VILNIAUS GEDIMINO TECHNIKOS UNIVERSITETAS

STATYBOS FAKULTETAS

GELŽBETONINIŲ KONSTRUKCIJŲ IR GEOTECHNIKOS KATEDRA

Simona Abezgaus

KOLONŲ PADŲ TYRIMAS
INVESTIGATION OF COLUMN SHOES

Baigiamasis magistro darbas

Statinių konstrukcijų studijų programa, valstybinis kodas 6211EX040

Pastatų konstrukcijų specializacija

Statybos inžinerijos studijų kryptis 02T

Vilnius, 2020

VILNIAUS GEDIMINO TECHNIKOS UNIVERSITETAS

STATYBOS FAKULTETAS

GELŽBETONINIŲ KONSTRUKCIJŲ IR GEOTECHNIKOS KATEDRA

TVIRTINU
Katedros vedėjas

____
(Parašas)

Juozas Valivonis
(Vardas, pavardė)

____
(Data)

Simona Abezgaus

KOLONŲ PADŲ TYRIMAS

INVESTIGATION OF COLUMN SHOES

Baigiamasis magistro darbas

Statinių konstrukcijų studijų programa, valstybinis kodas 6211EX040

Pastatų konstrukcijų specializacija

Statybos inžinerijos studijų kryptis 02T

Vadovas Dr. doc. Mykolas Daugevičius 20200528
(Moksl. laipsnis/pedag. vardas, vardas, pavardė) (Parašas) (Data)

Lietuvių kalbos konsultantas __________ __________
(Moksl. laipsnis/pedag. vardas, vardas, pavardė) (Parašas) (Data)

Vilnius, 2020

VILNIAUS GEDIMINO TECHNIKOS UNIVERSITETAS
STATYBOS FAKULTETAS
GELŽBETONINIŲ KONSTRUKCIJŲ IR GEOTECHNIKOS KATEDRA

TVIRTINU
Katedros vedėjas

____
(parašas)

Juozas Valivonis
(vardas, pavardė)

Technologijos mokslų sritis

Statybos inžinerijos studijų kryptis

Statinių konstrukcijų studijų programa, valstybinis kodas 6211EX040

Pastatų konstrukcijų specializacija

BAIGIAMOJO MAGISTRO DARBO
UŽDUOTIS
Nr. ....
Vilnius

Studentei *Simona Abezgaus*..
(vardas, pavardė)

Baigiamojo darbo tema *KOLONŲ PADŲ TYRIMAS*

patvirtinta 2020 m. balandžio 28 d. dekano įsakymu Nr. 129st

Baigiamojo darbo užbaigimo terminas 2020 m. birželio 1 d.

BAIGIAMOJO DARBO UŽDUOTIS:

Atlikti surenkamų gelžbetoninių kolonų jungties su pamatu sprendinių analizę. Nustatyti, kokie įvairių gamintojų siūlomi sprendiniai projektuojant jungtį naudojant plienines įdėtines detales. Atlikti kolonos ir pamato jungties skaitinę analizę panaudojant pasirinkto gamintojo įdėtines detales, kitu variantu vertinant, kad kolona su pamatu sumonolitinta. Pateikti skaitinės analizės įtempių pasiskirstymą betone ir armatūroje, kai jungtis su pamatu užtikrinta įdėtinėmis detalėmis ir jungtis yra monolitinė. Pateikti išvadas.

....

....

Baigiamojo darbo rengimo konsultantai: ....

....

....

(vardas, pavardė, mokslinis laipsnis ir vardas)

Vadovas ....
(parašas)

Mykolas Daugevičius dr. doc.
(vardas, pavardė , mokslinis laipsnis ir vardas)

Užduotį gavau

....
(parašas)

Simona Abezgaus
....
(vardas, pavardė)

2020-05-28
(data)

Vilniaus Gedimino technikos universiteto egzaminų sesijų ir baigiamųjų darbų rengimo bei gynimo organizavimo tvarkos aprašo

2 priedas

(Baigiamojo darbo sąžiningumo deklaracijos forma)

VILNIAUS GEDIMINO TECHNIKOS UNIVERSITETAS

Simona Abezgaus, 20123264

(Studento vardas ir pavardė, studento pažymėjimo Nr.)

Statybos fakultetas

(Fakultetas)

Statinių konstrukcijos, PKfmu-18

(Studijų programa, akademinė grupė)

BAIGIAMOJO DARBO (PROJEKTO)
SĄŽININGUMO DEKLARACIJA

2020 m. gegužės 28 d.

Patvirtinu, kad mano baigiamasis darbas tema „Kolonų padų tyrimas" patvirtintas 2020 m. balandžio 28 d. dekano potvarkiu Nr. 129st, yra savarankiškai parašytas. Šiame darbe pateikta medžiaga nėra plagijuota. Tiesiogiai ar netiesiogiai panaudotos kitų šaltinių citatos pažymėtos literatūros nuorodose.

Parenkant ir įvertinant medžiagą bei rengiant baigiamąjį darbą, mane konsultavo mokslininkai ir specialistai: docentas daktaras Mykolas Daugevičius. Mano darbo vadovas docentas daktaras Mykolas Daugevičius.

Kitų asmenų indėlio į parengtą baigiamąjį darbą nėra. Jokių įstatymų nenumatytų piniginių sumų už šį darbą niekam nesu mokėjęs (-usi).

(Parašas)

Simona Abezgaus

(Vardas ir pavardė)

Vilniaus Gedimino technikos universitetas
Statybos fakultetas
Gelžbetoninių konstrukcijų ir geotechnikos katedra

ISBN ISSN
Egz. sk. ....
Data ..........-.....-.....

Antrosios pakopos studijų **Statinių konstrukcijų** programos magistro baigiamasis darbas

Pavadinimas	**Kolonų padų tyrimas**
Autorius	**Simona Abezgaus**
Vadovas	**Mykolas Daugevičius**

Kalba: anglų

Anotacija

Šiame darbe yra tiriama surenkamos kolonos-pamato jungtis naudojant kolonų padus. Darbą sudaro dvi pagrindinės dalys: literatūros apžvalga ir bendros žinios, kolonos-pamato jungties baigtinių elementų analizė. Pirmoje dalyje yra apžvelgiami pagrindiniai sprendiniai, naudojami surenkamos kolonos ir pamato jungčiai, apžvelgiami kitų autorių darbai, susiję su kolonų padų elgsenos tyrimais ir apžvelgiami įvairių gamintojų siūlomi sprendiniai. Antroje dalyje, naudojant baigtinių elementų analizės programinę įrangą DIANA, yra sumodeliuoti ir apkrauti trys bandiniai. Pirmas bandinys yra kolonos padų jungtis su skiediniu, antras – kolonos padų jungtis be skiedinio, trečiasis – monolitinė kolona. Apžvelgiami ir palyginami skaitinės analizės rezultatai. Darbe pateikiamos išvados, bei rekomendacijos.

Darbą sudaro šešios dalys: įvadas, literatūros apžvalga ir bendrosios žinios , kolonos-pamato jungties baigtinių elementų analizė, išvados ir rekomendacijos, literatūros sąrašas, priedai.

Darbą sudaro – 75 puslapiai teksto be priedų, 58 iliustracijos, 8 lentelės, 28 bibliografiniai šaltiniai.

Atskirai pridedami darbo priedai.

Prasminiai žodžiai: kolonos padai, surenkamas gelžbetonis, baigtinių elementų analizė, PEIKKO, kolonos - pamato jungtis

Vilnius Gediminas Technical University
Faculty of Civil Engineering
Department of Reinforced Concrete Structures and Geotechnics

ISBN ISSN
Copies No. ....
Date ..........-.....-.....

Master Degree Studies **Structural Engineering** study programme Master Graduation Thesis

Title	**Investigation of column shoes**
Author	**Simona Abezgaus**
Academic supervisor	**Mykolas Daugevičius**

Thesis language: English

Annotation

In this work, prefabricated column-foundation connections using column shoes are investigated. The work consists of two main parts: literature overview and general knowledge, column-foundation connection finite element analysis. The first part reviews the main solutions used for the assembly of the prefabricated column and foundation, reviews the work of other authors related to the study of the behavior of the column shoes and reviews the solutions offered by various manufacturers. In the second part, three samples are modeled and loaded using DIANA finite element analysis software. The first sample is the connection of the column shoes with the grout, the second is the joint of the column shoes without grout, and the third is a cast-in-situ column. The results of numerical analysis are reviewed and compared. The work presents conclusions and recommendations.

The work consists of six parts: introduction, literature overview and general knowledge, FEA analysis of column- to foundation connection, conclusions and recommendations, bibliography, annexes.

Thesis consists from - 75 pages of text without annexes, 58 figures, 8 tables, 28 bibliographical entries.

Work annexes are attached separately.

Keywords: column shoes, precast reinforced concrete, PEIKKO, column-to-foundation connection

Contents

LIST OF TABLES

LIST OF FIGURES

ABBREVIATIONS AND DESCRIPTIONS

CE marking – certification mark that indicates conformity with safety, health and environmental protection standards for products sold within the European Economic Area

CEN – European Committee for Standardization

CUAP – Common Understanding of Assessment Procedures

CAGR – Compound annual growth rate

DCM – medium level of absorbing energy that allows high levels of ductility and there are responsive design demand

DCH – high level of absorbing energy that allows an even higher level of ductility and there are responsive strict and complicated design demands

EC2 - EN 1992-1-1 Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for Buildings, 2004 CEN

EC3 - Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings, 2005 CEN

EC8 - Eurocode 8: Design of structures for earthquake resistance -Part 1: General rules, seismic actions and rules for buildings, 2005 CEN

ETA – European Technical Assessment

TS2 - TS 1992-2 Technical specification: Design of fastenings for use in concrete – Part 2: Headed studs, 2007 CEN

TS – Technical Specification

EN – European standards

RTD – Research and Technological Development

SFS-EN – Finish standards of Eurocode

DIN-EN - German standards of Eurocode

Cast in situ – a structural concrete element that is being constructed and poured on the site

Precast – structural element that is manufactured off-site and being brought to the site for installation

ACI – American Concrete Institute

ISO – International Organization for Standardization

FEA – finite element analysis

1. INTRODUCTION

Following the different market research, the global precast concrete market size was valued at 99.4 Billion EUR in 2018 and 82.56 Billion EUR in 2019 and from 2020 till 2027 it is expected to increase at a CAGR of 6.3%. (1) So, nowadays to be an innovative and competitive market player it is very important to be ready to reshape and transform traditional engineering and construction business models by learning and adapting new manufacturing and prefabrication techniques. It allows to work faster, cleaner, and safer.

This movement is happening across the globe. It is happening in developed countries~~,~~ due to prefabrication and labour cost being comparatively expensive, according to references, the duration of construction can be reduced to as much as eight weeks. In developing countries (mainly in Asia) due to rapidly growing middle-class population that demands affordable housings. Thus, developers can meet demand faster.

Relating to it, rapidly growing proposed solutions for prefabricated constructions, however, beside main technical characteristics and price, it is not always clear how they differ from each other.

In prefabrication one of the most common connection is column-foundation connection. A column without a reasonably strong connection to the foundation~~,~~ can cause the structure to collapse. It can happen due to the slightest movement or sway caused by wind, earthquake, vibrations due to equipment or occupants, etc. This is why it is so important to understand well how this connection behaves. One of the most popular solutions for column-foundation connection in Lithuania is column shoes.

It provides a convenient solution for connection to foundations and linking columns together. The column shoes are set into the precast columns, the anchor bolts into the foundation on site. During assembly, the elements are connected mechanically using nuts. To reach moment-resisting stiff connection between precast concrete columns and foundations, or between precast concrete columns, the joint must be grouted by low-shrinking seal mortar. Grout must be hardened before the column is loaded by other structures.

In the European market, main companies that provide such a solution are Finish companies Anstar, Peikko, Semko, Rsteel, and German companies Halfen, Pfeifer. In Lithuania leading company is Peikko. However, living in an increasingly globalized market, where more and more companies are carrying out projects around the world, an engineer must be prepared to adapt to different market standards. Thus, it is important to be interested in different solutions that go beyond the local market. Also~~,~~ the engineer during the design is aware of the differences and similarities between the parts supplied by different suppliers

The manufacturers state that this solution is not in any way inferior to cast-in-situ construction and bending moment resistance of connection is achieved accordingly. This statement may seem strange to some engineers. In both cases, connections are assumed as a rigid one. Despite it, in real life, some rotation is still possible because the column shoes is a bolted connection, it seems that it should be less rigid and reliable than the connection of the cast-in-situ column.

In this study differences between column shoe connections from different companies are analyzed. One of the aims is to introduce main guidelines which can be important for a designer and other engineers, compare the performance of the column with column shoe connection with and without grout and with cast-in-situ column.

Below is presented a graphical scheme of this study.

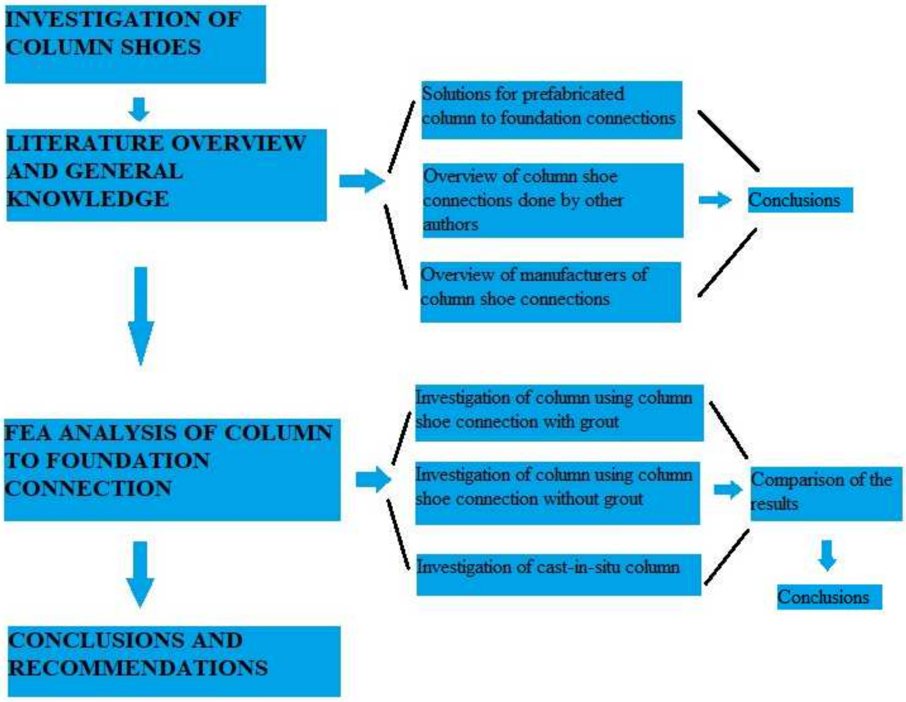


Figure 1. Graphical scheme of the study

1.1. Aim

The aims of this work is investigate column shoes connection, more specifically i.e.:

- Do analysis of solutions for prefabricated column to foundation connection;
- Determine what kind of solutions suggest different manufacturers of column shoe inserts;
- Do numerical analyses using chosen column shoe details, in another case, cast-in-situ column;
- Investigate the distribution of stresses in reinforcement and concrete.

1.2. Research objectives

The main objectives of this thesis include:

- Review the advantages and disadvantages of column shoe connections and compare it with other column-foundation connection types in prefabrication;
- Make an overview of other authors studies who were investigating such type of connection;
- Make an overview of companies which are manufacturing column shoe connections in the European market~~,~~ investigate the information provided by them, go through their guidelines;
- Provide links to design codes that are used for design calculation of column shoes and approves correct selection of it;
- Identify differences of column shoe connections manufactured by different manufacturers, try to identify key differences in the given guidelines.
- Model column shoe connection using FEA, compare results in different stages (erection stage, final stage) and compare its behaviour with cast-in-situ column;
- Make conclusions and proposals.

1.3. Limitations and scope

The boundaries of this paper include the following:

- Despite small introduction to other connection types in prefabrication, the main topic of this thesis is on column shoe connection intended for column-foundation connection;
- In this study are given design guidelines, on which choice of reinforcement in the practical part is based on;
- The practical part includes investigation of just Peikko column shoes using Finite Element Analysis;
- Way of load application in the practical part is not intended to make a comparison of experimental results made by other authors.

2. LITERATURE OVERVIEW AND GENERAL KNOWLEDGE

2.1. Solutions for a prefabricated column to foundation connections

Column shoe connection is not the only one, moment resisting column-foundation connection used. For a good structural design, it is necessary to understand where and in what cases this connection can be used. For a better understanding of what column shoe connection is good for, compared to others, it is necessary to give a brief overview of the advantages and disadvantages of other connections.

2.1.1. Baseplate connection

This system is analogical to base plate connection used in steel columns. A steel plate is attached to the bottom of the column base and it is bolted to the foundation (see Figure 2). A steel plate may directly or indirectly be joined to the longitudinal bars of the column. It can be reached by welding reinforcement bars to the plate. However, such connection can be uneconomical when loads or column cross-section (600x600 mm or more) is big. This can greatly enlarge the plate. The main advantage of such a connection that it provides instant stability to the column right after erection. It is challenging to get correct boundary conditions and calculate the size of baseplate which would be appropriate.

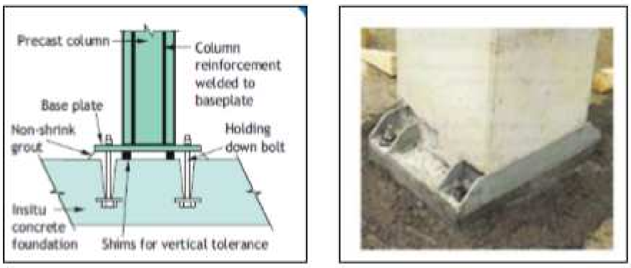


Figure 2. Baseplate connection (2)

Advantages:

- Immediate column stability
- Moment resisting connection
- Quick and easy erection on site
- Easy to fix and suitable for temporary structures

Disadvantages:

- High accuracy in setting bolt position
- Cost of bolts
- Requires analysis of load transfer into the baseplate, also boundary conditions are required for a footing connection.
- When a column dimensions are big, the required baseplate can become too large.

2.1.2. Pocket foundation

The column is inserted into the pocket delimited by the four walls of the foundation and footing slab at the bottom. After the column is centred, it is fixed by temporary bracing supports. Finally, a non-shrinkage mortar is poured to fill voids at the gap between the bottom of the column and footing, also between column and pocket walls.

Design of pocket foundation for a column is specified in Eurocode 2~~.~~ Pocket foundation with a keyed joint surface is described in section 10.9.6.2 and foundation with a smooth joint surface is described in section 10.9.6.3. The last one is encountered in practice more often. This type of connection works by transferring forces and moment from the bottom of the column by compressive forces F_1, F_2 and F_3 through concerete grout and corresponding friction forces (see Figure 3).

Commented [AK1]: Maybe concrete?

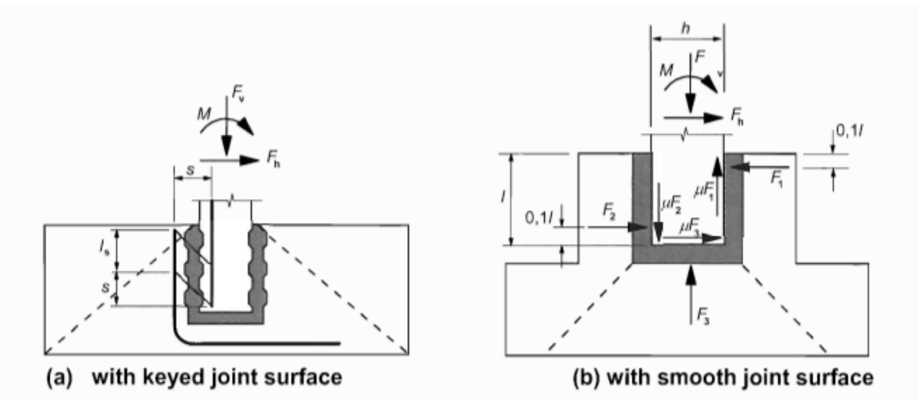


Figure 3. Pocket foundation: (a) with keyed joint surface (b) with smooth joint surface (3)

Following Eurocode 2 requirements, during the design of such connection shear resistance of column within the pocket and punching resistance of the footing slab for the force F_3 must be checked. At the top of the pocket walls, special reinforcement must be placed to transfer F_1 force along the vertical walls to the footing. Also, special attention should be paid to the anchorage of the main reinforcement in the column and pocket walls.

Taking account materials, it can increase rapidly amount of concrete and reinforcement depending on loads and column size, additional anchorage reinforcement shouldn't be forgotten. Also, frameworks should be used, however, it can be reused for the same shape of pocket foundations. So taking into account, economical evaluation, it is important to consider having as much as possible same shape of pockets. Further, a non-shrinkage concrete filling must be used which should be compacted well that no air gap would be left.

Advantages:

- Moment resisting foundation
- Quick and easy erection on site
- Law cost of the foundation and no need for additions to column

- Minimum tolerance is required

Disadvantages:

- Temporary propping is required
- When column dimensions are big, such a solution becomes uneconomical
- Deep foundation work
- Must be ensured that column is perfectly strict during erection and casting of void

2.1.3. Sleeve / projected bar connection

Depending on market requirements such a connection can be done in two ways. The first one, when rebars are jutted out from the foundation (see Figure 4). During prefabrication in the bottom of the column sleeves for starter bars are being formed. For this process smooth or grouted steel tubes are used. The column is erected in a construction site on starter bars. Non-shrinkage grout is poured to secure the connection. The sockets are anchored to the column by bars welded to them and spliced to the longitudinal reinforcement by lapping. In this case, other transversal bars can be attached to the socket to avoid their separation.

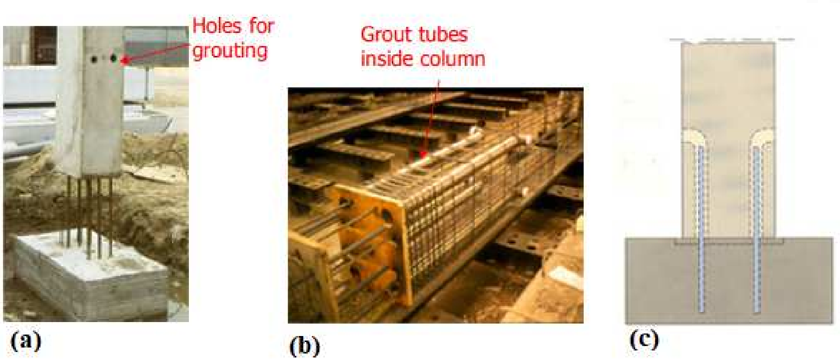


Figure 4. Sleeve connection: (a) installation process (b) production (c) scheme (4)

The second method is when sockets are formed in the foundation and longitudinal bars are projected from the column. Due to the size of the sleeve (80 to 100 mm in diameter) bars can enter into sleeves without deviating from their straight peripheral position in the column (5).

Such type of connection is easily manufactured and fixed, also thin joint is achieved. Further, the continuity of high tensile reinforcement is guaranteed. From an economical point of view, such type of connection is cheap, doesn't require a lot of grout. On the other hand, in comparison it is difficult effectively fill the voids, temporary support is required and high accuracy in projecting bars must be achieved.

For this type of connection some studies were conducted for seismic behaviour which concludes that it ensures ductility similar to cast in situ and pocket foundation connection, although just a slightly smaller dissipation capacity is observed. Test results are showing that the damage is localized at the thin grout layer at the base of the column (6). Respectively, one more advantage occurs – an easy repair of column-foundation connection after post-seismic actions.

Advantages:

- Law cost of foundation
- Moment resisting connection
- Quick and easy erection on site
- Good seismic response

Disadvantages:

- Accuracy in projecting rebar position
- Temporary propping is required
- An erection must be done carefully, avoiding bar bending
- Hard to fill the voids effectively

2.1.4. Column shoe connection

Column shoe system consists of anchor bolts which are anchored into foundation and steel shoes which are embedded into column base. Anchor bolts and steel shoes are mechanically connected by tightening nuts and washers.

In case that column-foundation connection plays vital role in ~~a~~ whole stability of the structure, it extra care must be taken in designing this connection. It is especially important for structures which are subjected to seismic or dynamic loads. Following studies, the seismic performance of the system has been evaluated numerically as well as experimentally which resulted in good seismic response of the examined specimens. (7). So, it plays a huge advantage in column-foundation connection.

References are mentioning that the tendency these days is towards the application of anchor bolts and column shoes. The function of such joints is more straightforward and closer to cast in situ approach. The main advantage of that solution from a technical point of view is easy rectification in a vertical direction, the joint is capable to carry out loads immediately after column erection and tightening of nuts and feet can be significantly smaller. (8). Also, a huge advantage of this system is the availability of design software and other design tools provided by manufactures (bending moment and normal forces diagrams). The biggest disadvantage of this system is to install anchorage bolts in the right location. However, this system is universal for any type of foundation and can be used for connection of column with slab, pile, footing and similar. Furthermore, it is still easier to reach the required vertical and horizontal position column plane than in other systems of connection.

Talking about materials, a number of shoes and anchor bolts depends on acting loads and cross-section size of the column. Also, there are no big requirements for reinforcement at the bottom of the column. Erection of column shoes into column requires casting boxes, however, their cost is low and they can be used several times. For final installation of the column, grouting of non-shrinkage mortal is required, so casting mould is needed for it. On the other hand, these casting moulds can be used repeatedly for other columns with the same cross-section.

In a study (7) economical comparison between a pocket and column shoe connection solutions is done. It is mentioned that the time of erection for column shoe system is 2 times shorter. It was concluded that the cost of column shoes represents 44% to 90% of the cost of pocket foundations. Further, another study (9) concluded that comparing these two systems under same assumed design conditions, column shoe systems results in about 20% less volume of concrete and about 30% less weight of steel.

Advantages:

- Immediate fixation
- A small quantity of needed grout
- Moment resisting connection, no bracing is needed
- Good performance during seismic and dynamic loading.
- Availability of design software and other design tools

Disadvantages:

- Relatively difficult to fix
- May be considered an expensive solution in some markets

2.2. Overview of column shoe connections done by other authors

It is necessary further to review the scientific works of other authors who have studied exactly the connection of column shoes. Below is provided a description of these works, laboratory test conditions, aims and results. This work continues to focus just on column shoe system.

2.2.1. Comparison between the numerical and experimental cyclic response of alternative column to foundation connections of reinforced concrete precast structures (10)

In this paper is examined the cyclic response of a column to foundation connection system based on the column shoes. This system is considered as an alternative solution to the pocket foundation. Authors perform three tests during which full-scale specimens are subjected to a constant axial load and to a quasi-static cyclic horizontal top displacement history at increasing drift levels. In this way, they are trying to compare hysteric behavior and the global collapse mechanism of the connection with an equivalent cast-in-situ solution. Additionally, 2D and 3D element models are developed which are characterized by non-linear material and geometry properties. The point of authors is to develop a global numerical model which would be able to predict the response to cyclic loading of similar connection typologies.

This type of mechanical connection based on the contact between steel shoes embedded into the column base and protruding anchor bolts anchored into the foundation. For vertical position controlling – nuts and washer are being attached to the anchor bolts. For complete system cement grout is being injected into the void below the column.

The stress transfer from the anchor bolts – steel shoes system (see Figure 5(c)) is based on a couple of longitudinal bars welded at the top of each shoe and on additional overlapped bars, which represent the steel reinforcement of the column (see Figure 5(b)). The seismic performances of the system have been evaluated.

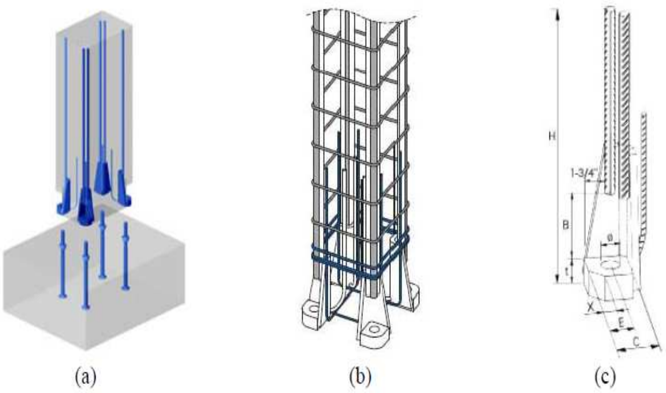


Figure 5. (a) Columns shoes and anchoring bolts in column connections; (b) details for additional steel reinforcement in square columns (welded bars and lap splice along with the height); (c) a single type of shoe (10)

Authors' objectives of this paper is to evaluate the real response of the welded connection between steel shoes and reinforcing bars; find a definition of the real global collapse mechanism, the displacement ductility and the dissipation capacity resources; find the capacity of the base section of the column to resist shear actions and the initial stiffness of the column compared to the stiffness of other precast structural typologies.

Researchers made a reference to reinforced concrete three-storey rectangular building with span length equal to 14 and 8 meters. Height of the building considered to be 12 meters which are subdivided into three interstory heights equal to 4 meters. So for specimens square columns with a cross-section of 400 x 400 mm were considered. It was made an assumption of high ductility class. During design, development was considered soil composed of medium dense sands. To resist the distribution of bending moment and shear action, respectively, without strong interaction of longitudinal bars, the additional longitudinal and transverse reinforcement is introduced along with the height of the column. Following the test set-up, it was necessary to permit anchor bolts to be subjected only to axial loads and resist the shear action. For this reason, was used four Ø 24mm steel pins with a length of 150mm (see Figure 6).

