Strengthening concrete members for punching shear

Tackling new problems with familiar solutions

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1. Introduction and background

The past two decades have witnessed a growing trend in the construction industry to reuse existing building stock to meet changing socio-economic demands and to reduce the environmental footprint. This is more prevalent in urban environments where a substantial portion of reinforced concrete (RC)-framed buildings and bridges are nearing the end of their service lives and require either refurbishment or outright demolition. Additionally, the need to strengthen structures may stem from several other factors: a change in use or occupancy class, an expansion of a building's footprint, addition of new floors, the introduction of new building regulations, the presence of errors or other deficiencies during the initial execution, and countering other durability-related issues caused by known hazards such as fires and earthquakes.

Depending upon the client's brief, the structure's current state, and its social, cultural, and historical importance, the engineer may find strengthening an existing building or bridge to be the superior choice than demolition and starting afresh, with evidence suggesting a 15 to 70% quicker "turn-around" time – time between stopping activity in the building or bridge and returning it into service – when compared to building a new structure. This advantage comes on top of a reduction of 10 to 75% in the resource burden through savings in labor and material [1].

After a local and global assessment of the existing structure, the engineer must choose between multiple strengthening methods to address any deficiencies in tension, compression, bending, shear, punching shear, and torsion, while meeting serviceability requirements. Strengthening on a global level is possible, for instance, by using frame encasement (e.g., additional shear walls), installing micro-piles, or installing base isolation or energy dissipation devices in case of earthquake loading. Conversely, strengthening of local, individual members includes concrete overlays; concrete-, steel-, or fiber-reinforced polymer (FRP)-jacketing, external- or near-surface-mounted FRP, external post-tensioning, or internally applied (post-installed) steel reinforcement [2]. The majority of strengthening projects usually involve multiple techniques to efficiently resist the additional loads and transfer them from the point of action to the foundations.

In many parts of the world, a large majority of existing buildings and civil infrastructure is currently undergoing or is scheduled for strengthening, therefore requiring careful deliberation on the adoption of the most appropriate intervention techniques. This paper provides an short overview of **punching shear** in concrete, summarizes existing methods or interventions typically employed to strengthen individual concrete members, and introduces Hilti's newest strengthening solution employing post-installed threaded rods that behave as punching shear reinforcement, the **HIT-Punching Shear** strengthening system, which in 2025 was granted a general construction technique permit (**aBG Z-15.5-387**) by the *Deutsches Institut für Bautechnik* (**DIBt**).

2. Overview of flat slabs and punching shear behavior in reinforced concrete

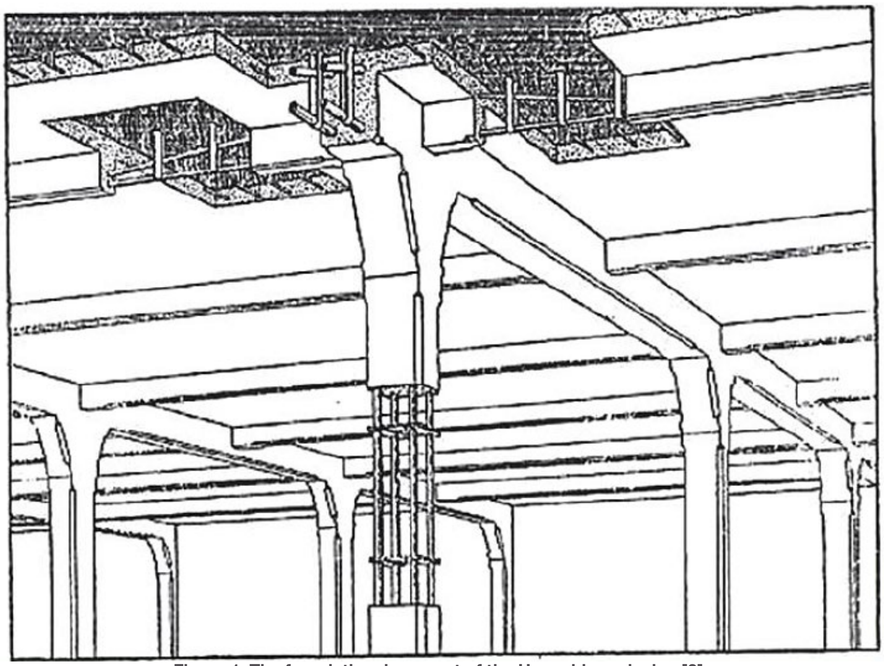


Figure 1: The foundational concept of the Hennebique design [3].

2.1 The development of flat slabs in reinforced concrete

The widespread adoption of early reinforced concrete buildings with beam-and-girder systems in the late 1800s, such as the patented *Système Hennebique* illustrated in Figure 1, reflected the same construction approach found in traditional timber and the more recent iron constructions. These systems, and particularly their individual beams, could be modelled reliably – thanks to the works of Emil Mörsch [4] and Wilhelm Ritter [3] – as analogous truss models consisting of a series of struts and ties. The structural frame consisted of complex formwork and reinforcement, forming discontinuous soffits that made positioning of building services underneath slabs a challenge, and imposed additional constraints on the interior floor space.

Introduced 120 years ago, the first reinforced concrete slab systems supported directly on columns represented an important break from the traditional arrangement of hierarchical linear structural elements. Pioneered simultaneously, although independently, by C.A.P Turner in the USA and Robert Maillart in Switzerland between 1905-1909, the design of their new slabs included a large mushroom-shaped column head (or capital) to facilitate the local introduction of forces from the slab into the column. Turner's design philosophy viewed columns, the capitals, and slabs as individual elements that could be optimized for rapid assembly. This approach maintained fundamental links to the traditional timber and iron frame constructions of the era, which helped gain widespread contractor acceptance due to their familiarity with traditional construction. The enlarged column capitals, with radial and diagonal reinforcement fanning out into the slab, flared seamlessly into the horizontal slabs, and were sized empirically to prevent punching failure [5].

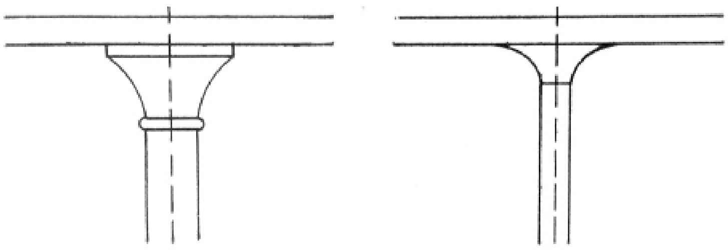


Figure 2: A comparison of the mushroom-shaped column heads for flat slabs by Turner (left) and Maillart (right) [6]

In contrast to Turner's approach, Maillart sought to reflect the inherent flow of stresses found in concrete's monolithic nature by conceiving of slabs and columns as a "unified structural entity", as seen in Figure 2 [6], in which forces flow continuously. This allowed the creation of a seamlessly merged slab-column connection through continuous reinforcement in only two directions, resulting in configurations that often negated capitals entirely or integrated capitals through thickened slab regions around the column. Both approaches allowed for flat soffits that provided sufficient punching shear resistance through the hyperbolic shape of the column capital, reflecting the hyperbolic flow of the stresses towards the column center [7]. Departing from Turner's empirical approach, Maillart developed novel elastic analysis methods to calculate bending in two-way slabs that complemented his practice of conducting full-scale load tests on his completed slabs and bridges. These are still used today.

The combined influences of both Turner and Maillart were felt from the 1950s, a period that saw many residential and office buildings, as well as multi-storey parking garages embracing the large spans (~9m) offered by flat slabs, with most structures entirely foregoing the distinct column capital. This further simplified formwork and reinforcement and provided a continuous flat soffit for easier positioning of building services.

2.2 The behavior and failure modes of reinforced concrete slab systems

2.2.1 Behavior and failure modes of slabs supported by beams

The behavior of one- and two-way linearly supported slabs under a uniformly distributed load is analogous to that of beams under shear. In both, the high compressive but low tensile strength of concrete causes it to crack perpendicularly to the tensile stress resulting from a sufficiently high applied load. Both beams and one- and two-way slabs resist shear by a combination of:

1. The uncracked concrete in the compression zone.
2. Dowelling action of any existing longitudinal reinforcement, and
3. Aggregate interlock across the tension cracks.

However, the haphazard nature of these three effects acting concurrently does not generate sufficiently large tensile strength to prevent concrete from cracking under a comparatively small tensile component of shear stress, leading to cracks developing diagonally near the supports where a significant upwards thrust exerted through the beam's web resists the downward applied load. Effectively resisting shear necessitates the addition of specific shear reinforcement – known as stirrups, links, or ties – that will activate after the formation of the first diagonal cracks to curtail their width within acceptable limits [8].

2.2.2 Behavior and failure modes of slabs supported by columns under concentrated loads

In contrast to slabs supported by beams (linear support), but with several similar characteristics, flat slabs transfer large, concentrated loads into a loaded area around the supporting column (point support). Under moderate loads, **radial** flexural cracks first form at the tension side of the slab and radiate outwards from the column, dividing the slab into segments that rotate about the column, leading to moment redistribution in the tangential direction where concrete is still uncracked and stiffer in comparison. At higher loads, the concrete then forms **circumferential** (or tangential) flexural cracks around the column.

These simultaneously generate inclined shear cracks that arise from the circumferential cracks in the tensile zone of the slab in the tangential direction and propagate towards the compression zone where the slab soffit meets the loaded area, which is the column face if no capitals are provided, highlighted by Figure 3. These cracks disturb the inclined compression struts resisting shear, and one of these is termed as the “critical shear crack”, which intercepts the compression strut near the loaded area. Wider critical shear cracks generated by higher slab rotations cause the compression strut to crush and lead to the slab-column connection experiencing a sudden loss of resistance, in turn resulting in a localized, brittle type of failure known as “punching” (or two-way) shear [9].

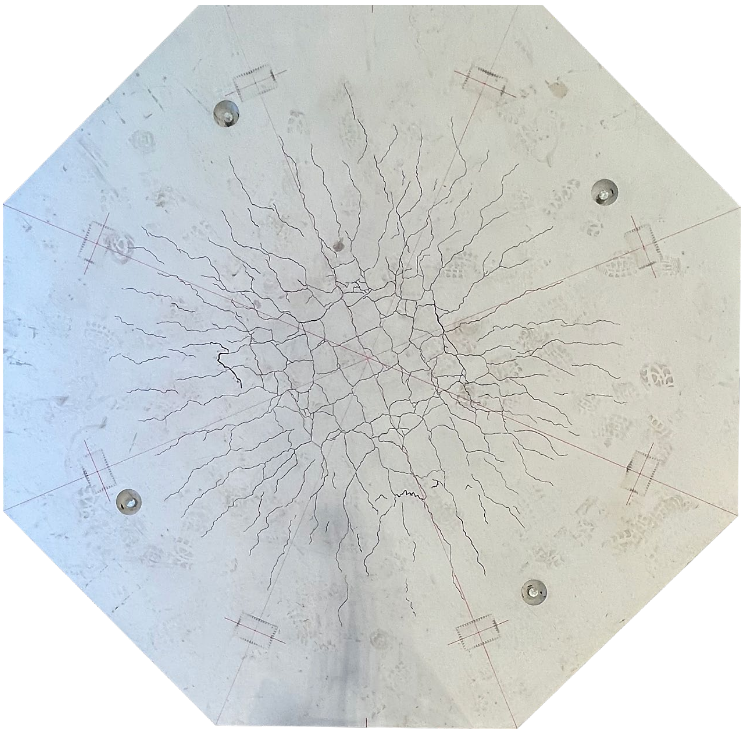


Figure 3: An example of radial and tangential cracks in a typical concrete slab under concentrated loads © Hilti

However, flat slabs rely not only upon the contribution of the strength of the uncracked concrete in the compression zone, but also on several other factors to resist punching shear:

1. Aggregate interlock due to tangential displacement across the crack surface.
2. Residual tensile stresses in the concrete at the opening of the crack surface.
3. Dowel action from longitudinal reinforcement in the tension zone.
4. Tensile and dowel forces transferred by any punching shear reinforcement.

Regardless, the resulting failure from a loss of equilibrium between the imposed actions and internal shear forces leaves a **truncated cone** forming around the column as illustrated by the two specimens in Figure 4.

Figure 4: Flat slab specimens highlighting formation of the shear cracks and the truncated cone © Hilti

Several types of shear reinforcement, most commonly stirrups but also double-headed studs, are cast within to increase the punching shear resistance of these slabs and, depending upon the amount and detailing of the shear reinforcement, failure occurs when the concentrated loads exceed the maximum punching shear resistance inside or outside the shear-reinforced zone. Providing insufficient shear reinforcement to limit growth of internal critical shear crack results in yielding or pullout of the anchored shear reinforcement inside this zone. Failure may occur beyond the shear-reinforced zone if insufficiently large. After ruling out failures within and beyond the shear-reinforced zone, the strength provided by the concrete struts limits maximum punching resistance of the slab [10], as seen in Figure 5.

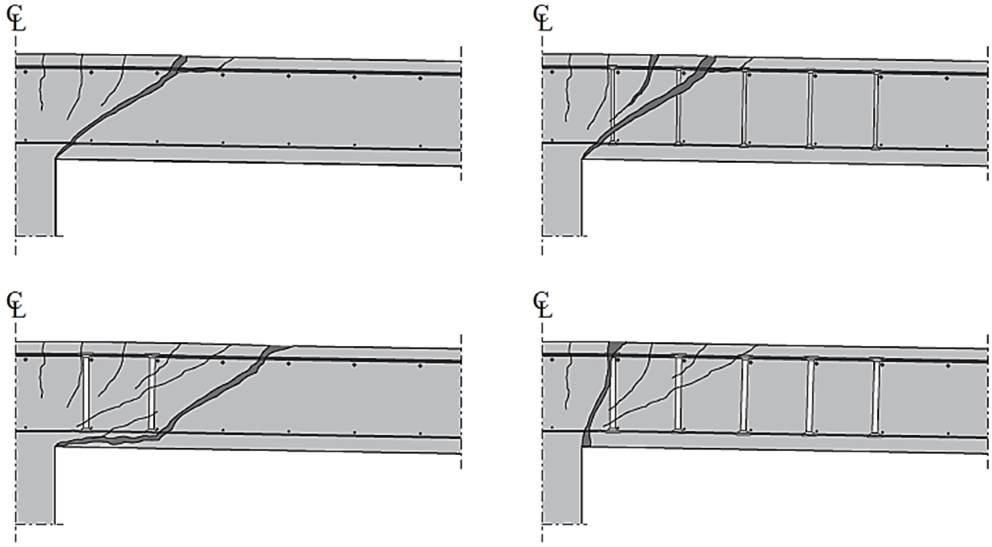


Figure 5: Punching shear failure modes: (top left) failure without shear reinforcement; (top right) failure inside the shear-reinforced zone; (bottom left) punching outside the shear-reinforced zone; (bottom right) failure of the compression strut at maximum resistance, adapted from [9]

At times, punching shear failure at one slab-column connection may trigger similar failures at other parts of the slab where multiple loaded areas penetrate the concrete slab, thereby compromising its structural integrity and leading to a progressive collapse that poses serious safety risks to the inhabitants, as demonstrated by several failures in the past:

- 1995 – Sampoong department store in Seoul, Korea, claimed over 500 lives and injured over 900.
- 1997 – Piper Row Car Park in Wolverhampton, UK, fortunately claimed no lives despite being in use.
- 2004 – Underground parking garage in Gretzenbach, Switzerland, claimed seven firefighters.
- 2021 – Champlain Towers in Florida, USA, claimed the lives of 98 residents.

