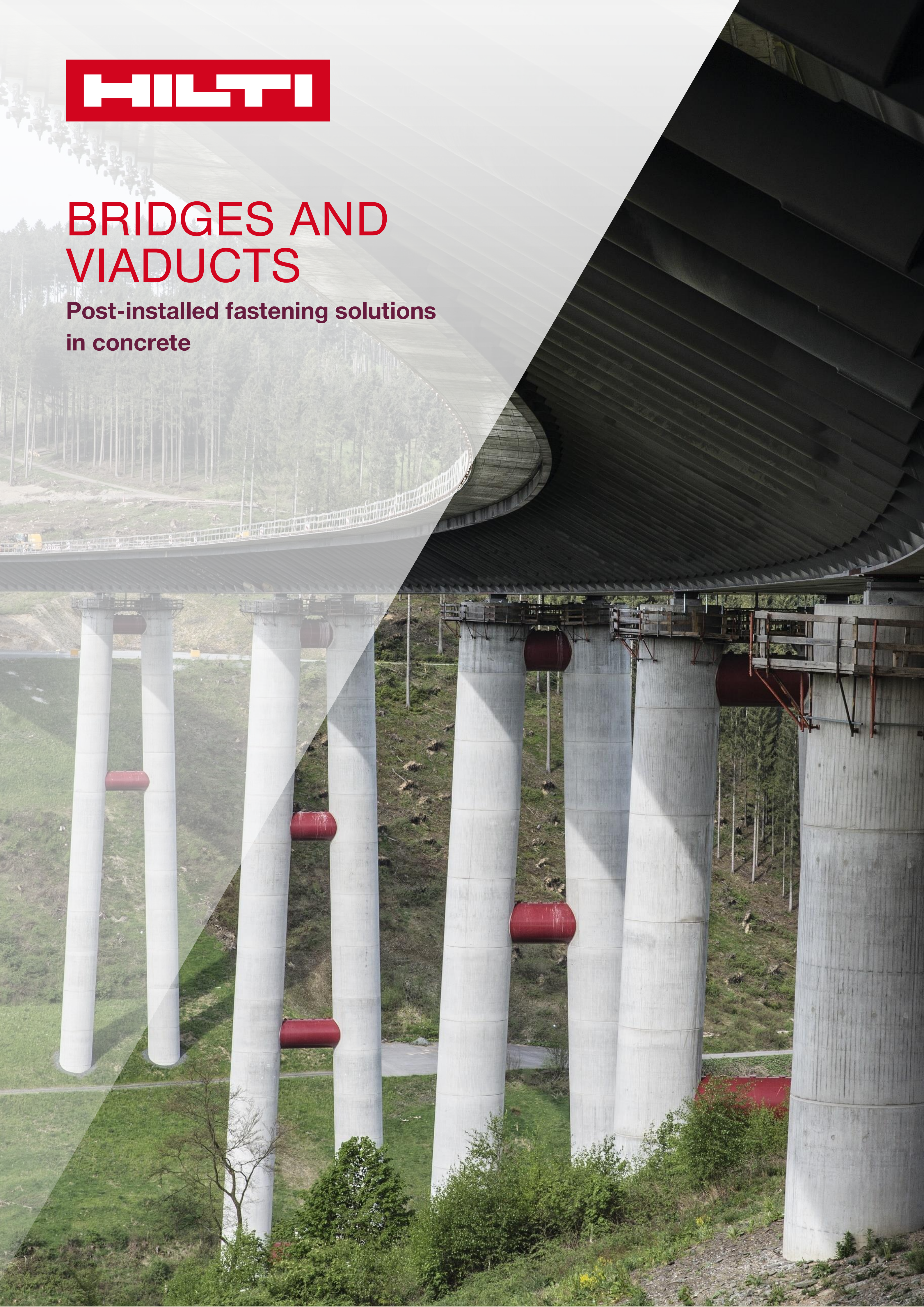
BRIDGES AND VIADUCTS

Post-installed fastening solutions in concrete



CONTENTS

1. SCOPE OF THIS WHITE PAPER

This consultation provides an overview of the main applications in the field of fastening systems that can be managed through post-installed solutions. For each application cluster, the regulatory framework, design method and approaches to evaluate and select the right solution will be introduced.