In case that aim of the research is a column and its connection, the foundation was over-designed to prevent the arising of crack pattern during the test and to keep it as fixed support.

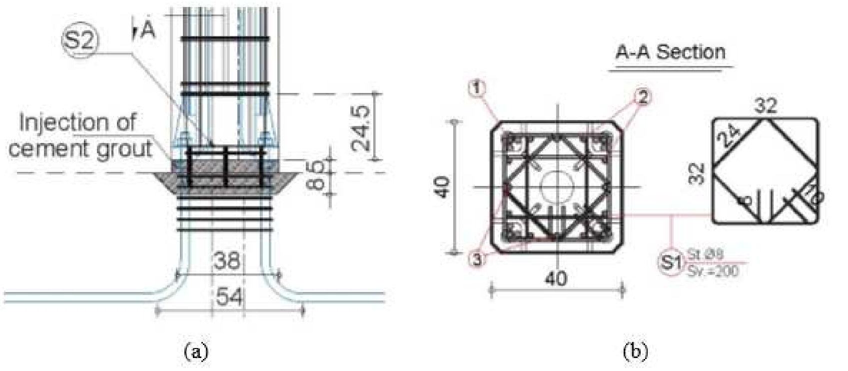


Figure 6. (a) Column to foundation details; (b) steel pins added to resist shear action (10)

During the test vertical loads of 200kN, 400kN and 600kN were imposed. It is expected that shear parts of the displacement profile will be neglected. In Figure 7 is shown instrumentation and test set-up. Relative displacement, deformation and curvature of a cross-section at different height levels are measured.

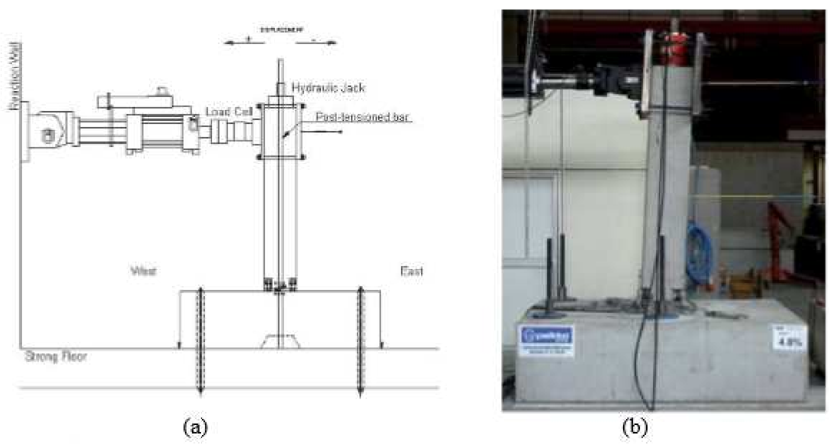


Figure 7. (a) Test set- up; (b) specimen at a drift of 4,8% (displacement 103.20 mm) (11)

Further Authors made FEA design. First of all, it was simulated cast-in-situ equivalent connection in 2D environment. purpose of it was to make a comparison with the system in terms of total damping capacity. Secondly, it was made 3D FEA design in order to capture the local behavior at the interface between grouting and concrete. The aim of this model is to increase the size of the shoes, using three specimens subjected to a series of horizontal displacement cycles, without making more tests. The comparison between test and numerical data was depicted.

After test on three specimens and FEA modeling these results were summarized:

- indecently of axial load value imposed on the top of the specimen, the collapse mechanism is being governed by behavior of anchorage bolts, without any significant damage of the column;
- only one component which reaches the plastic level (yielding) are anchorage bolts;
- the good seismic response of the examined specimens was noticed, obtained results very close to the typical behavior of cast-in-place frames for medium-low axial load values and close to the behavior of RC bridges axial load values;
- more rational design than the case of traditional reinforced concrete precast structures characterized by monolithic columns and pinned beams can be obtained since the column doesn't exceed elastic branch and it is not being significantly damaged.

2.2.2. Design Guidelines for Connection of Precast Structures under Seismic Actions (12)

A group of European associations of precast element producers and industrial partners with the assistance of a group of RTD providers prepared comprehensive research and development action for the project SAFECAST (Performance of Innovative Mechanical Connections in Precast Building Structures under Seismic Conditions). It was prepared guidelines which have a theoretical derivation supported by the experimental results of the testing campaigns performed, as well by the numerical simulations performed. General practical knowledge ~~on~~ of production practice and international literature on the subject have been also considered.

This document comprehensively examines bolted column to foundation connection in terms of seismicity.

Strength

Guidelines state that connection shall be verified for the ultimate moment $M_{Rd} = M_{Rd}(N)$ at the base of the column with the correspondent contemporary axial force N and the shear V. This verification can be made in two main directions independently. Depending which gives the smaller force – the steel area of the lower bars or the fasteners, that component should be assumed for calculation of the ultimate moment M_{Rd}.

The lap length of the column shoe bars with the longitudinal reinforcement bars of the column is calculated following EC2 Clause 8.7.3 applying γ_R factor:

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

Where:

$l_{b,rqd} = \left(\frac{\emptyset}{4}\right)\left(\frac{\sigma_{sd}}{f_{bd}}\right) -$ the basic required anchorage length;

$\sigma_{sd} -$ the design stress of the bar at the position from where the anchorage is measured from.;

$f_{bd} -$ the design value of the ultimate bond stress, for ribbed bars, may be taken as:

$f_{bd} = \eta_1 \eta_2 f_{ctd}$;

f_{ctd} – the design value of concrete tensile strength;

η_1 – a coefficient related to the quality of the bond condition and the position of the bar during concreting. For column shoes, good bond conditions are obtained, so this coefficient is $\eta_1 = 1{,}0$;

η_2 – coefficient related to the bar diameter, see EC2 Clause 8.4.2 for setting this value;

$\alpha_1, \alpha_2, \alpha_3, \alpha_5$ – values of the coefficients can be found in EC2 Table 8.2;

α_6 – value of the coefficient can be found in EC2 Table 8.3.

For a good performance during seismic actions, it is expected brittle failure mode, so a good ductile behavior should be reached in column shoes. For this reason, shoes should be over-dimensioned by γ_R with respect to the connected elements for which ductile behavior is required. The producer of the column shoe system should take care of such dimension.

Behavior models

Figure 8 shows the detail of the resisting mechanism of the foot section of the column subjected to combined bending moment $\gamma_R M_{Rd}$ and axial action N and to the shear $\gamma_R V$. The assumption is made that at such level of action the tensioned lower steel bars or the steel fasteners (depends which is the weaker) are at their maximum ultimate capacity F_u. The anchorage verification shall be referred to a correspondent pull-out force.

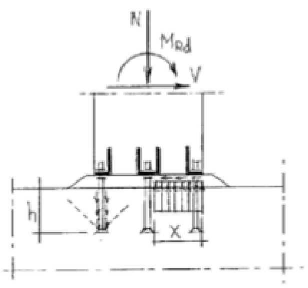


Figure 8. The resisting mechanism of the foot section of the column subjected to the combined bending moment, axial force and to the shear (12)

Failure modes

Three failure modes are distinguished:

I. failure of a non-ductile fastener subjected to the tensile force coming from the upper reinforcement;
II. pull-out of the head-fastener subjected to the maximum upper force F_u with concrete cone-failure;
III. sliding shear failure at the foot section in the design situation corresponding to $\gamma_R M_{Rd}$, N and $\gamma_R V$.

Calculation formulae

For fasteners, with reference to the symbols described in Figure 8, the following verifications shall be performed. The below calculations shall be adapted to the possible different solutions of other connectors systems.

I. Fastener failure (for non-ductile fasteners):

$F_{R.min} \geq \gamma_R A_s f_{ym}$.

Where:

$F_{R.min} -$ minimum steel ultimate capacity of the fastener declared by the producer;

γ_R – overstrength factor; the values $\gamma_R = 1{,}2$ for DCM and $\gamma_R = 1{,}35$ for DCH are recommended by EC8.

$A_s -$ sectional area of the corresponding upper reinforcement;

$f_{ym} = 1{,}08 f_{yk}$ - mean yielding stress of the steel bars (f_{yk} their characteristic yielding stress).

II. Pull-out of the head-fastener:

$R_d \geq \gamma_R F_u f_{ym}$;

$F_u = \min\{A_s f_{ym}, F_{R.max}\}$.

Where:

$F_{R.max} = 1{,}2 F_{R.min}$ – except differently declared by the producer;

$R_k = k\sqrt{(f_{ck.cube} h^3)}$;

$R_d = R_k/\gamma_c$;

$f_{ck.cube} -$ characteristic compressive cubic strength of concrete;

$h -$ effective length of the fastener;

$\gamma_c = 1{,}5 -$ value recommended by EC2 (see also TS2;

$k -$ may be taken from the relevant ETS (for current products the safe side value k=7,0 may be assumed);

$F_{R.min}, A_s, f_{ym} -$ defined before.

III. Sliding shear failure at the foot section:
$V_{Rd} \geq V$, with $V_{Rd} = V_{dd} + V_{fd}$;
$V = V(\gamma_R M_{Rd})$ – is the shear corresponding to $\gamma_R M_{Rd}$.

Where:

$V_{dd} = 1{,}3 A_d \sqrt{f_{cd} f_{yd}} -$ dowel resistance of the resisting fasteners;

$V_{fd} = 0{,}5 b x f'_{cd} -$ sliding resistance of the compressed mortar or concrete;

$f'_{cd} \approx 0{,}5 f_{cd}$;

$b -$ width of the section;

$x -$ depth of its compressed part;

$f_{cd} -$ design compressive strength of the mortar or of the column concrete if lower;

$A_d -$ area of the fasteners not yielded by the contemporary flexure;

$f'_{cd} -$ steel design yielding stress of the fasteners not yielded by the contemporary flexure.

Ductility

Ductility is a measure of a material's ability to undergo significant plastic deformation without losing strength or breaking, which may be expressed as percent elongation or percent area reduction from a tensile test. For the investigation of this and below listed parameter Authors performed a test on three different arrangements of the connection:

I. Weak fasteners of ductile steel coupled with strong bars in the column.

During this test failure occurred due to rapture of a fastener, in the column didn't appear any relevant cracking. The plastic deformation remained concentrated within the joint lap with an almost rigid rocking of the column. Obtained displacement ductility factor is $\mu_\delta \approx 2{,}2$.

II. Weak bars under the lap zone moved in an upper position.

During this arrangement, failure occurred due to the rupture of ~~a~~ defective welding of a socket just after the yielding limit of the bars. This points out the importance of ~~a~~ correct coupling technology. Defect during it can result in a non-ductile behavior which can become a decisive factor in the collapse of a structure during dynamic and seismic actions. Obtained displacement ductility factor is $\mu_\delta \approx 1{,}3$.

III. Inverted sockets position welded to the fasteners and bolted to the bars.

During the third arrangement, failure occurred after the formation of a plastic hinge at the base of the column with large cyclic deformations and has been produced by the localized rupture of the bars at their bottom end close to the coupling device. Obtained displacement ductility factor is $\mu_\delta \approx 3{,}0$.

Dissipation

Under seismic action aspiration for designers is to reach high dissipation capacity of the structure. Below is listed results of this parameter during all three assumptions.

I. Assembly foundation-connection-column showed a low-dissipation capacity. This behavior can be explained due to limited plastic length of the fasteners.
II. During second assumption assembly showed non-dissipative behavior. It one more time showed how is important to avoid defects during manufacturing on column-shoe details.
III. Dissipation during the third arrangement with inverted sockets was measured slightly bigger than during the first arrangement, but it is still in the limits of low-dissipation. This behavior is attributed to the column, however, it is slightly affected by the alternate opening of the base joint.

Deformation

During the cyclic test, the ultimate drifts were reached. Below are listed results of it during all three arrangements:

I. 4,4% mainly due to the plastic rotation concentration in the joint lap.
II. 2,0% with no evident signs of plastic deformation was visible on the specimen.
III. 4,5% due to deformation of the column and partly due to the opening of the base joint interface.

Decay

During the cyclic test, for all the three arrangements, at any displacement level before failure no relevant strength decay show up after the three cycles.

Damage

At serviceability limit state, taken as 1% of drift, during all three arrangements testing an elastic behaviour with no sensible residual deformations has been registered.

I. During arrangement with weak fasteners at 2,0 % of drift, the yielding limit set out, at 3,0% of drift a residual deformation was measured, at 4,4% of drift mainly due to the plastic rotation concentration in the joint lap.
II. 2,0% with no evident signs of plastic deformation was visible on the specimen.
III. 4,5% due to deformation of the column and partly due to the opening of the base joint interface.

2.2.3. ETA tests and design of HPKM Column Shoe Connections (13)

CE marking is mandatory in for construction products intended for sale within European Union, the European Free Trade Association and Turkey. This marking indicates that the European Union legislation applicable to a product, regardless of the place of manufacture. ETA approval is the only one solution for Column Shoes to get this marking and proves compliance with the basic requirements such as mechanical resistance, stability and safety in use. To prepare available practice for verification of such connection, it was necessary to get ETA and CUAP approval for the rules of verification for mechanical behavior and fire resistance by full-scale tests. Advantage of it is in simplification of designers' work due to the same design rules and methods used in the EU.

Further are presented results of ETA tests made for Peikko precast column connection with HPKM column shoes and HPM anchor bolts. It was tested 24 precast column connections with different types of column shoes and different dimensions of the column. The objective of the tests was the performance of column shoe system in terms of bending resistance, bending stiffness resistance, shear resistance and fire resistance. A common target of all tests to obtain reliable and real behavioral information about Peikko column connections and their main components such as column shoes, anchor bolts and grout. The aim of the test is confirmation that connection behaves rigidly and ductile failure mode appears in all conditions. Another purpose of it is to measure the bending or shear resistance of the connection and compare the bending stiffness of the column inside and outside the column shoe zone. For a column specimens concrete is used C30/37, for the connection zone it was used self-compacting, rapidly hardening, low-shrink grout which grade is C50/60. Before concreting the test specimens, the real strength of steel material of column shoes and anchor bolts were verified by tension test.

Bending resistance tests

In Figure 9 is shown an illustration of full-scale bending resistance test arrangement.

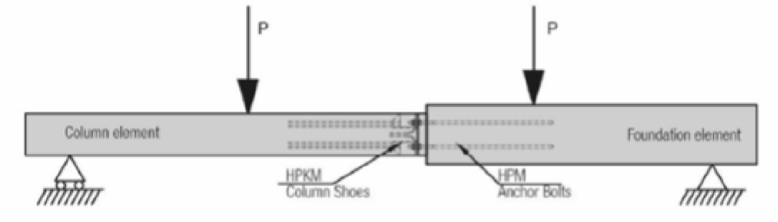


Figure 9. Arrangement of full-scale bending resistance test (13)

It was calculated theoretical bending resistance M_t based on the measured material properties and nominal geometry. During test was received experimental bending resistance M_e. Author presents ratio M_e/M_t which varies within 1.23-1.30. So, this means that the applied design method is safe for axial and bending resistance of column shoes.

Bending stiffness resistance tests

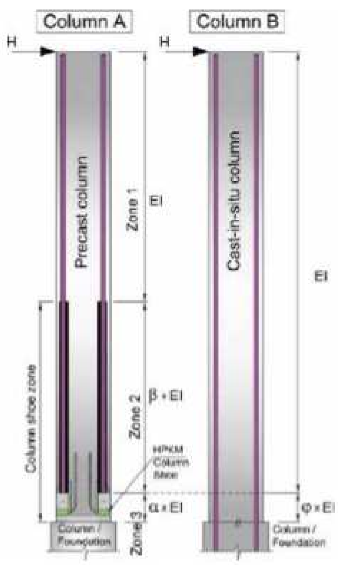


Figure 10. Different stiffness zones of cantilever column (13)

Figure 10 shows different stiffness zones in the precast column with column shoes and cast-in-situ column. The idea was to find the stiffness in different zones and make a comparison between cast-in-situ and precast columns. The design following EC2 assumes a purely theoretical value for buckling length of the column. The aim was to apply these rules, developed for a slender cast-in-situ column with continuous reinforcement, to a precast column with column shoe system.

For a precast column in Zone 1 column shoes have no effect on stiffness and it assumed to be (EI), In Zone 2 flexural stiffness is increasing due to overlapping of longitudinal reinforcement with anchor bars and assumed to be equal to $\beta(EI)$, where $\beta > 1$. In Zone 3 the flexural stiffness is the smallest due to reduced effective concrete section and eccentric tension on the column shoes. The assumption is made that it is equal to $\alpha(EI)$, where $\alpha < 1$.

For cast-in-situ column in Zone 3 flexural stiffness is $\varphi(EI)$, where $\varphi < 1$. Continuous and constant reinforcement is chosen because in EC2 design rules were developed for this case. The test is also focused on finding the real stiffness in the connection zone of cast-in-situ column.

For this test cantilever column was chosen. In both cases, reinforcement is the same one for both types of columns, just in Column A are used column shoe connections (HPKM 16 and HPKM 39).

In case that bending stiffness of a section typically increasing with increasing axial compression force, the stiffness is measured without axial force. For obtaining a conservative evaluation method, the deflection of the cantilever column is compared in a load case without axial force (see Figure 11). As shown in Figure 11, horizontal transducers, placed on the column, measured the differential displacement for determination of the axial strain on the top and at the bottom of the bended specimen. Based on the design principles, 90% of the nominal yield resistance was exploited.

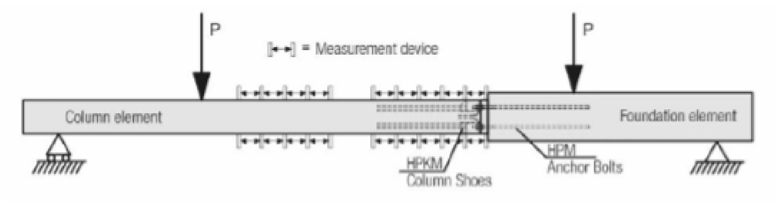


Figure 11.Arrangement of full-scale bending stiffness resistance test (13)

In figure 12 are presented results of relative stiffness of subzones. In short, relative stiffness of precast column A is lower in zone 3 than that of the cast-in-situ column. However, the difference is minor and in case that relative stiffness is significantly higher in the upper zone in comparison with column B, it should compensate for weaker stiffness in zone 3. All in all, a column with column shoes is stiffer than a cast-in-situ column. It proves that precast column with such a connection system behaves in the same way as a corresponding monolithic reinforced concrete column.

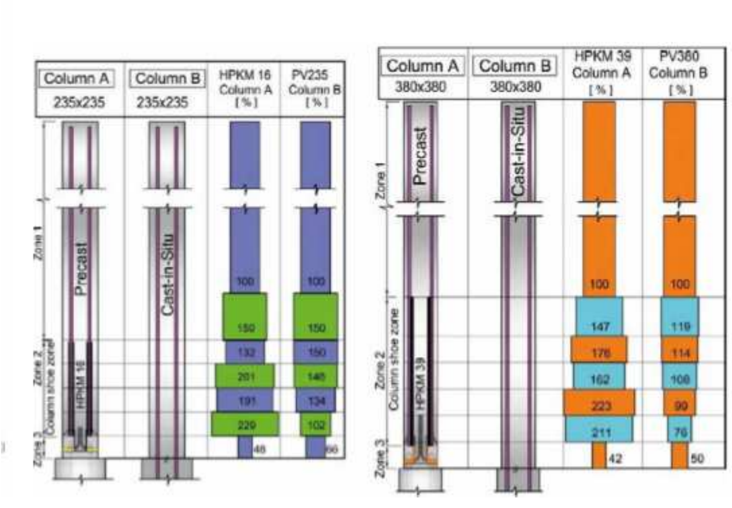


Figure 12. Relative bending stiffness of subzones in column A and B (13)

Shear tests

For a shear test assumption is made that the maximum shear force is caused by a moving vehicle which collides with a single column after hardening of the grout in the joint. Top of the column is laterally fixed and flexural rigidity of the top is between completely rigid and hinged. It is considered that the beams and floors carried by the column effectively redistribute the reaction forces and moments at the top of the column. The forces at the bottom of the column are slightly influenced by the bending stiffness at the top. An axial force is ignored in the test arrangement because it is assumed that in this case, it would have little or no negative effect. The shear test was carried out just for the final stage of connection (with a grouted joint).

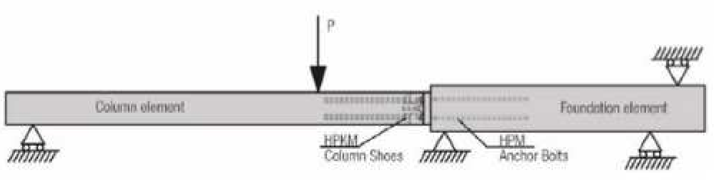


Figure 13.Arrangement of full-scale shear test (13)

Following EN 1993-1-8, Clause 6.2.2 it was calculated sum of theoretical shear resistance of two active column shoes V_t. Following CUAP requirements experimental shear resistance V_e should be bigger or equal to the theoretical one by 15%. It was obtained experimental shear resistance 54% bigger than theoretical one during the test for HPKM 16 column shoes and by 18% bigger for HPKM 39 column shoes.

Fire resistance tests

Fire resistance test in a full-scale was made for HPKM 16, HPKM 24 and HPKM 39 column shoes and corresponding anchor bolts. For HPKM 20 and HPKM 30 results were evaluated from a numerical model using FEA. The test was performed without mechanical loading and without any protective concrete cover in the outer edges of the column shoes. During the experimental and FEA testing results showed that critical temperature is being reached in anchor bolts and not the column shoe details. For this reason, in Table 1 are shown results of the measured temperature, which can be used in fire design, of anchor bolts. Due to higher heat absorption capacity, heat flow is slower in bigger columns. It causes lower design temperature which would be used in fire safety calculations.

Table 1. The average temperature t (°$\boldsymbol{C}$) in critical section of anchor bolts

Time (min)	HPKM 16	HPKM 20	HPKM 24	HPKM 30	HPKM 39
60	500	500	450	430	390
90	670	610	630	630	570
120	800	780	740	730	700

2.3. Overview of manufacturers of column shoe connections

There are many suppliers in the European market that produce column shoe connection. It is important to know at least the key differences between products, to understand when it is possible to choose which supplier. Below is the basic information provided by each of the known manufacturers about their product.

2.3.1. ANSTAR column shoes

ANSTAR is a Finnish company established in 1981. It offers concrete structure connections and composite structures manufactured in Finland. It is providing two types of column shoes:

- AHK shoes are used in office, commercial and public buildings for connecting light concrete frames to the foundation and for column extension. The shoes are also suitable for connecting concrete columns to foundations of industrial buildings. The AHK-K is a special application for round column connection.
- APK-C shoes are used for connecting columns to foundations in industrial concrete element frames. The shoes are also suitable for connecting heavy-duty concrete frames to foundation in office, commercial and public buildings. The APKK-C shoe is a special application for middle shoe connections of rectangular columns.

Products of this manufacturer have CE marking, so no additional national approval is required.

2.3.1.1. Design guidelines

AHK column shoes were tested and dimensioned to withstand demanding construction conditions. Test data is not shared, but ANSTAR provides very detailed User manual (14) following which is easy to get understanding about the system of column-foundation connection. Solutions suggested by this supplier stands out for the abundance of product types, the possibility to use column shoes for circular columns and in a non-symmetrical arrangement depending on the connection's force requirements (see Figure 14)

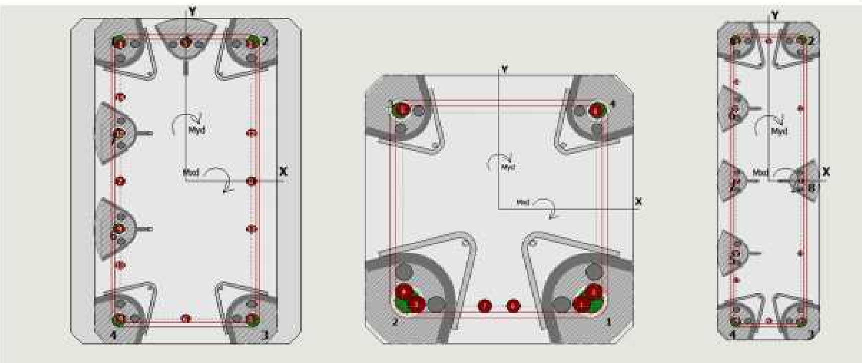


Figure 14. AHK and AHK-K shoes in asymmetrical rectangular column connections (14)

The company also manufactures parts fitted to round columns which successfully can be used as a middle shoe. There is an option to use just two column shoe connection, but it only transfers axial force, and a connection with 4 and more shoes transfers axial, shear force and bending moment. More detailed design aspects can be explored thanks to the comprehensive guide. For example, shoes are manufactured in execution class EXC2, but by special order, it can be manufactured in execution class EXC3. According to the standard ISO 1090 higher level should be used for building higher than 15 floors, pedestrian, bicycle, car, train bridges. So, an existing possibility is very important in the design of structures.

Based on EC1, EC2 and EC2 requirements, company gives axial force resistance which values are presented in Table 2. Design values were obtained using concrete C30/37 during final stage. So nominal strength resistance should be recalculated using ACOLUMN software for an erection stage. In Annex 1 are given recommendations of column reinforcement.

Table 2. Design values of nominal strength resistance of ANSTAR column shoes

Column shoes type	Nominal design tension resistance N_{Rd} (kN) (concrete C30/37)
AHK 16	61,7
AHK 20	96,3
AHK 24	138,7
AHK 30	220,2
AHK 36	315,9
AHK 39	383,5
AHK 45	493.4
APK-C 24	161,6
APK-C 30	299,2
APK-C 36	435,7
APK-C 39	520,5
APK-C 45	696,5
APK-C 52	937,6
APK-C 60	1260

2.3.1.2. AColumn software

ANSTAR has developed software AColumn which applications are used to specify the resistance and utilization rates of the concrete column connections with ANSTAR products as column shoe/anchor bolts. The software is used for designing moment stiff beam-to-column connections, shear wall coupler and foundation connections, element column coupler and foundation connections, rectangular and round columns. The software can be used to dimension the connection using basic Eurocode and Finnish, Swedish or German National Annex.

The software gives to enter initial data which is consisting of the following information:

- Connection type and materials.
- Dimensions of the column.
- Dimensions of the lower structures.
- Data for fire design.
- Supplementary reinforcement.

- Loading cases in erection, final and accidental stages.
- Column shoe and anchor bolt type.

After calculation it shows 3 types of results:

- at erection stage (resistance graph and loading combination points; axial and shear force interaction resistance);
- at the final stage for shoes;
- at the final stage for bolts.

2.3.2. PEIKKO column shoes

PEIKKO is a company founded in 1965 in Finland. It manufactures concrete connections and composite beams for both precast and cast-in-situ solutions. It suggests two types of column shoes:

- HPKM – for moderate load, has CE marking and widely approved not just in Europe, but also have National verification in UAE, Russia, Turkey, as well as, can be designed according to ACI standards.
- PEC – moment-resisting connection under heavy loading conditions. It doesn't have CE marking, however, it is approved product in Czech Republic, Finland, Germany, Hungary, Poland, Russia, Slovakia, Turkey and Ukraine.

Even if the company has CE marking through ETA assessments, the company takes time for more advanced research than it is required by the standards analysing their products for acting static, dynamic, seismic and fatigue actions.

2.3.2.1. Design guidelines

Column shoe resistance is equal to the resistance of corresponding anchorage bolts (see Table 3). These design values are pre-designed to bear a static load, in case of dynamic, seismic or fatigue loads, individual calculations have to be made. During the erection stage, it should resist loads from self-weight and wind loads. These forces are carried by anchor bolts. The final condition is fully verified by ETA requirements and fulfill the requirements of cast-in-situ columns.

PEIKKO gives main information in Technical manual (15) in which describes loading, environment, positioning conditions which must meet column shoes. Design is made following EN 1992-1-1, EN 1993-1-1, EN 1993-1-8 and ETA approvals.

The design value of the shear force for a single column shoe on the active side is calculated from:

$V'_{Ed} = \frac{V_{Ed} - \mu N_{Ed}}{n}$;

Where:

$V_{Ed} -$ total shear force of column connection;

$N_{Ed} -$ axal force of column connection (if column loaded just by tensile axial force, so $\mu N_{Ed} = 0$)

$\mu -$ friction coefficient between the base plate and grout = 0,20 (according to EN 1993-1-8, Chapter 6.2.2)

n – the number of the individual active column shoes resisting shear force.

It shall meet the requirements:

$V_{Ed} \leq V_{Rd}$;

Where:

V_{Rd} – shear resistance of a column shoe calculated according to EN 1993-1-8, Chapter 6.2.2 (these formulas see below in Chapter 2.3.4.1).

Annex 2 contains the manufacturer's column reinforcement recommendations.

