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Structural Design and Technology of Pocket Foundations for Long Precast Concrete Columns in Seismic Areas

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Abstract: The connection between a prefabricated reinforced concrete column and a pocket foundation is a case treated from a general perspective in the European Standard named EN 1992-1-1, and when the structural engineer deals with the dimensioning or verification of the connection, he tackles several unknowns. The present work aims to fill in the missing information by presenting detailed calculation models based on the strut-and-tie method for 4 of the widely used pocket foundations: pedestal pocket foundation with smooth, rough or keyed internal walls, and pad foundation with pocket possessing keyed internal walls. Additionally, detailed design prescriptions applicable to seismic areas are given. Afterwards, as a case study, a pocket foundation is designed in all 4 variations, having the structural design particularities, similitudes and differences pointed out. Finally, to conclude the advantages and disadvantages are mentioned for pocket foundations in respect to the type of internal wall surface used. Quantifiable data based on the case study undertaken is available.

Keywords: prefabricated pocket foundation; strut-and-tie; precast concrete column; structural connection; keyed surface; rough surface; smooth surface; socket connection between footing and prefabricated column

1. Introduction

The precast concrete column – foundation joint must be able to transfer axial force, bending moment and shear force from column to foundation [1]. Isolated footings for prefabricated reinforced concrete columns can be designed as pocket foundations (Figure 1) or as pad foundations (with anchor bolts, starter rebars etc.) made of reinforced concrete [2].

Pocket foundations can have smooth, rough or keyed internal walls, they can be designed with a socket (pedestal pocket foundation) or with a deep inner pocket (pad foundation with pocket), while the dimensions of the pocket interior must be sufficient to properly fill the space between the column and the foundation inner walls with fresh concrete [1–4]. For pocket foundations with keyed internal walls (Figure 2), if the geometry of the castellation fulfills the requirements given in [1–3] then the precast column – pocket foundation joint is considered to exhibit a structural response similar to a monolithic column – foundation. Moreover, for pocket foundations with smooth or rough internal walls, the transfer of the internal forces from column to foundation is considered to take place through normal pressure and friction [1–3]. When the column can be subjected to tensile axial forces, only the use of pocket foundations with keyed internal walls is allowed [2] (par. II.6.2.2).

Reinforced concrete pad foundations without pockets can be simple, slopped or have stepped footings [2]. The transfer of bending moments and tensile axial forces from the precast concrete column towards the foundation is carried out through anchor bolts or starter rebars, while shear force is transferred through shear dowels [2,5–8].

Foundations can be made of monolithic reinforced concrete, partially monolithic (monolithic and prefabricated collar) or fully prefabricated concrete [5,9]. Entirely prefabricated foundations are rare because of their size (they become oversized when their size in the horizontal plane exceeds 2,5x2,5 m) and the difficulties of placing them in a perfectly horizontal position, with contact between the entire bottom surface and the ground [9–11].

In earthquake resistant structures where the seismic energy is dissipated through formation of plastic hinges at the base of the columns, the column-foundation joints must be designed in such a way to allow yielding of the column`s longitudinal reinforcement without degradation and reduction of the load-bearing capacity in the joint region [5,6,12].

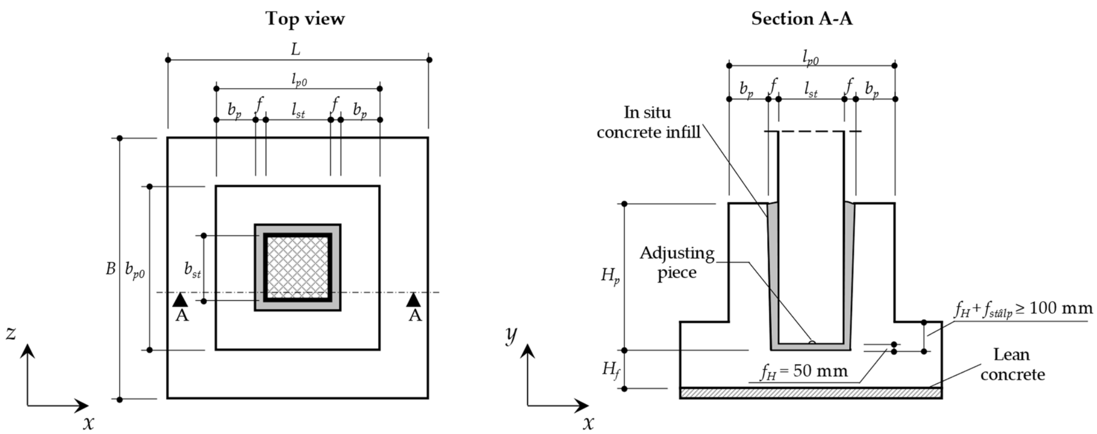

Figure 1. Precast concrete column – pedestal pocket foundation joint. Adapted from [2].

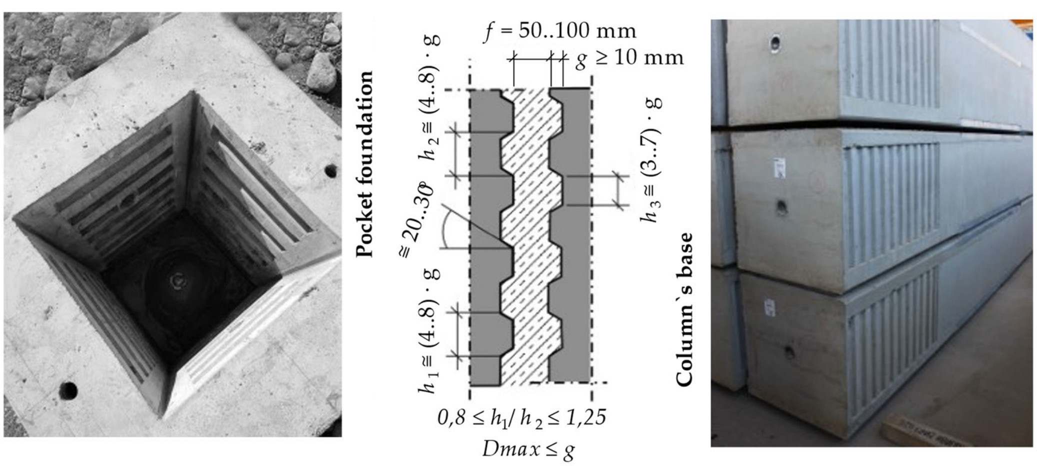

Figure 2. The precast socket of a pocket foundation with keyed internal walls (left), the geometry of the castellation (center) considering the most restrictive indications stated in [1–3,13–19] and the base of the prefabricated concrete columns stored in a stack (right).

The types of foundations for prefabricated concrete columns, in which the dimensioning of the column-foundation joint interfaces are considered are:

- pedestal pocket foundation with smooth internal walls;
- pedestal pocket foundation with rough internal walls;
- pedestal pocket foundation with keyed internal walls;
- pad foundation with pocket and keyed internal walls.

The criteria to be considered when choosing the prefabricated column-foundation connection are [2,5,20–22]:

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- level at which good soil is located (TBF) in relation to the natural land level (CTN), respectively in relation to the developed land level (CTA), and implicitly compared to the ±0.00 level of the future construction;
- nature of the soil and execution technology used for infrastructure;
- groundwater level (NAS);
- the reaction forces transmitted by the superstructure and their magnitude;
- owner`s requirements;
- quality control in conjunction with building costs.

The precast concrete column - pocket foundation keyed connection (Figure 2) can be considered in the design calculations as a monolithic joint between a concrete column and a foundation [1,2]. As a condition for the punching shear, the dimensioning is to be carried out in the assumption of a monolithic column-foundation, thus it is necessary to verify the shear transfer between the column and the footing [1,2,23].

2. Materials and Design Method

The steps in the structural design of the precast column – pocket foundation connection could be stated as:

- choosing the materials used (concrete classes, types of reinforcement) and the type of foundation;
- choosing the technology for mounting the column in the pocket foundation;
- pre-dimensioning of the pocket foundation (height of the pocket H_P, wall thickness b_P, concrete infill thickness f, f_H and $f_{stâlp}$);
- dimensioning of the pocket foundation (checking the concrete cross-section, calculating the amount of reinforcement necessary and reinforcement positioning in the concrete volume);
- dimensioning of the slab foundation (foundation footing);
- preparation of technical drawings;
- the inclusion of the construction particularities in the ``Technical specifications``.

2.1. Materials and Choosing the Type of Foundation

Concrete. The characterization of concrete in the fresh and hardened state is done by classifying it. According to the rules for the production and execution of concrete works [1,15,24–26] the following 5 types of classes are used:

(a) compressive strength class, denotes the characteristic value of the concrete compressive strength determined for a concrete cylinder, respectively the characteristic value of the compressive strength determined for a concrete cube, both expressed in MPa. Therefore, a concrete whose characteristic value for compressive strength is at least 30 MPa per cylinder, respectively at least 37 MPa per cube, will be marked with C30/37 (Figure 3);

(b) the environmental exposure class is correlated with the lifetime and maintenance of the structure (usually of 50 years). Depending on the environmental conditions to which the structure is exposed (infrastructure, respectively superstructure, as the case may be), the corresponding exposure classes will be identified. Reinforced concrete elements will be classified in one or more exposure classes, such as XC1..4 (risk of corrosion of steel by carbonation), XA1..3 (risk of chemical attack on concrete), XD1..3 (risk of corrosion of steel due to chlorides), XS1..3 (risk of corrosion of steel due to chlorides in seawater), XF1..3 (risk of attack on concrete by freeze-thaw) etc.. For example, a reinforced concrete foundation in a humid and rarely dry environment corresponds to exposure class XC2 and this implies the use of a minimum class of C25/30 concrete (Figure 3);

(c) the class of chlorides, is the maximum amount of chlorides contained in fresh concrete with reference to the mass of the cement in its composition. Both the amount of chlorides contained in the aggregates and the amount of chloride ions, Cl^-, in the fresh concrete mixture should be considered. For example, for steel-reinforced concrete, the maximum chloride content of the aggregates is 0,04%, and the maximum Cl^- content by mass of cement is 0,2% or 0,4%. As a result, reinforced concrete will be marked by Cl 0,20 or Cl 0,40, both classes being accepted;

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(d) consistency class, denotes the workability of fresh concrete. For example, fluid concrete is denoted by S4;
(e) density class, indicates the density of unreinforced (plain) concrete in its dry state. A concrete with a density equal to 2,4 tons/m^3 can be marked as D2,4.