The area of interest is bridges and viaducts. Due to differences in regulation, the geographical scope of the document is oriented mainly to the Eurocode-driven markets even if the applications and solution systems may be of broader interest.

In this paper we present three applications:

1. Concrete extensions: Concrete extensions of concrete structural elements through post-installed rebars connections
2. Concrete overlay: Concrete structure reinforcement or refurbishment of corroded surfaces through the application of new concrete layers connected with post-installed systems
3. Baseplate fastening: Anchoring of steel structures/equipment to concrete with post-installed systems

The document provides an overview of theory and equations but doesn't aim to provide a full and exhaustive explanation. In case the reader would desire to get a full picture of design methods, the author recommends the application's referring Hilti handbook.

2. OUTLOOK AND CHALLENGES IN BRIDGES AND VIADUCTS

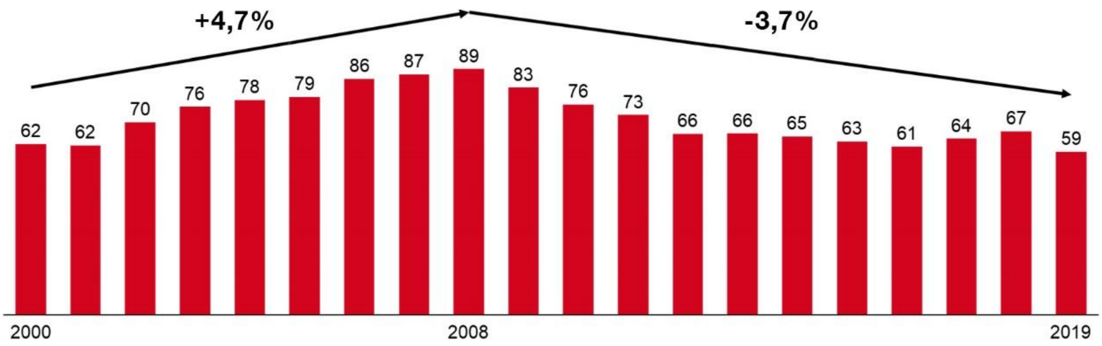
As shown from the TEN-T Performance Report [1] released in 2020 by the biennial Conference of European Directors of Roads (CEDR), around 2% of the entire Trans-European Road Network (71000 km) consists of bridges more than 100m in length.

Today two main challenges can be observed:

- Infrastructural transport network is aging: Many of the bridges were built during the economic boom of the 1950s and have now reached the end of their design life. Most bridges also carry significantly more vehicles than originally expected. This makes bridge maintenance, inspection, and monitoring of critical importance. Furthermore, recent tragic events, such as the Genoa bridge collapse, only highlight this urgency.
- Transport demand increase: As reported from the CEDR, demand on the network in term of traffic flow is increasing. National road administrations are planning to increase capacity by improving 31% of today total network sections.

Looking at previous investments and maintenance spending, the Organization for Economic Co-operation and Development (OECD) shows (Figure 2.1) a negative trend in the transportation sector since the outbreak of the 2008 economic and financial crisis [2]:

Figure 2.1 Transport infrastructural investment and maintenance spending in the Eurozone



Values in Bln €. Source: OECD data. Author's own calculation.

Though bridge and viaduct weakness due to aging and increase in demand across Europe is serious, Western countries, as reported from The New York Times [3], haven't yet approached the current situation as a priority.

The Italian Morandi bridge collapse is usually taken as an example, but similar patterns can be found in other European countries:

France: In July 2018 a study was commissioned by the Ministry of Transportation. The newspaper Le Monde [4], reporting the study, pointed out that around 12000 bridges are under investigation and that 7% have damages that could eventually result in collapse if not addressed.

Germany: Many studies [5] in recent years raised alarms stating underinvestment in infrastructure maintenance. According to the German Federal Highway Research Institute (BASt), out of the 39621 bridges monitored, 10.6% are in unsatisfactory condition while 1.8% are inadequate.

However, government awareness around the necessity to renovate infrastructural assets is increasing. As an example, in Italy's last report concerning Strategic Infrastructures planning [6], around 220 billion Euros in investments have been released, and a relevant share has been allocated on maintenance and improvement of current transportation assets.