Table 3 Design values of nominal strength resistance of PEIKKO column shoes

Column shoes type	Nominal design tension resistance N_{Rd} (kN) (concrete C30/37)	Design resistance V_{Rd} (kN)
HPKM 16	62,0	20,0
HPKM 20	96,0	31,0
HPKM 24	139,0	45,0
HPKM 30	220,0	71,0
HPKM 39	383,0	125,0
PEC 30	299,0	89,0
PEC 36	436,0	130,0
PEC 39	521,0	155,0
PEC 45	697,0	207,0
PEC 52	938,0	219,0

2.3.2.2. PEIKKO Designer software

For the selection of column shoe company has developed a software that simplifies the assessment of resistance, properties of the column and grout, position and arrangement of the column shoes in the column and design values of actions. The column is being verified for the erection stage, the final stage, fire situation design situations, as well as, for environmental exposure conditions.

The typical selection procedure is done by entering input parameters:

- Materials for column, grouting and structure under the column
- Geometries of the column and foundation
- Design values of the actions during erection, final and fire stage
- Type of column shoes and anchor bolts and its arrangement
- Column reinforcement (optional)

As output software can present a report which consists of:

- N-M interaction diagram (axial force-bending moment diagram) of joint in final and fire stage
- N-M interaction diagram of a reinforced column
- Calculation results for column connection in erection and final stage
- Supplementary reinforcement details
- Summary of products in the project

This report can be used in a design project as verification of product selection.

2.3.3. SEMKO column shoes

SEMKO is a Finish company established in 1975. It mainly manufactures concrete and fastening components, cement handling equipment and lightweight steel structures. It offers one type of column shoes OPK which can be used with two types of foundation bolts SUJ – one bar anchorage system and SELP – three bars anchorage system. The producer provides 8 types of shoes which are certified in Finland, Sweden and Russia.

2.3.3.1. Design guidelines

The company provides the direction of use this detail following Eurocode standards. It listed that strength resistance of shoes has been calculated for static load following SFS-EN 1992-1-1, SFS-EN 1993-1-1 and SFS-EN 1993-1-8. However, it states that hawing dynamic load calculations should be done separately.

Following tables (see Table 4) which is concluded following information given by manufactural and Figure 15 is possible to easily choose hook reinforcement and have sense up to which loads is are used each shoe type. It is also provided step by step installation process, quality control requirements, installation monitoring guidelines and instruction on how to use design templates which example is shown in Figure 16.

Table 4. Design values of nominal strength resistance and transverse reinforcement of SEMKO column shoes

Column shoes type	H_S (mm)	A_h (quantity-diameter)	Nominal design tension resistance N_{Rd} (kN) (concrete C35/45)
OPK 16	600	2-T8	63
OPK 20	700	3-T8	97
OPK 24	1100	4-T8	175
OPK 30	1350	4-T8	265
OPK 36	2150	8-T8	471
OPK 39	2400	8-T8	563
OPK 45	2550	10-T8	753
OPK 52	3000	14-T8	1013

Nominal design strength values which are presented in Table 4 were calculated for concrete C35/45. For concrete weaker than C35/45, the strength reduction factor can be calculated according to the tensile strengths of the concrete. For example, for column concrete C25/30 reduction factor is calculated as follows:

$$\frac{f_{ctd}(C25/30)}{f_{ctd}(C35/45)} = \frac{1{,}33MPa}{1{,}66MPa} = 0.8.$$

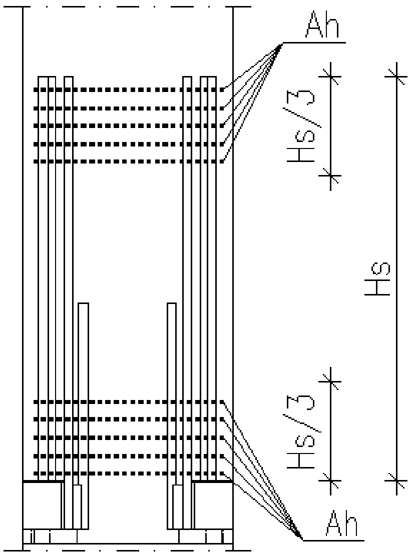


Figure 15.Position of hook bars for OPK column shoes (16)

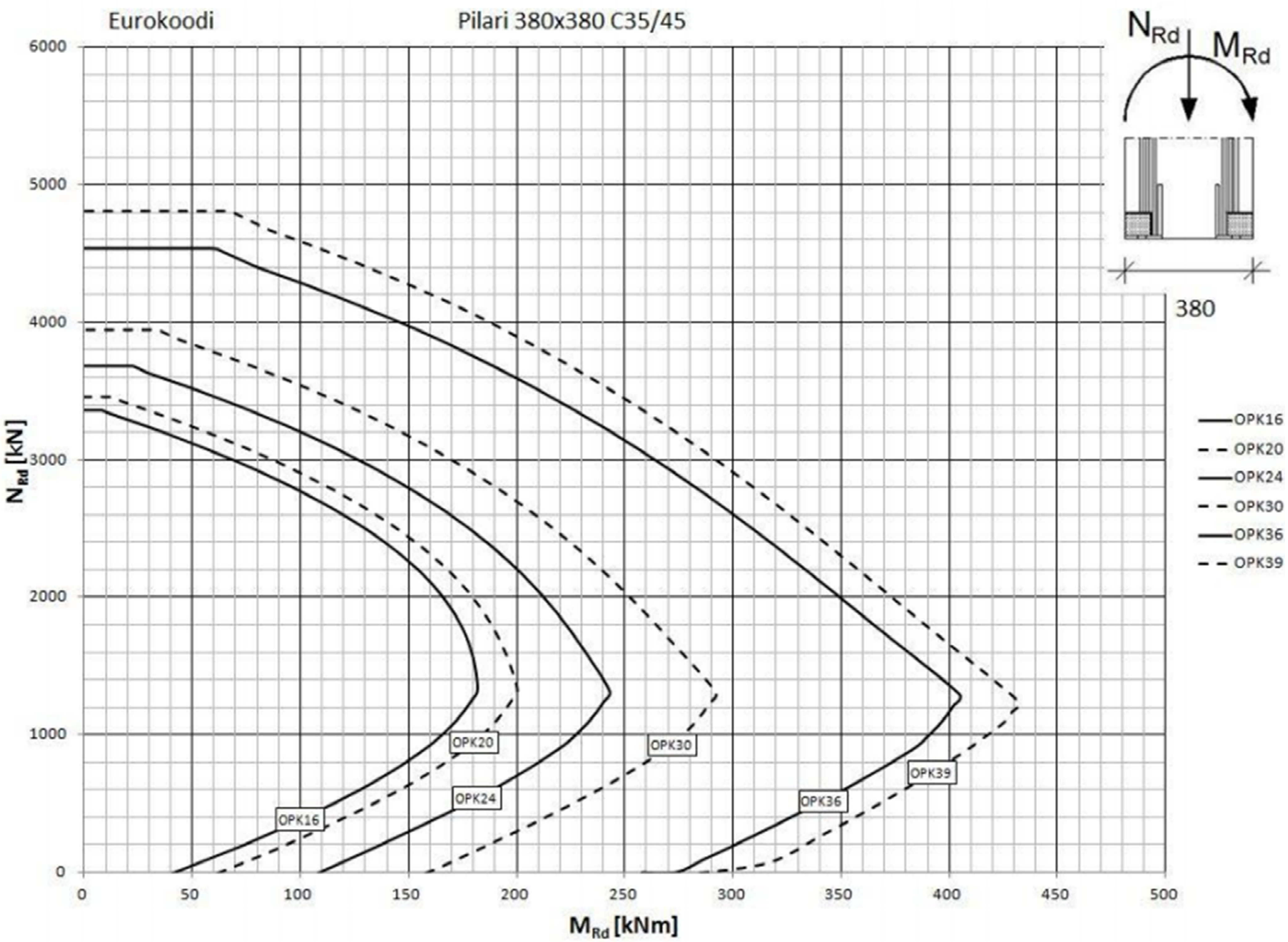


Figure 16. Example of design template provided for choice of OPK column shoes (16)

2.3.4. RSTEEL column shoes

RSTEEL is a Finish brand, manufacturing accessories and concrete connection for precast elements. They provide two types of column shoes – for moderate and high loads. The company states that it is designed according to Eurocodes and produced following EN 1090. However, this connection system has approval just in Finnish, Swedish, Norwegian and Russian markets.

2.3.4.1. Design guidelines

As a design tool company provides Technical Manual (17) in which presents main information about their product. Tension capacity of column shoe is calculated in accordance with EC2 and EC3. Minimal concrete grade of the column should be C30/37. Column shoes are being used with RPP anchor bolts for moderate loads and RPP-E anchor bolts for high loads. However, it is mentioned that base bolts can be used of other suppliers, that are of equivalent strength and that are is similarly approved by BY (Finish Concrete Association) product manual or has ETA approval.

Table 5. Design values of nominal strength resistance of RSTEEL column shoes

Column shoes type	Nominal design tension resistance N_{Rd} (kN) (concrete C30/37)
RPK-N2 M16	62,2
RPK-N2 M20	97,0
RPK-N2 M24	139,4
RPK-N2 M30	222,2
RPK-N2 M39	386,5
RPK-E2 M30	299,2
RPK-E2 M36	435,7
RPK-E2 M39	520,5
RPK-E2 M45	696,5
RPK-E2 M52	937,6

Design manual submits lap factors for anchorage length $l_0 = \alpha_1\alpha_2\alpha_3\alpha_5\alpha_6 l_{b,rqd} \geq l_{0.min}$ which are coming from EC2 and equals to: $\alpha_6 = 1{,}5, \alpha_2 = 0{,}7$, others $\alpha_1 \dots \alpha_5 = 1{,}0$.

The shear strength of the connection can be calculated in accordance with EN 1993-1-8 clause 6.2.2. The additional contribution of friction to shear strength can be taken into account; a friction coefficient of 0.2 can be adopted for a sand-cement grout, without additional tests.

Shear resistance:

$F_{v.Rd} = F_{f,Rd} + nF_{vb.rd}$,

Where:

n – number of bolts on the compression side of the column;

$F_{t.Rd} = C_{t.d}N_{c.Ed} -$ resistance due to friction;

$C_{t.d} = 0{,}2$;

$N_{c.Ed}$ – the axial compression applied by the column;

$V_{vb.rd} = min\{F_{1.vb.Rd}; F_{2.vb.Rd}\} -$ bolt shear resistance;

$F_{1.vb.Rd} = (k_1 a_b f_{base.u} d_b t_{base})/\gamma_{M2}$;

$F_{2.vb.Rd} = (\alpha_b f_{bolt.u} A_{bolt})/\gamma_{M2}$;

$\alpha_b = 0{,}44 - 0{,}003 f_{bolt.y}$;

k_1 and $a_b -$ values should be taken from EN 1993-1-8, Table 2.1;

$f_{base.u} -$ is the base plate ultimate tensile strength;

$f_{bolt.u} -$ is the bolt ultimate tensile strength;

$A_{bolt} -$ is the net tensile area of the bolt;

$\gamma_{M2} -$ is the material partial factor for resistance, EN 1993-1-8, Table 2.1.

Design criteria:

$V_{Ed} \leq V_{Rd}$;

$N_{Ed} \leq N_{Rd}$;

$\frac{N'_{Ed}}{1{,}4N_{Rd}} + \frac{V'_{Ed}}{1{,}4V_{Rd}} \leq N_{Rd}$;

Where N'_{Ed} and V'_{Ed} are applied coincident axial force and shear force.

The technical manual also provides calculation sequence, which is based on EN 1993-1-8, for anchor bolts during installation and prior to grouting of the base. It also gives required nominal concrete cover which ~~is~~ coincides with EN 1992-1-1 requirements. Adapting this nominal cover fire resistance is R90-120, without any additional protective layer it may be taken as R60.

In ~~a~~ Appendix 3 is submitted instruction for column reinforcement.

2.3.5. HALFEN column shoes

HALFEN, with the headquarters located in Germany, is one of the leading international companies in the fields of anchoring, reinforcement, framing, facade fixing technologies and transport anchor, tension rod systems. It offers two types of column shoes: HCC and HCC-M for higher stressed construction members. Company has CE marking for column shoe system.

2.3.5.1. Design guidelines

Following the test report of column shoes (18), the system was tested just for static load in the installation state (without grout). So, calculated design loads can be assumed in installation and final stage. The transfer (shear) load must be tested and evaluated separately. Also, it is mentioned that the level of fire protection must be confirmed in each case and fire resistance must be verified in accordance with the current regulations. As in previous cases, a minimal required concrete class is C30/37. HALFEN column shoes must be used together with HAB type anchor bolts and together they can form a rigid or hinged connection. In Table 6 presented results of the load-bearing capacity of the column shoes for tension and compression which are calculated following EN 1993-1-1. In Appendix 4 can be found recommendations for reinforcement of the column which based on DIN 1045-1:2001-07 normative.

Table 6. Design values of nominal strength resistance of HALFEN column shoes

Column shoes type	Nominal design tension/compression resistance N_{Rd} (kN) (concrete C30/37)
HCC 16	61,7
HCC 20	96,3
HCC 24	138,7
HCC 30	220,4
HCC 39	383,4
HCC M30	299,2
HCC M36	436,0
HCC M39	520,6
HCC M45	696,6
HCC M52	937,3

The company provides with Technical information manual (19) in which can be found main information about column shoes, application and installation instruction. However, the main tool given for designers is HCC Column Shoe software.

2.3.5.2. HCC Column Shoe software

For the section and dimensioning of the required column shoes and anchor bolts, the company developed software. It allows calculating loads of shoes during erection and final stage. Also, according to official approval, it calculates the anchoring length.

Input data:

- Position, dimensions of the structural member, concrete strength of column and foundation, edge distance;
- Automatic calculations of wind, dead loads (can be entered as an input value, too)
- Loads during erection and final stage
- Choice of shoes and anchor bolts type and number, concrete cover set up

Output data:

- The individual status of all values

- The position of the neutral line
- Part list

2.3.6. PFEIFER column shoes

PFEIFER is a German company, with headquarter in Memmingen which business started over 430 years ago. Companies' main specialization is ropes and cables production, however, it has a building system division which produces connecting and lifting systems. One of the suggested products is PCC column shoe connection which has CE marking. Company is also suggesting PGM/PSF column shoes for high loads, however, detail information about them is provided on request.

2.3.6.1. Design guidelines

In EC declaration of conformity in CE marking is listed that for the design of shoes following standards were applied: DIN-EN 1990, DIN-EN 1992-1-1, DIN-EN 1993-1-1 and DIN-EN 1993-1-8. Following these standards static test was done (20) which results are presented in Table 7. The derivation of shear forces was not part of this type of test. It must be proven separately in each case. Also, it has been checked just for stability, but not for other regulations or other official requirements. The type of the test of the PFEIFER column shoe PCC deals with the verifications in the limit states of the load-bearing capacity (stability) in accordance with DIN EN 1991-1. The required verifications with regard to serviceability in accordance with DIN EN 1992-1 (crack widths, rod spacing, deflections etc.) was not the subject of this type of test and must be carried out by the designer if necessary. So, concluding given information, additional evidence in individual cases must be proven:

- of the load-bearing capacity for the transfer of shear forces
- of the load-bearing capacity for the interaction of the actions with each other.

Column shoes are used together with anchor bolts PGS. Required stirrup reinforcement, anchorage length, lap length, minimum column cross-section dimensions, nominal concrete cover are provided in PFEIFER guidelines for column shoes and can be found out in Appendix 5. The guideline also suggests a few possible solutions when shear force exist (see Figure 17).

Table 7. Design values of nominal strength resistance of PFEIFER column shoes

Column shoes type	Nominal design tension resistance N_{Rd} (kN) (concrete C30/37)
PCC 16	68,0
PCC 20	97,0
PCC 24	139,0
PCC 30-1	220,0
PCC 30-2	299
PCC 36	436,0

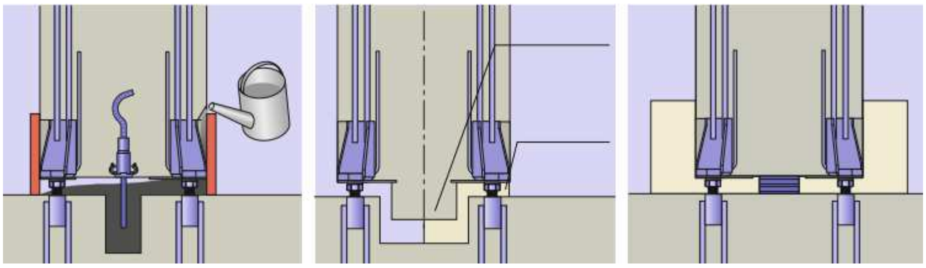


Figure 17. Possible solutions in existence of shear force (21)

2.4. Conclusions

After a brief overview of the types of column-foundation joints, it seems that the column shoe connection has the same advantages as other types of joints, but also far fewer disadvantages.

All types of connections installations process is described as quick and easy, however, both types of bolted connections don't need propping during it, due to immediate mechanical fixation and for both types it needs a small amount of grouting. Technologically it is easier to make grouting work in construction sites for bolted connections than for sleeve or pocket foundation type.

From the economic side, traditionally pocket foundation was described as a cheap solution and other connection types were presented as an alternative to it when due to loads or column cross-section pocket-size becomes too large. However, after some studies, it's obvious that the choice of column shoe connection can save the amount of steel and concrete but also save a time in a construction site. Comparing it with pocket foundation, it can save time and money because it doesn't require additional earthworks due necessity of deep column-foundation fixation. It is only necessary to mention that in some markets the details of the column shoes themselves can be considered as very expensive.

From the perspective of a structural engineer, column shoes have a huge advantage because it is a universal solution for connection column with any type of foundation. A very important aspect that due existence of design tools and software it is easy, fast and warranted way to make a design of such a connection. This may give a benefits for the duration and cost of the project design works. Also, it has a good performance during seismic or dynamic loads which gives additional safety to a whole structure.

It is limited accessibility on studies which investigates column shoe connections. However, from provided experimental data and conclusions done by other authors, it is clear that this connection system performs well in terms of seismicity and dynamic loads. The experiment described in Design Guidelines for Connection of Precast Structures under Seismic Actions doesn't provide final conclusions~~,~~ but obtained data shows that:

- it is important to have good quality control during the manufacturing process to avoid any imperfection of product;
- it is necessary to choose strong enough column shoe-and corresponding anchor bolt;
- the correct installation process is an indispensable part for performance of this type of connection system.

All these components listed above can become a main factor of whole structure failure during seismic or dynamic action. Also, information provided in references confirms that performance of column shoe connection is equivalate to cast-in-situ column performance.

Perhaps the main parameter given by manufacturers when choosing column shoes is designed resistance strength. All of which are summarized in Table 8. From it follows that for the same anchor bolt diameter, the bearing capacity does not differ or differs vary slightly between different manufacturers. These results are obtained using the same design standards. It is clear that anchorage length is being designed following EN 1992-1-1 Clause 8.7.3, shear strength of the connection can be calculated in accordance with EN 1993-1-8 clause 6.2.2. The resistance of this type of connection is being based on EN 1992-1-1, EN 1993-1-1 and EN 1993-1-8 design codes.

So resistance for the static load can't be the main criteria in a choice of column shoes between different suppliers. Much more important than the suggested product would have European or at least National approval of a country for which structural design is made.

Another important factor is the suitability of the product for the individual situation and the assurance of the manufacturer that their product can be used in such a case. Also, it is very important what design tools, design support the manufacturer provides.

Table 8. Summarize of design resistance

Colum shoes for moderate loads						
Anchorage bolt size (mm)	Nominal design tension resistance N_{Rd} (kN)					
	ANSTAR	PEIKKO	SEMKO[1]	RTEEL	HALFEN	PFEIFER
16	61,7	62	56,7	62,2	61,7	68
20	96,3	96	87,3	97	96,3	97
24	138,7	139	157,5	139,4	138,7	139
30	220,2	220	238,5	222,2	220,4	220
36	315,9	-	-	-	-	-
39	383,5	383	-	386,5	383,4	-
45	493.4	-	-	-	-	-
Colum shoes for high loads						
Anchorage bolt size (mm)	Nominal design tension resistance N_{Rd} (kN)					
24	161,6	-	-	-	-	-
30	299,2	299		299,2	299,2	299
36	435,7	436	423,9	435,7	436	436
39	520,5	521	506,7	520,5	520,6	-
45	696,5	697	677,7	696,5	696,6	-
52	937,6	938	911,7	937,6	937,3	-
60	1260	-	-	-	-	-

[1] Provided resistance by supplier is for concrete C35/45. Reduction coefficient equal to 0,9 was applied.

3. FEA ANALYSIS OF COLUMN TO FOUNDATION CONNECTION

3.1. A numerical model of column-foundation connection using FEA software

The main purpose of the experimental calculation is to investigate bending moment resistance of column shoe connection during erection and final stages and compare it with cast-in-situ column. Among the goals is to thoroughly investigate the behavior of each element of the connection, how the stresses are distributed in concrete and reinforcement.

For experimental column modeling, we chose the finite element analysis solver DIANA. The loading scheme is taken in such way as to meet axial force, which is an unavoidable element of external forces in the column design in a real situation, and to highlight effect on the connection of the bending moment. So the column was loaded axially at the top by force equal to 15kN and from the side, at 1428 mm distance from the bottom of the column, by force equal to 1kN (see Figure 18). Step of loading is 1kN. However, for acting bending moment calculations effect of axial force is not taken into account.

During numerical modeling the symmetry condition was applied, and fully rigid support was adopted. The cross-section of the column is 400x400 mm square. Height of the column is 1500 mm. following minimal requirements columns and grout, concrete is taken of class equal to C30/37.

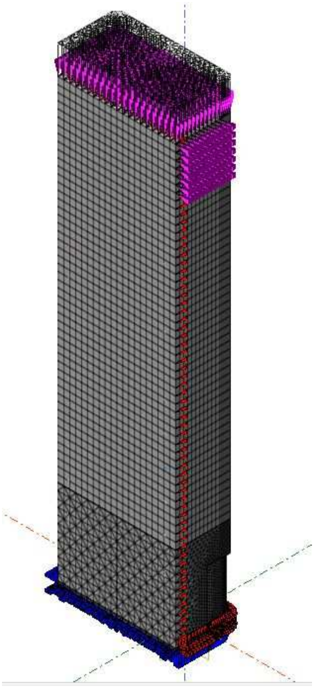


Figure 18. Rigid connection, symmetry conditions, loading

The following concrete linear material parameters were adopted:

$E = 32kN/mm^2$ –Young‘s model

$\nu = 0{,}2$ – Poisson's ratio

$\rho = 0{,}0025\ T/mm^3$ – density

For tensile behaviour is adopted exponential curve and tensile strength equal to $f_{ctm} = 2{,}9\ MPa$. Compressive behaviour of concrete described by parabolic curve, compressive strength is $f_{cm} = 38{,}0\ MPa$. Total strain based crack model is rotating.

The following reinforcement linear material parameters were adopted:

$E = 205kN/mm^2$ –Young‘s model

$\nu = 0{,}3$ – Poisson's ratio

$\rho = 0{,}00785\ T/mm^3$ – density

$\sigma_y = 550\ MPa$ – yield stress of reinforcement

For steel of column shoe plates, Young‘s model is $E = 210kN/mm^2$ and yield stress is $\sigma_y = 235\ MPa$.

For all steel is adopted Von Mises plasticity model.

By modeling the connection of the column, three main calculation schemes of the columns are obtained:

1. A model with HPKM24 column shoes and grouted joint
2. A model with HPKM24 column shoes without grouted joint
3. A model without in-situ column shoes with a rigid connection.

The first model simulates the conditions when the column is in the final stage, so non-shrinking grout has reached design strength and column is ready to be loaded by other structures. The tensile forces, based on the theory, is transferred through the bolt/shoe and the compressive force is transferred through the concrete/grout of the column and the shoe/anchor bolts.

The second model simulates the condition when the column is in the erection stage without grouting. So all forces being transferred to the foundation just by the shoe/anchor bolts.

For both models are used PEIKKO HPKM24 column shoes. This choice was made because this company is a leader not only in Europe but also worldwide in the production of these inserts and it is their column shoes that are most often found in practice. Another thing that led to this choice is that from a theoretical point of view it is about this type of column shoe that is most widely known.

The column was reinforced following PEIKKO recommendations and is as following (see Figure 19 and 20):

- Pos 1 – hook Ø8, S500, in compression side.
- Pos 2 – hook Ø8, S500, in compression and tension side.
- Pos 3 – column shoe reinforcement bar Ø10, S500, in compression side.
- Pos 4 – column shoe reinforcement bar Ø16, S500, in compression side.

- Pos 5 – column shoe reinforcement bar Ø16, S500, in compression side.
- Pos 6 – longitudinal reinforcement Ø20, S500, in compression side.
- Pos 7 – a group of transverse stirrups Ø8, S500.
- Pos 8 – transvers stirrup Ø8, S500.
- Pos 9 – transvers stirrup Ø8, S500.
- Pos 10 – longitudinal reinforcement Ø20, in tension side.
- Pos 11– hook Ø8, S500, in tension side.
- Pos 12 – column shoe reinforcement bar Ø16, S500, in tension side.
- Pos 13 – column shoe reinforcement bar Ø16, S500, in tension side.

These positions of bars are used throughout the work below for simplicity talking about the behavior of each separate bar. As well as these marking of positions found in the graphs below.

The same reinforcement was applied to the third model which simulates cast-in-situ connection.

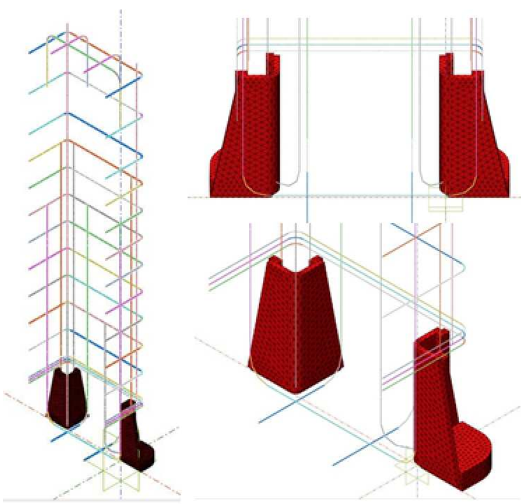


Figure 19. View of the column reinforcement in model and HPKM24 column shoes

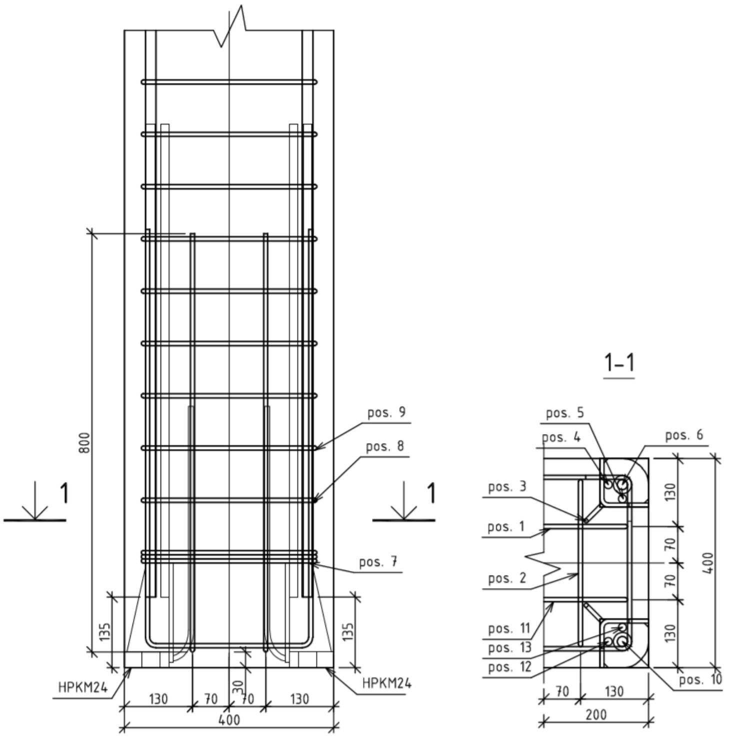


Figure 20. Reinforcement of column

3.2. Investigation of a column using column shoe connection with grout

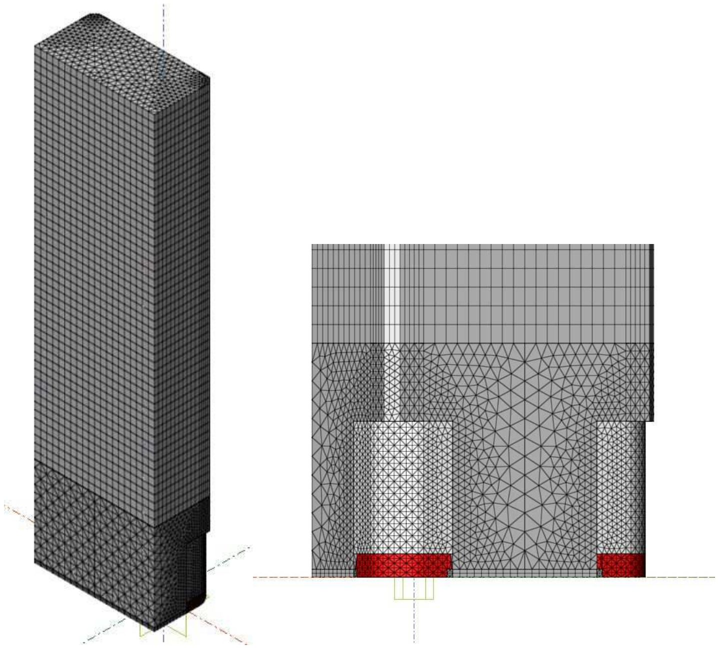


Figure 21. A model with a grouted connection

In this computational model of the column shoe connection to foundation, a grout is modeled (see Figure 21). The load is being increased step by step while reaching failure mode. As shown in Figure 22, at maximum loading we have a decomposition of concrete in the corner of the column at the top of grouting. It causes the development of crack.