In addition, the thickness of the concrete cover for reinforcement is also influenced by the structural class [1,15] (e.g. a building designed for a service life of 50 years corresponds to the structural class S4), and the values of the internal forces resulting from the structural analysis used in the dimensioning of the load-bearing elements are also influenced by importance-exposure class [6,15] (ex. a current-type construction corresponds to the importance-exposure class III).

The concrete strength class is determined considering:

- the maximum tension in concrete for the considered structural member,
- the ductility class of the structure (seismic design) and of
- exposure class.

In Romania, it is a widespread practice to produce prefabricated elements with concrete class C30/37 or higher [5].

Infill concrete. The connection between the precast column and the pocket foundation is a wet type and is carried out by concreting the space between the two members. Before mounting the column in the pocket and pouring concrete in the gap, it is mandatory that the interfaces of the connection between column and pocket are cleaned and moistened with water. The strength class of the infill concrete will be at least the strength class of concrete in the pocket foundation concrete [2]. Aggregates used are recommended to have the maximum size of Dmax ≤ minimum (f/3, 16 mm) [14,15]. It is mandatory that the concrete used to fill in the column-pocket joint has a fluid consistency (consistency class S4 or F5) and does not shrink during hardening (non-shrink concrete).

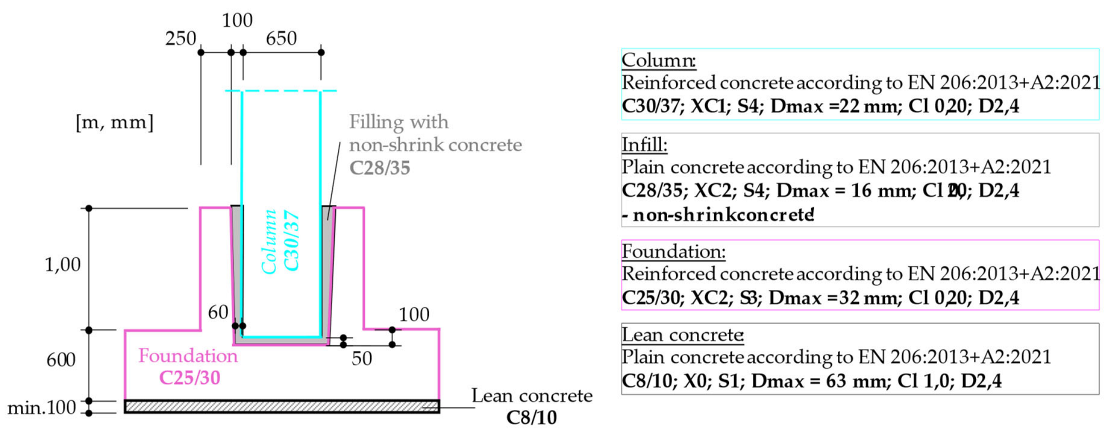

Figure 3. Exemplifying the characterization of concrete by classes on a structural joint between precast concrete column – pedestal pocket foundation.

Compliance with the provisions contained in the technical regulations regarding concrete strength classes in correlation with the exposure classes [1,2,24–26], leads to a reduction in the risk of premature degradation of reinforced concrete elements, or in other words, contributes to ensuring adequate durability of the superstructure and infrastructure of the buildings designed and constructed.

Reinforcement. The most commonly used reinforcement in the vertical members of the superstructure (columns, walls), as well as in the foundations, is the steel with strength class of B500 with ductility class B or C. This reinforcement is currently also an economical choice. Other strength classes of reinforcing steel used are B400, B420, B450 and B490 with ductility category B or C. In the past, hot-rolled steel was used as OB37 (smooth rebar, equivalent B235Cs), PC52 (rebar with periodic profile, has been used, equivalent, B345Cs) or PC60 (periodic profile rebar, equivalent to B405Cs). The use of a ductility class A of reinforcing steel is permitted for foundations and columns/walls [2,6]

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only when the members are designed to remain in the elastic range (without cracking). The second generation of EC2 [3] specifies the use of strength classes of reinforcing steel B400 to B700.

The technical regulations describing the types of reinforcing steel to be used are ST 009-2011 „ Technical specification for steel products used as reinforcement /Specificație tehnică privind produse din oțel utilizate ca armături", SR 438-1:2012 „ Steel products for concrete reinforcement/Produse de oțel pentru armarea betonului/", respectively SR EN 10080:2005 „Steel for the reinforcement of concrete/Oțel pentru armarea betonului".

For structures with columns subjected to small axial compressive stress, the pedestal pocket foundation with keyed, rough or smooth internal walls and shallow isolated footing (with a foundation depth of approx. -2,00 m underneath the finished floor), is often a convenient one. When the column is subjected to tensile axial force, it is necessary to use a pocket foundation with keyed internal walls and, possibly, with isolated footing on piles. It is appropriate to use a pad foundation with pocket and keyed internal walls in the following cases: the column is subjected to compression with small or medium eccentricity and the good soil is near to the level of the finished floor, if the soil is a hard rock (e.g. marl) and its upper level is near to the future finished floor, or if the groundwater level is near the level of the finished floor.

2.2. The Technology of Mounting the Column in the Pocket Foundation

The sequence of stages for mounting the prefabricated reinforced concrete column in the pocket foundation [5,20,26–28] can be summarized as:

(a) marking the axes of the foundation and positioning the centering device on the bottom of the pocket on a concrete layer approx. 50 mm thick (at least of the same strength class as the infill concrete);
(b) marking the station points of the mobile crane (by beating poles or painting the ground surfaces) and parking points for the trailer (when the installation is done directly with unloading from the transport equipment), in accordance with the technological project;
(c) transporting the columns in the vicinity of the final mounting position;
(d) cleaning the inner surfaces of the pocket and the outer surfaces at the base of the column;
(e) after positioning and centering the mobile crane, the handling device is fixed on its hook and mounting of the columns starts;
(f) after placing the column horizontally on the ground, next the column is lifted again to be brought to a vertical position (by crawling, turning or tipping);
(g) cleaning the column`s leaning areas and marking down the structural axes on the sides;
(h) fastening the guide ropes to the column;
(i) the slow lowering of the column into the foundation and adjusting its position by workers with the help of ropes and two crowbars, so that the axes marked on the column coincide with the position of the axes marked on the foundation;
(j) verifying the verticality of precast column on the two orthogonal sides using a plumb line, bubble level (spirit level) or laser level, followed by the initial temporary fixation of the column at the foundation level by hammering special hardwood wedges (e.g. oak, beech, acacia, etc.) on each side (minimum 6 wedges in total, of which on two parallel faces there are at least 2 pieces) [20]. The geometry of a wedge is a quadrilateral prism type, with one of the faces chamfered under an angle of 10 sexagesimal degrees [20], with a length of about 300 mm and a width of about 100 mm (Figure 4). The geometry of the wedge is made in such a way that after hammering it remains above the pocket foundation approx. 120 mm. The use of wedges with too large angles, or wedges made of soft wood (e.g. spruce, fir, pine, etc.) or overlapping wedges to compensate for their insufficient thickness – may lead to the rotation of the columns in the foundation under the wind action or under an accidental action (e.g. the collision of heavy members with columns) and their collapse in the intermediate phases of construction execution (Figures S01-S09, Video V1 and V2). In the mentioned figures and videos one can see the domino effect following the rotation of a column in the pocket foundation caused by a gust of wind with speed of up to 80 km/h. The dynamic action of wind can cause the expulsion of the wooden wedges as a result of the cyclical/balanced movement of the precast column. From a row of 11 precast reinforced concrete columns, the first column did not rotate in the pocket foundation,

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the second column rotated (the wooden wedges being crushed or expelled) and the deformed shape of the column was large enough to hit the third standing column (S01). Following the impact, the third column rotated in the pocket foundation (by crushing the wooden wedges or by expelling them, S02) hitting the fourth column. After the impact, the fourth rotated at the base and deformed and broke under its own-weight. Now broken, in the fall the fourth column hit the fifth column with amplification from the shock, and the rest of the failures happened similarly, ending with a total of 8 prefabricated columns collapsing (S03-S09). It can be noticed that the first fallen columns (fourth and fifth) developed a single plastic joint each (the cross-section broke at the base), but the next 5 columns developed two plastic joints each (one at the base and one at the middle of the height as a result of the change of the static scheme: from a cantilevering beam to that of a beam hinged in the foundation and supported at the top). The causes of the failure could be one or more of the following: high wind loading in the intermediate construction phase (the precast column – pocket foundation interspace was not completed), the fixing with wooden wedges was not correctly carried out (insufficient wedges, wedges with a chamfering of only about 20°...30 °, the wooden wedges were short, the thickness of the wooden wedges was supplemented by using short ends of wooden slats or softwood was used) and the distance between the pocket foundation internal wall and the precast concrete column (thickness of the concrete infill layer) was excessively large, a situation in which after the expulsion of the wedges, the rotation of the column at the base was so great that its tip collided with the neighboring column;

(k) checking the verticality of the precast column using more precise equipment (with topometric devices), adjusting the position of the column by loosening/hammering the oak wedges;

(l) for slender precast reinforced concrete columns as well as for those whose length exceeds 12 m, it is recommended to use tilt props or scaffoldings, for additional temporary fixing (Figure 5). The tilt props will be mounted in such a way that they can take compression forces (and tensile forces, if possible), their inclination should be at 30°...45° (angle measured between the longitudinal axis of the prop and the column), and their arrangement must provide stability in two orthogonal directions in the horizontal plane. In the case of exceptionally long columns, scaffolding or other temporary constructions with a similar effect will be used for temporary support;

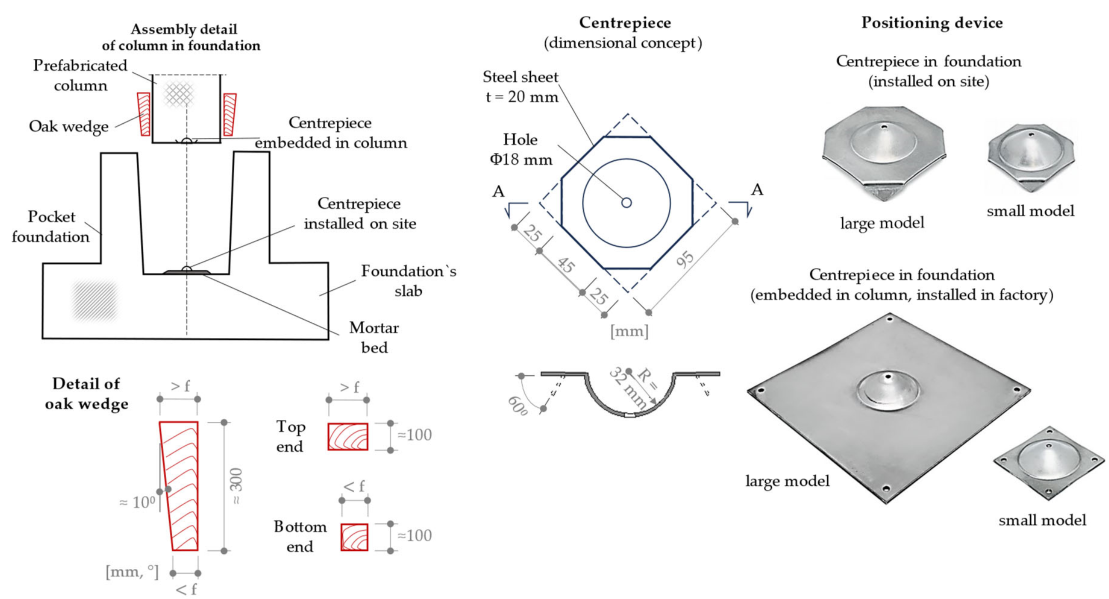

Figure 4. Details of precast concrete column mounting in the foundation and centering device. Adapted from [29] and stylized photos with branded products [30].