2.2.3 Punching shear resistance models

Depending on the load and design, punching shear failure may derive from the previously mentioned failure modes and continues to attract intense efforts to further investigate this phenomenon, leading to the current state-of-the-art. Investigations typically consider an isolated slab element that represents the slab surface surrounded by a column, delimited by the line of contraflexure for radial moments [11]. The result of these investigations led to the development of several models [9], classified as:

- Empirical or semi-empirical.
- Based on linear or non-linear fracture mechanics.
- Based on the theory of plasticity.
- Kinematic failure mechanisms.

While each of these models capture the complex failure mechanisms, the (semi)-empirical models are the easiest to apply in design practice as they sufficiently capture the main influencing parameters (despite limitations with the previously evaluated test data) and are the foundation for punching shear verifications with and without punching shear reinforcement in design standards such as EN 1992-1-1:2004 [12]. In such standards, for the truss model to function reliably for punching shear, any provided reinforcement must enclose (or hook) around the compression chord as a tension tie to allow the transfer of forces in the node. Achieving this requirement in practice is possible via: bond, the concrete's tensile strength or, most commonly, through direct supports where the shear reinforcement bends with or without the presence of longitudinal reinforcement in the compression zone [13].

3. Designing concrete members for punching shear

3.1 General principles

Design distinguishes between concrete members with and without punching shear reinforcement, such as floor slabs supported by columns and columns resting on isolated and mat footings, which typically have slender cross-sections and are subject to concentrated loads. To design both types of members, the design approach described in the first generation of EN 1992-1-1:2004 [12] adopts an empirically derived formulation [14] that is similar to the approach for one-way slabs and beams failing in shear. The formulations are practical and maintain a consistent resistance model for both shear and punching shear verifications with minor differences in key design parameters, chief amongst them being that the variable strut angle used for shear resistance verifications ($1 \leq \cot\theta \leq 2.5$) is replaced by a fixed compressive strut angle ($\tan\theta = 0.5$) for punching shear resistance verifications.

The following sections describe the approach a designer would take to verify the requirement for punching shear reinforcement with the common text in Section 6.4 of EN 1992-1-1:2004. This section of the Eurocode contains several Nationally Determined Parameters (NDPs) and Non-Contradictory Complementary Information (NCI) in the country-specific National Annexes (NAs), such as the German DIN EN 1992-1-1/NA [15] and the Austrian ÖNORM B 1992-1-1 [16]. The former is covered below due to its significantly more thorough nature.

3.2 The loaded area and the control section for slabs and foundations

3.2.1 The loaded area, u_0

Prior to conducting resistance verifications in EN 1992-1-1:2004, Sections 6.4.1 and 6.4.2 [12] require knowledge of both the lengths of the loaded area, u_0, and the control section, u_1, respectively. Referring to Figures 6.12 and 6.13 of [12], the former relates to specific areas of the compression member (column or wall) on which load is applied, typically modified by the position of the column or wall in relation to the slab or foundation; for instance, not all faces of a column are loaded if the column is positioned at the slab's edge.

For rectangular columns with large sections (aspect ratios $a/b > 2$), the resistance to punching shear only develops fully if the loaded area is small enough to generate a triaxial stress in the concrete, which implies that punching resistance will not develop over the entire cross-section of the column, but rather only at specific parts, thereby separating the cross-section into regions of shear and punching shear, as illustrated by Figure 6.

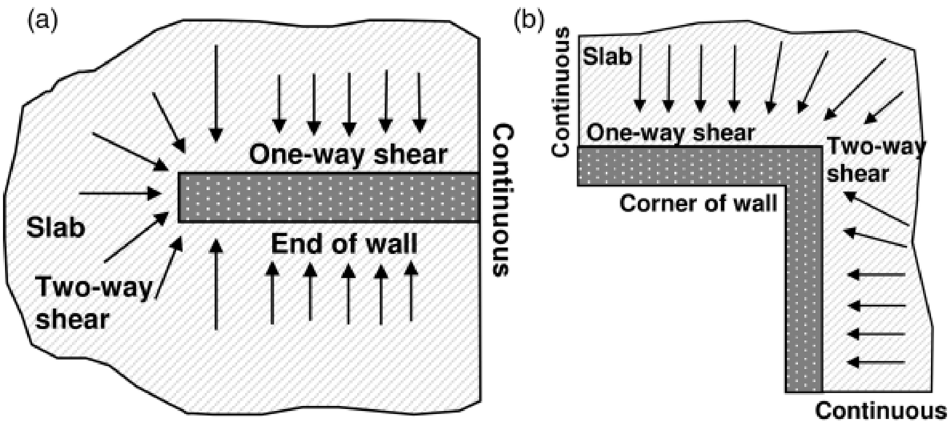


Figure 6: Shear and punching shear forces transferred from the slab to the support at wall ends (left) and wall corners (right), reproduced from Figure 1 [17].

While the common text of [12] does not contain such provisions, leaving designers to employ their own judgement for such cases, DIN EN 1992-1-1/NA [15] provides additional recommendations by limiting the loaded area to this aspect ratio, following which Section 6.2.2 [15] can be used to verify the resistance to shear. Additionally, the National Annex applies a ratio of $u_0/d_{ef} \leq 12$ (i.e., $u_0/4 \leq 3d_{ef}$ per corner), where the slab's effective depth is d_{ef}, as illustrated by Figure 7. This rationale also extends to columns with reinforced column heads (also known as "drop panels").

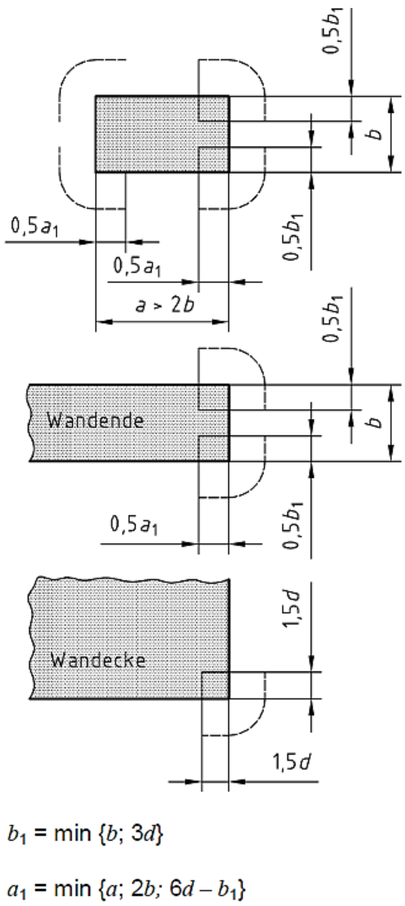


Figure 7: Loaded areas and control sections used for punching shear verifications large columns (top), ends of walls (center), and corner of walls (bottom), reproduced from Fig. NA.6.12.1 [15]

Columns with circular sections and $u_0/d_{ef} > 12$ employ a similar rationale, requiring checks of shear force distribution along the circumference of the column. For punching shear verifications, however, this requires reductions to the empirical pre-factor, $C_{Rd,c}$, and is discussed further in the following sections.

3.2.2 The control section area, u_1

For **slabs**, using a fixed strut angle of $\tan\theta = 0.5$ implies that the control section for the verification of punching shear resistance is set at a distance of $2d_{ef}$ from the edge of the loaded area and its length, u_1 (or u_{crit}). Constructed to minimize the length, it typically follows the shape of the loaded area, u_0, determined from the previous section. Exceptions include slabs that cantilever beyond the edge of the loaded area. As illustrated by Figure 6.14 [12], the length of the control section, u_1, is reduced by the presence of openings.

For **foundations**, the length of the control section u_1 is not set at $2d_{ef}$, but rather the length bound by a variable distance a_{crit} that must be determined iteratively using the smallest ratio of the design punching stress to the resistance, $v_{Rd,c}/v_{Ed}$. The distance a_{crit} may be set at $1.0d_{ef}$ as a simplification for slender foundations with a shear span-to-depth ratio $\lambda = a_\lambda/d_{ef} > 2$, where a_λ is the ratio of the distance to the smallest edge (or to the point of contraflexure) and d_{ef} is the effective depth [15]. Foundation slenderness has a proportional impact on a_{crit}, with higher slenderness increasing a_{crit} and decreasing the design punching stress. Conversely, in squat foundations where $\lambda = a_\lambda/d_{ef} \leq 2$, steeper inclinations of the failure crack reduce a_{crit}. Consequently, the proportion of soil pressure opposing the design punching stress is affected by the area bound by a_{crit}.

3.2.3 Column heads

Section 6.4.2 of EN 1992-1-1:2004 requires verification for punching shear resistance either within and beyond, or only beyond the column head depending on its slenderness ratio l_H/h_H (see Figure 6.17 [12]), where squat column heads $l_H/h_H < 2$ require verification only beyond the head and slender column heads $l_H/h_H \geq 2$ require verification both within and beyond the head.

DIN EN 1992-1-1/NA reduces the slenderness limit to $l_H/h_H < 1.5$ and introduces an additional verification for column heads with slenderness ratios between $1.5 < l_H/h_H < 2.0$ to rule out any potential failures from crack inclinations between 30°-35°, illustrated below in Figure 8. These also applies to checks for columns with heads on foundations.

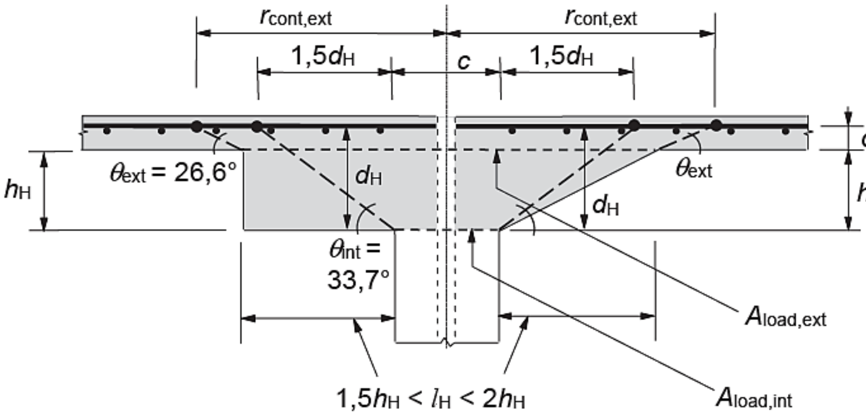


Figure 8: Additional verification for column heads with slenderness ratios between $1.5 < l_H/h_H < 2.0$, reproduced from Figure H6-33 [13]

3.3 Verification for punching shear resistance to EN 1992-1-1 & DIN EN 1992-1-1/NA

3.3.1 Control section for verification

As previously mentioned in Section 2.2.2 of this document, the inclined shear cracks propagate from the tensile zone of the slab towards the compression zone where the slab soffit meets the loaded area. Since one of these cracks – termed as the critical shear crack – intercepts the compression strut near the loaded area, it thus determines the control section used to verify the resistance of the compression strut.

In its main text, EN 1992-1-1:2004, 6.4.3 (2) requires conducting three verifications at different control sections, which are detailed in Table 1 and compared to the control sections required by DIN EN 1992-1-1/NA.

Table 1: Control sections used for the verifications for punching shear according to EN 1992-1-1 and DIN EN 1992-1-1/NA

	Control section used in:	
Verification for:	**EN 1992-1-1**	**DIN EN 1992-1-1/NA**
Maximum punching resistance, $\boldsymbol{v_{Ed} \leq v_{Rd,max}}$	u_0	u_1
Requirement for punching shear reinforcement, $\boldsymbol{v_{Ed} \leq v_{Rd,c}}$	u_1	
Limits of punching shear reinforcement, $\boldsymbol{v_{Rd,cs} \leq k_{max} v_{Rd,c}}$		

Summarized in Table 2, the design punching stress, $\boldsymbol{v_{Ed}}$, and the strut crushing limits, $\boldsymbol{v_{Rd,max}}$, are determined by:

Table 2: Differences in the evaluation of design stress and maximum resistance according to EN 1992-1-1 and DIN EN 1992-1-1/NA

EN 1992-1-1	DIN EN 1992-1-1/NA
$\boldsymbol{v_{Ed} = \frac{\beta \cdot V_{Ed}}{u_0 \cdot d_{ef}} \leq v_{Rd,max}}$	$v_{Ed} = \frac{\beta \cdot V_{Ed}}{u_1 \cdot d_{ef}} \leq v_{Rd,max}$
$\boldsymbol{v_{Rd,max} = 0.4\, v \frac{\alpha_{cc} f_{ck}}{\gamma_c}}$, with $\boldsymbol{v = 0.6(1 - \frac{f_{ck}}{250})}$	$v_{Rd,max} = 1{,}4 \cdot v_{Rd,c}$

3.3.2 Load eccentricity factor, $\boldsymbol{\beta}$

Eq. (6.38) of [12], reproduced in Table 2, converts the design shear force into stress at the control perimeter. The equation introduces a load eccentricity factor, β, that accounts for uniaxial or biaxial bending that unevenly distributes the shear force and increases stress around one side of the control perimeter. It also accounts for the eccentricity between the column centroid and the centroid of the control section bound by u_1.

EN 1992-1-1:2004 distinguishes between two approaches to calculate the β-factor loosely termed as:

- **"Approximated"**: values in EN 1992-1-1:2004, 6.4.3 (6) & the various National Annexes are valid only where the lateral stability of the structure does not rely on the frame action between the slab and columns and in which the two spans do not differ in length by more than 25%, and are reproduced in Table 3.

Table 3: Approximated values of the load eccentricity factor for EN 1992-1-1 and DIN EN 1992-1-1/NA

Design Standard	Approximated values of β				
/ National Annex	Inner Column	Edge Column	Corner Column	Wall Corner	Wall End
EN 1992-1-1	1.15	1.4	1.5	-	
DIN EN 1992-1-1	1.10			1.20	1.35

- **"Refined"**: more precise values of the load increase factor are evaluated using a fully plastic shear stress distribution approach. Here, a certain portion of the moment, M_{Ed}, generates additional shear stresses in the control section, further magnified since increasing the column dimension perpendicular to the moment axis, c_1, also increases shear stresses in the control section as demonstrated in Figure 6.19 of [12], reproduced in Figure 9. The remaining portion of the moment is transferred into the column via bending and torsion. The moment of resistance, $\boldsymbol{W_1}$, is determined along the control section, u_1, according to Eq. (6.40) [19]. Reproduced in Table 4, DIN EN 1992-1-1/NA introduces Eq. (NA.6.39.1) that enables an accurate evaluation of the β-factor in case of biaxial eccentricity as a vector sum, shown in Table 4.