Maximum step of loading: 123

A maximal moment at the bottom of the column: $M_{R.max} = 175{,}64\ kNm$

Displacement of top node of the column at maximum load: $s = 7.06\ mm$

Stresses in the concrete at the top node: $\sigma = -6.67\ MPa$

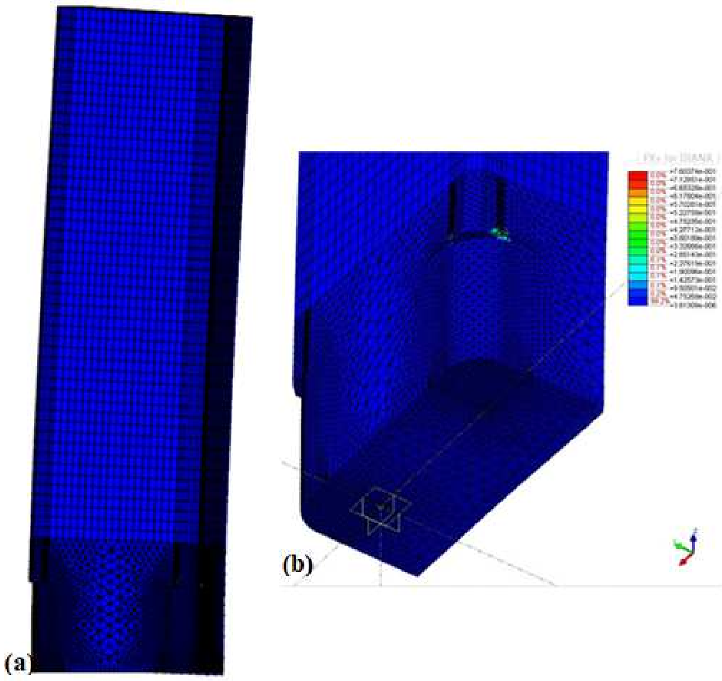


Figure 22. (a) Deformed diagram; (b) local decomposition of concrete at maximum load

Continuing to analyse concrete, at the bottom of the grout and at the bottom edge of the column, at the middle of it, concrete reaches its mean compressive stress $f_{cm} = 38,0\ MPa.$ (Figure 22). Removing the grout from the model, we can see that mean compressive stress is also reached in the edge of the opening for column shoe (dark blue colour). It should also be mentioned, that maximum tensile stress equals to $f_{ct.max} = 2{,}86\ MPa$ which is almost reaching mean tensile stress.

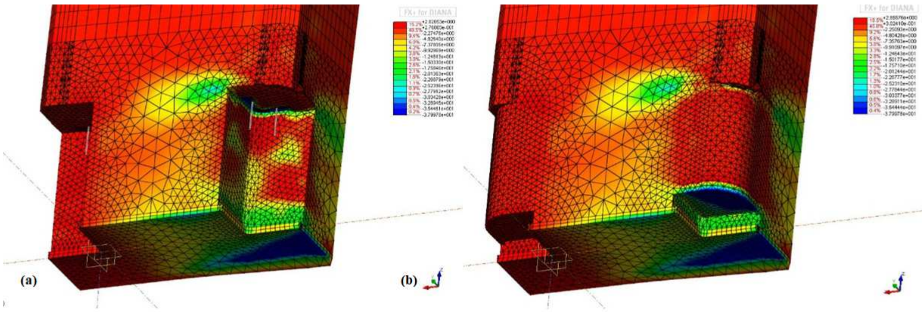


Figure 23. Compressive stresses in concrete at bottom of the column: column shown (a) without grout and (b) with grout

Looking at stresses in the grout, in one side of the column, it is fully in tension, while in the compressive side, as it was mentioned above, concrete fails at the bottom of grout (Figure 23).

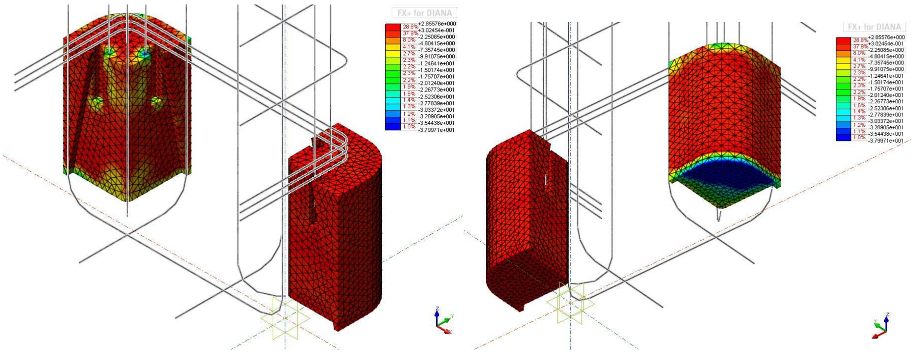


Figure 24. Compressive stresses of concrete in grouting

In Figure 24 is shown how changes stresses in the reinforcement due to the increase of the moment at the bottom of the column.

First of all, stresses in the compressive side increase more rapidly and evenly than on the tensive side. Compressive stresses are being reach first. When moment reaches 111,38 kNm, column shoe reinforcement starts to yield (Pos. 4). After follows another bar of column shoe (Pos. 5). Reaching 157,08 kNm moment~~,~~ fails hook reinforcement in the compressive side (Pos. 1). After it happens, stresses in tension side starts to increase rapidly in reinforcement of column shoe (Pos. 12), hook reinforcement (Pos. 11) and transfers stirrup (Pos. 8). Finally, fails hook (Pos. 2), column shoe reinforcement (Pos. 3) and transfers stirrup (Pos. 8) reaching moment 169,93 kNm, 172,79 kNm and 174,12 kNm respectively.

After the final failure of a column, in the compressive side, just longitudinal reinforcement doesn't reach its yielding strength, while due to tension fails just one of the stirrups.

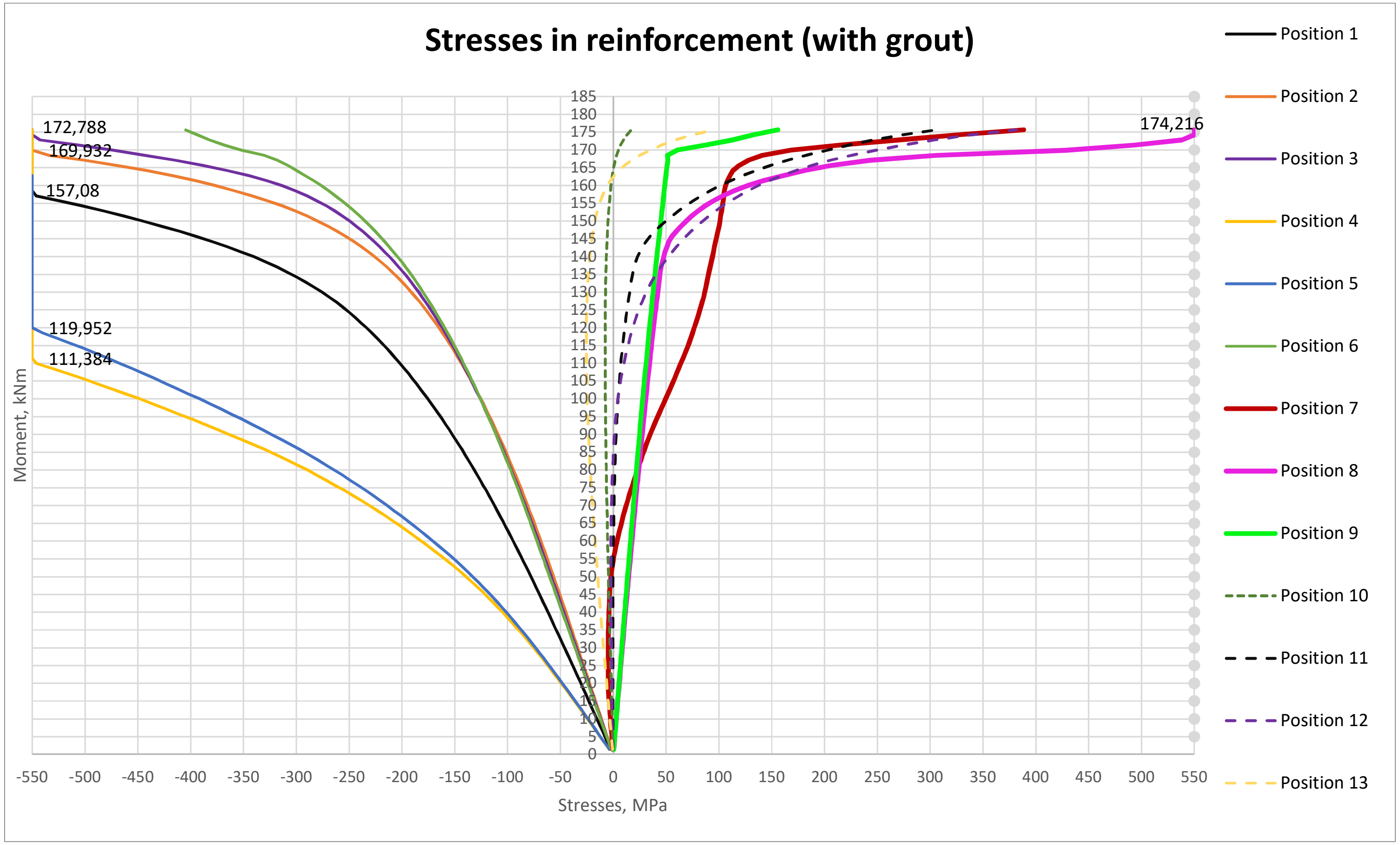


Figure 25 . Stresses in reinforcement (column with grout)

In Figure 26 is shown in which zones of reinforcement accurse failure. The most damaged reinforcement is in the bottom of the column, in the zone of column shoe, where acts the biggest bending moment. From Figure 27 we can see that column shoe plates doesn’t reach ultimate strength neither due tension (left), neither due compression (right).

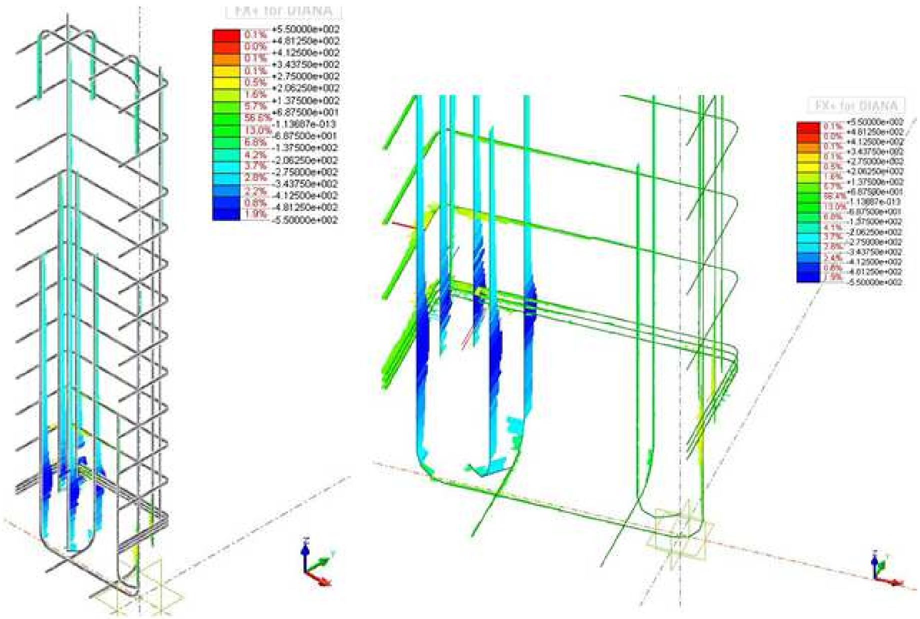


Figure 26. Stresses in reinforcement

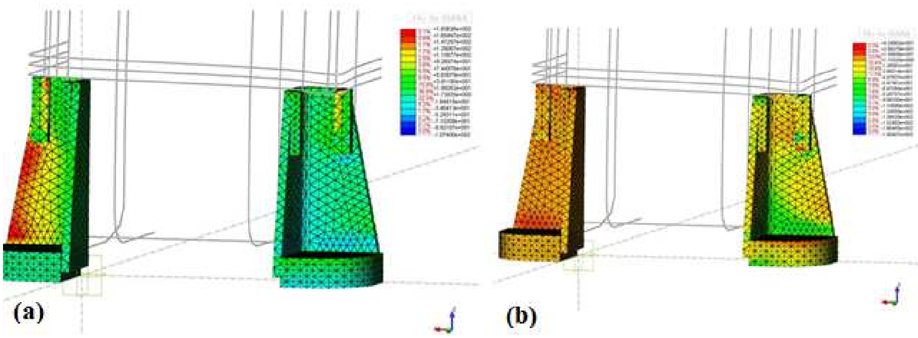


Figure 27. Tensile (a) and compressive (b) stresses in column shoes at maximum load

In Figure 28 mentioned areas in which maximal stresses in concrete investigated (see Figure 29). In these zones are the biggest concetration of stresses. In Zone 1.2 near column shoe, mean compressive stresses reached near 30% smaller acting load than in the edge of column. From information provided above, exactly in this zone happens decomposition of concrete.

If to compare load near which concrete fails with load when first reinforcement bar starts to yield, so we see that failure mode starts from failure of reinforcement.

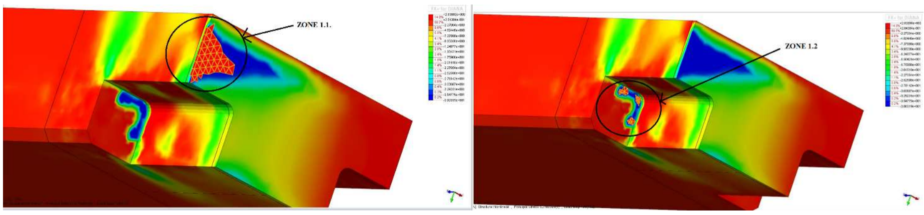


Figure 28 Zones of maximal stresses in concrete

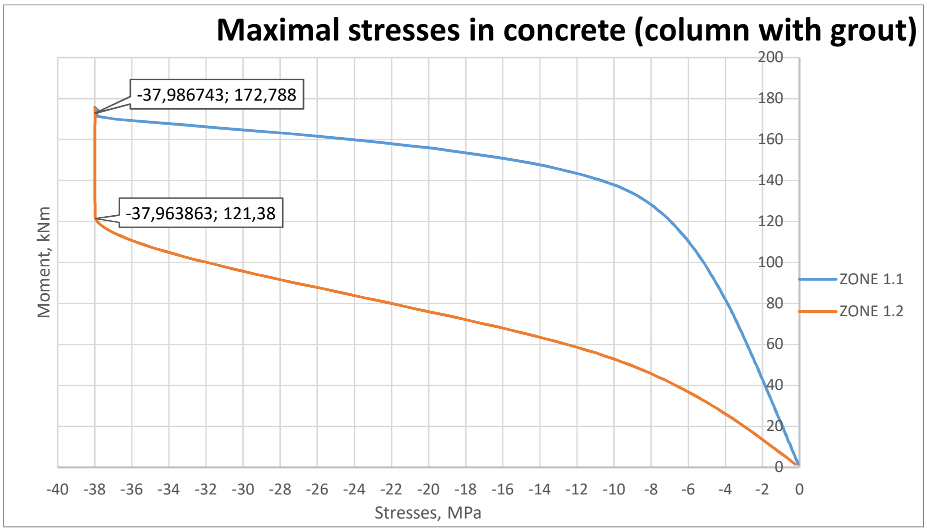


Figure 29. Maximal stresses in concrete (column with grout)

3.3. Investigation of the column using column shoe connection without grout

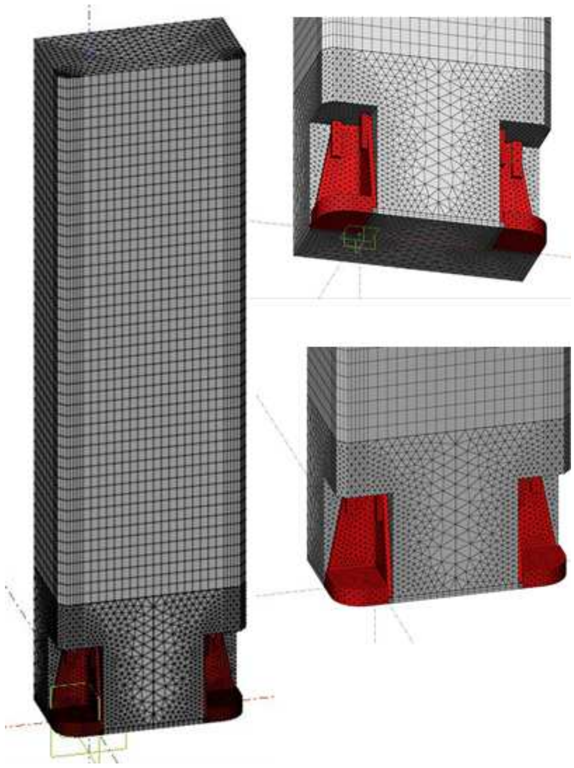


Figure 30. Column shoe connection without grout

In this variant of the model, where grout is excluded. So, it is can be equalized to column shoe connection during the erection stage. This is done to better examine the effect of the mortar on the behavior of the column connection. Reinforcement and its marking (positions) are the same as in the model above. The load is being increased step by step while reaching failure mode.

The moment at maximum load: $M_{R.max} = 162{,}7\ kNm$

Displacement of top node of the column at maximum load: $s = 7.9\ mm$

Stresses in the concrete at the top node: $\sigma = -7.54\ MPa$

From Figure 31 we see that in this model happens much more visible decomposition of concrete. In the tension side, appears transverse cracks which go from the corner of the opening for a column shoe. Also, the corner of the column in the tension side is being fully decomposed.

In compressive side appears longitudinal crack, which goes from the corner of the opening for a column shoe. All corner, above the opening for a column shoe, is being sheared. Decomposition happens also in the middle edge of the column bottom.

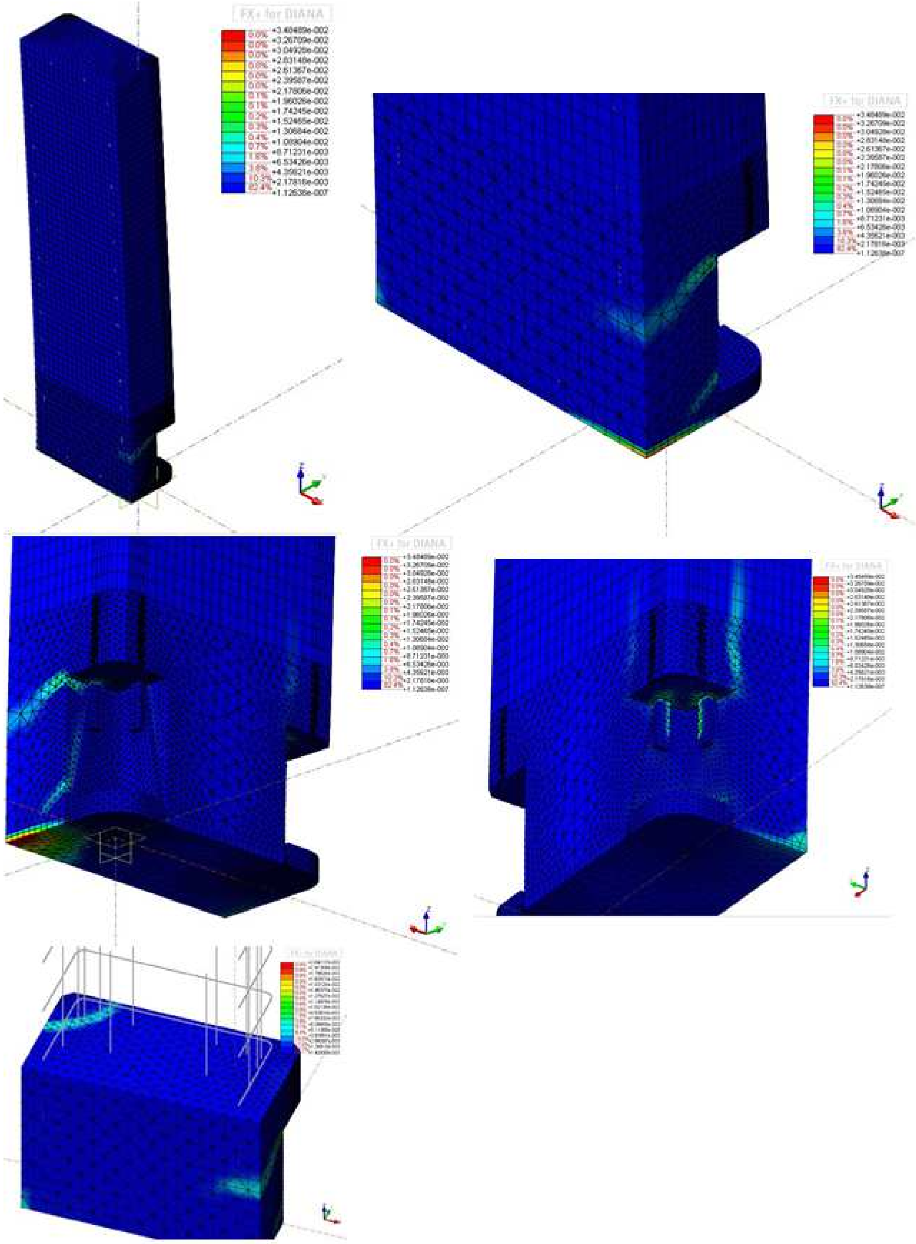


Figure 31. Local decomposition of concrete at maximum load

Mean compressive stress $f_{cm} = 38,0\ MPa$ is being reached in the bottom edges of the column (Figure 32, dark blue colour). Furthermore, mean tensile stress $f_{ctm} = 2,9\ MPa$ is being reached, too.

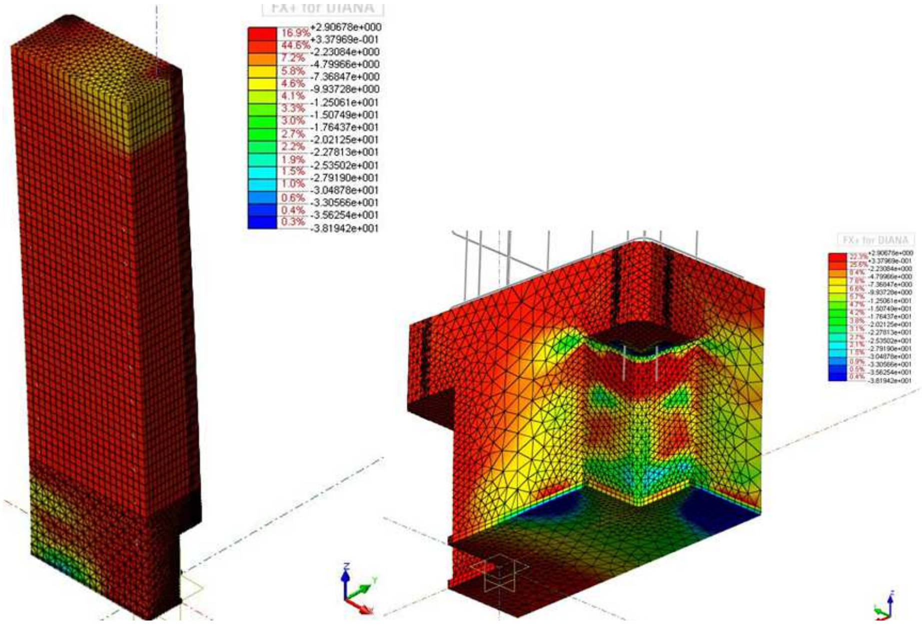


Figure 32. Compressive stresses in concrete

In Figure 33 is shown how changes stresses in the reinforcement due to the increase of the moment at the bottom of the column without grout.

Same as in the first model, first reinforcement which starts to yield is a reinforcement of column shoe (Pos.4). However, it starts to yield being exposed by 14% bigger bending moment when in the first model. It can be explained that stresses in the column are being distributed with concrete and shoe plates. From the graphic below, we see that tensile stresses are rapidly increasing in the group of transverse stirrups (Pos. 7). It has direct contact with reinforcement of column shoe and, as shown in Figure 33, it fails when moment reaches 129,95 kNm exactly at the node which interacts with reinforcement in Pos. 4.

Reinforcement in Position 5 starts to yield already near the smaller bending moment when was in the first model. After its failure, in the last reinforcement bar which belongs to column shoe (Pos.3) and in the transverse stirrup (Pos. 8) stresses are starting to increase very rapidly and soon it reaches ultimate strength, too. Reaching bending moment equal to 161,34 kNm fails hook reinforcement in Position 1.

After the final failure of a column, in compressive side longitudinal reinforcement (Pos. 6) and hook (Pos. 2) doesn't reach its maximum stress capacity. In tensive side, column shoe reinforcement (Pos. 12) reach steel characteristic strength, however, doesn't start to yield. The least exposed reinforcement is a longitudinal one (Pos. 10).

Also, it should be mentioned, that transverse stirrup (Pos. 8) which is above of a group of stirrups fails at the node of interaction with column shoe reinforcement (Figure 34). From the same Figure, it seems that some nodes of a bar in Position 5 are being in compression and some are being in tension, however, it reaches stress capacity just in compression, while in tension maximum stress in one of the nodes is being reached till 466 MPa.

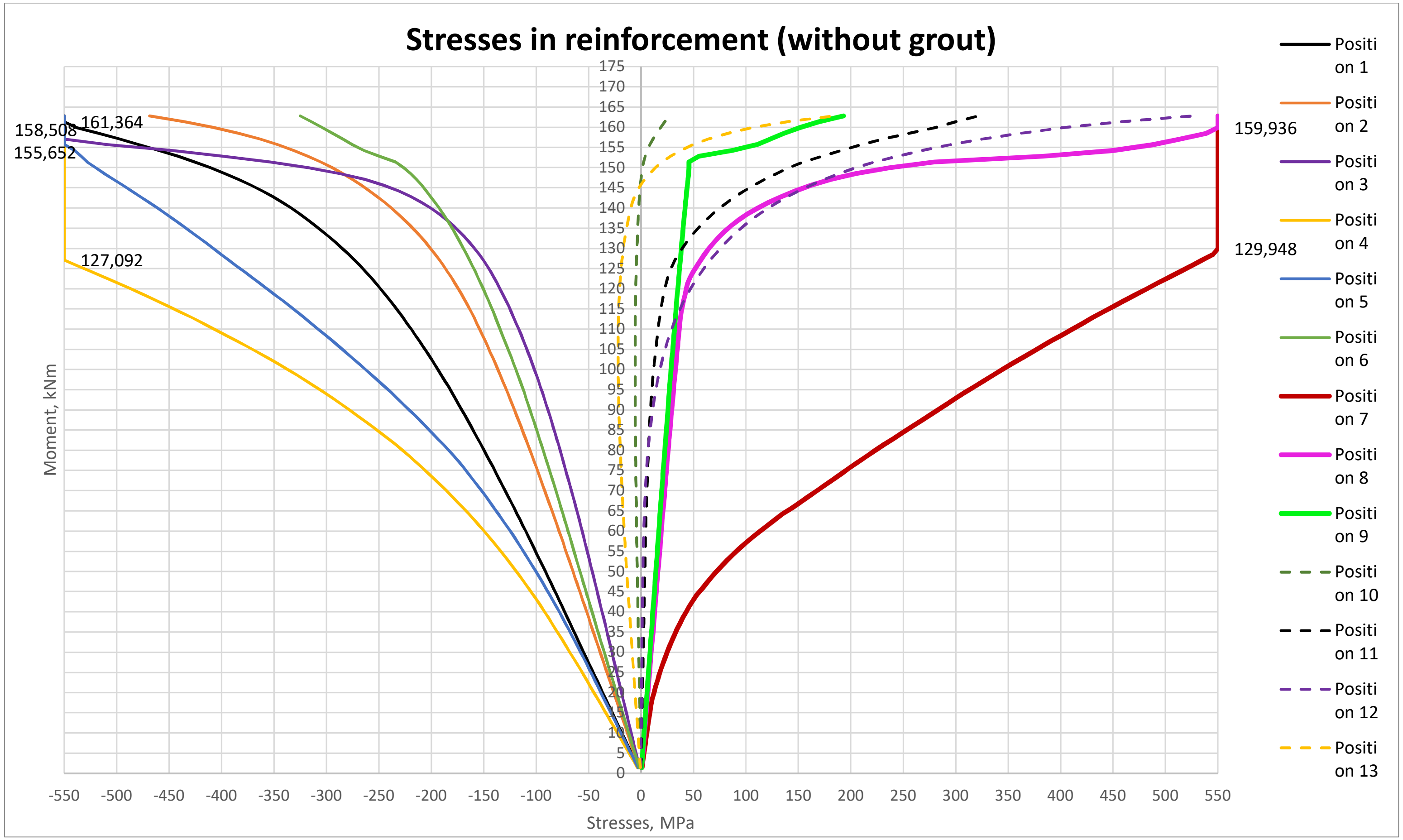


Figure 33. Stresses in reinforcement (column without grout)

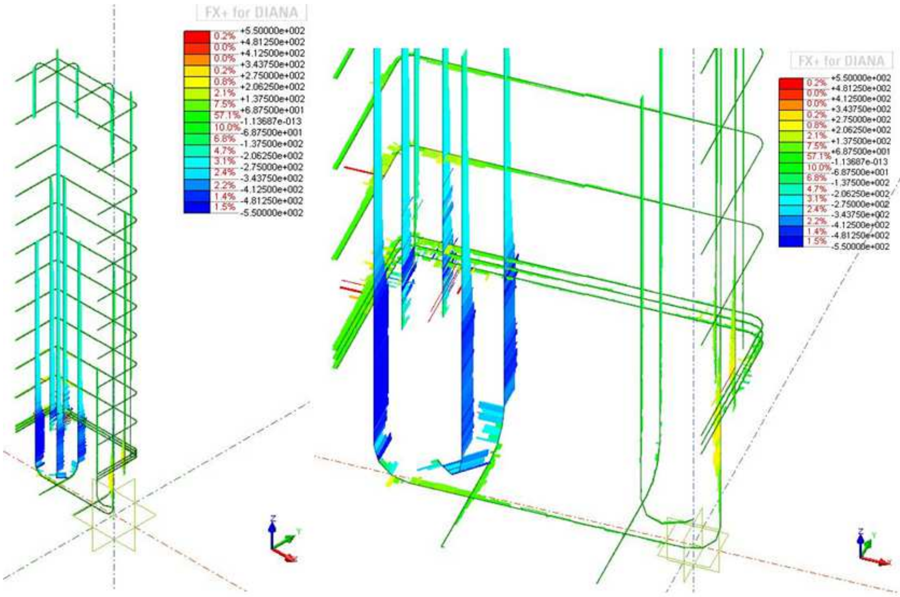


Figure 34. Stresses in reinforcement

From Figure 35 we can see that column shoe plates almost reach yielding strength due tension (left), while due compression (right) stresses are small. Tensile tresses are 33 % bigger than in the first model.

