angle\ between\ the\ shear\ reinforcement\ and\ the\ plane\ of\ the\ slab$$

$$k_{max} - coefficient\ of\ maximum\ resistance\ depends\ on\ a\ type\ of\ shear\ reinforcement$$

The control perimeter where the shear reinforcement is not necessary and its radius:

$$u_{out} = \frac{\beta \cdot V_{Ed}}{\upsilon_{Rd,c} \cdot d} \qquad r_{out} = \frac{u_{out}}{2 \cdot \pi} \tag{6,7}$$

The punching shear reinforcement should not be apportioned more than a distance 1.5d inside of an outer perimeter u_{out} or $u_{out,ef,}$ (see Fig 2.19). The punching shear reinforcement needs to be provided at least in two lines and the radial spacing should not be more than $s_r \leq 0.75d$. The spacing of rails should not exceed $s_t \leq 2d$ and first shear reinforcement should be placed in a range of 0.3d~0.5d from the face of the column. And one more important thing is that spacing of rails in the critical perimeter u_1 should be less than 1.5d.

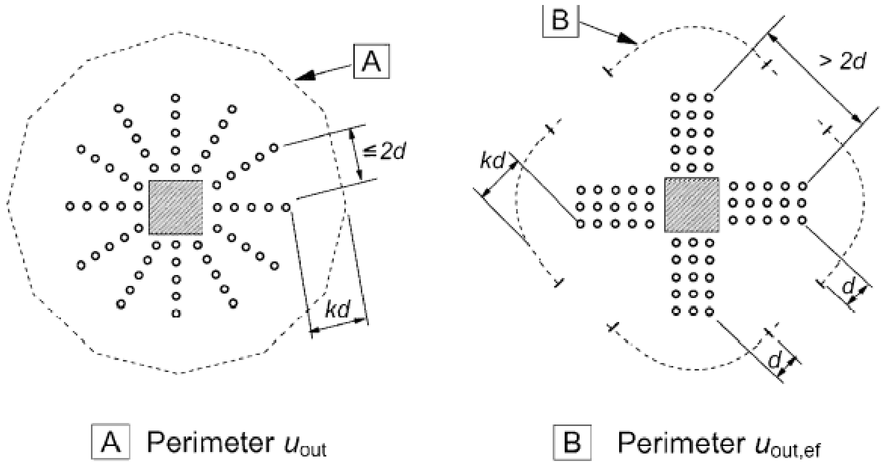


Figure 2.19 Control perimeters at inner columns

2.2.2.2 Design procedure of punching shear capacity according to ACI-318

A building code of "American Concrete Institute" requires only the minimum that needs to provide quality and safety for any structure and in this case, there could be some differences with Eurocode 2, generally such as the use of different safety and reduction factors, limitations, property of the material, and its significance.

All equations and figures related to the design of punching shear were taken from (ACI Committee 318, 2008). The punching shear capacity needs to be checked at the critical perimeter b_0 which is situated 0.5d from the face of the column (Fig. 2.20) and it should be verified and ensured that the shear resistance of concrete is greater than shear stress.

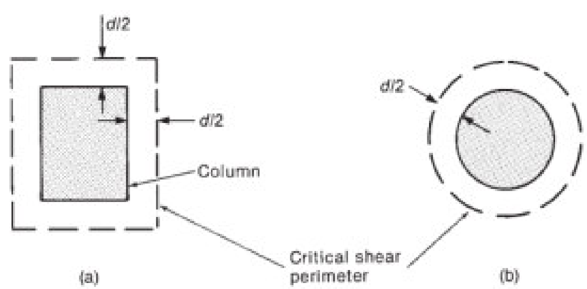


Figure 2.20 Location of critical perimeter

The calculation is presented, as follows:

$$\emptyset \cdot V_c \geq V_u \tag{8}$$

$\emptyset - reduction\ factor\ of\ strength\ and\ its\ equal\ to\ 0.75$

$V_c - shear\ strength\ provided\ by\ concrete$

$V_u - \ applied\ shear\ stress$

Applied shear stress is calculated along the critical perimeter b_0 and with including the effect of unbalanced moment:

$$V_u = \frac{\upsilon_u}{b_0 \cdot d} + \frac{\gamma_{v1} \cdot M_{u1}}{J_{c1}} + \frac{\gamma_{v2} \cdot M_{u2}}{J_{c2}} \tag{9}$$

$b_0 = \sum c + 4 \cdot d - critical\ perimeter$

$\gamma_{v1}\gamma_{v2} = (1 - \gamma_f) = \left(1 - \frac{1}{1 + \frac{2}{3} \cdot \sqrt{\frac{b_1}{b_2}}}\right) - fraction\ of\ the\ moment\ transferred\ by\ shear$

$M_{u1}, M_{u2} - unbalanced\ column\ moment\ about\ the\ center\ of\ geometry\ of\ critic.section$

$J_{c1}, J_{c2} = \frac{b_1 \cdot d \cdot (b_1 + 3 \cdot b_2) + d^3}{3} - moment\ of\ inertia\ of\ the\ critical\ section$

$b_1, b_2 = c_{1,2} + d - width\ and\ depth\ of\ critcal\ sections$

Allowable stress of shear strength provided by concrete should be taken as the minimum of those equations:

$$V_c = 0.17 \cdot \left(1 + \frac{2}{\beta_c}\right) \cdot \sqrt{f'_c} \tag{10}$$

$$V_c = 0.083 \cdot \left(2 + \frac{\alpha_s \cdot d}{b_0}\right) \cdot \sqrt{f'_c} \tag{11}$$

$$V_c = 0.33 \cdot \sqrt{f'_c} \tag{12}$$

$V_c - punching\ shear\ strength$

$f'_c - specified\ concrete\ cylinder\ strength$

$\alpha_c - equal\ to\ 40\ for\ interior\ columns\ (depends\ on\ the\ position\ of\ the\ column)$

$\beta_c - ratio\ of\ longer\ to\ shorter\ side\ of\ the\ critical\ section$

When the check requirement of Ø.V_c has been controlled and the condition doesn't meet, it says that at the critical perimeter b_0 the slab has insufficient shear resistance of concrete, and shear reinforcement is needed to be provided and checked, as follows:

$$V_u \leq \emptyset \cdot V_n \tag{13}$$

$\emptyset - reduction\ factor\ of\ strength\ and\ its\ equal\ to\ 0.75$

$V_n - the\ nominal\ shear\ strength$

ACI-318 code provides the calculation of the maximum shear resistance which depends on the type of the shear reinforcement and can be calculated as follows:

$$V_n = V_c + V_s \leq 0.5\sqrt{f'_c} - with\ rebars, single\ or\ multiple\ stirrups \quad (14)$$

$$V_n = V_c + V_s \leq 0.58\sqrt{f'_c} - with\ steel\ I\ shear\ heads \quad (15)$$

$$V_n = V_c + V_s \leq 0.66\sqrt{f'_c} - with\ shear\ head\ studs \quad (16)$$

$V_c - the\ shear\ strength\ provided\ by\ concrete$

$$V_s = \frac{\emptyset \cdot A_v \cdot f_y}{b_0 \cdot s} - \ the\ shear\ strength\ provided\ by\ shear\ reinforcement$$

$$A_v = \frac{(V_u - \emptyset \cdot V_c) \cdot b_0 \cdot s}{\emptyset \cdot f_y} - area\ of\ shear\ reinforcement\ within\ a\ distance\ s$$

The arrangement of shear reinforcement is shown in (Fig. 2.21). Then talking about the maximum shear resistance, do not forget that in case of using the shear reinforcement, so according to ACI-318 the maximum resistance provided by only concrete will change and be different for different types of shear reinforcement, and it should not be taken greater than as follows:

$$V_c \leq 0.17\sqrt{f'_c} - with\ rebars, single\ or\ multiple\ stirrups \quad (17)$$

$$V_c \leq 0.33\sqrt{f'_c} - with\ steel\ I\ shear\ heads \quad (18)$$

$$V_c \leq 0.25\sqrt{f'_c} - with\ shear\ head\ studs \quad (19)$$

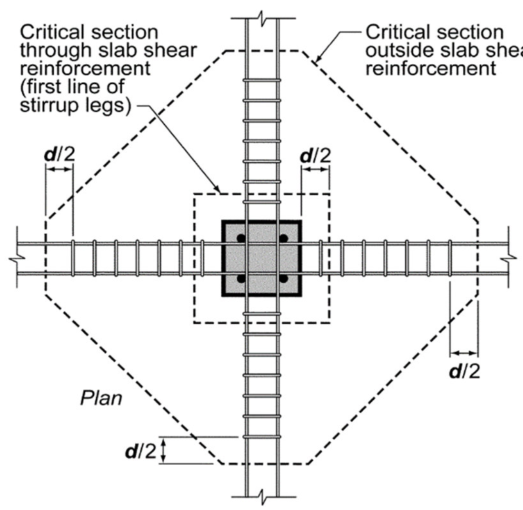


Figure 2.21 Arrangement of shear reinforcement according to ACI-318

2.2.2.3 Difference of codes

The provisions of ACI-318 and Eurocode 2 look different at the first glance, but the theory of use is similar because both provisions calculate the punching shear capacity based on a critical perimeter. Talking about the differences:

- The first thing to note is that in the ACI-318 the critical perimeter is placed at a distance of 0.5d from the face of the column, while Eurocode 2 takes it at 2d from the face of the column.
- The second thing to note is the use of concrete strength $\sqrt{f'_c}$ by ACI-318, and $\sqrt[3]{f_{ck}}$ by Eurocode 2. From analytical point of view, it could be taken from the regression analysis and testing. However, no information or unanimous opinion until now about which one is better: use of square root or cube root of the concrete strength.
- The third thing to note is that Eurocode 2 provides the design by using the effect of flexural reinforcement and the effect of size on the punching strength of the connection, while the ACI-318 considers and provides the design by using only concrete strength.
- The fourth thing to note is that the distribution of the shear reinforcement should be provided equally around at least two perimeters according to Eurocode 2, while the ACI-318 adopts cross-line shear reinforcement.

2.3 Deterioration processes of a slab structure

Different types of deterioration can lead to damage to the slab. Most frequent factors are due to the freeze-thaw attack, the penetration of carbonation or chlorides subsequently that appear corrosion (Fig. 2.22) and so on. Some defects could be due to poor quality construction such as defects in design, materials, workmanship, or maintenance that have influence on the safety and service life of the structure. However, the deterioration is usually the result of a combination of factors, and I would like to introduce several types of factors that cause the deterioration of a slab structure.

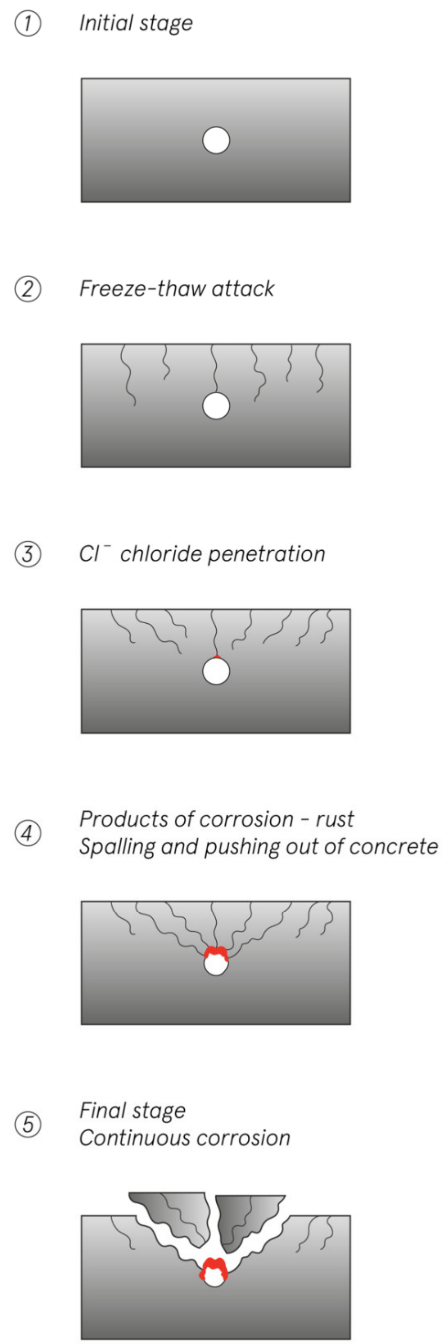


Figure 2.22 Deterioration due to several factors: frost, corrosion due to chloride ingress

Freeze-thaw attack

Frost damage is a deterioration process that appears mostly on the top of concrete structures (Fig. 2.23) (Freeze-thaw resistance, n.d.). When water freezes, it expands about 9% that means that if for instance, water freezes in wet concrete, it creates pressure in the concrete pores and capillaries. When the pressure reaches the limit of the tensile capacity of the concrete, the cavity will expand and break. Fortunately, the effect of freezing and thawing can be prevented. One of the ways is with deicing chemicals such as sodium chloride or calcium chloride. But it can lead to another problem named corrosion. Other ways are to apply a sealer or use high-strength concrete.

Figure 2.23 Typical examples of frost damaged slab

Corrosion

Steel corrosion is an electrochemical process by simultaneous effect of oxygen and moisture. There are several types of corrosion such as pitting, uniform, and galvanic. Talking about pitting corrosion, it's localized corrosion that creates pinholes in the steel. This type of corrosion is dangerous than others because of the difficulties in the prediction. Uniform corrosion is well-known corrosion that acts equally over the entire surface region (Fig. 2.24).