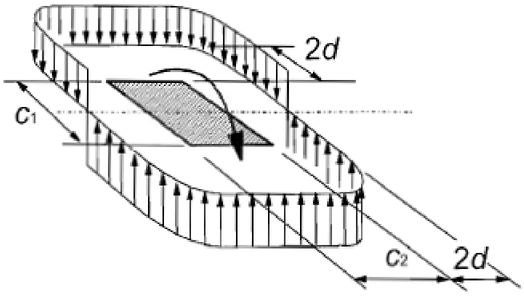


Figure 9: Shear distribution from unbalanced moments with the span, c_1, perpendicular to the moment axis, from Fig. 6.19 [12]

EN 1992-1-1	DIN EN 1992-1-1/NA
$\boldsymbol{\beta = 1 + k \cdot \frac{M_{Ed}}{V_{Ed}} \cdot \frac{u_1}{W_1}}$	$\beta = 1 + \sqrt{\left(k_x \cdot \frac{M_{Ed,x}}{V_{Ed}} \cdot \frac{u_1}{W_{1,x}}\right)^2 + \left(k_y \cdot \frac{M_{Ed,y}}{V_{Ed}} \cdot \frac{u_1}{W_{1,y}}\right)^2}$

Table 4: Evaluation of the load eccentricity factor according to EN 1992-1-1 and DIN EN 1992-1-1/NA

3.3.3 Verification without punching shear reinforcement

EN 1992-1-1:2004, 6.4.4 contains the following resistance verification of slabs and foundations without punching shear reinforcement, with the NDPs in DIN EN 1992-1-1 marked in bold red text and detailed in Table 5. Thus, for slabs:

$$v_{Rd,c} = max\left[\boldsymbol{C_{Rd,c}} k(100\rho_l f_{ck})^{\frac{1}{3}}, \boldsymbol{v_{min}}\right] + \boldsymbol{k_1}\sigma_{cp} \quad \text{(in N/mm}^2\text{)} \quad (1)$$

Parameter	EN 1992-1-1	DIN EN 1992-1-1/NA
$\boldsymbol{C_{Rd,c}}$	$C_{Rd,c} = 0.18/\gamma_c$	Slabs in general: $C_{Rd,c} = 0.18/\gamma_c$ Slab-inner columns with $\frac{u_0}{d_{ef}} < 4$: $C_{Rd,c} = \frac{0.18}{\gamma_c}\left(0.1 \cdot \frac{u_0}{d} + 0.6\right)$ Foundations: $C_{Rd,c} = 0.15/\gamma_c$ Slab-circular columns with $\frac{u_0}{d_{ef}} > 12$: $C_{Rd,c} = \left(\frac{12d}{u_0}\right) \cdot \frac{0.18}{\gamma_c} \geq \frac{0.15}{\gamma_c}$
$\boldsymbol{v_{min}}$	$v_{min} = \frac{0.0525}{\gamma_c} k^{3/2} f_{ck}^{1/2}$	For $d_{ef} \leq 600mm$, $v_{min} = \frac{0.0525}{\gamma_c} k^{3/2} f_{ck}^{1/2}$ For $d_{ef} > 800mm$, $v_{min} = \frac{0.0375}{\gamma_c} k^{3/2} f_{ck}^{1/2}$ Linear interpolation permitted for $600mm \leq d_{ef} < 800mm$
$\boldsymbol{k_1}$	$k_1 = 0.10$	

Table 5: Nationally determined parameters (NDPs) for Eq. (6.47) in EN 1992-1-1 and DIN EN 1992-1-1/NA

For **foundations**, $v_{Rd,c}$ is modified by the ratio $2d_{ef}/a_{crit}$ that arises from the more compact dimensions, particularly of isolated footings, and the interaction between them and the soil, resulting in:

$$v_{Rd,c} = max\left[C_{Rd,c} k(100\rho_l f_{ck})^{\frac{1}{3}}, v_{min}\right] \cdot \frac{2d_{ef}}{a_{crit}} \quad \text{(in N/mm}^2\text{)} \quad (2)$$

The distance to the control perimeter, a_{crit}, is determined iteratively using the smallest ratio of $v_{Rd,c}/v_{Ed}$, and provides the possibility to deduct the entirety of the relieving soil pressure, σ_{gd}, in the net upwards force, ΔV_{Ed}, in Eq. (6.48) [12] to calculate $V_{Ed,red}$. This results in a higher resistance to punching shear resistance, rather than using an approximation. Detailed in [15] for slender and squat foundations ($\lambda = a_\lambda/d_{ef} > 2$), a simplified calculation using $a_{crit} = 1.0d_{ef}$ may instead be used, but enables a deduction of only half the soil pressure in ΔV_{Ed}. To evaluate the slenderness, a_λ uses either the smallest edge distance from the column face to the foundation's edge or the smallest distance to the point of contraflexure (typically $0.22L_x$). In both cases, the favorable effect of soil pressure acts only within the area bound by a_{crit}, as illustrated in Figure 10.

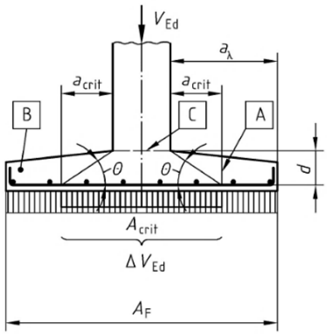


Figure 10: Reduction of the soil pressure inside the area bound by the control section, A_{crit}, reproduced from Fig. NA.6.21.1 [15]

3.4 Design with punching shear reinforcement to EN 1992-1-1 & NA-DE

A key feature of punching shear design that distinguishes itself from the shear design of beams and linearly supported one- and two-way slabs is that when concrete by itself cannot resist all acting punching shear stresses, any provided punching shear reinforcement will supplement this resistance. Based on strut-and-tie models, design standards like EN 1992-1-1:2004 and its National Annexes prescribe that a certain resistance from concrete can be activated.

The provision of punching reinforcement, most notably via stirrups but also with double-headed studs, is the most suitable solution to enhance the resistance and deformation of flat-slabs and, depending upon the amount and detailing of the shear reinforcement, three distinct failure modes govern the design:

1. Failure inside the shear-reinforced zone.
2. Failure due to crushing of the concrete struts.
3. Failure outside the shear-reinforced zone.

3.4.1 Failure inside the shear-reinforced zone, $\boldsymbol{V_{Rd,cs}}$

Where design demands punching shear reinforcement, several factors determine the amount of reinforcement required:

1. Contribution from concrete without punching shear reinforcement, $\boldsymbol{V_{Rd,c}}$.
2. Minimum cross-section per reinforcing element (e.g., stirrup) to avoid yielding of the shear reinforcement when the first shear cracks develop, $\boldsymbol{A_{sw,min}}$.
3. Total punching shear reinforcement required per perimeter to carry the design load, $\boldsymbol{A_{sw,crit}}$.
4. Inclination of the provided reinforcement, $\boldsymbol{\theta}$.
5. Effective depth of the section, $\boldsymbol{d_{ef}}$.
6. Radial spacing between the perimeters, $\boldsymbol{s_r}$, as a function of $\boldsymbol{d_{ef}}$.

EN 1992-1-1:2004 determines the amount of punching shear reinforcement with the "strut-and-tie" or "stress field model", using a shallower fixed strut inclination of ~33° (from $cot\,\theta = 1.5$). Thus:

- Effective yield stress: $f_{ywd,ef} = 250 + 0{,}25 \cdot d_{ef} \leq f_{ywd}$ (3)
- Effective yield force per stirrup: $F_{sw,i} = A_{sw} \cdot f_{ywd,ef}$ (4)
- Number of stirrups per perimeter: $A_{sw,i} = n \cdot A_{sw}$ (5)
- Forces in all stirrups at u_1: $\boldsymbol{V_{Rd,s} = f_{ywd,ef} \cdot A_{sw,crit} \cdot \frac{d_{ef} \cdot \cot\theta}{s_r}}$ (6)
- If stirrups are inclined ($\alpha \neq 90°$): $V_{Rd,s} = \left[1.5 \cdot f_{ywd,ef} \cdot A_{sw,crit} \cdot \frac{d_{ef}}{s_r}\right] \cdot \sin\alpha$ (7)
- With contribution from concrete: $V_{Rd,cs} = 0.75 v_{Rd,c}(u_1 d_{ef}) + \left[1.5 f_{ywd,ef} A_{sw,crit} \frac{d_{ef}}{s_r}\right]$ (8)
- Minimum punching reinforcement: $A_{sw,min} = 0.08 \frac{\sqrt{f_{ck}}}{1.5(f_{ywd} \cdot \gamma_s)}(s_r \cdot s_{t,max})$ (9)

The required amount of punching shear reinforcement in Eq. (8) can be determined by equating $V_{Rd,cs} = V_{Ed}$ and rearranging the equation to directly evaluate the total reinforcement required, $A_{sw,crit}$:

$$A_{sw,crit} = \frac{V_{Ed} - 0.75 v_{Rd,c}(u_1 d_{ef})}{1.5 f_{ywd,ef}\left(\frac{d_{ef}}{s_r}\right)} \quad (\text{mm}^2) \qquad (10)$$

For both slabs and foundations, this reinforcement must then be placed in all reinforcing perimeters within the shear-reinforced zone.

DIN EN 1992-1-1/NA [15] introduces several additional provisions to Section 6.4.5 of [12]:

1. The favorable impact of any prestress, $\boldsymbol{\sigma_{cp}}$, considered in $v_{Rd,c}$ from Eq. (1) is limited to $0.5 \cdot k_1 \cdot \sigma_{cp}$, where σ_{cp} cannot be larger than 2 MPa, thus resulting in:

$$V_{Rd,cs} = 0.75\left[v_{Rd,c} + 0.5 \cdot k_1 \cdot min(\sigma_{cp}; 2)\right] \cdot (u_1 d_{ef}) + \left[1.5 f_{ywd,ef} A_{sw,crit} \frac{d_{ef}}{s_r}\right] \quad (\text{kN}) \qquad (11)$$

2. An increase to the required punching shear reinforcement in the first two reinforcing perimeters as opposed to the same reinforcement: the magnification factors, $\boldsymbol{\kappa_{sw,1} = 2.5}$ and $\boldsymbol{\kappa_{sw,2} = 1.4}$, are applied to $A_{sw,crit}$ only for the **first** and **second** reinforcing perimeters, respectively.
3. Punching shear verification of foundations uses a modified approach to reflect the **steeper inclinations** of the shear cracks, requiring the shear-reinforced zone and any punching shear reinforcement to be closer to the support. The approach excludes any contribution to the punching shear resistance from concrete, $v_{Rd,c}$, and includes only the contribution from the punching reinforcement provided in the first two rows, $\boldsymbol{A_{sw,1+2}}$, that is equally split between the two perimeters that must be positioned between $0.3d_{ef}$ and $0.8d_{ef}$ from the support face. Any subsequent reinforcing perimeters (third, fourth, and so on) do not contribute to the overall resistance and therefore only require provision of **33%** of $A_{sw,1+2}$ per perimeter. Thus:

$$\beta \cdot V_{Ed,red} \leq V_{Rd,s} = f_{ywd,ef} \cdot A_{sw,1+2} \quad (\text{kN}) \qquad (12)$$

3.4.2 Failure due to crushing of the concrete struts, $V_{Rd,max}$

Similar to the design provisions for one-way shear in 6.2.2 [12], the maximum punching shear resistance with and without punching reinforcement, $\boldsymbol{v_{Rd,max}}$, is limited to the resistance of the compression struts at the support periphery, u_0, and consists of the design compressive strength of concrete, f_{cd}, and the strength reduction factor for concrete cracked in shear, ν, from Eq. (6.6N) [12]:

$$v_{Ed} = \frac{\beta V_{Ed}}{u_0 d_{ef}} \leq 0.4 \nu f_{cd} = v_{Rd,max} \quad (\text{N/mm}^2) \qquad (13)$$

Since Eq. (13) significantly overestimates the maximum punching shear resistance [18], the 2014 A1 Amendment to EN 1992-1-1:2004 [19] included an additional limit on the punching shear resistance of slabs and foundations **with** punching reinforcement as a factor of $v_{Rd,c}$ (evaluated at the control section u_1), leading to:

$$v_{Rd,max} = 1.5 \cdot v_{Rd,c} \qquad \text{(N/mm}^2\text{)} \qquad (14)$$

Note 1: The factor marked in red (1.5) is a Nationally Defined Parameter and several National Annexes may define higher values.

DIN EN 1992-1-1/NA introduces several changes in line with Eq. (14) that determines the maximum punching resistance as a factor of the punching resistance without punching reinforcement evaluated at the control section, u_1. For slender slabs, failure of the compression strut near the periphery of the support area is not as decisive as the failure of the concrete compression zone because the triaxial stress is significantly influenced by the slab rotation and the permissible crack width, both of which are controlled by the longitudinal reinforcement ratio, ρ_l. Moreover, the small depth of the compression zone and incomplete confinement by the punching shear reinforcement at the periphery of the loaded area causes the concrete cover to spall well before the maximum compressive strut resistance is reached [18]. Eq. (NA.6.53.1) of DIN EN 1992-1-1/NA thus limits the maximum resistance to:

$$v_{Rd,max} = 1.4 \cdot v_{Rd,c} \qquad \text{(N/mm}^2\text{)} \qquad (15)$$

Note 2: Although not explicitly specified in EN 1992-1-1:2004, the evaluation of $v_{Rd,max}$ according to DIN EN 1992-1-1/NA cannot consider any contribution from the axial stress, σ_{cp}, due to a lack of experimental evidence.

For foundations, the control section, u_1, is not determined at $2d_{ef}$, but iteratively at a_{crit} for all verifications, including for $v_{Rd,max}$.

3.4.3 Failure outside the shear-reinforced zone, $V_{Rd,c,out}$

Determining the extent of the shear-reinforced zone is crucial to prevent punching failure outside this zone. The provision of additional rows expands the zone until the concrete can, by itself, resist the applied stress. Thus, resistance provided by concrete at the outer perimeter, $\boldsymbol{V_{Rd,c,out}}$, is determined by:

$$\beta \cdot V_{Ed} \leq V_{Rd,c,out} = v_{Rd,c,out} \cdot u_{out} \cdot d_{ef} \qquad \text{(kN)} \qquad (16)$$

The extent of this zone is determined by the distance from the support area, u_0, to the outer perimeter where punching reinforcement is **not** required, $\boldsymbol{u_{out}}$. The latter's length is determined by equating $\beta V_{Ed} = V_{Rd,c,out}$ and rearranging Eq. (16):

$$u_{out} = \frac{\beta \cdot V_{Ed}}{v_{Rd,c,out} \cdot d_{ef}} \qquad \text{(mm)} \qquad (17)$$

As mentioned previously in Section 3.4.1 and Eq. (8), a fixed strut inclination of ~33° (from $\cot\theta = 1.5$) means that the reinforcing perimeter furthest from the support must be positioned within a distance of kd_{ef} from u_{out}, as highlighted by Figure 11. The value of $\boldsymbol{k}$ in the main text of EN 1992-1-1:2004 and most National Annexes is 1.5.

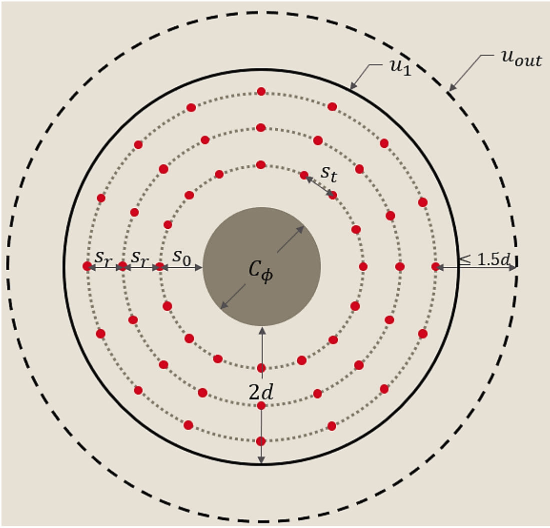


Figure 11: Schematic image of a punching reinforcement layout, with the outermost reinforcing perimeter within the distance "1.5d"

Determining the resistance and the required number of reinforcing perimeters is possible using two distinct approaches:

1. The length of u_{out} is evaluated according to Eq. (17) and the required number of reinforcing perimeters is then determined by using the maximum radial spacing, $s_r = 0.75d_{ef}$, that also maximizes the reinforcement per perimeter (see Eq. (10)), but optimizes the required perimeters.
2. In this more iterative approach, an appropriate radial spacing $s_r \leq 0.75d_{ef}$ is chosen and an outer perimeter, u_{out}, is positioned at $1.5d_{ef}$ beyond each reinforcing perimeter and verification is conducted to ensure that $\beta \cdot V_{Ed} \leq V_{Rd,c,out}$ (see Eq. (16)). More reinforcing perimeters are positioned until the verification is successful.