(m) additional topometric verification after mounting all columns in both horizontal directions (in plane) as well vertically (in elevation). At the end, the final hammering of the oak wedges is carried out, and if tilt props/scaffolding are used, they will be firmly fixed to the concrete blocks and to the columns;

(n) spraying the concrete surfaces that come into contact with the fresh infill concrete (base of the column and inner surfaces of the pocket) with water. The infill concrete is poured until it reaches the bottom level of the oak wedges. After pouring, the fresh infill concrete is vibrated with a concrete compactor;

(o) removing the oak wedges after the infill concrete has reached approx. 70% of the compressive strength (approx. 3 days) that the infill concrete has at 28 days. Sprinkling with water (watering) the hardened concrete surfaces that come into contact with the rest of the fresh concrete for filling the remaining column-pocket interspace;

(p) removal of the tilt props/scaffolding after the last part of the infill concrete poured in the interspace has reached approx. 70% of the compressive strength that the infill concrete has at 28 days;

(q) mounting the beams is carried out after the concrete filling in the column-pocket interspace is completed (Figure 5), has hardened and has reached the strength class mentioned in the project`s technical documentation (e.g. technical drawings, technical specifications etc.), unless there are other specifications from the structural engineer.

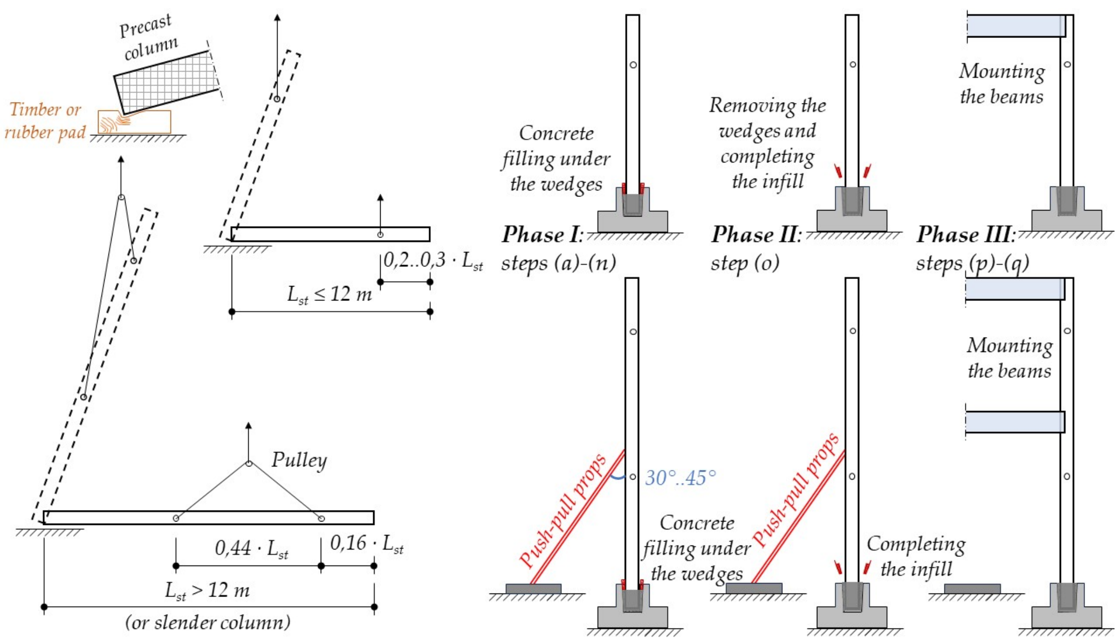

Figure 5. Temporary support of prefabricated columns belonging to a frame structure, adapted after [5,20].

Images during construction of pedestal pocket foundations with precast sockets (Figures S10-S15) and monolithic sockets (Figures S16-S20) are added as supplementary materials to this article.

2.3. Pre-Dimensioning of Socket in Pedestal Pocket Foundation

Pocket`s wall thickness, b_P. The minimum thickness specified in the standard [2] is 150 mm for sockets cast in factory, respectively 200 mm for those cast on site, but not less than one third of the short side of the column's cross-section. However, Tillmann et al. [7,11,22] recommend a minimum thickness of 250 mm. An overview of the minimum wall thickness is presented in Table 1. Sometimes, sockets are made with variable wall thicknesses and with an inclination of about 3° to 5° [2,20].

Thickness of the concrete infill, f. It is recommended that the infill thickness is f = 50..75 mm at the base of the pocket, respectively f = 85..120 mm at the top [2]. In the case of pockets with vertical internal walls (0° inclination) f ≈ 75..120 mm is used. For pockets with two or more columns, when

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no intermediate walls are used, the concrete infill should be at least 50 mm in thickness between two neighboring columns, that is to ensure the concreting of the entire interspace [2].

Thickness of the concrete infill underneath the column, f_H, respectively the embedding depth of the column`s base in the slab`s foundation, $f_{stâlp}$. In current design practice [5] f_H and $f_{stâlp}$ are considered to be 50 mm each (Figure 1). For large horizontal reaction forces, these values can be increased.

Slab`s foundation thickness, H_f. It is done after checking for punching (in the mounting and final phases) and considering $H_f \geq 250$ mm.

Depth of the pocket, H_p. It is established from the conditions of embedding the column in the pocket foundation and ensuring the anchorage length (l_{bd}) of the column`s longitudinal reinforcement. In Table 1, the pre-dimensioning equations are centralized for the pocket`s height provided in EC2 [1,3] and are used as a structural design standard in many countries in Europe as well as in other countries worldwide. Additionally, pre-dimensioning provisions given in the design norms issued in countries with moderate and high seismicity (Romania [2,6,31] and Brazil [32]) are inserted in Table 1.

Table 1. Pre-dimensioning of the pocket foundation (b_p and H_p) using as bibliographic references the lecture notes of the pioneer professor in reinforced and prestressed concrete, Fritz [33], respectively modern design standards from Europe [1,3] , Romania [2,6,31] and Brazil [32].

Type of surface	**Pocket`s wall thickness b_p**	**Depth of the pocket H_p**	**Bibliographic source**
smooth	$\geq max\begin{cases} min[(l_{st}+2\cdot f);(b_{st}+2\cdot f)]/3 \\ 100\ mm \end{cases}$	$\geq f_H + max(l_{st}; b_{st}) \cdot 1{,}2 \cdot 1{,}4$ dacă $M_{Rd.st}/N_{Ed.st} \leq 0{,}15$ $\geq f_H + max(l_{st}; b_{st}) \cdot 2{,}0 \cdot 1{,}4$ dacă $M_{Rd.st}/N_{Ed.st} \geq 2{,}00$	Leonhardt & Mönnig (1975) [33]
rough		$\geq f_H + max(l_{st}; b_{st}) \cdot 1{,}2$ dacă $M_{Rd.st}/N_{Ed.st} \leq 0{,}15$ $\geq f_H + max(l_{st}; b_{st}) \cdot 2{,}0$ dacă $M_{Rd.st}/N_{Ed.st} \geq 2{,}00$	
For intermediate values of the ratio between the value of the bending moment and that of the axial force, a linear interpolation is to be used.			
smooth rough keyed	$\geq max\begin{cases} 1{,}5\cdot[M_{Rd.1}/H_P + V_{Ed.st}]/(H_P \cdot f_{ctd} \\ 200\ mm\ (monolit)\ sau \\ 150\ mm\ (prefabricat) \end{cases}$ where, correlated with the direction of $V_{Ed.st}$, we have: $M_{Rd.1} = max(M_{Rd.st} - N_{Ed.st} \cdot l_0/3\,; M_{Rd.st}/2)$ or $M_{Rd.1} = max(M_{Rd.st} - N_{Ed.st} \cdot b_0/3\,; M_{Rd.st}/2)$	$\geq max\begin{cases} max(l_{st}; b_{st}) \cdot 1{,}2 \\ 500\ mm \\ Hs/11\ \ (**) \\ l_{bd,st} + 250\ mm \\ M_{Rd.st}/(3 \cdot l_{st} \cdot b_{st} \cdot f_{ctd,st}) \end{cases}$	NP 112-*04 (2004) [31]
smooth	No specification.	$\geq f_H + max(l_{st}; b_{st}) \cdot 1{,}2$	EN 1992-1-1:2004 (2004) [1]
keyed		$\geq l_{0,pv} + s + (f_H + c_{nom.st} + c_{nom.pv})$	

