



**HILTI**

**POST-INSTALLED  
REINFORCING BAR GUIDE**

HILTI ED 3500-A

CPC

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## 1. Post-installed reinforcing bars – what are they?

### 1.1 Post-installed reinforcing bars and their application in construction

A common and long-standing application of anchoring adhesives is the installation of deformed reinforcing bars (rebar) in holes drilled in concrete to emulate the behavior of cast-in-place reinforcing bars (Figure 2). These are commonly referred to as *post-installed reinforcing bars*. This application can be characterized as follow:

- Post-installed reinforcing bars* are embedded in adhesive in a hole drilled in existing concrete on one side of the interface and are usually cast into new concrete on the other side of the interface (Figure 1). The bars may be equipped with hooks or heads on the cast-in end, but are by necessity straight on the post-installed end;
- Post-installed reinforcing bars*, in contrast to *adhesive anchors*, are often installed with small concrete cover ( $3\phi > c > 2\phi$ , where  $\phi$  is the reinforcement bar diameter and  $c$  is the concrete cover). In such cases, the strength under tension loading of the *post-installed reinforcing bar* connection is typically limited by the splitting strength of the concrete (as characterized by splitting cracks forming along the length of the bar);
- Post-installed reinforcing bars* are typically not designed to resist direct shear loading in the manner of an anchor bolt and
- Post-installed reinforcing bars* are generally embedded as required to “develop” their design stress  $\sigma_{sd}$  using the basic required anchorage length, design anchorage length and splice length provisions of Eurocode2 [5]. In order to achieve ductility of the structure the design stress will often be close to the design yield strength.

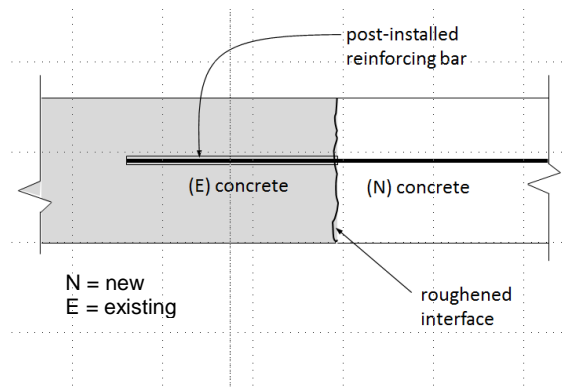


Figure 1– *Post-installed reinforcing bar*.

In some specific cases the tensile stress in *post-installed reinforcing bars* must be directly transferred into the concrete. In such cases they are designed as anchors taking account of the CCD-Method. The two design theories rebar theory and anchor theory, have peculiar differences which are summarized in Table 1 [16].

This Guide provides information regarding the design, detailing and installation of *post-installed reinforcing bars*. It does not address the design of *adhesive anchors* as governed by ETAG 001, part5 [1], CEN/TS 1992-4 [2] and TR 029 [6] anchor qualification and design provisions.

Table 1 – Main differences between anchor theory and rebar theory

<u>Key differences</u>	<u>Anchor theory</u>	<u>Rebar theory</u>
<b>Design standard</b>	EN 1992-4	EN 1992-1
<b>Load direction</b>	Tension, shear, combination of both	Tension
<b>Load transfer mechanism</b>	Utilization of tensile concrete strength	Equilibrium with local or global concrete struts
<b>Failure modes</b>	Steel failure, pull-out, splitting, concrete cone	Steel failure, pull-out, splitting
<b>Design results</b>	Capacity	Anchorage length
<b>Minimum concrete cover</b>	According to ETA	According to Eurocode2
<b>Allowable anchorage length</b>	$20\phi \geq l_b \geq 4\phi$	$60\phi \geq l_b \geq \max(0.3l_{brqd}; 10\phi; 100\text{mm})$

## 1.2 Application range

As noted above, *post-installed reinforcing bars* are typically used to develop a monolithic connection between new and existing concrete elements or structures. *Post-installed reinforcing bars* are used in both retrofit work and in new construction and are suitable for a wide range of applications.



Figure 2– Injecting Hilti HIT adhesive in a drilled hole with Hilti dispenser ED 3500-Ato to simplify installation of *post-installed reinforcing bars*.

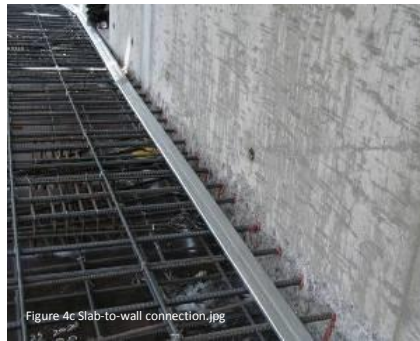
One of the most common classes of applications for *post-installed reinforcing bars* is the **extension of existing reinforced concrete (R/C) structural elements** such as slabs, walls, and columns (Figure 3), either to facilitate expansion of floor space or to address other functional changes in the use of the structure. Such applications usually involve the placement of large numbers of bars with close spacing. In some cases the *post-installed reinforcing bars* are installed close to the surface of the concrete (e.g., using the minimum concrete cover according to Eurocode2 provisions, whereby the presence of existing reinforcing must be taken into account). Where applicable, such as in a column, slab, or wall extension, it is generally preferable to place the *post-installed reinforcing bars* inside of the existing reinforcing bar cage, both to minimize spall during drilling and to ensure adequate cover. Avoidance of existing reinforcing is facilitated by the use of reinforcing detection equipment, such as the Hilti PS 250 or Hilti PS 1000 scanning systems (Figure 4).



a. Column extension



b. Slab-to-slab connection



c. Slab-to-wall connection



d. Wall extension



e. Steel column encasement extending from wall



f. Slab extension

Figure 3– Applications involving extension of existing construction with new elements using Hilti HIT-RE 500 and Hilti HIT-HY 200

Since the required embedment to satisfy anchorage length provisions of the building code (typically -40 to 60 bar diameters) often greatly exceeds typical anchoring embedment (generally limited to 20 bar diameters), special precautions may be necessary to ensure that the holes are drilled straight, the drilling process does not spall or otherwise damage the existing concrete or the existing reinforcing or other embedment. These may include the use of specialized tools such as the Hilti drilling alignment system and Hilti ferric- and GPR-based detection systems.

Another class of applications includes the **strengthening of existing concrete structures**, often to improve performance e.g. due to refurbishment (Figure 5).



Figure 4– Scanning for reinforcing bars and other embedded items with a Hilti GPR scanner.



a. Slab-to-wall connection



b. Wall-to-wall connection



c. Onlay shotcrete wall



d. Slab-to-column connection strengthening

Figure 5– Structural strengthening applications using Hilti HIT-RE 500

A third application class with unique requirements is the **extension, rehabilitation, and strengthening of existing concrete bridges** and other civil engineering structures (Figure 6). These applications are often distinguished by the need for enhanced resistance to corrosion and temperature extremes. Hilti has developed unique shear-friction solutions for bridge deck overlays<sup>1</sup> and offers hybrid adhesives (e.g., Hilti HIT-HY 200) with superior resistance to elevated temperatures [16].

<sup>1</sup> Contact Hilti for further information.



a. Bridge widening



b. Bridge deck rehabilitation



c. Bridge deck augmentation

Figure 6– Applications in bridge rehabilitation with *post-installed reinforcing bars*

### 1.3 Compatibility of post-installed reinforcing bars with cast-in-place reinforcing

*Post-installed reinforcing bars* are designed to transfer tension loads only. Extensive research programs have been conducted [7], [8], [9], [10], [11], [12], [13], [14] at laboratories in Europe and U.S. to verify that *post-installed reinforcing bars* installed with Hilti adhesive systems (HIT-HY, HIT-RE) demonstrate load transfer and load vs. displacement behavior comparable to cast-in-place reinforcing ( see Section 5).

When the value of minimum concrete cover is greater than  $3\phi$  (where  $\phi$  is the bar diameter) (i.e., where splitting does not limit the bond strength), *post-installed reinforcing bars* typically exhibit higher pull-out strength than cast-in-place bars of equivalent bar diameter and anchorage length. In fact, Hilti adhesive systems can generate equivalent uniform bond

strengths in *post-installed reinforcing bars* of up to 2.5 times that measured in tests with cast-in-place bars (see Section 5).

It should be noted that the method of load transfer between *post-installed reinforcing bars* and concrete in which they are anchored is bearing of the reinforcing deformations (lugs) on the adhesive surrounding them. These bearing stresses in turn are transferred from the adhesive to the surrounding concrete via adhesion and micro-friction, whereby the lateral dilation of the adhesive layer in response to the bearing stresses enhances the friction mechanism. The concrete in turn develops circumferential (hoop) stresses around the bars that can result in splitting cracks at certain load levels. This response is identical to that observed for cast-in-place reinforcing bars loaded in tension.

For near-edge bars subjected to tension loads the ultimate limit state behavior is characterized by splitting of the concrete along the bar or splice in response to the hoop stresses developed around the bar. Provided that the adhesive used can accommodate redistribution of stresses along the bar length in a manner similar to cast-in-place bars, *post-installed reinforcing bars* exhibit similar peak strengths to cast-in-place bars (for small values of concrete cover).

In 2006, the European Organization for Technical Approvals issued the technical report TR023 *Assessment of Post-installed Reinforcing Bar Connections* [15]. This document provides guidance for verifying that *post-installed reinforcing bar* connections built with a specific mortar system will exhibit comparable behavior to cast-in-place reinforcing bar connections in terms of load, and displacement behavior under several environmental conditions (see Section 5).

NOTE: Where it has been verified through appropriate qualification testing (in accordance with EOTA TR 023 [15] or similar procedures) that a given *post-installed reinforcing bar* system results in similar bond strength and displacement behavior as cast-in-place reinforcing bars, the design of *post-installed reinforcing bar* connections employing that system can follow the provisions for cast-in-place reinforcing bars.

In addition the intended use of the technical report are applications which are allowed with straight rebar according to Eurocode2 [5] as shown in Figure 8. This “limitation” is based on the fact that the tension forces from the rebar transferred to the concrete by bond should be balanced either by local (e.g. splices) or global compression struts to avoid brittle concrete failure. A tension loaded rebar may cause brittle concrete cone failure by using the tensile capacity of the surrounding concrete if not balanced by compression struts. Since this failure mode is prevented by means of global or local compressive struts, it is not considered in the national concrete code design rules.

#### 1.4 Application range of post-installed rebar

As a consequence of what has been declared in the previous chapter, the application range of post-installed rebar is limited by EOTA TR 023 [15] to:

- (a) Overlap joint for rebar connections of slabs and beams and overlap joint at a foundation of a column or wall by means of a non-contact splice. In this case the tension loads are transferred between adjacent bars via compression struts. The tension forces generated by the hoop stresses are taken up by the stirrups or transverse reinforcement, respectively, in the splice area.
- (b) Simply supported beams and anchoring of reinforcement to cover the line of acting tensile force



Based on strut and tie modelling (see Section 6.5.3), at the end support of a simply supported beam the node consist of a diagonal strut, the force from the tie and the supporting force (Figure 7).

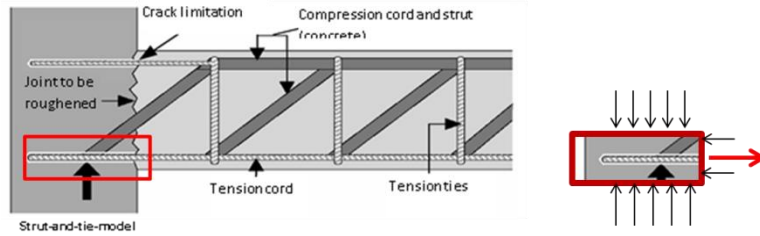


Figure 7 – Load bearing behavior of the end support of a simple supported beam avoiding the occurrence of concrete cone failure due to compression struts

(c) System connections stressed primarily in compression

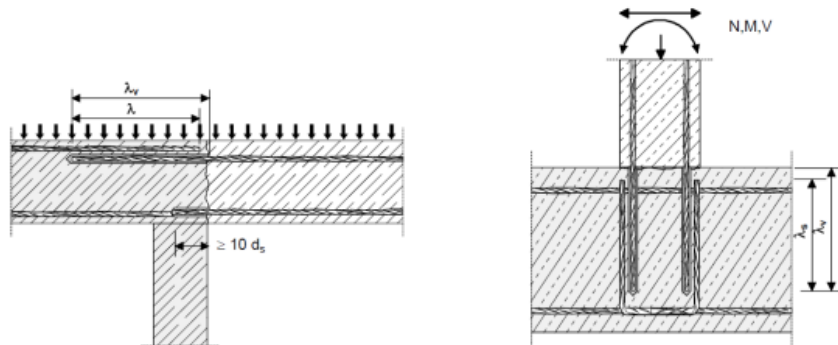


Figure 1.1: Overlap joint for rebar connections of slabs and beams

Figure 1.2: Overlap joint at a foundation of a column or wall where the rebars are stressed in tension

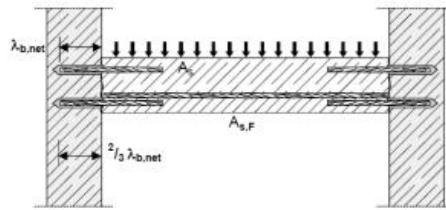


Figure 1.3: End anchoring of slabs or beams, designed as simply supported

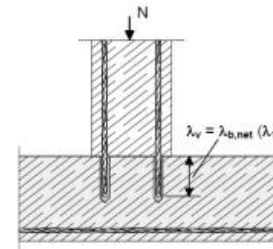


Figure 1.4: Rebar connection for components stressed primarily in compression. The rebars are stressed in compression

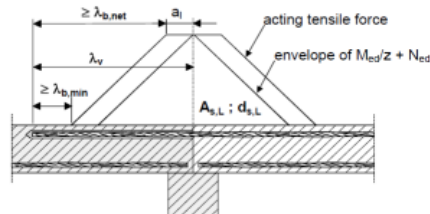
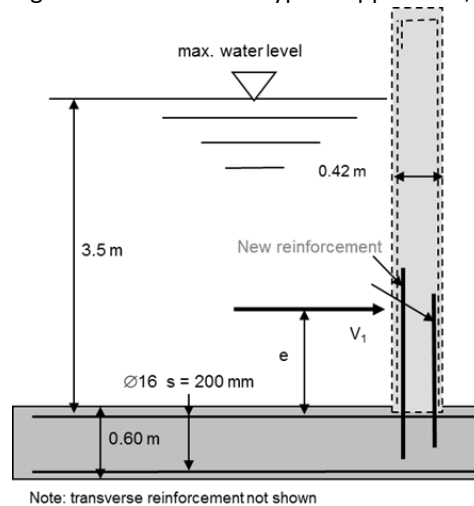


Figure 1.5: Anchoring of reinforcement to cover the line of acting tensile force

Figure 8 – Application range of post-installed rebar by EOTA TR023 [15]

The equilibrium of this node is provided by bond between rebar, adhesive and concrete. The strut and tie model following the loading path is shown in Figure 8. The anchorage condition of the post-installed rebar can be assessed as a CCT node (compression-compression-tension node). The post-installed reinforcing bar is loaded against the horizontal component of the diagonal strut, while the vertical component of the diagonal strut is in equilibrium with the supporting force. Due to the presence of the horizontal compression stresses in the area of the anchorage, the post-installed rebar may fail by pullout or steel failure but not by concrete cone failure. Similar conditions can be expected for anchoring of reinforcement to cover the line of acting tensile forces where post-installed rebar are used as reinforcement in tension areas. Therefore these applications are also covered by EOTA TR 023 [15].

Figure 9 shows also a typical application, connections subjected to a bending moment, of



*post-installed reinforcing bars* explicitly excluded from the applications of post-installed rebar by EOTA TR023 [15]. Such cases must be considered with specific models going beyond the approval based approach. After an extensive experimental and numerical analysis Hilti developed a design method for moment resisting connections made out of straight bars. On contrary, in cast-in place construction, such nodes are realized with hooked bars according to the Eurocode2 provisions.

Further details on the modelling can be found in section 6.

Figure 9– Examples of structural detailing with post-installed reinforcing bars not explicit covered in EOTA TR 023 [15]

## 2. How are they designed?

### 2.1 Design requirements

Design of *post-installed reinforcing bar* connections requires that the type, size, spacing and quantity and anchorage / splice length be established for the connection. This is typically based on either direct calculation of section forces, or a requirement to match existing reinforcement.

Additional design considerations may include:

- loading type (sustained, seismic, shock)
- fire requirements
- corrosion resistance
- detailing requirements based on element type (integrity reinforcement, etc.)

### 2.2 Jobsite constraints

Prior to designing a *post-installed reinforcing bar* connection, identification of the jobsite constraints is vital. Key parameters that should be accounted for in the design may include:

- existing reinforcement layout as given in drawings and confirmed on site using detection equipment (see Section 3.1).
- required proximity of new to existing reinforcing to satisfy conditions for non-contact lap splices, etc.
- required drilling method (hammer drill, core drill, Hilti Hollow Drill Bit)
- orientation of connection (downward, overhead, etc.)
- ambient air and concrete temperatures at time of installation
- type and condition of the concrete e.g. cracked, carbonized
- access and geometrical constraints

### 2.3 Required anchorage length

In general, the required bar embedment is based on the design anchorage length provisions of the code. Where geometrical or other practical constraints dictate, alternate procedures may be appropriate to establish bond length. The size of the bar and required anchorage length may also influence the type of adhesive system to be used. Adhesives with longer working time (e.g., Hilti HIT-RE 500-V3) are usually more appropriate for large diameter bars in combination with deep holes, whereby for small to medium bar diameters and shorter holes, systems with accelerated cure (e.g., Hilti hybrid adhesive HIT-HY 200) can increase installation efficiency. These considerations may be affected by the anticipated job site conditions (e.g., access and ambient air and concrete temperatures).

### 2.4 Connection detailing

The location of *post-installed reinforcing bars* with respect to existing reinforcement should be clearly indicated in the project documentation. In addition, the specifications and details should include:

- adhesive system
- bar type and size
- required design/splice anchorage length
- hole diameters and drilling method(s)
- requirements for preparation/roughening of existing concrete surface
- instruction on inviolability of existing reinforcement and embedded items as required
- requirements on training/certification of installers as required
- inspection/proof loading requirements

## 2.5 System specification

Specifications should correlate to the design assumptions and the specific job site requirements addressed in the project documentation. Substitutions based on a simple specification of bond stress may not be sufficient to ensure proper execution of the work.

## 2.6 Design of development and overlap length based on Eurocode2

The following reflect the design relevant sections from EOTA TR023, chapter 4 “Assumptions under which the fitness of use is to be assessed” and from the specific European Technical Approvals:

Design method for post-installed rebar connections

- The post-installed rebar connections assessed according to this Technical Report shall be designed as straight cast-in-place rebar according to Eurocode2 using the values of the design bond resistance  $f_{bd}$  for deformed bars as given in the relevant approval.
- Overlap joint for rebar: for calculation of the lap length of overlap joints the concrete cover at end-face of the post-installed rebar  $c_1$  shall be considered:

$$l_v \geq l_0 + c_1$$

with:  $l_0$  = required lap length

$c_1$  = concrete cover at end-face of bonded-in rebar

- The definition of the bond region in Eurocode2 is valid also for post-installed rebar.
- The conditions in Eurocode2 concerning detailing (e.g. concrete cover in respect to bond and corrosion resistance, bar spacing, transverse reinforcement) shall be complied with.
- The transfer of shear forces between new and old concrete shall be designed according to Eurocode2 [1].

Additional provisions

- To prevent damage of the concrete during drilling the following requirements have to be met:

- Minimum concrete cover:

$c_{min} = 30 + 0,06 l_v \geq 2ds$  (mm) for hammer drilled holes

$c_{min} = 50 + 0,08 l_v \geq 2ds$  (mm) for compressed air drilled holes

The factors 0,06 and 0,08 should take into account the possible deviations during the drilling process. This value might be smaller if special drilling aid devices are used.

Furthermore the minimum concrete cover given in clause 4.4.1.2, EC2: EN 1992-1-1: 2004 shall be observed.

- Minimum clear spacing between two post-installed bars  $a = 40 \text{ mm} \geq 4ds$

- To account for potentially different behavior of post-installed and cast-in-place rebar in cracked concrete,

- In general, the minimum lengths  $l_{b,min}$  and  $l_{o,min}$  given in the Eurocode2 for anchorage lengths and lap lengths shall be increased by a factor of 1.5. This increase may be neglected under certain conditions. The relevant approval states under which conditions the factor can be neglected for a specific adhesive.

### 3. How are they installed?

Jobsite constraints should be taken into consideration while designing and installing *post-installed reinforcing bars*.