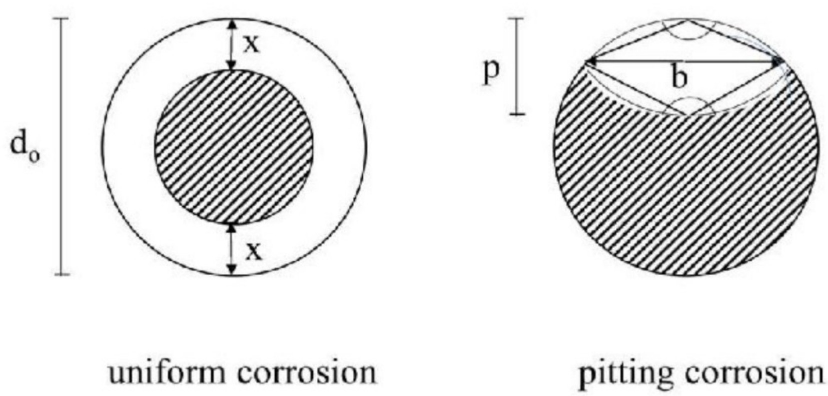


Figure 2.24 Types of corrosion

Corrosion may occur due to carbonation or chloride ingress (Henderson & et. al, 2002). They are two main damage mechanisms that can break down the passivity of steel. After the depassivation, the steel starts to corrode which in fact will produce rust. Then due to the expansion of rust the concrete is subjected to cracking, delamination, and spalling.

- **Chlorides.** One of the causes of deterioration is linked with the ingress of high levels of chloride salts by diffusion which subsequently induces corrosion of steel (Fig. 2.25). It can be induced by using deicing chemicals or due to the marine environment. The chloride destroys the passive layer of iron oxide and contributes to the corrosion rate. Corrosion appears if the concentration of chloride is about 0.6 % of the binder mass.

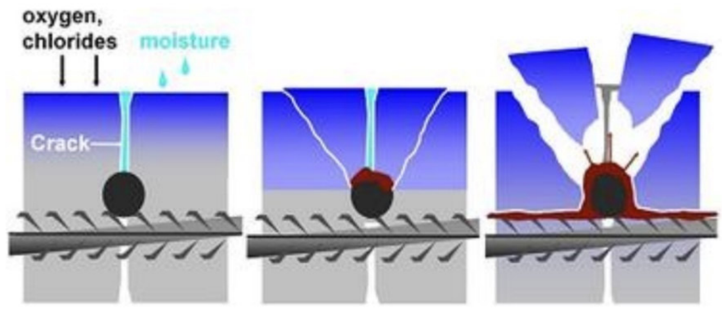


Figure 2.25 Corrosion due to chlorides

Analysis and research have found that over 50% of the structure had cast-in chlorides. This comes from admixtures that consist of chlorides or from chloride-contaminated aggregates. There is another story by considering car park structure. In that case, vehicles can bring chloride salts into the structure. Analysis of the region where the chlorides gain and cause the corrosion shows that chlorides are concentrated in the most often driven through positions or in the parking bays (Fig. 2.26).

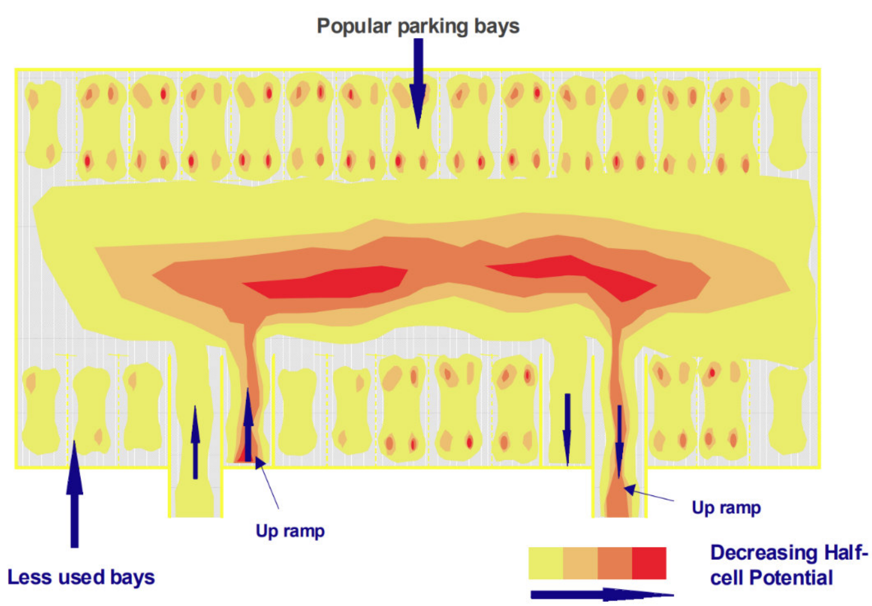


Figure 2.26 Areas of chloride concentration beneath vehicle tracks (Henderson & et. al, 2002)

- **Carbonation.** It's a calcium carbonate given from the reaction of carbon dioxide (CO_2) and the calcium hydroxide. This reaction reduces the pH from 13 around to 9 and it means that at this value the passive layer of steel breaks down and corrosion may occur. The most suitable condition for the reaction of carbonation is when there is enough moisture to react but not enough to operate as a barrier (Fig 2.27).

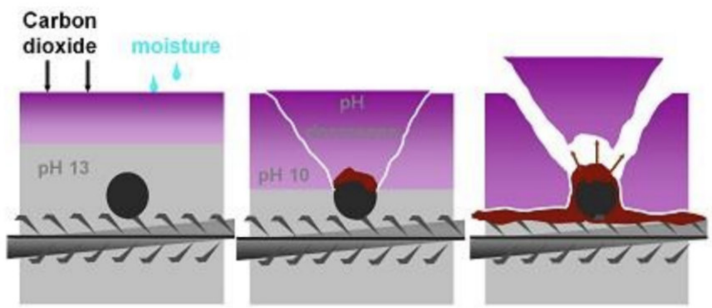


Figure 2.27 Corrosion due to carbonation

Again, speaking about the car park structure. There are many cases of premature corrosion caused by carbonation due to low concrete cover or low concrete quality, and so on. The environment in the car park cyclically changes from the dry conditions, which velocity carbonation, to the wet conditions, which accelerate corrosion. One thing to note is that cars create carbon dioxide from exhausts and if there is poor ventilation in the car park structure, subsequently it can lead to the accumulation of carbonation.

- **Service life of the structure** solely depends on its design and detailing, proportional mixing of materials, manufacturing of concrete and its placement, design methods, and its timely repairs (maintenance), moreover, environmental changes and load changes are also necessary (Karolina, n.d.). Since chemicals or fluids such as water participate in the degradation of steel and concrete, so it's also necessary to consider the permeability of concrete. The actual meaning of service life is the period of time where the structure satisfies the construction requirements and performances. Two time periods related to the service life of the reinforced concrete are introduced (Fig 2.28):
  - The initiation period t_i is the time from the concrete placing to the point where the reinforcement is depassivated.
  - The propagation period t_p is the time where the corrosion in steel starts until the moment where the structure fails.

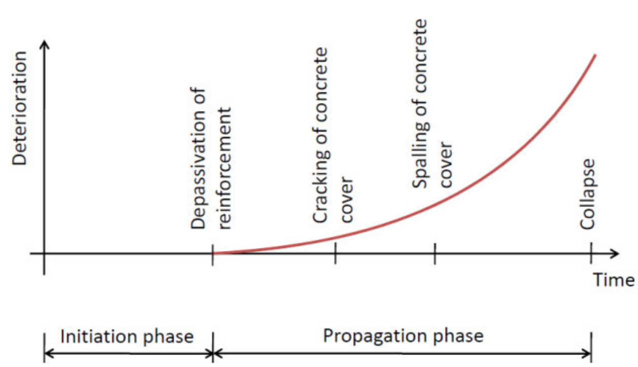


Figure 2.28 Service life

3 Summary

Nowadays, the flat slab is widely used in construction, despite the fact that there is a big problem in the face of punching shear. However, thanks to many types of research and experiences gained from the experiments and from the previous collapses due to punching shear, the design codes were invented, and the engineers have found the appropriate solutions for preventing the punching shear failure such as the column heads and drop panels, or combination of them. Obviously, increasing the strength of the concrete, thickness of the column or the depth of the slab along the slab-column connection of course will increase the punching shear strength but it will also affect the costs of the structure and its appearance. Therefore, using various types of shear reinforcement would be a good solution that will enhance the punching shear capacity of the structure. The strength of shear reinforcement depends on the type of reinforcement, the quantity, position and spacing between bars and so on. There are many types of it such as bent-up bars, headed shear studs, stirrups, shear heads, and so forth.

Speaking about the failures due to punching shear, it is necessary to note that there are many factors related to the deterioration that consequently leads to the collapse. The most frequent factors are due to the freeze-thaw attack, the carbonation or chlorides ingress which appear corrosion, and so on. However, there are many possibilities to prevent or make some repairs of such defects caused by those factors. What needs is a detailed inspection and good maintenance in time.

Summarizing the above section 2.2.2., the design procedure according to ACI-318 and Eurocode 2 is different, and the method of operation is different, but the theory of use and the goal is one, to design an appropriate structure that prevents the punching shear failure.

4 Objectives

The objectives of the thesis are as follows:

- To provide an overview of different retrofitting techniques used in modern flat slab structures.
- To provide an actual state-of-the-art based on current trending theoretical and experimental investigations for the problem.
- To study both European and American design practices, get to know their approaches and design limitations.
- To understand physical mechanisms behind shear punching failures, their prerequisites and development during service life of structures.
- To collect an appropriate research information and use it to modify the design code approaches with respect to possible deterioration processes by creating a damage model, and expand code usage when these processes couldn't be dispatched.

5 Methodology

The state-of-the-art will be based on current state of the research. The related research review will be performed for deeper understanding of the problem.

The experimental or, rather analytical, part of the thesis will be based on procedure steps described below.

First of all, it was decided to examine a real case in history (Piper's Row Car Park, Wolverhampton, U.K.) when punching shear caused failure of upmost floor which was designed as a car parking. For that reason, a real geometry of the parking floor was taken from open-access sources. It is very important to account for spans between columns. Also, it was decided to stick with real materials used in construction, such as concrete and reinforcement classes. Concrete cover also plays an important role in crack propagation and corrosion effect on reinforcement and to be designed same. The loads and loading conditions for the design will be prescribed by Eurocode 1 "Actions on Structures". For the further analysis it is required to have similar loading conditions in the both European and American design approaches.

Then, the geometry of the floor with an opening in slab structure for a ramp will be designed in SCIA Engineer software environment. The finite element method-based software will provide us with more precise values of internal forces in the slab, not to mention quick computation time, in comparison with slow and relatively sophisticated manual calculations. The calculation results will be used for the further analysis and design, such as values of unbalanced moments on top of vertical supports and design of flexural reinforcement.

To proceed with the assignment task, the design of the slab structure should be manually calculated by both Eurocode and American approaches. The concrete class, as well as reinforcement types, will be designed same in both cases. Based on the mentioned design conditions and code limitations, it will become clear which design procedure provides users with more conservative or less conservative solutions.

The next step is associated with the design range possibilities by two approaches. For that reason, by using the code provided functions it is possible to draw design ranges in dependence of few variable parameters, such as effective depth, amount of punching reinforcement and ratio of flexural reinforcement. The calculation procedure will depend upon a hypothesis when the deterioration process is subjected to action of both carbonation shrinkage, which causes crack formation, and chloride penetration, which causes corrosion of reinforcement. In return, products of corroded steel swell and cause strains in surrounding concrete leading to delamination of covering concrete. To reflect a reasonably realistic behavior under degradation in the model, we are able to linearly decrease the parameters. By doing so, the damage function could be easily drawn from the results.

6 Case study

The case study is about the real car parking Piper's Row that happened to collapse. One of the main goals of the thesis is to understand the reasons that lead to failure of the upmost floor. Therefore, the following chapter describes the case study related properties of the structure both geometry- and material-wise.

6.1 Structural system

Structural system and geometry of columns of the case study are the same of Piper's Row Car Park. The structural system of the car parking is represented by two-way flat slab supported by column structure. The longest span is 7,32 m, while the shortest span is 2,86 m long. The largest cantilevering part of the slab has a length 1,21 m. The slab height is 230 mm. The column dimensions are 300x300 mm and 250x250 mm. There is an opening in the slab structure for a ramp.

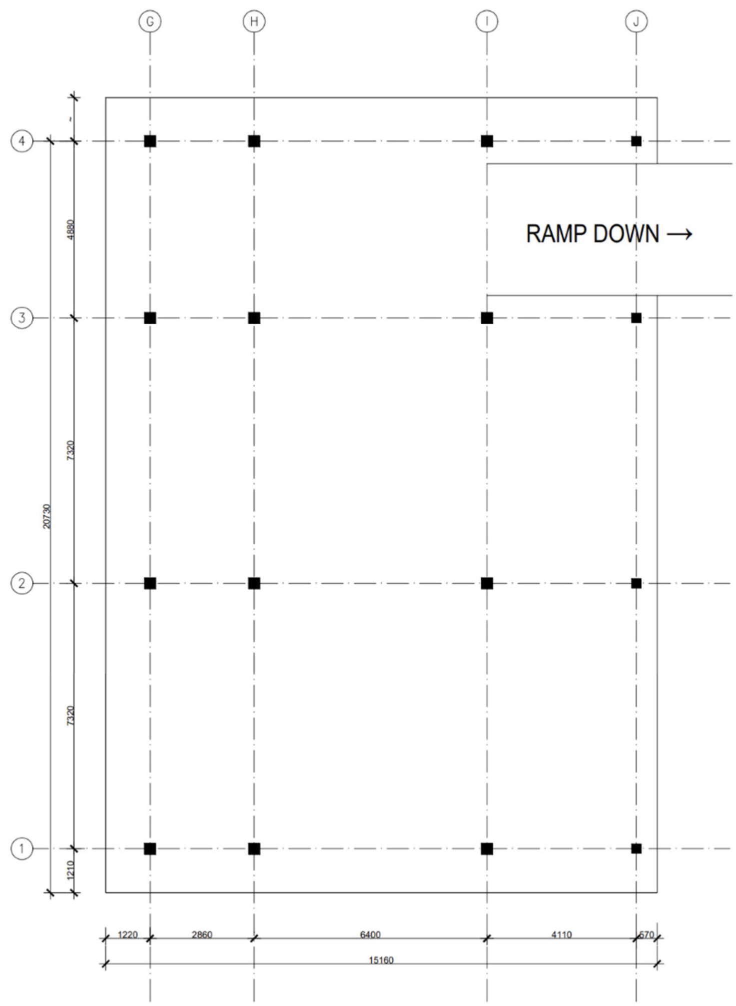


Figure 6.1 Structural system of Piper's Row Car Park

6.2 Materials

Following materials are considered for this structure according to ČSN EN 206-1: Concrete class for both slab and columns C25/30 -XC1-Dmax=22-Cl 0,20-S3. Reinforcing steel grade: B500A.