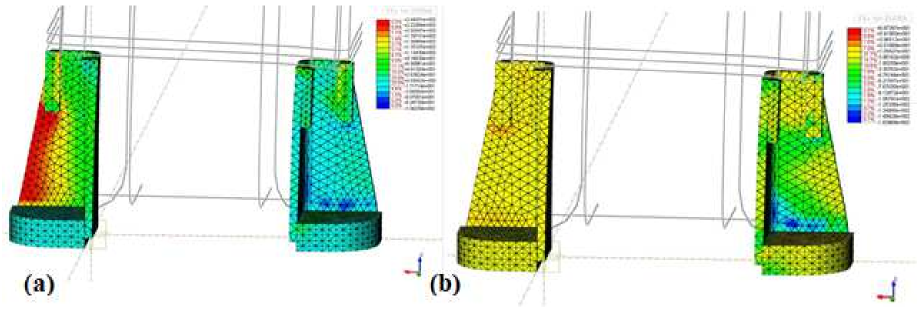


Figure 35. Tensile (a) and compressive (b) stresses in the column shoes at the maximum loading

In Figure 36 mentioned areas in which maximal stresses in concrete investigated (see Figure 37). In these zones are the biggest concetration of stresses. In Zone 2.3 near column shoe, mean compressive stresses reached near 28% smaller acting load than in the edge of column (zone 2.2).

this situation is similar to 1st sample, however, additionally mean compressive stresses reached in the corner of the column. From information provided above, exactly in these zone happens decomposition of concrete.

If to compare load near which concrete fails with load when first reinforcement bar starts to yield, so we see that failure mode starts from failure of reinforcement.

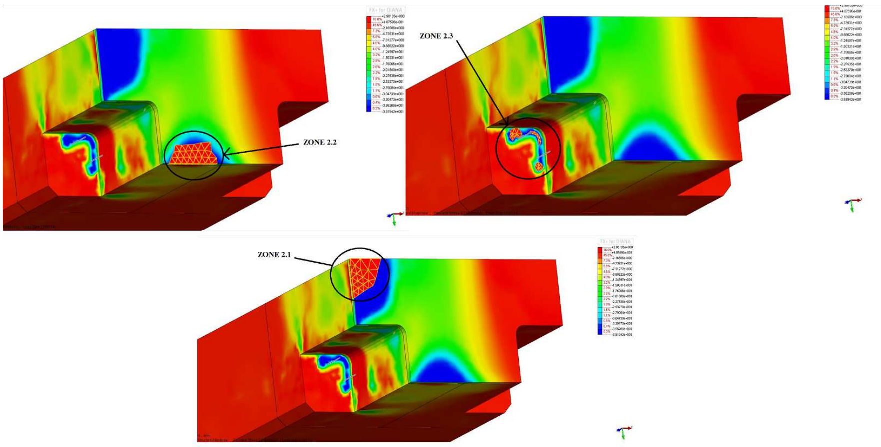


Figure 36 Zones of maximal stresses in concrete

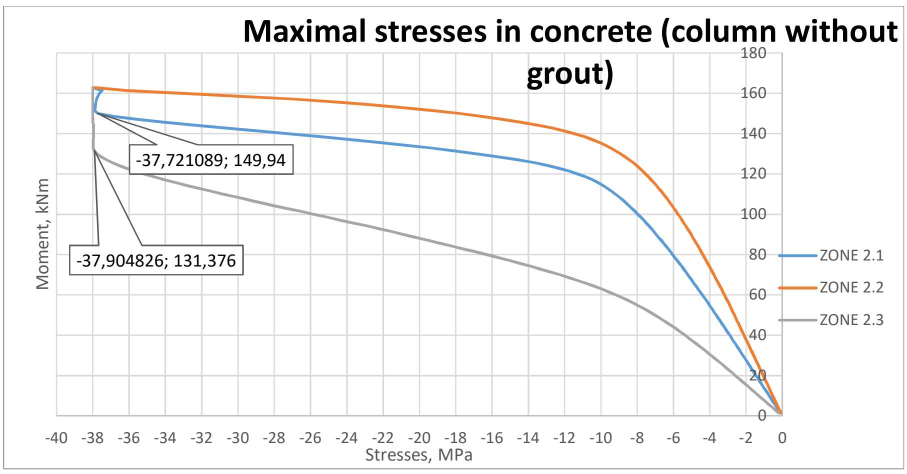


Figure 37. Maximal stresses in concrete (column without grout)

3.4. Investigation of cast-in-situ column with rigid connection

In this computational model, the column is modeled as cast-in-situ (see Figure 38). It is reinforced analogous to other models. This model is done to compare connections of column shoes with a monolithic one. As in previous cases, the load is being increased step by step while reaching failure mode. As it is shown in Figure 38, at maximum loading we have a decomposition of concrete in the support zone.

Maximum step of loading: 128

The maximal moment at the bottom of the column: $M_{R.max} = 177{,}07\ kNm$

Displacement of top node of the column at maximum load: $s = 6.51\ mm$

Stresses in the concrete at the top node: $\sigma = -6.11\ MPa$

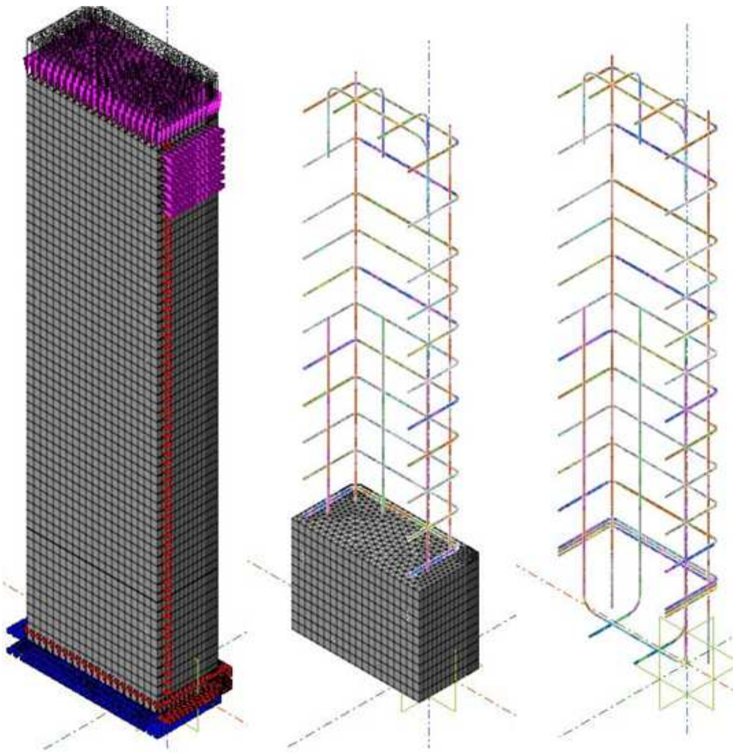


Figure 38. Model of the cast-in-situ column

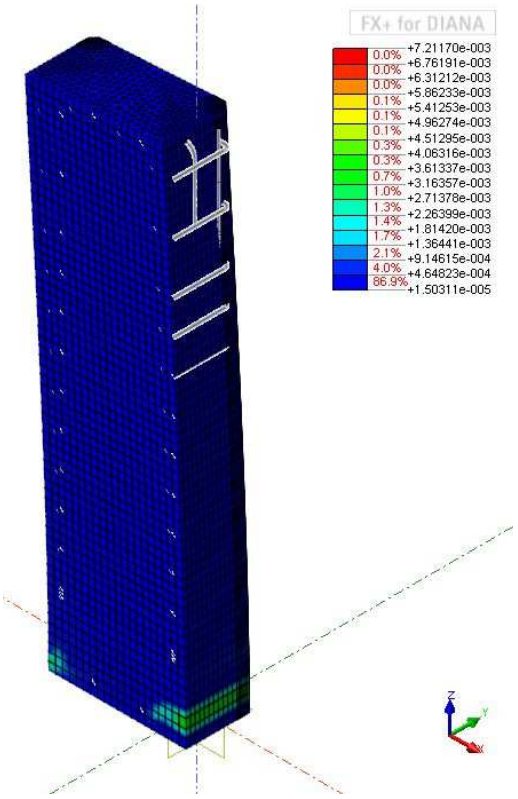


Figure 39. Development of a crack at the support

Mean compressive stress $f_{cm} = 38,0\ MPa$ is being reached in the bottom of the column in the compressive side (Figure 39). The maximum tensile stresses are being overreached and are equal to $f_{ctm.max} = 4{,}2\ MPa$ during final failure of the column.

From Figure 41 and 42 we see that maximal stresses are concentrating on longitudinal reinforcement (Pos. 6). The final failure of the column occurs just after it is reaching its ultimate strength. So a conclusion can be made that exactly it causes failure. All shear bars (Pos. 7,8 and 9) works in a very similar habit. It shows an equal distribution of stresses between them.

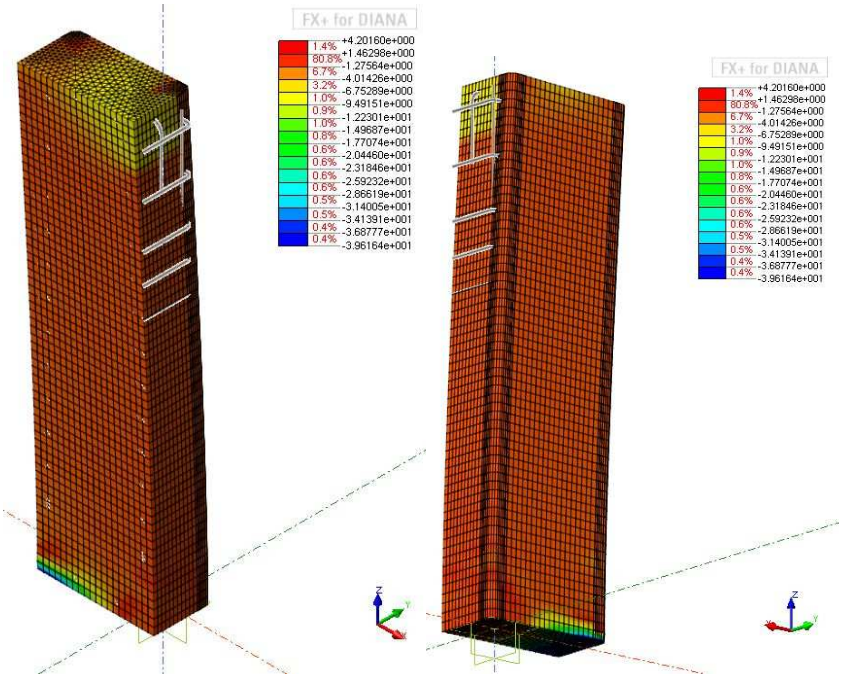


Figure 40. Compressive stresses in concrete

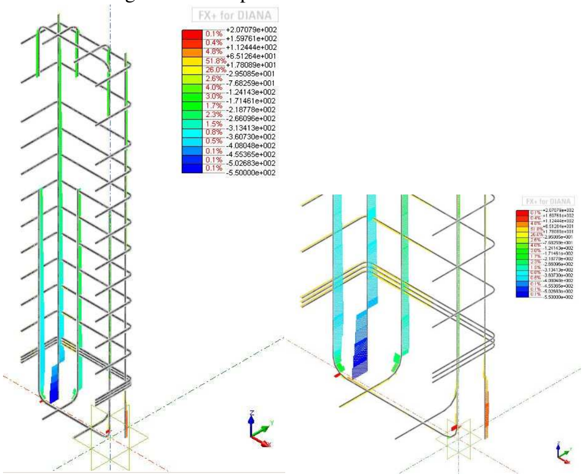


Figure 41. Stress in reinforcement

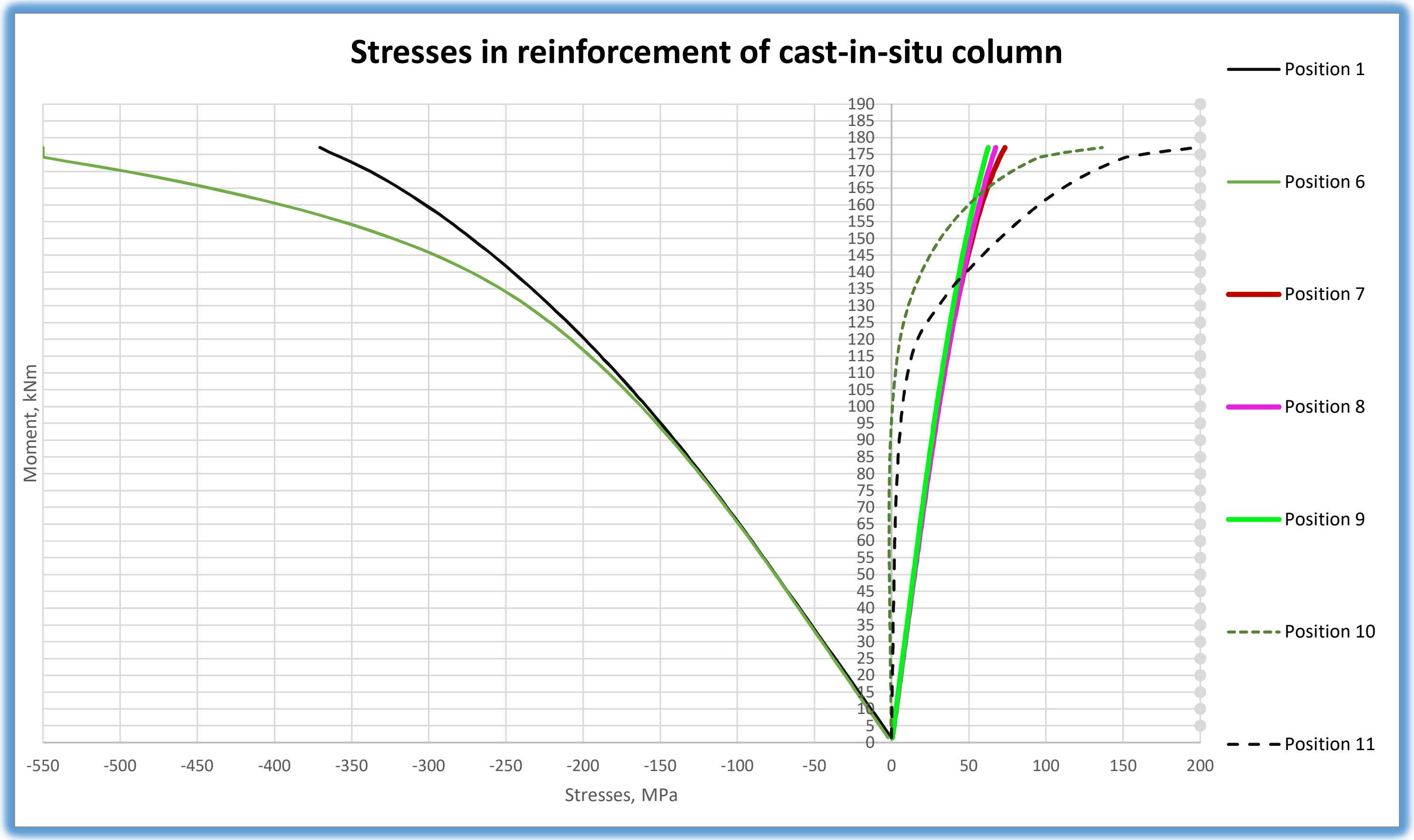


Figure 42 Stresses in reinforcement of cast-in-situ colum

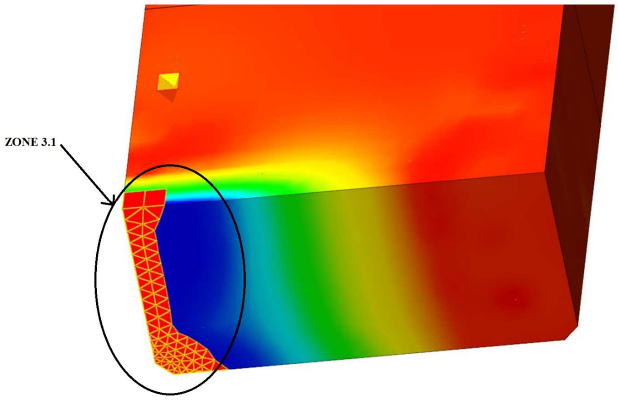


Figure 43 Zones of maximal stresses in concrete

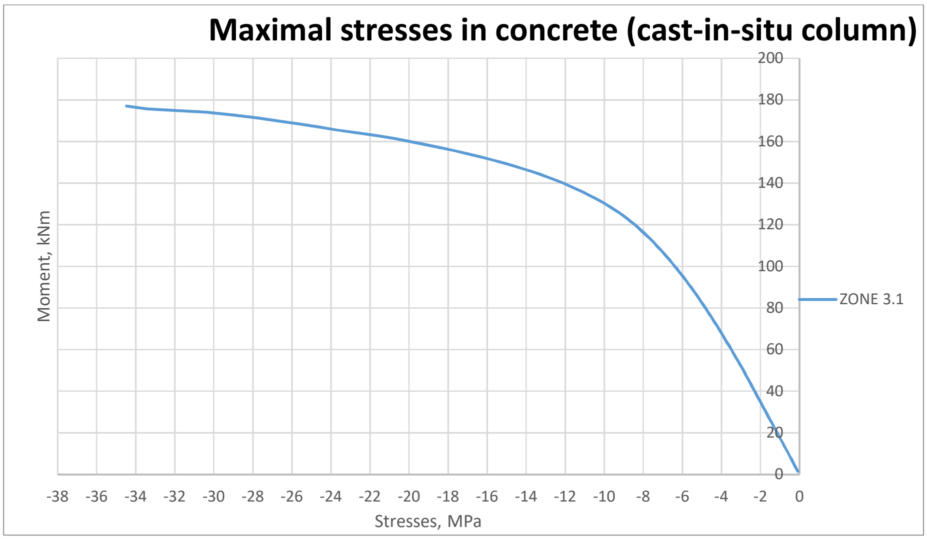


Figure 44 Maximal stresses in concrete (column without grout)

As shown in Figure 43 the highest stresses are in the edge of column, however as it shown in Figure 45, mean compressive stress is being not reached. So failure mode in this case happens just due failure of longitudinal reinforcement.

3.5. Comparison of the results

In Figure 45 is presented bending moment/displacement diagram of all 3 models. The best performance is a cast in place column, having displacement equals to 6,52 mm and reaching 177,07 kNm bending moment capacity. A column with grout carries bending moment which is just 0,77% less, however, displacement is bigger by 8,28%. From the diagram we see that 1st and 3rd models curves meet while reaching approximately 161,36 kNm bending moment. Before it, better performance shows the 1st model. It can be explained by bigger stiffness due to additional reinforcement from column shoes. The curve flattens approximately when fails reinforcement of column shoe and one of the hook bar. Because of its stiffness of column bottom decreases.

2nd model curve from the beginning is more flat than the other two. It is due to the lack of grout which causes smaller stiffness of connection. This model fails from 8,06% smaller bending moment than in cast in place column model and reaches 21,17% bigger displacement. However, taking into account, that this model can be equalized to the column during erection stage and in a real situation it will never be fully loaded, the difference in strength and displacement would be much smaller if to compare all 3 models with loading during erection stage.

In Annex 6 are presented graphics of streeses comparison of each reinforcement position.

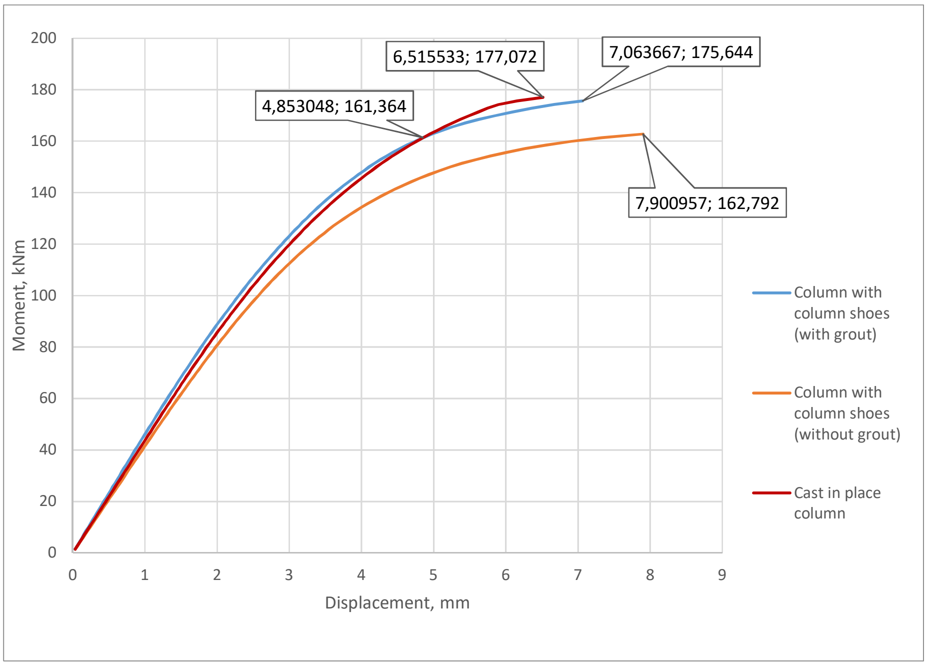


Figure 45. Bending moment/displacement diagram.

To understand the behavior of the column shoes in real situations, the stress diagrams at approximately 60% load are reviewed below.

In Figures 46-50 are presented stresses in reinforcement bars which belongs to column shoes. Stresses are higher by 5-75% in case when we have column shoe with grout. Just in one of the bars, in tension side, stresses are 3 times smaller than in the same bar when we have sample without grout. However, in sample without grout stresses increases 6,5 times more in the group of shear reinforcement (see Figure 51)

In Figues 51-53 are presented stresses in shear bars. In the group of shear bars stresses of 1st sample are higher by 81% in camparison with cast-in-situ column. Thought above shear bars takes till 5 % less stresses.

In Figure 54-55 are presented stresses in longitudinal bars. In compressive side, stresses in cast-in-situ column are 37% higher than stresses in column without grout and 30% higher than stresses with grout. In tension side, in columns with shoes longitudinal bars are being compressed while in cast-in-situ column bar works in tenson.

In Figures 56-58 are presented stresses in hook reinforcement. In sample without grout compressive stresses are higher by 8-9% than in sample with grout, and 20% higher than in cast-in-situ column. Tensile stresses are 236% higher than stresses in the 1st sample, and 65% higher than stresses in the 3rd sample. Meanwhile in compressive side stresses 10% higher in sample with column shoes with grout than in cast-in-situ sample. In tensive side is reverse situation and stresses in cast-in-situ sample are higher by 42%.