Regardless of the approach, a minimum of two reinforcing perimeters must be provided [13].

The National Annex DIN EN 1992-1-1/NA introduces minor modifications to the evaluation of the shear resistance $v_{Rd,c,out}$ used in Eq. (17) by replacing the variables $C_{Rd,c}$ and k_1 found in Eq. (1) with those found in NDP to 6.2.2 (1) [15] for linearly supported slabs, thus:

$$v_{Rd,c,out} = max\left[\frac{\mathbf{0.15}}{\gamma_c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{\frac{1}{3}}, \boldsymbol{v_{min}}\right] + \mathbf{0.12} \cdot \sigma_{cp} \qquad (\text{N/mm}^2) \qquad (18)$$

4. Approaches to strengthen members deficient in punching shear

A significant majority of building stock with flat-slab structures built in the past 50-70 years today requires strengthening against punching stemming from several reasons, for instance initial design or execution errors, environmental deterioration / corrosion, changes in use, and so on. Inadequately addressing these reasons with the appropriate strengthening techniques has resulted in a few notable episodes of failure across the world by partial or total collapse [20], mentioned previously in Section 2.2.2.

Five key parameters govern the resistance of a concrete slab or foundation against punching shear:

a) Concrete strength, f_{ck}.
b) The effective depth, d_{ef}, to the flexural reinforcement from the compression fiber.
c) Size of the support, u_0, and control perimeter, u_1.
d) The amount of longitudinal reinforcement, ρ_l.
e) The amount of punching shear reinforcement in each reinforcing perimeter, A_{sw}.

The various methods or interventions typically employed to strengthen individual concrete members enhance the member's shear resistance, yet incur a trade-off in terms of invasiveness, cost, availability, and other secondary parameters. Although improving one or several of these parameters enhances punching shear resistance, the concrete strength *(a)* in an existing structure cannot be modified *a posteriori*. Introducing new supports is generally unfeasible as these supports will need to transfer load to the foundations while imposing loads on other members that may also require strengthening. Depending on functional requirements, enhancing one or more parameters *(b)* to *(f)* by using different interventions is possible, as shown in the following subsections. Typically, only a part of the strengthening interventions is performed with proprietary products and, more often, solutions are tailored to the project at hand and combined where feasible.

4.1 Increasing the slab thickness

Employing a **concrete overlay** increases the section height, h, and the effective depth, d_{ef}, of floor slabs and foundations. As illustrated in Figure 12, this approach simultaneously enhances the flexural resistance and the stiffness, thereby also reducing deflection, and is useful when punching shear resistance is not the only deficiency to address. In scenarios where members require strengthening solely for punching shear, both approaches may have notable drawbacks:

1. The concrete overlay adds a substantial additional weight that affects other members in the load path, including the foundation.
2. Moreover, the increase in effective depth is less than the thickness of the overlay, with the resulting effective depth resting in the center of gravity of all flexural reinforcement in both the existing concrete and the overlay, i.e., below the flexural reinforcement of the overlay.

Examples of proprietary solutions in the industry:

Hilti HCC- series: HCC-K, HCC-B, HCC-HUS4, and HCC-U.

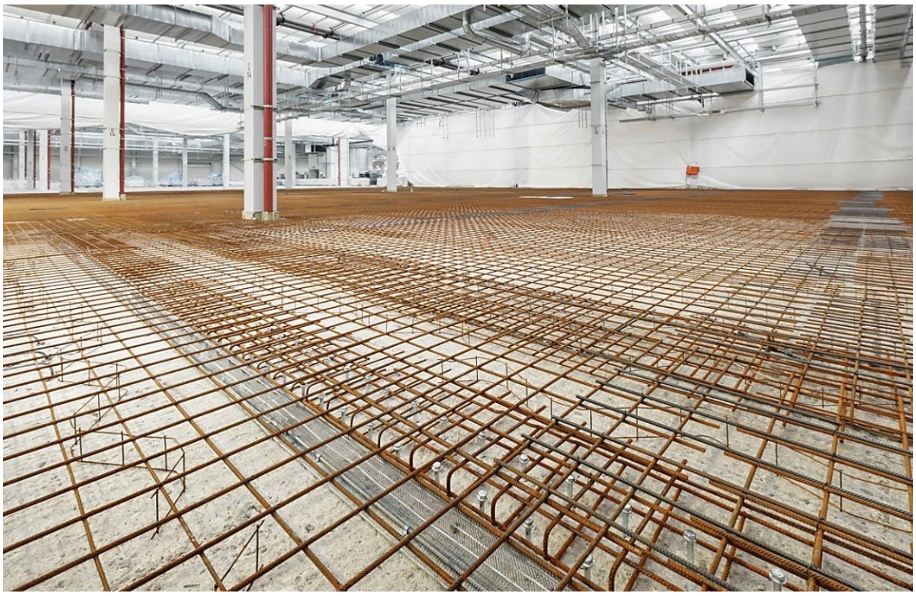


Figure 12: Example of post-installed reinforcement used in concrete overlays

4.2 Increasing the size of the support area

Increasing the size of the support (column or wall) by employing a **concrete jacket**, as exemplified in Figure 13, increases the stiffness and compressive resistance of the column, which is useful when additional loads, such as from a change in use, necessitate the strengthening of the existing column. A larger support distributes the concentrated load, V_{Ed}, over a larger area, A_{load}, and consequently reduces the design punching shear stress, v_{Ed}. For this technique to be effective, the size of the enlarged column or wall must significantly increase the column perimeter, u_0, and thus the control section, u_{crit}.

However, solely increasing the column cross-section to increase punching resistance requires that the columns in the floors below are also enlarged and holes must be drilled through the slab to allow for the positioning of longitudinal reinforcement, which then must be anchored securely in the foundation. A more effective approach to increase the support area is by employing post-installed steel collars (consisting of beams) or concrete column heads (or drop panels), illustrated in Figure 14.

Examples of solutions in the industry: post-installed steel or concrete column head or drop panel; concrete jacketing of the column.

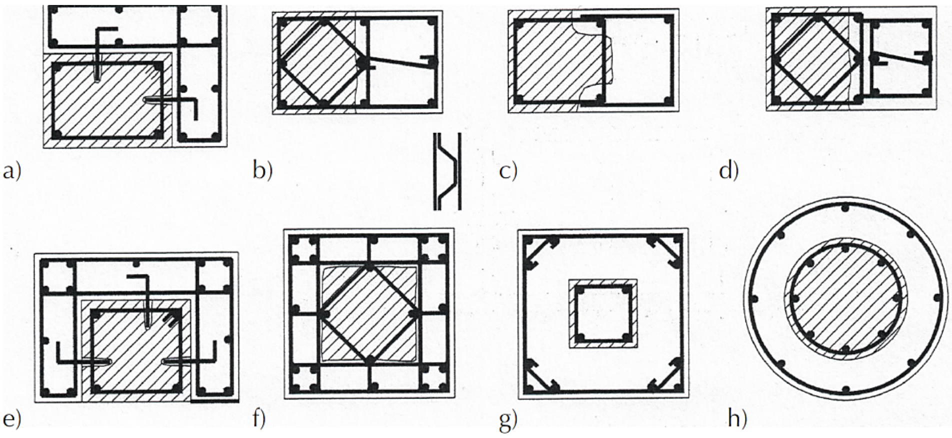


Figure 13: Examples of concrete jacketing, reproduced from [2]

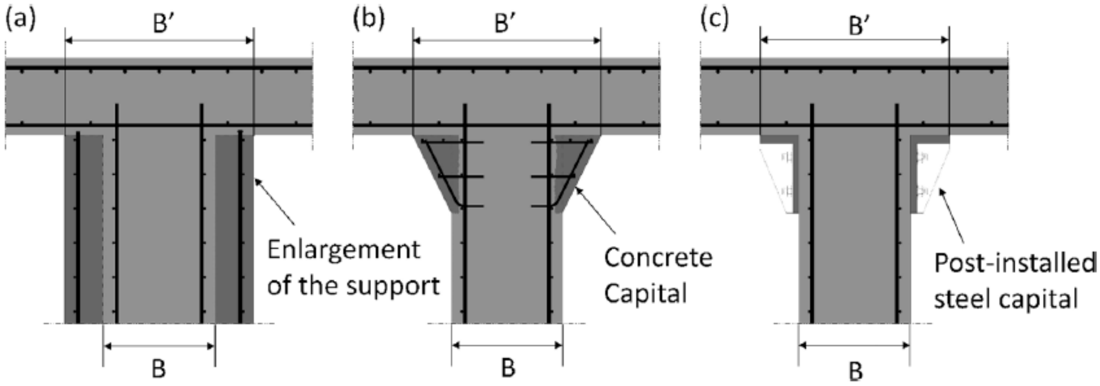


Figure 14: Enhancing punching shear resistance by: (a) column jacketing; (b) casting a new concrete column head; (c) post-installing a new steel capital, reproduced from [21]

4.3 Increasing the flexural resistance

Increasing the amount of flexural reinforcement enhances the section stiffness and reduces crack widths by improving aggregate interlock over the cracks and reduces slab rotation, which in turn increases the shear resistance. Illustrated by Figure 15, enhancements to flexural reinforcement are possible by applying glued laminates or installing near-surface-mounted reinforcement at the supports where flexural demand is the highest, with the reinforcement consisting of glass (GFRP) or carbon (CFRP) fiber-reinforced polymers or steel plates.

The effect of increasing flexural resistance has an "under-proportional" effect on shear resistance; for instance, doubling the amount of flexural reinforcement per Eq. (6.47) of EN1992-1-1:2004 results in the shear resistance, $V_{Rd,c}$, increasing by not more than 26%. Moreover, the deformation capacity is reduced due to the higher stiffness that increases the danger of progressive collapse.

Examples of solutions in the industry: glued or mechanically fastened CFK laminates, memory steel laminates, memory steel bars, near-surface-mounted reinforcement.

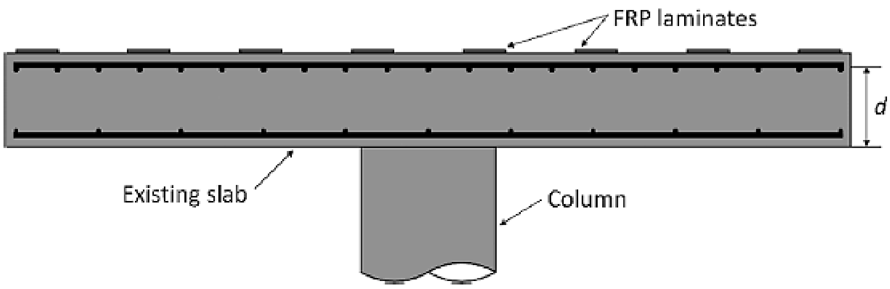


Figure 15: Cross-section of a slab strengthened using FRP strips, reproduced from [22]

4.4 Increasing punching shear resistance using steel reinforcement

Alternatively, another solution involves drilling holes through the concrete member on both sides and fixing threaded steel rods with a nut and washers, also understood as "through-bolting". Filling the annular gap between the threaded rod and the borehole with a suitable mortar is essential for engaging the reinforcement as the concrete cracks. This helps maintain the width of the cracks within serviceability limits and prevents corrosion of the reinforcement, which is crucial for ensuring the design's intended service life. As with post-installed reinforcement, drilling through the concrete member entails risks of cutting or damaging longitudinal reinforcement, which is particularly dense near the supports (typically rigid supports) where flexure demands are high. Mitigating this risk is possible by using ferro-scanners that aid in the detection of the flexural reinforcement on both sides of the member prior to drilling.

In most scenarios, nevertheless, drilling through the slab either is not possible or is not desired due to issues stemming from a lack of accessibility or the desire to maintain interior aesthetics, leading to a partially embedded installation of the strengthening elements from one side. This approach is less invasive than drilling through the full length of the concrete section but contains a stipulation: detailing rules in all modern standards, such as Section 9.2.2 of EN1992-1-1, require standard shear reinforcement such as stirrups to enclose and “confine” the longitudinal reinforcement or, at least, anchor at or beyond the longitudinal reinforcement layers. This means the only possible failures are the yielding of steel or crushing of the concrete struts. However, such anchorage may not be possible here and, therefore, requires a verification of the anchorage and the installation, in general based on specific tests wherever possible.

Currently available Hilti solutions: Figure 16 shows three different options of using HZA-P and the HAS-U rods embedded with epoxy RE 500 either partially (HZA-P and HAS(-U)) and through (only HAS(-U)) the full height of the section.

Examples of solutions in the industry: CFRP laminates, through bolts, concrete screws installed from one side, adhesive / undercut anchors installed from one side.

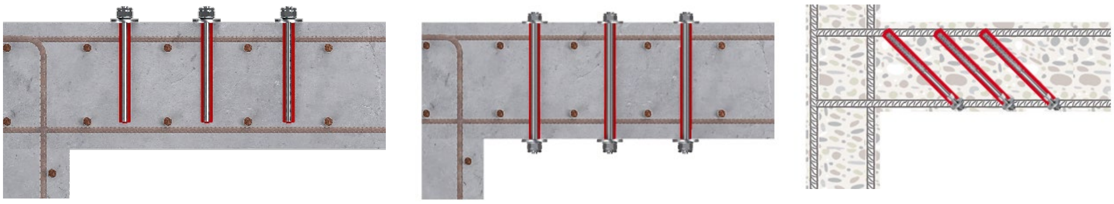


Figure 16: Increasing shear reinforcement using: (left) partially embedded HAS(-U) rods installed perpendicular to the beam length; (middle) through-bolted HAS(-U) installed perpendicular to the beam length; and (right) partially embedded HZA-P inclined to the beam length

4.5 Special solutions & combinations

When loads are exceptionally high, special solutions or combinations of the previously mentioned solutions can be applied. An example of a special solution is a carbon-fiber laminate that is installed through two inclined holes and prestressed, as opposed to a normal installation without specially created holes.

Figure 17 illustrates another example that may significantly enhance the punching shear resistance and combines post-installed punching shear reinforcement with a concrete overlay. Another combination that does not increase the slab or foundation thickness may include fiber laminates with post-installed punching shear reinforcement to meet the respective flexure and shear demands. Additional checks for strain compatibility may be required to ensure the system behaves as expected.