6.3 Loads

The self-weight load case accounts for the self-weight of the slab, while other permanent load case accounts for the dead load of layering. The variable loads, such as snow load and imposed load for car park buildings, were taken as per Eurocode 1 and adapted for the British National Annex (Table 1).

Table 1 Loads of the slab

Slab load	**Name**	**fk[kN/m2]**	**γ[-]**	**fd[kN/m2]**
Permanent	Self-weight	5.75	1.35	7.76
	Other perm.	0.5	1.35	0.68
			Σ	8.44
Variable	Snow load	0.52	1.5	0.78
	Category F	2.4	1.5	3.6
			Σ	4.38
			Σfd	12.82

6.4 SCIA model

For detailed analysis of flat slab was used SCIA Engineer software version 19.1. The structural system of the upper floor of the car park was modeled as a 3D structure (Fig. 6.2), which is represented by horizontal plate element (slab) and member elements (columns) rigidly interconnected to each other. Rigid supports were assigned for the bottom parts of columns.

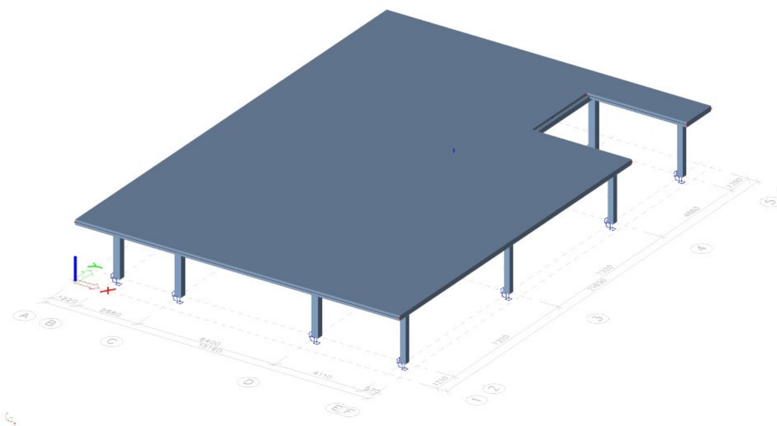


Figure 6.2 Structural model of the slab

The maximum displacement is 5.6 mm for the serviceability limit state combination.

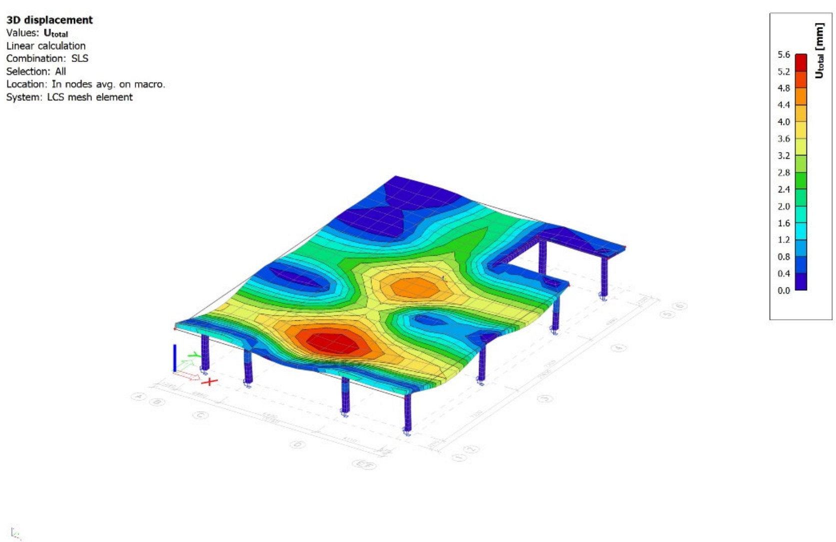


Figure 6.3 Displacement of the slab

Bending reinforcement design. Firstly, it is necessary to calculate the bending moments. By using the software required loading the slabs inserts, after that need to specify the load combinations. The load combinations are set as ULS and SCIA Engineer will automatically calculate by those combinations according to Eurocode.

Here are the results of internal forces for upper surfaces and lower surfaces respectively with x and y directions.

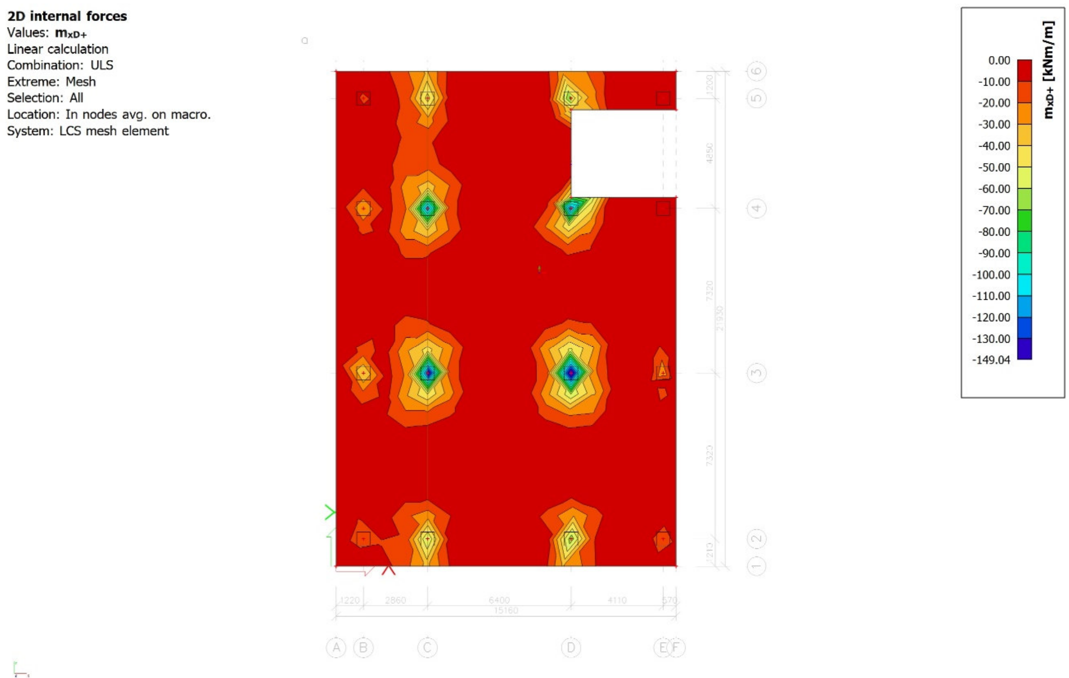


Figure 6.4 Bendng moments of upper surface in x-direction

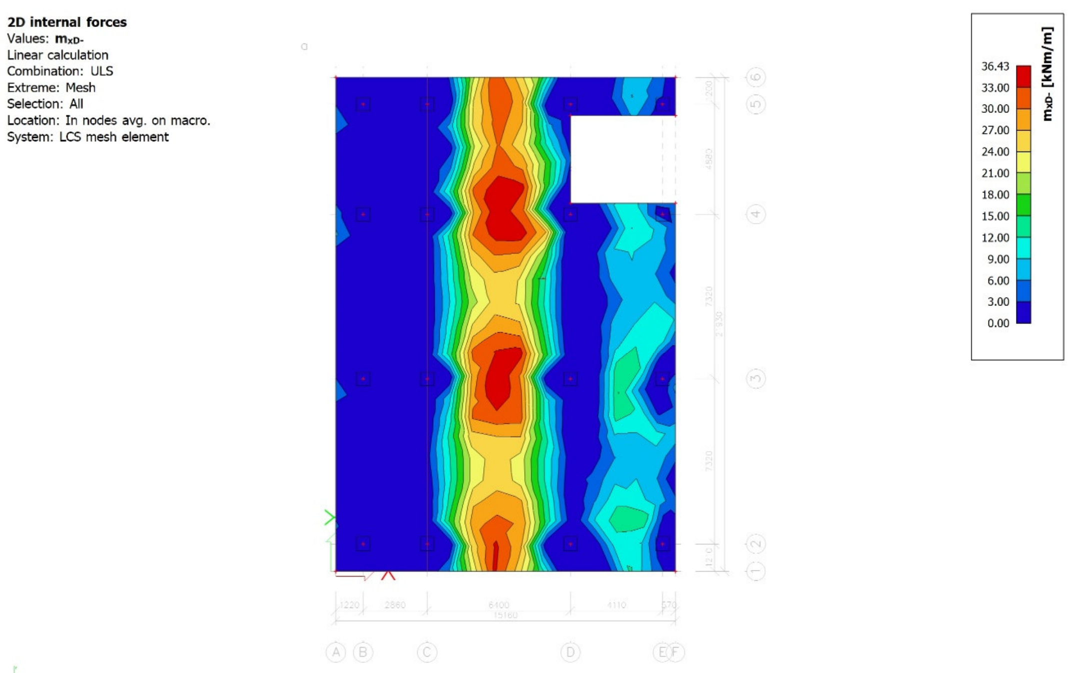


Figure 6.5 Bending moments of lower surface in x-direction

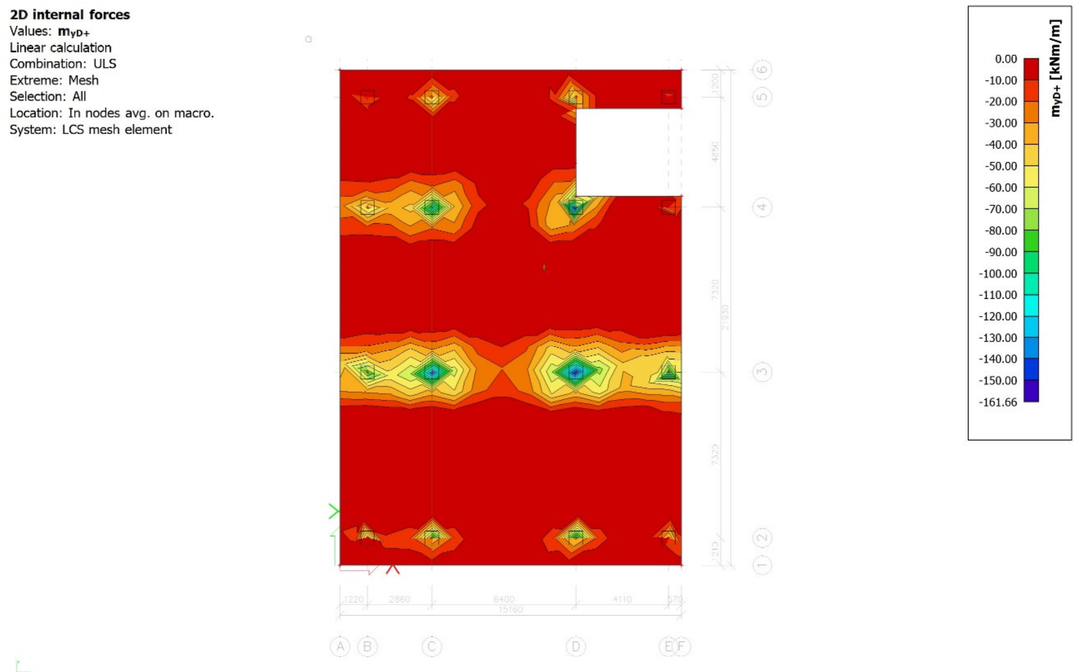


Figure 6.6 Bending moments of upper surface in y-direction

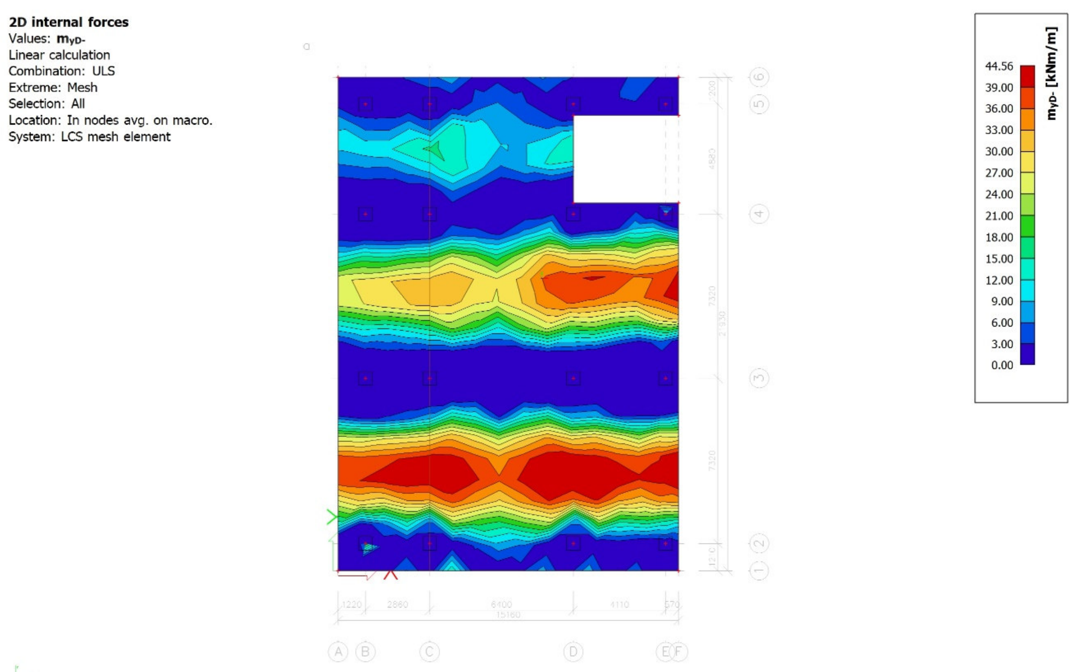


Figure 6.7 Bending moments of lower surface in y-direction

Required reinforcement design. The bending moments that obtained from the results were used to compute the required area of the reinforcements. Here are the results of required reinforcement area for upper surface and lower surface respectively with x and y directions.

