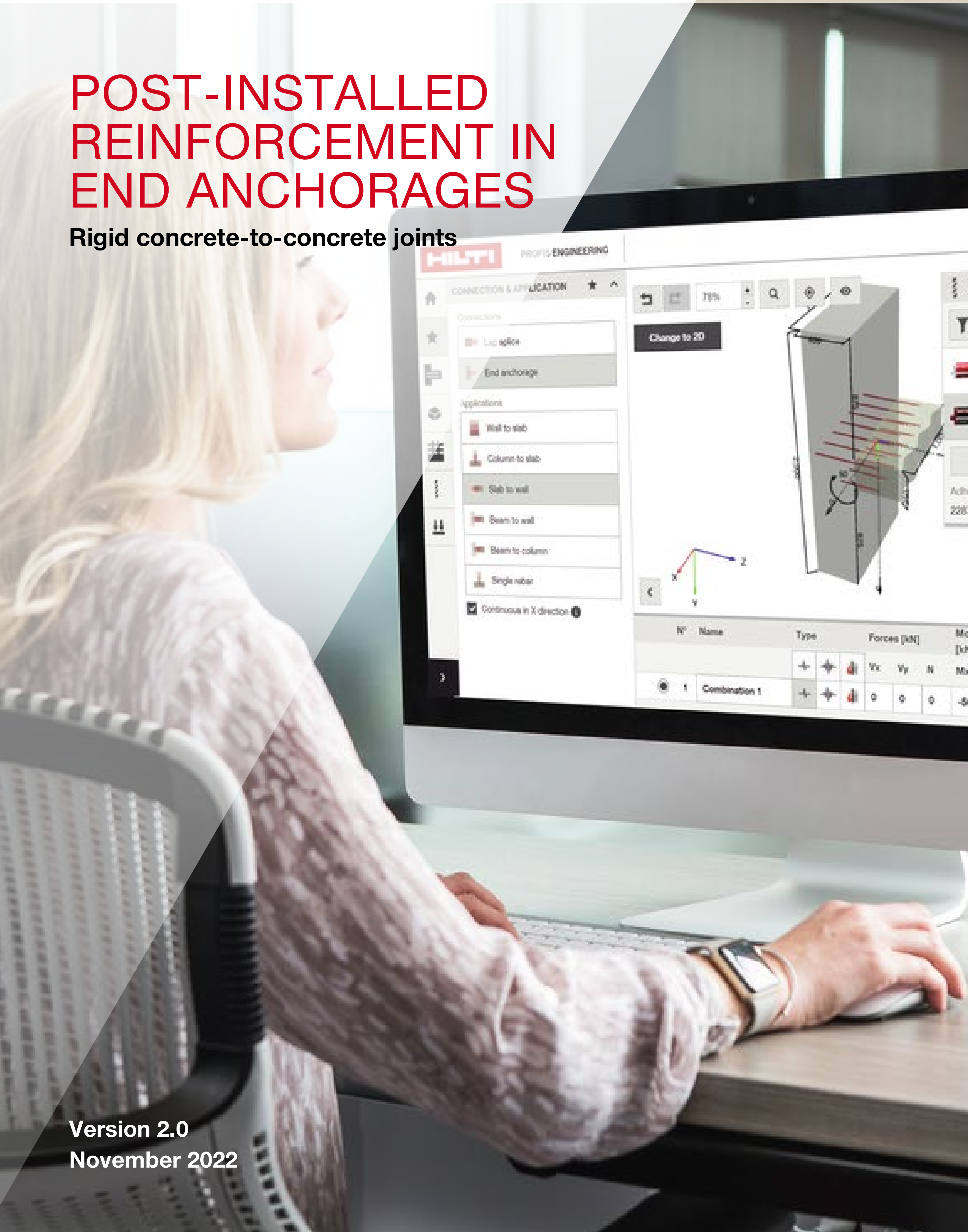
# POST-INSTALLED REINFORCEMENT IN END ANCHORAGES

**Rigid concrete-to-concrete joints**

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# 1. ABSTRACT

Connecting structural concrete members with post-installed rebar (PIR) is today a trusted and reliable solution, especially considering the significant advances in the assessment and qualification of PIR solutions during the past 15 years.

Until recently, post-installed rebar connections assessed using European Assessment Document (EAD) 330087 [1] could only be executed with straight rebar, which is permitted in accordance with EN 1992-1-1 [2] (for static loading) or EN 1998-1 [3] (for seismic action). This means that EAD 330087 [1] was limited to simply supported end anchorages or splice connections. Furthermore, the bond strength of high-performance adhesive was limited to cast-in bars. This was because assessment using the provisions of EAD 330087 is based on the demonstration of the equivalence with the bond strength of cast-in bars. The only mechanism through which rigid, moment-resisting end anchorages could be addressed with EAD 330087was if the reinforcement in the existing concrete element was detailed in a manner that created an overlap with the new bars. This, however, is not feasible in many cases where advance planning is required. Alternative strut & tie methods applicable in some situations for static loading are usually not valid if the connection must resist seismic actions.

Late 2019 saw the introduction of a new qualification EAD 332402 [4] and the accompanying design method TR 069 [5] valid for static loading. In 2020 and 2021 both EAD and TR were updated to consider design working life up to 100 years and seismic actions, respectively. This enabled, for the first time, regulation of the assessment, design, and execution of PIR connections at a European level considering their product dependent performance. This article introduces the new updated EAD 332402 [4], [6], [7] qualification procedure and the Technical Report (TR) 069 design method. Furthermore, this article briefly explains the relevance of installation of mortar systems that have a direct impact on design assumptions and vice versa.

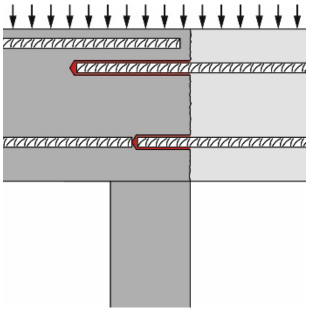
# 2. THE STATUS QUO: AN OVERVIEW ON THE ASSESSMENT & DESIGN OF POST-INSTALLED REBAR

Over the past decades, connecting structural concrete members with post-installed rebar (PIR) connections has gradually become a widespread practice in the global construction industry and is used for both new and existing construction. In 2006, the European Organisation for Technical Assessment (EOTA) published the first assessment for Post-installed Rebar – Technical Report 023 [8] – which was superseded in 2018 by the European Assessment Document (EAD) 330087 [1].

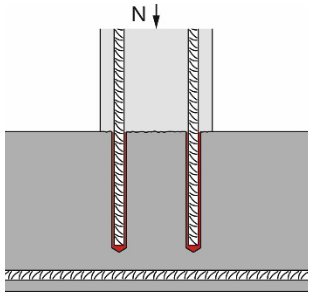
This EAD introduced the criteria for assessing PIR systems to establish their equivalence to cast-in rebars (CIR) in terms of load-displacement behavior, bond-splitting resistance, and robustness under differing installation, environmental, and loading conditions such as static, seismic, and fire exposure.