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smooth rough	$\geq max\begin{cases} min(l_{st};b_{st})/3 \\ 200\ mm\ \ (monolit)\ sau \\ 150\ mm\ \ (prefabricat) \end{cases}$	$\geq max\begin{cases} max(l_{st};b_{st})\cdot 1{,}2 \\ 500\ mm \\ Hs/8\ \ (*) \\ l_{bd,stv}+100\ mm \end{cases}$	NP 112-2014 (2014) [2] P100-1/2013+2019 [6]
keyed		$\geq max\begin{cases} max(l_{st};b_{st})\cdot 1{,}2 \\ 500\ mm \\ Hs/8\ \ (*) \\ l_{bd,st}+100\ mm \\ l_{0,pv}+s\ +(f_H+c_{nom.st}+c_{nom.pv} \end{cases}$	
For columns subjected to tensile axial force, the use of column-pocket foundation joint with smooth internal walls is not permitted. The longitudinal reinforcements of the columns subjected to tensile axial force in the seismic combination must be anchored with an anchorage length of 1,50·l_{bd}. (par. 5.7.2.2(1)) [6] and (par. 5.6.2.1(2)) [12]. The longitudinal reinforcement in the critical areas of the columns belonging to structures designed for ductility class high (DCH), require anchoring starting at 5·ϕ_{sl} inside the element in which the anchoring is made (in the pocket). In addition, the anchorage length for the rebars in tension will be 1,20·l_{bd}, where ϕ_{sl} is the diameter of the longitudinal rebars. (par. 5.7.1(4)) [6] (par. 5.6.1(3)) [12].			
smooth rough	$\geq 150\ mm$ for pedestal pocket foundations $\geq 400\ mm$ for pad foundations with pocket	$\geq f_H+max\{400\text{ mm};[max(l_{st};b_{st})\cdot 1{,}5]\}$ $dacă\ M_{Rd.st}/N_{Ed.st}\leq 0{,}15$ $\geq f_H+max\{400\text{ mm};[max(l_{st};b_{st})\cdot 2{,}0]\}$ $dacă\ M_{Rd.st}/N_{Ed.st}\geq 2{,}00$	ABNT NBR - 9062:2017 (2017) [32]
keyed		$\geq f_H+max\{400\text{ mm};[max(l_{st};b_{st})\cdot 1{,}2]\}$ $dacă\ M_{Rd.st}/N_{Ed.st}\leq 0{,}15$ $\geq f_H+max\{400\text{ mm};[max(l_{st};b_{st})\cdot 1{,}6]\}$ $dacă\ M_{Rd.st}/N_{Ed.st}\geq 2{,}00$	
For intermediate values of the ratio between the value of the bending moment and that of the axial force, a linear interpolation is to be used. In the dimensioning of the pocket foundation an overstrength factor for the column resistance must be used: $\gamma_{Rd}=1{,}2$. When rough interfaces are used, lower values of H_p can be used as long as they are validated experimentally. For columns subjected to tensile axial force it is mandatory to use keyed interfaces and $H_p\geq f_H+max\{400\text{ mm};[max(l_{st};b_{st})\cdot 2{,}0]\}$			
smooth rough	No specification.	$\geq f_H+max(l_{st};b_{st})\cdot 1{,}2$ $dacă\ M_{Rd.st}/N_{Ed.st}\leq 0{,}15$ $\geq f_H+max(l_{st};b_{st})\cdot 2{,}0$ $dacă\ M_{Rd.st}/N_{Ed.st}\geq 2{,}00$	EN 1992-1-1:2023 (2023) [3]
keyed		$\geq l_{0,pv}+s\ +(f_H+c_{nom.st}+c_{nom.pv})$	
For intermediate values of the ratio between the value of the bending moment and that of the axial force, a linear interpolation is to be used.			

Notes:

(*) Hs is the clear height of the column from the top face of the foundation to the roof purlin or to the runway beam. It is applicable only for columns shorter than 10 m.

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(**) Hs is the clear height of the column from the upper face of the foundation to the roof purlin. It only applies to the columns of single-story buildings with overhead bridge cranes and flyovers.

Notations:

- $l_{bd,st}$ is the anchorage length of the column`s longitudinal rebars. It is to be determined according to [1,6,12] and considering the maximum diameter of the rebar in tension.
- $f_{ctd,st}$ is the design value of the tensile strength of concrete used for the column calculated according to [1,6,12].
- $l_{0,pv}$ is the overlapping length of the vertical rebars in the pocket with the ones in the column calculated according to [1,6,12].
- $c_{nom.st}$ is the concrete cover for longitudinal rebars in the column.
- $c_{nom.pv}$ is the concrete cover for vertical rebars in the pocket foundation.
- $M_{Rd.st}$ is the resisting bending moment of the column associated with the axial force at the base of the column $N_{Ed.st}$.
- $V_{Ed.st}$ is the shear force of the column (in the cross-section at the base of the column) associated with the occurrence of the resisting bending moment $M_{Rd.st}$.

Conclusions are to be drawn based on the study in Table 1. The most numerous and at the same time restrictive provisions regarding the pocket`s wall thickness are those in NP 112-2014 [2] and NP 112-04* [31] The maximum pocket height results by applying the pre-dimensioning mathematical expressions from Leonhardt & Mönnig (1975) [33], where unlike the calculation models in the design rules [1–3,31,32], when transmitting reaction forces from the column`s base to the pocket foundation, the friction at the column – pocket foundation interfaces is neglected [34]. Following several experimental investigations on columns embedded in pocket foundations [35–39], it was observed that not taking into account the friction that occurs on these interfaces, load-bearing capacity of the pocket foundation determined experimentally is equal to 3 times the bearing capacity of the pocket foundation calculated using a calculation model that neglects friction [34]. Consequently, friction is especially important and must be considered in the calculation model of the column-pocket foundation joint in order to reach a more rational design [34]. Considering these observations, in the case study undertaken (chapter 3) the mathematical expressions from [1,2,6] will be used, as they are the most recent versions of the studied design standards enforced, and considered together are applicable for designing structures located in seismic areas.

2.4. Sizing the Pocket Foundation Using Strut-and-Tie Models

In mathematical calculation models for sizing pocket foundations, the column is considered to have the behavior of a solid rigid body over the entire embedding depth in the foundation. When using pocket foundations with thin walls ($b_p < l_{st}/3$), as well as when the shear force at the column`s base, $V_{Fd}$, is high (e.g. in the case of shorter columns, design for accidental truck impact situations, etc.) checking the pocket`s wall to out of plane bending becomes relevant and can be decisive in the sizing of the pocket foundation. Figures 6, 7 and 8 show the mechanical model using struts and ties (also named truss model) for determining the internal forces. For the dimensioning of the concrete cross-section and the reinforcement, the indicated equations can be used, or new ones can be determined analytically or with numerical methods keeping the calculation assumptions specified in the design standards. Experiment-based design is an alternative that can also be used in the design of pocket foundations.

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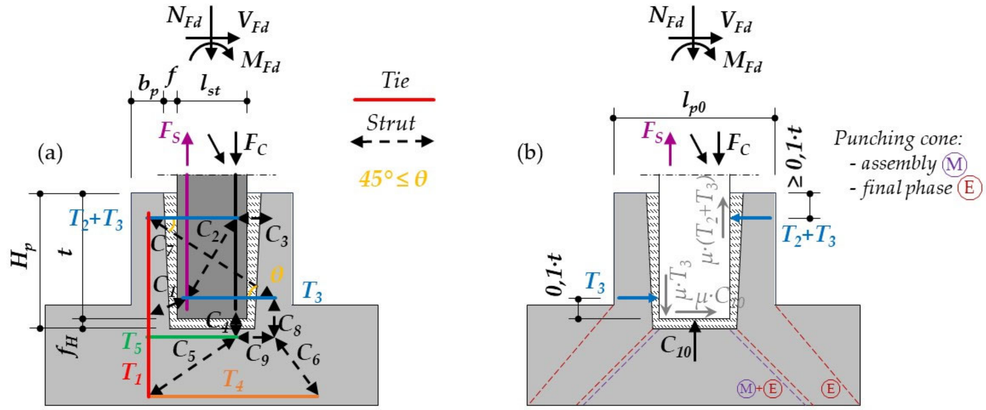


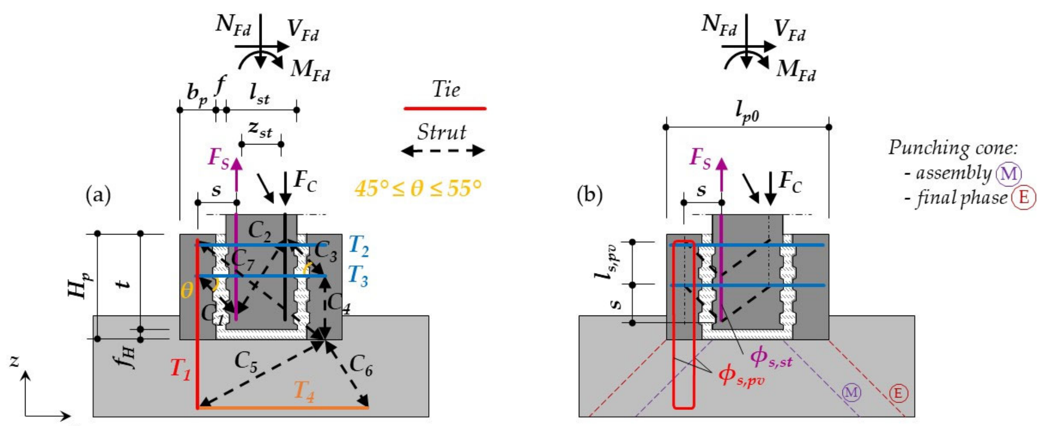
Considering friction (NP 112-2014, EC2-2004, EC2-2023) *for* ***smooth*** *or* ***rough*** *joint surfaces:*

$T_1 = (M_{Fd} + V_{Fd} \cdot H_p) / (l_{p0} - b_p)$ $\quad T_2 = V_{Fd}$ $\quad T_3 = 1{,}25 \cdot M_{Fd} / H_p$

Neglecting friction (Leonhardt & Mönnig 1975):

$T_1 = (M_{Fd} + V_{Fd} \cdot t) / (l_{p0} - b_p)$ $\quad T_2 = V_{Fd}$ $\quad \begin{cases} T_3 = 1{,}5 \cdot M_{Fd} / t + 0{,}25 \cdot V_{Fd} & \textbf{smooth} \text{ joint surfaces} \\ T_3 = 1{,}2 \cdot M_{Fd} / t + 0{,}20 \cdot V_{Fd} & \textbf{rough} \text{ joint surfaces} \end{cases}$