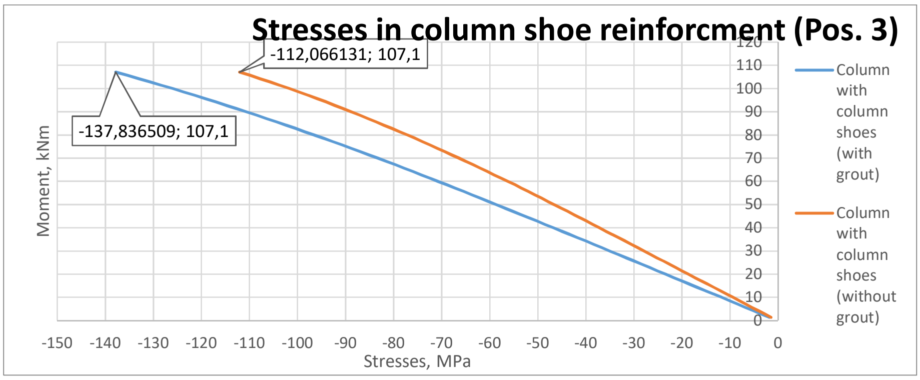


Figure 46. Stresses in column shoe reinforcement at ~60% load (Pos.3)

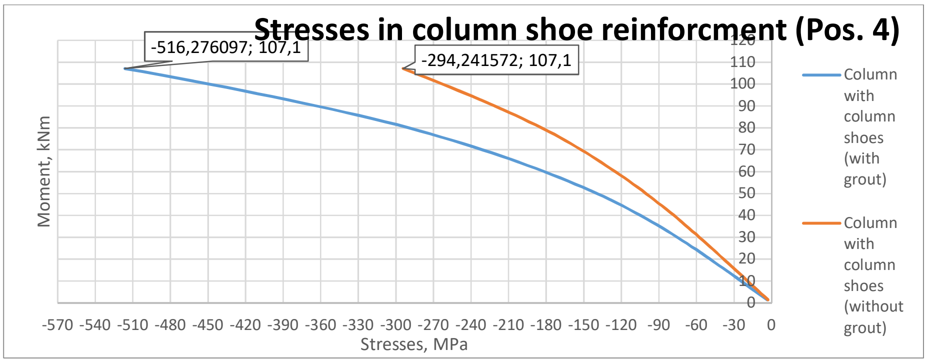


Figure 47. Stresses in column shoe reinforcement at ~60% load (Pos.4)

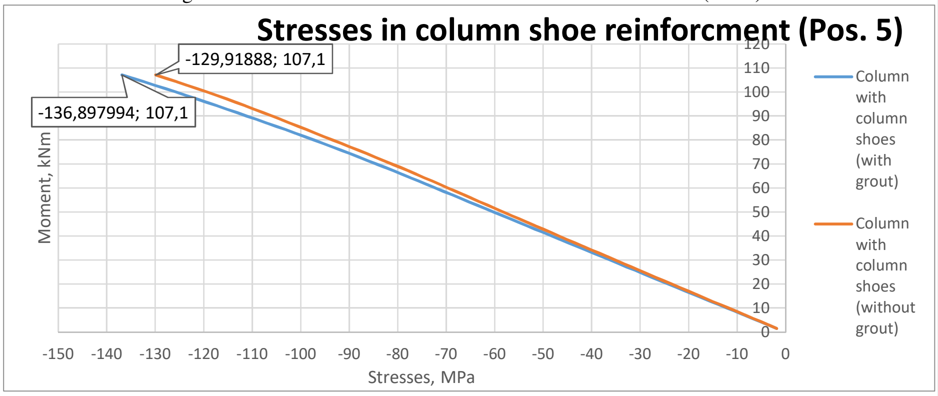


Figure 48. Stresses in column shoe reinforcement at ~60% load (Pos.5)

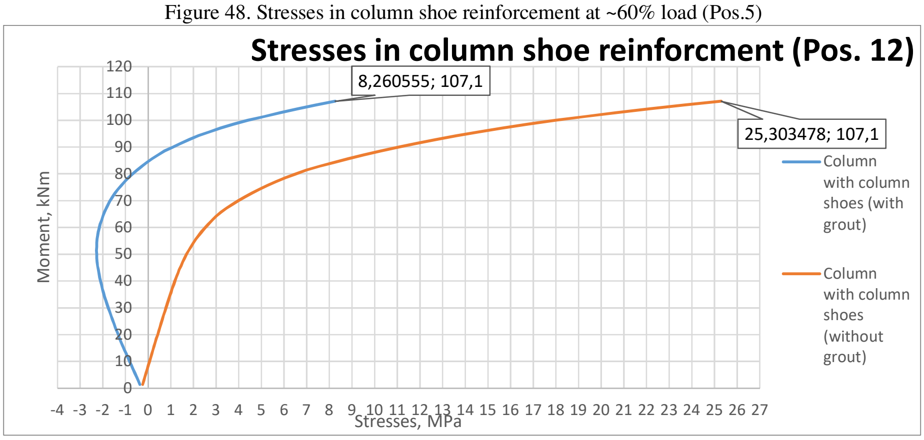


Figure 49. Stresses in column shoe reinforcement at ~60% load (Pos.12)

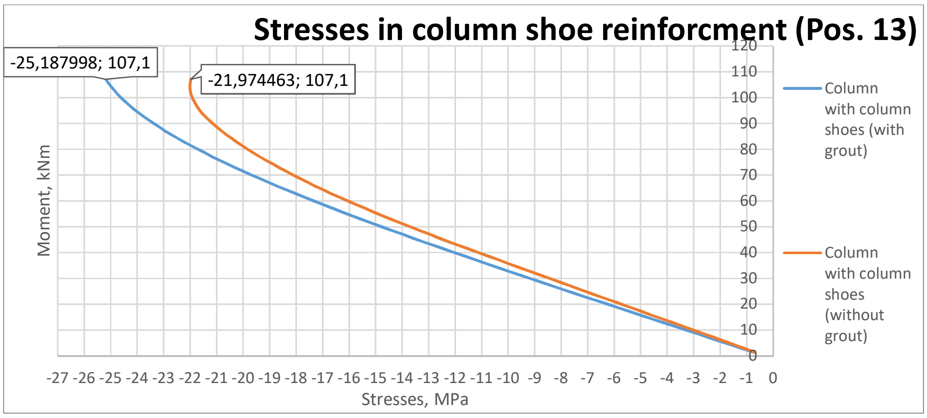


Figure 50. Stresses in column shoe reinforcement at ~60% load (Pos.13)

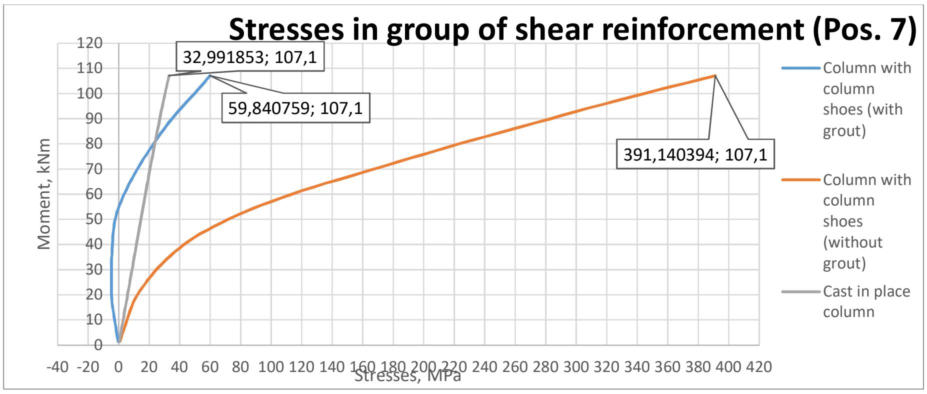


Figure 51. Stresses in group of shear reinforcement at ~60% load (Pos.7)

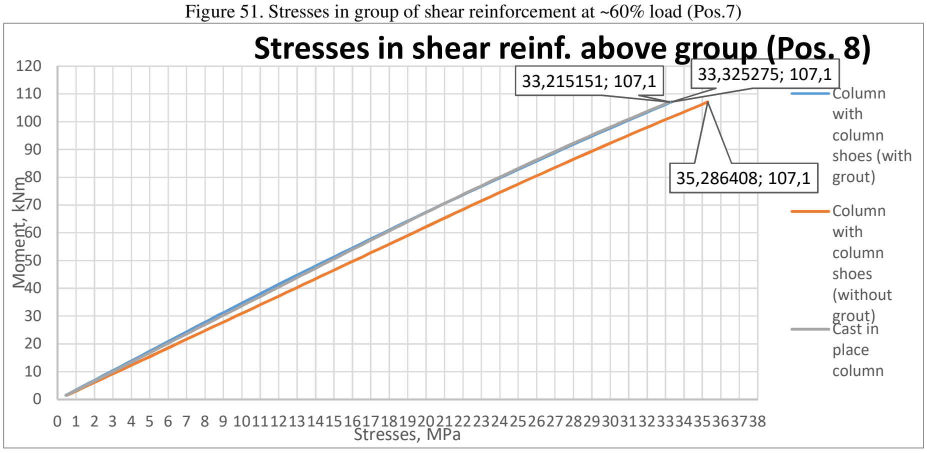


Figure 52. Stresses in shear reinforcement at ~60% load (Pos.8)

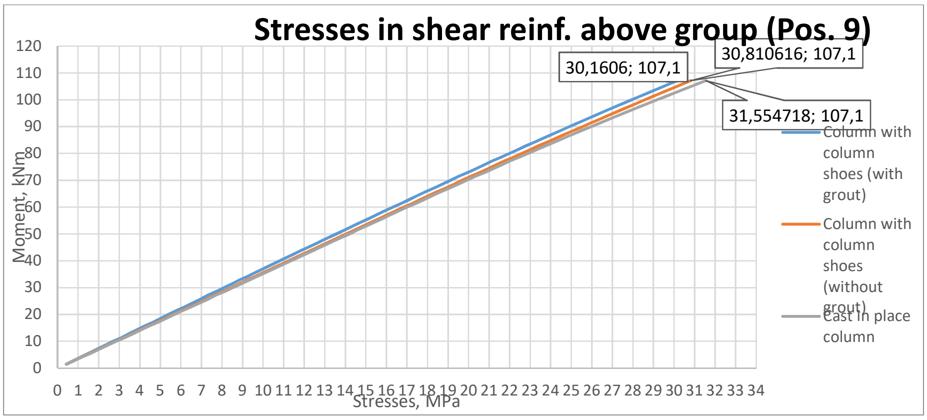


Figure 53.. Stresses in shear reinforcement at ~60% load (Pos.9)

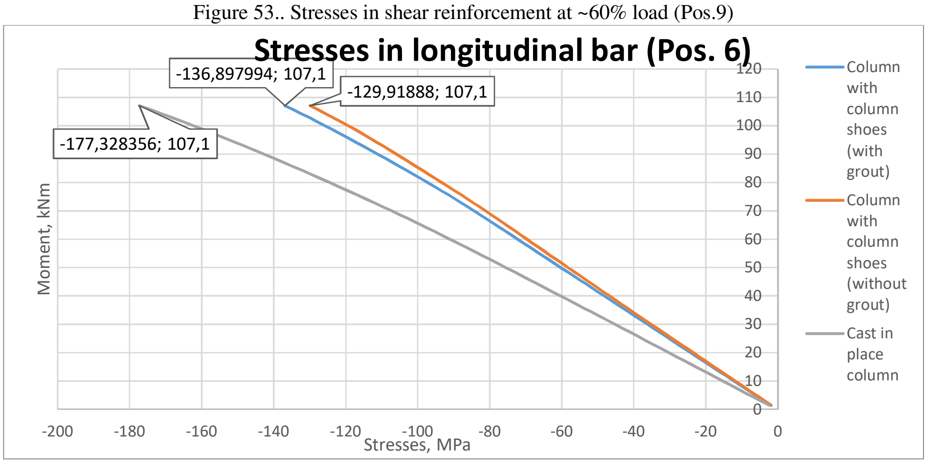


Figure 54. Stresses in longitudinal reinforcement at ~60% load (Pos.6)

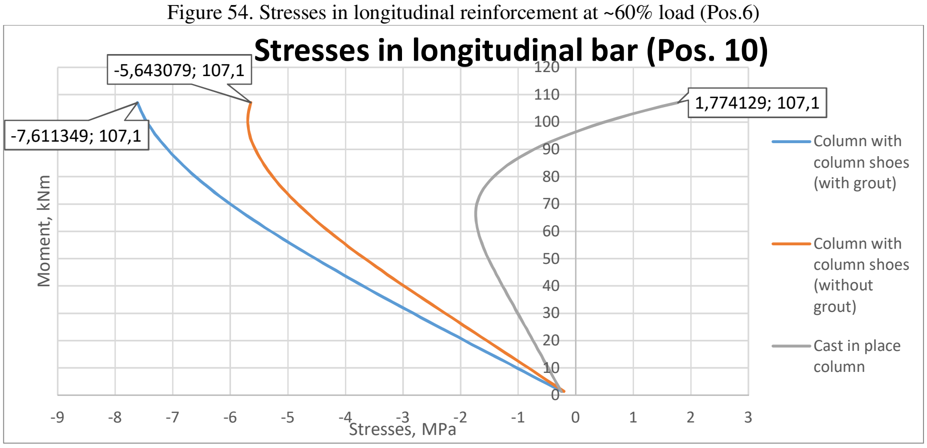


Figure 55. Stresses in longitudinal reinforcement at ~60% load (Pos.10)

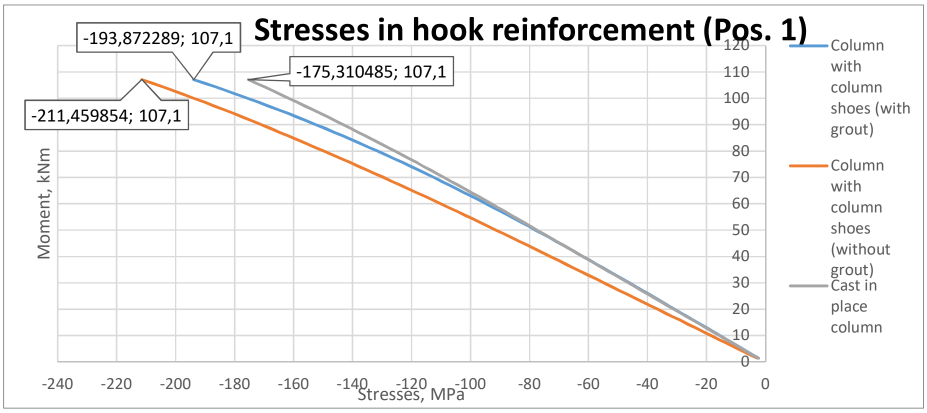


Figure 56. Stresses in hook reinforcement at ~60% load (Pos.1)

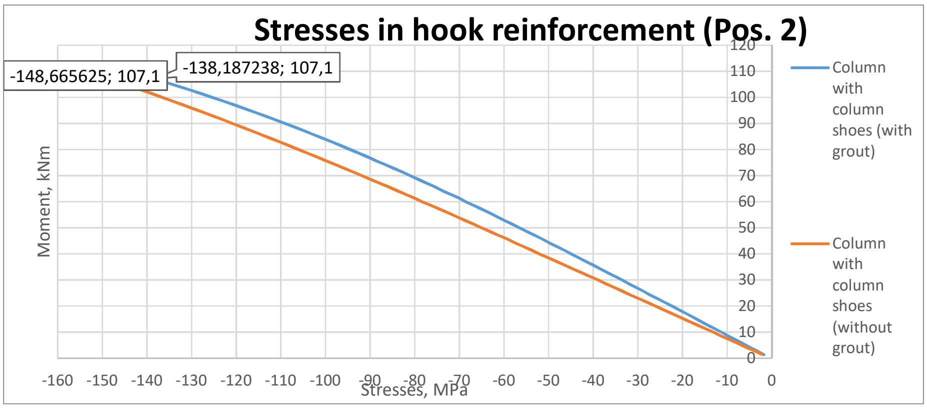


Figure 57. Stresses in hook reinforcement at ~60% load (Pos.2)

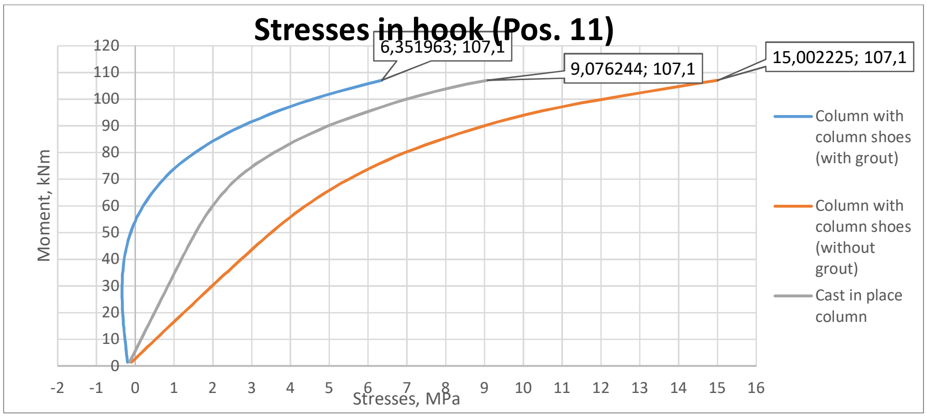


Figure 58. Stresses in hook reinforcement at ~60% load (Pos.11)

3.3. Conclusions

In columns during erection stage stresses are higher in hook and shear reinforcement than during the final stage. Also, shear reinforcement was first component in assembly which reached its ultimate strength. So it must be considered during design of column shoe connection. This reinforcement must withsatnd vertical loads caused by wind and should be strong enough that fail wont appear during mounting works. Also, in most of shear and hook reinforcement stresses are higher in comparison with cast-in-situ column. So, when choosing this reinforcement, more attention should be paid to these groups of bars when we have a column shoe connection.

Meanwhile, stresses in longitudinal bars are smaller with column shoes than in cast-in-situ column. During final stage most stresses are being concentrated in column shoe bars. So this could allow for a more economical choice of main reinforcement when we use the column shoe system.

In models with column shoes, concrete reaches its mean compressive stress and decomposition happens first of all in zone where is installed column shoe, secondly in edges/corners of column. In all three models first being reached limits of compreesive stresses, also failure mode accurs due to yielding of reinforcement. In models with column shoe first start to yield bars of column shoes, in 3rd model – longitudinal bars.

Bending moment capacity of column with shoes during final stage and cast-in-situ column can be equated to each other.

CONCLUSIONS AND RECOMMENDATIONS

1. Column shoe connection is an innovative column to foundation solution comparing it to other connection types for prefabricated concrete elements. It can save not only the amount of steel and concrete used in construction~~,~~ but also the time needed for design and in a construction site. The column shoe connection has not only the same advantages (immediate fixation, a small quantity of needed grout, moment resisiting connection, no bracing is needed, good performance during seismic and dynamic loading, availability of design softwares, templates) as other types of connecting systems used in prefabricated column to foundation connection, but also far fewer disadvantages. It is universal solution for connection column with any type of foundation.
2. The main downside is that this can be considered a relatively expensive solution, especially in certain markets. However, it is difficult to evaluate from an economic point of view. When choosing this solution should keep in mind that it will save materials, there is a much faster installation process than other solutions and simple design dimensioning.
3. From the literature review, it is clear that behavior during seismic and dynamic actions is similar to the behavior of cast-in-situ column. Good dissipation of connection is reached. Also, more rational design than the case of traditional reinforced concrete precast structures characterized by monolithic columns and pinned beams can be obtained.
4. Following completed studies, this type of connection reach very similar to the cast-in-situ solution bending resistance, stiffness resistance and shear resistance.
5. It is very important to have good quality control during manufacturing to avoid any possible incongruity in inserts. Also, the correct installation process is an indispensable part of the performance of this type of connection system. Another important thing is the right choice not only shoes but also anchor bolts. Only by implementing all these points can the desired behavior of the column shoes be ensured.
6. Despite the wide choice of manufacturers, the solutions they present vary little. Both the production process and the design are based on the same standards and design rules. Strict quality control is used in production. Thus, it cannot be said that one product is better than another. So, the main factor in choosing between column shoes of different manufacturers can become price and European or at least National approval of a country for which structural design is made.
7. Choice of column shoes and anchorage bolts can be made easily using design tools. However, it is very important to keep more attention in cases where seismic, dynamic and fatigue loads are acting.
8. Numerical analyses showed that for better performance and more close behavior to cast-in-situ column, grout is necessary. But even without it, good results are achieved. So, column can work immediately after installation.
9. Stresses in longitudinal bars are smaller with column shoes than in cast-in-situ column. During final stage most stresses are being concentrated in column shoe bars. So this could allow for a more economical choice of main reinforcement when we use the column shoe system.
10. The behavior of connection with column shoes with grout is very similar to behavior of cast-in-situ column. However, when connection is not grouted bending moment resistance is smaller by 8% and displacement increase by 21% in comparison with cast-in-situ column.

11. When we use column shoe system, it should be taken extra care for design of additional reinforcement like hooks and shear bars due to higher stress concentration in them than in cast-in-situ solutions.

12. When we use column connection with shoe, higher stresses are being concentrated in concrete than in cast-in-situ column. Having 60% of loading in column with grouted connection stresses in the most critical zone are almost 5 times bigger than in cast-in-situ column. In connection without grout stresses are 4 times bigger.

13. In reinforcement yield stress is reached at 111kNm for column with grout (column shoe bar), at 127 kNm for column without grout (column shoe bar) and at 177kNm for cast-in-situ column (longitudinal bar). While mean compressive stresses is reached at 121 kNm for column with grout, at 131 kNm for column without grout and in cast-in-situ column mean compressive stress is not reached.

14. In connections with column shoes, the vertical steel stiffening edges create a concentration of compressive and tensile stresses, so the concrete above those edges begins to crack earlier, and high tensile stresses are achieved in the transverse reinforcement. In an aggressive or unfavorable operating environment, strengthening such connections will be inevitable. Therefore, I propose to use a steel stiffness element as a quarter of the rotation shell instead of the vertical stiffening edges of the shoes.

.

BIBLIOGRAPHY

1. ***Precast Concrete Market Size, Share & Trends Analysis Report By Product Type (Structural, Architectural, Transportation, Water & Waste Handling), By End Use, By Region, And Segment Forecasts, 2020 - 2027.*** **San Francisco : Grand View Research, 2020. p. 80.**

2. Mostert, Louwrens Hubert. ***Design and construction preferences for connections in the precast concrete industry of South Africa.*** **s.l. : Stellenbosch Universit, 2014.**

3. ***EN 1992-1-1:2004 Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for building.***

4. Walraven, Joost. Connections between precast. ***http://www.uct.ac.za/sites/default/files/image_tool/images/333/Events/PP_2013_3_Precast_Concrete_SA.pdf.*** **[Online] 2013.**

5. Holly, Ivan and Abrahoim, Iyad. Connection and joints in precast concrete structures. ***Slovak Journal of Civil Engineering.*** **2020, pp. 49-56.**

6. Riva, Paolo. Advances in Engineering Structures, Mechanics & Construction. [book auth.] M Pandey, Wei-Chau Xie and Lei Xu. ***Springer.*** **Canada : s.n., 2006, pp. 121-128.**

7. ***FIXING OF PRECAST CONCRETE COLUMNS TO FOUNDATIONS.*** **Vimmr, Vaclav and Sharif, Zahra. Prague : s.n., 2011, Concrete engineering for excellence and efficiency, pp. 607-610.**

8. Vimmr, Vaclav. ***ADVANCED CONNECTIONS FOR PRECAST CONCRETE.*** **Prague : s.n., 2020. p. 4.**

9. Santagati, S, et al. Seismic response of columns connected to the foundation through a fastening technique. ***Peikko News.*** **2009, pp. 13-20.**

10. Fagà, Ettore , et al. ***COMPARISON BETWEEN NUMERICAL AND EXPERIMENTAL CYCLIC RESPONSE OF ALTERNATIVE COLUMN TO FOUNDATION CONNECTIONS OF REINFORCED CONCRETEC PRECAST STRUCTURES.*** **s.l. : 3rd fib International Congress, 2010. p. 11.**

11. —. ***COMPARISON BETWEEN NUMERICAL AND EXPERIMENTAL CYCLIC RESPONSE OF ALTERNATIVE COLUMN TO FOUNDATION CONNECTIONS OF REINFORCED CONCRETEC PRECAST STRUCTURES.*** **s.l. : 3rd fib International Congress, 2010. pp. 1-10.**

12. Paolo , Negro and Giandomenico , Toniolo . ***Design Guidelines for Connections of Precast Structures under Seismic Actions.*** **Luxembourg : Joint Research Centre of the European Commission, 2012. p. 84.**

13. Jorma, Kinnunena. ***ETA tests and design of HPKM Column Shoe Connections.*** **s.l. : Elsevier Ltd, 2017. p. 8.**

14. AHK column shoe. User manual. ***https://www.anstar.fi/wp-content/uploads/2020/02/AHK_Column_Shoes_User_Manual_2020.pdf.*** **[Online] 01 2020. [Cited: 05 14, 2020.]**

15. HPKM® Column Shoe. Technical manual. ***https://d76yt12idvq5b.cloudfront.net/file/dl/i/lhtR6A/2P2ddfwoa6hpzpPJFRoYRg/HPKMPeikkoGroup002TMAWeb.pdf.*** **[Online] 05 2014. [Cited: 05 20, 2020.]**

16. OPK-PILARIKENGÄT. Käyttö- ja suunnitteluohjeet / OPK column shoes. Instructions for use and design according to Eurocodes. ***https://semko.fi/wp-content/uploads/2017/04/opk-pilarikenkien-kayttoohje-EC.pdf.*** **[Online] 01 28, 2013. [Cited: 12 10, 2019.]**

17. Technical Manual. RPK-N2, RPK-E2 Column Shoes. ***https://rsteel.fi/wp-content/uploads/2018/12/RSTEEL-RPK-N2-RPK-E2-COLUMNSHOE-instructions-Eurocodes-12.6.2018-1.pdf.*** **[Online] 06 12, 2018. [Cited: 02 13, 2020.]**

18. Halfen HCC Column Shoe type test report. ***https://downloads.halfen.com/catalogues/de/media/typetests/reinforcementsystems/t-hcc-e.pdf.*** **[Online] 06 08, 2009. [Cited: 12 02, 2019.]**

19. Halfen HCC Column Shoe Technical Product Information. ***https://downloads.halfen.com/catalogues/de/media/catalogues/reinforcementsystems/hcc-e.pdf.*** **[Online] 2009. [Cited: 12 02, 2019.]**

20. Typenprüfung PFEIFER-Stützenfülše PCC / Test of the PFEIFER column shoe PCC. ***https://www.pfeifer.info/out/assets/PFEIFER_PCC-TYPP_ZUDE.PDF.*** **[Online] 04 30, 2015. [Cited: 02 13, 2020.]**

21. PFEIFER Column Shoe System PCC. ***https://www.pfeifer.info/out/assets/PFEIFER_COLUMN-SHOE-PCC_PPEN.PDF.*** **[Online] 09 2017. [Cited: 02 13, 2020.]**

22. ***Cyclic behaviour of a column to foundation joint for concrete precast structures.*** **Metelli , Giovanni , Consuelo , Beschi and Paolo , Riva. Paris : European Journal of Environmental and Civil Engineering, 2012, pp. 1298-1318.**

23. ***CAPACITY DESIGN CRITERIA FOR CONNECTIONS IN PRECAST STRUCTURES.*** **Biondini, F, Ferrara, L and Toniolo, G. Beijing : The 14th World Conference on Earthquake Engineering, 2008. p. 10.**

24. ***Adanced connections for precast concrete structures.*** **Vimmr, Vaclav. Tel Aviv : TECHNION, 2013. Engineering a Concrete Future: Technology, Modeling and Costruction. pp. 495-498.**

25. ***Structural Connections for Precast Concrete Buildings.*** **Engström, Björn. Guimarães : s.n., 2008, Encontro Nacional Betão Estrutural, pp. 35-42.**

26. Buratti, Nicola, Bacci, Lorenzo and Mazzotti, Claudio. ***Seismic behavoiur of grouted sleeve connections between foundations and precast columns.*** **Istanbul : Second European conference of Earthquake Engineering and Seismology, 2014. pp. 1-11.**

27. ***EN 1993-1-1:2005 Eurocode 3: Design of steel structures - Part 1-1: General rules.***

28. ***EN 1993-1-8:2005 Eurocode 3: Design of steel structures - Part 1-8: Design of joints.***

A_{st1}	- Vertical stirrups 1 + 1 pc. are placed on both sides of the shoe, and the stirrups transfer the shoe's shear force during the erection state and ultimate limit state in accordance with Section 4.2.2. - However, at least the minimum number of stirrups 1 + 1 pc. according to Table 7 is always placed symmetrically on both sides of the shoe. The number of stirrups increases if the shear force so requires. - For a round column, the stirrups are placed in the diameter direction of the column. - The stirrup anchoring length begins from above the shoe housing.
A_{st2}	- The stirrups bind the horizontal forces caused by the eccentric axial force of the shoe. - The stirrups are placed immediately above the shoe housing as a bundle. Table 7. - Circular stirrups are placed above the shoe for a round column and closed stirrups for a rectangular column.
A_{st3}	- The stirrups are located at the bottom and top ends of the shoe bonds according to EN 1992-1-1, Section 8.7.3.1. - The stirrups are needed when the shoe's bond or column's main piece of rebar is in the shoe area ≥ T20. - The distance between stirrups/location area is ≤ 150 mm. - The number of stirrups/location area = A_{st3}, which has been calculated according to the size of the shoe's bond.

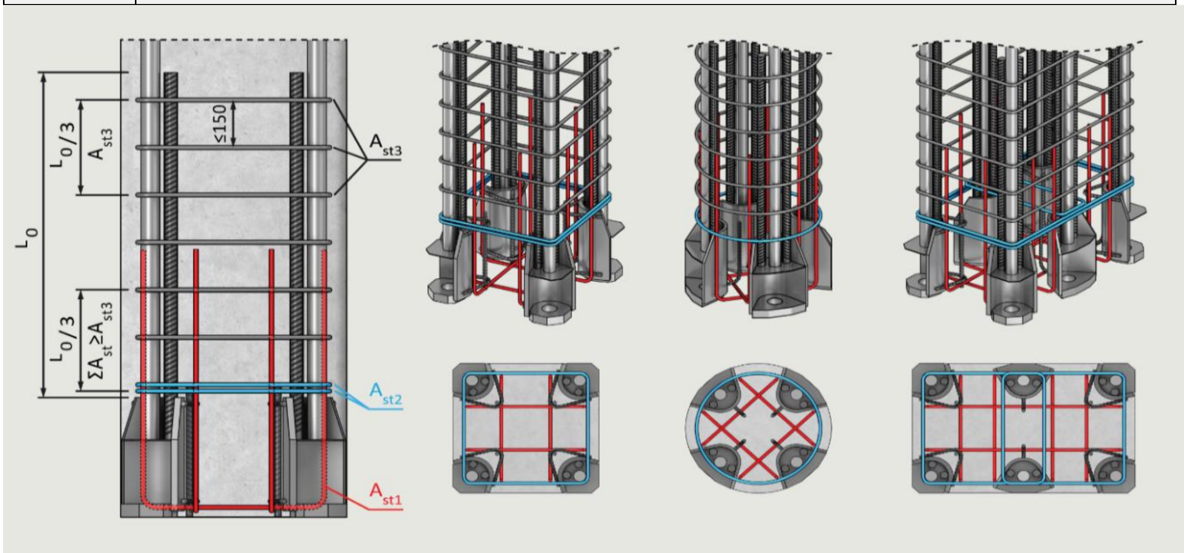


Figure 22. Supplementary reinforcement for AHK and AHK-K shoes.

1. General	- Table 7 presents the supplementary reinforcement for the shoe connection as calculated for the shoe's resistance values. - At least these stirrups must be placed in the column. - If the connection includes shoes of different sizes, the number of stirrups is determined according to the largest shoe. - The stirrup size can be changed and adapted to the other stirrup reinforcement of the column.

Table 7. Supplementary reinforcement for shoes with the shoe design values

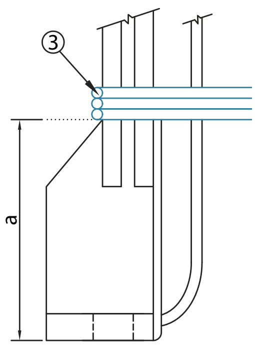
Shoe	A_{st1} T	A_{st2} T	A_{st3} mm^2	Shoe	A_{st1} T	A_{st2} T	A_{st3} mm^2
AHK16	2T8	T8	–	AHK16K	2T8	T8	–
AHK20	2T8	T8	–	AHK20K	2T8	T8	–
AHK24	2T8	2T8	–	AHK24K	2T8	T8	–
AHK30	2T8	2T8	157	AHK30K	2T8	T8	157
AHK36	2T10	3T8	245	AHK36K	2T10	T8	245
AHK39	2T10	2T10	245	AHK39K	2T10	2T8	245
AHK45	2T12	3T10	308	AHK45K	2T12	2T8	308

Annex A – Transverse reinforcement in the lap zone and supplementary reinforcement

Details of transverse reinforcement in the lap zone and supplementary reinforcement for HPKM Column Shoes are shown in following figures. Required quantities and lengths of stirrups are given in the Table 7.

