STRENGTHENING CONCRETE MEMBERS IN SHEAR

Tackling new problems with familiar solutions

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1. INTRODUCTION AND BACKGROUND

Over the past two decades, the construction industry has come under increasing scrutiny to reduce its environmental footprint and under pressure to reuse existing building stock to meet rising socio-economic demands. 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 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, 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 labour 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, torsion, etc., 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 fibre-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 overview of 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 shear reinforcement, which in 2024 was granted a general construction technique permit (**aBG**) by the Deutsches Institut für Bautechnik (**DIBt**).

2. OVERVIEW OF SHEAR BEHAVIOR IN CONCRETE

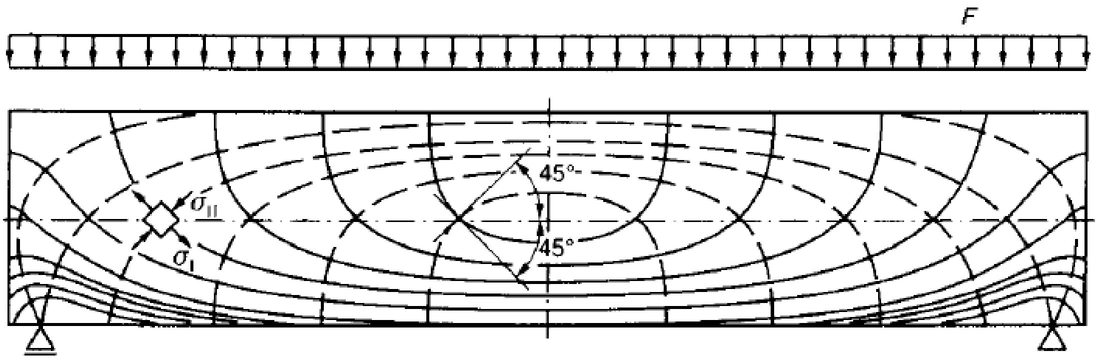


Figure 1: Representation of tensile and compressive stresses in a beam under a uniformly distributed load, represented by solid and dashed lines respectively [3].

A typical beam, uniformly loaded transverse to its length, causes compressive stresses produced by the supports to form an arch (dashed lines), while the tensile stresses curve into a catenary or a suspended chain (solid lines). These two principal stresses, denoted by σ_I and σ_{II}, act perpendicular to each other on a representative square element in Figure 1, generating a "shear" stress along the sides of the element if it is rotated such that its sides are parallel to the beam's longitudinal and transversal directions. The resulting shear stress stretches and compresses the opposite corners. The shear stress is highest at the middle of the beam depth – where it is inclined at ±45° to both stresses – and reduces to zero near the top and bottom surfaces where compressive and tensile stresses are dominant.

Since concrete has high compressive but low tensile strength, it will crack perpendicular to the tensile stresses at a certain load. To further maintain the integrity of the beam, reinforcement is placed to curtail the cracks within acceptable limits. In the cracked state, the beam resists by a combination of: (1) the uncracked concrete in the compression zone; (2) the dowelling action of any longitudinal reinforcement present; and (3) aggregate interlock across the tension cracks. However, the haphazard nature of these three effects acting concurrently does not generate a 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 [4].

Together with longitudinal reinforcement, a beam also containing shear reinforcement thus has four elements, illustrated by Figure 2:

1. A compression chord of concrete due to bending,
2. A tension chord of flexural steel reinforcement in tension due to bending,
3. Concrete struts in compression between the inclined cracks caused by shearing, and
4. Vertical ties in tension that link the compression and tension chords.

This forms an analogous truss consisting of struts and ties, which along with the stress field model was first developed by Mörsch [5] and Ritter [3] in the early 1900s based on crack patterns observed whilst conducting load tests on reinforced concrete beams, and assists designers in visualising the load paths.

Later in the 1960s, the theory of plasticity provided a theoretical foundation for the design with "strut-and-tie" models. Under this theory, any state of equilibrium in a member – the truss model being one such state of equilibrium between the applied loads and support reactions – that does not violate plasticity conditions requiring plastic deformations until the reinforcement yields, provides a lower-bound estimate of the real resistance. This results in a conservative estimate and forms a sound foundation for the design of members with shear reinforcement in all modern concrete design codes.

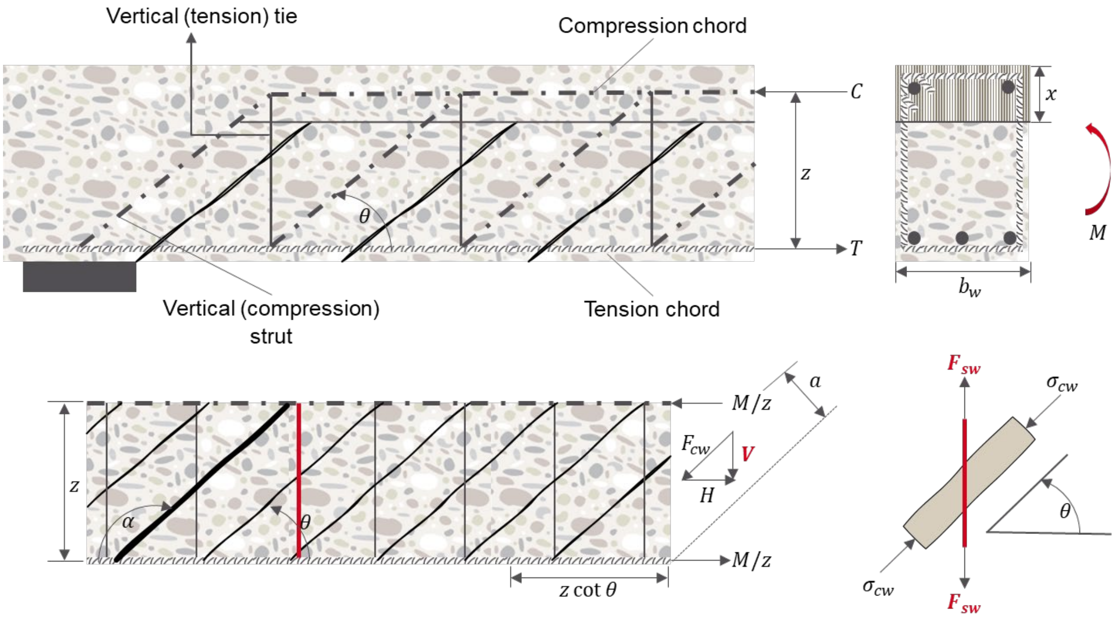


Figure 2: A concrete beam with shear reinforcement, represented by the Mörsch-Ritter truss containing the compression and tension chords, the inclined compression strut, and the vertical tension tie [5] & [6].

For the truss model to function reliably for shear, the shear 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, via the concrete's tensile strength or, most commonly, via direct supports where the shear reinforcement bends with or without the presence of longitudinal reinforcement in the compression zone [7].

3. DESIGNING CONCRETE MEMBERS FOR SHEAR

3.1 General principles

Design distinguishes between concrete members with and without shear reinforcement, where the latter are lightly loaded floor slabs and certain foundations that typically have a wider cross-section and are not subject to concentrated loads. For both types of members, a designer would first check the requirement for shear reinforcement based on empirically derived formulations.