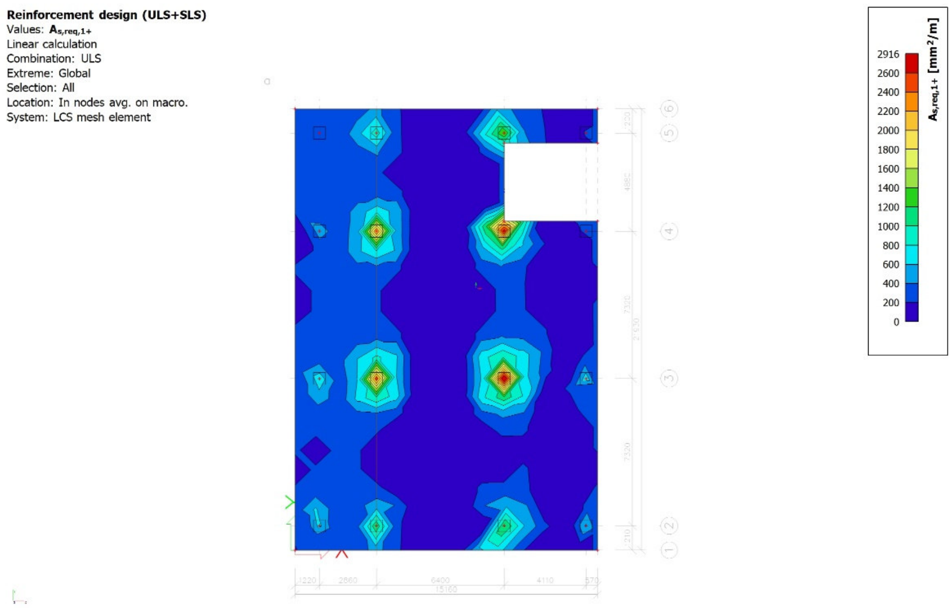


Figure 6.8 Required area in upper surface in x-direction

Figure (6.8) shows that the required reinforcements area $A_{s,req\ 1+}$ in x direction in upper surface of H2 column is 2100mm²/m.

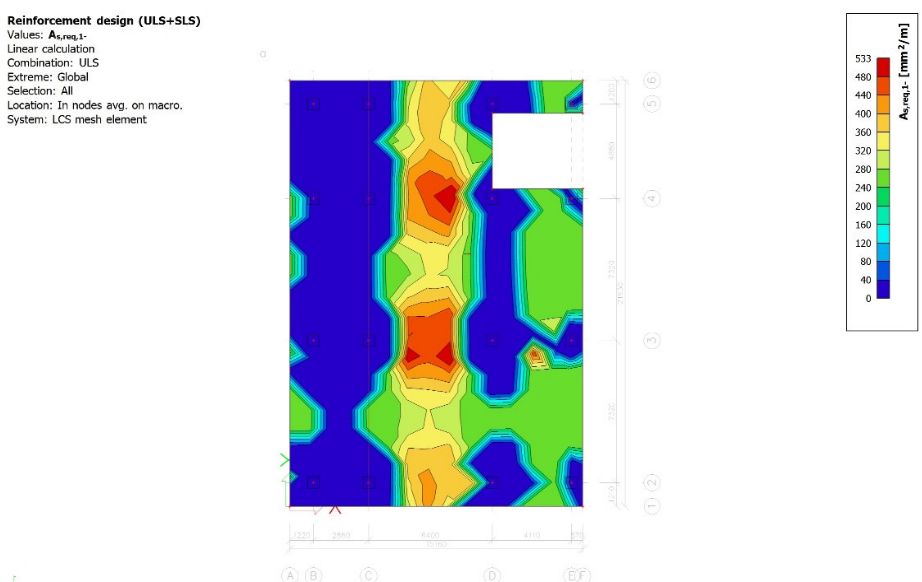


Figure 6.9 Required area in lower surface in x-direction

Figure (6.9) shows that the required reinforcements area $A_{s,req\ 1-}$ in x direction in lower surface with highest value 533mm²/m placed in mid span.

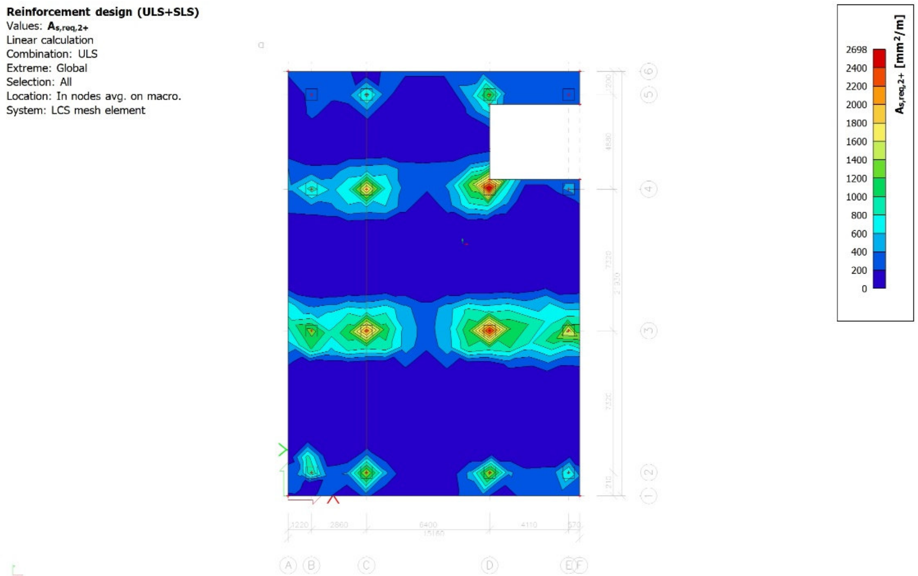


Figure 6.10 Required area in upper surface in y-direction

Figure (6.10) shows that the required reinforcements area $A_{s,req\ 2+}$ in y direction in upper surface of H2 column is 2100mm²/m.

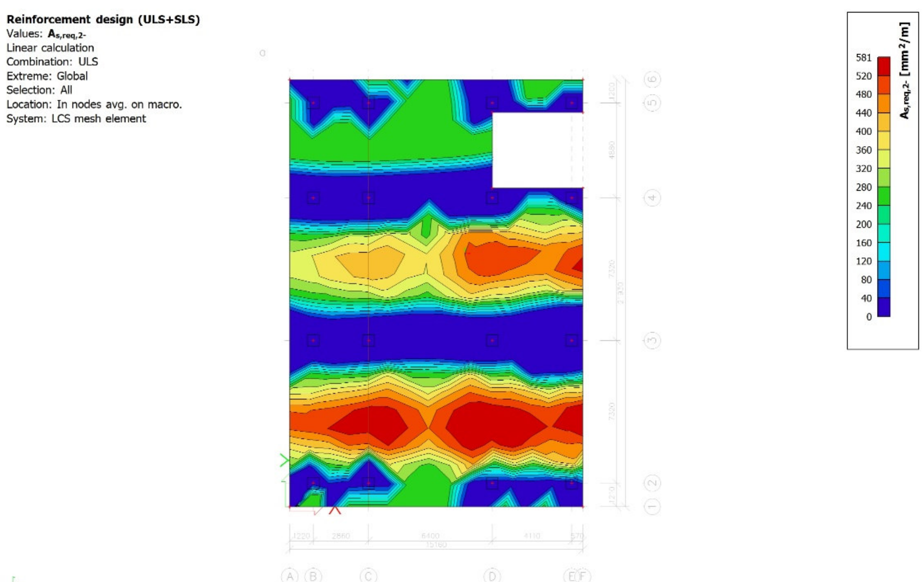


Figure 6.11 Required area in lower surface in y-direction

Figure (6.11) shows that the required reinforcements area $A_{s,req\ 2}$- in y direction in lower surface with highest value 581mm²/m placed in mid span.

For upper slab in x and y direction has provided reinforcement Ø20/140mm bars $A_{s,prov}$=2199mm²/m for required area $A_{s,req\ 1+,2+}$=2100mm²/m. For now, it is enough to consider only the upper part because it's necessary for calculation of reinforcement ratio, just to be sure if design of punching reinforcement needed.

6.5 Punching shear design according to Eurocode 2

Calculation and check of punching:

H2 is investigated column for calculation.

Input data

h_s	230 mm	f_d	12.82 kN/m2	Column dimensions	
C cover	30 mm	f_{ck}	25 MPa	c_1	305 mm
$A_{tributary}$	33.89 m2	f_{cd}	16.7 MPa	c_2	305 mm
Ø	20 mm	f_{yk}	500 MPa		
		f_{yd}	435 MPa		

Total shear force which is acting on the column H2 is V_{ed} 434.4 kN

Firstly it's needed to consider the effective depth of the slab which is taken from the value of x,y directions.

$$d_x = h - c - 1{,}5.\emptyset \qquad d_y = h - c - 0{,}5.\emptyset \qquad d = (d_x + d_y).0{,}5$$

dx 190 mm dy 170 mm d 180 mm

Preliminary check of punching

Perimeter of column

$$u_0 = 2(c_1 + c_2)$$

u_0= 1220 mm

Critical perimeter located 2*d from the loaded area

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

u_1= 3481.95 mm mm

Maximum punching shear resistance

By this check in perimeter u_0 will be seen whether the resistance of compressed concrete is sufficient or not

$$v_{Ed} = \beta \frac{V_{Ed}}{u_i d}$$

$V_{ed,0}$ 2.59 MPa

Coeff. expressing position of the column, for inner column factor ß 1.31

Due to the different spans(more than 25%), the increasing facotr needs to be find out, as follow:

$$\beta = 1 + k \cdot \frac{M_{Ed}}{V_{Ed}} \cdot \frac{u_1}{W_1}$$

Where: k-value depending on column dimensions is equal to 0.6

Med- moment about the centroidal axis of the critical perimeter

Med is taken from SCIA Engineer

W_1-modulus of resistance of the critical perimeter

$$W_1 = \frac{c_1^2}{2} + c_1c_2 + 4c_2d + 16d^2 + 2\pi dc_1$$

W_1= 1222484mm^2

$$v_{Rd,max} = 0{,}4\nu\, f_{cd}$$

$V_{rd,max}$ 3.6 MPa

Coeff. expressing effect of additional stresses v:

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

v= 0.54

$$v_{Ed} = \frac{\beta V_{Ed}}{u_0 d} \leq v_{Rd,max}$$

After the following calculation we can see that condtion is: OK

Max. resistance with reinforcement

Next check is in perimeter u_1. By this we will know if it is possible to anchor the punching sufficiently or not

$$v_{Ed} = \beta \cdot \frac{V_{Ed}}{u_1 \cdot d}$$

$V_{ed,1}$ 0.908 MPa

$$Kmax \cdot Vrd,c = Kmax \cdot Crd,c \cdot K \cdot \sqrt[3]{(100 \cdot \rho l \cdot fck}$$

Where:
Kmax is a coeff. of max. resistance
For double headed studs K_{max} 1.8

By this simple calculation we can easily get the maximum resistance with reinforcement:
$V_{rd,c}$ 1.388 MPa

Crd,c is the reduction factor, which is 0.12
K is effect of depth, which can be found:

$$Ved \leq Vrd,c$$

$$k = 1 + \sqrt{\frac{200}{d}}$$ K= 2

Check show if the condition is satisfied: OK

Ø	20	mm
dx	190	mm
dy	170	mm
d	180	mm

Reinforcement ratio for tensile reinforcement:

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

To find out the ratio, first we need to know our cross-sectional areas for upper reinforcement
$A_{s,prov\ 1,2}$ 2199 mm²
the areas were taken from SCIA Engineer

ρ_{lx}	0.0116	
ρ_{ly}	0.0129	
ρ_l	0.0122	OK
ρ_l limit	0.02	

Resistance without reinforcement

Further calculation requires to check if the slab able to carry the load without reinforcement

$$Ved,1 = \frac{\beta \cdot Ved}{u1 \cdot d} \leq Vrd,c = \max[Crd,c \cdot k \cdot \sqrt[3]{(100 \cdot \rho l \cdot fck)};\ 0.035 \cdot \sqrt{k^3 \cdot fck}]$$

$V_{ed,1}$ 0.908 MPa
$V_{rd,c}$ 0.771 MPa
Since the following condition is not satisfied: NOT OK
It says that punching shear reinforcement is required

DESIGN OF PUNCHING REINFORCEMENT

For the design, stud rails have been taken because it has high shear load resistance, moreover it's simple and fast on-site installation.
The outer control perimeter at which shear reinforcement is not needed, calculates as:

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

u_{out} 4100.95 mm

At the same time radius of the outer control perimeter should be found: r_{out} 652.69 mm

$$r_{out} = \frac{u_{out}}{2.\pi}$$

For the layout of punching reinforcement further steps should be taken into account:
Radius of the last perimeter of studs has to be more than:

r_{out}-1.5d   382.69 mm

The first stud should be placed from column face

54 ~ 90 mm

Last stud not more than (1.5*d) from u_{out}:   270 mm

Spacing of intermediate studs $s_r \leqslant 0.75*d$

$s_r \leqslant$ 135 mm

Spacing of rails $s_t \leqslant 2*d$

$s_t \leqslant$ 360 mm

In the distance of 1.5*d from u_{out} rails must be
max. 2*d from each other
And in perimeter u_1, rails must be max 1.5*d