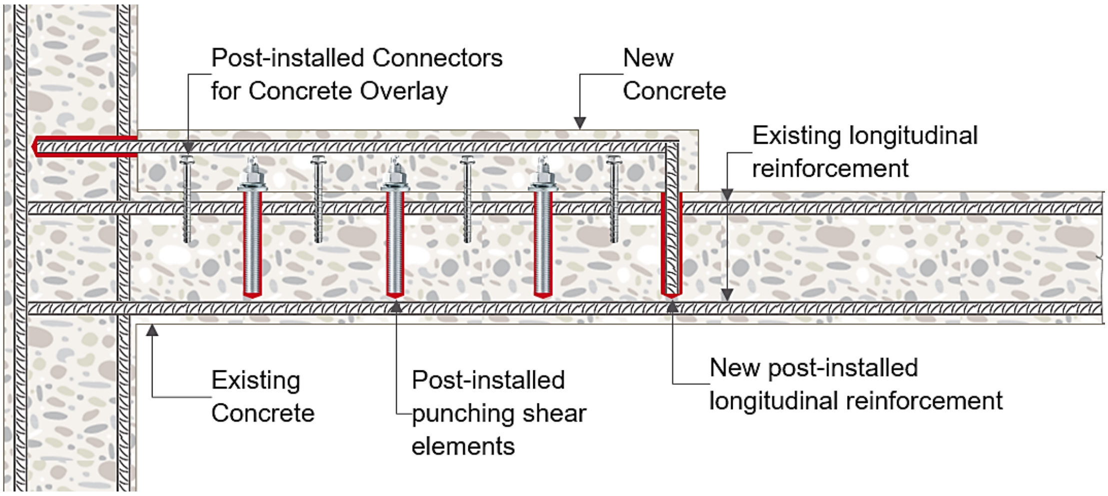


Figure 17: Example of a special solution combining post-installed punching shear reinforcement with a concrete overlay (the overlay may also extend over the entire span of the existing slab)

5. Qualification overview of post-installed Punching shear reinforcement

Whilst cast-in systems of punching shear reinforcement see widespread application in the construction industry, the use of bonded *post-installed* steel elements to strengthen concrete elements deficient in punching shear is not covered presently by any existing European Assessment Document (EAD) nor harmonized under a European standard (hEN). Such systems, therefore, require appropriate qualification to assess performance for design and use for punching shear resistance. In such scenarios, Annex D of EN 1990:2002 [23] provides the state-of-the-art guidance to calibrate, by a combination of testing and modelling, a design equation that is consistent with the target reliability levels of EN 1990.

According to the European Technical Assessment (ETA)-20/0541 [24], the combination of HIT-RE 500 V4 epoxy mortar and HAS(-U) rods of carbon and stainless steel with the Hilti Filling Set is assessed and qualified for use as a fastener in concrete. Yet, its use as a strengthening system installed perpendicular to the longitudinal axis of slabs and foundations to enhance their punching shear resistance was not investigated previously. Therefore, a comprehensive testing plan was conducted to assess the behavior of this innovative punching shear strengthening solution and to determine the influence of the main governing parameters, such as:

1. The diameter, spacing, and installation length of the rods,
2. The depth of the concrete member, and
3. The concrete strength.

Additional tests investigated the system's robustness under practical scenarios that involve unfavorable installation conditions, such as but not limited to the positioning eccentricity, accidental inclination while installing the rods, as well as the presence of existing shear cracks under service loads. This extensive experimental campaign enabled the calibration of a punching shear resistance model consistent with the reliability assessment procedure outlined in Annex D of EN 1990, yielding a design equation consistent with EN 1992-1-1:2004 detailed in the following section.

The entire experimental campaign conducted at Ruhr Universität Bochum (**RUB**) was evaluated and verified for its fitness for application by DIBt, which granted the system a General Construction Technique Permit, or **aBG Z-15.5-387** [25], thus fulfilling the national requirement for construction works under the *MVV TB*, or *Muster-Verwaltungsvorschrift Technische Baubestimmungen*. The **MVV TB** serves as a model for the Administrative Provisions – Technical Building Rules that are implemented at a federal level in Germany.

6. Design & Detailing approach with HIT-Punching shear strengthening system

The new Hilti *HIT-Punching shear strengthening* solution for shear involves the HIT-RE 500 V4 mortar and HAS or HAS-U threaded rods with the Hilti Filling Set, nuts, and washers. The installation of this solution is akin to installing a bonded anchor: i.e., drilling at a fixed embedment perpendicular to the concrete surface, cleaning the debris from the boreholes, and then injecting the mortar and inserting the rods. Once the mortar cures, the nuts are torqued according to the Instruction of Use. The solution is granted a national general construction technique permit (*aBG*) **Z-15.5-387** by DIBt and uses the provisions for **Design assisted by testing** contained in **Annex D of EN 1990** [23]. This section contains an overview of the assessment, design, and installation of post-installed threaded rods as reinforcement in punching shear deficient concrete members.

The adopted resistance model is consistent with the design provisions in DIN EN 1992-1-1/NA [15] and DIN EN 1992-2/NA [26]. The required verifications closely resemble Equations (6.47) and (6.52) of DIN EN 1992-1-1/NA for the resistance to punching shear without and with shear reinforcement, respectively, since the resistance model uses the same empirical strut-and-tie method explained in Section 3 of this document that covers the background to these equations.

The direct use of both equations, however, is not possible without the modifications that derive from the results of the qualification procedure and, overall, a successful verification must fulfil the following check of the compression struts and the strengthening rods at the ultimate limit state for a given design shear stress, τ_{Ed}:

Note: aBG Z-15.5-387 replaces the symbols for shear stress, v, with τ, and the control perimeter, u_1, with u_{crit}.

$$\tau_{Ed} = \frac{\beta \cdot V_{Ed}}{u_{crit} \cdot d_{ef}} \leq \tau_{Rd} = max\,(k_d k_{max} \tau_{Rd,c}\,, \tau_{Rd,cs,pi}) \qquad (19)$$

The following subsections highlight the additions and variations brought forth by the National Approval, aBG Z-15.5-387 [25].

6.1 Verification of the compression strut

Prior to verification, the following three conditions must be checked according to Table 6:

$\boldsymbol{\tau_{Ed} \leq \tau_{Rd,c}}$	Strengthening not required
$\boldsymbol{\tau_{Ed} \leq k_d k_{max} \tau_{Rd,c}}$	Strengthening is possible
$\boldsymbol{\tau_{Ed} > k_d k_{max} \tau_{Rd,c}}$	Strengthening is **not** possible

Table 6: Conditions for verifying the resistance of the concrete compression strut

When strengthening is possible and required, verification closely resembles Eq. (15) in Section 3.4.2:

$$\tau_{Ed} \leq \boldsymbol{k_d k_{max}} \tau_{Rd,c} \qquad (20)$$

The verification for punching resistance without shear reinforcement, $\tau_{Rd,c}$, remains unaffected and follows the same design rules as for cast-in stirrups per DIN EN 1992-1-1/NA and can be found in Eq. (1) of Section 3.3.3.

Note: When evaluating $k_d k_{max} \tau_{Rd,c}$ according to [25], $\tau_{Rd,c}$ cannot consider any contribution from any axial stress, σ_{cp}.

Following DIN EN 1992-1-1/NA, NDP to 6.4.5 (3), the factor $\boldsymbol{k_{max} = 1.4}$. The additional coefficient $\boldsymbol{k_d}$ provided in Table 7 derives from testing and only affects the strut resistance when the M16 rod is installed in thinner sections with an effective depth between 160-280 mm, where the product of $k_d k_{max} = 1.33$, as opposed to 1.4 for $d_{ef} \geq 280\ mm$.

The slight reduction in the strut resistance is attributed to a slightly larger residual cover, c_{res}, required for the M16 rod in thinner slabs to prevent spalling of the concrete on the opposite side when drilling (see Table 9). In such scenarios, the larger residual cover required has a noticeable impact, implying that the critical shear crack easily passes above tip of the rod and additionally needs to traverse a greater distance to reach the flexural reinforcement. This also has an impact on the second coefficient, k_{pi}, when verifying the shear-reinforced zones.

HIT-Punching shear strengthening	Rod size	Effective depth, d_{ef} [mm]	Installation from the top or the bottom
Coefficient for post-installed strengthening, k_{pi}	M12	≥ 160	0.82
	M16	$160 \leq d_{ef} < 280$	**0.59**
		≥ 280	0.82
	M20	≥ 350	0.82
	M24	≥ 420	0.82
Coefficient between d_{ef} and the rod diameter, k_d	M12	≥ 160	1.00
	M16	$160 \leq d_{ef} < 280$	**0.95**
		≥ 280	1.00
	M20	≥ 350	1.00
	M24	≥ 420	1.00

Table 7: Coefficients k_{pi} and k_d used in the verifications, from Table 14 [25]

6.2 Verification within and beyond the shear-reinforced zone for slabs and foundations

6.2.1 Verification within the shear-reinforced zone for slabs and foundations

When the post-installed strengthening rods are installed orthogonal to the longitudinal axis of the concrete member, the installation angle $\alpha = 90°$ and the resistance resembles Eq. (11) from Section 3.4.1, with both coefficients k_d and k_{pi} applied from Table 7:

$$\tau_{Rd,cs,pi} = \boldsymbol{k_d}(0.75\tau_{Rd,c}) + \boldsymbol{k_{pi}}\left(1.5 \cdot \frac{d_{ef}}{s_r} \cdot A_{sw,crit} \cdot f_{ywd,ef} \cdot \frac{1}{u_{crit}d_{ef}}\right) \geq \tau_{Ed} \quad \text{(N/mm}^2\text{)} \tag{21a}$$

When including the impact from prestress, Eq. (18a) may be modified to resemble:

$$\tau_{Rd,cs,pi} = \boldsymbol{k_d}[0.75(\tau_{Rd,c} + 0.5 \cdot k_1 \cdot min(\sigma_{cp}; 2))] + \boldsymbol{k_{pi}}\left(1.5 \cdot \frac{d_{ef}}{s_r} \cdot A_{sw,crit} \cdot f_{ywd,ef} \cdot \frac{1}{u_{crit}d_{ef}}\right) \geq \tau_{Ed} \tag{21b}$$

For foundations, Eq. (12) from Section 3.4.1 is modified (no contribution from concrete is considered):

$$\tau_{Rd,s,pi} = \boldsymbol{k_{pi}}\left(f_{ywd,ef} \cdot A_{sw,1+2} \cdot \frac{1}{u_{crit}d_{ef}}\right) \geq \tau_{Ed} \quad \text{(N/mm}^2\text{)} \tag{22}$$

Derived from statistical evaluations of the experimental campaign, the coefficient $\boldsymbol{k_{pi}}$ accounts for the difference in efficiency between traditional steel cast-in stirrups and the bonded steel rods used in the HIT-Punching shear strengthening system and combines the impact of several factors, such as:

- Statistically derived reliability that compares the post-installed strengthening rods to cast-in reinforcement,
- Durability accounting for the long-term effects on mortar's bond strength (e.g., short- and long-term temperature), and
- Installation sensitivity due to hole drilling and cleaning methods.

Note: The coefficient $\boldsymbol{k_{pi}}$ is unaffected by the direction of installation, in turn ensuring design outcomes are unaffected if conditions at a site do not permit installation from one direction. However, the chosen drilling direction should remain consistent for all strengthening elements.

Geometric tolerances during installation due to positioning and deviation from vertical direction,

The effective design strength of the strengthening elements, $f_{ywd,ef}$, employed in Eq. (21a), (21b), and (22) remains unchanged from Eq. (3) in Section 3.4.1, apart from the upper limit, $\boldsymbol{f_{ywd}}$, that stems from assessment and is consistent for both A4 stainless steel and 8.8 carbon steel rods and can be found in Table 8 alongside the stressed cross-section area for each rod diameter:

$$f_{ywd,ef} = 250 + 0.25 \cdot d_{ef} \leq f_{ywd} \quad \text{(N/mm}^2\text{)} \qquad (23)$$

Material	Rod size	Design value of yield strength $\boldsymbol{f_{ywd}}$ $[\mathrm{N/mm^2}]$	Stressed cross-section area of a threaded rod $\boldsymbol{A_{sw}}[\mathrm{mm^2}]$
HAS 8.8, HAS-U 8.8, HAS A4, HAS-U A4	M12	390	84.3
	M16		157
	M20		245
	M24		353

Table 8: Geometrical and material parameters, from Table 13 [25]

6.2.2 Reinforcement increase factor, $\boldsymbol{\kappa_{sw,i}}$

As mentioned previously in Section 3.4.1, DIN EN 1992-1-1/NA requires, only for slabs, an increase to the punching shear reinforcement required in the first two reinforcing perimeters with the factors $\boldsymbol{\kappa_{sw,1} = 2.5}$ and $\boldsymbol{\kappa_{sw,2} = 1.4}$ magnifying $A_{sw,crit}$ for the first and second reinforcing perimeters, respectively. These two parameters correct an underestimation of the required punching resistance provided by the first two reinforcing perimeters.

Mechanically, when positioning the reinforcement closer to the support area, the smaller length of the reinforcing perimeter results in a smaller contribution from concrete to the overall resistance, which must be compensated by a higher contribution from steel. The choice of two fixed factors is provided for ease of use, saving the designer from calculating the required amount of steel, and thus verifying $\tau_{Rd,cs}$, at each reinforcing perimeter.

When transferring this equation to the post-installed HIT-Punching shear strengthening system, the National Approval aBG Z-15.5-387 [25] introduces a refined alternative to evaluate the reinforcement increase factor, $\kappa_{sw,i}$, more precisely, which will better reflect the real strut-and-tie mechanisms behind punching shear resistance in EN 1992-1-1:2004 and its National Annexes.

Thus, rearranging Eq. (21a) and equating $\tau_{Rd,cs,pi} = \tau_{Ed}$ at the control section $\boldsymbol{u_{crit}}$ allows:

$$A_{sw,crit} \geq \frac{\tau_{Ed} - 0.75 \cdot k_d \cdot \tau_{Rd,c}}{1.5 \cdot k_{pi} \cdot f_{ywd,ef}} \cdot s_r \cdot u_{crit} \quad \text{(mm}^2\text{)} \qquad (24a)$$

To evaluate the reinforcement at any perimeter, u_{crit} is replaced by $\boldsymbol{u_i}$:

$$A_{sw,i} \geq \frac{\tau_{Ed} - 0.75 \cdot k_d \cdot \tau_{Rd,c}}{1.5 \cdot k_{pi} \cdot f_{ywd,ef}} \cdot s_r \cdot u_i \quad \text{(mm}^2\text{)} \qquad (24a)$$

For the first and second reinforcing perimeters, the ratio $A_{sw,i}/A_{sw,crit}$ is the same as $\kappa_{sw,i}$, which can be expressed as:

$$\kappa_{sw,i} = \frac{\beta V_{Ed} - 0.75 k_d \tau_{Rd,c} \boldsymbol{u_i} d_{ef}}{\beta V_{Ed} - 0.75 k_d \tau_{Rd,c} \boldsymbol{u_{crit}} d_{ef}} \qquad (25)$$

Note: The reinforcement increase factor $\boldsymbol{\kappa_{sw,i}}$ applies only to slabs and not to foundations.

6.2.3 Verification beyond the shear-reinforced zone for slabs and foundations

Strengthening verifications beyond the shear-reinforced zone remain unaffected by the National Approval aBG Z-15.5-387 [25], and follow the same provisions described in Section 3.4.3.

6.3 Requirements for detailing the strengthening reinforcement

6.3.1 Installation length, l_{sw}

As evidenced from Equations (19-25), the design model does not require an explicit consideration of the installation length, $\boldsymbol{l_{sw}}$, in the verifications. Instead, l_{sw} is a function of the section height, $\boldsymbol{h}$, and the "residual" cover, $\boldsymbol{c_{res}}$, see Figure 18 (right). From an installation perspective, the residual cover prevents concrete blowout, or spalling, on the surface opposite to drilling, and does not require knowledge of the longitudinal reinforcement position close to that surface.