An EAD 330087-qualified mortar system can thus be designed using the straight bar anchorage length design provisions of EN 1992-1-1 [2], i.e. mainly for simply supported or compression-only end anchorages and splices, as illustrated by Figure 1.

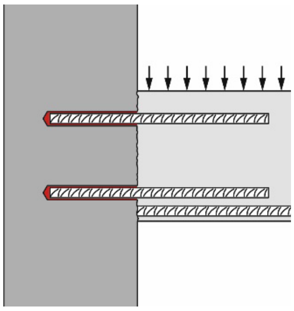
**Figure 1**
Execution of post-installed moment resisting connection of slab-to-slab and column or wall by splicing as required by EAD 330087 [1]



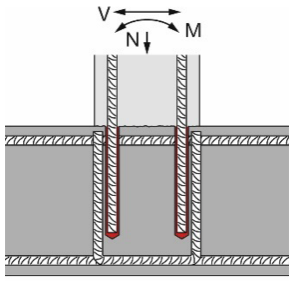
a) **Overlap joint for rebar connections of slabs and beams**




b) **Components stressed primarily in compression**



c) **End anchoring of slabs or beams (simply supported)**



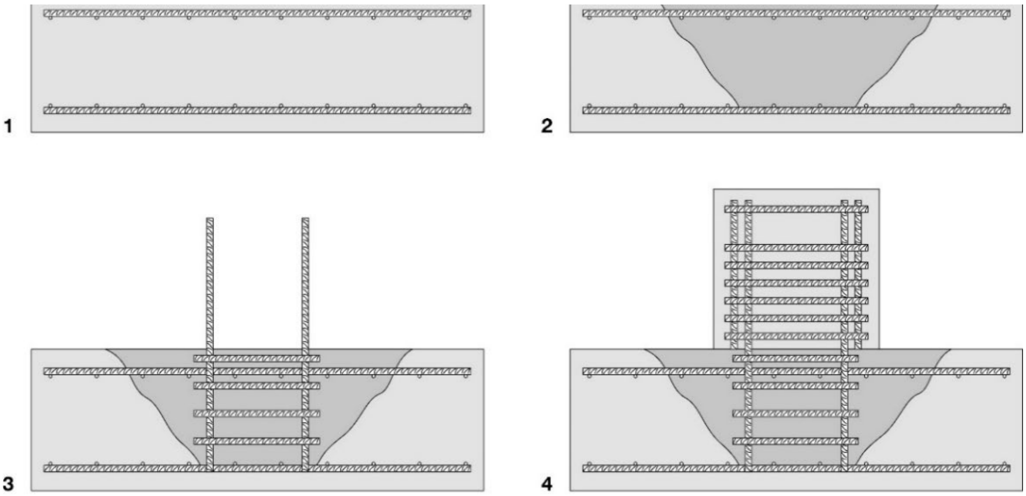

d) **Overlap joint at a foundation of a column or wall**

Once designed, installation of post-installed bars introduces changes to the construction workflow, its economics and its safety measures when compared to laying cast-in bars. This is particularly so if the requirement of post-installed bars is unplanned and arises from errors in execution or changes in design. Additional steps and competencies are required from installers to ensure design translates into appropriate installations on site. A major challenge for designers and contractors is accommodating the anchorage length of straight post-installed bars within the existing section's thickness. For splices, it is possible to use post-installed bars despite deep anchorage lengths, provided the existing section is sufficiently thick to accommodate the required anchorage length. Ensuring drilling remains perpendicular to the surface is the biggest challenge that can be overcome through use of drilling aids.

For end anchorages, EN 1992-1-1 usually covers such connections to resist bending if overlap bars are provided in a manner illustrated by Figure 1(d), otherwise these connections may only transfer shear (i.e., they are simply supported as in Figure 1(c)). Alternative methods, such as the strut & tie model of the anchorage region require deep knowledge of the forces in the connections and of the existing reinforcement. As a result, they are usually validated for specific geometries under static loading only, meaning their applicability is limited. A structural engineer wishing to transfer moment and comply with EN 1992-1-1 for a new construction must undertake additional planning to both provide and position the appropriate reinforcement at the right location in the existing member to accommodate the new post-installed bars. On the jobsite, this translates to exposed bars that often impede access and disrupt workflow in already congested spaces, leading to heightened safety risks. The exposed bars also run the risk of damage from construction equipment. In existing construction where these overlap bars are unavailable, transferring bending from the new to existing concrete may require partial demolition to accommodate new bars, as illustrated by Figure 2 [9]. In both situations, execution on-site is hampered and prone to further errors, so the use of overlap bars is best avoided.

One additional limitation of the assessment of post-installed rebar systems following the provisions of EAD 330087, is the limitation of the design bond strength, $f_{bd}$, used in calculating the anchorage length to that of cast-in bars. For example, the system must achieve a mean bond strength of 10 MPa that corresponds to a design bond strength, $f_{bd}$, of at least 2.3 MPa in C20/25 concrete, but a superior performance is not taken into account. The bond strength of industry-leading mortars used in post-installed rebar connections far exceeds that of cast-in bars. Limitations of EAD 330087 hampered the usage of more realistic performance in well-confined concrete, oftentimes causing an unfeasibility of post-installed end anchorages. (i.e., the drilling depth often exceeding the existing concrete's thickness).

**Figure 2**
Typical process to demolish existing member to place a new reinforcement system followed by concrete cast-in [9]

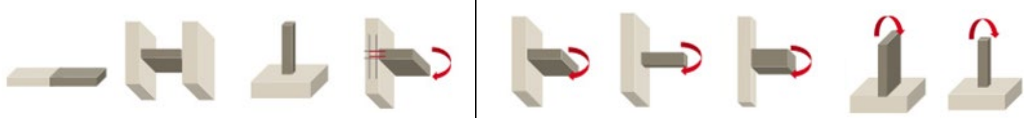



The consequence of EAD 330087 being unable to realistically assess the behavior of moment-resisting end anchorages while considering the bond-splitting performance of mortar in unconfined concrete necessitated a large experimental campaign in the late-2010s [10] [11], culminating in a new European Assessment Document (EAD) 332402 [4] and the accompanying Technical Report TR069 [5] published by EOTA.

## 3. A BREAKTHROUGH IN PIR: NEW ASSESSMENT & DESIGN CONCEPT WITH EAD 332402 & TR 069

First published in 2019 and then updated in 2020 and 2021, the new Technical Report EOTA TR069 [5] entitled "*Design method for anchorages of post-installed reinforcing bars (rebar) with improved bond-splitting behavior as compared to EN 1992-1-1*" allows for the design of post-installed, moment-resisting reinforced concrete connections under static and quasi-static loading conditions without using an overlap splice configuration, for static and seismic loading. Design with TR 069 requires that the mortar system is assessed following the requirements of EAD 332402 [4], [6], [7] and the subsequently published ETA contains specific factors unique to the mortar system that govern the bond-splitting behavior of the moment-resisting connections illustrated in Table 1.