3.2 Design without shear reinforcement to EN 1992-1-1 & SIA 262:2013

The design for shear strength of concrete members without shear reinforcement consists of empirical equations that derive from decades of extensive testing by experts across the world, in turn derived from national traditions and local testing. Although this leads to an absence of a unified and established physical model across the relevant standards, the results are usually comparable in most conditions. In its common text for instance, EN 1992-1-1:2004 recommends the following expression for the resistance of concrete members without shear reinforcement, with modifications to specific parameters, such as $C_{Rd,c}$ and k, contained within the different National Annexes:

- Shear strength without shear reinforcement: $V_{Rd,c} = \left[C_{Rd,c}k(100\rho_l f_{ck})^{\frac{1}{3}}\right] b_w \cdot d$ (1)

In SIA 262:2013, the resistance of concrete members without shear reinforcement has minor differences:

- Shear strength without shear reinforcement: $V_{Rd} = k_d \tau_{cd} d_v b_w$ (2)

3.3 Design with shear reinforcement to EN 1992-1-1 & SIA 262:2013

A key feature of modern design standards assumes that when the concrete by itself cannot resist all acting shear stresses, any provided shear reinforcement such as that in Figure 2 must solely resist all stresses. Therefore, much like the diagonal compression members in an open-form truss, varying the angle of inclination, $\boldsymbol{\theta}$, will play a significant role in determining the maximum resistance of the concrete compression strut before it crushes, $\boldsymbol{V_{Rd,max}}$, and the resistance of the shear reinforcement before it yields, $\boldsymbol{V_{Rd,s}}$, with the lesser of the two governing the total shear resistance, $\boldsymbol{V_{Rd}}$. Where design demands shear reinforcement, three criteria determine the amount of shear reinforcement required:

1. A minimum amount to avoid yielding of the shear reinforcement when the first shear cracks occur, $\boldsymbol{A_{sw,min}}$.
2. The actual shear reinforcement required to carry the design load, $\boldsymbol{A_{sw}}$.
3. The ratio between shear reinforcement and the concrete section, $\boldsymbol{\rho_w}$, that induces yielding of the shear reinforcement and precludes a brittle crushing failure of the concrete under compression.

Design standards such EN 1992-1-1:2004 and SIA 262:2013 determine the amount of shear reinforcement with the “strut-and-tie” or “stress field model” shown in Figure 3 using a consistent design formula with only a minor variation to the limits on the strut angle, $\boldsymbol{\theta}$. Thus:

- Yield force per stirrup: $F_{wi} = A_{sw} \cdot f_{ywd}$ (3)
- Number of stirrups within Δx: $n = \frac{z \cdot cot\,(\alpha)}{s}$ (4)
- Forces in all stirrups within Δx: $\boldsymbol{V_{Rd,s} = f_{ywd} \cdot A_{sw} \cdot \frac{z \cdot cot\theta}{s}}$ (5)
- If stirrups are inclined ($\alpha \neq 90°$): $V_{Rd,s} = f_{ywd} \cdot \frac{A_{sw} \cdot z}{s} \cdot (cot\theta + cot\alpha) \cdot sin\alpha$ (6)

- For EN 1992-1-1:2004, the stirrup inclination is: $45° \leq \alpha \leq 90°$
- The corresponding minimum shear reinforcement: $\rho_{w,min} = 0{,}08 \cdot \frac{\sqrt{f_{ck}}}{f_{yk}}$ (7)
- For SIA 262:2013, the stirrup inclination is: $45° \leq \beta \leq 90°$
- The corresponding minimum shear reinforcement: $\rho_{w,min} = 0{,}001 \cdot \sqrt{\frac{f_{ck}}{30}} \cdot \frac{500}{f_{sk}}$ (8)

Maximum shear resistance limited by crushing of the compressive strut: $\boldsymbol{V_{Rd,max}} = \frac{\boldsymbol{b_w \cdot z \cdot \alpha_{cw} \cdot v_1 \cdot f_{cd}}}{\boldsymbol{\cot\theta + \tan\theta}}$ (9)

From the same model, the maximum *effective* amount of shear reinforcement is calculated using the following expression (assuming $cot\theta = 1$):

$$\frac{A_{sw,max} \cdot f_{ywd}}{b_w \cdot s} \leq \frac{1}{2} \alpha_{cw} \cdot v_1 \cdot f_{cd} \tag{10}$$

Assuming a variable strut inclination: $\left(\frac{A_{sw}}{s}\right)_{max} \leq \frac{\alpha_{cw} \cdot v_1 \cdot f_{cd}}{f_{ywd}} \cdot b_w \cdot \sin^2\theta$ (11)

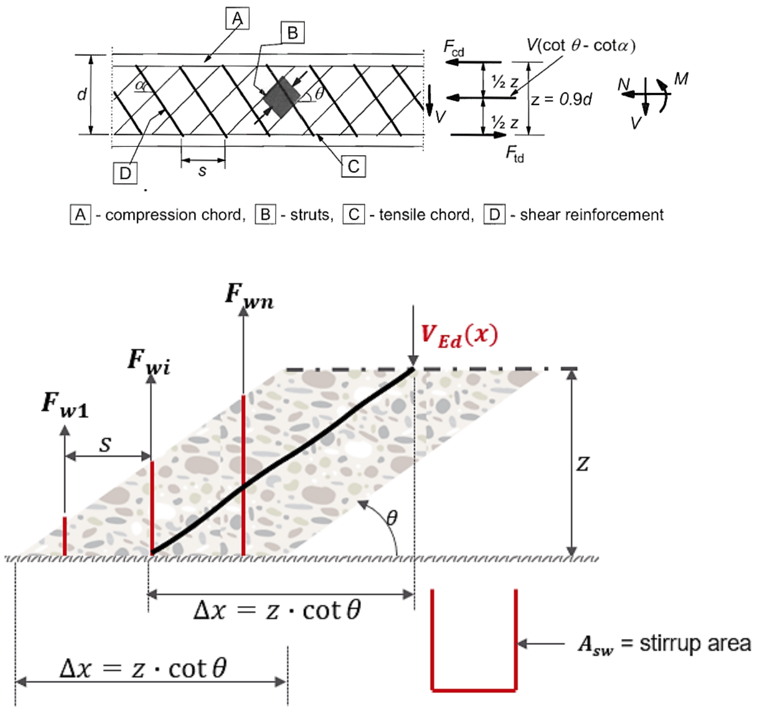


Figure 3: Design model for shear resistance with shear reinforcement: (top) Strut-and-tie model from EN 1992-1-1:2004 [8]; (bottom) Sketch with vertical stirrups

The angle of inclination, $\boldsymbol{\theta}$, increases proportionally to the magnitude of the shear force applied on a concrete member and design for shear according to modern design standards enables the structural engineer to select, between a specific range, a higher strut inclination that increases the strut resistance to deal with higher shear forces, $\boldsymbol{V_{Ed}}$. This reduces the contribution from the shear reinforcement and therefore requires more reinforcement to satisfy the demand, as illustrated in Figure 4.

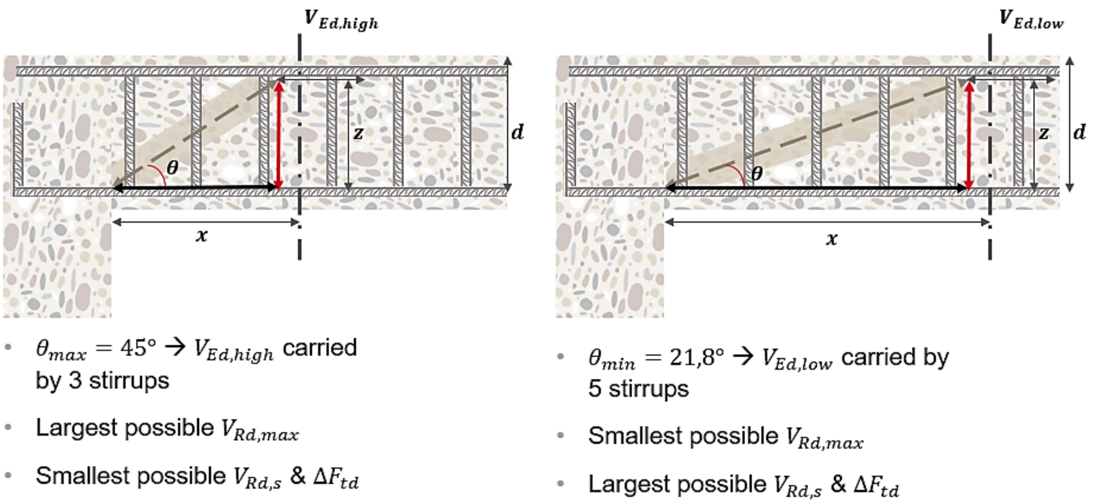


Figure 4: Schematic representation of the impact of the strut inclination on the resistance of the compression strut and the shear reinforcement: (left) maximum; and (right) minimum permissible inclination

Conversely, optimising the shear design by reducing the strut inclination, θ, for a given shear load, V_{Ed}, leads to:

a) A higher force in the inclined strut that leads to concrete crushing under a lower shear load, $\boldsymbol{V_{Ed}}$. The shear load under which the concrete will crush, $\boldsymbol{V_{Rd,max}}$, is the maximum shear strength of a section and, providing higher resistance through additional reinforcement, $\boldsymbol{V_{Rd,s}}$, becomes unnecessary. Higher amounts shift the failure mode towards crushing of the concrete struts.
b) A wider distribution of the vertical tension members means more stirrups can resist the shear load, $\boldsymbol{V_{Ed}}$, leading to a lower amount of required shear reinforcement.
c) A higher tensile force imposed the longitudinal reinforcement, $\Delta \boldsymbol{F_{td}}$, that requires anchorage at the supports.