Figure 6. Calculation model for pedestal pocket foundation (integrally monolithic: socket and slab foundation) with smooth or rough internal walls: (a) Strut-and-tie model and (b) Normal forces on the inner surfaces of the pocket and associated friction forces, considering [1–3,5–7,33]. Struts $C_{7\text{-}8}$ appear due to shear force action, V_{Fd}, struts $C_{1\text{-}6}$ due to eccentric compression, and strut C_9 due to combined action of the shear and eccentric compression. Tie T_2 appears due to shear force action, Ties T_1 and T_3 due to bending moment, and ties T_4 and T_5 due to the combined action of shear force and eccentric compression. $C_1 \geq T_3$, $C_3 = T_2 + T_3$, $C_7 = (T_2 + T_3)/\cos\theta$, the compressive force in the other struts depends on the geometry of the foundation, the coefficient of friction that characterizes the foundation-column joint, the axial forces in the column in the intermediate phases of the construction and the position of the reinforcement. The anchorage and overlapping lengths are indicated in EC2 from 2004 [1] and NP 112-2014 [2]. The height of the pocket will be chosen in such a way as to ensure the anchorage length of the vertical reinforcement in the column.



due to V_{Fd}	$T_1` = T_2 \cdot t / (l_{p0} - b_p)$ $T_2 = V_{Fd}$	$T_1 = T_1` + T_1``$	$l_{0,pv} = (\alpha_2 \cdot \alpha_5) \cdot \alpha_6 \cdot l_{b,rqd}$
due to $M_{Fd} + N_{Fd}$	$T_1`` = F_S \cdot z_{st} / (z_{st} + s)$   $T_3 = C_{1x} = T_1`` \cdot ctg\ \theta$	$T_2 + T_3 = V_{Fd} + T_1`` \cdot ctg\ \theta$	$l_{0,pv} \geq 0{,}7 \cdot 1{,}5 \cdot l_{b,rqd} = 1{,}05 \cdot l_{b,rqd}$

Figure 7. Calculation model for pedestal pocket foundation (with precast socket and monolithic slab foundation) with keyed internal walls: (a) Strut-and-tie model and (b) Vertical reinforcement

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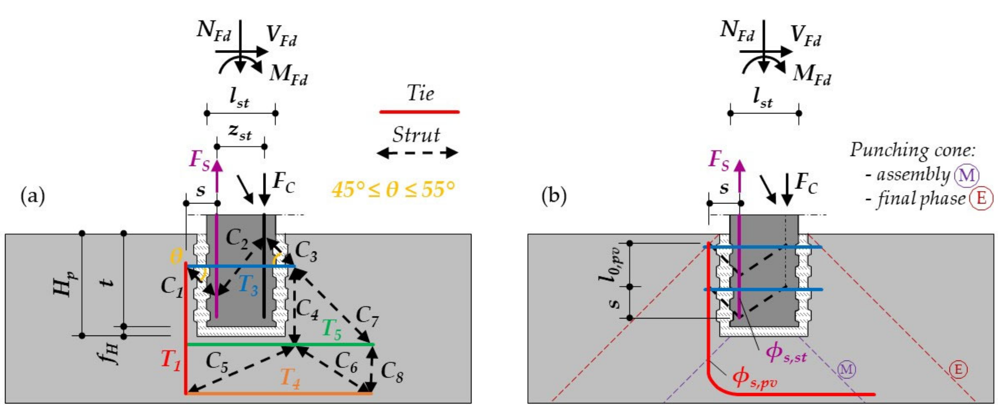
overlapping, considering [1–3,5–7,9,11,40]. Strut C_7 appears due to shear force action, V_{Fd}, struts C_{1-4} due to eccentric compression, and struts C_{5-6} due to combined shear and eccentric compression. Tie T_2 appears due to shear force action, tie T_3 due to bending moment, and ties T_1 due to combined action of the two. $C_1 = C_3 = F_s/\cos\theta$, $C_7 = (T_2+T_3)/\cos\theta$, the compressive force in the other struts depends on the geometry of the foundation, the axial forces in the column in the intermediate phases of the construction and the position of the reinforcement. The anchorage and overlapping lengths are indicated to EC2 din 2004 [1], however, according to German literature [5,7,9,11,40], anchorage and overlapping lengths can be substantially reduced. The height of the pocket will be chosen in such a way as to ensure the overlap lengths for the vertical rebars in the pocket with those in the column.

The design of **pedestal pocket foundation with smooth or rough internal walls** has been analyzed and experimentally validated in several studies [41–45], as well as the design of the **pedestal pocket foundation with keyed internal walls** [43,46–48]. The values of the internal forces that can be obtained using the equations in the bottom of Figures 6, 7 and 8 are conservative. For pedestal pocket foundation with smooth or rough internal walls, if $\mu \cdot C_{10} \geq T_3$ then the horizontal reinforcement in the lower half is arranged according to prescribed rules in [1–3].

The structural design of pad foundations with pocket and keyed internal walls is meant to be conducted like the one for a monolithic foundation, mentioning that the internal walls of the pocket are reinforced with rebars arranged vertically and horizontally, for which the sizing is carried out on the basis of internal forces T_1 and T_3. The geometry of the castellation shall comply with the provisions set out in paragraph ``2.5. Calculating the load-bearing capacity of the concrete-infill concrete-pocket interfaces``. The transfer of internal forces in the column`s cross-section at the top of the pocket shall be deemed to be carried out as follows: N_{Fd} is transmitted through the lateral keyed surface of the column embedded in foundation; M_{Fd} is transmitted through the couple of internal forces F_S and F_C which results in the reinforcement areas required in the pocket`s wall $A_{s,pv}$ and $A_{s,ph}$; and V_{Fd} is considered to be transmitted straight into the block by compressive forces (if the foundation is massive enough, no additional reinforcement is required to resist the shear force from the base of the column/bottom of the pocket). The group of internal forces N_{Fd}, V_{Fd} and M_{Fd} cannot be extracted directly from the static calculation but must be analyzed so that the bending moment for which the foundation is dimensioned in the seismic combination is at least equal to or larger than the resisting bending moment (M_{Rd}) of the column in the embedding section (top of the pocket), and the resisting shear force is at least equal to the value of the shear force associated with the occurrence of M_{Rd}. For each loading combination (fundamental and seismic) [49] groups of internal forces will be considered for maximum and minimum values of the axial force in the column. The calculation model and arrangement of reinforcement are illustrated in Figure 8.



$T_1 = F_S \cdot z_{st} / (z_{st} + s)$

$T_3 = T_1 \cdot ctg\ \theta$

$l_{0,pv} = (\alpha_2 \cdot \alpha_5) \cdot \alpha_6 \cdot l_{b,rqd}$

$l_{0,pv} \geq 0{,}7 \cdot 1{,}5 \cdot l_{b,rqd} = 1{,}05 \cdot l_{b,rqd}$

Figure 8. Calculation model for pad foundation with pocket and keyed internal walls: (a) Strut-and-tie model and (b) Vertical reinforcement overlapping, considering [1–3,5–7,9,11,40]. Struts C_{7-8} appear

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due to shear force action, V_{Fd}, and struts C_{1-6} due to eccentric compression. Ties T_1 and T_3 appear due to eccentric compression, and ties T_{4-5} due to combined action of shear force and eccentric compression. $C_1 = C_3 = F_s/\cos\theta$, $C_7 = (V_{Fd} + T_3)/\cos\theta$, the compressive force in the other struts depends on the geometry of the foundation, the axial forces in the column in the intermediate phases of the construction and the position of the reinforcement. The anchorage and overlapping lengths are indicated in EC2 din 2004 [1], however, according to German literature [5,7,9,11,40], anchorage and overlapping lengths can be substantially reduced. The height of the pocket will be chosen in such a way as to ensure the lap lengths for the vertical reinforcement in the pocket with those in the column.

The provisions regarding anchorage of the vertical reinforcement in the pocket foundation and in the column, applicable to the column – pocket foundations, are [1–3,5–7,9,11,40,50]:

(a) the vertical reinforcement placed in the pocket foundation shall be properly anchored at both ends. At the bottom end, the vertical rebars will be continued in the slab`s foundation until above the horizontal slab reinforcement layer the bottom of the footing and anchoring it with a length at least equal to l_{bd} [1], considering the tension in the reinforcement as equal to the design yield strength of the steel rebars, $\sigma_{sd} = f_{yd}$. While at the top end, it is recommended that the anchorage length be at least equal to $l_{b,min} = \max(0{,}3 \cdot l_{b,rqd}; 15 \cdot \phi; 250\ \text{mm})$, with hooks bent at 90^0;

(b) when the column can be subjected to axial tensile force, the anchorage length of the longitudinal reinforcement, l_{bd} established according to EC 2 [1], is increased by 50% (5.7.3) [6];

(c) for columns in structures designed for ductility class high (DCH), the longitudinal reinforcement in the critical region will have the anchorage length increased to $l_{bd,DCH} = 5 \cdot \phi + 1{,}2 \cdot l_{bd}$ (par. 5.7.1(4)) [6]. Columns belonging to structures designed for ductility class medium (DCM) and low (DCL) will have the longitudinal rebars anchored with lengths calculated according to EC2 [1], lbd, without increases;

(d) for pocket foundations with keyed internal walls, when the distance between the vertical rebars in the pocket and the ones in the column, s, is greater than max(50 mm; $4 \cdot \phi$), lapping length will be increased to l_0+s (par. 8.4.4(2)+10.9.6.2) [1];

(e) the depth of the pocket with keyed internal walls shall be sufficient to ensure the overlapping between the vertical reinforcement in the column and pocket, l_0+s, and orthogonal transversal reinforcement shall be calculated with (8.7.3+10.9.6.2) [1] and (II.6.2.2) [2];

(f) the depth of the pocket with smooth or rough internal walls shall be sufficient to ensure the anchorage of the column`s vertical reinforcement (8.4+10.9.6.3) [1], (II.6.2.1) [2] and [50];

(g) when calculating anchorage and lapping lengths, not only the different diameter of the rebars to be anchored/overlapped shall be taken into account, but also the different classes of concrete. The longer anchoring/lapping length will be chosen.

(h) for pocket foundations with keyed internal walls, in the calculation of the lap length, l_0, of vertical rebars overlapped with the longitudinal rebars in the column, the design value of the ultimate bond stress, f_{bd}, may be increased by 50% due to the presence of transverse compression at 90^0 (par. 8.4.4(2)) DIN EN 1992-1-1:2004+AC:2010+NA:2011-01, or due to a concrete cover of minimum $6 \cdot \phi$ EC2 [1].