**Table 1**
Extension in scope of post-installed reinforcement connection with EOTA TR069 [5] design method.

| | Splices / Simply supported / Compression load only / Rigid connection with overlap | | | Rigid connection (without overlapping) | |
|---|---|---|---|---|---|
| |  | | | | |
| **Load** | **Static** | **Fire** | **Seismic** | **Static** | **Seismic** |
| **Product qualification** | EAD 330087 | | | EAD 332402 | |
| **Technical data** | ETA I | | | ETA II | |
| **Design method** | EC2-1-1 | EC2-1-2 | EC8-1 | TR069 | |

## 3.1 Overview of EAD 332402 – "Post-installed reinforcing bar (rebar) connections with improved bond-splitting behavior under static loading"

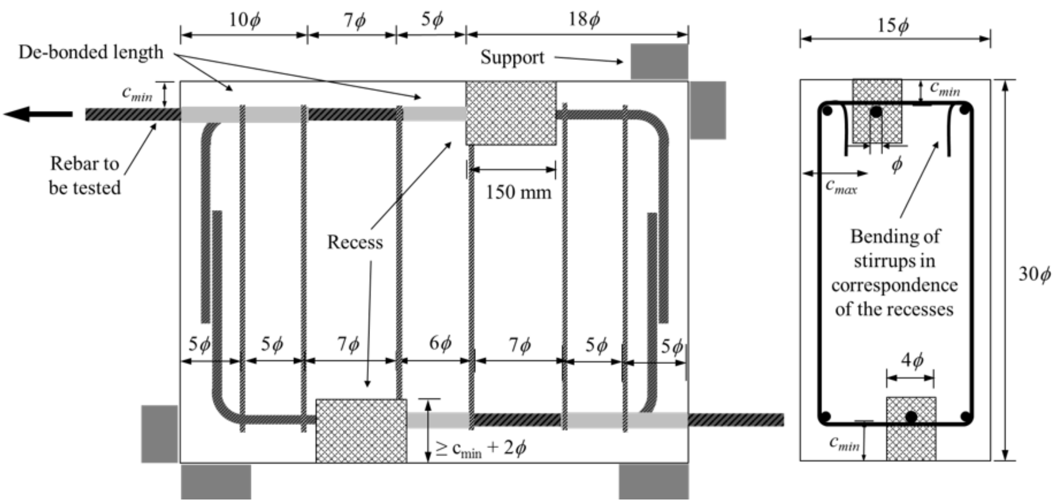
While EAD 330087 [1] assesses the equivalence of mortar system to cast-in-situ concrete, the new EAD 332402 [4] goes a step beyond and enables a product-dependent assessment of the bond-splitting resistance of the post-installed rebar system when used with EOTA TR 069 [5].

Although a "stand-alone" qualification document, EAD 332402 builds upon specific aspects of two existing EADs: 330087 (for equivalence to cast-in, covered in Section 2 of this document) & 330499 (bonded anchors) [12]. The first one solely assesses equivalence to cast-in bars, meaning a mortar must first be qualified using both these documents prior to an assessment with EAD 332402. The latter does not cover small edge distances or spacing and anchorages beyond 20 times the bar diameter but does provide the characteristic pull-out resistance of the mortar that forms the upper limit of the bond-splitting resistance.

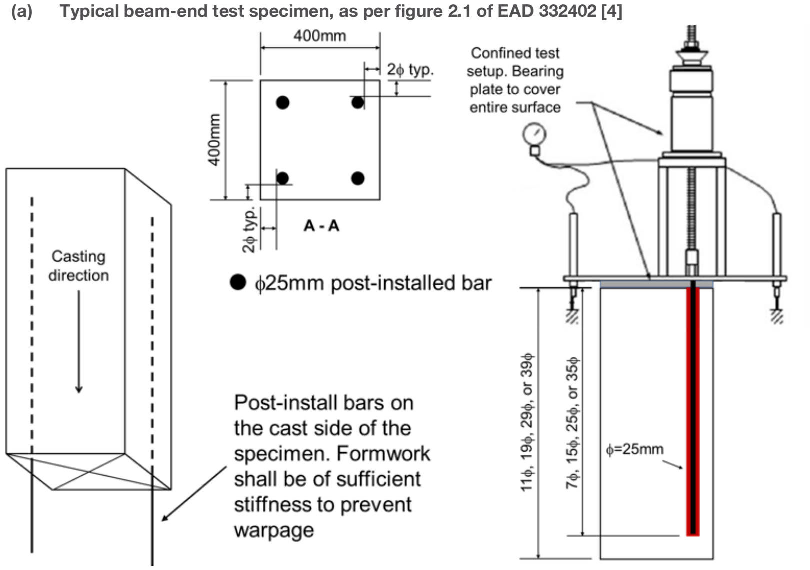
EAD 332402 implements a novel beam-end test (BET) specimen (see Figure 3(a)) to determine the bond-slip curve from the *fib* Model Code 2010 [13] for post-installed bars using mortars. This BET specimen simulates unconfined stress conditions, resulting in a compression and tension zone comparable to a typical beam under bending, being post-installed rebar loaded in tension with small edge distance. When compared with the pull-out specimen from EAD 330087 that uses a **confined** setup to prevent cone breakout, large concrete cover to all edges and large confinement, this setup enables a comprehensive evaluation of bond-splitting behavior under realistic boundary conditions in which concrete is **unconfined**, cover is small and not uniform to different edges, and the layout of the transverse reinforcement is included.

Figure 3(a) illustrates a representative BET specimen designed to evaluate the **influence** of a specific mortar on the bond-splitting behavior of the concrete, resulting in the following mortar-specific calibration factors that influence the characteristic bond-splitting resistance. The second important test specimen and setup is the bond-splitting test (see Figure 3(b)). A prismatic unreinforced specimen is used to investigate the influence of anchorage length lb on the splitting resistance and to derive the product dependent parameter lb1 that analytically describes the bond-strength degradation as a function of the anchorage length.