**NOTE:** In addition to state-of-the-art adhesive anchoring systems, Hilti offers best in class detection and drilling equipment to facilitate the installation of *post-installed reinforcing bars* over a wide range of bar diameters and lengths.

#### 3.1 Location of existing reinforcement and other embedded items

The location of existing reinforcement is generally identified with scanning methods categorized as:

- a) Scanners that locate ferrous materials using magnetic fields (ferrous scanners, see Figure 10),
- b) Scanners that utilize GPR (ground-penetrating radar), and
- c) X-ray scanning equipment.

For reinforcing bars located within 200-250 mm of the concrete surface, ferrous scanners provide both bar location and size. For location of both ferrous and non-ferrous embedded items (e.g., aluminum conduit), GPR-based scanners are appropriate. For areas of heavy congestion or where existing reinforcing is too deep for ferrous or GPR systems, x-ray scanning methods may be necessary. Where available, it is generally preferable to supplement scanning results with as-built or original design documents.



Figure 10 – Using a Hilti PS 200 hand-held ferroskan scanner to locate and map existing reinforcing prior to beginning drilling

#### 3.2 Roughening the existing concrete surface

Surface roughening prior to casting new concrete against existing provides not only increased adhesion, but also increases the ability of the joint to transfer shear through friction. Where new concrete is to be applied to an existing concrete surface, roughening should be prepared according to the envisaged intended use according to Eurocode2 [5], the surface should be with at least 3 mm roughness at about 40 mm spacing. In case the surface layer of existing concrete is carbonated, the carbonated layer should be removed in areas that are to receive *post-installed reinforcing bars*. A rule of thumb is to remove the carbonated concrete over a circular area with a diameter given by the diameter of the bar plus 60mm ( $d_{\text{rough}} = \phi + 60\text{mm}$ ), see Figure 11.

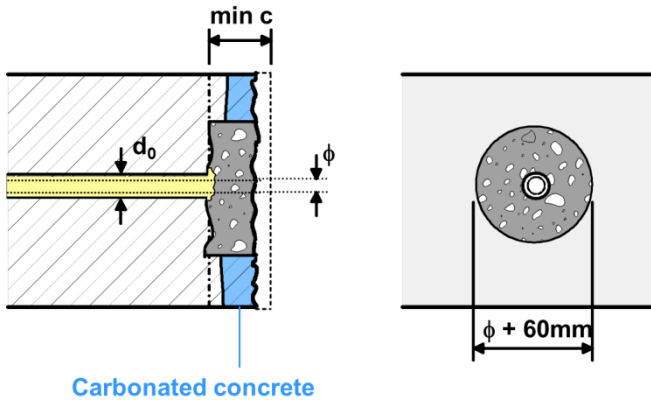


Figure 11 – Required removal of carbonated concrete over a circular area with  $d_{\text{rough}} = \phi + 60\text{mm}$ )

The required roughening may be accomplished by mechanical means (e.g., using a Hilti TE 76 ATC equipped with a bushing tool, see Figure 12), sand-blasting or water-blasting. It should be ascertained that the resulting surface does not contain loose material prior to placing the new concrete.



Figure 12 – Roughening a concrete surface with a Hilti TE 76 Combiammer

In the following a summary of Eurocode2 [5] provisions for cast-in is given as far as it concerns the design of post-installed reinforcing bar.

$$V_{\text{Edi}} \leq V_{\text{Rdi}}$$

$V_{\text{Edi}}$  is the design value of the shear stress in the interface and is given by:

$$V_{\text{Edi}} = \beta \cdot V_{\text{Ed}} / (z \cdot b_i)$$

Where:

$B [-]$  is the ratio of the longitudinal force in the new concrete area and the total longitudinal force either in the compression or tension zone, both calculated for the section considered.

$V_{\text{Ed}} [\text{N}]$  is the transverse shear force

$z [\text{mm}]$  is the lever arm of composite section

$b_i [\text{mm}]$  is the width of the interface

$V_{\text{Rdi}}$  is the design shear resistance at the interface and is given by:

$$V_{Rdi} = c \cdot f_{ctd} + \mu \cdot \sigma_n + \rho \cdot f_{yd} \cdot (\mu \cdot \sin \alpha + \cos \alpha) \leq 0.5 \cdot v \cdot f_{cd}$$

Where:

$c$  [-] and  $\mu$  [-] are factors which depend on the roughness of the interface

$f_{ctd}$  [N/mm<sup>2</sup>] is the value of the design tensile strength wo

$\sigma_n$  [N/mm<sup>2</sup>] is the stress per unit area caused by the minimum external normal force across the interface that can act simultaneously with the shear force, positive for compression and negative for tension. When  $\sigma_n$  is tension,  $c$  is equal to zero.

$\rho$  [-] is the ratio between the area of reinforcement ( $A_s$ ) crossing the interface, including ordinary shear reinforcement and the area of the joint ( $A_i$ ).

$\alpha$  is between 45° and 90° and it is the inclination of the rebar with the axis parallel to the contact interface

$v$  [-] is a strength reduction factor.

When the surface is roughened with the Hilti tool it can be assumed safely that  $c = 0.45$  and  $\mu = 0.7$ .

### 3.3 Installation of post-installed reinforcing bars with small cover

As with cast-in bars, *post-installed reinforcing bars* installed with Hilti HY products must be provided with sufficient concrete cover to prevent corrosion. Contrary, with Hilti RE products the cover is not needed, since the corrosion protection is only by density of the coating.

If the bar has been properly installed with adhesive surrounding the bar over its entire length, additional protection against corrosion is provided by the adhesive. Qualification of *post-installed reinforcing bar systems* under EOTA TR 023 [15] includes verification (through an accelerated aging test) of the protection of the adhesive against corrosion.

In addition, sufficient distance must be provided from the existing concrete edge to facilitate drilling without splitting and/or spalling, particularly when hammer- or rock-drilling equipment is used. Hilti drilling alignment aids can be employed with Hilti hand-held hammer drills to improve drilling accuracy (Figure 14) and as a consequence to reduce the distance either from the edge or from the adjacent bars. Detailed rules related to minimum concrete cover in case of drilling aid usage are available in the ETA of each product.

In the absence of other guidance, and where alignment aids or other techniques to maintain drilling accuracy are not used, the following relationships may be used to account for possible deviation of the drilled hole from its intended path in both edge distance and spacing:

Hammer drilled holes:  $c_{min,rqd} = 30 + 0,06l_v \geq 2\phi$  (mm)

Compressed-air (rock) drilled holes:  $c_{min,rqd} = 50 + 0,08l_v \geq 2\phi$  (mm)

Drill stand (e.g., core-drilled holes):  $c_{min,rqd} = 30 + 0,02l_v \geq 2\phi$  (mm)

Where:

$l_v$  is the drilling/anchorage length of the rebar in concrete in mm; and

$\phi$  is the diameter of the reinforcing bar in mm

$c_{min,rqd}$  is the minimum concrete cover to prevent damage of the concrete during drilling (Figure 13).

*Example:  $l_v = 255\text{mm}$ ,  $d_s = 12\text{mm}$*

*Hammer drilling:  $c_{min,rqd} \sim 45\text{mm}$*

*Compressed-air drilling:  $c_{min,rqd} \sim 75\text{mm}$*

Diamond coring:  $c_{min,rqd} \sim 35mm$

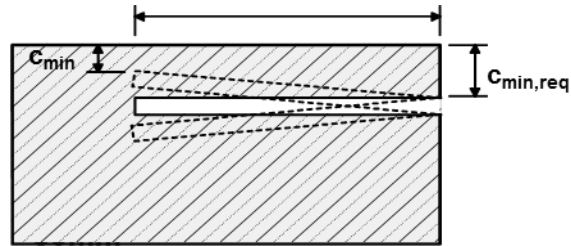


Figure 13 –  $c_{min,rqd}$  is intended to increase the probability that the end of the installed bar will remain within the minimum required concrete cover  $c_{min}$



Figure 14 – Drilling with alignment tool

Clearance requirements for core-drilled holes vary according to the type, diameter and length of core bits being used. Embedded items may also cause drill bits to deviate from the intended path.

As a matter of practicality, spacing of adjacent *post-installed reinforcing bars* should in general be maintained at 4 bar diameters or greater as clear spacing. Where applicable, Eurocode 2 [5] provisions for cover and bar spacing should be observed.



### 3.4 Drilling method

*Post-installed reinforcing bars* typically require deeper embedment to satisfy anchorage length requirements. The deep holes required for installing reinforcing bars are drilled using one of the following three methods:

- rotary-impact drills (hammer drills) equipped with standard or cruciform carbide bits or with Hilti Hollow Drill Bits (HDB)<sup>2</sup>
- compressed air drills
- diamond core drills utilizing either wet or dry coring technology

Each method is associated with advantages and disadvantages. Hammer drills (Figure 15) are readily available and are the preferred approach for most applications given their portability and ease of use. Hilti hammer drills produce a non-uniform hole surface especially suitable for enhancing bond (provided correct hole cleaning procedures are used). For longer holes, hammer drills may not be practical; they are also not always suitable for drilling through existing reinforcing where this is required.

The Hilti SafeSet™ system consists of Hollow Drill Bits (HDB) used in combination with Hilti Vacuum Cleaners (VC 40-U or VC 20-U). Hilti HDBs utilize the same state-of-the-art carbide drilling technology as Hilti TE-CX and Hilti TE-YX bits. The Hilti SafeSet system performs equally well in dry and wet concrete and eliminates the most load-affecting and time-consuming step in the installation process: cleaning the hole before injection of the adhesive.



Figure 15 – Drilling with a Hilti rotary-percussive drill equipped with Hilti SafeSet™ technology.

Compressed air drills offer speed and efficiency and produce a rough drilled hole surface which is suitable for bond, but the larger impact energy associated with compressed air may increase the tendency for damage in the concrete member, particularly if used in applications with small edge distance or reduced backside cover. Compressed air drills typically require larger edge distances/spacing/member thickness (see Section 3.3). For more information, contact Hilti.

For longer anchorage lengths, core drills are generally the preferred option (Figure 16 and Figure 17).

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<sup>2</sup> Hilti Hollow Drill Bits automatically remove concrete dust during drilling as part of the Hilti SafeSet™ System.



Figure 16 – Core drilling with a hand-held Hilti wet core drill with water-capture technology.



Figure 17 – Inclined core drilling with a Hilti drill stand.

In contrast to hammer drills, which fracture the concrete with impact energy, core drill bits utilize a sacrificial matrix containing diamond fragments to abrade the concrete. Hilti diamond core bits with laser-welded segments offer long life and exceptional drilling efficiency. Using extensions, core drills can produce very long, straight holes. The stiffness of the core barrel permits holes to be drilled with less deviation from the intended path, and they are capable of drilling through existing reinforcing without great effort. On the other hand, where the existing reinforcing must be protected, this feature of core drilling may be a liability. More importantly, core drills typically produce a very smooth hole that is usually covered with a thin film of dust deleterious to bond. Accordingly, core drilled holes must be thoroughly cleaned prior to injecting adhesive. Note also that some adhesive systems are not suitable for use with core drilled holes and in addition when the product is qualified, due to the smooth drilled holes surface, the bond strength for certain rebar diameter is reduced compare to hammer drilled holes. For qualified systems, specific hole cleaning procedures have been developed to optimize bond under these conditions, and are detailed in the Hilti *Instructions for Use* (generically, these instructions are known as the Manufacturer's Printed Installation Instructions, or MPII).

The Hilti SafeSet™ consists of the TE-Y RT "Flex fork" roughening tool which roughens the sides of the smooth diamond cored hole allowing increased mechanical interlock between mortar and concrete. This results in much higher bond strength values and furthermore reduced and simplified cleaning steps.



Figure 20 – Hilti TE-YRT roughening tool to specifically address the challenges of diamond cored drilled holes

For more info contact Hilti.

NOTE: Drilling through existing reinforcing or other embedded objects should in general not be undertaken prior to consultation with the engineer of record or other authority having jurisdiction.

NOTE: Correct hole drilling and cleaning are critical for the performance of *post-installed reinforcing bars*. Detailed instructions, referred to as *Instructions for Use*, accompany all Hilti anchoring products. For questions regarding correct installation Hilti offers expert advice through Hilti field representatives, nationwide Hilti Centers, Hilti Customer Service, and online at [www.hilti.com](http://www.hilti.com).

Figure 18 shows the influence of the drilling technique on the load-displacement behavior of *post-installed reinforcing bar* at shallow embedment. The adhesive tested, is not approved for diamond drilling and as a consequence the bond strength is dramatically lower than the hammer drilling or compressed air drilling the product was approved for.

Where the drilling method to be used has not been predetermined, it is advisable to use an adhesive that is suitable for all drilling methods (e.g., Hilti HIT-RE 500 V3). However, the bond strength when diamond drilling without roughening tool is used can be much lower compared to hammer drilling.

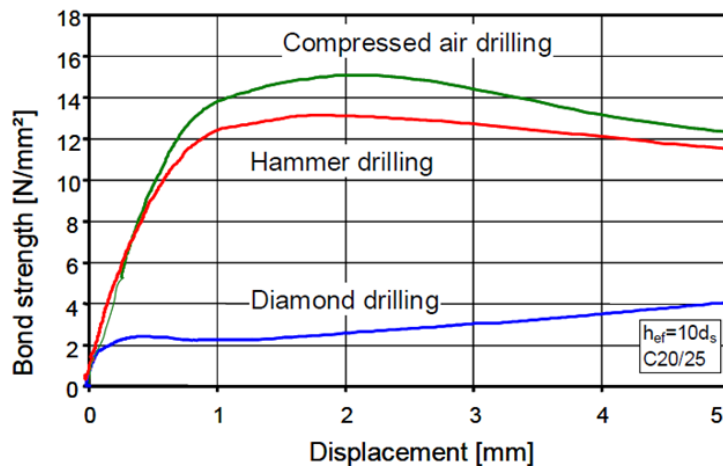


Figure 18 – Example of the influence of drilling method on the bond-displacement behavior of a *post-installed reinforcing bar* with an embedment of  $10 \phi$  in low-strength concrete and cleaned hole [14]

### 3.5 Hole cleaning

Bond between adhesive and concrete is directly influenced by the condition of the drilled hole wall at the time of adhesive injection. The concrete in which the *post-installed reinforcing bar* is to be installed may be dry, saturated or even partially or completely submerged at the time of installation.

NOTE: Where installation in water-saturated or submerged concrete is required, check that the adhesive system to be used is qualified for these conditions.

Wet diamond core drilling will necessarily result in a damp environment in the drilled hole. Cored hole cleaning generally involves sequential flushing the hole until clear water exits, blowing out of the hole with compressed air (Figure 19 and Figure 21) to remove debris and water, and the use of a wire brush (Figure 20) to mechanically scour the hole wall. All cleaning procedures end with the use of compressed air. (It is important to note that the use of compressed air may produce flying debris – eye protection should be worn at all times.)



Figure 19 – Hilti accessories for compressed air hole cleaning operations (partial) and Hilti Profi Rebar Accessory Set



Figure 20 – Hilti extension rod and Hilti HIT-RB matched-tolerance steel brushes for hole cleaning.

The importance of hole cleaning as specified in the Hilti *Instructions for Use* for the performance of *post-installed reinforcing bars* is indicated Figure 21b. For cases where adherence to multi-step hole cleaning procedures may not be possible, use of Hilti SafeSet™ technology with Hilti Hollow Drill Bits (HDB) is recommended.

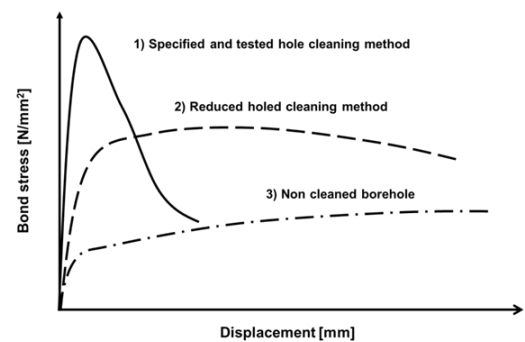


Figure 21 – Cleaning a hole using a compressed air blow-out tool (a) and Schematic representation of the potential influence of hole cleaning procedures on the measured bond and displacement of a post-installed reinforcing bar loaded in tension (b)

Hilti provides a number of accessories for cleaning deep drilled holes in accordance with the *Instructions for Use*. These include matched-tolerance wire brushes, brush extensions for long holes, attachments to facilitate power brushing, air wands, hose extensions, couplers and air nozzles. Hilti Profi Rebar Accessory Sets (Figure 19b) in a single package provide the necessary additional components for installation of *post-installed reinforcing bars*.

### 3.6 Selection of adhesive system

The suitability of Hilti adhesive systems for post-installed reinforcing bar applications has been verified for a wide variety of jobsite parameters. Nevertheless, the choice of the appropriate Hilti adhesive system (Figure 22) and injection equipment (Figure 23) for post-installed reinforcing bar installations is to a degree dependent on jobsite parameters, see Section 4.1.



Hilti HIT-RE 500 V3



Hilti HIT-HY 200-R

Figure 22 – Hilti anchoring adhesives suitable for *post-installed reinforcing bar* connections.

**NOTE:** Adhesives which have not been properly verified for *post-installed reinforcing bar* applications should not be used for structural or safety-related applications.

For example, if a fast-cure adhesive is specified for a large and deep bar installation, the time required to inject the adhesive may exceed the working time of the polymer. In such cases it may be impossible to insert the bar fully into the hole and/or the adhesive may not reach full strength. In particular, when adhesives are delivered in bulk quantities into a large drilled hole, the exothermic reaction associated with polymerization can result in excessive temperature rise which in turn can result in accelerated cure, further complicating bar installation.

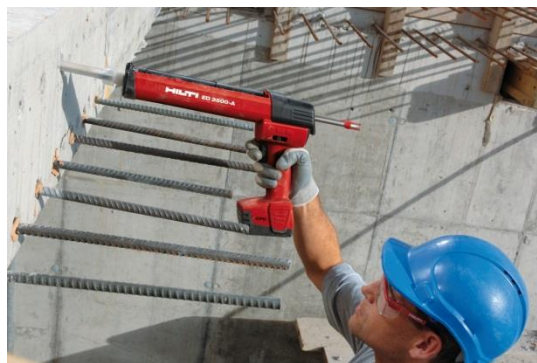


Figure 23 – Using Hilti battery dispenser in combination with Hilti HIT-RE 500 V3 / Hilti HIT-HY 200 for smaller bar diameters.

Conversely, injection of adhesives under sub-zero conditions can result in elevated viscosity, likewise making manual adhesive injection and bar installation difficult or impossible.

Basic considerations associated with adhesive selection should include:

- Can the adhesive be injected and the reinforcing bar installed within the gel time of the adhesive?
- Is the appropriate injection equipment available, including all necessary accessories, to ensure correct dispensing and mixing?
- Is the adhesive suitable for the concrete temperature and moisture conditions, hole orientation and drilling method?
- What mechanical effort or equipment is required to inject the adhesive and to install the reinforcing bar into the adhesive-filled hole?
- How will the bar be held in place during adhesive cure?

### 3.7 Injection of the adhesive

The objective of adhesive injection is to achieve a void-free installation. Aside from reducing bond area and inhibiting cure, air voids in the injected adhesive may lead to increased effort associated with bar installation and can cause uncontrolled ejection of the adhesive from the hole during bar installation as air is forced out of the adhesive matrix.

**NOTE:** Proper skin and eye protection should always be worn during injection of Hilti adhesives.

In order to inject the adhesive with minimal air voids in drilled holes, the Hilti injection system utilizes matched-tolerance piston plugs (Figure 24). The Hilti piston plug system provides positive feedback to the operator for controlling the injection process through the pressure of the adhesive on the plug and has been shown to dramatically improve injection quality and efficiency.



Figure 24 – Hilti HIT-SZ piston plugs, available in several diameters

Dispensing equipment used for injection is generally selected as a function of bar size and orientation, ambient temperature conditions and accessibility (Figure 25).



Figure 25 – The Hilti HIT-P8000D pneumatic dispenser, appropriate for large volume installations and large bar diameters.

### 3.8 Bar installation

Smaller bar diameters can be inserted in a vertical downward direction with (relatively) minimal effort. Large diameter bars in horizontal and upward-inclined orientations may require substantial effort to lift and be inserted into the adhesive-filled hole (Figure 26). **In all cases, it is advisable to test the fit of the bar in the hole prior to injecting adhesive.**

For overhead installations, particularly of larger diameter bars, provision must be made for securing the bar during adhesive cure. In addition, certification requirements for installers performing installation of bars to carry sustained tension loads, as well as additional special inspection requirements, may apply.