Number of rails must be taken the maximum value by the following calculation:

$$n \geq \max\left(\frac{2\pi(r_{out} - 1,5d)}{2d}; \frac{u_1}{1,5d}\right)$$

n ≥ 13 rails
n ≥ 6.679 rails

Design: 16 rails with stud diameter 10mm

Cross-sectional area of studs in one perimeter:

$$A_{sw} = n.A_{sw,1} \qquad Asw,1 = \frac{\pi \cdot \emptyset^2}{4}$$

Asw   1256.637 mm² A_{sw1} 78.540 mm²

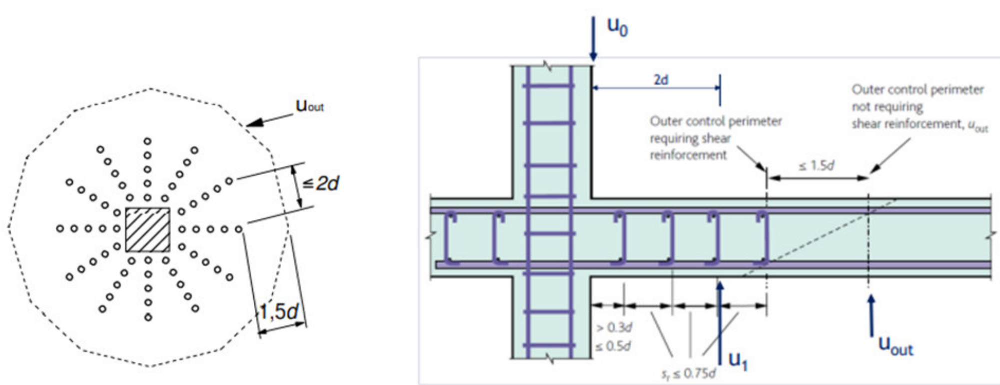


Figure 6.12 Arrangement of shear reinforcement

Check of punching reinforcement

Where shear reinforcement is required it should be calculated in accordance with the following expression:

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

$V_{ed,1}$	0.908	MPa
		OK
$V_{rd,c+s}$	1.761	MPa

Where:
s_r-spacing of intermediate studs 100mm
α-angel between the shear reinfrocement and the plane of slab
fywd,ef-effective design strength of punching shear reinforcement

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

fywd,ef	295	≤	fywd	435	MPa
	OK				

After all these check and calculations I can summarize that the shear resistance of shear reanforcement is adequate to transfer the effect of loads from the slab to column

Punching reinforcement ratio

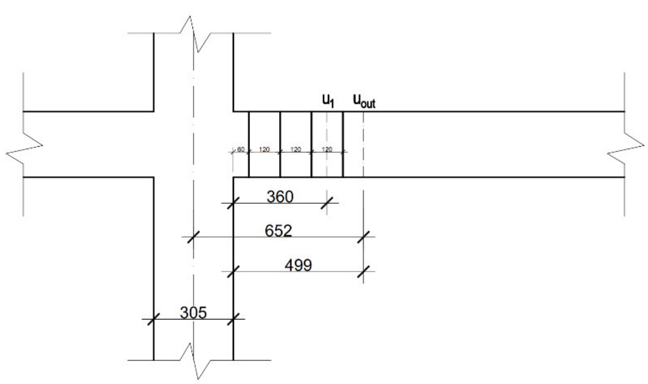
This check is to verify if we take the right amount of punching reinforcement for the design

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

Where:
ρ_{sw} - punching reinforcement ratio of slab
$\rho_{sw,min}$ - the minimum punching reinforcement ratio

ρ_{sw}	0.0024	≥	$\rho_{sw,min}$	0.0008
	OK			

Sketch of punching reinforcement:



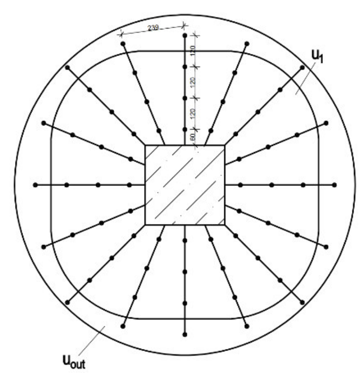


Figure 6.13 Layout of shear reinforcement according to EC2

6.6 Punching shear design according to ACI-318

Calculation and check of punching:

H2 is investigated column for calculation.

Input data

h_s	230 mm	f_d	12.82 kN/m2	Column dimensions	
C cover	30 mm	f_{ck}	25 MPa	c_1	305 mm
$A_{tributary}$	33.89 m2	f_{cd}	16.7 MPa	c_2	305 mm
Ø	20 mm	f_{yk}	500 MPa		
		f_{yd}	435 MPa		

Total shear force which is acting on the column H2 is V_{ed} 434.4 kN

Firstly it's needed to consider the effective depth of the slab which is taken from the value of x,y directions.

$$d_x = h - c - 1{,}5.\varnothing \qquad d_y = h - c - 0{,}5.\varnothing \qquad d = (d_x + d_y).0{,}5$$

dx	190 mm	dy	170 mm	d	180 mm

Shear stress resistance provided by concrete only:

$$v_u \le \varnothing.v_c$$

Where:

Ø=0.75 - strength reduction factor for shear

V_u - shear stress

V_c - shear strength of concrete

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

Where:

b_0 is critical control perimeter

b_1,b_2 are a width and depth of critical sections

$$b_0 = \sum c + 4.d \qquad b_1 = b_2 = c_1 + d$$

b_0 1940 mm

b_1,b_2 485 mm

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

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

γ_{v1}, γ_{v2} 0.4

J_c is a geometry property of critical section, analogues to polar moment of inertia

$$J_{c1} = J_{c2} = \frac{b_1.d.(b_1 + 3.b_2) + d^3}{3}$$

J_c 58398000 mm³

$M_{u1,2}$ are unbalanced column moment. In this case moments were taken from SCIA Engineer 19.1

M_{u1} 20.1 kNm

M_{u2} 0 kNm

By these values we can easily substitue them into equation and get the shear stress:

V_u 1.382 Mpa

The minimum shear stress resistance provided by concrete will be taken from following equations:

$$v_c = 0{,}17.\left(1+\frac{2}{\beta_c}\right).\sqrt{f'_c}$$

$$v_c = 0{,}083.\left(2+\frac{\alpha_s\, d}{b_0}\right).\sqrt{f'_c}$$

$$v_c = 0{,}33.\sqrt{f'_c}$$

Where:
β_c is a ratio of long side to short side of column c2/c1, which in our case equal to 1
α_s - constant for computing nominal shear strength; for inner column it's equal to 40

V_c	2.55 MPa	V_c	2.370 MPa	V_c	1.65 MPa

Then we should take the minimum value of shear strength of concrete and check it with shear stress:

$$v_u \leq \emptyset.v_c$$ Ø=0.75 - strength reduction factor for shear

Vu	1.382 Mpa	≤	ØV_c	1.238 MPa	NOT OK

As the condition doesn't met, it says that at the critical perimeter b_0 the slab has insufficient shear resistance of concrete and we need to provide shear reinforcement

DESIGN OF PUNCHING REINFORCEMENT

The shear resistance of concrete and shear reinforcement eventually will be checked by:

$$V_u \leq \emptyset.V_n$$

The maximum shear stress resistance provided by concrete and shear reinforcement will be taken from following equations:

$\emptyset \cdot Vn = \emptyset \cdot 0.5 \cdot \sqrt{f'_c}$ with rebar shear reinforcement

$\emptyset \cdot Vn = \emptyset \cdot 0.58 \cdot \sqrt{f'_c}$ with steel I or C shear heads

$\emptyset \cdot Vn = \emptyset \cdot 0.66 \cdot \sqrt{f'_c}$ with headed shear stud

ØVn	1.875 MPa	ØVn	2.175 MPa	ØVn	2.475 MPa

In consequence we can see that the remaining two results above might be sufficient for the design but to be more precise I will use the shear stress resistance of stud

Then we should take the maximum value of shear strength of concrete and shear reinforcement and check it with shear stress:

Vu	1.382 Mpa	≤	ØVn	2.475 MPa	OK

The maximum shear stress resistance provided by concrete only in case of using shear stud reinforcement:

$$v_c = 0{,}25.\sqrt{f_c'}$$

Vc 1.25 MPa

Assuming the design for one peripheral line was taken 8 rails with diameter 10mm and spacing 100mm

Calculating the required shear reinforcement:

$$A_v = \frac{(V_u - \emptyset.V_c).b_0.s_{stud}}{\emptyset.f_{y,stud}}$$

Where:

Sstud	100	mm
fy,stud	500	MPa

Av 229.76 mm²

Then it should be compared and checked with design: 8 rails Ø10/60mm

Av	229.76	mm²	≤	Av,prov	628.32	mm²	OK

By that we can find out the providing shear reinforcement:

$$v_s = \frac{A_{v,prov}.\emptyset.f_{y,stud}}{b_0.s_{stud}}$$

Vs 2.024 MPa

And this contribution of shear studs should be bigger than:

$$Vs \geq \emptyset.0{,}17.\sqrt{f_c'}$$

Vs	2.024	MPa	≥	0.64	MPa	OK

As the condition is met, we can check the shear strength with shear stud reinforcement and concrete:

$$V_u \leq \emptyset.V_n$$

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

Vn=Vc+Vs	3.274	MPa	≤	3.35	MPa	OK

It says that shear stress is limited until that point

Vu	1.382	MPa	≤	ØVn	2.456	MPa	OK

So, at the end we see by this checking that the provided shear stud reinfrocement is ok to resist punching shear

Here it's needed to determine the distance from sides of column where studs may be terminated:

$$a = \frac{\frac{V_u.b_0}{0{,}17.\emptyset.\sqrt{f_c'}} - 2.(c_1 + c_2)}{4.\sqrt{2}}$$

a 527.58 mm

Position of studs related to spacing between rails and studs shown as:

Spacing of intermediate studs:	$s_0 \leq 0{,}5.d$	90	mm
Spacing between rails:	$s \leq 2.d$	360	mm
Spacing of last stud:	$s_r \leq 0{,}5.d$	90	mm

Sketch of punching reinforcement:

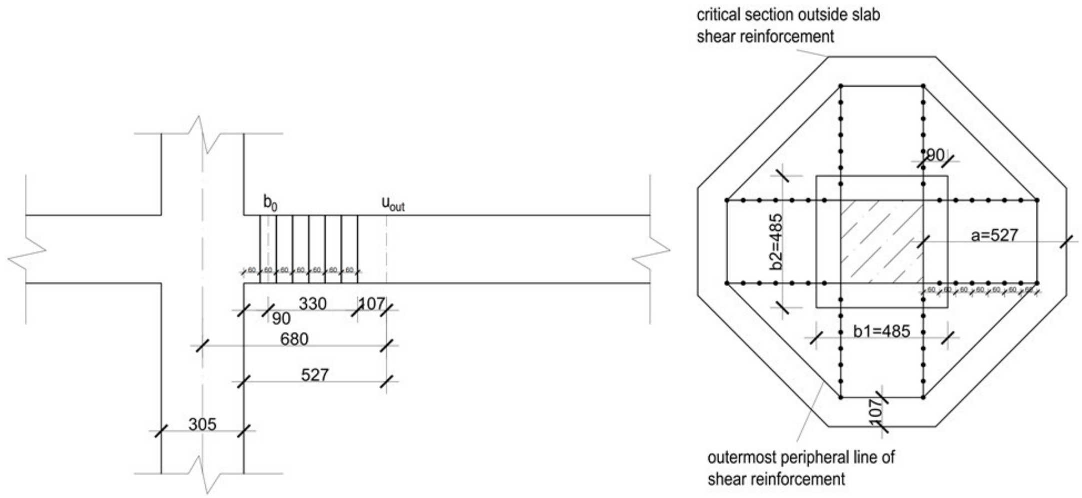


Figure 6.14 Layout of shear reinforcement according to ACI-318

6.7 Application of European and North American design codes

The reduction of mechanical properties is based on the design punching shear equations provided by both European and American codes. As it was discussed earlier, the variables are the effective depth, amount of flexural reinforcement and reinforcement ratio. The reduction factors are based on three parameters. One of them refers to degradation of effective depth by crack formation (by chloride penetration, freeze-thaw attack), another one depends on corrosion effect (area of reinforcement is reduced), and, in the case of flexural reinforcement, it depends on both effects simultaneously.