**Figure 3**
Test specimens from EAD 332402 [4]: a) Beam-end test (BET); b) Prismatic bond-splitting test




**(a) Typical beam-end test specimen, as per figure 2.1 of EAD 332402 [4]**

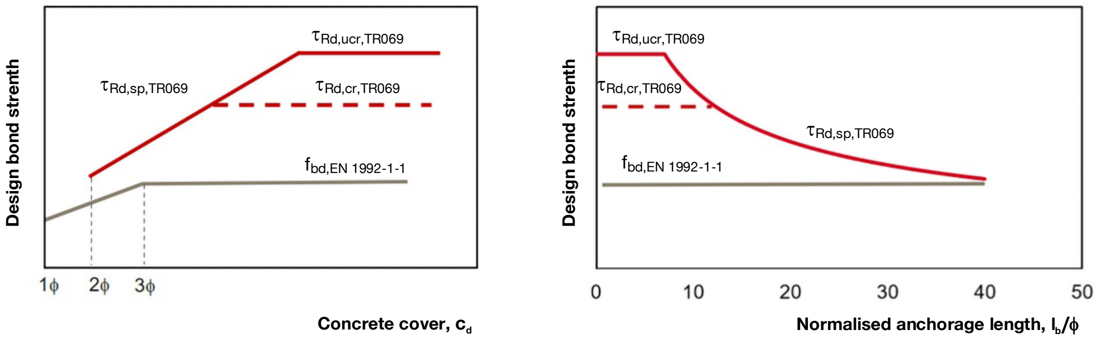



**(b) Typical specimen for bond-splitting test, as per figure 2.2 of EAD 332402 [4]**

The result of this approach represents a shift in design methods where using a different mortar will result in a different bond-splitting resistance and anchorage length, an approach that has many similarities to the design of post-installed bonded anchors. Figure 4(a) and (b) show a qualitative comparison of the bond-splitting resistance when the design methods TR 069 and EN 1992-1-1 [2] are respectively employed. The solid and dashed red lines for uncracked and cracked concrete illustrated by Figure 4(a) represent the bond stress in relation to the confinement by cover (the minimum cover-to-bar diameter) and demonstrate the increase in splitting bond stress until it reaches the pull-out resistance of the mortar, a feature not possible with the EAD 330087 and EN 1992-1-1 design approach, which does not permit increase on bond stress beyond a confinement limit of cover being three-times the bar diameter.

Additionally, concrete design standards such as EN 1992-1-1 use the “uniform bond model” that uses a mean bond stress along the entire length of the bar to simplify the design procedure for cast-in bars with post-installed bars also following suit. However, bond stress is known to degrade with increasing anchorage length and Figure 4(b) represents this scenario where the EN 1992-1-1 line represents the uniform bond model and the solid and dashed red lines represent the non-linear degradation of bond stress with TR 069.

**Figure 4**
Qualitative comparison of the bond-splitting resistance of a system [14] evaluated according to EAD 332402 [4] and EOTA TR069 [5] with a system according to EAD 330087 [1] and EN 1992-1-1 [2]




(a) Bond stress curve dependence on concrete cover (b) Bond stress curve dependence on anchorage length

## 3.2 Design concept of TR069 – "Design method for post-installed reinforcing bars (rebars) with improved bond-splitting behavior as compared to EN 1992-1-1"

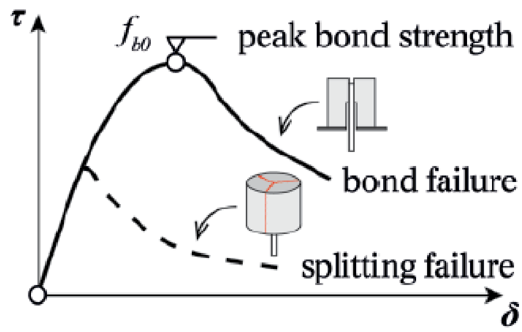
Structural concrete relies on the force transfer between reinforcement and concrete – denoted by "bond" and "anchorage" – which governs the behavior at the serviceability (SLS) and ultimate limit states (ULS). Fundamentally, however, bond is not a property of the bar; rather, the geometry of the bar and concrete section, material characteristics, stress state and the surface characteristics of the steel all combine to influence bond and, by extension, structural concrete [15].

For cast-in and post-installed bars, a bond failure may occur in one of the following ways:

1. **Splitting** – relying on the concrete's tensile strength as well as adhesion and friction at the interface, splitting is typified by cracks forming along the concrete's surface parallel to the bar. Radial, or hoop, stresses give rise to such cracks that develop at the surface of the nearest edge along the length of the bar and may cause the concrete to spall if the stress is sufficiently high and confinement low. Therefore, increasing the confinement, particularly through increasing concrete cover and spacing, in turn increases the splitting resistance.
2. **Pull-out –** with increasing confinement and stress, the force transfer increasingly relies on the ribs of the bar bearing against the concrete. Failure is marked when the force in the bar exceeds the concrete's shear strength, resulting in the bar shearing off along the tops of the ribs.

Figure 5 typifies the concept of splitting being weaker than pull-out.

**Figure 5**
Bond-slip laws for pull-out (solid line) and splitting (dashed line) failures [16].




Design provisions such as those in EN 1992-1-1 [2] and other standards specifically limit the positive impact of higher confinement. However, these provisions assume that concrete will not be subject to direct tension, as is the case for a cast-in or post-installed anchors, due to sufficient confinement provided by the concrete strength, clear cover and spacing, transverse reinforcement, bar diameter and geometry, pressure perpendicular to the bar axis (lateral pressure), or a combination of these. This confined model also ensures that any formation of concrete cone does not occur as the bars are embedded at sufficient depth to preclude such a failure.

TR 069 [5] represents a shift in the design approach to engage the higher strength of modern mortars used with post-installed bars, thereby allowing a more realistic and detailed evaluation of bond-splitting resistances. When coupled with the mortar-specific calibration factors from the ETA according to the EAD 332402 [4] assessment, the design method allows a strength assessment of post-installed bars in unconfined concrete – i.e. concrete subject to direct tension caused by a bending moment – while engaging the mortar to influence the bond-splitting resistance of the system. Many aspects of TR 069 connect to the existing understanding of bond behavior and design of structural concrete:

- The hierarchy of resistances follows limit state principles, where the weakest design resistance governs the design anchorage length.
- The detailing arrangements must still respect EN 1992-1-1 provisions.
- Roughening the contact interface between new and existing concrete must be undertaken to transfer shear in line with Section 6.2.5 of EN 1992-1-1.
- Durability requirements must be satisfied in line with EN 1992-1-1 1.
- Verification of the existing member to withstand the loads introduced by the post-installed reinforcement following the provisions of EN 1992-1-1 1.

However, the nature of the mortar assessment, with its unconfined setup, implies design must consider the possibility of concrete cone breakout and, consequently, requires the evaluation of its resistance similar to bonded anchor design when connecting a baseplate to a concrete substrate. Here, TR 069 draws upon the logic of EN 1992-4 [17], but without limitations in anchor group configurations while maintaining that bars in structural concrete are not subject to direct shear.

## 3.3 Design verification according to EOTA TR 069 under static and quasi static loading

Using the limit state design logic, TR 069 [5] requires evaluation of three design resistances using the appropriate material partial safety factors from Table 3.1 of TR 069, replicated below in Table 2. The three failure modes are:

- Design resistance to yielding of the bars ($N_{Rd,y}$), evaluated for the **highest loaded bar** in tension.
- Design resistance to concrete cone breakout ($N_{Rd,c}$), evaluated for the **group of bars** in tension.
- Design resistance to bond-splitting ($N_{Rd,sp}$), evaluated for **highest loaded bar** in tension.

The decisive design resistance, $R_d$, governing the anchorage length is provided by:

$$\boldsymbol{R_d} = \min\,(\boldsymbol{N_{Rd,y}}; \boldsymbol{N_{Rd,c}}; \boldsymbol{N_{Rd,sp}})$$

To prevent failure at the ultimate limit state, $\boldsymbol{E_d} \leq \boldsymbol{R_d}$, where $\boldsymbol{E_d}$ is the design action.