**NOTE:** Hilti dispensers provide efficient, void-free adhesive injection at all orientations, hole diameters and depths, and temperature conditions.



Figure 26 – Installing a long bar in an adhesive-filled hole can require the effort of several people

#### 4. How do I decide which system to use?

Options for the installation of *post-installed reinforcing bars* include cementitious grouts, polymer adhesives, and hybrid systems that combine cementitious components with polymers. The use of cementitious (e.g., baseplate) grouts is typically limited to down-hole applications and is not discussed further in this Guide. Adhesives (sometimes referred to as thixotropic adhesives) that have the correct viscosity to provide a void-free bond layer in the annular space between the bar and the concrete while still resisting unrestricted flow have been developed specifically for anchoring and bar embedment. These systems permit installation at all orientations with superior bond strength under a variety of use conditions. The proper selection of the system is dependent on a number of job-specific parameters.

Hilti design software PROFIS Rebar makes it easy to identify the optimal Hilti mortar for post-installed rebar applications. Simple to navigate with intuitive functions, filters and menus, PROFIS Rebar performs all calculations automatically and accurately with just one click taking account of the selected jobsite constraints.

##### 4.1 System selection considerations

Jobsite constraints impact both design values (bond strength) as well as installation effectiveness. Typical parameters for Hilti adhesive systems are shown in Table 2.

Table 2 – Typical parameters for HIT-HY 200-R and HIT-RE 500 V3

<b>Jobsite constraints</b>	<b>HIT-HY 200-R<sup>1</sup></b>	<b>HIT-RE 500 V3</b>
<b>Typical reinforcing bar diameter range</b>	8mm-32mm	8mm-40mm <sup>(2)</sup>
<b>Embedment range</b>	Up to 2m <sup>(3),(5)</sup>	Up to 3,2m <sup>(3),(5)</sup>
<b>Temperature of base material (installation)</b>	-10°C to 40°C	-5°C to 40°C
<b>Working time<sup>3</sup></b>	6 min. to 3 hrs.	10 min. to 2 hrs.
<b>Cure time<sup>3</sup></b>	1 hrs. to 20 hrs.	4 hrs. to 168 hrs.
<b>Holes drilled in dry and water-saturated concrete</b>	Yes	Yes
<b>Water-filled holes and underwater applications</b>	No	Yes
<b>Hammer-drilled holes</b>	Yes	Yes
<b>Core drilled holes</b>	No	Yes
<b>Hilti SafeSet™ technology using Hilti roughening tool</b>	No	Yes
<b>Hilti SafeSet™ technology using Hilti HDB and VC vacuum</b>	Yes	Yes <sup>4</sup>

<sup>1</sup> HIT-HY 200-A (accelerated cure) available. Not suitable for larger bar diameters due to short gel time.

<sup>2</sup> For larger bar sizes contact Hilti.

<sup>3</sup> Temperature dependent.

<sup>4</sup> Contact Hilti.

<sup>5</sup> Dispenser dependent.



System selection is therefore dependent on the combination of design requirements and jobsite constraints. Note also that each system is offered with a variety of options for injection in terms of cartridge size and injection equipment (manual vs. battery or pneumatic drive). Additionally, Hilti offers specialized drilling systems that substantially reduce hole cleaning requirements.

An aspect of system selection that is sometimes overlooked is the absolute volume of adhesive that must be placed in the hole. Large diameter and very deep holes may require a greater volume of adhesive than can be reasonably placed even with pneumatic delivery equipment. Furthermore, injection of large quantities of adhesive can result in excessive heat generation due to the exothermic nature of polymerization. These issues should be carefully considered for cases outside of the normal range of *post-installed reinforcing bar* applications.

**NOTE:** Hilti technical staff can provide assistance with unique or non-standard applications.

## 5. Development of design data

### 5.1 Background

Over the past two decades, extensive investigations have been conducted to evaluate *post-installed reinforcing bar* connections subjected to a variety of loading conditions. Much of this work has been conducted at leading research institutions in Europe and the U.S. as well as Hilti's own research laboratories [7],[8],[9],[10],[11],[12],[13],[14]. The tests have consistently shown that *post-installed reinforcing bars* installed with *qualified systems* exhibit performance that is at least equivalent to cast-in reinforcing bars under similar conditions, even if the basic load transfer is not identical.

Figure 27 shows schematically the load transfer mechanism of a cast-in and a post-installed rebar under tension. At the reinforcing bar-concrete interface, load is mainly transferred by mechanical interlock provided by the ribs. The mechanical interlock of the ribs leads to compression struts starting from the ribs, leading to hoop tensile stresses perpendicular to the loading direction. If the concrete cover is "small" these hoop tensile stresses may cause splitting cracks in the concrete. If the concrete cover is "large" either the concrete crushes above the ribs and shares off (by means of pullout failure) or steel failure may occur.

Post-installed rebar follow the same load transfer principle, however at the adhesive-concrete interface, load is transferred by adhesion and micro-interlock due to the roughness of the borehole surface. Small concrete cover may also lead to splitting failure however pullout failure may occur between rebar and mortar or mortar and concrete, depending on the strength of the individual layers. Steel failure will also occur for sufficiently deep embedment.

*Bond strength* is typically defined as the average bond over the embedded length of the bar while a constant bond stress distribution along the length of the rebar is assumed (uniform-bond model) for both, cast-in and post-installed rebar.



Figure 27 – Load transfer mechanism of a cast-in rebar and a post-installed rebar, schematically

Figure 28 provides a comparison of the performance of post-installed and cast-in reinforcing bars from pull-out tests conducted at the University of Stuttgart [14]. For small concrete cover ( $c = 50\text{mm}$ ) the failure loads of *post-installed* and cast-in reinforcing bars are shown to be nearly identical, verifying that, for splitting failure, qualified *post-installed reinforcing bars* behave as cast-in-place bars. At larger concrete cover, the hoop stresses required to split the concrete are not sufficiently large and therefore pull-out failure of the bars occurs. Where pull-out (bond failure) controls the behavior, the bond strength of a *post-installed reinforcing bar* may be significantly higher than that of a cast-in reinforcing bar, depending on the adhesive used.

In addition, the load-slip performance of *post-installed reinforcing bars* installed with a qualified system is similar to that of cast-in-place reinforcing bars. Thus, the design provisions for cast-in reinforcing bars in tension can be extended to *post-installed reinforcing bars* in combination with qualified products.

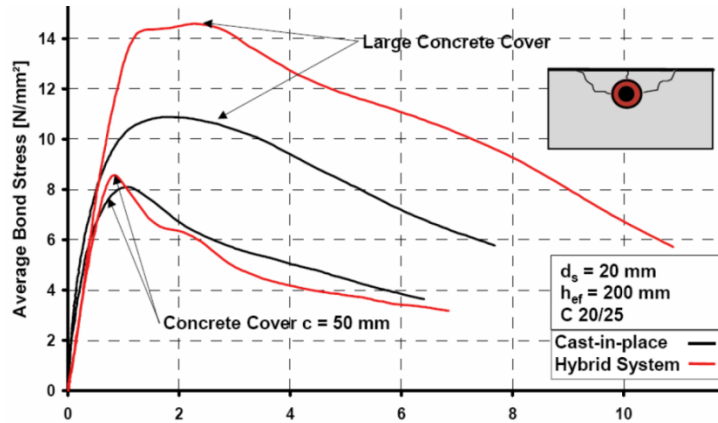


Figure 28 – Comparison of bond stresses as a function of edge distance [14]

Figure 29 shows additional test results by means of ultimate bond strength values calculated from tests for both cast-in and post-installed rebar in C20/25 as a function of the related concrete cover  $c_d/\phi$ . In addition the resulting design bond strength  $f_{bd}$  according to Eurocode2 [5] is plotted taking account of the related concrete cover by means of

$$f_{bd} = f_{bd,EC2} / \alpha_2 \text{ with } 0,7 \leq \alpha_2 \leq 1,0 \text{ (N/mm}^2\text{)}$$

$$f_{bd} = f_{bd,EC2} / (1,0 - 0,15(c_d - \phi) / \phi) \text{ (N/mm}^2\text{)}$$

with  $f_{bd,EC2} = 2,3 \text{ N/mm}^2$  according to [12]

while for  $c_d/\phi = 1,0$ , and  $\alpha_2 = 1,0$   $\rightarrow f_{bd} = 1,0 f_{bd,EC2}$

$c_d/\phi = 2,0$ , and  $\alpha_2 = 0,85$   $\rightarrow f_{bd} = 1,18 f_{bd,EC2}$

$c_d/\phi = 3,0$ , and  $\alpha_2 = 0,7$   $\rightarrow f_{bd} = 1,43 f_{bd,EC2}$

Figure 29 indicates that the scatter of the ultimate bond strength values decreases with increasing concrete cover and that the derived ultimate bond strength values for post-installed rebar are up to 2.5 times higher compared to the ultimate bond strength values of cast-in rebar.

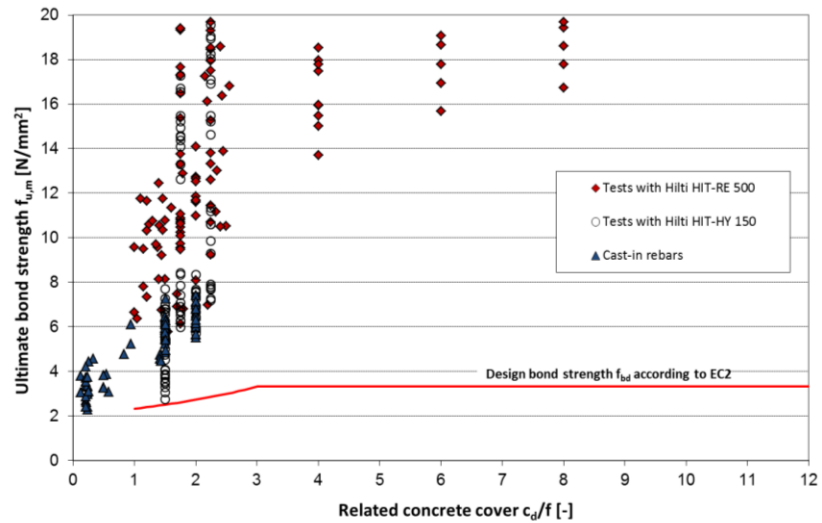


Figure 29 – Ultimate bond strength values of post-installed rebar with different Hilti mortars and cast-in rebar calculated from tests. As comparison the design bond strength of cast-in rebar according to

Eurocode2 [5] is plotted.

The importance of the right combination of bond strength and stiffness in case of splices can be seen in Figure 30.

Figure 30a shows the steel stress distribution of a cast-in rebar spliced with a cast-in rebar and Figure 30b cast-in rebar spliced with post-installed rebar for different load levels measured with strain gauges for comparison. The upper curves represent the steel stress distribution at ultimate load. Highest steel stress values are indicated at the loaded end of each bar. If a cast-in rebar is spliced with a cast-in rebar the steels stress values are nearly identical for both bars, while in addition the distribution can be assessed as axial symmetric, and the axis of reflection can be assumed in the middle of the splice.

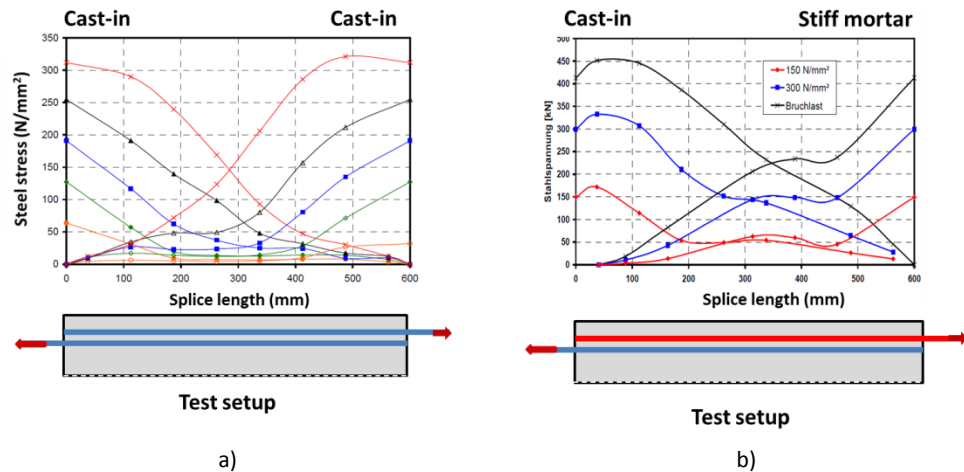


Figure 30 – Steel stress distribution of splices at different load levels (a) cast-in rebar spliced with cast-in and (b) cast-in rebar spliced with post-installed rebar with a mortar approved for anchors but not for rebar according to EOTA TR 023 [15]

In contrary to Figure 30a, Figure 30b shows the steel stress distribution with an identical test setup while in this test a cast-in rebar was spliced with a post-installed rebar using an extreme stiff epoxy which is approved for anchors but not for post-installed rebar. For that arrangement the splice shows an unequal stress distribution along the splice length. This unequal stress distribution may lead to large crack width in the area of the post-installed rebar and may therefore be assessed as not suitable for post-installed rebar applications.

Hilti HIT RE/HY adhesives passed all test successfully and can be seen as a reliable system solution for post-installed reinforcing bars ensuring a balance between stiffness and performance. Tests show an equal stress distribution along the splice length.

In 2006, the European Organization for Technical Approvals (EOTA) issued TR023 *Assessment of Post-installed Reinforcing Bar Connections* [15]. This document provides a path for verifying that *post-installed reinforcing bar* connections performed with a specific system will exhibit comparable behavior to cast-in-place reinforcing bar connections in terms of load and displacement behavior.

## 5.2 Establishment of required system performance (qualification)

The suitability of an adhesive system for *post-installed reinforcing bar* applications is dependent on many factors. Systems that may be otherwise appropriate for anchoring applications will not necessarily fulfill the requirements for safe and reliable reinforcing bar installations.

Figure 31 lists the full range of tests required to qualify adhesive anchor systems for *post-installed reinforcing bar* applications as provided in TR023 [15] which can be summarized as:

1. The ability of the adhesive to develop the required bond resistance from concrete strength class C20/25 to C50/60;
2. The sensitivity of the bond resistance to hole cleaning, freezing and thawing conditions, concrete temperature extremes in service, installation orientation, and alkalinity/sulfur exposure;
3. The ability of the system to successfully execute long bar installations (up to 60 bar diameters) without substantial voids in the adhesive around the *post-installed reinforcing bar*;
4. The corrosion resistance of the *post-installed reinforcing bar*;

While it is generally the case that modern structural-grade adhesives are capable of developing bond resistances in excess of those shown by cast-in rebar, the effects of job-site installation conditions, temperature, and other factors included in the assessment can reduce bond resistance substantially. Therefore, system performance is critical for qualification, not just adhesive bond strength as determined under optimum conditions.

	Purpose of test	Concrete (1)	Rebar (2)		Minimum number of tests	Criteria req. $\alpha$ (3)	Test procedure
			size	length (9)			
1	Bond resistance in C20/25	C20/25	12mm 25mm $d_{max}$	10 $d_s$ 10 $d_s$ 10 $d_s$	5 5 (4) 5	see 3.3.2	2.2
2	Bond resistance in C50/60	C50/60	$d_{max}$	7 $d_s$	5	see 3.3.2	2.3
3	Installation safety dry concrete	C20/25	$d_{max}$ (5)	10 $d_s$	5	$\geq 0.8$ (6)	2.4
4	Installation safety wet concrete	C20/25	$d_{max}$ (5)	10 $d_s$	5	$\geq 0.75$ (6)	2.5
5	Functioning under sustained loads	C20/25	12mm	10 $d_s$	5	$\geq 0.9$	2.6
6	Functioning under freeze/thaw conditions	C50/60 (7)	12mm	7 $d_s$	5	$\geq 0.9$	2.7
7	Functioning with installation direction	C20/25	$d_{max}$	10 $d_s$	5	$\geq 0.9$	2.8
8	Installation at maximum embedment depth	C20/25	$d_{max}$	max $l_v$	5	see 3.2.3	2.9
9	Correct injection		$d_{max}$	max $l_v$	5	see 3.2.4	2.10
10	Checking durability of mortar (8)	C20/25	12mm	10 $d_s$	3 x 10	see 3.3.3	2.11
11	Corrosion resistance of rebar (10)	C20/25	12mm	70mm	3	see 3.3.4.3	3.3.4.2

**Notes to Table 2.1**

- (1) All tests performed in non-cracked concrete
- (2) size: diameter of the rebar;  $d_{max}$ : max. diameter of the rebar specified by the manufacturer; length: embedment length of the bar in the concrete
- (3) see 3.3.1.
- (4) Tests are necessary only, if tests according to line 3 and 4 are done with  $d_s = 25\text{mm} < d_{max}$
- (5) Tests shall be done with  $d_s = 25\text{mm}$ , if comparison tests according to line 1 are carried out with  $d_s = 25\text{mm}$  instead of  $d_{max}$
- (6) The required  $\alpha$  shall not be changed, because the concept of installation safety factor  $\gamma_2$  is not supported by EC2.
- (7) The test result shall be normalised to C20/25 by using a factor reflecting the influence of concrete strength or bond resistance as established by tests according to line 1 and 2
- (8) Tests are not required for mortars based on cement only
- (9) 10  $d_s$  and 7  $d_s$  shall be reduced in case of steel failure mode. It is the objective of these tests to determine bond resistance.
- (10) No prove of the corrosion resistance of the rebar is needed if post-installed rebars are used in building components in dry surroundings according to exposure class X0 and XC1 of EC2. Also no prove is needed when only corrosion resistant rebars are specified for all applications; see 3.3.4.1.

Figure 31 – Test Program for Evaluating Reinforcing Bars for Use in *Post-installed Reinforcing Bar Connections* (EOTA TR 023) [15].

## 6.0 Design concepts

### 6.1 Establishing the required bar anchorage length

Systems qualified under EOTA TR 023 [15] are required to demonstrate bond resistance and stiffness characteristics that are compatible with cast-in reinforcement. Therefore, *post-installed reinforcing bars* installed with qualified systems can be designed and detailed using the same provisions that are applicable to the development of straight cast-in-place bars.

### 6.2 Overview of Eurocode2 [5] anchorage length provisions for straight reinforcing bars

The Eurocode2 [5] concept of anchorage length is based on the attainable average bond stress over the length of embedment of the reinforcement. Anchorage length can be defined as the shortest length in which the bar stress increases from zero to the design steel stress  $\sigma_{sd}$  acting at the point where the anchorage or splice starts. . The bar stress is the force per unit area of the bar cross-section.

Structural reinforced concrete design is in practice based on the assumption that the reinforcing bar will develop its nominal yield strength before premature failure occurs due to inadequate bond. Basic required anchorage length, design anchorage length and splice length is intended to ensure that the nominal yield strength (the minimum bar stress at which permanent (inelastic) deformation occurs) of the bar can be developed under structure loading. Although Eurocode2 does not require a ductile design of the system connection, it is strongly recommended apply an anchorage length able to ensure the development of the nominal yield strength.

In the following a summary of Eurocode2 [5] provisions for cast-in is given as far as it concerns the design of post-installed reinforcing bars.

The anchorage length is closely associated to the design bond strength,  $f_{bd}$ , which is given as follows:

$$f_{bd} = 2,25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \quad (\text{N/mm}^2)$$

Where:

2,25 = basic value of the design bond strength (N/mm<sup>2</sup>)

$\eta_1$  = coefficient related to the quality of the bond condition and the position of the bar during concrete pouring.  $\eta_1=1.0$  stands for good bond conditions and  $\eta_1=0.7$  is taken for all other cases. Note for post-installed rebar  $\eta_1=1.0$  should be taken (-)

$\eta_2$  = coefficient related to the rebar diameter:  $\eta_2 = (132-\phi)/100 \leq 1.0$  where  $\phi$  is the nominal rebar diameter [mm] while  $\eta_2 = 1,0$  for  $\phi \leq 32\text{mm}$  (-)

$f_{ctd}$  = the design tensile strength of the concrete

The basic required anchorage length  $l_{b,rqd}$  is given as follows:

$$l_{b,rqd} = (\phi/4) / (\sigma_{sd}/f_{bd}) \quad (\text{mm})$$

Where:

$\phi$  = the reinforcing bar diameter (mm)

$\sigma_{sd}$  = design steel stress at the beginning of the anchorage (N/mm<sup>2</sup>)

$f_{bd}$  = design value of the ultimate bond stress (N/mm<sup>2</sup>)

The design anchorage length  $l_{bd}$  is calculated from the basic required anchorage length  $l_{b,rqd}$  taking into account the influence of five parameters ( $\alpha_1$  to  $\alpha_5$ ) and it should not be less than a minimum anchorage length  $l_{b,min}$ . The design anchorage length  $l_{bd}$  is given as follows:

Rebar under tension:	$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_{b,rqd} \geq l_{b,min}$	(mm)
Rebar under compression:	$l_{bd} = \alpha_4 l_{b,rqd} \geq l_{b,min}$	(mm)

Where:

$\alpha_1$  considers the form of the bar (for straight bar ends  $\alpha_1=1.0$ , for other shapes in certain conditions  $\alpha_1=0.7$ ) (-)

$\alpha_2$  takes into account the concrete cover:  $0,7 \leq \alpha_2 = 1 - 0,15(c_d - k\phi) / \phi \leq 1,0$  where  $c_d$  is the smallest of (a) the concrete cover and (b) half of the clear spacing of bars and  $k=1$  for bars without hook and  $k=3$  for bars with hook (-).  
 $\alpha_2$  takes account of passive confinement provided by the surrounded concrete.