The reduction of mechanical properties of the slab according to Eurocode 2:

The first check is associated with the compressive resistance of concrete under punching shear. The effective depth is introduced into formulas for the shear stress, and the damage parameter $\kappa_d = (\frac{degraded\ effective\ depth}{initial\ effective\ depth})$ ($\kappa_d \in [1; 0{,}93]$) is introduced:

$$v_{Ed,0} = \frac{\beta \cdot V_{Ed}}{u_0 \cdot \kappa_d \cdot d} \leq v_{Rd,max} = 0.4 \cdot \nu \cdot f_{cd} \quad (20)$$

The reversed factor was used in order to show the $\kappa_{d,Ed,0} = \kappa_d^{-1}$ ($\kappa_{d,Ed,0} \in [1; 1{,}09]$) as a multiplier:

$$v_{Ed,0} = \kappa_{d,Ed,0} \cdot \frac{\beta \cdot V_{Ed}}{u_0 \cdot d} \leq v_{Rd,max} = 0.4 \cdot \nu \cdot f_{cd} \quad (21)$$

The second check is related to whether the shear reinforcement is needed or not. Where the critical perimeter is taken at a distance 2d from the loaded area and the reducing of effective depth of the shear stress is shown in detail, as follow:

$$v_{Ed,1} = \frac{\beta \cdot V_{Ed}}{u_1 \cdot \kappa_d \cdot d} \leq v_{Rd,c} \quad (22)$$

$$u_1 = 4 \cdot c + 2 \cdot \pi \cdot 2 \cdot \kappa_d \cdot d \quad (23)$$

Then the resultant $\kappa_{d,Ed,1}$ ($\kappa_{d,Ed,1} \in [1; 1{,}14]$) was taken by simple calculation from Excel, as $(\frac{v_{Ed,1,degraded}}{v_{Ed,1,initial}})$ and was introduced as a multiplier:

$$v_{Ed,1} = \kappa_{d,Ed,1} \cdot \frac{\beta \cdot V_{Ed}}{u_1 \cdot d} \leq v_{Rd,c} \quad (24)$$

The punching shear resistance is also assumed to be reduced since the ratio of flexural reinforcement is decreasing by the reduction of mechanical properties of flexural reinforcement, such as $\kappa_s = \left(\frac{corroded\ amount\ of\ flexural\ rc}{initial\ amount\ of\ flexural\ rc}\right)$ ($\kappa_s \in [1; 0{,}38]$) is the reduction parameter of the cross-sectional areas of reinforcement and κ_d is the reduction parameter of effective depth in x and y directions:

$$v_{Rd,c} = C_{Rd,c} \cdot k \cdot \sqrt[3]{100 \cdot \rho_l \cdot f_{ck}} \tag{25}$$

$$\rho_l = \sqrt{\rho_{lx} \cdot \rho_{ly}} \leq 0.02 \tag{26}$$

$$\rho_x = \frac{\kappa_{s,x} \cdot A_{s,prov.x}}{1000 \cdot \kappa_{d,x} \cdot d_x} \tag{27}$$

$$\rho_y = \frac{\kappa_{s,y} \cdot A_{s,prov.y}}{1000 \cdot \kappa_{d,y} \cdot d_y} \tag{28}$$

Then, the simultaneous reduction effect of reinforcement ratio has been expressed and transformed into the total effect $\kappa_{Rd,c} = \sqrt[6]{\frac{\kappa_{s,x}}{\kappa_{d,x}} \cdot \frac{\kappa_{s,y}}{\kappa_{d,y}}}$ ($\kappa_{Rd,c} \in [1; 0{,}75]$) in order to clearly express the parameter:

$$v_{Rd,c} = C_{Rd,c} \cdot k \cdot \kappa_{Rd,c} \cdot \sqrt[3]{100 \cdot \rho_l \cdot f_{ck}} \tag{29}$$

The third check is associated with the punching shear reinforcement. Where the combined capacity of concrete, flexural reinforcement, and shear reinforcement have been taken into account and, accordingly, the degradation is considered by reduction of mechanical properties of materials. Here, the damage parameter of shear reinforcement $\kappa_{sw} = (\frac{corroded\ amount\ of\ shear\ rc}{initial\ amount\ of\ shear\ rc})$ ($\kappa_{sw} \in [1; 0{,}38]$), and the corresponding damage parameter of effective depth κ_d are added:

$$\kappa_{Ed,1} \cdot v_{Ed,1} \leq v_{Rd,c+s} = 0.75 \cdot v_{Rd,c} \cdot \kappa_{Rd,c} + 1.5 \cdot \frac{\kappa_d \cdot d}{s_r} \cdot A_{sw} \cdot \kappa_{sw} \cdot f_{ywd,ef} \cdot \frac{1}{u_1 \cdot \kappa_d \cdot d} \cdot sin\alpha \tag{30}$$

Step by step, all the reduction parameters have been transformed into one united parameter by simple mathematic calculation in Excel, and it is introduced as $\kappa_{Rd,c+s} = \frac{v_{Rd,c+s,degraded}}{v_{Rd,c+s,initial}}$ ($\kappa_{Rd,c+s} \in [1; 0{,}54]$) into the formula below:

$$\kappa_{Ed,1} \cdot v_{Ed,1} \leq v_{Rd,c+s} = \kappa_{Rd,c+s} \cdot \left(0.75 \cdot v_{Rd,c} + 1.5 \cdot \frac{d}{s_r} \cdot A_{sw} \cdot f_{ywd,ef} \cdot \frac{1}{u_1 \cdot d} \cdot sin\alpha\right) \tag{31}$$

The reduction parameter $\kappa_{Rd,c+s}$ is linearly decreasing and $\kappa_{Ed,1}$ is increasing, the results under the observations are shown below:

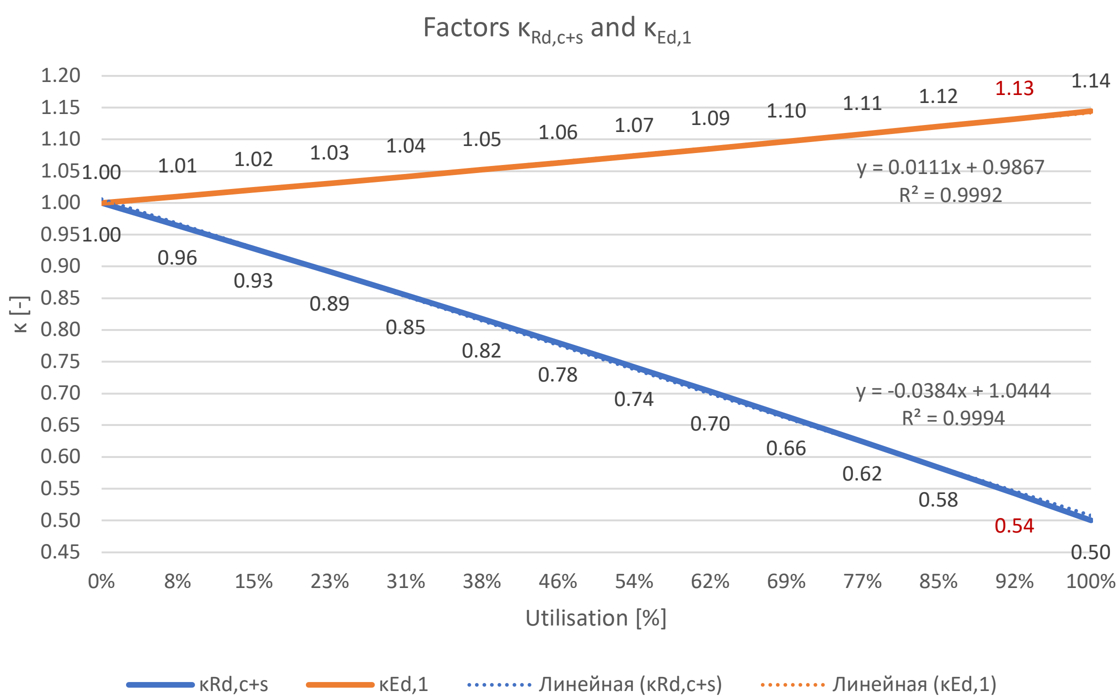


Figure 6.15 Reduction parameters for EC2

The corresponding check of punching resistance with shear reinforcement is not fulfilled and failure comes when the punching resistance reduces and reaches the point of 0.54, at the same time the shear stress increases due to degrading of mechanical properties of the materials and reaches the point of 1.13.

The reduction of mechanical properties of the slab according to ACI-318:

American Concrete Institute's code considers the punching shear capacity only with 2 checks: First check is related to compressive strength of concrete under the shear stress. The effective depth is introduced into formulas for the shear stress, and the damage parameter κ_d ($\kappa_d \in [1; 0{,}93]$) is introduced:

$$V_u = \frac{v_u}{b_0 \cdot \kappa_d \cdot d} + \frac{\gamma_{v1} \cdot M_{u1}}{J_{c1}} + \frac{\gamma_{v2} \cdot M_{u2}}{J_{c2}} \leq V_c = 0.33 \cdot \sqrt{f'_c} \tag{32}$$

$$b_0 = \sum c + 4 \cdot \kappa_d \cdot d \tag{33}$$

$$b_1, b_2 = c_{1,2} + \kappa_d \cdot d \tag{34}$$

$$J_{c1}, J_{c2} = \frac{b_1 \cdot \kappa_d \cdot d \cdot (b_1 + 3 \cdot b_2) + \kappa_d^3 \cdot d^3}{3} \tag{35}$$

Then the resultant κ_u ($\kappa_u \in [1; 1{,}11]$) was taken by simple calculation from Excel, as ($\frac{V_{u,degraded}}{V_{u,initial}}$) and was introduced as a multiplier to simplify the equation:

$$V_u = \kappa_u \cdot \left(\frac{v_u}{b_0 \cdot d} + \frac{\gamma_{v1} \cdot M_{u1}}{J_{c1}} + \frac{\gamma_{v2} \cdot M_{u2}}{J_{c2}}\right) \leq V_c = 0.33 \cdot \sqrt{f'_c} \tag{36}$$

So that we could compare the results at the end, the conditions should be the same. But since the ACI-318 doesn't take into account the reinforcement ratio, it is not. In spite of this, if consider the effect of the reinforcement ratio by the degradation parameter according to EC2, it's seen that the effect is not significant, hence, the effect of reinforcement ratio by ACI-318 can be disregarded.

The second check is associated with the punching shear reinforcement (headed shear studs). Here, the reduction of mechanical properties of the shear reinforcement is introduced as $\kappa_v = \left(\frac{corroded\ amount\ of\ shear\ rc}{initial\ amount\ of\ shear\ rc}\right)$ ($\kappa_v \in [1; 0{,}38]$) and shown in the following equations:

$$V_s = \frac{\emptyset \cdot \kappa_v \cdot A_v \cdot f_y}{b_0 \cdot s} \tag{37}$$

$$b_0 = \sum c + 4 \cdot \kappa_d \cdot d \tag{38}$$

$$V_c = 0.25\sqrt{f'_c} \tag{39}$$

$$V_n = V_c + \kappa_s \cdot V_s \tag{40}$$

Finally, the manipulations were made by simple mathematic calculation in Excel to simplify the equation and to get the total reduction parameter of punching resistance, and it is introduced as $\kappa_n = \frac{V_{n,degraded}}{V_{n,initial}}$ ($\kappa_n \in [1; 0{,}62]$) into the formula below:

$$\kappa_u \cdot V_u \leq \emptyset \cdot \kappa_n \cdot V_n \tag{41}$$

The final outcome of the reduction parameter for punching shear resistance is shown in the following chart:

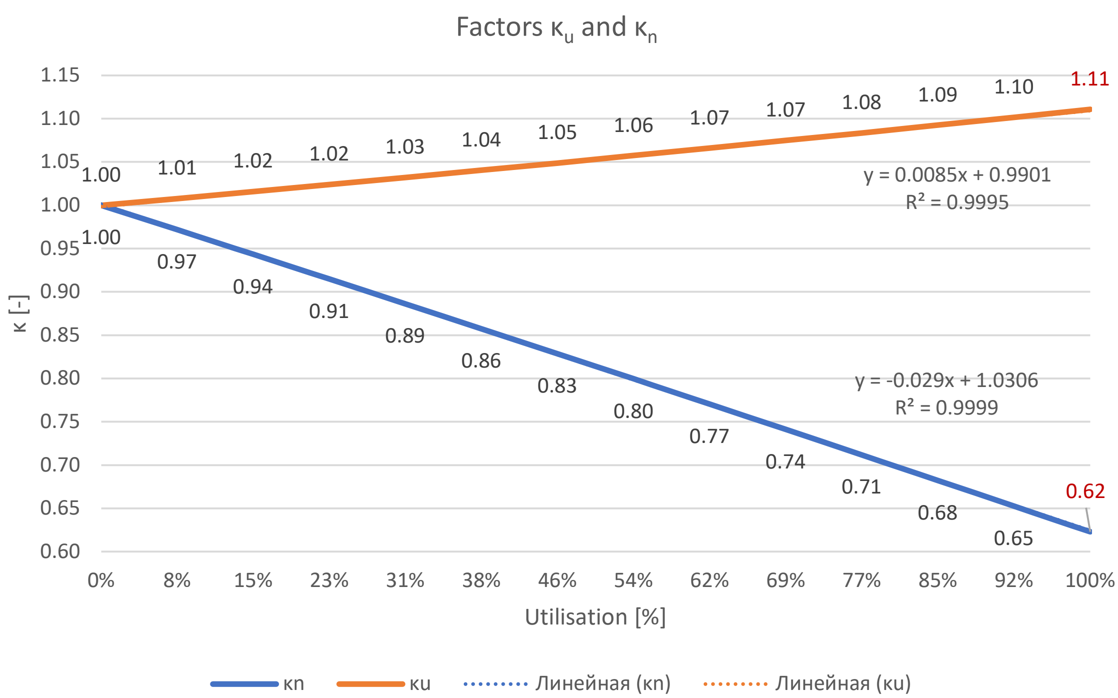


Figure 6.16 Reduction parameters for ACI-318

In the case of the North American approach, the corresponding check of punching resistance with shear reinforcement is not fulfilled and failure comes at the point of 0.62. And when the shear stress exceeds the point of 1.11.

7 Results and discussions

7.1 The damage model as per Eurocode 2 approach

The following graph (Fig. 7.1) represents punching shear resistance of compressed concrete as per the designed structure. The skewed increasing behavior of the shearing stress in the slab section is accounted for decrease of affected area. With the degradation of effective depth up to value of 166 mm no failure of concrete is expected.