4. APPROACHES TO STRENGTHEN MEMBERS DEFICIENT IN SHEAR

From the two aforementioned design standards, the resistance of a concrete member against shear depends on these parameters:

a) Concrete strength, $\boldsymbol{f_{ck}}$
b) Section width, $\boldsymbol{b_w}$, and height, $\boldsymbol{h}$
c) Effective depth to the flexural reinforcement from the top of the compression fibre, $\boldsymbol{d}$
d) Length of the support, $\boldsymbol{a_v}$
e) The amount of longitudinal reinforcement, $\boldsymbol{A_{sl}}$
f) The amount of shear reinforcement (also called transverse reinforcement), $\boldsymbol{A_{sw}}$

Existing 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 parameters. Although improving one or several of these parameters enhances shear resistance, in an existing structure the concrete strength (a) cannot be modified *a posteriori*, and adding more supports reduces the slenderness of the member, such as adding columns to beams or adding beams to slabs. However, these additional supports will need to transfer load to the foundations. Depending on functional requirements, enhancing one or more parameters (b) to (f) by using different interventions is possible as shown in the following paragraphs. Typically, only a part of the strengthening interventions is performed with proprietary products and, more often, solutions are tailored to the project at hand.

4.1 Increase the section width and / or height

The width and effective depth of a concrete section can be increased by employing either a concrete overlay for planar members, such as floor slabs, foundations and walls, or a reinforced concrete jacket for linear members, such as beams and columns. Illustrated in Figure 5 & Figure 6, both approaches will simultaneously enhance the flexural resistance and the stiffness, thereby also reducing deflection, and are useful when shear resistance is not the only deficiency to address. In scenarios where members require strengthening solely for shear, both approaches may have notable drawbacks:

1. The concrete overlay or jacket adds a substantial additional weight that affects other members in the load path, including the foundation.
2. Moreover, the increase in effective depth is even less than the thickness of the overlay, with the resulting effective depth laying in the center of gravity of all tension reinforcement in existing concrete and in overlay, i.e., below the tension reinforcement of the overlay.
3. The reinforcement is concentrated in the vicinity of the supports where shear is the highest.

Examples of proprietary solutions in the industry:

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

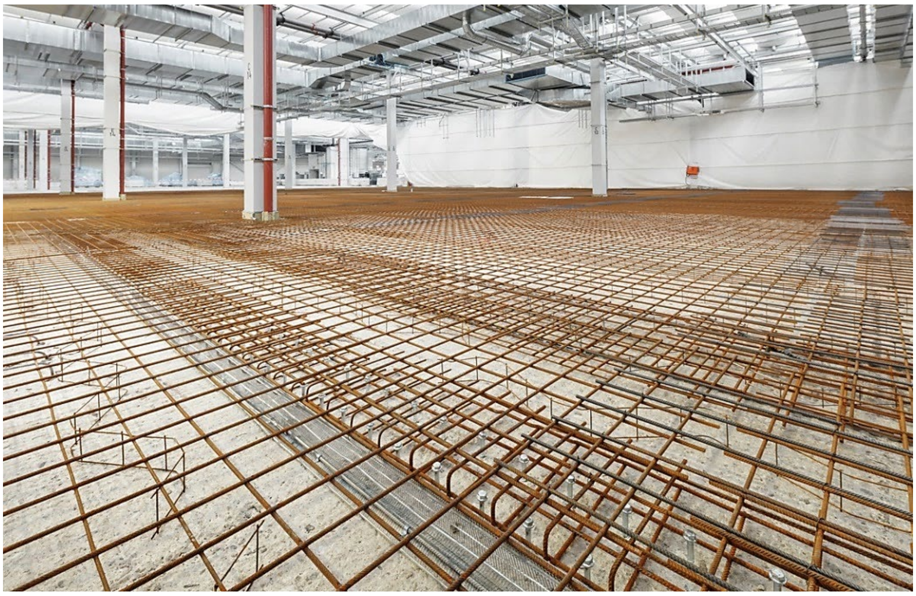


Figure 5: Example of post-installed reinforcement used in Concrete Overlays

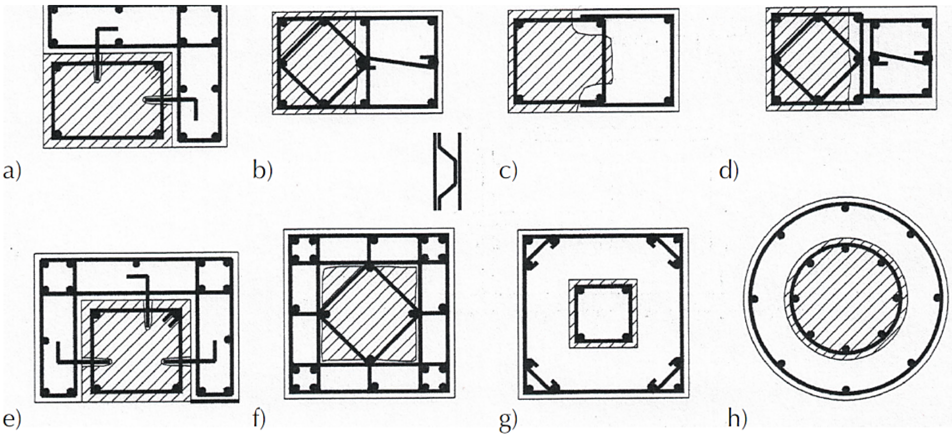


Figure 6: Examples of concrete jacketing, reproduced from [2]

4.2 Increase the support length

Increasing the size of the column or support distributes the shear load over a larger area, which reduces the average shear stress by shifting the face of the support closer to the midspan of the beam or slab. Several solutions can be employed:

Examples of solutions in the industry: post-installed steel capital or strut; extending the concrete supports (e.g., corbels).

4.3 Increase the flexural resistance

Increasing the amount of flexural reinforcement enhances overall ductility, the section stiffness, and reduces crack widths by improving aggregate interlock over the cracks, which in turn increases the shear resistance. Enhancements to flexural reinforcement are possible by applying glued laminates or installing near-surface-mounted reinforcement at the supports and midspan of sections where flexural demand is the highest, with the reinforcement consisting of fibre-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. 6.2a of EN1992-1-1 results in the shear resistance, $V_{Rd,c}$, increasing by not more than 25%.

Examples of solutions in the industry: CFK laminates, memory steel laminates, memory steel bars, near surface mounted reinforcement.

4.4 Increase the shear resistance using laminates

Exemplified by Figure 7, strengthening members deficient in shear, such as beams, is also possible by gluing laminates onto the sides perpendicular to the beam length or at an incline. Typically, this method is deemed to be highly effective, but the end anchorage of the laminates can pose challenges when subject to high loads.