**Table 2**
Partial safety factors for different failure modes, from Table 3.1 of TR069 [5]

| Failure Modes | Partial Factor |
|---|---|
| Reinforcement Yielding | $\gamma_{Ms}$ = 1.15 |
| Concrete cone failure | $\gamma_{Mc} = \gamma_{inst} \cdot \gamma_c$<br>$\gamma_{inst} \geq 1.0$ see relevant ETA<br>$\gamma_c$ = 1.5 |
| Bond failure and Bond-splitting failure | $\gamma_{Mp} = \gamma_{Msp} = \gamma_{Mc}$ |

### 3.3.1 Design resistance to yielding (Section 4.2, TR 069):

Resistance to yielding of the bars is based on the bar diameter and yield strength of the bar using the following relationship:

$$N_{Rd,y} = \frac{N_{Rk,y}}{\gamma_{Ms}} = \frac{A_s f_{yk}}{\gamma_{Ms}}$$

Where:

$A_s$ – cross-sectional area of highest loaded bar in tension

$f_{yk}$ – characteristic yield strength of the bar

$\gamma_{Ms}$ – See Table 2

### 3.3.2 Design concrete cone breakout resistance (Section 4.3, TR 069)

As elaborated in earlier sections of this paper, evaluating the resistance of concrete breakout is necessary considering the assumption that concrete will be subject to direction tension. Evaluating the resistance is similar to EN 1992-4 [17] for bonded anchors:

$$N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{Mc}}$$

with:

$\gamma_{Mc}$ = See Table 2 of this paper.

$$N_{Rk,c} = \left(k_1 \cdot \sqrt{f_{ck}} \cdot l_b^{1.5}\right) \cdot \frac{A_{c,N}}{A_{c,N}^0} \cdot \psi_{s,N} \cdot \psi_{ec,N} \cdot \psi_{re,N} \cdot \psi_{M,N}$$

*Where:*

$k_1$ = 7.7 or 11.0 for cracked or uncracked concrete, respectively, from the ETA

$f_{ck}$ = concrete compressive strength (in cylinder)

$l_b$ = anchorage length of the bar

$A_{c,N}/A_{c,N}^0$ = factor for geometric effect of axial spacing and edge distance

$\psi_{s,N}$ = factor for the disturbance of the distribution of stresses in the concrete due to the proximity of an edge of the concrete member

= $0.7 + 0.3\frac{c}{c_{cr,N}} \leq 1.0$

$\psi_{re,N}$ = effect of dense reinforcement in existing concrete when anchorage, $l_b$, is less than 100mm

= $0.5 + \frac{l_b}{200} \leq 1$

$\psi_{ec,N}$ = for the effect of tension acting eccentric to the group of bars.

= $\frac{1}{1+2e_N/s_{cr.N}} \leq 1$

$\psi_{M,N}$ = the positive effect of a compression force between fixture and concrete in cases of bending moments, with or without axial force. and is expressed in below equation:

= $2 - \frac{z}{1.5l_b} \geq 1$, where *z* represents the lever arm.

A lack of experimental evidence suggests that $\psi_{M,N}$ should be assumed as 1.0 when the concrete cover is less than 1.5 times the anchorage length (i.e., near edge conditions) [18].

### 3.3.3 Design bond splitting resistance (Section 4.4, TR 069)

The design bond-splitting resistance, $N_{Rd,sp}$, is based on the analytical formulation found in the *fib* Model Code 2010 [13] and it is function of several parameters as explained in the following:

$$N_{Rk,sp} = \frac{\boldsymbol{\tau_{Rk,sp}} \cdot l_b \cdot \phi \cdot \pi}{\gamma_{Mp}}$$

$$\boldsymbol{\tau_{Rk,sp}} = \boldsymbol{\eta_1} \cdot \boldsymbol{A_k} \cdot \left(\frac{\boldsymbol{f_{ck}}}{\mathbf{25}}\right)^{sp1} \cdot \left(\frac{\mathbf{25}}{\boldsymbol{\phi}}\right)^{sp2} \cdot \left[\left(\frac{\boldsymbol{c_d}}{\boldsymbol{\phi}}\right)^{sp3} \cdot \left(\frac{\boldsymbol{c_{max}}}{\boldsymbol{c_d}}\right)^{sp4} + \boldsymbol{k_m K_{tr}}\right] \cdot \left(\frac{\mathbf{7}\boldsymbol{\phi}}{\boldsymbol{l_b}}\right)^{lb1} \cdot \boldsymbol{\Omega_{p,tr}}$$

The equation for $\boldsymbol{\tau_{Rk,sp}}$ models the influence of the mortar on concrete strength, bar diameter, minimum and maximum cover, transverse reinforcement, and transverse pressure (this pressure applies only to uncracked concrete), and anchorage length, thereby providing the bond-splitting strength ($\tau_{Rk,sp}$). The splitting bond stress then is limited by the pull-out resistance of the mortar influenced by cracks and anchorage length as shown below:

$$\tau_{Rk,sp} \leq \boldsymbol{\tau_{Rk,ucr}} \cdot \boldsymbol{\Omega_{cr,03}}(\boldsymbol{or}\ \boldsymbol{\Omega_{p,tr}}) \cdot \boldsymbol{\psi_{sus}} \qquad \rightarrow for\ 7\phi \leq l_b \leq 20\phi$$

$$\tau_{Rk,sp} \leq \boldsymbol{\tau_{Rk,ucr}} \cdot \left(\frac{\mathbf{20}\boldsymbol{\phi}}{\boldsymbol{l_b}}\right)^{lb1} \cdot \boldsymbol{\Omega_{cr,03}}(\boldsymbol{or}\ \boldsymbol{\Omega_{p,tr}}) \cdot \boldsymbol{\psi_{sus}} \quad \rightarrow for\ l_b \geq 20\phi$$

In the above equations:

$\boldsymbol{\gamma_{Mc}}$ = see Table 2 of this paper.

$\boldsymbol{A_k}$ = basic mortar fitting parameter, evaluated by the EAD 332402 [4] & published in the ETA, represents the basic splitting resistance.

$\boldsymbol{\eta_1}$ = from EN 1992-1-1, this factor accounts for the quality of good (1.0) or poor (0.7) bond conditions and bar position while casting the new section. For post-installed bars, the same rules as for cast-in bars may be conservatively applied.

$(\boldsymbol{f_{ck}}/\mathbf{25})^{sp1}$ with sp1 from mortar's ETA = the combined term accounts for the influence of the concrete on the bond-splitting strength of the mortar.

$(\mathbf{25}/\boldsymbol{\phi})^{sp2}$ with sp2 from the mortar's ETA = the combined term accounts for the diameter-dependent size effect on the splitting bond strength.