The beneficial effect of normal pressure on the plane of lapping, by preventing concrete splitting and spalling in the vicinity of the overlapping reinforcement means that the 50% increase of the ultimate bond stress, f_{bd}, can be applied to concrete covers up to $10 \cdot \phi$. In EC 2 [1] this increase in bond is introduced in the calculation of the anchorage length by the α_2 coefficient (which results in $\alpha_2 = 0{,}7$ for straight rebars in tension, with a concrete layer around them of minimum $6 \cdot \phi_{sl}$) or by the coefficient α_5 (confinement by transverse pressure), unlike DIN 1045-1:2008 (German edition) and 2010 (English edition), where the design value of the ultimate bond stress is increased to $1{,}5 \cdot f_{bd}$. Nevertheless, in DIN EN 1992-1-1:2004+AC:2010+NA:2011-01, the German norm enforced, along with its national annex, this adhesion increase is introduced by considering the coefficient $\alpha_2 = 2/3 = 1/1{,}5 \cong 0.67$;

(i) when columns are horizontally concreted (e.g. precast columns) and external vibrating devices are used, then satisfactory/good bond conditions may be considered over the entire cross-sectional height when $l_{st}(b_{st}) \leq 500$ mm (par. 8.4.4(2)) DIN EN 1992-1-1:2004+AC:2010+ NA:2011-01).

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2.5. Calculating the Load-Bearing Capacity of the Concrete-Infill in Concrete-Pocket Interfaces

In the case of using column-foundation joints with keyed interfaces, with the provisions regarding the geometry of the castellation and of the joint, as well as the quality of the infill concrete (fluidity and mechanical strength), it is observed that the joint develops a sufficient bearing capacity. However, when calculating the strength of the joint, the lack of a judicious choice of coefficients α_i (Table 8.2 from [1]) involved in the calculation of the anchorage length [1], can lead to significant underestimations of the real bearing capacity.

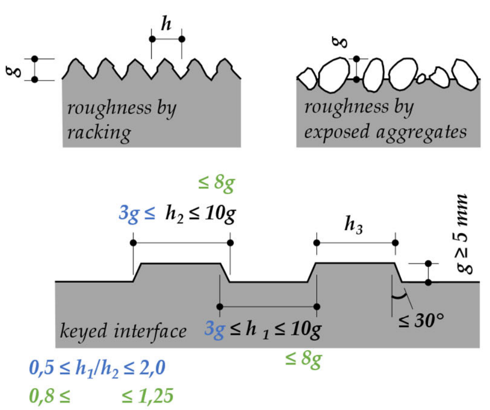
In the projects where precast column – pocket foundation with keyed interfaces were used, during the use of the building – service life - no failures or damage were observed at the level of the joint, proving a high degree of safety and a sufficient overstrength . By filling in the space between two concrete elements (column and pocket foundation) with concrete cast in previous phases (in a caisson), the interlocking of the joined elements takes place, thus the phenomenon of separation does not occur. Unlike the case of over-concreting the elements (pouring a new layer of concrete over another hardened one), where the lack of connectors or their insufficient provision, leads to failure of the joint through the separation of concrete poured in different phases.

Pocket foundations with keyed internal walls can be considered in the structural design as acting as one with the column, according to (10.9.6.2) [1] and (II.6.2.2) [2].

If tensile stresses occur in the column`s longitudinal reinforcement, then the overlapping length of the reinforcement in the foundation, l_0, will be increased by the distance s (measured from the center of gravity of the tension reinforcement in the column to the center of gravity of the tension reinforcement in the pocket foundation) and transverse reinforcement shall be provided, according to (10.9.6.2) [1] and (II.6.2.2) [2].

If the shear transmission between the column and the pocket/socket in a vertical plane is checked, then the dimensioning of the foundation for punching is conducted similarly to the monolithic column-foundation joints. Otherwise, the check for punching shall be carried out similarly to pocket foundations with smooth or rough surfaces of internal walls, according to (10.9.6.2) [1] and (II.6.2.2) [2].

The adequate geometry of the shear keys is in several technical norms [1–3,13] and is illustrated in Figure 2, considering the most restrictive criteria. The angle of the shear keys is limited to less than 30^0 to obtain a maximum shear strength in the joint plane, respectively it is limited to less than 20^0 [16] to ensure proper infill with fresh concrete.



Description of each type of interface:

(1) very smooth ($g \cong 0$ mm; achieved after casting against steel, plastic or specially prepared wooden molds);
(2) smooth ($g < 3$ mm; achieved as free surface left without further treatment after compacting);
(3) rough ($g \geq 3$ mm și $h \cong 40$ mm; achieved by racking, exposing of aggregate or other methods);
(4) very rough ($g \geq 6$ mm și $h \leq 40$ mm; achieved by racking, exposing of aggregate or other methods);
(5) keyed (with shear keys complying with drawing).

Type of interface	Coefficient for shear resistance at interfaces, c	Coefficient of friction at concrete interfaces, μ
Very smooth	0,25* (0,01*) [0]	0,5 (0,5) [0,5]
Smooth	0,35* (0,08*) [0,2**]	0,6 (0,6) [0,6]
Rough	0,45* (0,15*) [0,4**]	0,7 (0,7) [0,7]
Very rough	- (0,19*) [-]	- (0,9) [-]
Keyed	0,5* (0,37) [0,5]	0,9 (0,9) [1,0]

Notes:

Values given in table are according to EN 1992-1-1:2004, EN 1992-1-1:2023, respectively DIN 1045-1:2008.

, in case of tensile stresses perpendicular to the interface, for smooth and very smooth c = 0, while for other types is considered c/2.

** in case of tensile stresses perpendicular to the interface, or in case of dynamic loading, or in case of fatigue, then c = 0.

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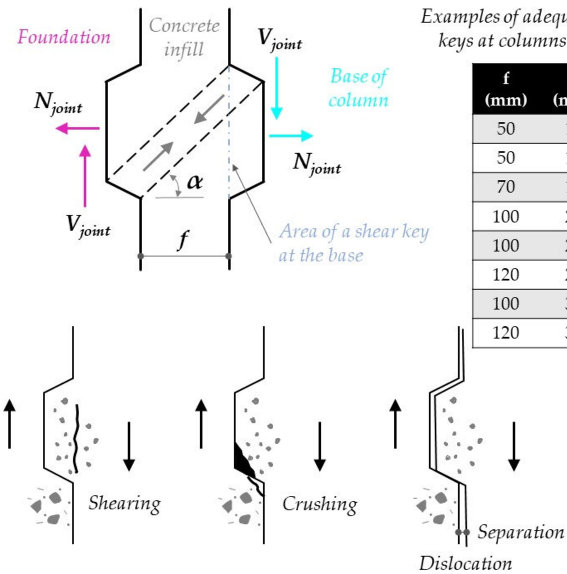
Figure 9. Classification of interfaces, according to EN 1992-1-1:2004 [1], EN 1992-1-1:2023 [3] and DIN 1045-1:2008 [13].

In [17] is recommended the maximum size for the used aggregates to be less than or equal to the depth of the shear key, g. Even though the second generation EC2 [3] explicitly classifies the types of concrete interfaces (Figure 9), however, the criteria for a surface to be considered as keyed remain more stringent than those of the German standard DIN 1045-1 [13], where it is stipulated that the depth of the shear key, g, must be at least 10 mm, and the maximum length of the teeth $h_1 \leq 0{,}8 \cdot g$ and $h_2 \leq 0{,}8 \cdot g$, while $0{,}8 \leq h_1/h_2 \leq 1{,}25$. Possible failure mechanisms of the keyed surfaces and examples of compliant geometries for column-pocket interfaces are illustrated in Figure 10.



Examples of adequate geometry for the shear keys at columns and pocket foundations

f (mm)	g (mm)	h_1 (mm)	h_3 (mm)
50	10	50	40
50	10	60	50
70	15	60	45
100	20	100	80
100	25	125	100
120	25	120	90
100	30	150	120
120	30	150	120

Note:
It is recommended for the column-foundation interface to have the same geometry for the shear keys (depth of shear key, distance between keys, angle).

Figure 10. Possible failure mechanisms for constellation [20], examples of compliant geometries for shear keys on the base of the column and on the internal walls of the pocket foundation, respectively a rubber molding is to be placed on the formwork in the area meant to have of keyed interface.

The following should be verified at the interface between concrete cast at different times:

$$\tau_{Edi} = V_{Ed,i}/A_i \leq \tau_{Rdi} \tag{1}$$

where $\boldsymbol{\tau_{Edi}}$ is the design value of the shear stress at the interface, $\mathbf{V_{Edi}}$ is shear force acting parallel to the interface, and $\mathbf{A_i}$ is the area of the keyed interface (when the area of the keyed interface related to the elements differs, the minimum area shall be considered).

Shear stress resistance at interfaces, $\boldsymbol{\tau_{Rdi}}$, is influenced by:

- coefficient of the shear resistance at interfaces, **c**, with values given in the table shown in Figure 9;
- design value of the tensile strength of concrete, $\mathbf{f_{ctd}}$;
- coefficient of friction at concrete interfaces, $\boldsymbol{\mu}$;
- the compressive stress (with absolute value) acting perpendicular to the plane of the interface, $\boldsymbol{\sigma_n \leq 0{,}60 \cdot f_{cd}}$, and if the normal stress at the interface surface is tensile, then it is considered in eq.(2) $\boldsymbol{\sigma_n = 0}$.

The load-bearing capacity to longitudinal shear at the interface, without interface reinforcement, calculated with EC2 (6.2.5 and eq. (6.25)) [1]:

$$\tau_{Rdi} = min(c \cdot f_{ctd} + \mu \cdot \sigma_n;\ 0{,}5 \cdot \upsilon \cdot f_{cd}) \tag{2}$$

where it can be considered [51]:

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$$\sigma_n = \frac{C_1 \cdot cos\,\theta}{A_i} = \frac{V_{Fd} + T_3}{b_{st} \cdot t} \quad (3)$$

Strength reduction factor for concrete cracked due to shear, **ν**, is calculated as:

$$\upsilon = 0{,}6 \cdot \left(1 - \frac{f_{ck}[MPa]}{250}\right) \quad (4)$$

In the second generation of EC2 (8.2.6(5) and eq. (8.76), respectively 10.7(2) and eq. (10.8)) [3], this load-bearing capacity to longitudinal shear at the interface, without interface reinforcement, is:

$$\tau_{Rdi} = min\left(c \cdot \frac{\sqrt{f_{ck}}}{\gamma_C} + \mu \cdot \sigma_n;\ 0{,}3 \cdot f_{cd}\right) \quad (5)$$

where $\mathbf{f_{ck}}$ denotes characteristic concrete cylinder compressive strength, γ_C represents a partial safety factor for concrete, and $\mathbf{f_{cd}}$ is the design value of concrete compressive strength.