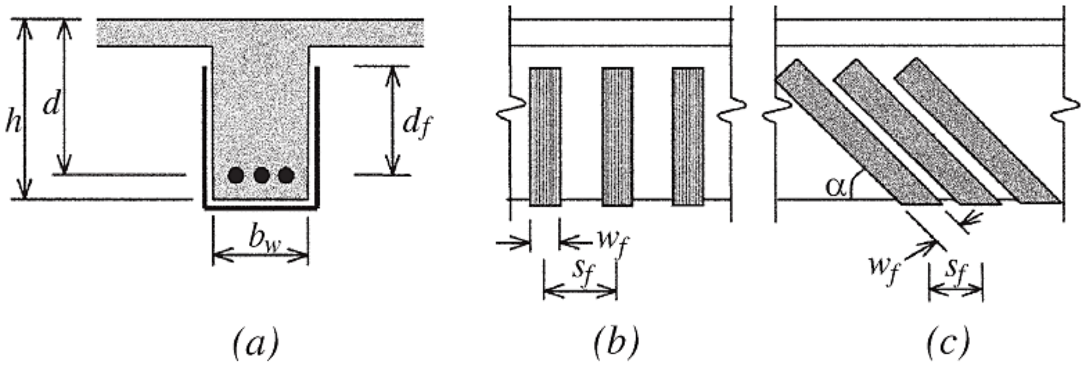


Figure 7: Use of FRP laminates perpendicular or inclined to the beam length, reproduced from Figure 11.4 [9]

4.5 Increase the shear resistance using post-installed reinforcement

Alternatively, a simpler 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, 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 it has 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 failure is the yielding of steel. 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 8 shows three different options of using HZA-P and HAS(-U) threaded rods embedded partially (HZA-P and HAS(-U)) and through-bolted (only HAS(-U)).

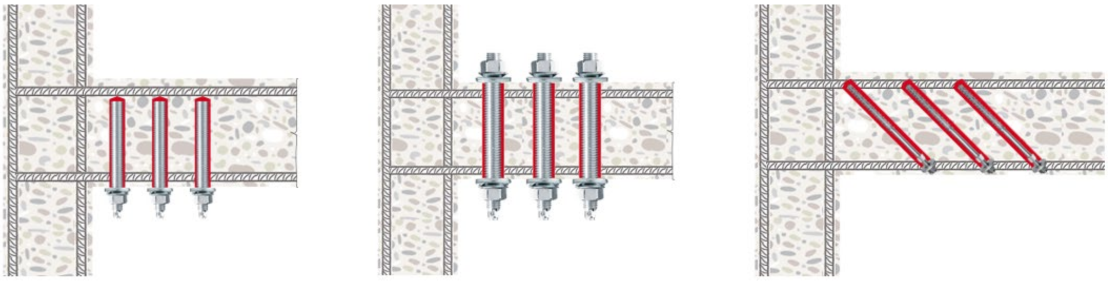


Figure 8: Increasing shear reinforcement using: (a) partially embedded HAS(-U) rods installed perpendicular to the beam length; (b) through-bolted HAS(-U) installed perpendicular to the beam length; and (c) partially embedded HZA-P inclined to the beam length

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

4.6 Special solutions & combinations

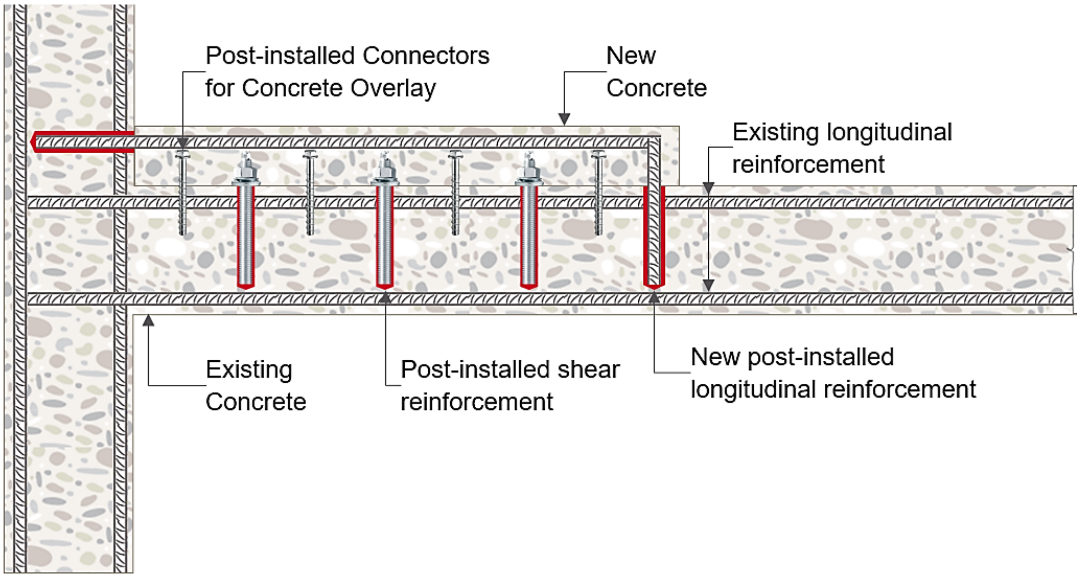


Figure 9: Example of a special solution combining post-installed shear reinforcement with a concrete overlay (the overlay may also extend over the full length of the existing length)

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-fibre laminate that is installed through two inclined holes and prestressed, as opposed to a normal installation without specially created holes.

Figure 9 illustrates another example that may significantly enhance shear resistance combines post-installed shear reinforcement with a concrete overlay. Other combinations that cannot increase concrete member thickness may combine fibre laminates with post-installed shear reinforcement to meet the respective flexure and shear demands.

5. QUALIFICATION OVERVIEW OF POST-INSTALLED SHEAR REINFORCEMENT

Whilst cast-in systems of shear reinforcement see widespread application in the construction industry, the use of *post-installed* steel elements to strengthen concrete elements deficient in shear is presently not covered 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 shear resistance. In such scenarios, Annex D of EN 1990:2002 [10] 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.

As per the European Technical Assessment (ETA)-20/0541 [11], 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 concrete members to enhance their shear resistance in reinforced concrete members has not been investigated previously. Therefore, a comprehensive testing plan was recently conducted to assess the behavior of this innovative shear strengthening solution and 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 transverse eccentricity and, 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 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 that is 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-15.5-383*, 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 HILTI HAS / HAS-U THREADED RODS

Apart from the HZA bonded reinforcement solution, a new Hilti 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 to a fixed value. The solution is granted a national general construction technique permit (*aBG*) **Z-15.5-383** by DIBt and uses the provisions for **Design assisted by testing** contained in **Annex D of EN 1990**. This section contains an overview of the assessment, design and installation of post-installed threaded rods as reinforcement in shear deficient concrete members.

The adopted resistance model is entirely consistent with the design provisions in DIN EN 1992-1-1/NA [12] and DIN EN 1992-2/NA [13]. The required verifications resemble Equations (6.8) and (6.9) of DIN EN 1992-1-1/NA, for the resistance of steel reinforcement and the maximum strut resistance, respectively, with Section 4 of this document covering the background of these equations. This resistance model also uses the Variable Strut Inclination method that permits adjustment of the inclined compressive strut angle to balance the forces between the compression strut and the strengthening elements, potentially leading to economies in design from fewer required rods.