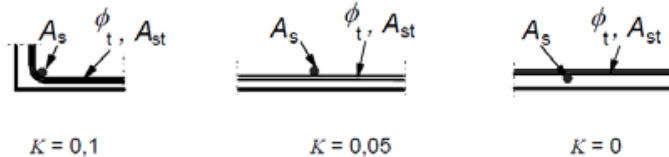
For simplification  $\alpha_2 = 1.0$  maybe assumed.

$\alpha_3$  takes account for the effect of transverse reinforcement where  $0,7 \leq \alpha_3 = 1 - K\lambda \leq 1,0$  with  $\lambda = (\Sigma A_{st} - \Sigma A_{st,min}) / A_s$ . (-)

$\Sigma A_{st}$  = cross-sectional area of the minimum transverse reinforcement along the design anchorage length  $l_{bd}$  (mm<sup>2</sup>)

$\Sigma A_{st,min} = 0.25A_s$  for beams and  $\Sigma A_{st,min} = 0$  for slabs with  $A_s$  = area of a single anchored bar with maximum bar diameter (mm<sup>2</sup>)

K: coefficient related to the position of the post-installed rebar



$\alpha_3$  takes account of passive confinement provided by the lateral reinforcement. Concrete structural members that are confined react to the Poisson type lateral expansion and generate side pressures. With the increase in lateral steel, the ductility of the concrete increases (its ability of sustaining large permanent changes in shape without breaking). For simplification  $\alpha_3 = 1.0$  maybe assumed.

$\alpha_4$  = 0.7 if transverse reinforcement is welded to the reinforcement to be anchored, otherwise  $\alpha_4 = 1.0$  (-)

$\alpha_5$  = takes account of transverse pressure while  $\alpha_5 = 1 - 0.04p \geq 0.7$  where  $p$  is the transverse pressure along the anchorage length (active confinement). The confining pressure which is applied to pre-stress the concrete element laterally prior to loading exerts an initial volumetric strain due to compaction. In order to overcome this, additional axial strain and stress

are needed, and the load capacity of the concrete is increased compared to the passively confined concrete.

The cumulating of the influences is limited by  $\alpha_2 \cdot \alpha_3 \cdot \alpha_5 \geq 0.7$

The minimum anchorage length  $l_{b,min}$  is given as follows:

$$l_{b,min} = \max(0.3l_{b,rqd}; 10\phi; 100\text{mm}) \text{ for bars under tension (mm)}$$

$$l_{b,min} = \max(0.6l_{b,rqd}; 10\phi; 100\text{mm}) \text{ for bars under compression (mm)}$$

The design splice length  $l_o$  is also calculated from the basic required anchorage length  $l_{b,rqd}$  with:

$$l_o = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{o,min} \quad (\text{mm})$$

For bars in compression (usually considered only in highly loaded columns), all  $\alpha$ -factors except  $\alpha_6$  are the same as for anchorage, see above. For bars in tension (and  $\alpha_4$  also in compression) the factors are defined as follows:

$\alpha_6 = 1.5$  if all bars are spliced in the same area (i.e. the splices are not staggered) which is usually the case with post-installed splices (-)

Note: If the clear distance between lapped bars  $e$  exceeds four times the bar diameter  $\phi$  or 50mm, then the overlap length shall be increased by a length equal to  $e-4\phi$  or  $e-50\text{mm}$ .

The minimum splice length  $l_{o,min}$  can be calculated as follows:

$$l_{o,min} = \max(0.3 \cdot \alpha_6 \cdot l_{b,rqd}; 15\phi; 200\text{mm})$$

### Special provisions for post-installed rebars

To prevent damage of the concrete during drilling the following requirements have to be met:

- Minimum concrete cover:
  - $c_{min} = 30 + 0.06 l_v \geq 2d_s$  (mm) for hammer drilled holes
  - $c_{min} = 50 + 0.08 l_v \geq 2d_s$  (mm) for compressed air drilled holes

The factors 0.06 and 0.08 take into account the possible deviations during drilling. However, if special drilling aid devices are used, this value could be smaller.

- Minimum clear spacing between two post-installed bars should be  $4\phi \geq 40\text{mm}$ .

Note that the Eurocode2 [5] limits the extent to which bond stresses in the concrete may be utilized via a 0.7 cap on the value of the quotient  $0.7 \leq \alpha_2 = 1 - 0.15(c_d - k\phi)/\phi \leq 1.0$  in the basic anchorage length equation.

Figure 32 provides a schematic representation of this limit, whereby for bars with a high value of concrete cover ( $c_d/\phi \geq 3$ ), it is assumed that splitting no longer controls the behavior at ultimate load.

Hilti anchoring adhesives can generate bond stresses that far exceed this limit represented by the "actual design bond capacity" line plotted below. The background of this function will be explained in Section 6.3.



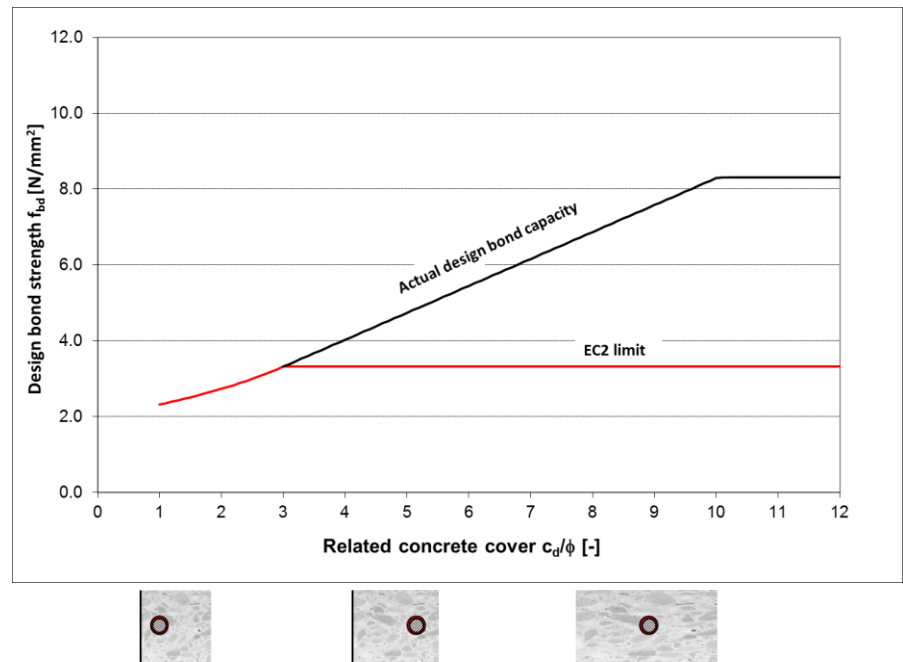


Figure 32 – Effective limit on bond stress for post-installed rebar using Hilti mortar systems represented by the “actual design bond capacity” and design bond strength values as provided by Eurocode2 [5]

### 6.3 Alternative approaches to establishing bar embedment: Hilti Rebar design method

The anchorage length provisions of the Eurocode2 [5] are predicated on the assumption that cast-in bars may be closely spaced and may be placed far from the concrete surface.

No reduction in anchorage length is afforded for bars with a concrete cover equal to or greater than  $3\phi$ , since it is assumed that pullout should control for these bars (as opposed to splitting). Increases in concrete cover should permit further reductions in anchorage length or further increase of the design bond strength for post-installed bars with Hilti mortar; however, bars carrying bond stresses could lead to concrete cone failure, if the rebar is not loaded against a local or global compression strut.

However, when *post-installed reinforcing bars* are applied in a moment resisting connection, not lapped with existing reinforcing, and are installed sufficiently far from edges with a high value of concrete cover, it may be appropriate to employ other design concepts which will be discussed in the following.

#### 6.3.1 Design of post-installed reinforcing bars using Hilti Rebar design method

Eurocode2 [5]/ETA [15] design method for post-installed reinforcing bars has two main drawbacks

- The connection of simply supported slabs to walls is only possible if the wall is thick enough to accommodate the anchorage length. As reductions of the anchorage length with hooks or welded transverse reinforcement cannot be made with post-installed reinforcement, it often occurs that the wall is too small. However, if the confinement of the concrete is large enough, it is actually possible to use the full bond strength of the adhesive rather than the bond strength given by Eurocode2 [5]. The so-called Hilti Rebar design method or, to be more precise, the “splitting design” allows to design for the full strength of the adhesive [7, 8], see Section 6.5.2

- According to traditional reinforced concrete principles, moment resisting frame node connections require bent connection bars. In this logic, they can therefore not be made with straight post-installed rebar connections. The frame node model is a proposed strut and tie model to design moment resisting frame node connections with straight connection bars [7,9], see Section 6.5.4

However, why in most of the cases where post-installed reinforcing bars are lapped with existing reinforcement, the Hilti HIT Rebar design method does not produce any benefit?

As discussed in Section 1.3 in case of splices the load is transferred from one bar (cast-in) to the adjacent bar (post-installed) via compression struts. Therefore the ultimate load carrying behaviour of the splice is given by weaker element within this interaction represented by the cast-in rebar. Even if the spliced post-installed rebar would have infinite bond strength values the splice would fail by pull-out of the cast-in rebar. Therefore an utilization of the bond strength values of the used mortar type as followed by the Hilti Rebar design method is limited by the spliced cast-in rebar embedded using the bond strength values given in Eurocode2 [5].

However there is one case where the Hilti Rebar design method may be beneficial for the overall splice length. It is common that reinforcement at the end of slabs/beams is hooked, and therefore for cast-in  $\alpha_1 = 0.7$  and for post-installed rebar  $\alpha_1 = 1$ . The result is that the anchorage length of the post-installed rebar governs the overall anchorage length of the system connection. By using the Hilti Rebar design method the anchorage length of the post-installed rebar can be reduced up to the anchorage length of cast-in reinforcement.

### 6.3.2 Use of confinement to increase bond efficiency (Hilti splitting design)

The factor  $\alpha_2$  of Eurocode2 [5] gives an explicit consideration for splitting and spalling as a function of related concrete cover  $c_d/\phi$  (passive confinement).

European Technical Approvals recommend the same procedure for post-installed rebar connections. The  $\alpha_2$  –within its limitation – results in a maximum reduction of the anchorage length of approximately 43%, if the concrete cover is larger than  $3\phi$ . The reduction of anchorage length can also be interpreted by means of an increase of the bond strength of max. 43% ( $\alpha_2 = 0.7$ ) while  $0.7 \leq \alpha_2 \leq 1.0$  as described in Section 5.1. This can be interpreted as follows:

as long as  $\alpha_2$  exceeds 0.7, spalling of the concrete cover or splitting between bars will be the controlling mode of failure. If  $\alpha_2$  is less than 0.7, corresponding to cover dimensions of  $c_d/\phi > 3$ , the cover is large enough so that splitting cannot occur anymore and pullout will control.

$$f_{bd} = f_{bd,EC2}/\alpha_2 \text{ with } 0,7 \leq \alpha_2 \leq 1,0 \text{ (N/mm}^2\text{)}$$

$$f_{bd} = f_{bd,EC2}/(1,0 - 0,15(c_d - \phi)/\phi) \text{ (N/mm}^2\text{)}$$

$$\text{while for } c_d/\phi = 1,0, \alpha_2 = 1,0$$

$$c_d/\phi = 2,0, \alpha_2 = 0,85$$

$$c_d/\phi = 3,0, \alpha_2 = 0,7$$

This philosophy is adapted and extended to post-installed reinforcement via the HIT-Rebar design concept. In the following the footnote “<sub>Hilti</sub>” is added to the denominations of the equations to indicate that the design is done according to the “Hilti splitting method”. The basic design anchorage length can be given as follow:

$$f_{bd,Hilti} = f_{bd,EC2}/\alpha_2 \text{ (N/mm}^2\text{)}$$

For a related concrete cover of  $1 \leq c_d/\phi \leq 3$  "splitting" controls and the obtained design bond strength from the "Hilti splitting method" for post-installed rebar becomes the same as for cast-in rebar obtained from Eurocode2 [12] by means of:

$$f_{bd,Hilti,1} = f_{bd,EC2}/\alpha_2 \text{ (N/mm}^2\text{)}$$

For cover dimensions exceeding the range of Eurocode2 [5], ( $c_d/\phi > 3$  (post-installed rebar with Hilti mortar, only)), an adapted factor  $\alpha_2'$  is used to create a linear extension of the bond strength function related to the basic bond strength value  $f_{bd}$  while the linear approach was derived from a large number of tests to describe the increase of bond strength as a function of the related concrete cover. The increase in the design bond strength is limited by the maximum pull-out bond stress  $f_{bd,p}$ , which is a value taken from the relevant anchor approval. Thus, the limitation for bond failure in the code has been replaced by the specific design bond stress of the bonding agent for the specific application conditions and the splitting function given by:

$$f_{bd,Hilti,2} = f_{bd,EC2}/\alpha_2' \leq f_{bd,p} \text{ (N/mm}^2\text{)}$$

while

$$\alpha_2' = \frac{1}{\frac{1}{0.7} + \delta \cdot \frac{c_d - 3\phi}{\phi}} \geq 0.25$$

Where:

$\delta$  = describes the increase in design bond strength with increasing related concrete over taking account of the different mortar types.

$f_{bd,p}$  = maximum bond strength in case of pullout taken from the relevant anchor approval

0.25 = factor to avoid unreasonably low values of  $\alpha_2'$

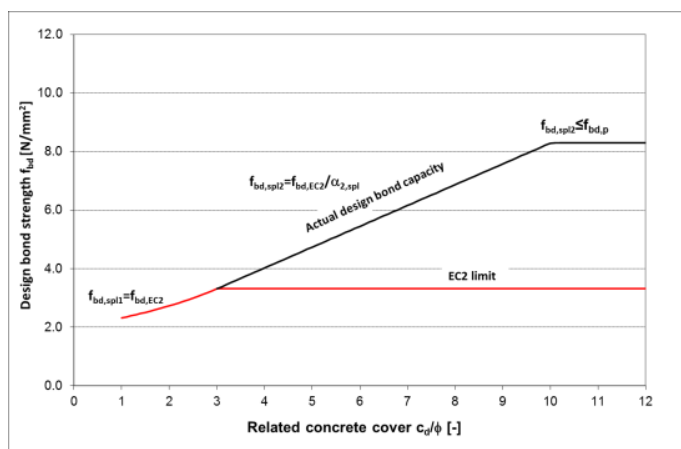


Figure 33 – Effective limit on bond stress for post-installed rebar using Hilti mortar systems represented by the “actual design bond capacity” and design bond strength values as provided by Eurocode2 [5]

Figure 33 – Effective limit on bond stress for post-installed rebar using Hilti mortar systems represented by the “actual design bond capacity” and design bond strength values as provided by

Eurocode2 [5]Figure 33 shows a typical design bond strength  $f_{bd}$  curve as a function of the related concrete cover, shown for a concrete class C20/25 and for a rebar with a diameter not greater than 32mm.

In this figure the equivalent design bond stresses according to Eurocode2 [5] (red line) and the ones resulting from the above described definition of  $\alpha_2$  and  $\alpha_2'$ , are plotted as a function of the related concrete cover (actual design bond capacity). The design bond strength is defined by an inclined line and it increases with larger values of  $c_d$ .

In the following design examples are given following the Hilti Rebar design concept.

## 6.4 Design examples

The following design examples are intended for illustration purposes only. System connections based on both Eurocode2 and HIT Rebar Design Method are shown.

### 6.4.1 Design of end support of a simply supported slab according to Eurocode2

Requirement:

Provide post-installed reinforcing bar for a new simply supported slab/beam on a concrete structure as shown below.

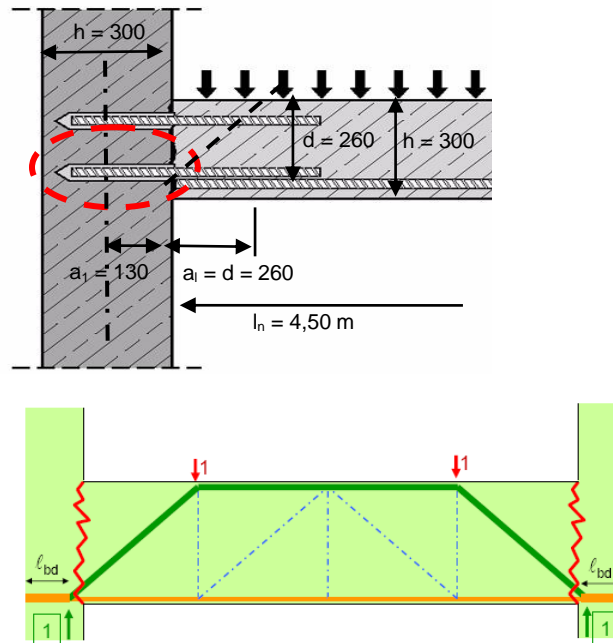


Figure 34– Cross section through structure

Existing structure:	wall: $h = 300 \text{ mm}$
New construction:	slab: $l_n = 4,50\text{m}$ , $Q_k = 20 \text{ kN/m}^2$ , $h = 300 \text{ mm}$ , $d = 260 \text{ mm}$
Concrete strength class:	C20/25, dry concrete
Reinforcement:	$f_{yk} = 500 \text{ N/mm}^2$ , $\gamma_s = 1.15$

Loads:

$$G_k = 25 \text{ kN/m}^3 \cdot h = 7.5 \text{ kN/m}^2;$$

$$S_d = (1.50 \cdot Q_k + 1.35 \cdot G_k) = 40.1 \text{ kN/m}^2$$

Structural analysis (design forces):

$$M_{Ed} = S_d \cdot l_n^2 / 8 = 102 \text{ kNm/m}$$

$$V_{Ed} = S_d \cdot l_n / 2 = 90.3 \text{ kN/m}$$

Set design conditions:

Drilling method:	Rotary-hammer drilling
Orientation of connection:	Horizontal/wall position
Installation and in-service temperature:	-5°C - +40°C
Type of base material:	Normal weight concrete
Condition of base material:	Dry
Choice of adhesive based on condition:	HIT-RE 500 V3

Bottom reinforcement required at mid span:

$$A_{s,rqd,m} = (M_{Ed} \cdot \gamma_s) / (0.9 \cdot d \cdot f_{yk}) = (102 \cdot 10^6 \cdot 1.15) / (0.9 \cdot 260 \cdot 500) = 1002 \text{ mm}^2/\text{m}$$

Reinforcement provided at mid span:

$$\phi 16, s = 200 \text{ mm} \rightarrow A_{s,prov,m} = 1005 \text{ mm}^2/\text{m}$$

Bottom reinforcement at support:

$$\text{Tension force to be anchored: } F_E = |V_{Ed}| \cdot a_l / (0.9d) = 90.3 \cdot 260 / (0.9 \cdot 260) = 100.3 \text{ kN/m}$$

{Clause 9.2.1.4(2), EC2: EN 1992-1-1:2004}

$$\text{Steel area required: } A_{s,rqd} = F_E \cdot \gamma_s / f_{yk} = 100 \cdot 10^3 \cdot 1.15 / 500 = 231 \text{ mm}^2/\text{m}$$

Minimum reinforcement to be anchored at support:

$$A_{s,min} = k_c \cdot k \cdot f_{ct,eff} \cdot A_s / \sigma_s = 0.4 \cdot 1 \cdot 2.2 \cdot 150 \cdot 1000 / 500 = 264 \text{ mm}^2/\text{m}$$

{Clause 7.3.2(2), EC2: EN 1992-1-1:2011}

$$A_{s,min} = 0.50 \cdot 998 = 499 \text{ mm}^2/\text{m}$$

{Clause 9.3.1.2(1), EC2: EN 1992-1-1:2011}

$$A_{s,min} = 0.25 \cdot 1005 = 251 \text{ mm}^2/\text{m}$$

{Clause 9.2.1.4(1), EC2: EN 1992-1-1:2011}

Decisive is 499 mm<sup>2</sup>/m

$$\Rightarrow \text{reinforcement provided: } \phi 12, s = 200 \text{ mm} \Rightarrow A_{s,prov} = 565 \text{ mm}^2/\text{m};$$

(Required) basic anchorage length

{EC2: EN 1992-1-1:2004, section 8.4.3}

$$l_{b,rqd} = (\phi / 4) \times (\sigma_{sd} / f_{bd}) \quad [\text{mm}]$$

with:  $\phi$  = diameter of the rebar = 12 mm  
 $\sigma_{sd}$  = calculated design stress of the rebar  
 $= (A_{s,rqd} / A_{s,prov}) \cdot (f_{yk} / \gamma_s)$   
 $= (231 / 565) \cdot (500 / 1.15)$   
 $= 177 \text{ N/mm}^2$

$f_{bd}$  = design value of bond strength according to ETA-16/0142  
 $= 2.3 \text{ N/mm}^2$  for C20/25

$$l_{b,rqd} = (12 / 4) \times (177 / 2.3)$$

$$= 231 \text{ mm}$$

### Design anchorage length

{EC2: EN 1992-1-1:2011, section 8.4.4}:

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_{b,rqd} \geq l_{b,min}$$

with:

$\alpha_1$	= 1,0 for straight bars
$\alpha_2$	= $1 - 0,15(c_d - \phi)/\phi$ = 0,7 ( $0,7 \leq \alpha_2 \leq 1,0$ ) with $c_d = (200-12)/2$ = 94mm ( $d_s = 12$ mm)
$\alpha_3$	= 1,0 (no transverse reinforcement)
$\alpha_4$	= 1,0 (no welded transverse reinforcement)
$\alpha_5$	= 1,0 (influence of transverse pressure is neglected)

while  $c_d = \min(a/2, c_1, c)$

$$l_{bd} = 0,7 \cdot 231 = 162 \text{ mm}$$

### Minimum anchorage length:

{Clause 8.4.4(1), EC2: EN 1992-1-1:2011}:

$$l_{b,min} = \max \{0,3l_{b,rqd}; 10\phi; 100\text{mm}\}$$

$$= 120 \text{ mm} \leq l_{bd}$$

$l_{bd}$  controls → anchorage length  $l_{bd}$  is equal to the drill hole length  $l_{inst} = 162 \text{ mm}$ .