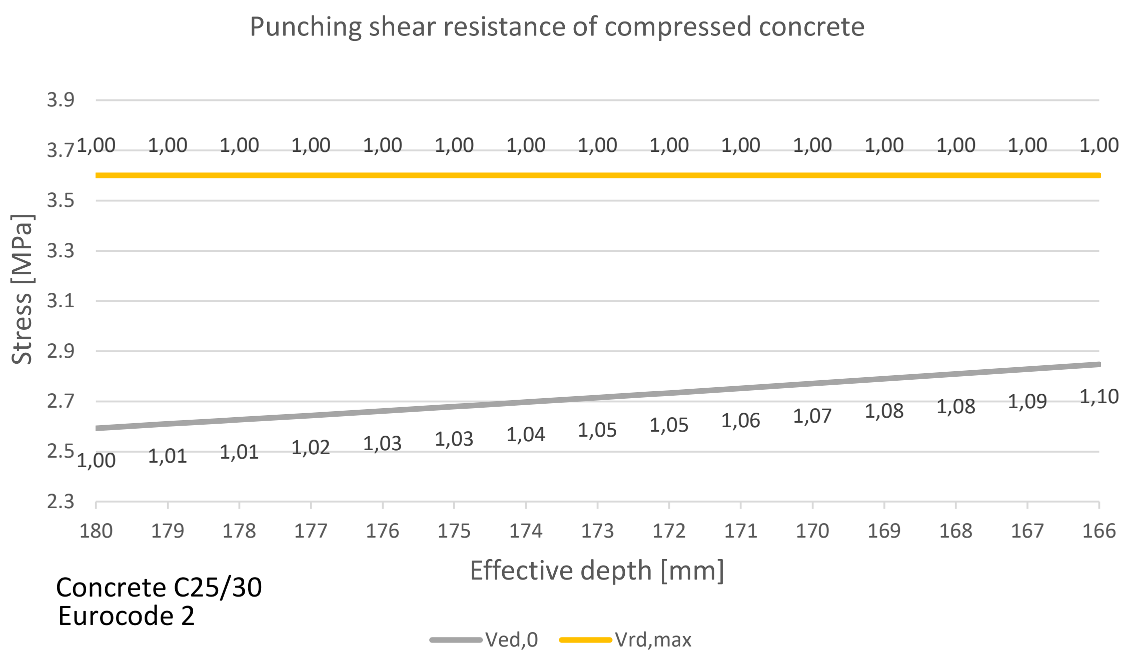


Figure 7.1 Chart of shear resistance of compressed concrete

The Figure 7.2. demonstrates the graphical understanding of the case.

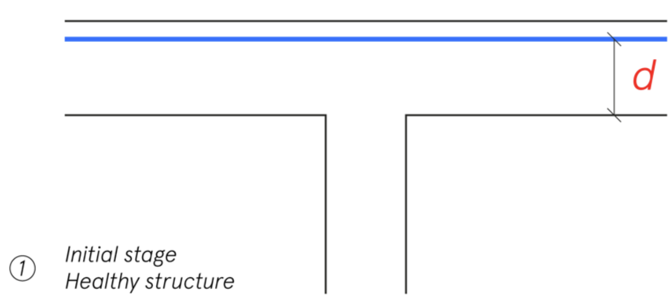


Figure 7.2 Illustration of the structure

While accounting for the punching shear resistance as per the second check which includes the effect of flexural reinforcement disregarding any effect of punching reinforcement, we could observe (Fig. 7.3.) a strong conclusion that the structure cannot withstand the loading from the very beginning. Regardless to that fact, the continuous degradation of the effective depth leads to even larger values of the shearing stress by the same principle. It could be mentioned that the effect of reinforcement ratio has a minor effect on the shear resistance, with the assumption that the effective depth is simultaneously decreased together with the amount of reinforcement.

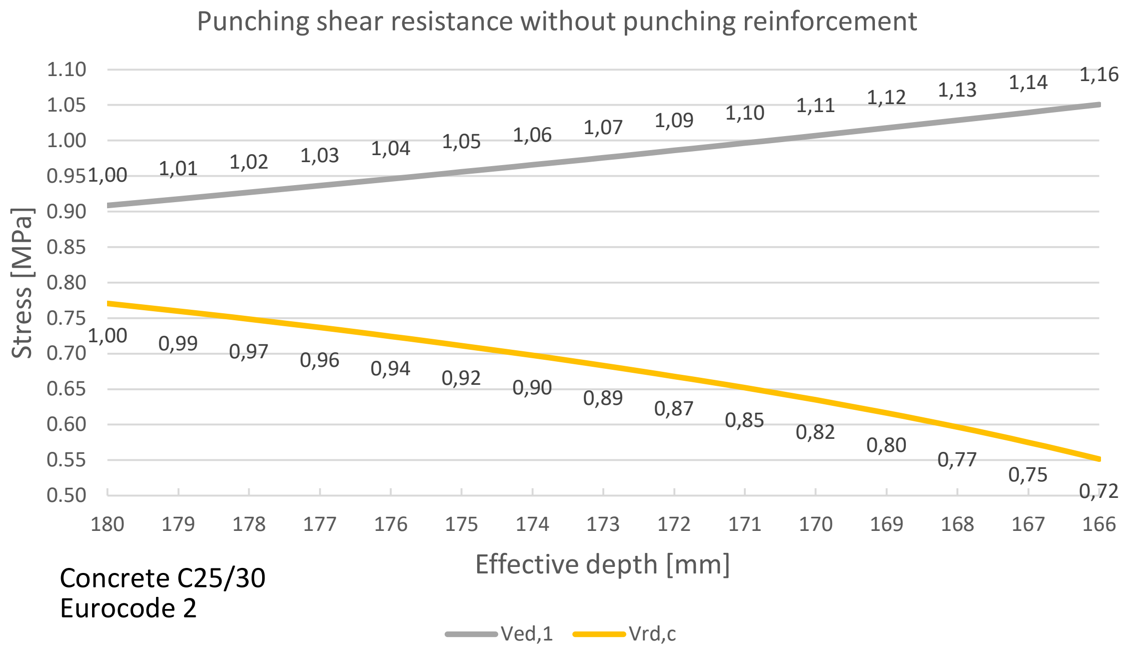


Figure 7.3 Chart of shear resistance without shear reinforcement

The Figure 7.4. illustrates the failure mode of the second check, when the rupture of flexural reinforcement is expected.

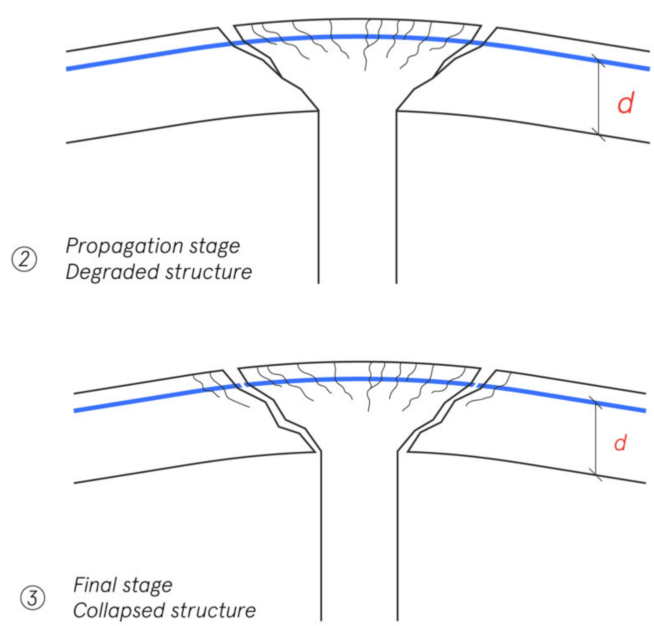


Figure 7.4 Illustration of the structure

The following graph (Fig. 7.5) expresses application of the decreasing factor κ. The strong decreasing trend for the resisting stress and for the increasing shearing stress could be observed. The failure of the structure is expected when the effective depth reaches 167 mm (7.8% degradation) and amount of shear reinforcement reaches 416.6 mm^2 (66.8% degradation).

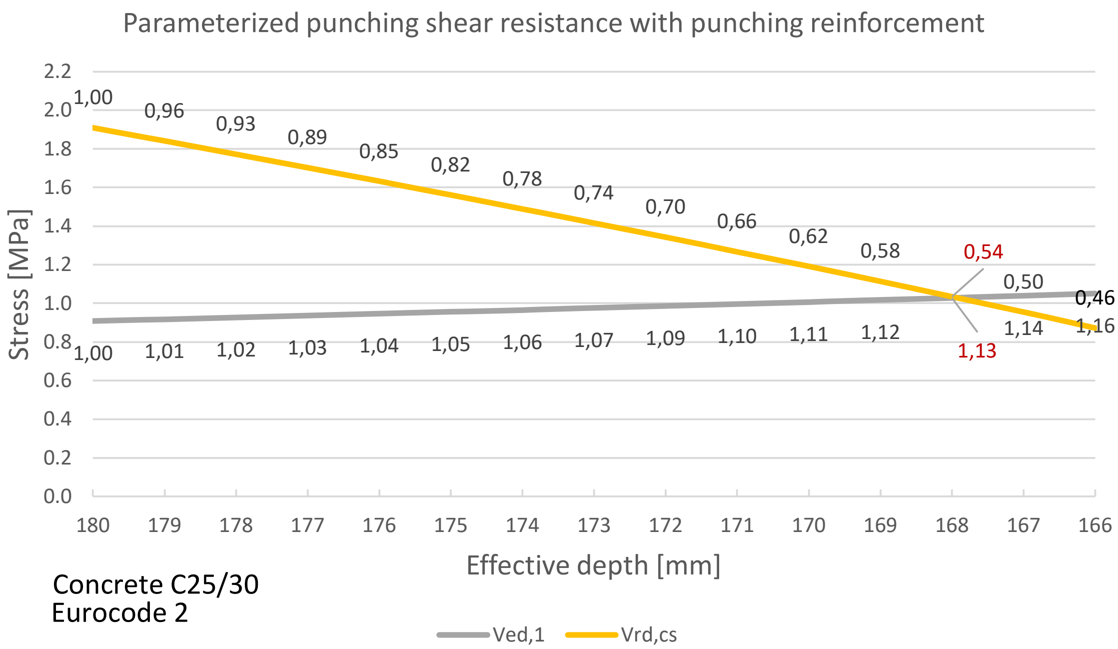


Figure 7.5 Chart of the punching shear resistance with shear reinforcement

The Figure 7.6. clearly shows the failure mode of the third check, where the structure is gradually degraded by the effect of the reduction parameters.

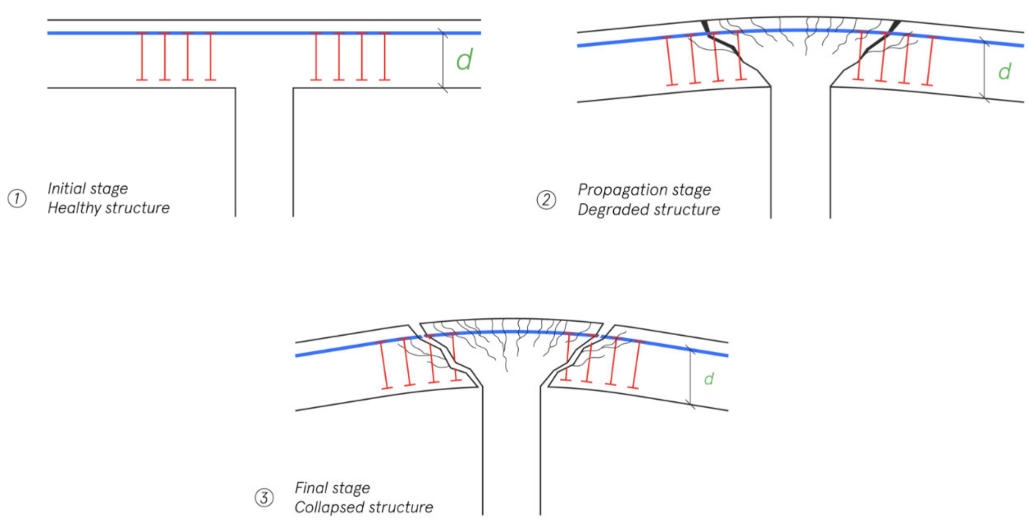


Figure 7.6 Illustration of the structure under the degradation

The Table 2 concludes the output of the calculations.

Table 2 Calculation of changed properties by EC2

ß	u0=	u1=	d	Ved,0	Vrd,max	Check	Ved,1	Vrd,c	Check	fywd,ef	Asw	Sr	Vrd,cs	Check
1,31	1220	3482	180	2,59	3,6	Ok	0,91	0,77	Not OK	295	1256,6	120	1,91	Ok
1,31	1220	3469	179	2,61	3,6	Ok	0,92	0,76	Not OK	294,75	1196,6	120	1,84	Ok
1,31	1220	3457	178	2,63	3,6	Ok	0,93	0,75	Not OK	294,5	1136,6	120	1,77	Ok
1,31	1220	3444	177	2,64	3,6	Ok	0,94	0,74	Not OK	294,25	1076,6	120	1,70	Ok
1,32	1220	3432	176	2,66	3,6	Ok	0,95	0,72	Not OK	294	1016,6	120	1,63	Ok
1,32	1220	3419	175	2,68	3,6	Ok	0,96	0,71	Not OK	293,75	956,6	120	1,56	Ok
1,32	1220	3407	174	2,70	3,6	Ok	0,97	0,70	Not OK	293,5	896,6	120	1,49	Ok
1,32	1220	3394	173	2,71	3,6	Ok	0,98	0,68	Not OK	293,25	836,6	120	1,42	Ok
1,32	1220	3381	172	2,73	3,6	Ok	0,99	0,67	Not OK	293	776,6	120	1,34	Ok
1,32	1220	3369	171	2,75	3,6	Ok	1,00	0,65	Not OK	292,75	716,6	120	1,27	Ok
1,32	1220	3356	170	2,77	3,6	Ok	1,01	0,63	Not OK	292,5	656,6	120	1,19	Ok
1,32	1220	3344	169	2,79	3,6	Ok	1,02	0,62	Not OK	292,25	596,6	120	1,11	Ok
1,33	1220	3331	168	2,81	3,6	Ok	1,03	0,60	Not OK	292	536,6	120	1,04	Ok
1,33	1220	3319	167	2,83	3,6	Ok	1,04	0,57	Not OK	291,75	476,6	120	0,95	Not OK

7.2 The damage model as per ACI-318 approach

The following chart (Fig. 7.7) shows the punching shear resistance as per the first check that relates to the strength of the concrete. Based on the result we could make a conclusion that the structure is not able to resist the loading from the very beginning i.e. the design stage. Moreover, the gradual degradation of the effective depth leads to increase of the shearing stresses.