$(\boldsymbol{c_d}/\boldsymbol{\phi})^{sp3}$ with sp3 from the mortar's ETA = the combined term accounts for the influence of mortar on confinement from small concrete covers. Similar to EN 1992-1-1 1 [2], the minimum cover, $c_d$, is lowest of the cover to the nearest edge and half the clear spacing between the bars. The ETA also sets the minimum concrete cover to be not less than $2\phi$ and the design equation sets $\phi$ as 12mm in the denominator when using bar sizes less than 12mm.

$(\boldsymbol{c_{max}}/\boldsymbol{c_d})^{sp4}$ with sp4 from the mortar's ETA = the ratio of the largest ($c_{max}$) to the smallest cover ($c_d$) accounts for the influence of the mortar on confinement from large concrete covers. $c_{max}$ is the larger of the cover to the farthest edge and half the bar spacing. Smaller ratios of $c_{max}$ & $c_d$ represent bars positioned near corners where low confinement from cover reduces the splitting bond resistance. While the lower limit cannot be lower than 1.0, TR 069 sets the upper limit of $c_{max}/c_d$ as 3.5.

$\boldsymbol{k_m K_{tr}}$ = the combination of $\boldsymbol{k_m}$ and $\boldsymbol{K_{tr}}$ highlights the positive impact of transverse reinforcement on splitting by increasing ductility. While $\boldsymbol{K_{tr}}$ (the amount of reinforcement crossing a potential splitting surface) is limited to an upper value of 0.05 by TR 069 and EN 1992-1-1, $\boldsymbol{k_m}$ can only take values of 0, 6, or 12 based on the effectiveness of transverse reinforcement. In most of the cases for anchorages such as connecting a new beam to an existing column, transverse reinforcement in the existing concrete column will be parallel to the post-installed longitudinal bars of the new beam, and therefore cannot improve the splitting bond resistance, hence this combined term may be usually ignored in design.

$(7\phi/l_b)^{lb1}$ = covered in earlier sections and by Figure 5, the **splitting** bond strength of the mortar degrades with increasing anchorage length, and the factor lb1 – from the mortar's ETA – influences this degradation. Since all design anchorage lengths, $l_b$, will exceed $7\phi$ as required by the minimum anchorage length rules in Section 3.4 of this paper, a mortar with a lower lb1 factor proves beneficial for deeper anchorages ($l_b >> 7\phi l_b$).

$(20\phi/l_b)^{lb1}$ = similar to the reduction in splitting bond strength, **pull-out** bond strength of the mortar also declines in a non-linear manner with increasing anchorage length, but this effect only becomes noticeable at anchorages beyond $20\phi$ in combination with large concrete cover & spacings. The same factor lb1 from the mortar's ETA further influences this degradation and wherever $l_b > 20\phi$ mortars with a lower lb1 factor slightly reduce the combined term.

$\psi_{sus}$ = applied to the pull-out resistance, the sustained load ratio accounts for the influence of creep and relaxation on the mortar caused by sustained tension. Since post-installed bars are always under tension and the portion of permanent loads is usually significant, mortar-specific sustained load factor ($\psi^0_{sus}$) will reduce the pull-out resistance. Therefore, as with bonded anchors, a sustained load factor close to unity is beneficial.

$\Omega_{cr,03}$= the presence of cracks parallel to the bond length of the post-installed bar reduce the pull-out resistance of the mortar and this factor, alongside the sustained load ratio, reduces the upper limit of the bond-splitting resistance. This value is taken from relevant ETA. The dashed red lines in Figure 4 demonstrate schematically this reduced upper limit, and this is diameter- and mortar-dependent

$\Omega_{p,tr}$ = in instances of concrete where cracks in concrete have not yet arisen or can be proven never to arise during the structure's service life, a compressive transverse pressure contributes to a higher bond-splitting resistance, or a lower resistance if the transverse pressure is tensile. The pressure is based on compressive or tensile stress in concrete perpendicular to the bar axis averaged over a volume of 3ϕ around the bar.

## 3.4 Design verification according to EOTA TR 069 under seismic action

Under seismic action the same failure modes as under static loading need to be verified, i.e. steel yielding, concrete cone breakout and bond-splitting. However, following the principles of EN 1998-1 [3] for the majority of the applications the decisive seismic design resistance, $R_{d,eq}$ shall correspond to steel yielding. Thus, the following equation must be satisfied:

$$R_d = N_{Rd,y,eq} \leq \min\left(N_{Rd,c,eq}; N_{Rd,sp,eq}\right)$$

$N_{Rd,c,eq}$ or $N_{Rd,sp,eq}$ may be also be acceptable as decisive failure modes if the predicted plastic mechanism of the structural system is ductile at a demand level, at which the connection with post-installed rebars designed according to TR069 [5] is still elastic.

Under seismic action the crack width should be evaluated as it can be significantly larger than in static loading. The assessment of the crack width to be expected in a specific connection under seismic actions should be carried out by the designer. Principles to estimate the crack width are related to the deformability of the connection (i.e., ductility class and behavior factor), the ratio between anchorage length and member thickness and the level of compression in the existing element. TR069 gives some recommendations in table 3.6.1, replicated below in Table 3. Crack width influences the concrete breakout cone resistance through $\alpha_{eq}$ depending on $w_k$ and the bond splitting resistance through $\alpha_{cr,eq}$ factor depending also on $w_k$ as described in the next chapters.

In addition to the provisions of the EOTA TR 069 specific requirements of EN 1998-1 for different type of connections (e.g., in terms of reinforcement detailing) apply.

**Table 3**
Partial safety factors for different failure modes, from Table 3.6.1 of TR069 [5]

| Ductility class according to EN 1998-1 | Behaviour factor, $q$ according to EN 1998-1 | $l_b$ / $h$ [-] | Assumed crack width, $w_k$ [mm] | Comment |
|---|---|---|---|---|
| DCL | 1,0 | All | 0,3 | Static design applies |
| DCM | 1,0 – 1,5 | ≥ 0,8 | 0,3 | |
| | | < 0,8 | 0,5 | |
| DCM / DCH | 1,5 – 3,0 | ≥ 0,8 | 0,5 | |
| | | < 0,8 | 0,8 | |
| DCM / DCH | > 3,0 | ≥ 0,8 | 0,8 | |
| | | < 0,8 | Not covered by this TR | |

### 3.4.1 Seismic design resistance to yielding (Section 5.2, TR 069):

Seismic resistance to yielding of the bars is based on the bar diameter and yield strength of the bar using the following relationship:

$$N_{Rd,y,eq} = \gamma_{Rd} \frac{N_{Rk,y}}{\gamma_{Ms}}$$

Where:

$N_{Rk,y}$ – calculated as per Section 3.3.1 of this paper

$\gamma_{Ms}$ – as per Table 2 of this paper.

$\gamma_{Rd} \geq 1.0$

The value of the overstrength factor $\gamma_{Rd}$ should be related to level of ductility for which the connection is designed.

The values 1.0 and 1.2 for DCM and DCH, respectively, can be used in accordance with the provisions of EN 1998-1, Cl. 5.6.2.2.