Note: Value of drilled hole length may differ from anchorage length, see “splice” example

### Alternative drilling method and impact:

*Drilling method: wet diamond core drilling*

*Installation by wet diamond core drilling: Hilti HIT-RE 500 is suitable adhesive (see Tech data), however if wet diamond core drilling is used for installation, the minimum anchorage length according to Eurocode2 has to be multiplied by 1.5 (ETA 09-0295, sect. 4.3.3).*

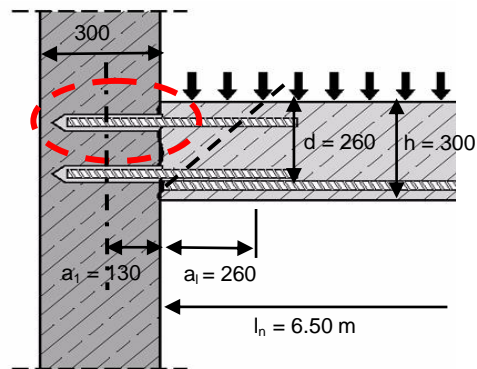
$$l_{b,min} = \max \{0,3l_{b,rqd}; 10\phi; 100\text{mm}\} \cdot 1.5$$

$$= 180 \text{ mm} > l_{bd}$$

$l_{b,min}$  controls → anchorage length  $l_{bd}$  is  $l_{inst} = 180 \text{ mm}$  in diamond cored drilled holes

Note that when roughening tool is used, this amplification factor is not applied.

Top reinforcement at support:



Minimum reinforcement:

{Clause 9.3.1.2(2), EC2: EN 1992-1-1:2004}

25% of bottom steel required at mid-span:

$$A_{s,req} = 0,25 \cdot 998$$

$$= 247 \text{ mm}^2/\text{m}$$

Requirement for crack limitation :

{Clause 7.3.2(2), EC2: EN 1992-1-1:2004}

$$A_{s,min} = 0,4 \cdot 1 \cdot 2,2 \cdot 150 \cdot 1000 / 435$$

$$= \underline{303 \text{ mm}^2/\text{m}} \text{ (decisive)}$$

⇒ reinforcement provided:  $\phi=10$ ,  $s = 200$  mm;  $A_{s,prov} = 393 \text{ mm}^2/\text{m}$

Design steel stress in bar:

$$\sigma_{sd} = f_{yd} \cdot A_{s,min} / A_{s,prov}$$

$$= 335 \text{ N/mm}^2$$

$$l_{b,rqd} = (\phi / 4) \times (\sigma_{sd} / f_{bd})$$

$$= (10 / 4) \times (335 / 2.3)$$

$$= 364 \text{ mm}$$

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_{b,rqd}$$

$$= 1 \cdot 0,7 \cdot 1 \cdot 1 \cdot 1 \cdot 364 = 255 \text{ mm}$$

$$l_{b,min} = \max \{0,3l_{b,rqd}; 10\phi; 100\text{mm}\}$$

$$= 110 \text{ mm}$$

Therefore, drill hole length  $l_v = l_{bd} = \mathbf{255 \text{ mm}}$

Alternative drilling method and impact:

*Drilling method: wet diamond core drilling*

*Installation by wet diamond core drilling: Hilti HIT RE-500 V3 is suitable adhesive (see Tech data), however if wet diamond core drilling is used for installation, the minimum anchorage*



length according to Eurocode 2 have to be multiplied by 1.5 if the roughening tool is not used.

$$l_{b,min} = \max \{0,3l_{b,rqd}; 10\phi; 100mm\} \cdot 1.5$$

$$= 180 \text{ mm} \leq l_{bd}$$

$l_{bd}$  controls → anchorage length  $l_{bd}$  is  $l_{inst} = 255 \text{ mm}$  in diamond cored drilled holes also

Drilling method	Borehole conditions	$l_{bd, lower}$	$l_{bd, upper}$	$c_{min}^3)$ , see Section 3.3
Diamond coring	wet	180mm	255mm	34mm/35mm
Hammer drilling	wet/dry	162mm	255mm	40mm/45mm

According to Eurocode2 the shear transfer at the interface between existing concrete and new concrete must be ensured by friction.

Specification: Provide post-installed rebar at size, spacing and anchorage length as indicated on construction documents.

Anchoring system: Hilti HIT RE-500 V3 epoxy, install as per Manufacturer`s Printed Installation Instructions (MPII) permissible concrete temperature range for installation: -5°C to 40°C, concrete shall be dry or wet during installation.

Drill hole using a rotary hammer drill with carbide bit or diamond coring. Locate existing reinforcing prior to drilling – do not damage reinforcing without prior authorization of the Engineer of record (EOR)

Shear transfer at the interface:

$$V_{Ed} = 90.3 \text{ kN/m}$$

$$v_{edi} = 90.3 / b_i = 90.3 / 0.3 = 301 \text{ kN/m}^2$$

$$c = 0.45$$

$$f_{ctm} = 0.3 \cdot f_{ck}^{2/3} = 0.3 \cdot 20^{2/3} = 2.21 \text{ N/mm}^2 \text{ (EN 1992-1-1:2004 (Table 3.1))}$$

$$f_{ctk} = 0.7 \cdot f_{ck} = 0.7 \cdot 2.21 \text{ N/mm}^2 = 1.55 \text{ N/mm}^2 \text{ (EN 1992-1-1:2004 (Table 3.1))}$$

$$f_{ctd} = \alpha_{ct} \cdot f_{ctk,0,05} / \gamma_C = 1 \cdot 1.55 / 1.5 = 1.5 \text{ N/mm}^2$$

$$V_{Rdi,c} = c \cdot f_{ctd} = 0.45 \cdot 1 \cdot 10^3 = 450 \text{ kN/m}^2$$

$$v = 0.6 \cdot (1 - f_{ck} / 250) = 0.6 \cdot (1 - 20 / 250) = 0.55 \text{ (EN 1992-1-1:2004 (6.2.2))}$$

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_C = 1 \cdot 20 / 1.5 = 13.33 = \text{N/mm}^2 \text{ (EN 1992-1-1:2004 (3.1.6))}$$

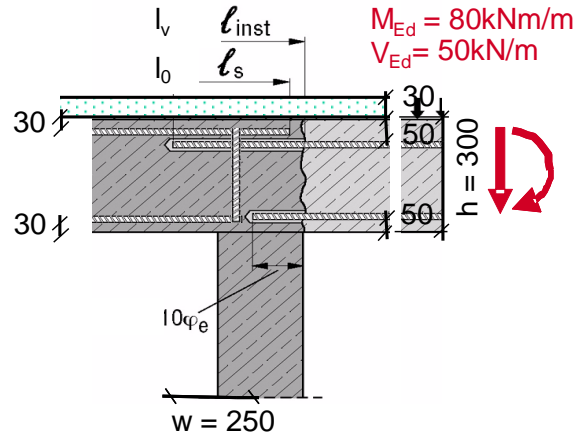
$$V_{Rdi,max} = 0.5 \cdot v \cdot f_{cd} = 0.5 \cdot 0.55 \cdot 13.33 \cdot 1000 = 3670 \text{ kN/m}^2$$

$$V_{Rdi} = \min(V_{Rdi,max} ; V_{Rdi,c}) = \min(400 ; 3670) = 400 \text{ kN/m}^2$$

$$V_{Rdi} \leq v_{edi}$$

### 6.4.2 Design of splice on support according to Eurocode 2

Requirement: Provide post-installed reinforcing bar for a new balcony extension on an existing concrete structure as shown below.



Actions:  $M_{Ed}=80 \text{ kNm/m}$ ; shear:  $V_{Ed} = 50 \text{ kN/m}$   
 Concrete strength class: C25/30  
 Reinforcement:  $f_{yk} = 500 \text{ N/mm}^2$   
 Slab: Cover cast-in bars  $c_c = 30 \text{ mm}$  (top, bottom);  
 Cover new bars:  $c_n = 50 \text{ mm}$   $h = 300 \text{ mm}$ ;  
 Top reinforcement (new and existing):  
 $\phi 16, s = 200 \text{ mm} \rightarrow A_{s,prov} = 1005 \text{ mm}^2/\text{m}$ ; cover to face  $c_1 = 30 \text{ mm}$   
 Bottom reinforcement:  
 $\phi 10, s=200 \text{ mm} \rightarrow A_{s,prov}=393 \text{ mm}^2/\text{m}$

#### Set design conditions:

Drilling method:	Rotary-hammer drilling
Orientation of connection:	Horizontal/wall position
Installation and in-service temperature:	-5°C
Type of base material:	Normal weight concrete
Condition of base material:	Dry
Choice of adhesive based on condition:	HIT-RE 500 V3

#### Cast-in reinforcement(<sub>ci</sub>) top

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

$$\eta_1 = (d - \phi/2 > 250 \text{ mm}) \quad 0.7 \text{ poor bond condition}$$

$$z_{ci} = 239 \text{ mm} \quad (\text{from static calculation})$$

$$A_{s,req} = (M_{Ed}/z) \cdot (\gamma_s/f_{yk}) = (80/0.239) \cdot (1.15/0.5)$$

$$= 770 \text{ mm}^2/\text{m}$$

$$\sigma_{sd} = (A_{s,rqd} / A_{s,prov}) \cdot (f_{yk} / \gamma_s) = (770 / 1005) \cdot (500 / 1.15) \\ = 333 \text{ N/mm}^2$$

$$f_{bd} = 2.25 \cdot \eta_1 \cdot 0.7 \cdot 0.3 \cdot f_{ck}^{2/3} / \gamma_c \\ = 2.25 \cdot 0.7 \cdot 0.7 \cdot 0.3 \cdot 25^{2/3} / 1.5 \\ = 1.89 \text{ N/mm}^2 \quad (\text{ETA 16/0142})$$

$$l_{b,rqd,ci} = (\phi / 4) \cdot (\sigma_{sd} / f_{bd}) \\ = (16 / 4) \cdot (333 / 1.89) \\ = 705 \text{ mm}$$

$$\alpha_1 = 0.7 \text{ (hooked end of cast-in bars)}$$

$$\alpha_2 = (1 - 0.15(c_d - \phi) / \phi) \geq 0.7 \\ = 1 - 0.15(30 - 16) / 16 \\ = 0.87$$

$$\alpha_3 = 1.0 \text{ (no transverse reinforcement)}$$

$$\alpha_5 = 1.0 \text{ (no transverse pressure)}$$

$$\alpha_6 = 1.5 \text{ (splice factor)}$$

$$l_{0,min} = \max\{0.3 \cdot 1.5 \cdot 705; 15 \cdot 16; 200\} \\ = 317 \text{ mm}$$

$$l_{0,ci} = 0.70 \cdot 0.87 \cdot 1.5 \cdot 705 \\ = 643 \text{ mm}$$

#### Post-installed reinforcement ( $l_{pi}$ ) top

The required design lap length  $l_0$  shall be determined in accordance with EC2: EN 1992-1-1:2004, section 8.7.3:

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

$$d = h - c_n - \phi / 2 = 300 - 50 - 16 / 2 = 242 \text{ mm}$$

$$\eta_1 = (d - \phi / 2 < 250 \text{ mm}) 1.0 \text{ (good bond condition)}$$

$$z = 228 \text{ mm (from static calculation)}$$

$$A_{s,req} = (M_{Ed} / z) \cdot (\gamma_s / f_{yk}) \\ = (80 / 0.228) \cdot (1.15 / 0.5) \\ = 807 \text{ mm}^2 / \text{m}$$

$$\sigma_{sd} = (A_{s,rqd} / A_{s,prov}) \cdot (f_{yk} / \gamma_s) = (807 / 1005) \cdot (500 / 1.15) \\ = 349 \text{ N/mm}^2$$

$$f_{bd} = \text{design value of bond strength according to ETA-16/0142} \\ = 2.7 \text{ N/mm}^2$$

$$l_{b,rqd,pi} = (\phi / 4) \cdot (\sigma_{sd} / f_{bd}) = (16 / 4) \cdot (349 / 2.7) = 516 \text{ mm}$$

$$\alpha_1 = 1.0 \text{ (straight bars)}$$

$$\alpha_2 = (1 - 0.15(c_d - \phi) / \phi) \geq 0.7 \\ = 1 - 0.15(50 - 16) / 16 \\ = 0.7$$

$$\alpha_3 = 1.0 \text{ (no transverse reinforcement)}$$

$$\alpha_5 = 1.0 \text{ (no transverse pressure)}$$

$$\alpha_6 = 1.5 \text{ (splice factor)}$$

$$l_{0,\min} = \max\{0.3 \cdot 1.5 \cdot 515; 15 \cdot 16; 200\} = 240 \text{ mm}$$

$$l_{0,\text{pi}} = 0.7 \cdot 1.5 \cdot 530 = 542 \text{ mm}$$

Embedment depth for post-installed rebars top:

Clear distance between lapped bar:

$$e = \left[ \left( \frac{s}{2} \right)^2 + (c_n - c_c)^2 \right]^{0.5} - \phi = \left[ 100^2 + (50 - 30)^2 \right]^{0.5} - 16 \\ = 86 \text{ mm clear spacing between spliced bars}$$

Increase of lap length due to clear distance:

$$\Delta l_0 = e - 4\phi = 86 - 4 \cdot 16 = 22 \text{ mm}$$

$$l_0 = \max(l_{0,\text{pi}}; l_{0,\text{pi,fi}}; l_{0,\text{ci}}; l_{0,\min}) + \Delta l_0$$

Where:

$l_{0,\text{pi}}$  = design overlap length of post-installed bar = 542mm

$l_{0,\text{pi,fi}}$  = design overlap length of post-installed bar under fire = not required

$l_{0,\text{ci}}$  = design overlap length of cast-in bar = 643 mm

$l_{0,\min}$  = Minimum overlap length = 317 mm

$$= 643 + 22$$

$$= 665 \text{ mm}$$

$c_f = 30 \text{ mm}$

$w/2 = 125 \text{ mm}$

Drilling length

$$l_{\text{inst}} = l_0 + \max(w/2; c_f)$$

$$= 665 + 125$$

$$= 790 \text{ mm}$$

Embedment depth for post-installed rebars bottom:

Concrete in compression, no force on bars → anchorage with minimum embedment length.

$f_{\min} = 1.0$  (ETA 08/0105)

$$l_{b,\min} = f_{\min} \cdot \max(10\phi; 100\text{mm})$$

$$= 1.0 \cdot \max(10 \cdot 10; 100)$$

$$= 100 \text{ mm}$$

$w/2 = 125 \text{ mm}$

$$l_{\text{inst}} = l_{b,\min} + w/2 = 100 + 125 = 225 \text{ mm}$$

Drilling method	Borehole conditions	$l_{o,top}$	$l_{o,bottom}$	$C_{min}$ , see Section 3.3
		respectively $l_{drill,top}$	respectively $l_{drill,bottom}$	
Diamond coring	wet	790 mm	225 mm	82 mm/44 mm
Hammer drilling	Wet/dry	790 mm	225 mm	48 mm/35 mm

Specification: Provide post-installed rebar at size, spacing and anchorage as indicated on construction documents.

Anchoring system: Hilti HIT RE 500 V3 epoxy, install as per Manufacturer`s Printed Installation Instructions (MPII) permissible concrete temp. range for installation: -5°C to 40°C, concrete shall be dry or wet during installation.