3. APPLICATIONS OVERVIEW IN BRIDGE AND VIADUCT

Aging bridges and viaducts and the need to accommodate traffic increase require refurbishing part of the current infrastructural assets. The refurbishment, in this context, includes not only strengthening, repair and upgrading of bridge structures, but also geometric changes such as widening the bridge deck to provide more traffic capacity.

Post-installed fastening solutions are well known worldwide and widely utilized along the entire lifetime of the infrastructures, including renovation and maintenance. As innovative systems, post-installed solutions have been subject to a fast evolution of the related regulatory framework both in terms of a product qualification's procedures and its design methods.

Given this background, the scope of this white paper is to support the designer in providing an overall framework of when post-installed anchor solutions can be adopted to manage specific applications in renovation and maintenance of bridges and viaducts.

3.1 Applications overview

The first main distinction for describing a post-installed fastening system is the nature of the element being fastened through the post-installed anchor. The new element can be a new concrete layer or a steel baseplate (Figure 3.1). Because concrete structures are the most common in the infrastructural landscape, this paper addresses mainly those applications with concrete as base material.

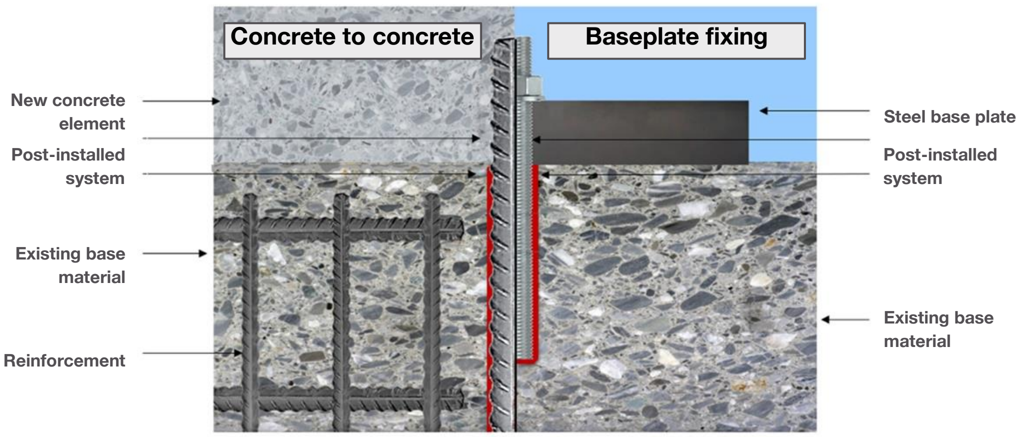
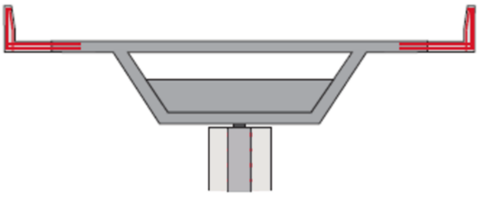


Figure 3.1
Distinction between concrete to concrete (C2C) and baseplate (BP) applications

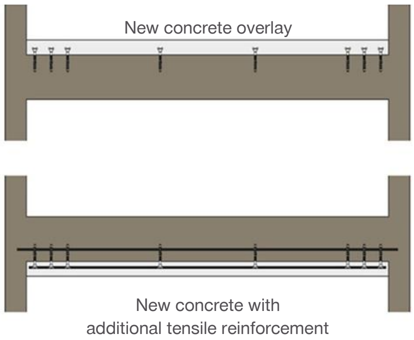
When the new element being fastened is concrete, applications will be further divided (Figure 3.2) in two groups: concrete extension through post-installed reinforcing bars, typically used to develop a new monolithic structural connection (e.g.: beam to column extension); and concrete overlay, frequently adopted to strengthen or repair an existing structure by pouring a new concrete layer.