### 3.4.1 Seismic design concrete cone breakout resistance (Section 5.3, TR 069)

The seismic resistance corresponding to concrete cone failure is calculated as per:

$$N_{Rd,c,eq} = \alpha_{eq} \cdot \frac{N_{Rk,c}}{\gamma_{Mc}}$$

Where:

$\alpha_{eq}$ = 1.0 if $w_k$ = 0.3 mm or uncracked concrete or 0.85 if $w_k$ = 0.5 mm or 0.8 mm

$N_{Rk,c}$ calculated as per Section 3.3.2 of this paper

$\gamma_{Mc}$ – as per Table 2 of this paper.

### 3.4.1 Seismic design bond splitting resistance (Section 5.4, TR 069)

The design bond-splitting resistance, $N_{Rd,sp,eq}$, is in principle the same formulation as used for static loading (see Section 3.3.3). However, the sensitivity of the mortar to cyclic loading as quantified in the assessment according to the EAD 332402-00-0601-v02 [7] for the failure modes splitting ($\alpha_{eq,sp} \leq 1.0$) and pullout ($\alpha_{eq,p} \leq 1.0$) are included in the equation to calculate the characteristic bond strength, $\tau_{Rk,sp,eq}$. Furthermore, the factor taking into account for the expected maximum crack of 0.3, 0.5 or 0.8 mm width $\Omega_{cr,eq}$ varies depending on the design assumption. All other factors in the following equation remain unchanged compared to what explained in Section 3.3.3.

$$\tau_{Rk,sp,eq} = \eta_1 \cdot \alpha_{eq,sp} \cdot A_k \cdot \left(\frac{f_{ck}}{25}\right)^{sp1} \cdot \left(\frac{25}{\phi}\right)^{sp2} \cdot \left[\left(\frac{c_d}{\phi}\right)^{sp3} \cdot \left(\frac{c_{max}}{c_d}\right)^{sp4} + k_m K_{tr}\right] \cdot \left(\frac{7\phi}{l_b}\right)^{lb1} \cdot \Omega_{p,tr}$$

$$\leq \boldsymbol{\tau_{Rk,ucr}} \cdot \boldsymbol{\Omega_{cr,eq}} \cdot \boldsymbol{\alpha_{eq,p}} \qquad \rightarrow for\ 7\phi \leq l_b \leq 20\phi$$

$$\leq \boldsymbol{\tau_{Rk,ucr}} \cdot \left(\frac{\boldsymbol{20\phi}}{\boldsymbol{l_b}}\right)^{\boldsymbol{lb1}} \cdot \boldsymbol{\Omega_{cr,eq}} \cdot \boldsymbol{\alpha_{eq,p}} \rightarrow for\ l_b \geq 20\phi$$

Where:

$\Omega_{cr,eq}$ = $\Omega_{cr,0.3}$ or $\Omega_{cr,0.5}$or $\Omega_{cr,0.8}$ depending on the design assumptions on the crack width (see Table 3 for recommended values from TR069) taken from the relevant ETA

$\alpha_{eq,sp}$ and $\alpha_{eq,sp}$ are the seismic reduction factors for splitting and pull-out, respectively, taken from relevant ETA

# 4. SIGNIFICANT ADVANTAGES WITH QUALIFIED INJECTABLE MORTARS

Hilti offers designers a choice of two high performing mortars, **HY 200R V3** and the new **RE 500 V4**, both qualified to EAD 332402 [4], [6], [7] for use in moment-resisting end anchorages designed to TR 069 [5]. Both have unique characteristics based on the mortar chemistry and perform excellently under different conditions.

- **HY 200R V3** (ETA-19/0665) is recommended where a faster curing time is dictated by overall project timelines. From a TR 069 design perspective, this mortar provides high performance in scenarios where confinement is small, under static loading
- **RE 500 V4** (ETA-20/0539) provides a longer curing time but superlative performance in the widest of project requirements. This latest mortar from Hilti outperforms HY 200R V3 in scenarios where confinement is large, under static and seismic loading.

As a note to designers, it is difficult to provide a straightforward comparison on which mortar is “always better” without a comparative design. This is because mortar-specific factors merely influence the bond-splitting resistance in the design flow. If a design is governed by steel failure or by concrete cone breakout rather than bond-splitting, then the choice of mortar matters little from a design perspective, assuming both met the initial project requirements.

Table 4 shows the comparison between HIT-RE 500 V4 and HIT-HY 200-R V3

**Table 4**
Comparison between HIT-RE 500 V4 and HIT-HY 200-R V3

| Application conditions | HIT-RE 500 V4 | HIT-HY 200-R V3 |
|---|---|---|
| Connection type | Anchorage, splice,<br>moment resisting connection | Anchorage, splice,<br>moment resisting connection |
| Load Type / Design Method/ EAD | Static, Seismic, Fire / EN 1992-1-1 / EAD 330087-01-0601<br>Static, Seismic / TR 069/ EAD 332402-00-0601-v2 | Static, Seismic, Fire / EN 1992-1-1 / EAD 330087-01-0601<br>Static / TR 069 / EAD 332402-00-0601-v1 |
| Service life | 50y / 100y | 50y / 100y |
| Rebar diameter | 8 mm – 40 mm | 8 mm – 40 mm (up to 32 mm for TR 069 design) |
| Maximum embedment depth | ≤ 3,2 m | ≤ 1 m |
| In-service temperature range | –5 °C up to 40 °C | –10 °C up to 40 °C |
| Working time | 10 min – 2 h | 6 min – 3 h |
| Curing time | 4 h – 168 h | 1 h – 20 h |
| Dry and wet drilled hole | Yes | Yes |
| Water-filled drilled hole | Yes (only TR069 design) | No |
| Hammer drilled hole | Yes | Yes |
| Diamond drilled hole | Yes | Yes (14 mm – 28 mm rebar diameter) |
| Hilti SafeSet technology with roughening tool | Yes | Yes |
| Hilti SafeSet technology with hollow drill bit HDB and Hilti vacuum | Yes | Yes |

# 5. SAFETY IN DESIGN AND EXECUTION

**PROFIS Software: planning, designing, and documenting in one tool**

Design with TR 069 [5], like bonded anchor design and unlike traditional post-installed rebar design to EN 1992-1-1 [2], is ill-suited to hand calculations. The calculation cycle starts with assuming a design anchorage length and a particular mortar system that will satisfy the cone breakout and bond-splitting resistances. A hand calculation would require several iterations to arrive at a feasible solution.

For a fast and optimized design, **Hilti PROFIS ENGINEERING** software enables engineers to design and resolve any post-installed reinforced concrete connection, from simply supported to moment-resisting to splice. **PROFIS ENGINEERING** offers you flexibility and efficiency, always according to the latest regulations and standards (EOTA TR069, EN 1992-1-1 and EN 1998-1 [3]), see Figure 6.