Figure 18: Simplified schematic of the HIT-Punching shear strengthening system installed from above (left) or below (right) the concrete member

From a design perspective, a fixed installation length ensures that the punching shear reinforcement is anchored in the compression or tension chord of the member, enabling the formation of the strut-and-tie model on which design is predicated. As mentioned previously in Sections 2 and 3, the provided punching shear reinforcement must enclose, or hook, around the compression chord as a tension tie to allow the transfer of forces in the node. To this effect, the combination of large diameter strengthening reinforcement, such as M24 rods, in thinner slabs, say 200mm, can result in potentially dangerous scenarios where the remaining cover, $\boldsymbol{c_{res}}$, of 60 mm leaves the installation length, $\boldsymbol{l_{sw}}$, at a mere 140 mm, which is inadequate to effectively anchor the strut-and-tie mechanism at the nodes. Such combinations are, therefore, not permitted and a correlation between the effective depth of the member and the reinforcement diameter is required per Table 9 [25].

Installation Parameter		M12	M16	M20	M24
Rod diameter	d [mm]	12	16	20	24
Drill hole diameter	d_0 [mm]				
Minimum effective depth of the concrete member	$d_{ef,min}$ [mm]	160	160	350	420
Maximum section height of the concrete member	h_{max} [mm]	1100			
Residual cover	c_{res} [mm]	35	40	45	60

Installation length	l_{sw} [mm]	$h - c_{res}$			
Maximum torque moment	$T_{inst} \leq$ [Nm]	40	80	150	200

Table 9: Correlation between the minimum section height, residual cover, and strengthening reinforcement diameter, from Table 3 [25]

6.3.2 Minimum and maximum spacing, s

Apart from easing the distribution of concrete aggregates evenly during casting, DIN EN 1992-1-1/NA does not define a minimum spacing, s_{min}, between punching reinforcement such as stirrups. Without exceptions, the HIT-Punching shear strengthening system requires a defined minimum spacing to avoid splitting between the rods and a potential reduction in the overall shear resistance. Additionally, Table 10 provides the minimum spacing that applies to both the radial (between reinforcing perimeters) and transverse (within each reinforcing perimeter) directions inside and outside the control section, u_{crit}.

Diameter of the strengthening reinforcement	Minimum spacing, s_{min} [mm]	Maximum transverse spacing, $s_{t,max}$ within u_{crit} [mm]	Maximum transverse spacing, $s_{t,max}$ beyond u_{crit} [mm]
M12	72	$1.5d_{ef}$	$2.0d_{ef}$
M16	96		
M20	120		
M24	144		

Table 10: Minimum center-to-center radial spacing and the maximum transverse spacing within and beyond the control section, reproduced from Table 15 of [25]

The radial spacing, s_0 (from the support area to the first reinforcing perimeter) and s_r (spacing between subsequent reinforcing perimeters), has different upper limits for slabs and foundations and the various rules are summarized in Table 11.

Concrete member	Spacing from the support area to the first perimeter, s_0	Spacing between first and second perimeters, s_r	Spacing for subsequent perimeters, s_r
Slabs	$0.3d_{ef} \leq s_0 \leq 0.5d_{ef}$	$s_{min} \leq s_r \leq 0.75d_{ef}$	
Slender foundations ($a_\lambda/d_{ef} \leq 2$)	$s_0 \leq 0.3d_{ef}$	$s_r \leq 0.5d_{ef}$	$s_r \leq 0.5d_{ef}$
Squat foundations ($a_\lambda/d_{ef} > 2$)			$s_r \leq 0.75d_{ef}$

Table 11: Maximum spacing between reinforcing perimeters for slabs and foundations

6.3.3 Edge distance, c

Setting a minimum distance between the position of the strengthening rods and any concrete edge, such as an opening or the edge of slab / foundation, reduces the risk of splitting, with this minimum evaluated in the mortar's assessment ETA 20/0541 [24]. The base minimum is increased by a percentage of the installation length that accounts for the maximum permitted inclination of the borehole (5°) perpendicular to the concrete surface and is summarized in Table 12.

Drilling system	Rod size	Minimum edge distance, c_{min} [mm]	
		Without drilling aid	With drilling aid
Hammer drilling with or without Hilti hollow drill bits, and diamond coring with roughening tool	M12	$45\ mm + 0{,}06 l_{sw}$	$45\ mm + 0{,}02 l_{sw}$
	M16	$50\ mm + 0{,}06 l_{sw}$	$50\ mm + 0{,}02 l_{sw}$
	M20	$55\ mm + 0{,}06 l_{sw}$	$55\ mm + 0{,}02 l_{sw}$
	M24	$60\ mm + 0{,}06 l_{sw}$	$60\ mm + 0{,}02 l_{sw}$
Pneumatic drilling	M12	$50\ mm + 0{,}08 l_{sw}$	$50\ mm + 0{,}02 l_{sw}$
	M16		
	M20	$55\ mm + 0{,}08 l_{sw}$	$55\ mm + 0{,}02 l_{sw}$
	M24	$60\ mm + 0{,}08 l_{sw}$	$60\ mm + 0{,}02 l_{sw}$

Table 12: Minimum edge distances based on drilling methods and tolerances, reproduced from Table 16 of [25]

6.3.4 Positioning tolerances

To curtail the radial and tangential cracks associated with punching actions, punching shear reinforcement is typically positioned in a radial manner around the support area that, when drilling and installing strengthening elements, coincides with the orthogonal layout of the existing longitudinal reinforcement within the slab or foundation. Aborting and drilling in new positions may have a detrimental impact on the resistance of the slab or foundation due to the asymmetry between the flow of shear stresses and the reinforcement positions. In turn, limiting the asymmetry helps limit any potential loss in punching resistance of the slab.

The experimental campaign underpinning the HIT-Punching shear strengthening system replicated such asymmetries to avoid triggering potential redesign based on as-built positioning of the individual strengthening elements. The resulting evidence suggests that individual stirrups may deviate from their original positions by a maximum distance of $\pm\mathbf{0.2}\boldsymbol{d_{ef}}$, with design requiring no additional considerations or reduction in resistance as long as the minimum and maximum spacing rules for both slabs and foundations adhere to Table 10 and Table 11. The red dashed circle in Figure 19 highlights this tolerance.

Note: the minimum clear distance from the original position should be maintained at $\boldsymbol{2d_0}$, with the or the aborted hole filled with a low-shrinkage mortar such as HIT-RE 500 V4.

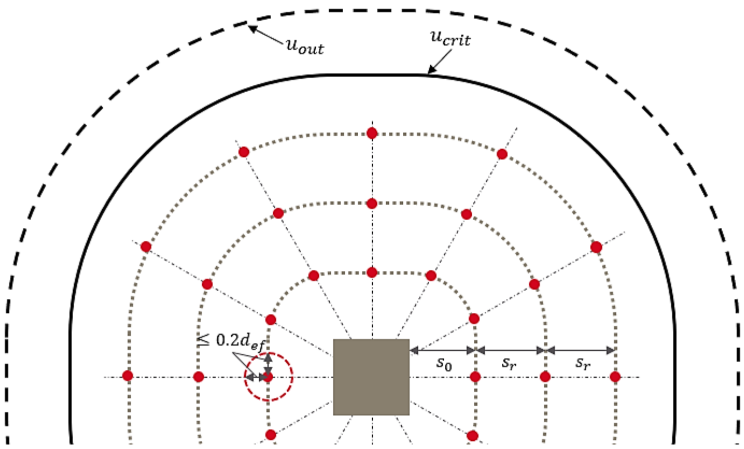


Figure 19: Schematic layout illustrating the potential deviation in positioning of individual strengthening elements, represented by a red dashed circle

7. Design examples

7.1 Foundation – Column on an Isolated Footing

7.1.1 Inputs

- Design shear force: $V_{Ed} = 5700\ kN$
- Column dimensions ($C_x \times C_y$): $600 \times 1400\ mm$
- Load eccentricity factor: $\beta = 1.15$
- Slab thickness: $h = 800\ mm$
- Effective depth in *x* & *y*: $d_x = 745\ mm\ ;\ d_y = 735\ mm$
- Slab concrete strength: $f_{ck} = 20\ N/mm^2$
- Concrete partial safety factor: $\gamma_c = 1.5$
- Prestress: $\sigma_{cp} = 0\ kN/m^2$
- Uniform soil pressure: $\sigma_{gd} = 350\ kN/m^2$
- Unit weight of concrete, $\gamma = 25.0\ kN/m^3$
- Concrete parameters:

f_{ck} [N/mm²]	α_{cc} [-]	γ_c [-]	f_{cd} [N/mm²]
20.00	0.85	1.50	11.33

The longitudinal reinforcement ratio is assumed constant across the specific slab width in both directions, b_{sx} and b_{sy}:

- In the *x*-direction, $\rho_{lx} = \frac{1885+8042}{3900\cdot745} = 0.00342$ [from 24-10 mm bars and 10-32 mm bars]
- In the *y*-direction, $\rho_{l,y} = \frac{1885+8042}{3900\cdot735} = 0.00346$ [from 24-10 mm bars and 10-32 mm bars]

NDP to 6.4.4 (1) [15]

Mean longitudinal reinforcement ratio, $\rho_l = \sqrt{0.00342 \cdot 0.00346} = 0.00344 \leq \min\left(0.02\ ;\ 0.5\frac{f_{cd}}{f_{yd}}\right)$

7.1.2 Perimeter definitions

Since the ratio of the larger to the smaller column dimension exceeds 2.0, “partial sections” according to Figure NA.6.21.1 [15] are used to evaluate the various perimeters.

Description	Variable	Value
Column perimeter	u_0	$3600\ mm$
Mean effective depth	d_{ef}	$740\ mm$
Control section from column face at $a_{crit} = 600\ mm$ (by iteration)	u_{crit}	$7370\ mm$
Area contained within a_{crit}	A_{crit}	$4.371\ m^2$

Outer perimeter where reinforcement is not required	u_{out}	$23058\,mm$

7.1.3 Verification of concrete without punching shear reinforcement

6.4.3 (1) [12] Verification of the concrete resistance without punching shear reinforcement, $\tau_{Rd,c}$, is conducted at the critical perimeter, u_{crit}, determined by a_{crit}.

Net upwards force within a_{crit}: $\Delta V_{Ed} = (A_{crit} \cdot \sigma_{gd}) - G_k(\gamma \cdot A_{crit} \cdot h)$

Eq. 6.48 [12]

$$\Delta V_{Ed} = (4.371 \cdot 350) - 1.35 \cdot (25 \cdot 4.371 \cdot 0.8) = \mathbf{1412\ kN}$$

Net applied force: $V_{Ed,red} = V_{Ed} - \Delta V_{Ed} = 5700 - 1412 = \mathbf{4288\ kN}$

Design punching stress at u_{crit}: $\tau_{Ed} = \frac{\beta \cdot V_{Ed,red}}{u_{crit} \cdot d_{ef}} = \frac{1.15 \cdot 4288 \cdot 10^3}{7370 \cdot 740} = \mathbf{0.904\ N/mm^2}$

Eq. (2) [25] Verification of the existing section: $\tau_{Ed} \leq \tau_{Rd,c} = max\left[C_{Rd,c}k(100\rho_l f_{ck})^{\frac{1}{3}}, \tau_{min}\right] + k_1\sigma_{cp}$

NDP Zu 6.4.4 (1) [15] Empirical pre-factor for foundations: $C_{Rd,c} = 0.18/1.5 = \mathbf{0.10}$

Eq. 6.2.2 (1) [25] Member height-dependent coefficient: $k = 1 + \sqrt{200/740} = \mathbf{1.52} < 2.0$

Eq. (3) [25] Minimum design punching resistance (interpolated for $600\,mm \leq d_{ef} \leq 800\,mm$):

$$\tau_{min} = \frac{0.042}{\gamma_c} k^{3/2} f_{ck}^{1/2} = \frac{0.042}{1.5} \cdot 1.52^{3/2} \cdot 20^{\frac{1}{2}} = 0.235\ N/mm^2$$

Eq. 6.50 [12] Design punching resistance:

$$\tau_{Rd,c} = max\left[0.10 \cdot 1.52 \cdot (100 \cdot 0.00344 \cdot 20)^{\frac{1}{3}}, 0.235\right] \cdot \frac{2 \cdot 740}{600} = \mathbf{0.713\ N/mm^2}$$

Eq. NA.6.53.1 [15] Maximum punching resistance ($k_{max} = 1.4$): $\tau_{Rd,max} = k_{max}\tau_{Rd,c} = 1.4 \cdot 0.713 = \mathbf{0.998\ N/mm^2}$

Since $\tau_{Rd,c} \leq \tau_{Ed} \leq \tau_{Rd,max}$, strengthening is required!

7.1.4 Verification of concrete with HIT-Punching Shear strengthening

Eq. (4) [25] Verification of the strengthened section: $\tau_{Ed} \leq \tau_{Rd,cs,pi} \leq k_d k_{max} \tau_{Rd,c}$

Maximum punching resistance ($k_d = 1.0$): $k_d \tau_{Rd,max} = 1.0 \cdot 0.998 = \mathbf{0.998\ N/mm^2}$

Since $\tau_{Ed} \leq \tau_{Rd,max}$, strengthening is possible!

Eq. (10) [25] Design punching resistance with strengthening elements must satisfy:

$$V_{Rd,s,pi} = k_{pi}(f_{ywd,ef} \cdot A_{sw,1+2}) \geq \beta V_{Ed,red}$$

Eq. 9.11 [15] Check for minimum cross-sectional area of **each** strengthening element:

$$A_{sw,min} = 0.08 \frac{\sqrt{f_{ck}}}{1.5(f_{ywd} \cdot \gamma_s)}(s_r \cdot s_{t,max}) = 0.08 \frac{\sqrt{20}}{1.5 \cdot (390 \cdot 1.15)} \cdot 350 \cdot (1.5 \cdot 740) = \mathbf{206.6\ mm^2}$$

The **M24 8.8 HAS(-U)** with $\boldsymbol{A_{sw}} = \mathbf{353\ mm^2}$ is sufficient to proceed with the verification, with $d_{ef} = 740\,mm$, $\boldsymbol{k_d = 1.0}$, and $\boldsymbol{k_{pi} = 0.82}$ adopted to verify $V_{Rd,cs,pi}$ according to Eq. 5 [25].

Eq. (6) [25] Effective design strength of the strengthening elements, $f_{ywd,ef} = 250 + 0,25 \cdot d_{ef} \leq f_{ywd}$

$$f_{ywd,ef} = 250 + 0.25 \cdot 740 = 435\,N/mm^2 > \mathbf{390\,N/mm^2}$$

Spacing of post-installed punching shear strengthening elements:

Parameter	Check for minimum and maximum	
$s_0 = 200\,mm$	$0.3d_{ef} \leq s_0$	Fulfilled

$s_r = 350\ mm$	$s_{min} \leq s_r \leq 0.5d_{ef}$ where $s_{min,M24} = 144\ mm$	Fulfilled
s_t within u_{crit}	$s_{min} \leq s_t \leq 1.5d_{ef}$ where $s_{min,M24} = 144\ mm$	Fulfilled (see Section 7.1.5)
s_t beyond u_{crit}	$s_{min} \leq s_t \leq 2.0d_{ef}$ where $s_{min,M24} = 144\ mm$	Fulfilled (see Section 7.1.5)

Rearranging Eq. (10) [25] allows calculation of the combined punching reinforcement required in the first two perimeters: $A_{sw,1+2} = \frac{\beta V_{Ed,red}}{k_{pi} \cdot f_{ywd,ef}}$.

$$A_{sw,1+2} = \frac{1.15 \cdot 4288 \cdot 10^3}{0.82 \cdot 390} = \mathbf{15420\ mm^2}$$

NCI Zu 6.4.5 [15]

The minimum punching reinforcement required per perimeter for any further reinforcing perimeters beyond the first two is: $0.33 \cdot A_{sw,1+2} = \mathbf{5089\ mm^2}$.