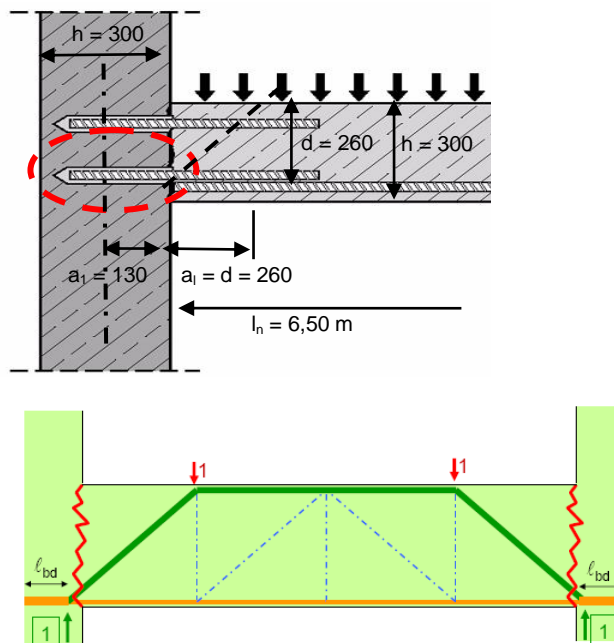
Drill hole using a rotary hammer drill with carbide bit or diamond core drilling. Locate existing reinforcing prior to drilling – do not damage reinforcing without prior authorization of the Engineer of record (EOR)

### 6.4.3 Design of end support of a simply supported slab according to HIT Rebar Design Method

Requirement:

Provide post-installed reinforcing bar for a new slab/beam on a concrete structure as shown below, **using the Hilti Rebar design concept.**

#### a) End support of slab, simply supported



Slab:  $l_n = 4,50\text{m}$ ,  $Q_k = 20\text{ kN/m}^2$ ,  $h = 300\text{ mm}$ ,  $d = 260\text{ mm}$

Wall:  $h = 300\text{ mm}$

Concrete strength class: C20/25, dry concrete

Reinforcement:  $f_{yk} = 500 \text{ N/mm}^2$ ,  $\gamma_s = 1.15$

Loads:  $G_k = 25 \text{ kN/m}^3 \cdot h = 7.5 \text{ kN/m}^2$ ;  
 $S_d = (1.50 \cdot Q_k + 1.35 \cdot G_k) = 40.1 \text{ kN/m}^2$

Structural analysis (design forces):

$M_{Ed} = S_d \cdot l_n^2 / 8 = 102 \text{ kNm/m}$

$V_{Ed} = S_d \cdot l_n / 2 = 90.3 \text{ kN/m}$

Bottom reinforcement required at mid span:

$A_{s,rqd,m} = (M_{Ed} \cdot \gamma_s) / (0.9 \cdot d \cdot f_{yk}) = 998 \text{ mm}^2/\text{m}$

Reinforcement provided at mid span:  $\varnothing 16$ ,  $s = 200 \text{ mm}$

$A_{s,prov,m} = 1005 \text{ mm}^2/\text{m}$

Bottom reinforcement at support:

Tension force to be anchored:  $F_{Ed} = |V_{Ed}| \cdot a / (0.9d) = 100 \text{ kN/m}$

(Clause 9.2.1.4(2), EC2: EN 1992-1-1:2004)

Steel area required:  $A_{s,rqd} = F_{Ed} \cdot \gamma_s / f_{yk} = 231 \text{ mm}^2/\text{m}$

Minimum reinforcement to be anchored at support:

$A_{s,min} = k_c \cdot k_{ct,eff} \cdot A_s / \sigma_s = 0,4 \cdot 1 \cdot 2,2 \cdot 150 \cdot 1000 / 500 = 264 \text{ mm}^2/\text{m}$

(Clause 7.3.2(2), EC2: EN 1992-1-1:2011)

$A_{s,min} = 0.5 \cdot A_{s,rqd,m} = 0,50 \cdot 988 = 499 \text{ mm}^2/\text{m}$

(Clause 9.3.1.2(1), EC2: EN 1992-1-1:2011)

$A_{s,min} = 0.25 \cdot A_{s,prov,m} = 0,25 \cdot 1010 = 251 \text{ mm}^2/\text{m}$

(Clause 9.2.1.4(1), EC2: EN 1992-1-1:2011)

Decisive is  $499 \text{ mm}^2/\text{m} \Rightarrow$  reinforcement provided:  $\varnothing 12$ ,  $s = 200 \text{ mm} \Rightarrow A_{s,prov} = 565 \text{ mm}^2/\text{m}$ ;

Installation by hammer drilling; Hilti HIT-RE 500 V3

Minimum anchorage length

$\sigma_{sd} = (A_{s,rqd} / A_{s,prov}) \cdot (f_{yk} / \gamma_s) = (23 / 565) \cdot (500 / 1,15) = 177 \text{ N/mm}^2$

$f_{bd,EC2} = 2,3 \text{ N/mm}^2$

(ETA-16/0142)

$l_{b,rqd} = (\phi / 4) \times (\sigma_{sd} / f_{bd}) = (12 / 4) \times (177 / 2.3) = 231 \text{ mm}$

$$l_{b,\min} = \max \{0.3\ell_{b,rqd}; 10\phi; 100\text{mm}\} = 120 \text{ mm}$$

(Clause 8.4.4(1), EC2: EN 1992-1-1:2011)

Development length:

$$\text{Cover dimension: } c_d = (s - \phi)/2 = 94 \text{ mm}$$

$$\text{Confinement } c_d/\phi = 94/12 = 7.8$$

Splitting bond strength for  $c_d/\phi > 3$ :

$$\alpha_2' = \frac{1}{\frac{1}{0.7} + \delta \cdot \frac{c_d - 3\phi}{\phi}} = \frac{1}{\frac{1}{0.7} + 0.306 \cdot \frac{94 - 3 \cdot 12}{12}} = 0.344$$

$$f_{bd,spl,2} = \frac{f_{bd,EC2}}{\max(\alpha_2'; 0.25)} = \frac{2.3}{0.344} = 6.7 \text{ N/mm}^2$$

$$\text{Pull-out bond strength: } f_{bd,p} = 9.3 \text{ N/mm}^2 \text{ (ETA-16/0143)}$$

$$\text{Applicable design bond strength: } f_{bd} = \min(f_{bd,spl}; f_{bd,p}) = 6.7 \text{ N/mm}^2$$

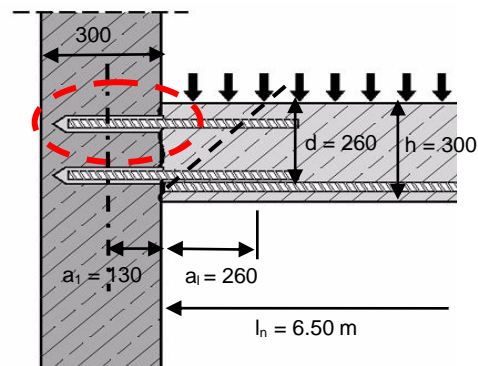
$$\text{Design development length: } \ell_{bd} = (\phi/4) \cdot (\sigma_{sd}/f_{bd}) = 80 \text{ mm}$$

Minimum length controls  $\rightarrow$  drill hole length  $l_{inst} = 120 \text{ mm}$

Note: Using the Hilti rebar design approach the minimum anchorage length controls with  $l_{b,\min} = 120\text{mm} = l_{inst}$ . In the same design example, following the Eurocode2 the design anchorage length controlled with  $l_{bd} = 162\text{mm} = l_{inst}$ .

Using the Hilti rebar design approach reduced the anchorage length of around 26%.

Top reinforcement at support:



Minimum reinforcement:

- a) 25% of bottom steel required at mid-span {Clause 9.3.1.2(2), EC2: EN 1992-1-1:2004}:

$$A_{s,req} = 0,25 \cdot 988 = 247 \text{ mm}^2/\text{m}$$

b) requirement for crack limitation {Clause 7.3.2(2), EC2: EN 1992-1-1:2004}:

$$A_{s,min} = 0,4 \cdot 1 \cdot 2,2 \cdot 150 \cdot 1000 / 435 = \underline{303 \text{ mm}^2/\text{m}}$$

Decisive is 303 mm<sup>2</sup>/m

⇒ reinforcement provided: Ø10, s = 200 mm; A<sub>s,prov</sub> = 393 mm<sup>2</sup>/m

Design stress in bar:  $\sigma_{sd} = f_{yd} \cdot A_{s,min} / A_{s,prov} = 335 \text{ N/mm}^2$

#### Minimum anchorage length

$$\sigma_{sd} = (A_{s,rqd} / A_{s,prov}) \cdot (f_{yk} / \gamma_s) = (23 / 565) \cdot (500 / 1,15) = 335 \text{ N/mm}^2$$

$$f_{bd,EC2} = 2,3 \text{ N/mm}^2 \text{ (ETA-16/0142)}$$

$$l_{b,rqd} = (\phi / 4) \times (\sigma_{sd} / f_{bd}) = (10 / 4) \times (335 / 2.3) = 364 \text{ mm}$$

$$l_{b,min} = \max \{0.3 l_{b,rqd}; 10\phi; 100\text{mm}\} = 110 \text{ mm}$$

(Clause 8.4.4(1), EC2: EN 1992-1-1:2011)

#### Development length:

$$\text{Cover dimension: } c_d = (s - \phi) / 2 = 95 \text{ mm}$$

$$\text{Confinement } c_d / \phi = 95 / 10 = 9.5$$

Splitting bond strength for  $c_d / \phi > 3$ :

$$\alpha_2' = \frac{1}{\frac{1}{0.7} + \delta \cdot \frac{c_d - 3\phi}{\phi}} = \frac{1}{\frac{1}{0.7} + 0.306 \cdot \frac{95 - 3 \cdot 10}{10}} = 0.293$$

$$f_{bd,spl,2} = \frac{f_{bd,EC2}}{\max(\alpha_2'; 0.25)} = \frac{2.3}{0.293} = 7.9 \text{ N/mm}^2$$

$$\text{Pull-out bond strength: } f_{bd,p} = 9.3 \text{ N/mm}^2 \text{ (ETA-16/0143)}$$

$$\text{Applicable design bond strength: } f_{bd} = \min(f_{bd,spl}; f_{bd,p}) = 7.9 \text{ N/mm}^2$$

$$\text{Design development length: } l_{bd} = (\phi / 4) \cdot (\sigma_{sd} / f_{bd}) = 97 \text{ mm}$$

Minimum length controls → drill hole length **l<sub>ef</sub> = 110 mm**

Therefore, drill hole length **l<sub>ef</sub> = 110mm**

Note:

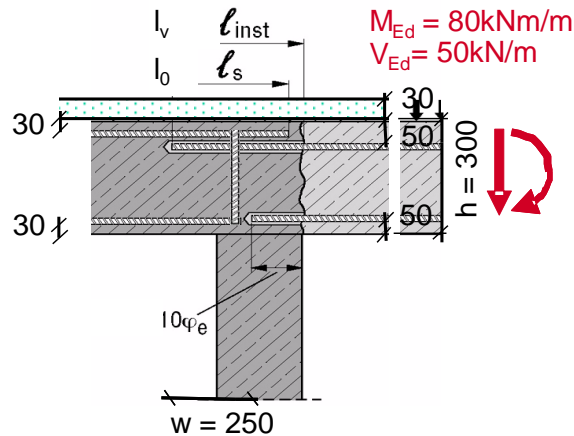
Using the Hilti rebar design approach the minimum anchorage length controls with  $l_{b,min} = 110\text{mm} = l_{inst}$ . In the same design example following the EC2 the design anchorage length controlled with  $l_{bd} = 255\text{mm} = l_{inst}$ .

Using the Hilti rebar design approach reduced the anchorage length of around 57%.



### 6.4.4 Design of splice on support according to HIT Rebar Design Method

Requirement: Provide post-installed reinforcing bar for a new balcony extension on an existing concrete structure as shown below, using the **Hilti Rebar design concept**.



As described in Section 6.3.1 the Hilti Rebar design concept does not bring beneficial in case of overlapping. Therefore the design according to Hilti Rebar design method will result in the same anchorage length as if Eurocode 2 was followed.

### 6.5 Strut-and-tie models

Eurocode2 [5] provides procedures for the development of strut-and-tie models to design reinforced concrete structures or members. This approach is particularly suitable for the design of system connections made out of *post-installed reinforcing bars*. The structure is divided into B- and D- regions. B-regions are parts of a structure in which Bernoulli's hypothesis of linear strain profiles applies. The internal stress state of B-regions can be easily derived from the sectional forces and the region can be designed on the basis of classical beam theory.

D-regions are parts of a structure with a complex variation in strain. They include portions near abrupt changes in geometry (geometrical discontinuities) or concentrated forces (statistical discontinuities). D-regions are assumed to extend a distance  $h$  (height of the member) from the force or geometric discontinuity (see Figure 35).

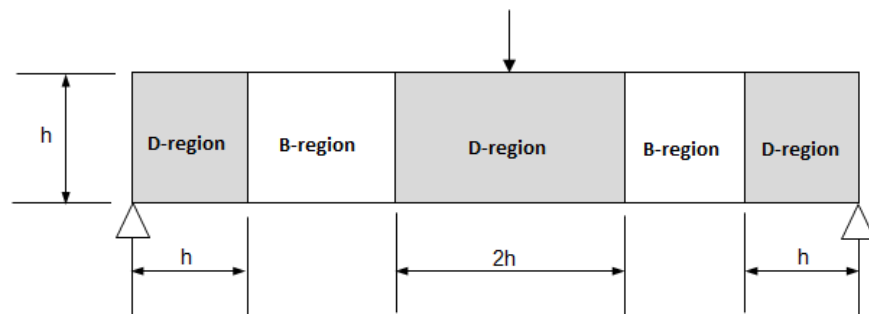


Figure 35 – Idealized D- and B-regions in a concrete beam

The design of D-regions is complex and requires a clear understanding of force flow. In strut-and-tie modelling, the complex state of internal forces is idealized as a truss. The compression (struts) and tension (ties) members are determined in the region by the user. The reinforcement is located in the determined tension areas while compression strength is verified in the struts. The strut and tie model, chosen among all, should result in small plastic deformations and economic reinforcement layout. The points of equilibrium where struts, ties, and concentrated forces intersect are denoted as nodes (Figure 36).

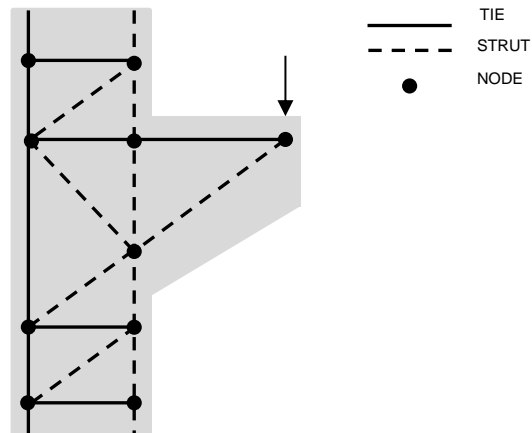


Figure 36 – Strut-and-tie model of a corbel

Nodes are in turn classified according to the sense and orientation of intersecting the ties and struts (Figure 37). The strut and tie model approach is treated extensively in the Eurocode 2 section 6.5.

Most *post-installed reinforcing bar* problems can be expressed with some variant of a C-C-T node, as shown in **Error! Reference source not found.**

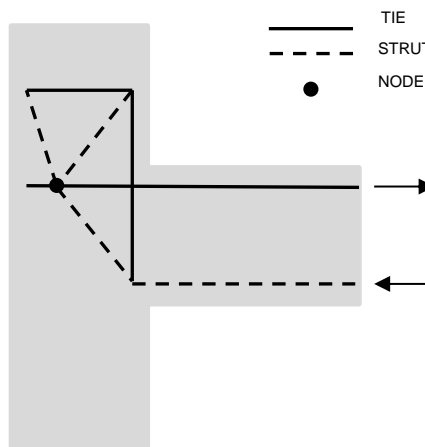


Figure 37 – Strut-and-tie model for a *post-installed reinforcing bar* connection

NOTE: A detailed description of strut-and-tie modeling of a *post-installed reinforcing bar* connection in a column-foundation joint subject to overturning moment is described in Kupfer, *et al.* [9]. Additional experimental verification of this connection is provided in

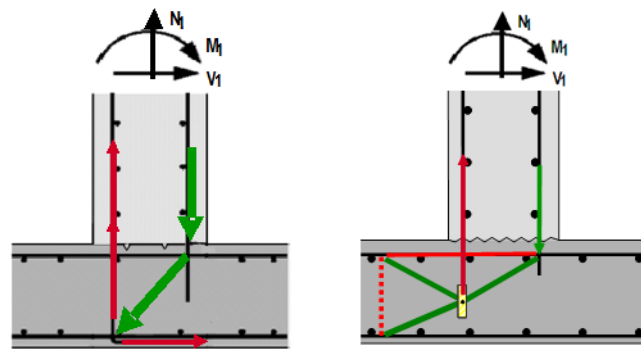
Hamad, B., *et al.*, "Evaluation of Bond Strength of Bonded-In or Post-Installed Reinforcement," ACI Structural Journal V. 103, No. 2, pp. 207-218 [7].

### 6.6 Strut-and-tie model for frame nodes (Hilti frame node concept)

If frame nodes (or moment resisting connections in general) are designed with cast-in reinforcement, they usually require bent bars according to the standard reinforced concrete design rules.

Anchoring the reinforcement of moment resisting connections with straight bars would, at least at first sight, result in concrete that is under tension, and therefore in a possible concrete cone failure. As this failure mode is brittle, such an anchorage is not in line with the philosophy of the national concrete design rules.

In cooperation with the Technical University of Munich, Hilti performed a research programme in order to provide a strut-and-tie model for frame nodes with straight connection bars [7, 9]. The main differences to the standard cast-in solution are that the compression strut is anchored in the bonding area of the straight bar (see Figure 38b) rather than in the bend of the bar (see Figure 38a) resulting in an reduced inner lever arm  $z_{1R}$  (see Figure 40b) inside the node and additional splitting forces in the transition zone between D- and B-region (see Figure 38b). These differences will be explained in detail in the following.



a)

b)

Figure 38 – Strut-and-tie model for a moment resisting connection with bended bar (a) and with *post-installed reinforcing bar* connection with straight end (red=tie, green= strut)

Global Equilibrium of the Node

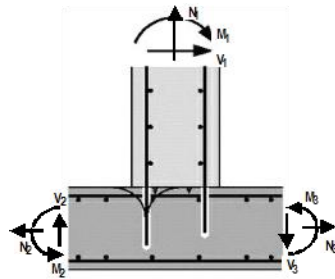
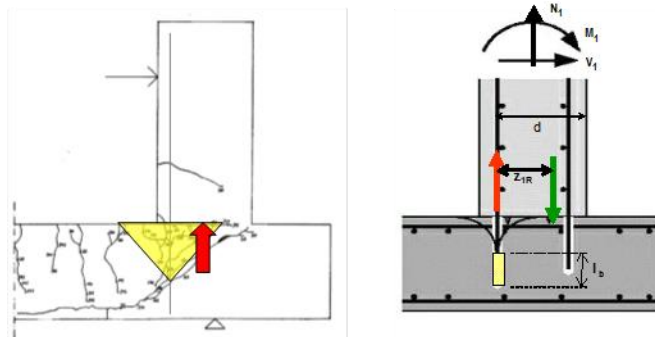


Figure 39 – Global equilibrium of node with *post-installed reinforcing bar connection with straight end*

In order to check the struts and ties inside the node, the reactions  $N_2$ ,  $V_2$ ,  $M_2$ ,  $N_3$ ,  $V_3$ ,  $M_3$  at the other ends of the node need to be defined (Figure 39). Normally, they result from the structural analysis outside the node region and will be determined by the designer in charge.

Tension in connecting bars

The loading of the wall (Figure 40b) results in a tensile force in the reinforcement on the left hand side (red arrow) and in a compression force on the right hand side (green arrow). Initial tests and numerical investigations [7, 9] lead to the consideration that the compression forces of the straight bar under tension due to bond has a tendency to orientate against the interface with the wall indicated by the crack pattern received at ultimate load (Figure 40a). Thus the compressive stress is not concentrated on the outside of the wall, but distributed over a large part of the interface, which leads to a reduced inner lever arm in the wall section ( $z_{1R}$ ). The recommended reduction factor for the inner lever arm is 0.85 for opening moments and 1.0 for closing moments.



a) Crack pattern resulted from tests, indicating a concentration of compression forces from concentrated loads on the left hand side  
 b) Reduced inner lever arm  $z_{1R}$  based on crack pattern (Fig. 43a) and numerical investigations [7, 9]

Figure 40 – Crack pattern and reduced inner lever arm with *post-installed reinforcing bar connection with straight end* (red=tie, green= strut)

Anchorage length

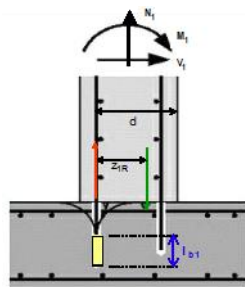


Figure 41 – Definition of anchorage length  $l_b$

While the equilibrium inside of frame nodes with cast-in hooked bars can be modeled with the compression strut continuing from the vertical compression force and anchored in the bend at the level of the lower reinforcement (Figure 38a), straight bars are anchored by bond stresses at a level above the lower reinforcement.

As bending cracks are expected to occur at the loaded end of the tension loaded rebar, the anchorage zone is developing from the lower end of the bar and its length  $l_b$  is that required to develop the steel stress but calculated from the section forces  $M_1$ ,  $N_1$  and  $V_1$ .

$$l_b = \frac{\sigma_{sd} \cdot \phi}{4 \cdot f_{bd}} \text{ [mm]}$$

with  $\sigma_{sd}$  design steel stress in the connection bars [N/mm<sup>2</sup>]  
 $\phi$  diameter of the vertical bar [mm]  
 $f_{bd}$  design bond strength of cast-in bar to concrete or of the adhesive mortar [N/mm<sup>2</sup>]

Installation length

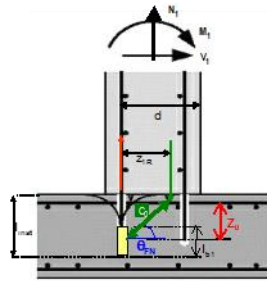


Figure 42 – Definition of strut angle  $\theta_{FN}$

The strut-and-tie model requires a strut angle,  $\theta_{FN}$ , between the inclined compression strut  $C_0$ , limited between 30° and 60°, based on current knowledge.

For short anchorage lengths, where the necessary resulting strut angle would be less than 30°, the design should not be applied. This is based on the fact that in such cases the struts from bond would not be balanced with the local compression strut from the wall and a brittle concrete cone failure may occur. A minimum drill hole length is also required in order to remain as close as possible to the solution with bent bars. Therefore it is recommended to drill the holes as deep as possible to achieve a large strut angle  $\theta_{FN} \leq 60^\circ$ .

The software developed by Hilti for the design of post-installed rebar connections, PROFIS Rebar, preferably propose the installation (drilling) length such that the strut angle  $\theta_{FN}$  is equal to 60°. In cases where the existing base material does not provide enough depth, the maximum possible embedment depth which is defined for bonded anchors in ETAG 001, part 5 [1] is taken as

$$l_{inst,max} = h_{member} - \max(2 \cdot d_0; 30\text{mm})$$

with  $l_{inst,max}$  maximum possible installation length [mm]  
 $h_{member}$  thickness of the existing concrete member [mm]  
 $d_0$  diameter of the drilled hole [mm]

Drilling longer than  $l_{inst,max}$  may lead to spalling of the concrete cover due to the impact energy and should therefore be avoided.