2.6. Dimensioning of the Slab Foundation (Isolated Footing)

The in-plane size of an individual footing, considering a shallow foundation type, must ensure the design requirements regarding the limitation of the maximum value of the earth pressure under the footing and must check the stability to overturning of the foundation, respectively the limitation of its settlement and rotation. The footing of the pocket foundation is then dimensioned for punching, bending and shear.

Dimensioning for punching is carried out for the assembly phase (without infill concrete) considering the weight of the column multiplied by a dynamic coefficient γ_{din} = 1,5 and considering the angle θ = 45^0. Afterwards, for the final phase (after the monolithic concrete has been poured, hardened and reached the class given in the project) the design forces resulting from the structural analysis (for the ultimate limit state phase and possible intermediate phases) are used and also the bending moment capacity of the column (for seismic design) are considered. In this phase the check for punching is done in respect to the type of the precast column-pocket foundation joint interface (with smooth, rough or keyed surfaces), Figures 6, 7 and 8.

Bending moment and shear design are relevant in the cross-sections at the outside face of the pocket.

The equations for dimensioning for punching, bending moment and shear force can be used as given in EC2 [1].

Provisions regarding the reinforcement of the pocket foundation footing [1,2]:

(a) longitudinal reinforcement (bottom side):
- minim reinforcement ratio in each direction: $\rho_{sl,min} = 0{,}1\%$;
- minimum diameter of the rebars used: $\phi_{sl,min} = 10\ mm$;
- maximum and minimum distances between rebars: $s_{l,max} = 250\ mm$ and $s_{l,min} = 100\ mm$;
- reinforcement will be anchored at the edge of the footing with $l_{b,min}$ [1] or with hooks having a length of $15 \cdot \phi_{sl}$ [31] or of d (effective depth of the footing at the edge) [2];

(b) longitudinal reinforcement (top side): minimum 3 rebars in each direction arranged orthogonally next to the column as a grid on the entire slab surface. Foundations that do not detach from the ground are constructively reinforced, and those that work with an active area are reinforced based on the dimensioning to the negative bending moments in the critical cross-sections (in this case, the provisions for bottom reinforcement are also respected) [2].

(c) transversal reinforcement: if transverse reinforcement is required due to checking for punching, then inclined or vertical reinforcements will be arranged, respecting the reinforcement provisions specified in EC2 [1].

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3. Case study. Results

This case study aims to determine the tension forces in the pocket foundations for a long reinforced concrete column belonging to a single-story warehouse built of prefabricated concrete structural elements. For the precast reinforced concrete column characterized below (Figure 11, Table 2 and Table 3) the tension forces in the ties (from the corresponding strut-and-tie model) for each of the 4 types of pocket foundations is calculated. The design forces calculated at the base of the column in the cross section that is placed at the upper part of the pocket, are noted in Table 3. For column – pocket foundation joints with keyed surfaces, the theoretical embedment depth of the column in the foundation to be used in the structural analysis of the superstructure is considered to be at the top of the pocket foundation in all of the cited literature sources. However, for column – pocket foundation joints with smooth surfaces the recommendations for the embedment depth vary, in most sources [1,3,5,52] it is recommended to be considered at the top of the pocket foundation; in the lectures on precast structures at TUCN, by professor Kiss Z. it is indicated at $H_P/3$ measured from the top to be bottom of the foundation; at t/2 or even at $H_P/2$ [53] (Ch. 12.2.5.1.1, Fig. 12.21 and pp. 388). However, for the case study presented below the recommendations stated in the latest publications [1,3,5] are considered, which indicate the fixation of the column in the foundation to be established at the upper part of the pocket foundation, regardless of the type of the pocket foundation or the roughness of the interfaces in the joint.

Table 2. Input values for the case study.

Parameter	Value	Meaning
		Input
Geometry		
H_{st_0}	7,0 m	Free height of the column measured vertically from the top of the slab on grade to the bottom of the roof`s main beam
H_{st_calc}	8,2 m	Design height of the column measured vertically from the top of the pocket foundation (-0,40 m) to the roof`s center of gravity (0,80 m above the bottom of the roof`s main beam)
f	100 mm	Thickness of the in situ concrete filling around the column
f_H	50 mm	Thickness of the in situ concrete filling below the column
f_{stalp}	50 mm	The embedment depth of the column in the footing of the foundation
b_{st} , l_{st}	600 mm	Size of the column`s rectangular cross section
$A_{sl,1\_lat,ef}$	1390 mm^2	Total cross sectional area of the longitudinal rebars on one side of the column (2ϕ20+3ϕ18)
ϕ_{sw}	8 mm	Diameter of the stirrups (with 6 legs) used as transverse reinforcement in the column
$c_{nom,st}$	30 mm	Concrete cover for longitudinal rebars (in the column)
Materials		
C30/37, C28/35, C25/30		Strength classes for concrete in column, joint and foundation
B500 C	-	Steel grade and ductility class of the steel used as reinforcement
Structural design		
DCH	-	Ductility class of the superstructure
γ_{Rd}	1,15	Overstrength factor which considers the overstrength of the superstructure in DCH. For DCM and DCL the value 1,00 can be consider.

θ	45⁰	Inclination of the strut crossing the joint
100	%	Minimum active area of the foundation footing in all Fundamental combinations, according to NP 112-2014 [2] (par. I.6.1.1(3.1). In the previous edition, NP 112-*04 [31] (6.2.5), the acceptance limit was set to 80%.
75	%	Minimum active area of the foundation footing in all Seismic combinations, including plastic hinge formation, according to NP 112-2014 [2] (par. I.6.1.1(3.2)). In the previous edition, NP 112-*04 [31] (6.2.6), the acceptance limit was set to 50%.

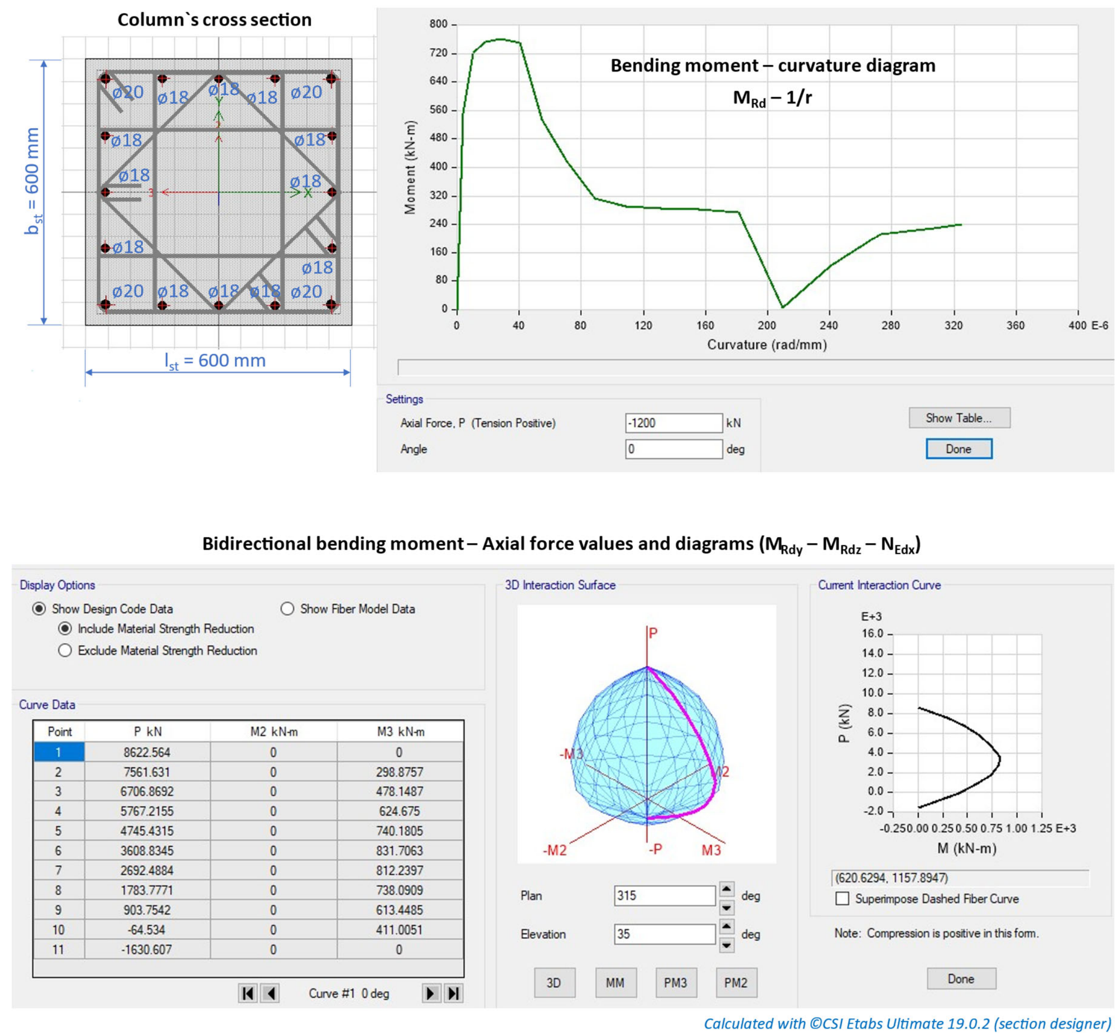

Figure 11. Input data for the precast reinforced concrete column: geometry, reinforcement and bending moment– curvature (M_{Rd} – 1/r) interaction diagrams and bi-directional bending moment – axial force (M_{Rdy} – M_{Rdz} – N_{Edx}).

Table 3. Reaction forces at the base of the column after structural analysis using load combinations from EC0 [49].

Load combination	N_{Ed} („-" is compression)	V_{Ed}	M_{Ed}
Fundamental combination (STR, GEO)	-2400 kN	51 kN	415 kN·m
Fundamental combination (ECH)	-1100 kN	51 kN	415 kN·m
Seismic combination (GS)	-1200 kN	70 kN	580 kN·m

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Seismic combination (GS) – plastic hinge	-1200 kN	76 kN	620 kN·m

Since the enforcement of the new European standard EC2 [3] for the design of reinforced concrete structures is planned for 2027, and the national annexes have not yet been developed, in this case study equations from EC2 [1] (which is the design standard enforced) are used for calculating the anchorage and overlap lengths of the rebars used in dimensioning of the necessary reinforcement for the pocket foundations.