7.1.5 Strengthening reinforcement layout and Installation Data

Outer perimeter where punching reinforcement is not required, $u_{out} = \frac{\beta \cdot V_{Ed,red}}{\tau_{Rd,c,out} \cdot d_{ef}}$, where $\tau_{Rd,c,out}$ is evaluated with $C_{Rd,c} = 0.15/1.5 = \mathbf{0.10}$:

Eq. (5) [25]

$$\tau_{Rd,c,out} = max\left[0.10 \cdot 1.52 \cdot (100 \cdot 0.00344 \cdot 20)^{\frac{1}{3}};\ 0.235\right] = 0.289\ N/mm^2$$

$$u_{out} = \frac{1.15 \cdot 4288 \cdot 10^3}{0.289 \cdot 740} = \mathbf{23058\ mm}$$

Distance from the column face to u_{out}, $r_{out} = \frac{23058 - 3600}{2\pi} = 3097\ mm$

With $s_0 = 200\ mm$ and $s_r = 350\ mm$, a maximum of nine reinforcing perimeters can fit within u_{out}; however, as punching reinforcement may only stop at a distance greater than $(3097 - 1{,}5d_{ef}) = 1987\ mm$ from the column face, **seven** reinforcing perimeters are sufficient.

Perimeter	Distance from column face [mm]	Perimeter length [mm]	Required steel area (mm²) [A_{sw}]	Elements provided per perimeter	Provided steel area [mm²]	Transverse Spacing [mm]
1	200	4857	7710	22	7766	$300 \leq s_{t,max}$
2	550	7056	7710	22	7766	$432 \leq s_{t,max}$
3	900	9255	5089	16	5648	$707 \leq s_{t,max}$
4	1250	11454	5089	16	5648	$982 \leq s_{t,max}$
5	1600	13653	5089	16	5648	$1257 \leq s_{t,max}$
6	1950	15852	5089	20	7060	$1021 \leq s_{t,max}$
7	2300	18051	5089	20	7060	$1204 \leq s_{t,max}$

Note: When the position of any strengthening element coincides with existing flexural reinforcement, the affected element can be adjusted by a minimum distance of $\mathbf{2d_0}$ and a maximum of $\mathbf{0,2d_{ef}}$. However, the minimum and maximum spacing rules for radial, s_0 and s_r, as well as tangential distances, s_t, from 7.1.4 must always be observed.

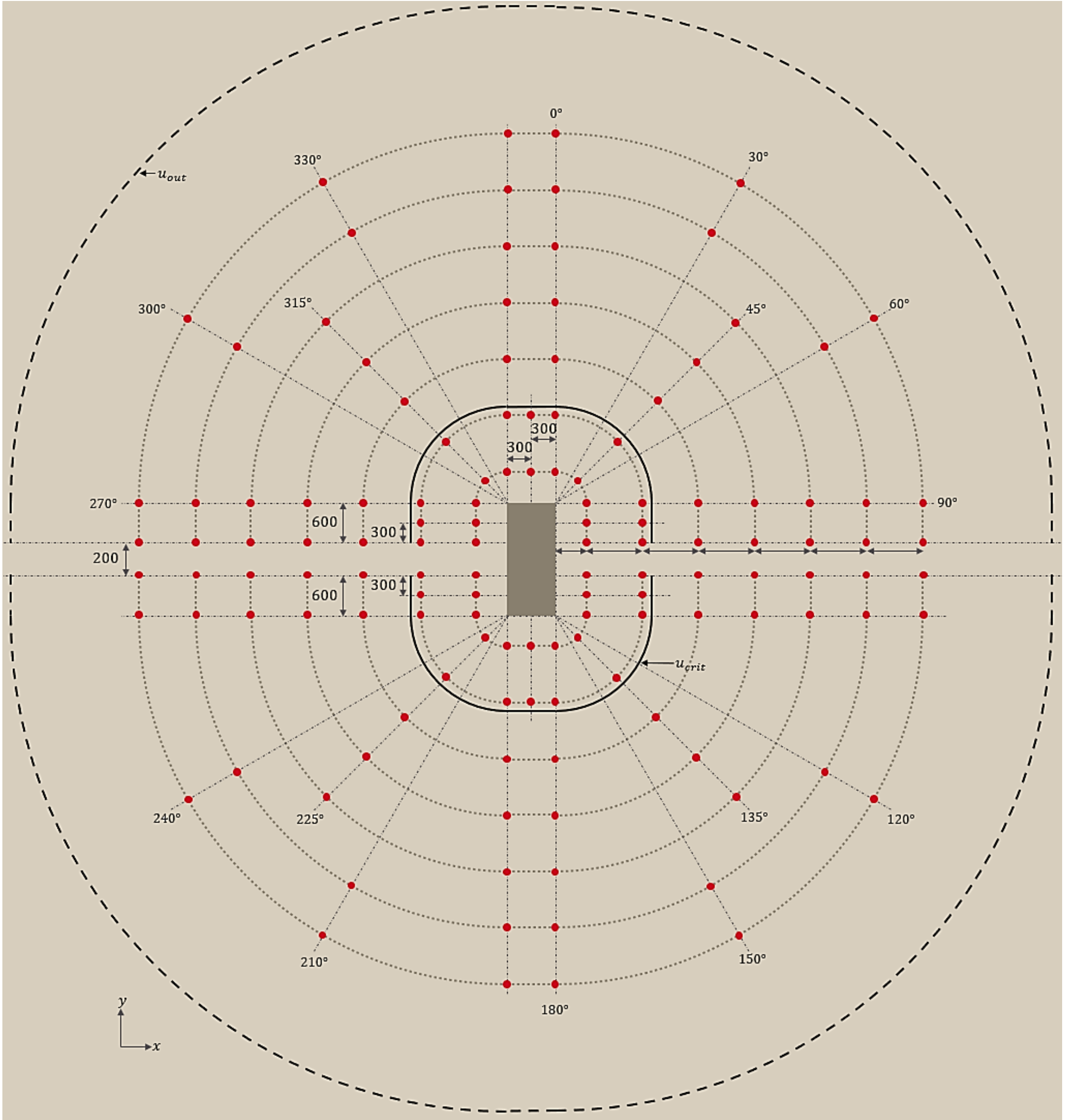
Installation according to the Manufacturer's Product Installation Instructions (MPII):

- Strengthening solution: HIT-RE 500 V4 + HAS-U A4 M24 + Filling Set
- Maximum installation torque, T_{inst}: 200 Nm
- Hole diameter in the foundation, d_0: 28 mm
- Residual cover, c_{res}: 60 mm
- Hole depth in the foundation, l_{sw}: 740 mm
- Proposed drilling method: Hammer-drilled (HD) with Drilling Aid
- Concrete condition: Dry Concrete

Specification:

132 pieces of Hilti HIT-RE 500 V4 + HAS-U **A4 M24** threaded rods + Filling Set embedded at **740 mm** per installation instructions in DIBt abG Z-15.5-387 for Hammer drilling **(HD)** in **Dry** concrete with Drilling aid is used. **First** reinforcement perimeter must be positioned **200 mm** from the face of the column, with **subsequent** perimeters spaced at **350 mm** from the first perimeter. Refer to the construction drawing for reinforcement spacing within each perimeter.

As an example, a suggested reinforcement layout is provided below:



Note 1: dimensions in millimeters and not to scale.

7.2 Slab – Re-entrant column

7.2.1 Inputs and layout

- Design shear force: $V_{Ed} = 565\ kN$
- Column dimensions ($C_x \times C_y$): $450 \times 450\ mm$
- Load eccentricity factor: $\beta = 1.367$, from $M_{Ed,x} = 75\ kNm$ & $M_{Ed,y} = 73\ kNm$
- Slab thickness: $h = 225\ mm$
- Effective depth in *x* & *y*: $d_x = 187\ mm\,;\ d_y = 171\ mm$
- Slab concrete strength: $f_{ck} = 35\ N/mm^2$
- Concrete partial safety factor: $\gamma_c = 1.5$
- Prestress in slab: $\sigma_{cp} = 0\ N/mm^2$
- Concrete parameters:

$\boldsymbol{f_{ck}}$ [N/mm²]	$\boldsymbol{\alpha_{cc}}$ [-]	$\boldsymbol{\gamma_c}$ [-]	$\boldsymbol{f_{cd}}$ [N/mm²]
35.00	0.85	1.50	19.83

The longitudinal reinforcement ratio is assumed constant across the specific slab width in both directions, b_{sx} and b_{sy}:

- In the *x*-direction, $\rho_{l,x} = 1.12\%$
- In the *y*-direction, $\rho_{l,y} = 1.225\%$

NDP to 6.4.4 (1) [15]

Mean longitudinal reinforcement ratio, $\rho_l = \sqrt{0.0112 \cdot 0.01225} = 0.01171 \leq \min\left(0.02\,, 0.5\frac{f_{cd}}{f_{yd}}\right)$

7.2.2 Perimeter definitions

Description	Variable	Value
Column perimeter	u_0	$1800\ mm$
Mean effective depth	d_{ef}	$179\ mm$
Critical perimeter at $2d_{ef}$ with reduction from openings	u_{crit}	$3787\ mm$
Ratio of column perimeter to effective depth	$\frac{u_0}{d_{ef}}$	12.0
Outer perimeter where reinforcement is not required	u_{out}	$6263\ mm$

7.2.3 Calculation of the load eccentricity factor, *β*, from [12] & [15]

Values of $W_{1,x}$ and $W_{1,y}$ are evaluated from Eq. 6.40 [12] for each direction, and the factors k_x and k_y are derived from the ratio of column dimensions in Table 6.1 [12].

Static Moment, $W_{1,x}$	Static moment, $W_{1,y}$	$\frac{C_x}{C_y}$	k_x	$\frac{C_y}{C_x}$	k_y
$988{,}418\ mm^2$	$1{,}434{,}508\ mm^2$	1.0	0.6	1.0	0.6

Eq. (NA.6.39.1) [15] Load eccentricity for the unbalanced moments, $\beta = 1 + \sqrt{\left(k_x \frac{M_{Ed,x}}{V_{Ed}} \cdot \frac{u_{crit}}{W_{1,x}}\right)^2 + \left(k_y \frac{M_{Ed,y}}{V_{Ed}} \cdot \frac{u_{crit}}{W_{1,y}}\right)^2} \geq 1.10$

$$\beta = 1 + \sqrt{\left(0.6 \cdot \frac{75 \times 10^3}{336} \cdot \frac{3787}{988418}\right)^2 + \left(0.6 \cdot \frac{73 \times 10^3}{336} \cdot \frac{3787}{1434508}\right)^2} = \mathbf{1.367} \geq 1.10$$

7.2.4 Verification of concrete without punching shear reinforcement

6.4.3 (1) [12] Verification of the concrete resistance without punching shear reinforcement, $\tau_{Rd,c}$, is conducted at the critical perimeter, u_{crit}.

Eq. (2) [12] Verification of the existing section: $\tau_{Ed} \leq \tau_{Rd,c} = max\left[C_{Rd,c}k(100\rho_l f_{ck})^{\frac{1}{3}}, \tau_{min}\right] + k_1\sigma_{cp}$

NDP Zu 6.4.4 (1) [15] Empirical pre-factor (inner columns with $\frac{u_0}{d} \geq 4$): $C_{Rd,c} = 0.18/1.5 = \mathbf{0.12}$

Eq. 6.2.2 (1) [12] Member height-dependent coefficient: $k = 1 + \sqrt{200/179} = 2.06 \geq \mathbf{2.0}$

Eq. NA 6.3a [15] Minimum design punching resistance (with $d_{ef} \leq 600\ mm$):

$$\tau_{min} = \frac{0.0525}{\gamma_c}k^{3/2}f_{ck}^{1/2} = \frac{0.0525}{1.5} \cdot 2.0^{3/2} \cdot 35^{\frac{1}{2}} = \mathbf{0.586\ N/mm^2}$$

Design punching resistance: $\tau_{Rd,c} = 0.12 \cdot 2.0 \cdot (100 \cdot 0.01171 \cdot 35)^{\frac{1}{3}} = \mathbf{0.828\ N/mm^2}$

Design punching stress at u_{crit}: $\tau_{Ed} = \frac{\beta \cdot V_{Ed}}{u_{crit} \cdot d_{ef}} = \frac{1.367 \cdot 565 \cdot 10^3}{3787 \cdot 179} = \mathbf{1.140\ N/mm^2}$

Eq. NA.6.53.1 [15] Maximum punching resistance ($k_{max} = 1.4$): $\tau_{Rd,max} = k_{max}\tau_{Rd,c} = 1.4 \cdot 0.828 = \mathbf{1.159\ N/mm^2}$

Since $\tau_{Rd,c} \leq \tau_{Ed} \leq \tau_{Rd,max}$, strengthening is required!

7.2.5 Verification of concrete with HIT-Punching Shear strengthening

Eq. (3) [25] Verification of the strengthened section: $\tau_{Ed} \leq \tau_{Rd,cs,pi} \leq k_d k_{max}\tau_{Rd,c}$

Maximum punching resistance ($k_d = 1.0$): $k_d\tau_{Rd,max} = 1.0 \cdot 1.159 = \mathbf{1.159\ N/mm^2}$

Since $\tau_{Ed} \leq \tau_{Rd,max}$, strengthening is possible!

Eq. (5) [25] Design punching resistance with strengthening elements must satisfy:

$$V_{Rd,cs,pi} = \boldsymbol{k_d}(0.75\tau_{Rd,c}u_{crit}d_{ef}) + \boldsymbol{k_{pi}}\left(1.5f_{ywd,ef}A_{sw,crit}\frac{d_{ef}}{s_r}\right) \geq \beta V_{Ed}$$

Eq. 9.11 [15] Check for minimum cross-sectional area of **each** strengthening element:

$$A_{sw,min} = 0.08\frac{\sqrt{f_{ck}}}{1.5(f_{ywd} \cdot \gamma_s)}(s_r \cdot s_{t,max}) = 0.08\frac{\sqrt{35}}{1.5 \cdot (390 \cdot 1.15)} \cdot 120 \cdot (1.5 \cdot 179) = \mathbf{22.7\ mm^2}$$

The **M12 8.8 HAS(-U)** with $\boldsymbol{A_{sw}} = \mathbf{84.3\ mm^2}$ is sufficient to proceed with the verification, with $d_{ef} = 179\ mm$, $\boldsymbol{k_d} = \mathbf{1.0}$, and $\boldsymbol{k_{pi}} = \mathbf{0.82}$ adopted to verify $V_{Rd,cs,pi}$ according to Eq. 5 [25].