Tension in Existing Reinforcement

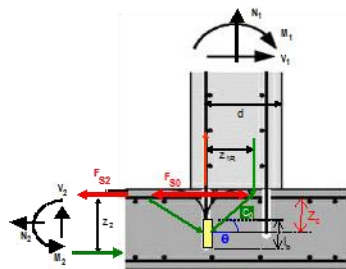


Figure 43 – Definition of the inner lever arm inside the node  $z_0$

Taking account of the installation length  $l_{inst}$  and a concrete cover measured from the upper reinforcement to the center of the transverse bars equal to  $c_s$ , the lever arm inside the node  $z_0$  can be calculated as follows:

$$z_0 = l_{inst} - \frac{l_b}{2} - c_s \text{ (mm)}$$

The value of the lever arm inside the node  $z_0$  is therefore smaller than the value of the lever arm of the slab  $z_2$ . As a consequence, the tension force in the upper slab reinforcement in the node region,  $F_{S0}$ , is higher than the tension force calculated for the slab using  $z_2$ . Based on this, the

tensile resistance of the existing upper reinforcement  $A_{s0,prov}$  must be verified separately as follows:

$$F_{s2} = M_2/z_2 + N_2/2 \text{ (N), (tension in existing reinforcement outside node area)}$$

$$H_{s2} = \left( M_1 + (V_2 + V_3) \cdot \frac{z_1}{2} \right) \cdot \left( \frac{1}{z_0} - \frac{1}{z_2} \right) + V_1 \cdot \left( \frac{z_{1R}}{z_0} - 1 \right) \text{ (additional tension in node due to reduced lever arm)}$$

$$F_{s0} = F_{s2} + H_{s2} \text{ (steel tension in node area)}$$

$$A_{s0,rqd} = F_{s0} / (f_{yk} / \gamma_s) \text{ (steel area required in existing part for forces from new part)}$$

If the provided steel area  $A_{s0,prov}$  is greater than or equal to the required one  $A_{s0,rqd}$ , the reinforcement of the existing part is sufficient. This is the analysis obtainable from PROFIS Rebar.

As mentioned above, a more sophisticated check needs to be made if there are also additional loads acting. In that case  $F_{s2}$  evaluated under “global equilibrium” should be evaluated under a complete static design.

The smaller the value of the embedment of the post-installed vertical bar, the higher the moment resistance of the slab in the node region is reduced compared to a node with hooked bar. Therefore it is also recommended to provide sufficient embedment of the connecting bars rather than trying to optimize mortar consumption by trying to provide shortest possible embedment depth.

Concrete Compressive Strut

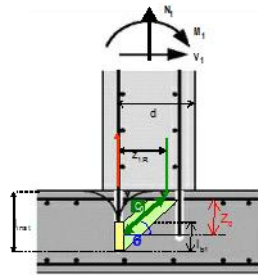


Figure 44 – Definition of the compression strut  $C_0$

The strut-and-tie model assumes that the compression strut  $C_0$  is anchored at the center of the anchorage zone and that its compression field width corresponds to the length of the anchorage zone  $l_b$ .

$$F_{c0} = \frac{M_1 + (V_2 + V_3) \cdot z_1 / 2}{z_0}$$

(horizontal component of concrete strut force)

$$D_0 = F_{c0} / \cos \theta_{FN}$$

(compression force in concrete in direction of strut)

$$\sigma_{Rd,max} = v' \cdot k_2 \cdot \alpha_{cc} \cdot f_{ck} / \gamma_c$$

(reduced concrete strength in tension-compression node according to ENV1992-1-1, 4.5.4(4b). Standard parameters:  $v'=1-f_{ck}/250$ ;  $k_2=0.85$ ;  $\alpha_{cc}=1.0$ ;  $\gamma_c=1.5$ , subject to variations in National Application Documents)

$$D_{0,R} = \sigma_{Rd,max} \cdot l_b \cdot w \cdot \cos \theta_{FN} \text{ (resistance of concrete in direction of strut, } w=\text{width of section)}$$

(resistance of concrete in direction of strut,  $w$ =width of section)

If  $D_{0R} \geq D_0$ , the concrete strut can take up the loads introduced from the new section.

Splitting of Concrete in Transition Area

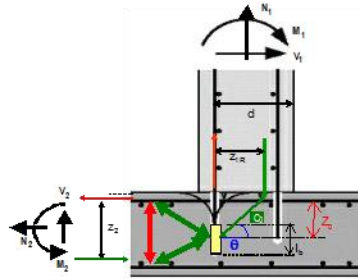


Figure 45 – Definition of anchorage length  $l_b$

On the left hand side of the anchorage zone, the compression force is continuing through additional struts to the tension and compression zones of the B-region of the slab where the equilibrium of the horizontal forces is given. The vertical components of these struts are taken up by tensile stresses in the concrete. Normally there is no vertical reinforcement in the slab to take up the tension force. The loads and thermal solicitations of a slab do not lead to horizontal cracking; therefore it is possible to attribute the tension force to the tensile capacity of the concrete. On the safe side, the maximum splitting stress has been taken as that caused by a concentrated load  $C_0$  on the center of the anchorage zone. It has been shown that the occurring splitting stress  $\max \sigma_{sp}$  can be calculated as

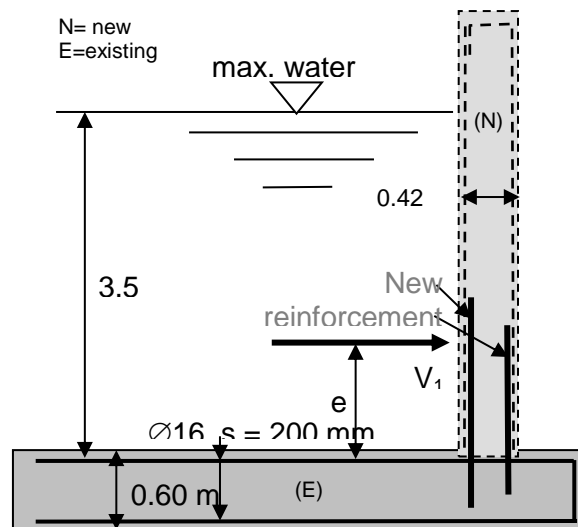
It has been shown that the occurring splitting stress  $\max \sigma_{sp}$  can be calculated as

$$\max \sigma_{sp} = \left( M_1 + \frac{(V_2 + V_3) \cdot z_1}{2} \right) \cdot \left( 1 - \frac{z_0}{z_2} \right) \cdot \left( 1 - \frac{l_b}{2 \cdot z_2} \right) \cdot \left( \frac{2.42}{b \cdot z_2^2} \right) \leq f_{ct}$$

- with:  $M_1, V_2, V_3$ : external forces on node according to figure 5
- $z_2$ : inner lever arm of slab section outside node region
- $b$ : width of the wall section
- $f_{ctd} = f_{ct} \cdot 0.7 \cdot 0.3 \cdot f_{ck}^{2/3} / f_c$ : tensile strength of concrete (Standard value in Eurocode2:  $\alpha_{ct}=1.0$ , subject to variations in National Application Documents)

If the calculated maximum splitting stress is smaller than the tensile strength of the concrete  $f_{ctv}$  then the concrete can take up the splitting forces without any additional shear reinforcement.

**6.7 Design of a moment resisting connection according to HIT Rebar Design Method**



- Geometry:
  - $h_1 = 420 \text{ mm}; h_2 = h_3 = 600 \text{ mm};$
  - $d_1 = 380 \text{ mm}; d_2 = d_3 = 560 \text{ mm};$
  - $z_1 = 360 \text{ mm}; z_2 = z_3 = 520 \text{ mm}$
  - $A_{s0} = A_{s2} = A_{s3} = 1005 \text{ mm}^2/\text{m} (\text{Ø}16 \text{ s} = 200 \text{ mm})$
  - $c_s = h_2 - d_2 = 40 \text{ mm}$
- Material:
  - Concrete: C20/25 (new and existing parts),  $\gamma_s = 1.5$
  - Steel grade: 500 N/  $\text{mm}^2$ ,  $\gamma_s = 1.15$
  - Safety factor for variable load:  $\gamma_Q = 1.5$

### HIT-RE 500 V3 (temperature range I, hammer drilling)

#### Acting loads:

$$\begin{aligned}
 V_{1d} &= \gamma_Q \cdot p \cdot h^2 / 2 = 1.4 \cdot 10 \cdot 3.5^2 / 2 &&= 92 \text{ kN/m} \\
 e &= h / 3 = 3.5 / 3 &&= 1.17 \text{ m} \\
 M_{1d} &= V_{1d} \cdot e = 92 \cdot 1.17 &&= 107 \text{ kNm/m}
 \end{aligned}$$

#### Force in post-installed reinforcement

$$\begin{aligned}
 z_{1r} &= 0.85 \cdot z_1 = 0.85 \cdot 360 &&= 306 \text{ mm} \\
 &\text{(opening moment} \rightarrow \text{reduced inner lever arm)} \\
 F_{s1d} &= M_{1d} / z_{1r} = 107 / 0.306 &&= 350 \text{ kN/m} \\
 A_{s1,rqd} &= F_{s1d} / (f_{yk} / \gamma_{Ms}) = 350'000 / (500 / 1.15) &&= 805 \text{ mm}^2/\text{m} \\
 \text{Select } \phi 12\text{mm, spacing } s_1 = 125\text{mm} \rightarrow A_{s1,prov} &&&= 905 \text{ mm}^2
 \end{aligned}$$

$$\begin{aligned}
 \rightarrow \text{drilled hole diameter: } d_0 &&&= 16 \text{ mm} \\
 \text{Stress in bar: } \sigma_{sd} = F_{s1d} / A_{s1,prov} &&&= 386 \text{ N/mm}^2
 \end{aligned}$$

#### Anchorage length

$$f_{bd,EC2} = 2.3 \text{ N/mm}^2 \text{ (ETA-16/0142)}$$

$$l_{b,rqd,EC2} = (\phi/4) \cdot (\sigma_{sd} / f_{bd,EC2}) = 504 \text{ mm}$$

$$l_{b,min} = \max \{0,3\lambda_{b,rqd,EC2}; 10\phi; 100 \text{ mm}\} = 151 \text{ mm}$$

$$f_{bd,b} = 9.3 \text{ N/mm}^2 \text{ (ETA-16/0143)}$$

$$c_d = s_1/2 - \phi/2 = 56.5 \text{ mm} > 3\phi$$

$$\alpha_2' = \frac{1}{\max \left[ \frac{1}{0.7} + \delta \cdot \frac{c_d - 3\phi}{\phi}; 0.25 \right]} = 0.512$$

$$f_{bd,sp12} = \frac{f_{bd}}{\max [\alpha_2'; 0.25]} = 4.5 \text{ N/mm}^2$$

$$f_{bd} = \min \{f_{bd,b}; f_{bd,sp12}\} = 4.5 \text{ N/mm}^2$$

$$l_{b1} = \max \{(\phi/4) \cdot (\sigma_{sd} / f_{bd}); l_{b,min}\} = 258 \text{ mm}$$

#### Drilled hole length

$$\begin{aligned}
 l_{inst,max} &= h_2 - \max \{2d_0; 30\text{mm}\} &&= 568 \text{ mm} \\
 &\text{(maximum possible hole length)}
 \end{aligned}$$

$$\begin{aligned}
 l_{inst,60} &= c_s + z_{1R} \cdot \tan 60^\circ + l_{b1} / 2 &&= 672 \text{ mm} \\
 &\text{(hole length corresponding to } \theta=60^\circ \text{)}
 \end{aligned}$$



$$l_{inst,60} > l_{inst,max} \rightarrow \text{select hole length } l_{inst} = l_{inst,max} = 568 \text{ mm}$$

$$\text{Strut angle with } l_{inst,max}: \tan \theta = (l_{inst,max} - c_s - l_{b1}/2) / z_{1R} \rightarrow \theta_{FN} = 53^\circ$$

check:  $\theta > 30^\circ \rightarrow \text{ok}$

#### Reaction in Foundation:

$$-M_{2d} = M_{1d} + V_{1d} \cdot z_2 / 2 = 107 + 0.25 \cdot 92 = 131 \text{ kNm/m}$$

$$N_{2d} = -V_{1d} = -92 \text{ kN/m}$$

$$M_{s3} = 0; V_{2d} = V_{3d} = 0; N_1 = N_3 = 0$$

#### Check of foundation reinforcement

$$F_{s2d} = M_{2d} / z_2 + N_{2d} / 2 = 298 \text{ kNm/m}$$

(tension outside node area)

$$z_0 = l_{inst} - c_s - l_{b1} / 2 = 568 - 40 - 258 / 2 = 399 \text{ mm}$$

(lever arm in node area)

$$H_{s2d} = M_{1d} \cdot (1/z_0 - 1/z_2) + V_{1d} \cdot (z_1/z_0 - 1) = 53 \text{ kNm/m}$$

(additional force in node area)

$$F_{s2d,node} = F_{s2d} + H_{s2d} = 351 \text{ kNm/m}$$

(tension in node area)

$$A_{s2,rqd} = F_{s2d,node} / (f_{yk} / \gamma_{Ms}) = 351 \cdot 1000 / (500 / 1.15) = 808 \text{ mm}^2/\text{m}$$

$$A_{s2} > A_{s2,rqd} \rightarrow \text{ok (} A_{s2} \text{ is given)}$$

#### Check concrete compressive strut

$$F_{c0d} = M_{1d} / z_0 = 268 \text{ kN/m}$$

$$D_{0d} = F_{c0d} / \cos \theta_{FN} = 441 \text{ kN/m}$$

$$\alpha_{cc} = 1.0$$

(EC2: EN 1992-1-1:2004, 3.1.6(1))

$$v' = 1 - f_{ck} / 250 = 0.92$$

(EC2: EN 1992-1-1:2004, 6.5.2(2))

$$k_2 = 0.85$$

(EC2: EN 1992-1-1:2004, 6.5.4(4b))

$$D_{0Rd} = \alpha_{ct} \cdot v' \cdot k_2 \cdot f_{ck} / \gamma_c \cdot l_{b1} \cdot \cos \theta_{FN} = 1639 \text{ kN/m}$$

$$D_{0Rd} > D_{0d} \rightarrow \text{ok}$$

### Check concrete splitting in plane of foundation

$$\alpha_{cc} = 1.0$$

(EC2: EN 1992-1-1:2004, 3.1.6(2))

$$f_{ctk,0.05} = \alpha_{ct} \cdot 0.7 \cdot 0.3 \cdot f_{ck}^{2/3} / \gamma_c = 1.03 \text{ N/mm}^2$$

(table 3.1, EC2: EN 1992-1-1:2004)

$$M_{sp,d} = F_{c0d} \cdot z_0 \cdot (1 - z_0/z_2) \cdot (1 - l_{b1}/(2z_2)) = 1.87 \cdot 10^7 \text{ Nmm/m}$$

$$W_{sp} = 1000\text{mm} \cdot z_2^2 / 2.41 = 1.12 \cdot 10^8 \text{ mm}^3/\text{m}$$

$$\max \sigma_{sp} = M_{sp,d} / W_{sp} = 0.17 \text{ N/mm}^2$$

$$f_{ctk,0.05} > \max \sigma_{sp} \rightarrow \text{ok}$$

NOTE: Additional checks for the adequacy of the model may be required. For further information, see Hamad, B., *et al.* "Evaluation of Bond Strength of Bonded-In or Post-Installed Reinforcement," ACI Structural Journal V. 103, No. 2, pp. 207-218 [7].

## 7. What else do I need to know?

### 7.1 Cracked concrete vs. uncracked concrete

According to TR023, to account for a possible different behavior of a post-installed rebar and cast-in rebar in cracked concrete, the minimum anchorage length should be amplified by a factor of 1.5. This increase is omitted if tests show that the bond strength of post-installed rebar and cast-in rebar in cracked concrete is similar [15]. In addition, in most of post-installed rebar applications, cracks are located perpendicular to the reinforcement axis without affecting the bond loading capacity of the system connection. In the related ETA for post-installed rebar of each product, values of bond strengths and amplification factors for minimum anchorage lengths are reported.

The differentiation between cracked and uncracked concrete needs more attention when the HIT Rebar Design Method is applied in order to reduce the anchorage length of post-installed rebar installed in a system connection. As a first step the direction of the cracks which could develop along the reinforcement should be identified. When the cracks are parallel to the reinforcement axis the upper limit of the HIT Rebar Design Method is the bond strength for cracked concrete shown in the anchor approval of the related product.

### 7.2 Additional factors influencing the bond strength

Sustained tension loading of adhesive anchors has been associated with excessive creep in certain anchor applications. For post-installed reinforcing bar applications, judgment is required to determine whether additional precautions for sustained tension load applications may be required. For specific cases, e.g., where a small number of bars are subjected to direct tension as a result of dead loads, use of the reduced bond stress may be appropriate. Additionally, reductions on bond strength for concrete temperature, presence of water (e.g., saturated concrete), and installation in lightweight concrete should be adopted as appropriate.

### 7.3 Fatigue

High-cycle fatigue loading is not specifically addressed in the qualification requirements contained in TR 023 [15] for post-installed rebar connections. However for load bearing elements which are subjected to considerable cyclic loading the bonded-in connections should be designed for fatigue. In case of evidence for fatigue resistance of reinforcing steel bars, concrete and bond should be provided separately.

For simple cases, it is reasonable to use simplified methods and be on the safe side.

The partial safety factors for loads are specified in the code for reinforced concrete.

The partial safety factors for material are specified in Table 4.3.

**Table 4.3:** Partial safety factors for materials subjected to cyclic loading

Evidence for	Concrete related failure modes (concrete cone and splitting)	Bond related failure modes (pull out)	reinforcing bars (steel)
Partial safety factor [-]	1.5	1.8	1.15

#### Fatigue of reinforcing bars (steel)

The resistance for fatigue of reinforcing bars (steel) is specified in the actual code for reinforced concrete. The behaviour of reinforcing bars bonded-in by means of HIT-products is at least as good as cast-in place reinforcement.

#### Fatigue of bond and concrete (simplified approach)

As a simple and conservative approach, evidence for fatigue is proven if the following equation is verified:

$$F_{Sd,fat} \leq N_{Rd} \cdot f_{fat} \quad (N)$$

where:

$F_{Sd,fat}$  Design value of the anchorage force for the rolling loading model for fatigue (N)

$N_{Rd}$  Design resistance for static load of the anchorage (bond and concrete) (N)

$f_{fat}$  Reduction factor for fatigue for bond and concrete:  $f_{fat}=0.5$

If max/min of cycles is known, reduction factors are shown in Figure 46.

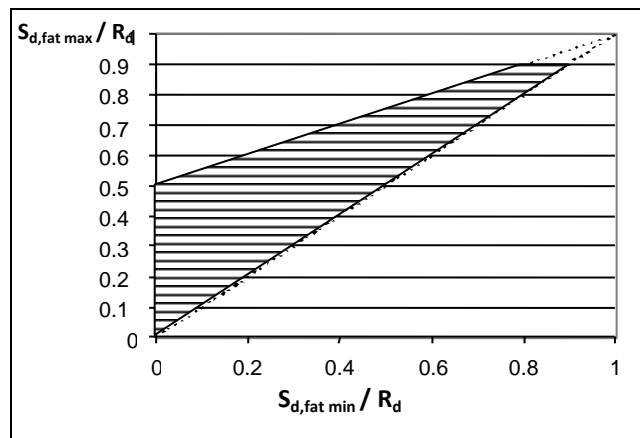


Figure 46 – Diagram for a simplified approach with  $2 \cdot 10^6$  cycles (Weyrauch diagram)

**Reduction factors for fatigue for bond and concrete**

If the simplified method is not satisfying, additional information using Woehler curves is available. Ask Hilti Technical Service for additional information.

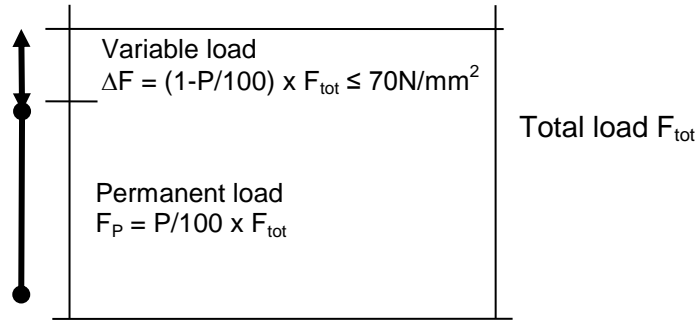
**Design Approach**

**Steel resistance:**

The steel resistance under fatigue load is calculated based on the part of the load which is permanent, the portion of load which is cycling and the steel yield strength. The safety factors are the same as those used for static design (taken from ENV 1992-2-2:1996, sect. 4.3.7.2).