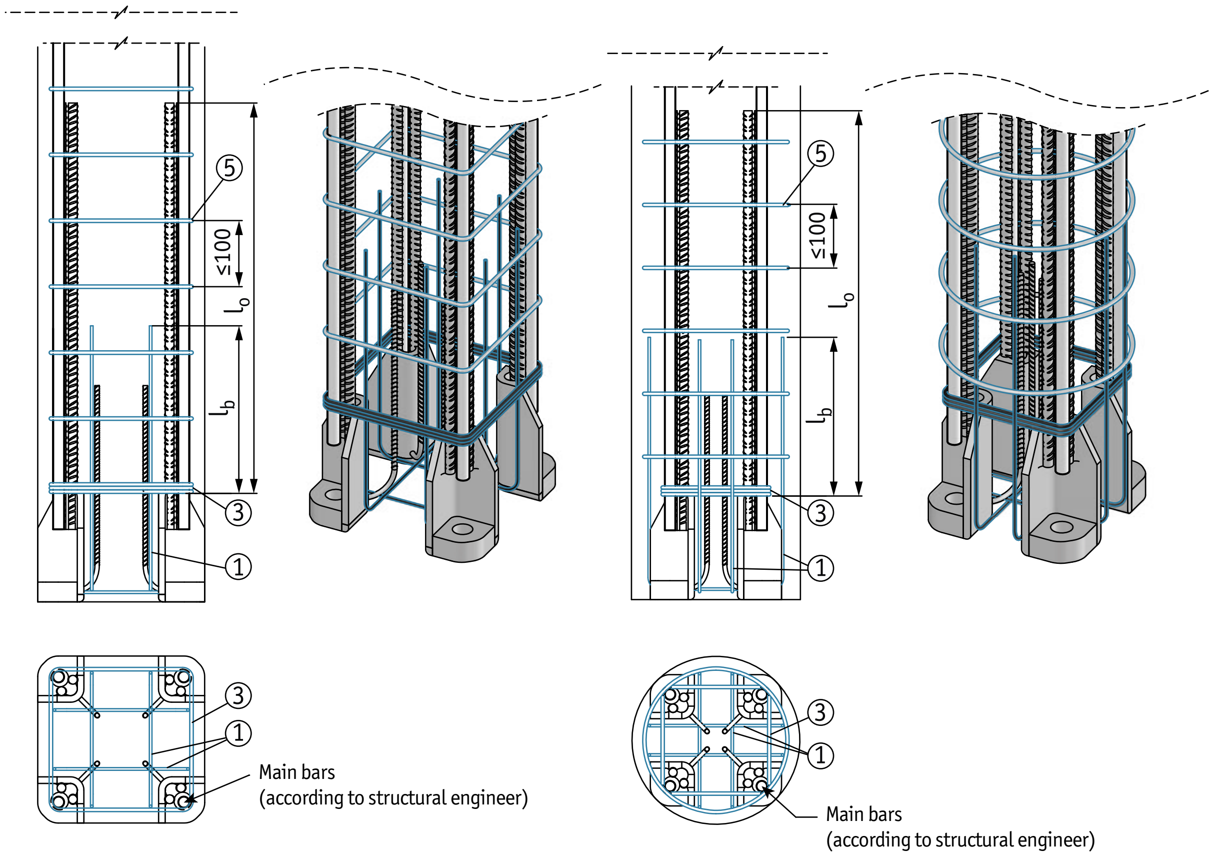
Table 7. Transverse reinforcement in the lap zone and supplementary reinforcement (B500B).

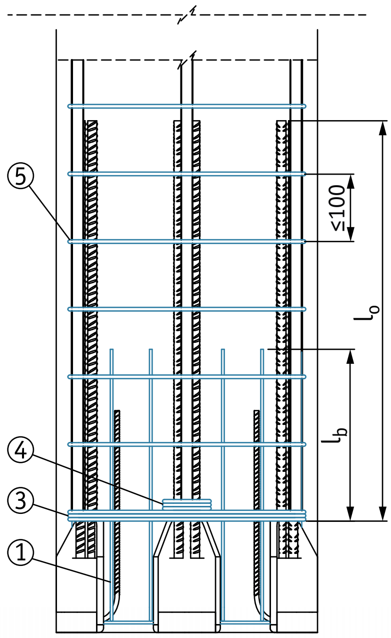
	HPKM 16	HPKM 20	HPKM 24	HPKM 30	HPKM 39
U-stirrup (1)	4 Ø 6	4 Ø 6	4 Ø 6	4 Ø 6	4 Ø 6
U-stirrup (2)	2 Ø 6	2 Ø 6	2 Ø 6	2 Ø 6	2 Ø 6
Stirrup (3)	2 Ø 8	2 Ø 8	3 Ø 8	3 Ø 8	3 Ø 10
Stirrup (4)	2 Ø 8	2 Ø 8	3 Ø 8	3 Ø 8	3 Ø 10
Stirrup (5)	Ø 8	Ø 8	Ø 8	Ø 8	Ø 10
a	140	165	200	250	300
l_b	300	300	300	300	300

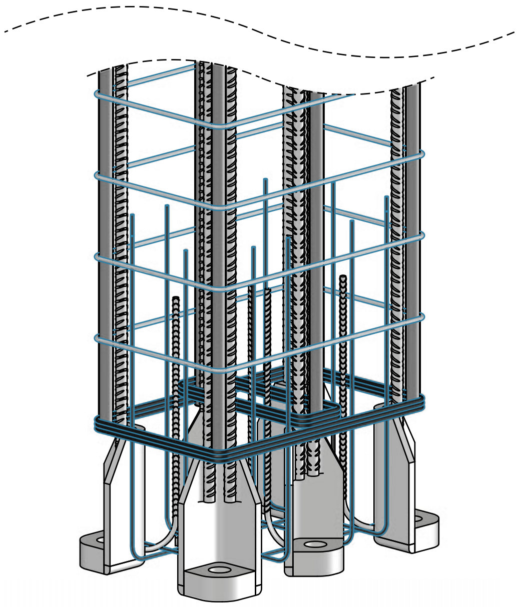


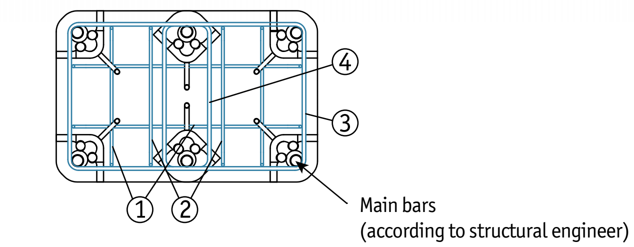
Recommended spacing ≤100 mm of transverse reinforcement (5) in the lap zone l_0

Figure 8. Transverse and supplementary reinforcement needed for HPKM Column Shoes (HPKM 30 shown in the pictures).



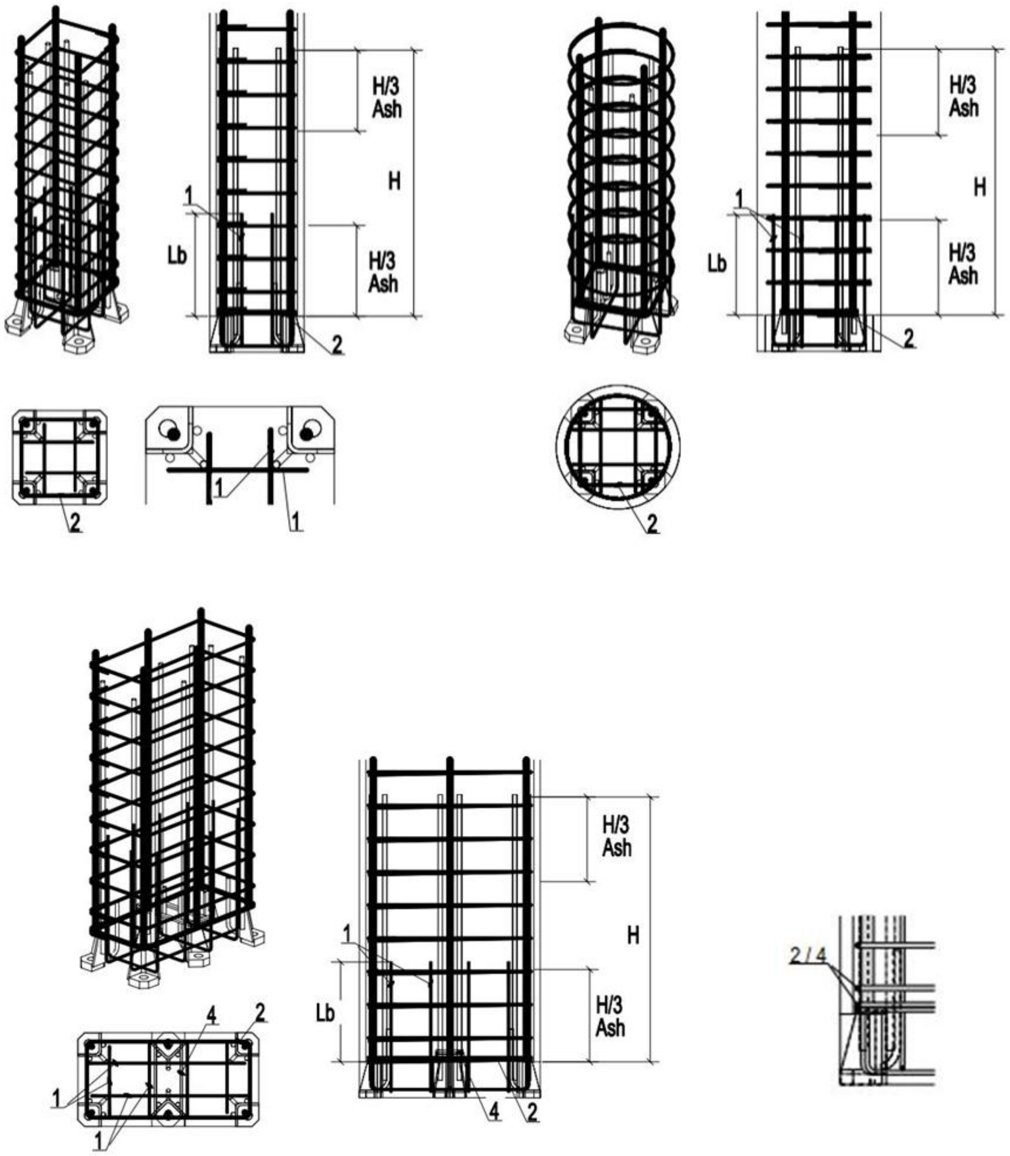

5
≤100
l_0
l_b
4
3
1




4
3
1
2
Main bars
(according to structural engineer)

5.3 Column Reinforcement Instructions

The reinforcement to the column adjacent to the column shoes is to be in accordance with instructions provided in EC 2, as well as the following details.



Additional Reinforcement for the RPK-N2 shoe:

RPK-N2 Shoe	Links					Dim.		Links	Main Rebar
	1 (vert.links)			2 (hor.)	4 (hor.)	H	H/3	A_{sh}	D_{max}
	pcs		Lb						
M16	min 4	T6	500	2 T8	2 T8	480	160	2 T8	20
M20	min 4	T6	500	2 T8	2 T8	750	250	2 T8	20
M24	min 4	T6	500	3 T8	3 T8	1100	350	3 T8	25
M30	min 4	T6	500	3 T8	3 T8	1300	430	6 T8	32
M39	min 4	T6	500	3 T10	3 T10	1800	600	6 T8	32

Within the zone defined by H, the maximum recommended spacing of shear links according to SFS-EN 1992-1-1 is 100mm.

Columns where centrally positioned shoes are used:

Link "1": add 2 pcs. no. "1" links per shoe pair (1no. to each side of each pair).

Link "4": add no. "4" links around each shoe pair. Required no. of links according to the table.

Additional Reinforcement for the RPK-E2 shoe:

RPK-E2 Shoe	Links					Dim.		Links	Main Rebar
	1 (vert.links)			2 (hor.)	4 (hor.)	H	H/3	A_{sh}	D_{max}
	pcs		Lb						
M30	min 4	T6	500	2+2 T8	2+2 T8	1400	370	3 T8	25
M36	min 4	T8	500	2+1 T10	2+1 T10	1400	470	6 T8	32
M39	min 4	T10	600	2+2 T10	2+2 T10	1650	550	6 T8	32
M45	min 4	T10	600	3+2 T12	3+2 T12	1800	600	6 T8	32
M52	min 4	T10	600	3+2 T12	3+2 T12	2600	870	6 T8	32

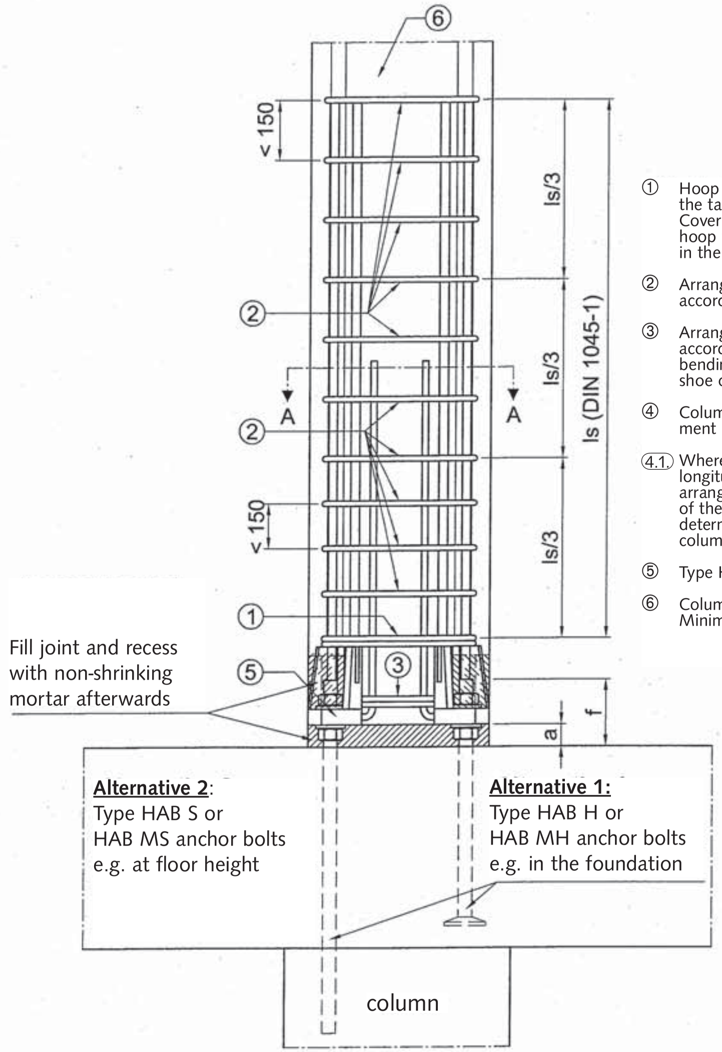
Within the zone defined by H, the maximum recommended spacing of shear links according to SFS-EN 1992-1-1 is 100mm.

Columns where centrally positioned shoes are used:

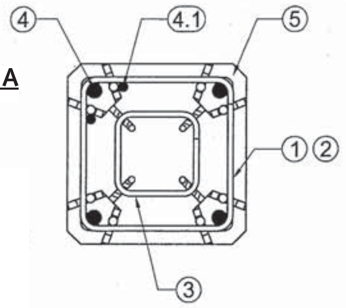
Link "1": add 2 pcs. no. "1" links per shoe pair (1no. to each side of each pair).

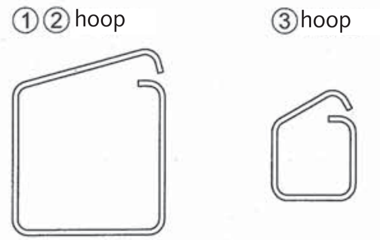
Link "4": add no. "4" links around each shoe pair. Required no. of links according to the table.

...1...Fertigung



Section A – A





① Hoop for column shoe according to the table flush with upper edge. Cover of 30 mm present for the hoop when fitting the column shoe in the corner.

② Arrange hoop in the corner in accordance with DIN 1045-1:2001-07

③ Arrange hoop for column shoe in accordance with the table, in the bending radius of the bent column-shoe concrete reinforcing steel.

④ Columns – longitudinal reinforce ment in the corner

4.1 Where necessary, additional longitudinal reinforcement can also be arranged outside the cross section of the shoe. The client must determine the lapped splice with the column-shoe reinforcement.

⑤ Type HCC column shoe

⑥ Column concrete
Minimum strength class C30/37

Column shoe	Anchor bolt	Joint height a [mm]	Bolt protrusion f [mm]	①	②	③
HCC 16	HAB H16/ HAB S16	50	105	1 Ø 8	Ø 8	1 Ø 8
HCC 20	HAB H20/ HAB S20	50	115	1 Ø 10	Ø 10	1 Ø 10
HCC 24	HAB H24/ HAB S24	50	130	1 Ø 10	Ø 10	1 Ø 10
HCC 30	HAB H30/ HAB S30	50	150	1 Ø 12	Ø 12	1 Ø 12
HCC 39	HAB H39/ HAB S39	50	165	2 Ø 12	Ø 12	2 Ø 12
HCC M30	HAB MH36/ HAB MS36 *)	50	165	2 Ø 10	Ø 10	2 Ø 10
HCC M36	HAB MH36/ HAB MS36	55	165	2 Ø 12	Ø 12	2 Ø 12
HCC M39	HAB MH39/ HAB MS39	55	185	2 Ø 12	Ø 12	2 Ø 12
HCC M45	HAB MH45/ HAB MS45	65	195	3 Ø 12	Ø 12	3 Ø 12
HCC M52	HAB MH52/ HAB MS52	70	240	3 Ø 12	Ø 12	3 Ø 12

*) Bolzen Typ HAB MH30/ HAB MS30 sind in Vorbereitung

Type tested for construction application

Test number; 03/30

Landesstelle für Bautechnik Baden-Württemberg

Tübingen, dated. 8th June 2009

Processor (signature)

Stamp Next endorsement by the federal office of construction required by 15th June 2014	Stamp Municipality steering commitee Tübingen

HALFEN

Fitting instructions
Halfen column shoe type HCC

HALFEN GmbH
Liebigstr. 14 D-40764 Langenfeld
Phone +49-(0) 2173-970 (0) • Fax +49-(0) 2173-970-123

Dimensioning

Table 2 – design resistances of Column Shoes PCC

Type	N_{Rd} [mm]	Associated Bolt	Possible horizontal tolerance [mm]
PCC 16	± 68	M 16	± 6
PCC 20	± 97	M 20	± 5
PCC 24	± 139	M 24	± 5,5
PCC 30-1	± 220	M 30	± 5,0
PCC 30-2	± 299	M 30	± 7,5
PCC 36	± 436	M 36	± 8,5
PGM/PSF 42	on enquiry		
PGM/PSF 48	on enquiry		
PGM/PSF 56	on enquiry		

Prerequisites:

Column:
- Concrete grade ≥ C 30/37, good bonding conditions
- Additional reinforcement according to section "Reinforcement"
- Standard reinforcement from column dimensioning

Foundation:
- Concrete grade ≥ C 20/25, good bonding conditions
- Additional reinforcement according to approval/standard
- Standard reinforcement from foundation dimensioning

Column reinforcement layout

The Column Shoes PCC are integrated in the column reinforcement. The two front reinforcing bars form an overlapping joint with the longitudinal reinforcement of the column. The transverse reinforcement in the region of the overlapping joints between the main anchoring bars of the Column Shoes PCC and the respective longitudinal reinforcement of the column is not part of this description. The proofs are to be provided in the individual case by the responsible planner within the context of the static calculation of the precast elements according to the applicable standard. The reinforcing steel stirrups, pos. 1/2 shown in figures 10-12 are intended for the absorption of regular tensile forces arising from tensile and compressive stresses acting on the Column Shoes PCC and must be installed without fail.

Table 3

Type	Size	Pos.1/2 [cm²]	Overlap length [mm]
PCC	16	0,13	650
PCC	20	0,19	800
PCC	24	0,29	1000
PCC	30-1	0,64	1260
PCC	30-2	0,90	1360
PCC	36	0,97	1780

Figure 10

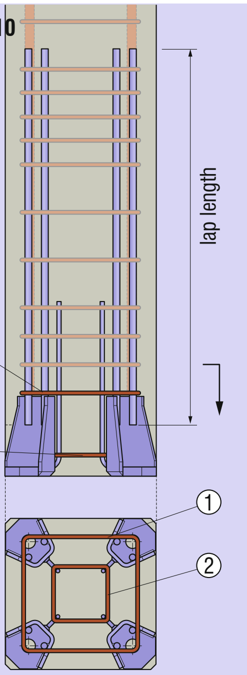


Figure 11

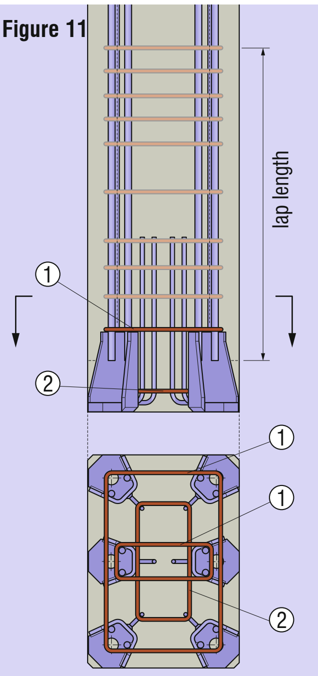
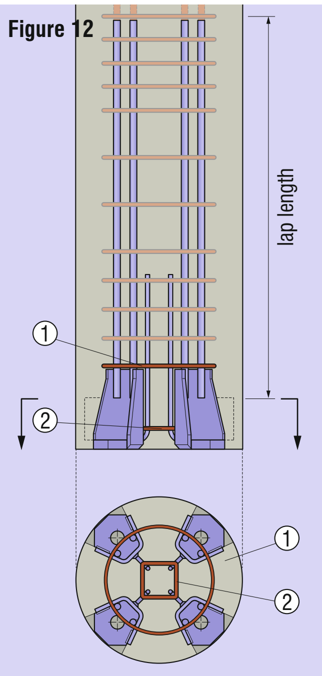


Figure 12



The definition of the overlapping length of the main vertical reinforcement bars is based on DIN EN 1992-1-1, paragraph 8.4 respectively 8.7.
According to the definition of the material the minimum concrete grade of the precast elements using column shoes PCC: C30/37. Further conditions are the assumption that the column shoes are installed within the framework of a plant production into bar-shaped concrete elements (e.g. columns), maximum section sizes of 500mm and the usage of common external/surface vibrators for compacting. In this case based on DIN EN 1992-1-1/NA, NCI 8.4.2 good bonding conditions can be applied.
The dimensioning of the transverse reinforcement in the area of overlapping between the main vertical reinforcement bars of the column shoes PCC and the particular longitudinal reinforcement of the column is not part of this type calculation. Appropriated verifications must be done in each individual case within the static calculation of the precast elements.
The overlapping length is the whole length of the main vertical reinforcement bars which are welded to the column shoes with the comparatively short length of the welds themselves.

Foundation reinforcement layout

Figure 13

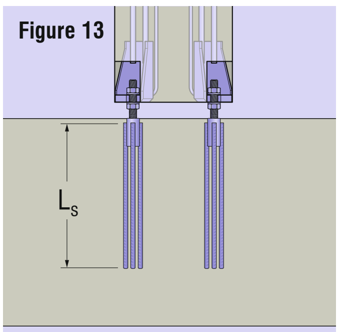
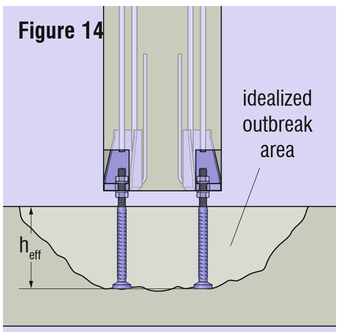


Figure 14



When laying out the foundation reinforcement in relation to the foundation anchors, one of two variants must be chosen:

1) Anchorage via end anchorage or overlapping length as per figure 13
   – Proofs are to be provided in this case by the responsible planner according to the applicable standard. The measure l_{bd} is calculated L_s in accordance with Product table minus 2x $\varnothing_s$.
2) Anchorage via anchor foot/Typ: G1-K und G1-DK as per figure 14
   – The anchorage proofs are to be provided in this case by the responsible planner according to the applicable approvals.

Minimum distances between Column Shoes PCC

Figure 15

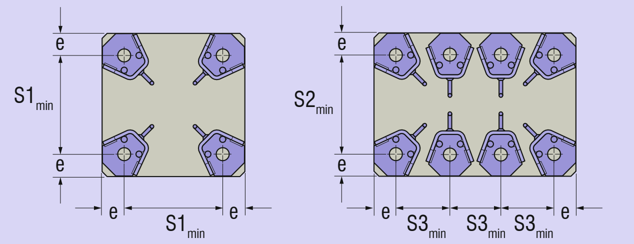
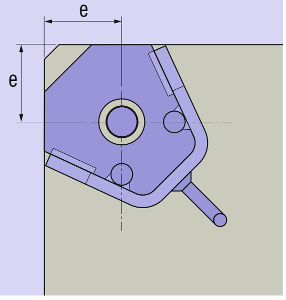


Figure 16



The minimum distances given in figure 15 and Table 3 are defined on the basis of the column shoe geometries. Other distances may be relevant within the context of the anchor dimensioning and the distances necessary there.

Notice: The minimum distances given in Table 3 are to be applied analogously in the case of round columns.

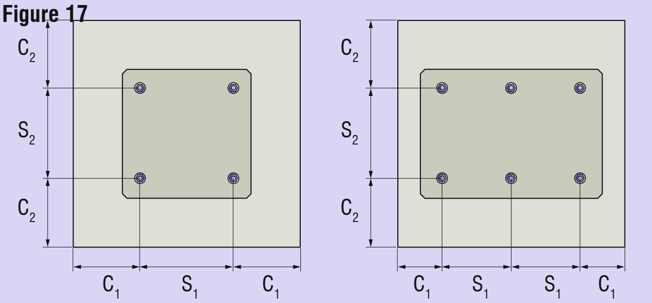
Table 3 – minimum distances

Type/Size	e [mm]	S_{1min} [mm]	S_{2min} [mm]	S_{3min} [mm]
PCC 16	50	145	190	105
PCC 20	50	155	205	120
PCC 24	50	180	240	125
PCC 30-1	50	220	295	160
PCC 30-2	50	265	355	160
PCC 36	60	275	370	175

Notice: Minimum distances in Table 3 (column shoes PCC) and 4 (foundation anchors G1-K and G1-DK) should be checked for relevance.

Minimum distances between foundation anchors PGS

Figure 17



Notice: the mentioned minimum distances do not provide any information about the load capacity as far as the verification of the concrete failure is concerned. This verification always must be done separately.

Notice: In the case of the anchor variants PGS-H and PGS-G, anchorage takes place via an overlapping joint or an end anchorage in accordance with the applicable standard. The applicable constructive rules of the standard must be adhered to when installing the anchors.

Table 4 – minimum distances for Foundation Anchors G1-K and G1-DK

Type/Size	C_{1min}/C_{2min} [mm]	S_{1min}/S_{2min} [mm]
PGS 16/G1-DK	50	90
PGS 20/G1-DK	55	100
PGS 24/G1-DK	60	110
PGS 30/G1-DK	73	135
PGS 36/G1-DK	80	150
PGS 42/G1-DK	88	165
PGS 48/G1-DK	95	180
PGS 56/G1-DK	105	200
PGS 16/G1-K	50	80
PGS 20/G1-K	70	100
PGS 24/G1-K	70	100
PGS 30/G1-K	100	130
PGS 36/G1-K	130	150

Shear force

The proof of shear forces is not part of the type testing of the column shoe system. If shear forces exist, they must be proven separately in the individual case. The proofs can be rendered, for example, using a concrete ring, a shear key or by friction according to DIN EN 1992-1-1. Figures 18-20 show a few possible solutions.