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

$$V_{Ed} \leq V_{Rd} = min\,(V_{Rd,max}\,, V_{Rd,s}) \tag{12}$$

6.1 Verification of the compression struts

When the post-installed shear strengthening rods are installed perpendicular to the concrete member's longitudinal axis, the installation angle is $\alpha = 90°$ and the resistance of the compression strut per Equation (6.9) of DIN EN1992-1-1 in conjunction with DIN EN1992-1-1/NA, *NCI to 6.2.3(1)* is:

$$V_{Rd,max} = \frac{\boldsymbol{b_{w,eff}} \cdot \alpha_{cw} \cdot z \cdot v_1 \cdot f_{cd}}{\cot\theta + \tan\theta} \tag{13}$$

Here, the effective width of the strengthened section, $\boldsymbol{b_{w,eff}}$, replaces the overall section width, $\boldsymbol{b_w}$, with a transverse eccentricity parameter, $\boldsymbol{e_{inst}}$, that increases to either a maximum of **50 mm** or $\boldsymbol{b_w/6}$ in relation to the beam's center of mass, depending on the positioning of the reinforcement during installation. Note, however, this eccentricity **applies only to a single row of reinforcement** since a single eccentric tension tie in the truss model induces torsion into the structural member proportional to the applied shear force. Thus:

$$b_{w,eff} = b_w - \min\left(50\,mm, \frac{b_w}{6}\right) \tag{14}$$

The other parameters in the equation are no different than those contained in DIN EN 1992-1-1/NA:

- Lever arm $z = 0{,}9d$, with an upper limit of $z = max\big(d - 2c_{v,l}; d - c_{v,l} - 30\,mm\big)$. Here, the effective section depth to the flexural reinforcement is $\boldsymbol{d}$, and $\boldsymbol{c_{v,l}}$ is the concrete cover of the longitudinal reinforcement in the compression zone, illustrated in Figure 10:

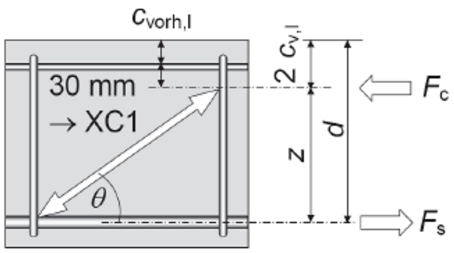


Figure 10: Definition of the inner lever arm, *z*, reproduced from Figure H6-11 [7]

Note: When $V_{Ed} \leq V_{Rd,cc}$, the expression $\frac{1{,}2+1{,}4\sigma_{cp}/f_{cd}}{1-V_{Rd,cc}/V_{Ed}}$ representing the upper limit of $\cot\theta$ becomes negative and is beyond the prescribed upper and lower bounds; in such scenarios, $\cot\theta = 3.0$.

- The dimensionless factor, $\boldsymbol{\alpha_{cw}} = 1$ accounts for the state of stress in the compression chord
- The strength reduction factor for concrete cracked in shear, $\boldsymbol{v_1} = 0.75$
- The design compressive strength of concrete, $\boldsymbol{f_{cd}} = \alpha_{cc} f_{ck}/\gamma_c$, where $\alpha_{cc} = 0.85$ and $\gamma_c = 1.5$
- The strut angle, $\boldsymbol{\theta}$, is bound by Eq. (6.7aDE), DIN EN1992-1-1/NA:

$$1 \leq \cot\theta \leq \frac{1{,}2+1{,}4\sigma_{cp}/f_{cd}}{1-\boldsymbol{V_{Rd,cc}}/V_{Ed}} \leq 3{,}0 \tag{15}$$

- In Eq. (15), $V_{Rd,cc} = 0{,}5 \cdot 0{,}48 \cdot f_{ck}^{1/3}\left(1 - 1{,}2\frac{\sigma_{cp}}{f_{cd}}\right) \cdot \boldsymbol{b_{w,eff}} \cdot z,$ (16)
- When designing using DIN EN1992-2/NA, the strut angle is limited by Eq. (6.107aDE):

$$1 \leq \cot\theta \leq \frac{1{,}2+1{,}4\sigma_{cp}/f_{cd}}{1-\boldsymbol{V_{Rd,cc}}/V_{Ed}} \leq 1{,}75 \tag{17}$$

As $V_{Ed} > V_{Rd,cc}$, this expression will continue to result in $\cot\theta > 3{,}0$ until a sufficiently large shear force $V_{Ed} \gg V_{Rd,cc}$ reduces $\cot\theta < 3{,}0$. This newly *calculated* upper limit corresponds to the maximum inclination of the compressive strut for members with minimum shear reinforcement, and a corresponding reduced resistance in the strut. Figure 11 below illustrates a scenario where an increasing shear force in turn increases the strut angle calculated from Eq. (6.7aDE), DIN EN1992-1-1/NA.

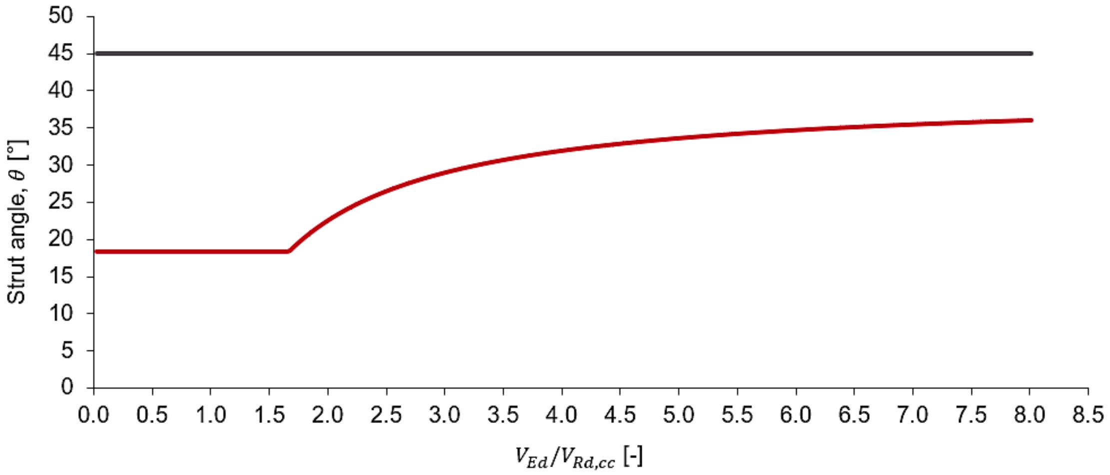


Figure 11: Representative relationship between the ratio $V_{Ed}/V_{Rd,cc}$ with the calculated strut angle (the horizontal line at 45° represents the maximum strut angle)

Note: As good practice, this verification should not be ignored when strengthening existing structures.

Although not explicitly required for the verification of the steel or strut resistance, the design shear, V_{Ed}, generates an additional tensile force, ΔF_{td}, in the longitudinal reinforcement that increases when the strut inclination decreases according to EN 1992-1-1, 6.2.3(7). This force is in addition to the tension imposed on the longitudinal reinforcement from the bending, requiring a separate check for potential yielding and anchorage failure of these bars. In a new-build structure, this verification for additional tension may not be required if the maximum bending along the concrete member, $M_{Ed,max}$, was used whilst designing the reinforcement and its anchorage.

6.2 Verification of the compression struts

With the post-installed strengthening rods installed orthogonal to the longitudinal axis of the concrete member, the installation angle $\alpha = 90°$ and the resistance of the steel rods per DIN EN 1992-1-1/NA is:

$$V_{Rd,s} = \boldsymbol{k_{pi}} \cdot \boldsymbol{k_s} \cdot f_{ywd} \cdot \boldsymbol{a_{sw}} \cdot z \cdot \cot\theta \tag{18}$$

While the parameters for the lever arm, $\boldsymbol{z}$, and the strut angle, $\boldsymbol{\theta}$, do not change from the verification of the compressive strut, the value for $\boldsymbol{f_{ywd}}$ stems from the assessment and is 390 MPa for both A4 stainless steel and 8.8 carbon steel rods. The stressed area of the post-installed strengthening rods, $\boldsymbol{a_{sw}}$, per unit length of the concrete member combines the number of rows of rods in the transverse direction, $\boldsymbol{n_{swt}}$, the spacing in the longitudinal direction, $\boldsymbol{s_{wl}}$, and the stressed cross-section area of each rod, $\boldsymbol{A_{sw}}$, provided below in:

$$a_{sw} = \frac{n_{swt} \cdot A_{sw}}{s_{wl}} \tag{19}$$

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

Table 1: Geometrical and material parameters for use in Eq. (18) and (19)

This expression introduces two new parameters that derive from the assessment:

- A size-dependent coefficient, $\boldsymbol{k_s}$, as a function of the lever arm, z (in meters), for member thicknesses, $\boldsymbol{h}$, between 200-2200 mm, with a larger lever arm reducing this coefficient by:

$$k_s = \begin{cases} 1{,}0 & when\ z \leq 0{,}75m \\ 1{,}15 - 0{,}20 \cdot z & when\ z > 0{,}75m \end{cases} \tag{20}$$

- The coefficient for post-installed strengthening, k_{pi}, that carries a fixed value combining:
  - A statistically derived **reliability** factor that compares the strengthening rods to cast-in reinforcement,
  - A durability factor accounting for the long-term effects on mortar's bond strength,
  - A factor for installation sensitivity and tolerances,
  - A factor accounting for the presence of bending and shear cracks in the anchorage zone of the rods relative to the installation direction. As shown below, this coefficient is consistent for all rod sizes.