Eq. (6) [25] Effective design strength of the strengthening elements, $f_{ywd,ef} = 250 + 0.25 \cdot d_{ef} \leq f_{ywd}$

$$f_{ywd,ef} = 250 + 0.25 \cdot 179 = \mathbf{294.75\ N/mm^2} \leq 390\ N/mm^2\ \therefore OK$$

Spacing of post-installed punching shear strengthening elements:

Parameter	Check for minimum and maximum	
$s_0 = 80\ mm$	$0.3d_{ef} \leq s_0 \leq 0.5d_{ef}$	Fulfilled
$s_r = 120\ mm$	$s_{min} \leq s_r \leq 0.75d_{ef}$ where $s_{min,M12} = 72\ mm$	Fulfilled

s_t within u_{crit}	$s_{min} \leq s_t \leq 1.5d_{ef}$ where $s_{min,M12} = 72\ mm$	Fulfilled (see Section 0)
s_t beyond u_{crit}	$s_{min} \leq s_t \leq 2.0d_{ef}$ where $s_{min,M12} = 72\ mm$	Fulfilled (see Section 0)

Eq. (7) [25]

Equating $\tau_{Ed} = \tau_{Rd,cs,pi}$ and rearranging the equation allows calculation of the punching reinforcement required at the critical perimeter: $A_{sw,crit} = \frac{\tau_{Ed}-0.75k_d\tau_{Rd,c}}{1.5k_{pi}f_{ywd,ef}} s_r u_{crit}$.

$$A_{sw,crit} = \frac{1.140 - 0.75 \cdot 1.0 \cdot 0.828}{1.5 \cdot 0.82 \cdot 294.75} \cdot 120 \cdot 3787 = \mathbf{651\ mm^2}$$

Eq. (8) [25]

The punching reinforcement required for the first two reinforcing perimeters must satisfy $A_{sw,i} \geq \kappa_i A_{sw,crit}$, where the factor $\boldsymbol{\kappa_i}$ is evaluated using Eq. (9) [25]:

$$\kappa_i = \frac{\beta V_{Ed} - 0.75k_d\tau_{Rd,c}\boldsymbol{u_i}d_{ef}}{\beta V_{Ed} - 0.75k_d\tau_{Rd,c}\boldsymbol{u_{crit}}d_{ef}}$$

For the first reinforcing perimeter: $\kappa_1 = \frac{1.367 \cdot 565 \cdot 10^3 - 0.75 \cdot 1.0 \cdot 0.828 \cdot \mathbf{2303} \cdot 179}{1.367 \cdot 565 \cdot 10^3 - 0.75 \cdot 1.0 \cdot 0.828 \cdot 3787 \cdot 179} = \mathbf{1.47} \leq 2.5 \therefore OK$

For the second reinforcing perimeter: $\kappa_2 = \frac{1.367 \cdot 565 \cdot 10^3 - 0.75 \cdot 1.0 \cdot 0.828 \cdot \mathbf{3042} \cdot 179}{1.367 \cdot 565 \cdot 10^3 - 0.75 \cdot 1.0 \cdot 0.828 \cdot 3787 \cdot 179} = \mathbf{1.24} \leq 1.4 \therefore OK$

7.2.6 Strengthening reinforcement layout and Installation Data

Outer perimeter where punching reinforcement is not required, $u_{out} = \frac{\beta \cdot V_{Ed}}{\tau_{Rd,c,out} \cdot d_{ef}}$, where $\tau_{Rd,c,out}$ is evaluated with $C_{Rd,c} = 0.15/1.5 = \mathbf{0.10}$:

Eq. (5) [25]

$$\tau_{Rd,c,out} = max\left[0.10 \cdot 2.0 \cdot (100 \cdot 0.0117 \cdot 35)^{\frac{1}{3}} ; 0.586\right] = 0.689\ N/mm^2$$

$$u_{out} = \frac{1.367 \cdot 565 \cdot 10^3}{0.689 \cdot 179} = \boldsymbol{6263\ mm}$$

Distance from the column face to u_{out}, $r_{out} = \frac{6263-1800-300}{1.5\pi} = 883\ mm$

With $s_0 = 80\ mm$ and $s_r = 120\ mm$, a maximum of seven reinforcing perimeters can fit within u_{out}; however, as punching reinforcement may only stop at a distance greater than $(566 - 1{,}5d_{ef}) = 298\ mm$ from the column face, **six** reinforcing perimeters are sufficient.

Perimeter	Distance from column face (mm)	Perimeter length (mm)	Required steel area (mm²) $\kappa_i \cdot A_{sw,crit}$	Elements provided per perimeter	Provided steel area (mm²)	Transverse Spacing (mm)
1	80	2303	957	12	1012	$200 \leq s_{t,max}$
2	200	3042	807	16	1349	$200 \leq s_{t,max}$
3	320	3608	651	14	1180	$251 \leq s_{t,max}$
4	440	4173	651	14	1180	$346 \leq s_{t,max}$
5	560	4739	651	17	1433	$293 \leq s_{t,max}$
6	680	5304	651	17	1433	$293 \leq s_{t,max}$

Note: When the position of any strengthening element coincides with existing flexural reinforcement, the affected element can be adjusted by a minimum distance of $\mathbf{2d_0}$ and a maximum of $\mathbf{0,2d_{ef}}$. However, the minimum and maximum spacing rules for radial, s_0 and s_r, as well as tangential distances, s_t, from Section 7.2.5 must always be observed.

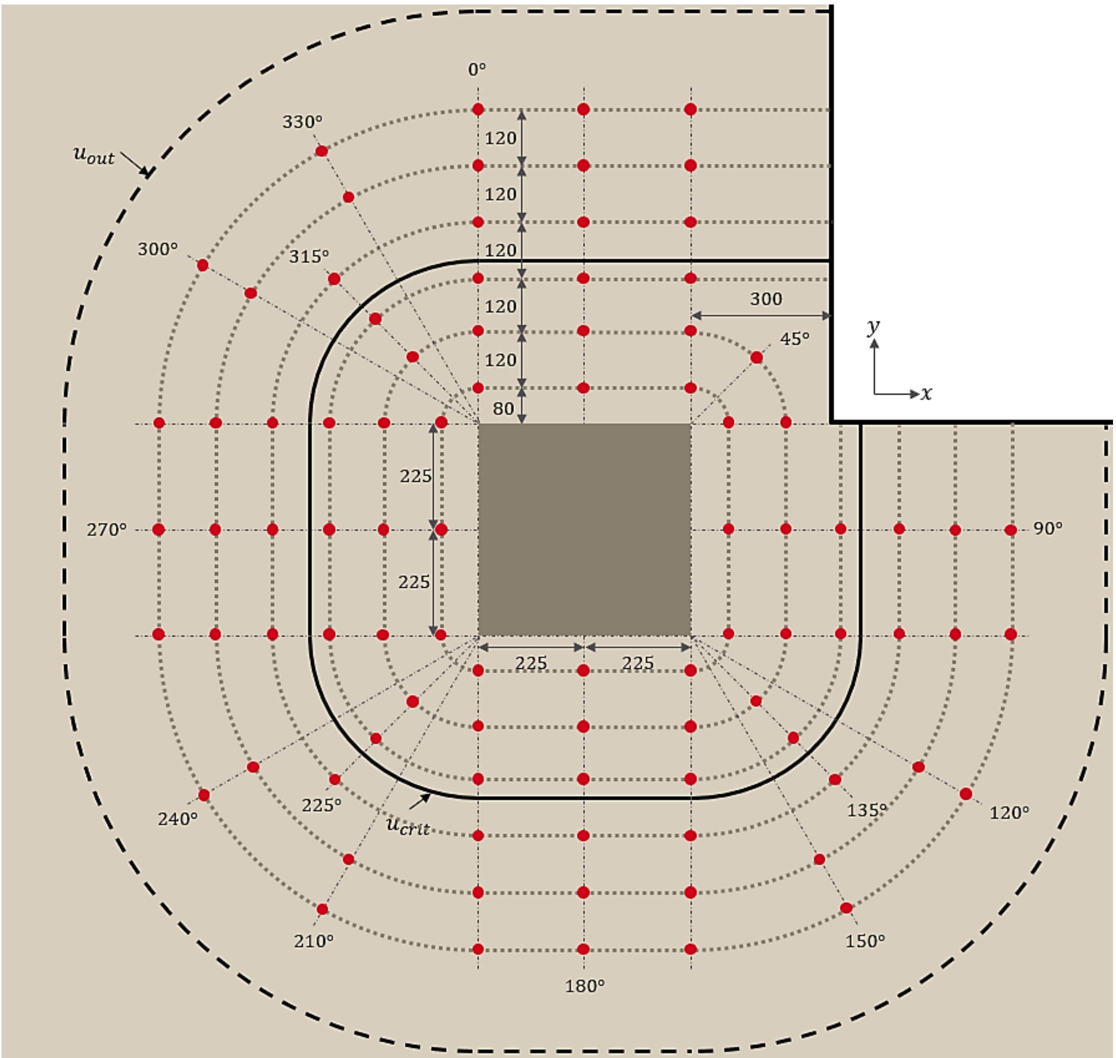
Installation according to the Manufacturer's Product Installation Instructions (MPII):

- Strengthening solution: HIT-RE 500 V4 + HAS-U 8.8 M12 + Filling Set
- Maximum installation torque, T_{inst}: 40 Nm
- Hole diameter in the slab, d_0: 14 mm
- Residual cover, c_{res}: 35 mm
- Hole depth in the slab, l_{sw}: 190 mm
- Proposed drilling method: Hammer-drilled (HD) with Drilling Aid
- Concrete condition: Dry Concrete

Specification:

90 numbers of Hilti HIT-RE 500 V4 + HAS-U **8.8 M12** threaded rods + Filling Set embedded at **190 mm** per installation instructions in DIBt abG Z-15.5-387 for Hammer drilling **(HD)** in **Dry** concrete with Drilling aid is used. **First** reinforcement perimeter must be positioned **80 mm** from the face of the column, with **subsequent** perimeters spaced at **120 mm** from the first perimeter. Refer to the construction drawing for reinforcement spacing within each perimeter.

The suggested reinforcement layout is provided below:



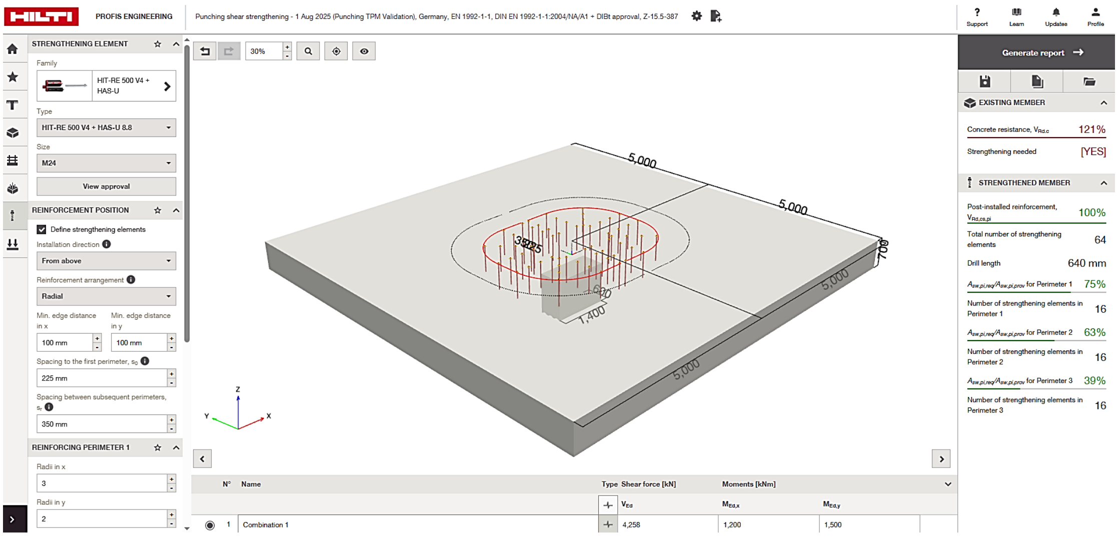
Note: dimensions in millimeters and not to scale.

8. PROFIS Engineering's Punching Shear strengthening module

As with the design of punching shear reinforcement such as stirrups cast within concrete members, manually finding the optimum strengthening solution for them can be a very repetitive and time-consuming exercise with the number of different choices of diameter, spacing, and positioning. Hilti's **cloud-based design software** PROFIS Engineering includes a **dedicated module** for assessing and strengthening concrete members deficient in punching shear that assists structural engineers when evaluating the resistance of existing members and strengthening them, thereby ensuring a safer and more efficient workflow.

Some **key benefits** of using PROFIS Engineering's Punching Shear Strengthening module include:

- Selecting the slab and the relevant compression member (e.g., column or wall).
- Defining the slab dimensions, geometry, and material parameters to verify the need for strengthening under a new punching shear force.
- Defining the strengthening reinforcement diameter and radial spacings.
- PROFIS Engineering generates the layout and calculates the total required strengthening elements based on the previously defined inputs.
- PROFIS Engineering displays the utilization ratios for verification of the existing and strengthened concrete, and the steel utilization per perimeter.
- For documentation, PROFIS Engineering produces a comprehensive design report with the calculation steps and provides the necessary information for detailing the reinforcement.

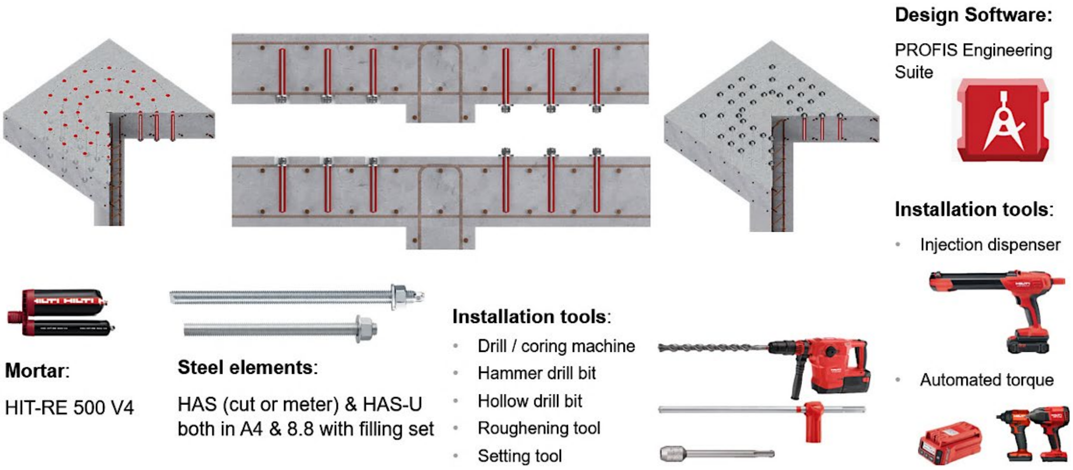


9. Hilti Solutions for punching shear strengthening

Hilti's new HIT-Punching Shear strengthening system is approved with **DIBt aBG Z15.5-387** and the key tools and accessories required for installation are summarized below.

HIT-RE 500 V4 mortar + following strengthening reinforcement:

- HAS(-U) rods A4: M12, M16, M20, and M24
- HAS(-U) rods 8.8: M12, M16, M20, and M24
- Hilti Filling Set (8.8 & A4): M12, M16, M20, and M24



10. Summary

Transforming and reusing older structures can offer many advantages over new-build, with each structure requiring fulfilment of specific objectives when strengthened. Based on the chosen design philosophy, the structural engineer can address punching shear deficiencies in slabs and foundations through various methods, some less invasive than others. The use of post-installed HIT-Punching shear strengthening system consisting of HAS(-U) threaded rods with the HIT-RE 500 V4 mortar, is a novel example of a minimally invasive method that can significantly enhance the punching shear resistance and deformation capacity of a slab or foundation.

Suitably assessed and granted a general construction technique permit (*aBG*) as a system by DIBt, engineers can use the familiar Eurocode 2-based design approach integrated into Hilti's PROFIS Engineering Suite to arrive at a feasible solution when selecting between the key design parameters such as diameter, spacing, and others. With its intuitive interface, the new Punching Shear Strengthening module aims to save designers and engineers time during the design phase, helping them bringing value to their clients while also contributing to a safer and more resilient built environment.

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