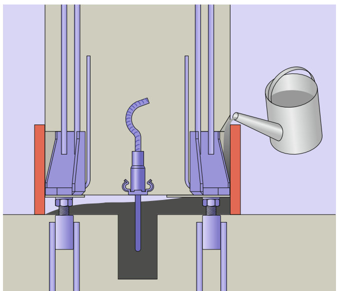


Figure 18

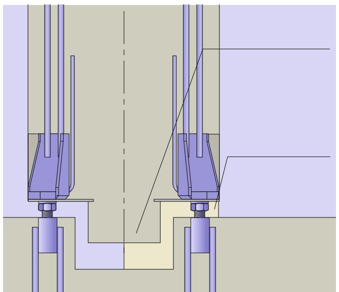


Figure 19

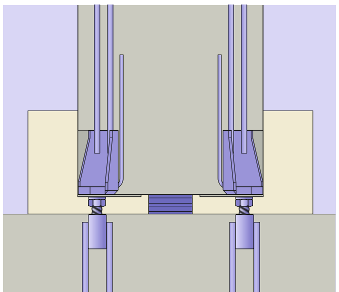
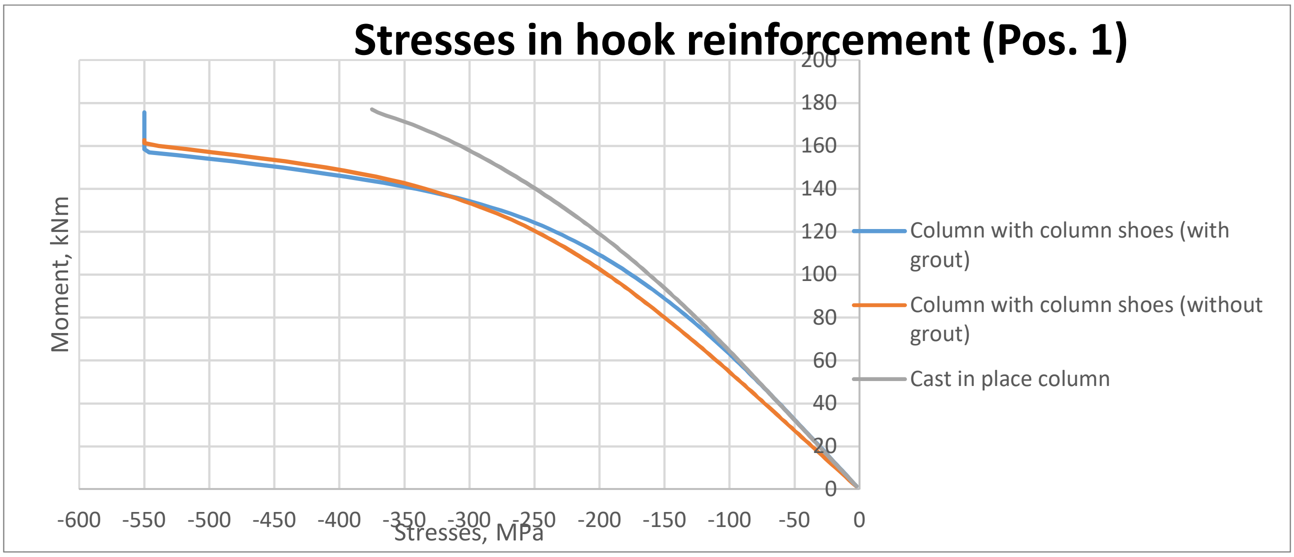
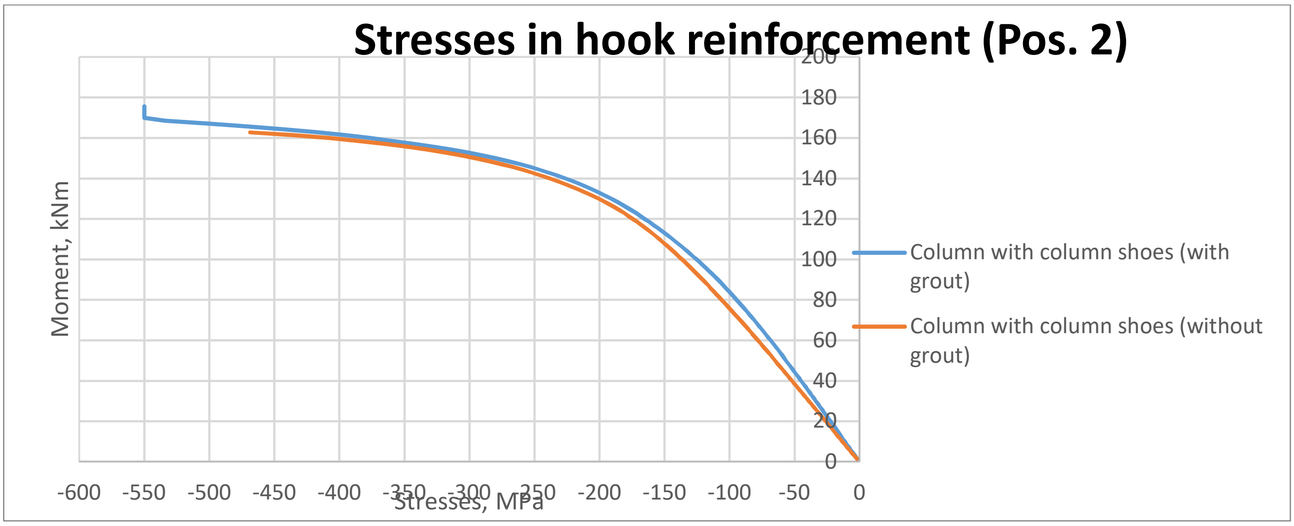
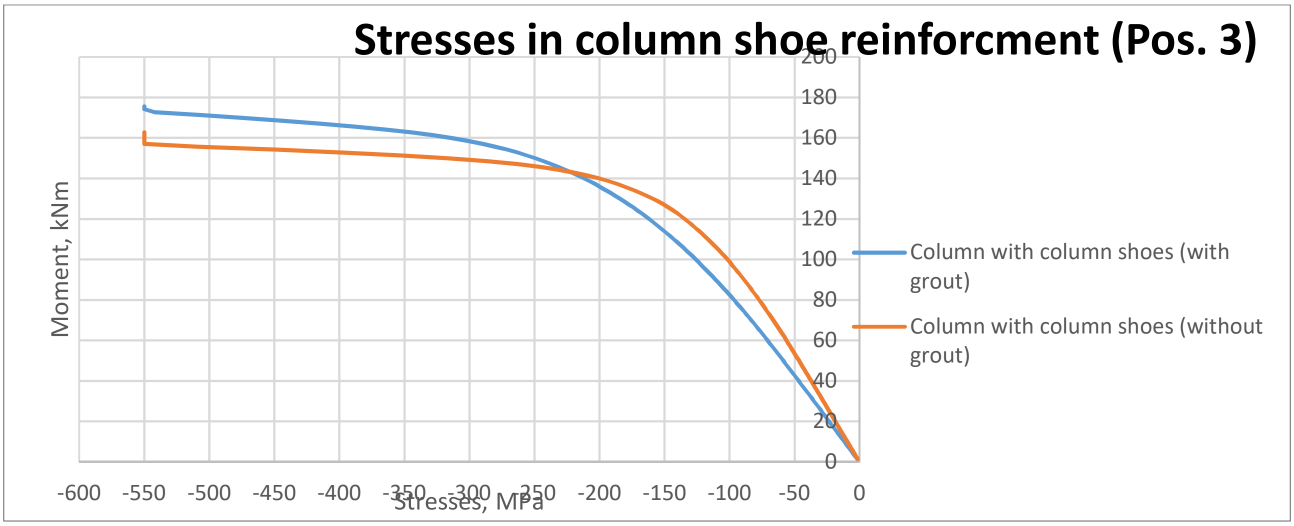
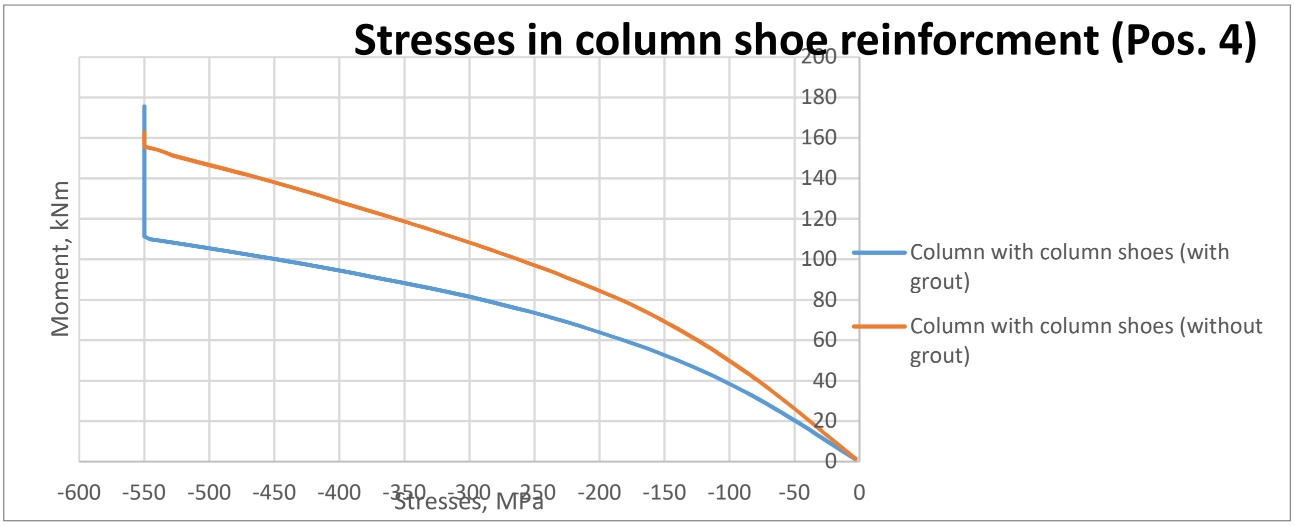


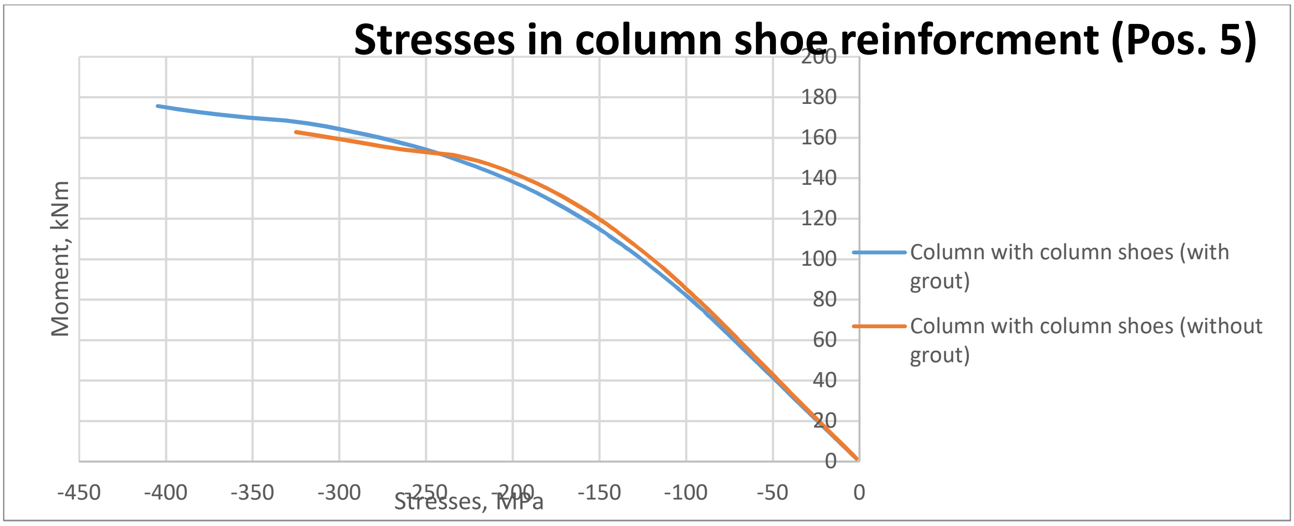
Figure 20

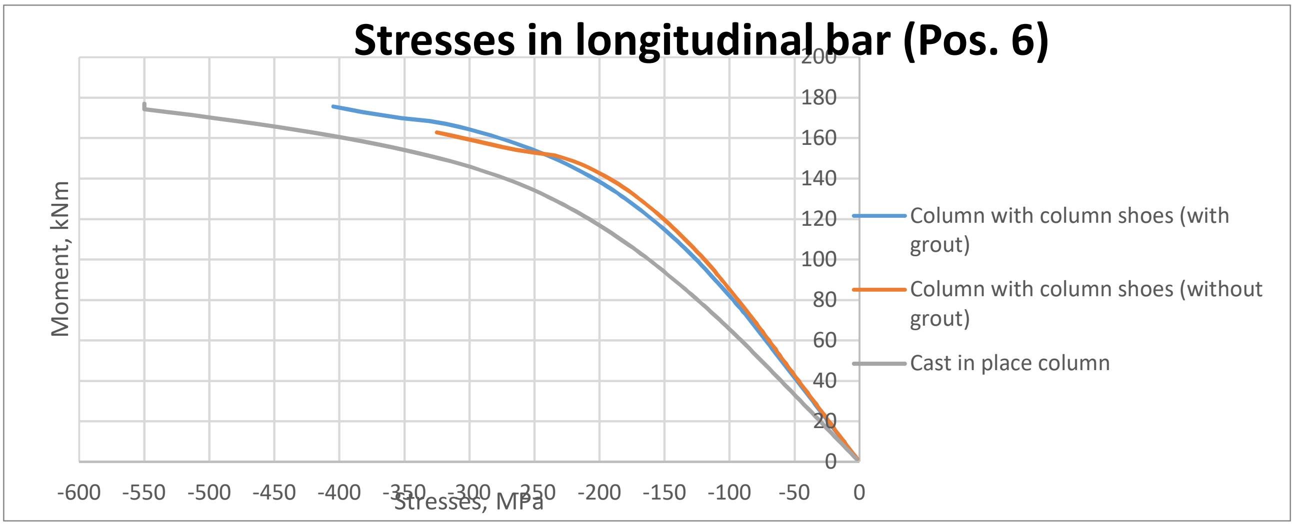

Stresses in hook reinforcement (Pos. 1)
Moment, kNm
Stresses, MPa
200
180
160
140
120
100
80
60
40
20
0
-600
-550
-500
-450
-400
-350
-300
-250
-200
-150
-100
-50
0
Column with column shoes (with grout)
Column with column shoes (without grout)
Cast in place column

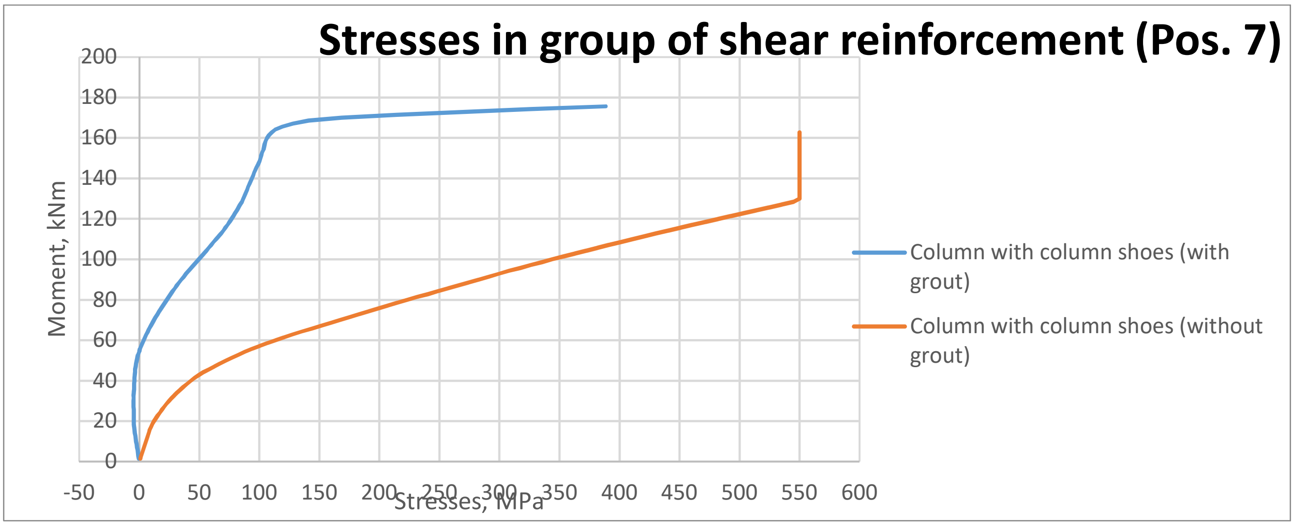

Stresses in hook reinforcement (Pos. 2)
Moment, kNm
Stresses, MPa
200
180
160
140
120
100
80
60
40
20
0
-600
-550
-500
-450
-400
-350
-300
-250
-200
-150
-100
-50
0
Column with column shoes (with grout)
Column with column shoes (without grout)

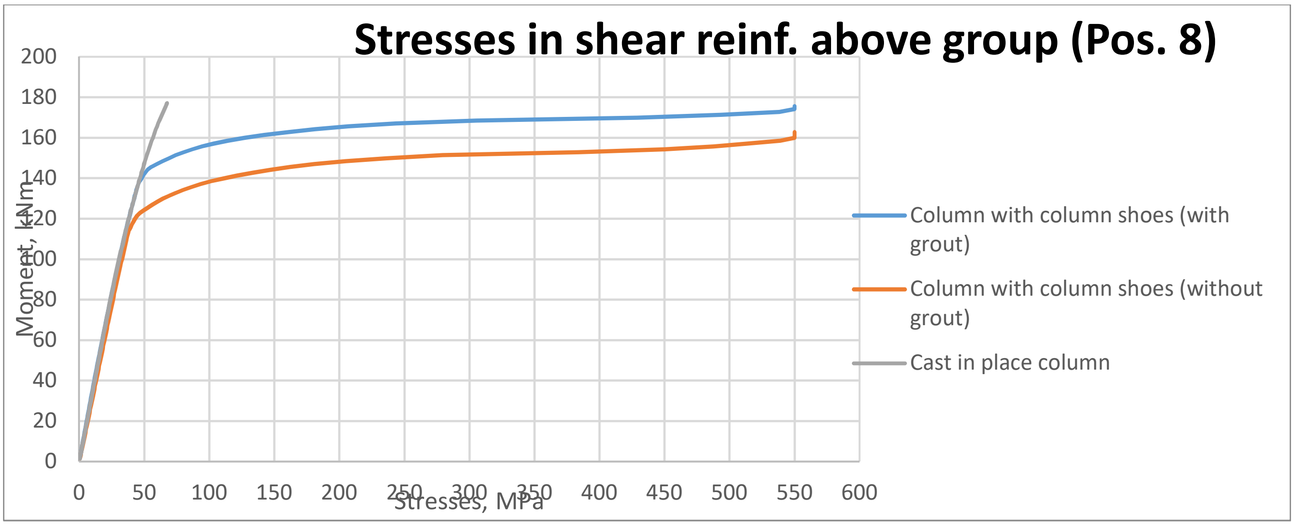

Stresses in column shoe reinforcment (Pos. 3)
Moment, kNm
Stresses, MPa
200
180
160
140
120
100
80
60
40
20
0
-600
-550
-500
-450
-400
-350
-300
-250
-200
-150
-100
-50
0
Column with column shoes (with grout)
Column with column shoes (without grout)

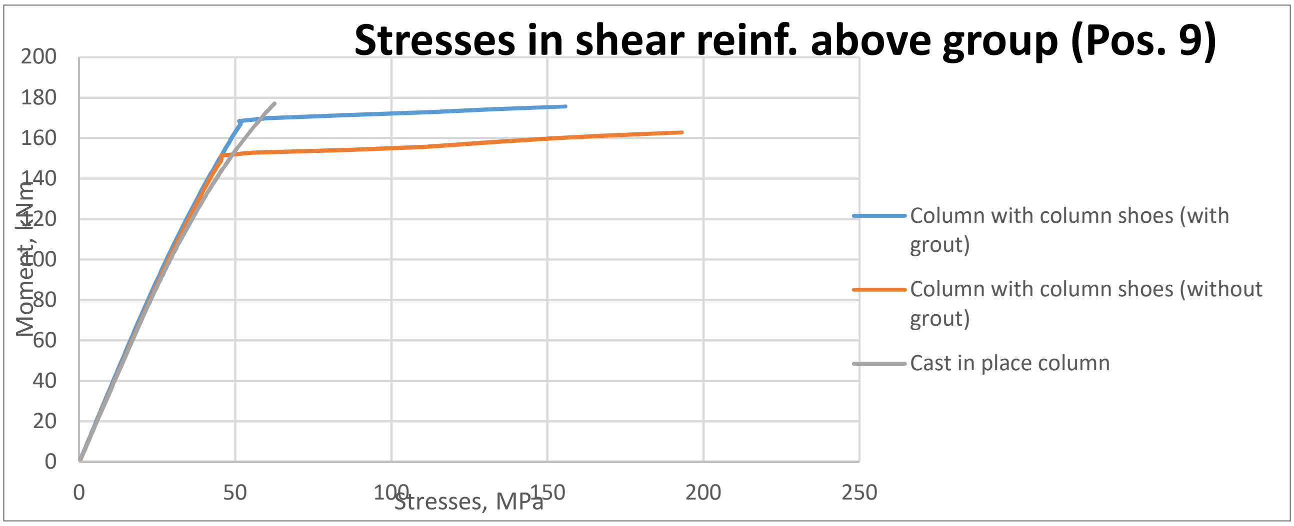

Stresses in column shoe reinforcment (Pos. 4)
Moment, kNm
Stresses, MPa
200
180
160
140
120
100
80
60
40
20
0
-600
-550
-500
-450
-400
-350
-300
-250
-200
-150
-100
-50
0
Column with column shoes (with grout)
Column with column shoes (without grout)

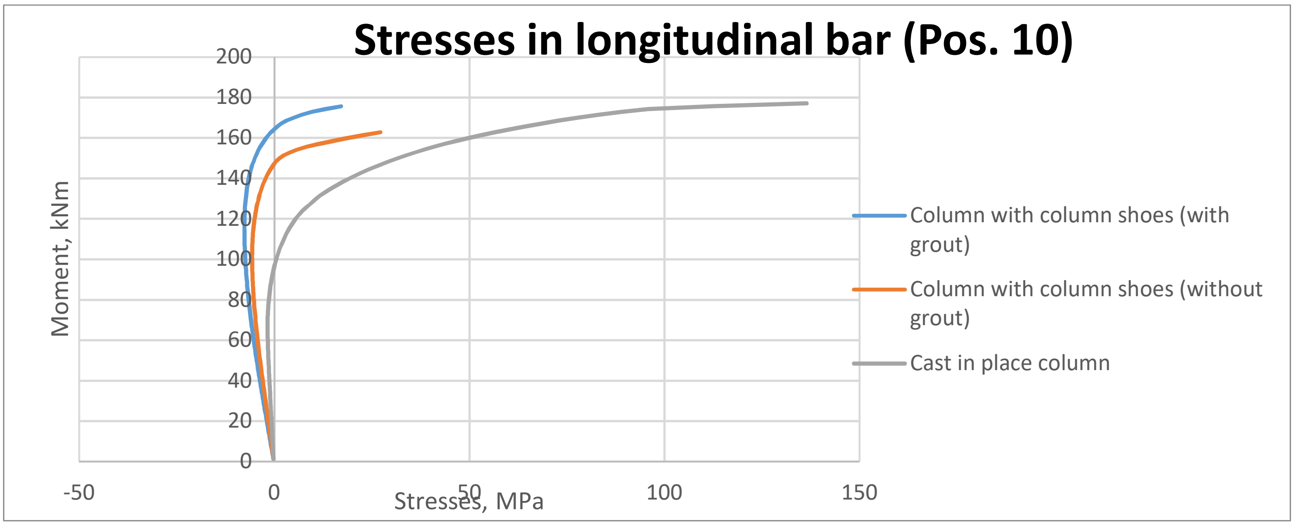

Stresses in column shoe reinforcment (Pos. 5)
Moment, kNm
Stresses, MPa
200
180
160
140
120
100
80
60
40
20
0
-450
-400
-350
-300
-250
-200
-150
-100
-50
0
Column with column shoes (with grout)
Column with column shoes (without grout)

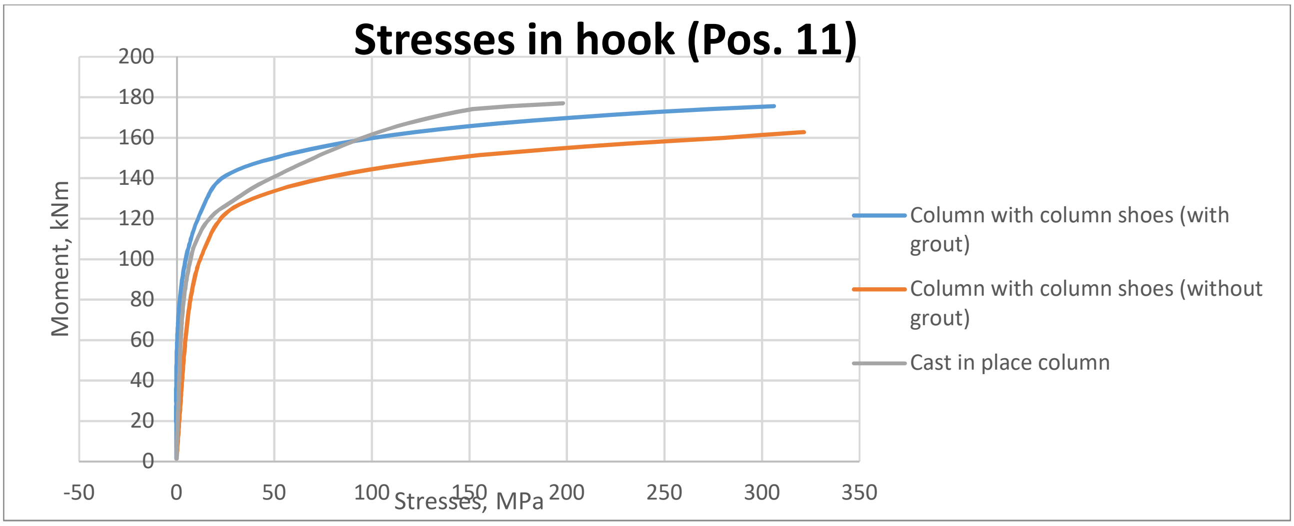

Stresses in longitudinal bar (Pos. 6)
Moment, kNm
Stresses, MPa
200
180
160
140
120
100
80
60
40
20
0
-600
-550
-500
-450
-400
-350
-300
-250
-200
-150
-100
-50
0
Column with column shoes (with grout)
Column with column shoes (without grout)
Cast in place column

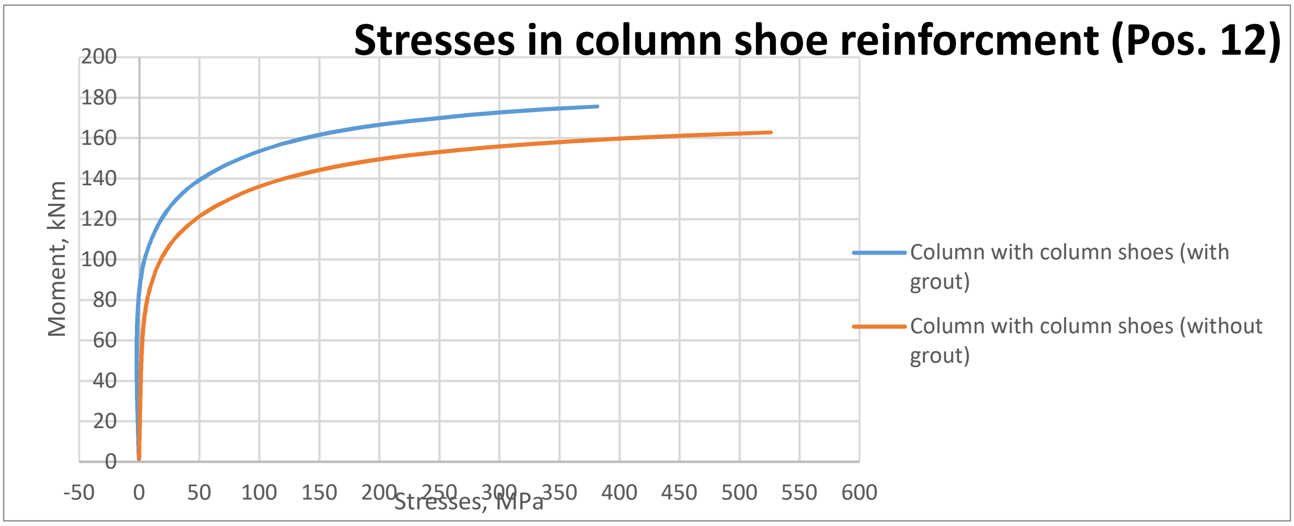

Stresses in group of shear reinforcement (Pos. 7)
Moment, kNm
200
180
160
140
120
100
80
60
40
20
0
-50
0
50
100
150
200
250
300
350
400
450
500
550
600
Stresses, MPa
Column with column shoes (with grout)
Column with column shoes (without grout)


Stresses in shear reinf. above group (Pos. 8)
Moment, kNm
200
180
160
140
120
100
80
60
40
20
0
0
50
100
150
200
250
300
350
400
450
500
550
600
Stresses, MPa
Column with column shoes (with grout)
Column with column shoes (without grout)
Cast in place column


Stresses in shear reinf. above group (Pos. 9)
Moment, kNm
200
180
160
140
120
100
80
60
40
20
0
0
50
100
150
200
250
Stresses, MPa
Column with column shoes (with grout)
Column with column shoes (without grout)
Cast in place column


Stresses in longitudinal bar (Pos. 10)
Moment, kNm
200
180
160
140
120
100
80
60
40
20
0
-50
0
50
100
150
Stresses, MPa
Column with column shoes (with grout)
Column with column shoes (without grout)
Cast in place column


Stresses in hook (Pos. 11)
Moment, kNm
200
180
160
140
120
100
80
60
40
20
0
-50
0
50
100
150
200
250
300
350
Stresses, MPa
Column with column shoes (with grout)
Column with column shoes (without grout)
Cast in place column


Stresses in column shoe reinforcment (Pos. 12)
Moment, kNm
200
180
160
140
120
100
80
60
40
20
0
-50
0
50
100
150
200
250
300
350
400
450
500
550
600
Stresses, MPa
Column with column shoes (with grout)
Column with column shoes (without grout)