Hilti shear strengthening rods	Rod size	One-sided installation (Configuration A)	One-sided installation (Configuration B)
Coefficient for post-installed shear strengthening, $\mathbf{k_{pi}}$	M12	0.735	0.588
	M16		
	M20		
	M24		

Table 2: Performance parameter k_{pi} used in the verification of the strengthening rods

Although the strengthening system allows for installation from either the tension or compression side of a concrete member, two aspects impact the system performance: (1) the presence of flexural cracks or axial tension around the tips of the strengthening rods; and (2) the possibility that shear cracks pass below the ends of the strengthening rods. The first aspect reduces system efficacy when the rods are installed from either direction in the presence of bending cracks, and the possibility of reduced efficacy for the second aspect arises only when installing from the compression side.

Illustrated in Figure 12 & Figure 13, **Configuration A** represents conditions when the rods are installed from the tension side of a concrete member and clearly pass through the shear cracks but are unaffected by flexural cracks that are common in simply supported conditions. Conversely, the **Configuration B** scenario arises when rods are installed from the compression side of a simply supported concrete member or when installed on either side of a fixed support subject to large hogging (negative) moments and a high shear force.

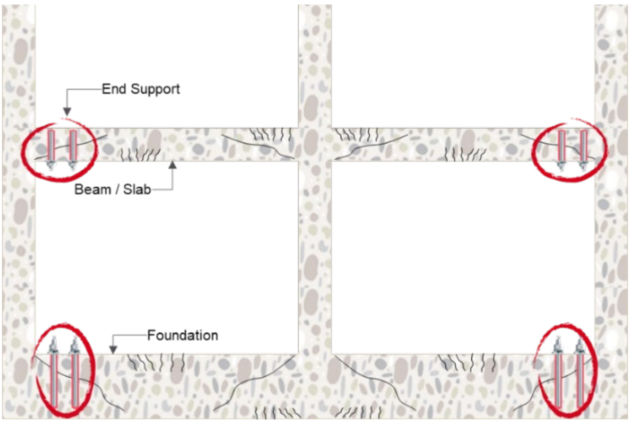


Figure 12: Configuration A, where installation is from the tension side of the member without the impact of flexural cracks

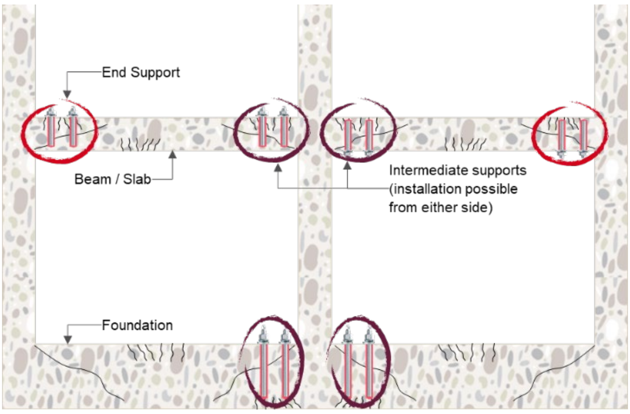


Figure 13: Configuration B, where installation is from the compression side of the member or installed at either the tension or compression side where flexural cracks develop together with shear cracks

6.3 Requirements for detailing the strengthening reinforcement

Installation length, l_{sw}

As evidenced from Equations (14) and (19) for the post-installed shear strengthening, the design model does not require an explicit consideration of the installation length, l_{sw}, in the verifications since this is a function of the section height, h, and the "residual" cover, c_{res}, see Figure 14. 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.

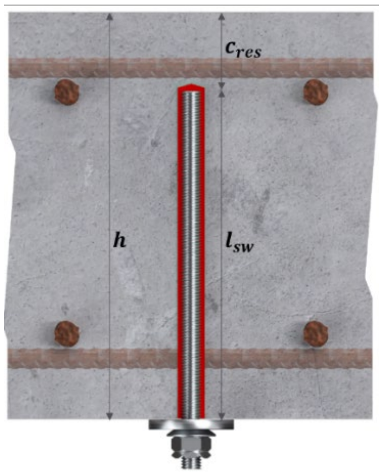


Figure 14: Simplified schematic of the installation, from [14]

From a design perspective, a fixed installation length ensures that the shear reinforcement is anchored in the compression chord of the member, enabling the formation of the truss model. As mentioned previously in Section 2 of this document, the truss model on which shear design is predicated on the shear reinforcement enclosing, or hooking, 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, c_{res}, of 60mm leaves the installation length, l_{sw}, at a mere 140mm, 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 section height and reinforcement diameter is required per the relationship in Table 3 [14].

	Diameter of the strengthening reinforcement			
	M12	M16	M20	M24
Minimum section height, h_{min} [mm]	200	400	600	
c_{res} [mm]	35	40	45	60

Table 3: Correlation between the minimum section height, residual cover, and strengthening reinforcement diameter, from Table 3 of [14]

Spacing, s_{wl}

Apart from easing the distribution of concrete aggregates evenly during casting, DIN EN 1992-1-1/NA does not define a minimum spacing between shear reinforcement such as stirrups. Without exceptions, post-installed reinforcement does require a defined minimum spacing to avoid splitting between the rods and a potential reduction in the overall shear resistance. Table 4 below provides the minima in the longitudinal and transverse directions.

Diameter of the strengthening reinforcement	Minimum longitudinal spacing, $s_{wl,min}$ [mm]	Minimum transverse spacing, $s_{wl,min}$ [mm]
M12	120	120
M16	160	160
M20	200	200
M24	240	240

Table 4: Minimum center-to-center spacing for each strengthening element, reproduced from Table 11 of [14]

As can be seen by Table 5 and Table 6, the maxima for spacing in both longitudinal and transverse directions follow the stipulations in DIN EN 1992-1-1/NA, *Tables NA.9.1 and NA.9.2* for linear members and *NCI to 9.3.2 (4)* for planar members, using $V_{Rd,max}$ from Eq. (1) without modifications.

Shear force to strut resistance ratio	Maximum longitudinal spacing, $s_{wl,max}$ [mm]	Maximum transverse spacing, $s_{wt,max}$ [mm]
$V_{Ed}/V_{Rd,max} \leq 0,3$	$\min\,(0.7h, 300\ mm)$	$\min\,(h, 800\ mm)$
$0,3 < \frac{V_{Ed}}{V_{Rd,max}} \leq 0,6$	$\min\,(0.5h, 300\ mm)$	$\min\,(h, 600\ mm)$
$\frac{V_{Ed}}{V_{Rd,max}} > 0,6$	$\min\,(0.25h, 200\ mm)$	

Table 5: Maximum center-to-center spacing in linear members (e.g., beams), reproduced from Tables NA.9.1 and NA.9.2 of [12]

Shear force to strut resistance ratio	Maximum longitudinal spacing, $s_{wl,max}$ [mm]	Maximum transverse spacing, $s_{wt,max}$ [mm]
$V_{Ed}/V_{Rd,max} \leq 0,3$	$0.7h$	h
$0,3 < \frac{V_{Ed}}{V_{Rd,max}} \leq 0,6$	$0.5h$	
$\frac{V_{Ed}}{V_{Rd,max}} > 0,6$	$0.25h$	

Table 6: Maximum center-to-center spacing in planar members (e.g., slabs), reproduced from NCI to 9.3.2 (4) of [12]

Edge distance, c_{wt}

Setting a minimum distance between the position of the strengthening rods and any concrete edge reduces the risk of splitting, with this minimum evaluated in the mortar's assessment ETA 20/0541 [11]. 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. Although not defined in DIN EN 1992-1-1/NA [12] for traditional shear reinforcement such as stirrups, the post-installed strengthening rods also require a maximum edge distance to generate at least one or more rows of strengthening elements in wider sections ($b_w \geq 350mm$), see Table 7.