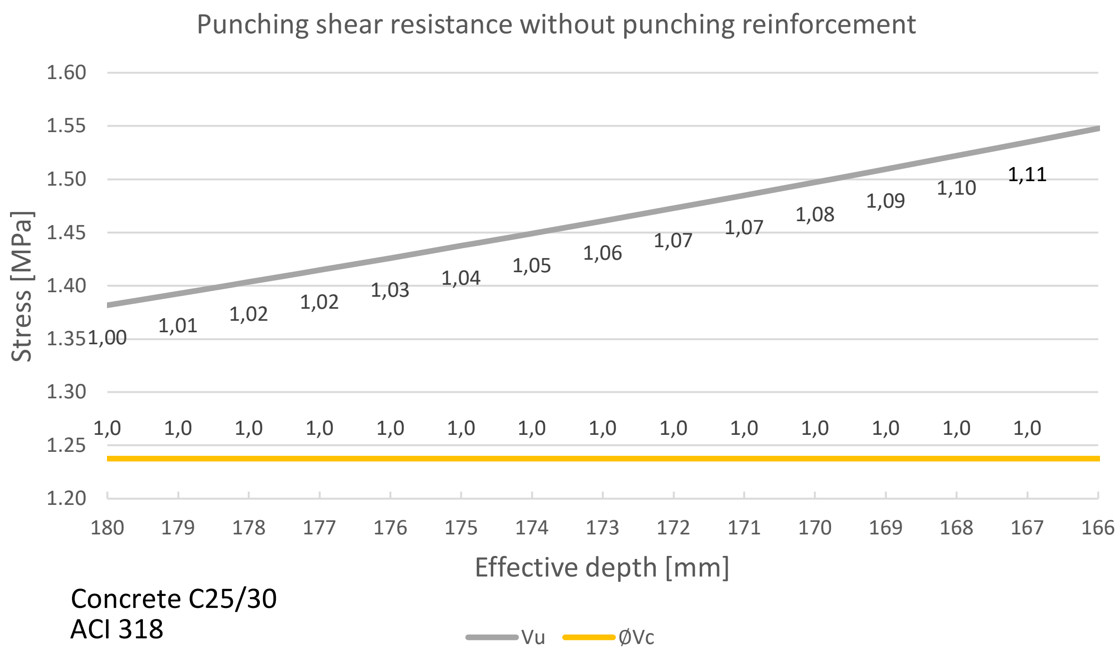


Figure 7.7 Chart of punching shear resistance without punching reinforcement

The Figure 7.8. indicates the failure mode of the first check:

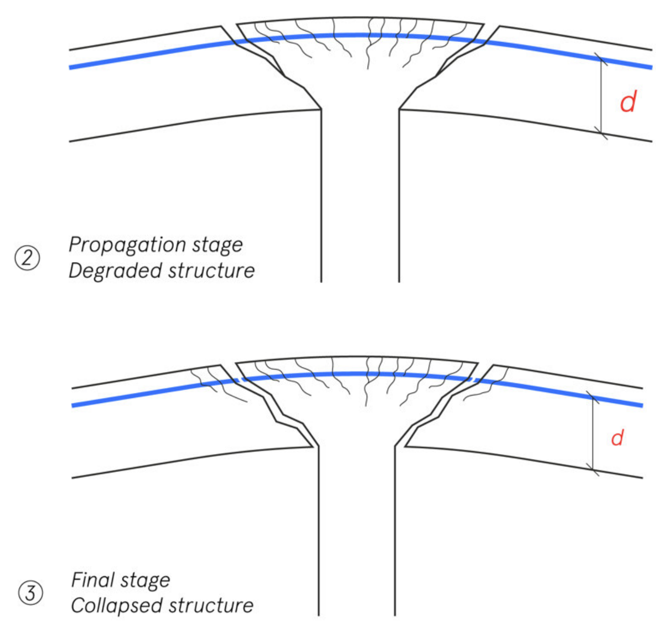


Figure 7.8 Illustration of the structure

The final result, in the following chart (Fig. 7.9) of punching shear resistance with shear reinforcement shows the expected failure of the structure by reducing the mechanical properties of the materials when the effective depth reaches 167 mm (7.2% degradation) and amount of shear reinforcement reaches 238.32 mm^2 (62.07% degradation).

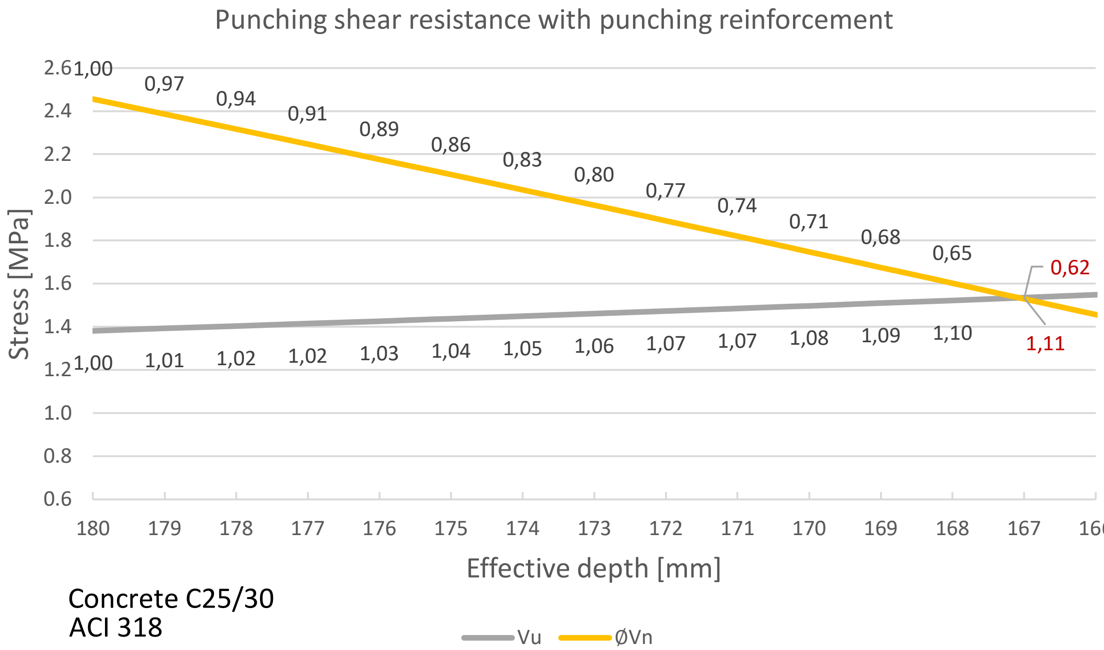


Figure 7.9 Chart of shear resistance with punching reinforcement

The Figure 7.10. clearly indicates the failure mode of the second check, where the structure is gradually degraded by the effect of the reduction parameters and the check is not fulfilled at the point of reduction factor 0.62.

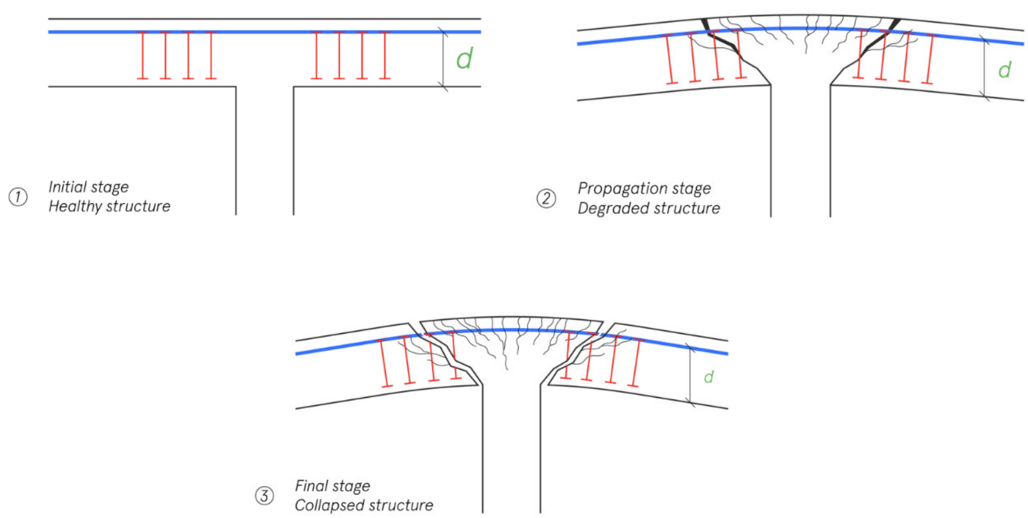


Figure 7.10 Illustration of the structure

The Table 3 concludes the output of the calculations.

Table 3 Calculation of changed properties by ACI-318

b1	d	Jc	b_0	Vu	Vc	∅Vc	Check	Vs	Vn=Vc+Vs	∅Vn	Vu	Check
485	180	58398000	1940	1,38	1,65	1,24	Not Ok	2,02	3,27	2,456	1,382	OK
484	179	57820878	1936	1,39	1,65	1,24	Not Ok	1,93	3,18	2,386	1,393	OK
483	178	57247173	1932	1,40	1,65	1,24	Not Ok	1,84	3,09	2,316	1,404	OK
482	177	56676875	1928	1,41	1,65	1,24	Not Ok	1,75	3,00	2,246	1,415	OK
481	176	56109973	1924	1,43	1,65	1,24	Not Ok	1,65	2,90	2,176	1,426	OK
480	175	55546458	1920	1,44	1,65	1,24	Not Ok	1,56	2,81	2,105	1,438	OK
479	174	54986320	1916	1,45	1,65	1,24	Not Ok	1,46	2,71	2,034	1,449	OK
478	173	54429548	1912	1,46	1,65	1,24	Not Ok	1,37	2,62	1,963	1,461	OK
477	172	53876133	1908	1,47	1,65	1,24	Not Ok	1,27	2,52	1,892	1,473	OK
476	171	53326065	1904	1,48	1,65	1,24	Not Ok	1,18	2,43	1,820	1,485	OK
475	170	52779333	1900	1,50	1,65	1,24	Not Ok	1,08	2,33	1,747	1,497	OK
474	169	52235928	1896	1,51	1,65	1,24	Not Ok	0,98	2,23	1,675	1,510	OK
473	168	51695840	1892	1,52	1,65	1,24	Not Ok	0,89	2,14	1,602	1,522	OK
472	167	51159058	1888	1,53	1,65	1,24	Not Ok	0,79	2,04	1,529	1,535	Not Ok

7.3 Comparison between European and North American design approaches

It could be clearly concluded that European approach is more complicated than the North American approach. The European approach takes into account the effect of flexural reinforcement by accounting for width of strips and effective depth in both directions. The downvote for the American approach is that we could not effectively design punching reinforcement when reinforcement ratio is too low or too high. It also could be concluded that the European approach is more conservative than the American approach following by the design code provisions we use twice as much of shear stud connectors to satisfy the design criterias. Nevertheless, the American approach is much more user-friendly since formulation of the design criterias is based on experimental investigations i.e. by introducing different factors. Not to mention that on average it took as twice as much time to design by the European approach than by the American.

In the European approach we use in total of 64 shear stud connectors radially distributed, and per one perimeter there are 16 shear stud connectors. In the American design we use in total of 56 shear stud connectors cruciformly distributed, and per one perimeter there are 8 shear stud connectors. It could be seen that total amount of shear studs is nearly the same (the difference is 8 pc.). Nevertheless, we account for twice as low in the American approach per one perimeter. However, in comparison with Eurocode 2 we receive nearly the same relative design resistance. This could be explained by the fact that the control perimeter is nearly twice as much in the Eurocode 2, as the result the shearing stresses are nearly twice lower. Regarding the punching resistance there are several formulations given in the American approach. These formulations are prescribed for different design conditions and simply prescribed by factor values. Therefore, it is not possible to draw where these factors are coming from. Nevertheless, the main conclusion is that both approaches show nearly identical results as per design conditions.

7.4 Discussion about the failure of Piper's Row Car Park

Based on the extensive literature review on the application of two approaches, which are very different in principle, it could be concluded that the main factors for the failure were the corrosion of reinforcement by leakage of water and oxide through cracks developed by mechanisms of either chloride penetration or carbonation. The application of both approaches shows a dramatic decrease of resistance with the drop of the amount of reinforcement. However, the effective depth plays a significant role, in reality, it didn't go below the change of 60mm. Therefore, damage to covering concrete has a huge role in the shear punching development. This observation shows the importance of correctly chosen exposure class, and, hence, covering depth. Additionally, there are many possibilities for how to retrofit and repair the damaged building. One of them is to use the post-installed punching shear reinforcement or inclined post-installed reinforcement. Other strengthening methods are concrete overlay, enlarged columns, concrete or steel collars, and so on. The main thing is to make those repairs in the correct way and to guarantee a permanent construction inspection.

8 Conclusions

The conclusions of the thesis are as follows:

- In the thesis, particularly in the state-of-the-art, the flat slab and possible retrofitting systems has been reviewed such as column head or drop panels. Their applicability and functionality with respect to punching shear.
- Also discussed, the possible enhancement of the strength of the slab that prevents punching shear by using shear reinforcements such as stirrups, bent-up bars, shear studs, band reinforcement, and so on.
- The code provisions related to punching shear design according to Eurocode 2 and ACI-318 were introduced and totally understood.
- The deterioration factors that cause punching shear failure were listed, particularly, the degradation due to the freeze-thaw attack, the carbonation or chloride ingresses, and their impact on the service life of the structure.
- In the case study, the collapse of the real car park (Piper's Row Car Park, Wolverhampton, U.K.) has been examined and the importance was to provide the punching shear design according to Eurocode 2 and ACI-318, after that, the aim was to analyze the failure by introducing the reduction parameter in those two provisions.
- From my point of view or correctly will be from my observations and results of view, the design of punching reinforcement by Eurocode 2 seems to be more complicated since it considers a flexural reinforcement while the ACI-318 does not take into account the longitudinal reinforcement because it's assumed that the punching failure will be due to shear since the flexural capacity of the structure is correctly designed and it has a constant strength.
- In the future it will be interesting to develop the topic by reinforcing the results with the experimental research.

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