$\Delta\sigma_{s,max}$  = maximum allowable stress variation, usually given by codes, e.g. ENV 1992-2-2:1996, sect. 4.3.7.5:  $\Delta\sigma_{s,max} = 70 \text{ N/mm}^2$

P percentage of the load which is permanent:  $0 \leq P \leq 100$



The reduction factor on steel resistance due to dynamic loading can be calculated as follows:

$$f_{red,s,dyn} = \frac{\min(f_{yk}, \frac{70}{1 - (\frac{P}{100})})}{f_{yk}} (-)$$

while the resulting maximum steel stress taken into account for fatigue loading can be given as follows:

$$\sigma_{s,max,dyn} = f_{red,s,dyn} \cdot f_{yk} \text{ (N/mm}^2\text{)}$$

**Concrete related resistance**

The concrete resistance calculated for static loading is reduced by a reduction factor for fatigue loads,  $f_{red,c,dyn}$ , which is applied to all types of concrete related failure modes, i.e. splitting, concrete edge failure under shear for cracked and non-cracked concrete. This factor is calculated from the Weyrauch diagram of Eurocode2 (ENV 1992-2-2:1996, section 4.3.7.4), by means of:

$$f_{red,c,dyn} = 0.5 + 0.45 \cdot \frac{P}{100} \leq 0.9$$

For P=100 (only permanent loads),  $f_{red,c,dyn}$  is, of course 1.0, but as soon as P<100,  $f_{red,c,dyn} \leq 0.9$ .

For simplification,  $f_{red,c,dyn} = 0.5$  can be conservatively taken.

### Bond Resistance

The bond resistance calculated for static loading is reduced by a reduction factor for fatigue loads,  $f_{red,b,dyn}$ . This factor is calculated from the Weyrauch diagram based on intend testing and literature reviews [8] and is valid for Hilti mortars only. It has to be chosen between two formulas. Depending on the drilling method,  $f_{red,b,dyn}$  can be determined as follow:.

General approach for Hilti mortars:

$$f_{red,b,dyn} = 0.63 + 0.37 \cdot \frac{P}{100} \leq 0.9$$

For Hilti mortar HIT-RE 500 V3 in diamond cored drilled holes (water saturated):

$$f_{red,b,dyn} = 0.53 + 0.47 \cdot \frac{P}{100} \leq 0.9$$

For simplification  $f_{red,c,dyn}=0.5$  can be conservatively taken.

Contact Hilti technical staff for additional guidance on this topic.

### 7.4 Fire

Organic adhesives are affected by high temperatures. For post-installed reinforcing bar connections that are part of a fire-rated assembly (floor, roof, etc.) it is important that the fire resistance of the connection be evaluated using test data for the time-dependent reduction in bond strength associated with typical geometries and time-temperature loading protocols.

The bond strength of post-installed rebar subjected to fire is evaluated through tests based on a new EAD [18]. The rebar ETA of the related product reports the reduction factor to be applied at the bond strength as function of the temperature. As example, Figure 46 shows the reduction factor vs. temperature for HIT-RE 500 V3.

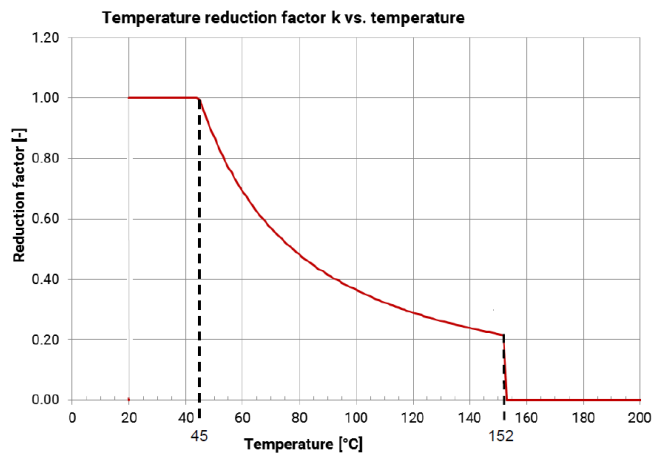


Figure 49 – Reduction factor  $K_{fi}(\theta)$  as function of temperature (from ETA – 16/0142)

The temperature profile along the anchorage length can be either constant (Figure – 48a) and function of concrete cover and time exposure or variable and function of anchorage length itself and time exposure (Figure – 48b). The slab-to-slab connection is, for instance representative of the first condition (“parallel”), while the simply supported connection is representative of the second condition (“anchor”).

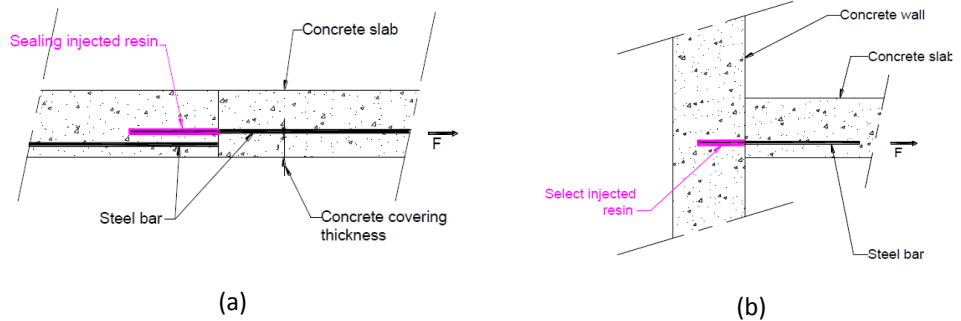


Figure 50 – Sketches of a slab-slab connection (a) and of a beam-wall connection (b)

The design follows the rules of the Eurocode2:

$$E_{d,fi} \leq R_{d,t,fi}$$

The action,  $E_{d,fi}$ , is 70% of the “cold” action (supposing that in case of fire the building is not subjected to the maximum design load), and the reaction,  $R_{d,t,fi}$ , is reduced based on the temperature the system connection is subjected to during the event.

Three are the connection components which need to be checked: concrete, steel and mortar.

Concrete have shown sensitivity to high temperature lower than mortar and steel. Therefore, the latters need more attention. For both, mortar and steel, the reduced strength due to high temperature is assessed by using the reduction coefficients available in the ETA and Eurocode2, respectively.

The evaluation of the temperature profile along the anchorage length needs, in some cases, the usage of a FEM programs and as a consequence the process can be long. To support our customers, Hilti developed performance tables for different type of applications, where the bond strengths / bond loads are reported. In particular, in case of overlapping the bond strength is function of the concrete cover, while in case of simply supported/moment resisting nodes the bond load is function of anchorage length itself and time exposure.

“Parallel” case

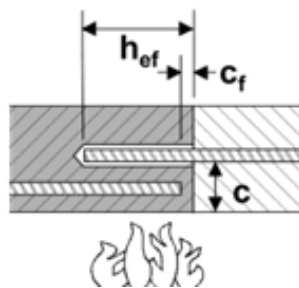


Figure 47 – Fire condition “parallel”

Concrete Cover (mm)	Fire Design Bond Resistance $f_{bd,fi}$ (N/mm <sup>2</sup> )					
	R30	R60	R90	R120	R180	R240
30						
40	0.8					
50	1.1					
60	1.5					
70	2.1	0.9				
80	2.9	1.2				
90	3.5	1.5	0.9			
100		1.8	1.1	0.8		
110		2.3	1.4	1.0		
120		2.8	1.6	1.2		
130		3.4	2.0	1.4	0.9	
140		3.5	2.3	1.6	1.0	

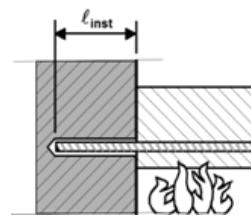
Figure – 48 Applicable design bond strength values  $\tau_{Rd,fi}$  ( $\tau_c$ ) under fire as a function of the clear concrete cover c, situation parallel (example)

In the fire condition “parallel”, the only decisive parameters are the clear distance from the fire exposed concrete surface to the perimeter of the bar (“clear concrete cover *c*”) and the specific fire durations. From these two parameters the applicable design bond strength of the adhesive for specific fire duration can be taken from the relevant tables for which Figure – 48 provides an extract.

For fire design, parameters to account for bond conditions like temperature profile along the anchorage length and reduction coefficient are not needed. It is sufficient to prove that the calculated splice or anchorage length is sufficient to transfer the load under fire with the given fire design bond strength  $\tau_{Rd,fi}$  (denominated  $\tau_c$  in Fig. 51). According to Figure – 48, for example, the design bond strength  $\tau_{Rd,fi}$  for a concrete cover of *c*=70mm and a fire duration time of R60 is equal to  $\tau_{Rd,fi}$ = 0.9 N/mm<sup>2</sup>. Based on this, the maximum tension force on the rebar under the assumed boundary conditions can be calculated as follows:

$$F_{fire} = \tau_{Rd,fi} \cdot \pi \cdot h_{ef} \cdot \phi \quad (N)$$

“Anchor” case



Concrete Cover = 10 mm		Fire Design Load Resistance N <sub>Rd,Fire</sub> (kN)					
Diameter (mm)	Length l <sub>v</sub> (mm)	R30	R60	R90	R120	R180	R240
8	100	3.4	1.1	0.5	0.2	0.0	0.0
	140	6.9	4.1	2.2	1.4	0.5	0.0
	180	10.3	7.6	5.4	3.8	1.9	1.0
	220	13.8	11.1	8.9	7.2	4.3	2.7
	260	16.8	14.5	12.4	10.7	7.6	5.2
	290		16.9	15.0	13.3	10.3	7.8
	320			16.8	15.9	12.9	10.4
	330				16.8	13.7	11.3
	370					16.8	14.7
	400						16.8
10	110	5.3	2.2	1.0	0.5	0.0	0.0
	150	9.6	6.2	3.6	2.4	0.9	0.3
	190	14.0	10.5	7.9	5.8	3.0	1.7
	230	18.3	14.5	12.2	10.1	6.3	4.0
	270	22.7	19.2	16.5	14.4	10.6	7.6
	310	26.2	23.6	20.9	18.8	15.0	11.9
	340		26.2	24.1	22.0	18.2	15.2
	360			26.2	24.2	20.4	17.3
	380				26.2	22.6	19.5
	420					26.2	23.8
450						26.2	

Figure 49 – Fire condition “anchorage”

Figure 50 – Applicable tension force *F<sub>fire</sub>* (*F<sub>s,T</sub>*) under fire as a function of the bar diameter, installation length and fire duration class (example)

In the fire condition “anchorage”, the tables directly show the fire resistance as a force [kN] for given diameters, embedment depths, concrete cover and fire durations.

The tables are providing maximum steel forces under fire as a function of the bar diameter, installation length and fire duration time. It must be noted that these values are derived from a specific value of the yielding strength of the rebar, *f<sub>yk,fi</sub>* and would be different for other values of *f<sub>yk,fi</sub>* ). According to Figure 50, for example, the maximum design bond load, *F<sub>bd,fi</sub>* for an anchorage length of *c*=270mm, a concrete cover of 10 mm and a fire duration time of R60 is equal to *F<sub>bd,fi</sub>* = 19.2 kN.

While intermediate values of the maximum steel force between those given in fire design taking account of the installation/anchorage length may be interpolated linearly, an extrapolation is not permitted.

Contact Hilti technical staff for additional guidance on this topic.

### 7.5 Seismic design of structural post-installed rebar

An increasing population density, the concentration of valuable assets in urban centers and society's dependence on a functioning infrastructure demand a better understanding of the risks posed by earthquakes. In several areas around the globe, these risks have been reduced through appropriate building codes and state of the art construction practices. The development of pre-qualification methods to evaluate building products for seismic conditions additionally contributes to safer buildings for generations to come.

DTAs approvals delivered by CSTB, a member of EOTA, recognizes Hilti products as HIT-RE 500 V3 and HIT-HY 200-A/R injectable mortars as products qualified for structural rebar applications in seismic zones. This national approval requires that qualified products have an ETA approval for rebar, an ETA approval for anchorage in cracked concrete, as well as an ICC-ES pre-qualification for seismic conditions.

The seismic design is restricted to:

- $\max f_{yk} = 500 \text{ N/mm}^2$
- concrete of class lower than C20/25 shall not be used
- only ductile reinforcement (class C)
- no combination of post-installed and e.g. bent connection bars to ensure displacement compatibility
- only specific injection systems for the anchoring of reinforcement:
  - HIT-HY 200-A/R
  - HIT-RE 500 V3

The design procedure is fully details in the approval and, in addition to detailing rules of Eurocode2/rebar ETA, consider the following detailing rules of EN1998-1:2004 (Eurocode8) [11]:

- Use of the system is limited to new constructions or structures undergoing major repair work
- The structure in which the rebar is set shall be designed for seismic action and the reinforcement shall be designed to pick up the forces that are generated by the subsequently post-installed rebar.
- The design study shall be performed at the same time as the reinforcement of the initial structure is tested.
- The process can be used to take some traction or compression stresses. The application is not designed to work as a stud.
- The calculation of the post-installed rebar and the determination of the internal forces shall be performed in accordance with EN 1992-1-1 and EN 1998-1-1, and shall coincide with the design of the structure.
- The loads applied to the reinforcements taking account of the acceleration shall be the responsibility of the design office. The office may decide to set additional reinforcements or greater anchorage lengths.
- The verification of the local transfer of loads in the concrete shall be performed.
- The verification of the transfer of anchored loads in the structure shall be performed.



- The distance between the reinforcement bars is specified for each product in the related European Technical Approval.
- The provisions for anchorages and splices shall follow the EN 1998-1-1.

Tables with design values of bond strengths in seismic zones are provided in the FTM injection system datasheets

## 7.6 Corrosion

Concrete is a naturally alkaline material and under normal conditions corrosion of embedded reinforcing is prevented by passivation of the bar surface. However, when concrete undergoes carbonation, its decreased pH can break the passivation film of rebar and allow corrosion. Furthermore, faster corrosion rates (pitting corrosion) are observed if the concrete is contaminated with chlorides.

The qualification of adhesive systems for *post-installed reinforcing bar* applications includes a specific test for the susceptibility of the system to long-term bar corrosion. Bars installed with qualified systems should exhibit similar corrosion rates to cast-in-place bars in the same concrete. It is important that the adhesive surrounding the bar be relatively void-free to minimize corrosion. Therefore, installation quality is important for corrosion resistance as well as for high bond strength.

The Swiss Association for Protection against Corrosion (SGK) was given the assignment of evaluating the corrosion behavior of fastenings post-installed in concrete using the Hilti HIT-HY 200 and Hilti HIT-RE 500 injection systems.

Corrosion tests were carried out. The behavior of the two systems had to be evaluated in relation to their use in field practice and compared with the behavior of cast-in reinforcement.

The result can be summarized as follows:

### Hilti HIT-HY 200

- The Hilti HIT-HY 200 systems in combination with reinforcing bars can be considered resistant to corrosion when they are used in sound, alkaline concrete. The alkalinity of the adhesive mortar safeguards the initial passivation of the steel. Owing to the porosity of the adhesive mortar, an exchange takes place with the alkaline pore solution of the concrete.
- If rebar are bonded-in into chloride-free concrete using this system, in the event of later chloride exposure, the rates of corrosion are about half those of rebar that are cast-in.
- In concrete containing chlorides, the corrosion behavior of the system corresponds to that of cast-in rebar. Consequently, the use of unprotected steel in concrete exposed to chlorides in the past or possibly in the future is not recommended because corrosion must be expected after only short exposure times.

### Hilti HIT-RE 500 V3

- If the Hilti HIT-RE 500 V3 system is used in corrosive surroundings, a sufficiently thick coat of adhesive significantly increases the time before corrosion starts to attack the bonded-in steel.
- The HIT-RE 500 V3 system may be described as resistant to corrosion, even in concrete that is carbonated and contains chlorides, if a coat thickness of at least 1 mm can be

ensured. In this case, the unprotected steel in the concrete joint and in the new concrete is critical.

- If the coat thickness is not ensured, the HIT-RE 500 V3 system may be used only in sound concrete. A rebar may then also be in contact with the wall of the drilled hole. At these points, the steel behaves as though it has a thin coating of epoxy resin.
- In none of the cases investigated did previously rusted steel (without chlorides) show signs of an attack by corrosion, even in concrete containing chlorides.
- Neither during this study an acceleration of corrosion was found at defective points in the adhesive nor was there any reference to this in literature. Even if a macro-element forms, the high resistance to it spreading inhibits a locally increased rate of corrosion.
- Information in reference data corresponds with the results of this study.

### 7.7 Design Programme PROFIS Rebar

The PROFIS Rebar™ design programme allows rapid and safe design of post-installed reinforcement connections taking account of all the parameters discussed in this guide.

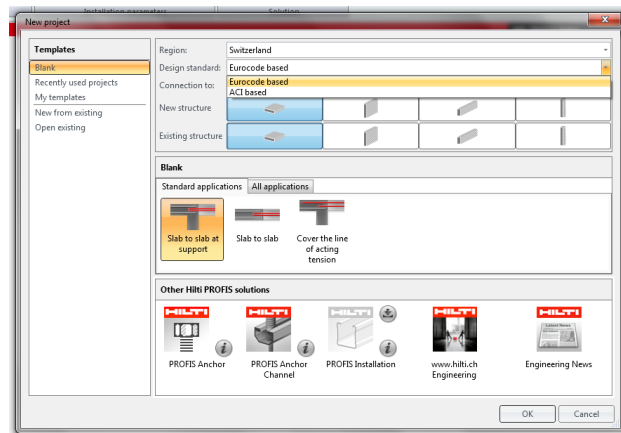


Figure 51 – PROFIS Rebar™

When a new project is opened, the user selects between the design methods “Eurocode2 [5] based” and “ACI based” design methods and the type of application. “Standard applications” covers the range of application according to TR023, “All applications” covers additional applications out of the scope of TR023, as moment resisting node whereby Hilti provides a design solution. After the correct application has been

chosen, the necessary data concerning existing and new structure have to be defined.

The results pane to the right of the drawing lets the user switch between the methods “EC / ETA” and “Hilti Rebar Design” (see section 6).

In the left hand ribbon of the screen, the user can select the adhesive to be used and either the bar size or the spacing for top and bottom layers. Based on the input data, the program calculates the section forces in steel and concrete as well as the position of the neutral axis. (Elastic-plastic behaviour of the steel is assumed, strain hardening is not taken into account.) The report summarizes the sequence of steps and the parameters/factors/formulas applied for the calculation of the design solution.

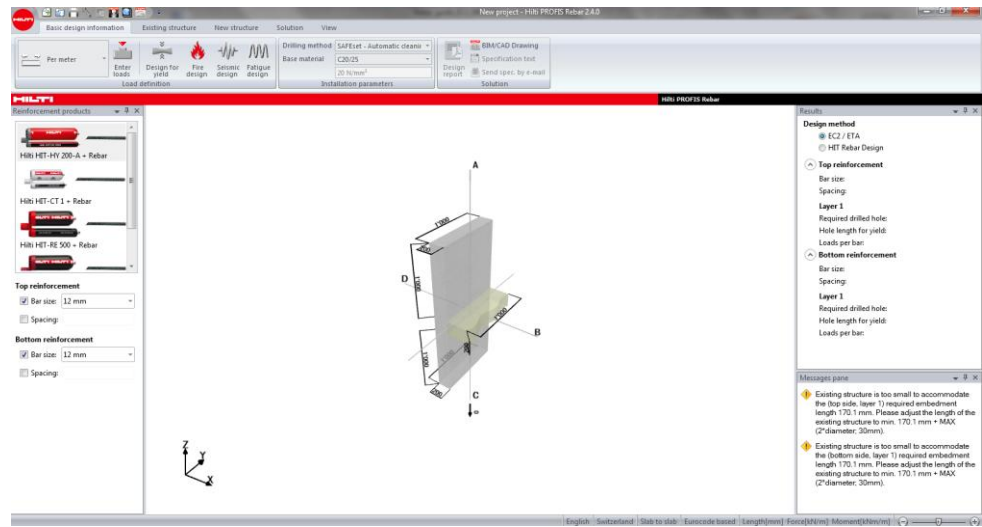


Figure 52 – PROFIS Rebar™

Contact Hilti technical staff for additional guidance on this topic.

## 8. References and suggestions for further reading

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