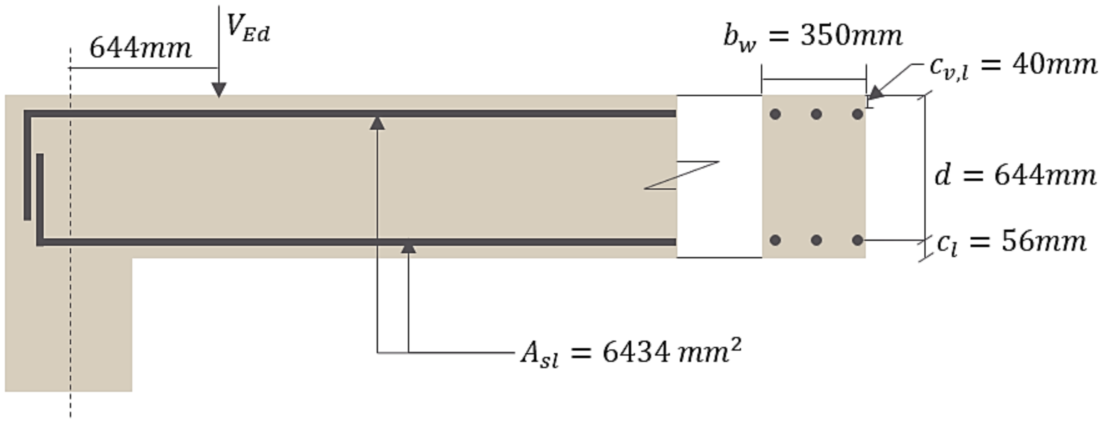
Drilling system	Rod size	Minimum edge distance, $c_{wt,min}$ [mm]		Maximum edge distance, $c_{wt,max}$ [mm]	
		Without Drilling Aid	With Drilling Aid	Linear members	Planar members
Hammer drilling with or without Hilti hollow drill bits	M12	$45\ mm + 0{,}06l_{sw}$	$45\ mm + 0{,}02l_{sw}$	$175\ mm$	$\max\,(175\ mm, 0{,}5h)$
	M16	$50\ mm + 0{,}06l_{sw}$	$50\ mm + 0{,}02l_{sw}$		
	M20	$55\ mm + 0{,}06l_{sw}$	$55\ mm + 0{,}02l_{sw}$	$250\ mm$	$\max\,(250\ mm, 0{,}5h)$
	M24	$60\ mm + 0{,}06l_{sw}$	$60\ mm + 0{,}02l_{sw}$		
Pneumatic drilling	M12	$50\ mm + 0{,}08l_{sw}$	$50\ mm + 0{,}02l_{sw}$	$175\ mm$	$\max\,(175\ mm, 0{,}5h)$
	M16				
	M20	$55\ mm + 0{,}08l_{sw}$	$55\ mm + 0{,}02l_{sw}$	$250\ mm$	$\max\,(250\ mm, 0{,}5h)$
	M24	$60\ mm + 0{,}08l_{sw}$	$60\ mm + 0{,}02l_{sw}$		

Table 7: Minimum and maximum edge distances based on drilling methods and tolerances, reproduced from Table 14 of [14]

Note: Unless explicitly considered in design, drilling and cutting through the flexural reinforcement should be avoided wherever possible to prevent an adverse impact to the structure. If this cannot be avoided, for instance to facilitate drilling in densely reinforced areas, additional measures with the explicit agreement of the Engineer in Charge of design are required to compensate for a loss of flexural reinforcement.

7. DESIGN EXAMPLE

An existing, simply supported beam with a cross section 350 mm x 700 mm of grade C30/37 spans 8.0 meters and is loaded by a uniformly distributed load of 142 kN/m at the ultimate limit state. The clear cover for top and bottom longitudinal reinforcement is 40 mm, with the cross-sectional flexural steel, $A_{sl} = 6434\,mm^2$ from 8H32. The resistance of the existing member is verified per DIN EN1992-1-1/NA & strengthened per *aBG Z15.5-383*.



Total ultimate load on the beam, $w = 142 \cdot 8.0 = 1136\ kN$

Support reaction, $R = \frac{1136}{2} = 568\ kN$

6.2.1 (8) [8] Shear at distance *d* from the support, $V_{Ed} = 568 - (142 \cdot 0.644) = \mathbf{477\ kN}$

6.2.1 (5) [8] Verification of the existing section, $V_{Rd,c} \geq V_{Ed}$

Eq. 3.15 [12] Design compressive strength of concrete, $f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c} = \frac{0{,}85 \cdot 30}{1.5} = \mathbf{17\ MPa}$

Eq. 6.2.a [8] Shear resistance of the existing section, $V_{Rd,c} = \left[C_{Rd,c} k (100 \rho_l f_{ck})^{\frac{1}{3}} + k_1 \sigma_{cp}\right] b_w d \geq V_{Rd,c,min}$

6.2.2 (1) [8] Ratio of longitudinal reinforcement, $\rho_l = \frac{A_{sl}}{b_w d} = \frac{6434}{350 \cdot 644} = 0.0285 \geq \mathbf{0.02}$

6.2.2 (1) [8] Member height-dependent coefficient, $k = 1 + \sqrt{200/644} = \mathbf{1.557} \leq 2.0$

NDP Zu 6.2.2 (1) [12] Minimum design shear stress (with interpolation $600\ mm < d \leq 800\ mm$),

$$v_{min} = \begin{cases} \frac{0{,}0525}{\gamma_c} k^{3/2} f_{ck}^{1/2} \text{ if } d \leq 600\ mm \\ \frac{0{,}0375}{\gamma_c} k^{3/2} f_{ck}^{1/2} \text{ if } d > 800\ mm \end{cases} = \frac{0{,}0492}{1.5} \cdot (1.557)^{\frac{3}{2}} \cdot (30)^{\frac{1}{2}} = \mathbf{0.349\ N/mm^2}$$

Eq. 6.2.b [8] Minimum design shear resistance, $V_{Rd,c,min} = v_{min} b_w d = 0.349 \cdot 350 \cdot 644 = \mathbf{78.7\ kN}$

Design shear resistance, $V_{Rd,c} = \left[0.1 \cdot 1.557 \cdot (100 \cdot 0.02 \cdot 30)^{\frac{1}{3}}\right] 350 \cdot 644 = \mathbf{137.4\ kN}$

$\therefore V_{Rd,c} \leq V_{Ed}$, strengthening required!

6.2.1 (5) [8] Verification of the strengthened section, $V_{Rd} = \min\,(V_{Rd,s}\ ;\ V_{Rd,max}) \geq V_{Ed}$

NCI Zu 6.2.3 (1) [12] Lever arm, $z = \min\,(0.9 \cdot 644\,; \max(644 - 2 \cdot 40\,; 644 - 40 - 30) = \mathbf{574\ mm}$

NDP Zu 6.2.3 (2) [12] The strut angle is determined by: $1.0 \leq \cot\theta \leq \frac{1{,}2+1{,}4\sigma_{cp}/f_{cd}}{1-V_{Rd,cc}/V_{Ed}} \leq 3.0$

Eq. 6.7bDE [12] Where $V_{Rd,cc} = 0.5 \cdot 0.48 \cdot f_{ck}^{\frac{1}{3}} \left(1 - 1.2\frac{\sigma_{cp}}{f_{cd}}\right) \cdot \boldsymbol{b_{w,eff}} \cdot z = 0.5 \cdot 0.48 \cdot \sqrt[3]{30} \cdot 350 \cdot 574 = \mathbf{149.8\ kN}$

Thus, the minimum strut angle, $\theta = \cot^{-1}\left(\frac{1.2}{1-149.8/477}\right) = 29.75°$ (use $\boldsymbol{\theta} = \mathbf{30}°$ for design)