Table 4. Results of the case study, using [1,2,6].

		Pedestal socket foundation			Pad foundation with pocket	
		having the internal walls				
		smooth	**rough**	**keyed**	**keyed**	
Parameter	Unit	Value for each foundation type				Meaning
b_p	mm	200	200	200	-	Wall thickness of the pocket foundation
$l_{bd,st}$	mm	1040	1040	1000	1000	Anchorage length for the longitudinal rebars in the column
$l_{0,pv}$	mm	-	-	590	590	Overlapping length for the vertical rebars in the pocket
s	mm	-	-	240	180	Distance between the vertical reinforcement in the internal wall of the foundation and the longitudinal reinforcement in the column
H_p	mm	1200	1200	1100	1100	Total height of the pocket`s internal walls
T_1	kN	711	711	494	449	Design value for tension force in tie T_1
T_1 %	[-]	100%	100%	69%	63%	Value of T_1 compared with the pedestal pocket foundation with smooth internal walls
T_2	kN	76	76	76	-	Design value for tension force in tie T_2
T_2 %	[-]	100%	100%	100%	-	Value of T_2 compared with the pedestal pocket foundation with smooth internal walls
T_3	kN	646	646	414	449	Design value for tension force in tie T_3
T_3 %	[-]	100%	100%	64%	70%	Value of T_3 compared with the pedestal pocket foundation with smooth internal walls
σ_n	N/mm^2	1,05	1,05	0,78	0,83	Compression stress per unit area caused by the normal force across the joint interface column-concrete filling calculated with eq.(3), as in [51]

A_i	mm²	690E3	690E3	630E3	630E3	Area of the interface
$V_{Rd,i}$	kN	740	900	994	994	Shear resistance (strength) at the interface column-concrete filling
$V_{Ed,i}$	kN	605	605	605	605	Design value of the shear force at the interface, column-concrete filling, $V_{Ed,i} = F_s$
$\tau_{Rd,i}$	N/mm²	**1,07**	**1,31**	**1,58**	**1,58**	Shear stress resistance (strength) at the interface column-concrete filling
$\tau_{Ed,i}$	N/mm²	**0,88**	**0,88**	**0,96**	**0,96**	Design value of the shear stress at the interface column-concrete filling

4. Discussion

Analyzing the results of the calculations performed for the case study, we can state the following:

- the shear stress resistance, $\tau_{Rd,i}$, is greater than the design value of shear stress, $\tau_{Ed,i}$, for all 4 types of the pocket foundations studied, which means that the pre-dimensioning mathematical expressions led to a correct final geometry of each of the pocket foundations. Whereas the ratio of longitudinal reinforcement in the column is 1,2% compared to the maximum permitted value of 4,0% [1,2,6], it is expected that for heavy reinforced columns, increased pocket heights, H_P, will be required, which means that a pre-dimensioning with consideration of M_{Ed}/N_{Ed} ratio as indicated in the new EC2 standard [3] could provide H_p values closer to those required in the final design for columns with large ratios of longitudinal reinforcement;
- the interface area, A_i, was taken as the area of the entire lateral side of the column over the pocket depth for all interface types. By doing so, when compared to pocket foundations with smooth internal walls, the shear resistance, $V_{Rd,i}$, was 18% higher for pocket foundations with rough internal walls and 26% higher for those with keyed internal walls (despite that the pockets with keyed internal walls are 8% shorter than those with smooth or rough internal walls). The values obtained seem plausible and we interpret them from the perspective of the roughness of the column-filled concrete interface, therefore, as the roughness increases, so does the shear resistance. However, this conclusion regarding joints with keyed interfaces becomes interpretable according to the text in EC2 [1] (par. 6.2.5(1)) and that of the new EC2 [3] (par. 8.2.6(4)), where for area of the joint, A_i, one would consider either the area of all individual castellations placed on one side and/or the keyed surface as a whole;
- pocket foundations with keyed internal walls are more efficient than those with smooth or rough walls, since for reinforcing of the pocket, 30% to 37% less reinforcement area and pocket heights lower by 8% are required.

Precast column - concrete infill - pocket foundation interface. However, when is it necessary to calculate the bearing capacity of the keyed precast column – concrete infill – pocket foundation interfaces? The calculation is required when we are in one of the following situations:

- when the geometry of the shear keys (castellations) does not comply with the provisions of the design norm;
- when the concrete used for filling the joint is not sufficiently fluid in its fresh state, or shows shrinkage after hardening and drying;
- when the longitudinal reinforcement of the column is meant to be anchored directly in the foundation and the castellations do not comply with the provisions of the design norm;
- when the castellations have been damaged to a great extent (e.g. during removal of the formwork, mounting, handling, etc.);
- when the column is subjected to large axial compression force (e.g. $\nu_{compresiune} = N_{Fd}/(b_{st} \cdot l_{st} \cdot f_{cd}) \geq 0{,}3$) or when the column is subjected to axial tensile force.

Calculating castellations in a simplified way - the authors' proposal. In precast concrete column – pocket foundation keyed joints, the ensemble behaves like a caisson (characterized by a state of confinement), as a result, the surfaces cannot detach or move away from each other. In this case, $0 \leq \sigma_n \leq 0{,}60 \cdot f_{cd}$ can be considered. It is proposed that for a column subjected to pure compression, the transfer of the axial force to the foundation should be considered to be on the side faces of the pocket and the base of the column, with $\sigma_n = 1{,}0 \cdot f_{ctd}$ (separation cannot take place), by equations in accordance with the design norm for reinforced concrete walls CR 2 -1-1.1/2013 (par. 7.6.3 and eq. (7.18)) [54] and with its previous edition CR 2-1-1.1/2006 [55], where for vertical joints with shear keys between precast wall panels, the shear strength of a concrete indentation (tooth) is $\tau_{Rd} = 1{,}5 \cdot f_{ctd}$. Finally, the following equations can be written to calculate the shear strength of the joint with keyed surfaces:

$$\tau_{Rdi} = \min(0{,}5 \cdot f_{ctd} + 0{,}9 \cdot 1{,}0 \cdot f_{ctd};\ 0{,}5 \cdot 0{,}5 \cdot f_{cd}) \cong \min(1{,}4 \cdot f_{ctd}; 0{,}25 \cdot 14 \cdot f_{ctd}) \\ = \mathbf{1{,}4 \cdot f_{ctd}} \tag{6}$$

For columns subjected to axial compressive force combined with shear force and bending moment, it is proposed to effectively calculate the value of σ_n, and if it is greater than $1{,}0 \cdot f_{ctd}$ the result in the calculation is to be considered, otherwise $\sigma_n = 1{,}0 \cdot f_{ctd}$ is to be used (under normal working conditions, the precast column – infill concrete – pocket foundation separation does not occur due to the large out-of-plane bending stiffness of the pocket`s walls).

It is also proposed that the beneficial effect of the confinement of the column – infill concrete interface is to be introduced in the dimensioning equations for keyed joints by considering the interface area, A_i, as the lateral area of the columns embedded in the pocket (as in NP 112-2014 [2]) and not just the area of all individual castellations placed on one side (as it might be interpreted from EC2 [1] when designing structural joints with keyed surfaces). Likewise, for the beneficial effect of confinement at the infill concrete – pocket foundation interface. Additionally, in the dimensioning of the keyed interfaces for column – pocket foundation joints the shear strength can be considered similar to that of a monolithic element, as it is stated [1] (par. 10.9.6.2(1)), where it is stated that pocket foundation with keyed surfaces may be considered to act as monolithic with the column. The shear stress resistance at the interface between two monolithically cast concrete elements (e.g. coupling beam - column) is equal to $\tau_{Rdi} = 1{,}0 \cdot f_{ctd}$ (for DCH), respectively $\tau_{Rdi} = 1{,}5 \cdot f_{ctd}$ (for DCM), according to CR 2-1-1.1/2013, (par. 7.7.2, eq. (7.19) and eq. (7.20)) [54]. While for the shear stress strength of a monolithic concrete wall, in the same technical regulation (par. 7.6.2. with eq. (7.8) and eq. (7.9)) [54] it is recommended to consider $\tau_{Rdi} = 0{,}15 \cdot f_{cd} \cong 2{,}1 \cdot f_{ctd}$ (DCH), respectively $\tau_{Rdi} \cong 0{,}18 \cdot f_{cd} \cong 2{,}5 \cdot f_{ctd}$ (DCM). Kiss Z. recommends in [5] to use $\tau_{Rdi} = 1.5 \cdot f_{ctd}$ in the calculation of the toping – beam/wall joint (chap. 4.5.4 and eq. (4.84)). Given these provisions from the technical regulations enforced (for wall, plate and beam type elements) and in the absence of specific provisions for column – pocket foundation joints with keyed internal walls, based on the previous equations one can consider as conservative: $\boldsymbol{\tau_{Rdi} \cong 1.4 \cdot f_{ctd}}$, or to be a reference value for the shear stress strength of the concrete in the column – infill concrete – pocket foundation ensemble with keyed interfaces.

Future research directions. Further optimization of the superstructures made of reinforced concrete is in great demand. The authors are focusing on two approaches for reducing the self-weight of long precast concrete columns for structures designed for seismic areas. The first consists in developing prestressed reinforced concrete columns with integrated viscous elastic damping and self-centering capacity bioinspired by coniferous trees [56–63]. And second, reducing the self-weight of the column through variations of the concrete`s density [64–67] in respect to the ratio of the utilization factor in each cross section. In both cases, the design method with strut-and-tie needs to be further studied for each type of pocket foundations and adapted, if applicable.

Supplementary Materials: The following supporting information can be downloaded at: https://drive.google.com/drive/folders/17qg7bHGTxoXyvc-8HCvpNThswc3lt2VD?usp=sharing, Figures S01-S09: Insufficient temporary fixing of long prefabricated concrete columns in pocket foundations due to strong wind conditions caused collapse with a domino effect.; Video S1: The collapse of the columns fixed with wooden wedges under the action of a gust of wind; Video S2: A row of 11 columns after collapse, of which 1 is intact, 2 are tilted and 8 are broken; Figures: S10-S15 execution of pocket foundations with prefabricated socket and keyed internal walls, respectively Figures S16-S20: execution of pocket foundations with monolithic socket and keyed internal walls.

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