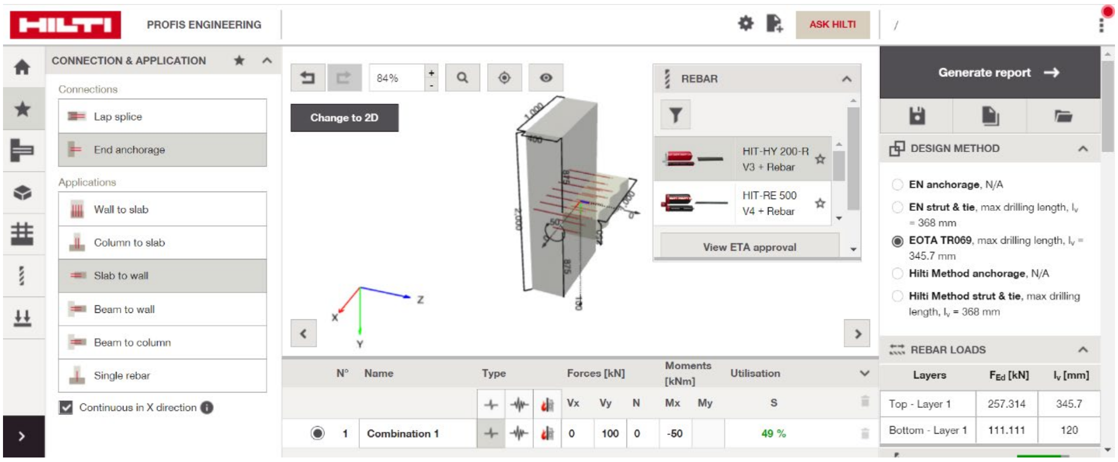



**Figure 6**

Profis Engineering, Post-installed rebar design with using TR069 method

**Figure 7**

A typical SafeSet system for concrete-to-concrete connections

**SAFESET: Consistent safety during installation**

The resistance of post-installed rebar connections is significantly influenced by the installation process. A clean borehole is key to ensuring virtually void-free installations during the adhesive injection to guarantee, in turn, that the installation behaves as designed. Inserting the rebar up to the necessary anchorage length within the working time of the mortar is another crucial factor in the installation process.

To minimize installation errors, HIT-RE 500 V4 and HIT-HY 200-R V3 injection mortars are compatible with the SafeSet system. When hammer drilling, the Hilti SafeSet system relies on hollow drill bits (HDB) connected to a vacuum cleaner (e.g., Hilti VC 40-U or VC 20-U vacuums) to drill and clean the hole in one step, minimizing the risk of dust adversely impacting the bond between the mortar and concrete. Hilti hollow-drill bits use the same state-of-the art carbide drilling technology as the Hilti TE-CX and Hilti TE-YX bits for optimal drilling performance. The Hilti SafeSet system performs equally well in dry and wet concrete and eliminates an important and time-consuming step in the installation process – cleaning the borehole before injecting the adhesive. Hilti SafeSet helps to minimize installation errors, contributing to a construction that performs as designed on the jobsite.

# 6. SUMMARY

The state-of-the-art EAD 332402 [4], [6], [7] & TR 069 [5] introduce a new assessment and design approach for post-installed bars to the industry and fill the gap of the existing EAD 330087 [1] assessment and EN 1992-1-1 [2] / EN 1998-1 [3] design standard. The ability to design moment-resisting, post-installed reinforced concrete connections while simultaneously considering the impact of the mortar system on the bond-splitting behavior is revolutionary in concept, since no two mortar systems can be considered comparable without design.

This paper gave a short glimpse inside the qualification program of the EAD 332402 and a detailed look at the design verification required by TR 069. With Hilti's PROFIS ENGINEERING software, an engineer's time is freed up to undertake several other verifications that are required beyond the anchorage length calculation – for instance, the shear transfer, stability checks for the existing member, and so on. Translating these designs into a productive and error-free installation at site is possible with the Hilti SafeSet systems.

# 7. REFERENCES

[1] EAD 330087-01-0601: Systems for post-installed rebar connections, Brussels: EOTA, 2020.

[2] EN 1992-1-1:2004: Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings, Brussels: European Committee for Standardization, 2004.

[3] EN 1998-1:2004 - Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings, Brussels: European Committee for Standardization, 2004.

[4] EAD 332402-00-0601: Post-installed reinforing bar (rebar) connections with improved bond-splitting behavior under static loading, Brussels: EOTA, 2019 (Pending for citation in OJEU).

[5] EOTA TR 069 - "Design method for anchorage of post-installed reinforcing bars (rebars) with improved bond-splitting behavior as compared to EN 1992-1-1", Brussels: EOTA, 2021.

[6] EOTA EAD 332402-00-0601-v01: Post-installed reinforcing bar (Rebar) connections with improved bond-splitting behaviour under static loading: 100 years working life, Brussels: EOTA, August 2020 (pending for citation in OJEU).

[7] EAD 332402-00-0601-v02: Post-installed reinforcing bar (rebar) connections with improved bond-splitting behaviour under static loading and seismic action, Brussels: EOTA, 2021 (Pending for citation in OJEU).

[8] EOTA TR023 Assessment of post-installed rebar connection, Brussels: EOTA, 2006 (superseded by EAD 330087).

[9] B. Campelo, J. Correia and J. Appl, "The revolution of concrete to concrete connections with post-installed rebar," *Revista Portuguesa de Engenharia de Estructuras,* vol. 3, no. 18, pp. 89-94, March 2022.

[10] J. Rex, A. Sharma and J. Hofmann, "A new test specimen to investigate the bond behavior," *Structural Concrete,* vol. 20, no. 2, pp. 583-596, 2019.

[11] V. Mahadik, A. Sharma and J. Hofmann, "Cast-in and post-installed rebar anchorage systems in RC column foundation joints: Experimental investigations," *Engineering Structures,* vol. 238, 2021.

[12] EAD 330499-01-0601: Bonded fasteners for use in concrete, Brussels: EOTA, 2018.

[13] fib, Model Code for Concrete Structures 2010, Lausanne: Ernst & Sohn, 2013.

[14] P. Wörle, J. Appl and G. Genesio, "Bewehrungsanschlüsse für momententragfähige Verbindungen nach EOTA TR 069," *Beton- und Stahlbetonbau,* vol. 115, no. 11, pp. 887-896, 2020.

[15] J. Cairns, "Bond and anchorage of embedded steel reinforcement in fib Model Code 2010," *Structural concrete,* vol. 16, no. 1, pp. 45-55, 2015.

[16] F. Brantschen, D. Faria, M. F. Ruiz and A. Muttoni, "Bond behaviour of straight, hooked, U-shaped and headed bars in cracked concrete," *Structural Concrete,* vol. 17, no. 5, pp. 799-810, 2016.

[17] EN 1992-4 - Eurocode 2: Design of concrete structures - Part 4: Design of fastenings for use in concrete, Brussels: European Committee for Standarization, 2018.

[18] S. Catteneo, G. Genesio and R. Piccinin, "Design of anchorages in connections using post-installed reinforcing bars," Naples, 2021.

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