Eq. 2.1 [14] Resistance of the compression strut, $V_{Rd,max} = \frac{1 \cdot 350 \cdot 574 \cdot 0.75 \cdot 17}{\cot(30)+\tan(30)} = \mathbf{1109.2\ kN}$

Eq. 6.17 [8] Additional tensile force from shear, $\Delta F_{td} = 0.5 \cdot 477 \cdot \cot(30) = \mathbf{413.1\ kN}$

Assuming installation of **two** rows of M16 ($\boldsymbol{A_{sw}} = \mathbf{157\ mm^2}$) spaced at **185 mm** centers along the beam length from the tension side, $\boldsymbol{k_{pi}} = \mathbf{0.735}$, the stressed area of the post-installed strengthening rods per meter length:

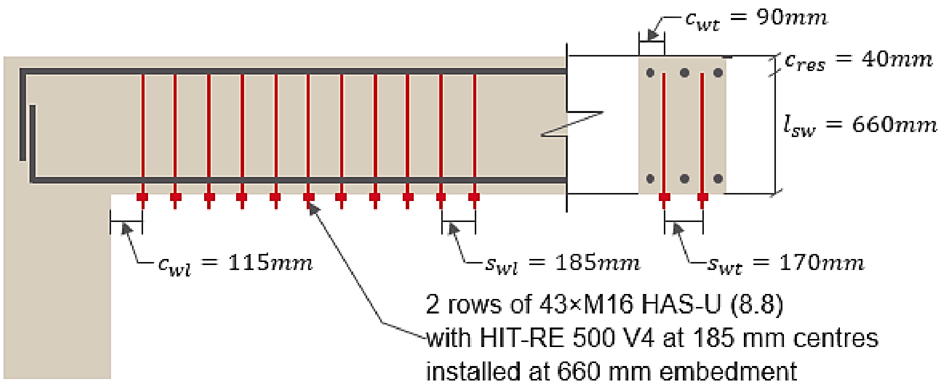
2.2.3 [14]

$$a_{sw} = \frac{n_{swt} \cdot A_{sw}}{s_{wl}} = \frac{2 \cdot 157}{185} \cdot 10^3 = \mathbf{1697.3\ mm^2/m}$$

Eq. 2.3 [14] Design shear resistance from post-installed shear reinforcement:

$$V_{Rd,s} = k_{pi} \cdot k_s \cdot f_{ywd} \cdot a_{sw} \cdot z \cdot \cot\theta = 0.735 \cdot 1 \cdot 390 \cdot 1.6973 \cdot 574 \cdot \cot(30) = \mathbf{483.7\ kN} > 477\ kN$$

$\therefore \boldsymbol{V_{Rd} > V_{Ed}}$**, design satisfied!**



Specification: Install 2 rows of **43×M16 HAS-U 8.8** with **HIT-RE 500 V4** spaced at **185 mm centers** along the beam length at an embedment of **660 mm,** with **rows 170 mm apart.**

Levers for Optimisation

For a uniformly distributed load used in the worked example above, the shear force, V_{Ed}, reduces in the central section of a structural member, effectively dividing the beam into three zones. Table 8 summarizes the results of the same calculation procedure for the above example, but with a stepped shear diagram shown in Figure 15 that demonstrates a **reduction** in the overall number of required elements **from 86 to 71.**

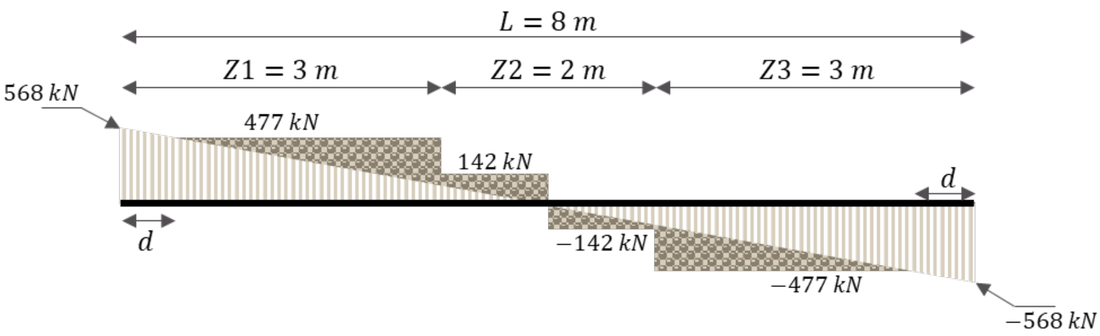


Figure 15: Stepped shear force diagram after sectioning the beam in three zones (Zone 2 is a common zone for $V_{Ed} = \pm 142\ kN$)

Zone	n_{wt} [-]	s_{wl} [mm]	θ [°]	a_{sw} [mm²/m]	$V_{Rd,max}$ [kN]	$V_{Rd,s}$ [kN]	Strengthening elements / zone
Z1	2	185	30	1697	1109.2	483.7	32
Z2	1	$s_{wl,max} = 300$	30	523	1109.2	149.1	7
Z3	2	185	30	1697	1109.2	483.7	32

Table 8: Design summary with three zones (maximum spacing in Zone 2 does not exceed the limits prescribed in [12]).

8. PROFIS ENGINEERING'S SHEAR STRENGTHENING MODULE OVERVIEW

As with the design of 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 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 Shear Strengthening module include:

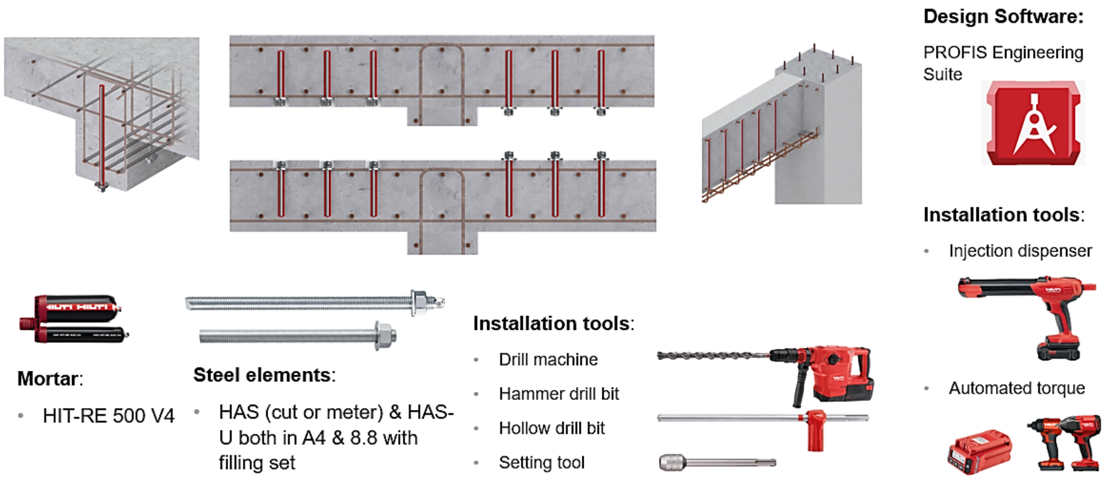
- Selecting the relevant linear and planar concrete member.
- Defining the existing concrete member type, geometry and material parameters to verify the need for strengthening under a new shear force.
- Dividing the concrete member into different zones to enable load inputs as per a stepped shear force diagram.
- Defining the strengthening reinforcement diameter, spacing and the optimal compression strut angle.
- PROFIS Engineering generates the layout and calculates the total required strengthening elements based on the previously defined inputs.
- PROFIS Engineering displays the utilisation ratios for verification of the existing concrete, the steel resistance from the shear strengthening reinforcement, and the maximum compressive strut resistance.
- 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 SHEAR STRENGTHENING

Hilti's new solutions for shear strengthening approved with **DIBt aBG Z15.5-383** 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 shear deficiencies in linear or planar concrete members through various methods, some less invasive than others. The use of post-installed shear reinforcement, such as Hilti's solution 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 shear resistance of a structural member.

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

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