Figure 3.2
Application examples: distinction between concrete extension and concrete overlay in bridges and viaducts

Concrete Extension

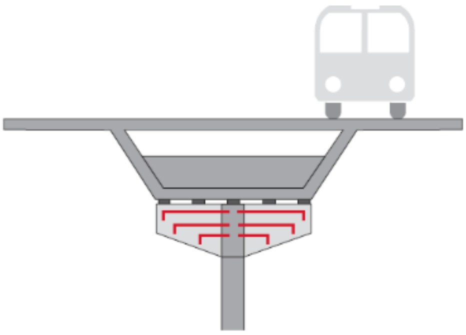


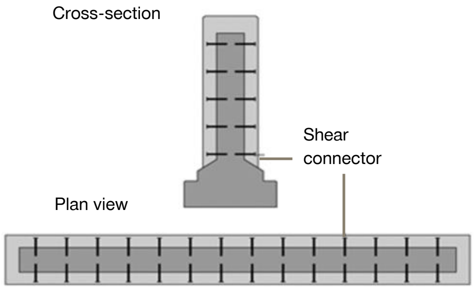
Concrete Overlay



Bridge deck extension

Bridge deck reinforcement





Repairing and strengthening a pie

Bridge corbels

Both extensions and reinforcements find application fields at deck level, along the piers and in the foundation.

Most common baseplate applications in bridges and viaducts concern the infrastructure at the roadway/railway's border at deck level (Figure 3.3). They include fastening of baseplates for retrofitting, as well as fastening of sound barriers, handrails and traffic signs. Other examples of common baseplate applications in motorway and railway segments include fastening crash barriers and anchoring posts for traction power.

Both concrete-to-concrete connections and baseplate applications in bridges and viaducts are in general selected based on structural considerations and are typically designed and detailed by a structural engineer. This is because failure of the system may pose a risk to life or result in significant economic loss.

Other applications that can be found in bridges and viaducts are related to safety, for example fastening of escape routes and gratings, services for anchoring piping/gas/electricity and communication systems, fastening of temporary elements and formworks.

The design establishes whether the requirements of the serviceability and ultimate limit state are met. Many are the dimensions impacting the design and that will be discussed in this whitepaper:

- Applicable design code or guideline
- Type of action: static, seismic, fatigue
- Design life
- Corrosion

Additional economical or quality aspects may be considered already in the design or the specification by specifying proof loading or test loads.

Figure 3.3
Application examples: baseplate applications in bridge and viaducts

Baseplate retrofitting applications

Sound/wind barrier fastening

Temporary structures fastening

Traction power fastening

Handrail fastening

Crash barrier fastening

3.2 Regulatory framework overview for post-installed fastening systems

European standards and regulatory framework guide testing, assessment and design of post-installed systems. The construction products regulation (CPR) lays down harmonized rules for marketing construction products in Europe. Below are definitions to help facilitate understanding as you read through the paper:

3.2.1 European Committee for Standardization (CEN)

CEN, recognized by the European Union as a European Standardization Organization, brings together knowledge and expertise from its members, from business and industry and from other stakeholders, in order to develop European Standards. CEN provides a platform for the development of European Standards and other technical documents in relation to various kinds of products, materials, services and processes. They help to protect the environment, as well as the health and safety of consumers and workers.

3.2.2 European Organization for Technical Assessment (EOTA)

EOTA is set up by the Regulation (EU) No 305/2011 and comprises all Technical Assessment Bodies (TABs) designated by Member States of the European Union and the European Economic Area.

EOTA co-ordinates the application of the procedures set for requests for European Technical Assessment (ETA) and for the procedures for adopting a European Assessment Document (EAD). EOTA also informs the European Commission and the Standing Committee on Construction of any question related to the preparation of EADs and suggests improvements to the European Commission based on its experience gained.

3.2.3 European Assessment Document (EAD)

A European Assessment Document, or EAD for short, is a harmonized technical specification developed by EOTA as the basis for European Technical Assessments (ETAs). The development of new, or the amendment of existing, EADs is usually triggered by an ETA request from a manufacturer.

3.2.4 European Technical Assessment (ETA)

The European Technical Assessment (ETA) provides an independent Europe-wide procedure for assessing the essential performance characteristics of a construction product. It provides the documented assessment of the performance of a construction product, in relation to its essential characteristic, in accordance with the respective EAD.

3.2.5 Technical Reports (TR)

EOTA Technical reports are developed as supporting documents to EADs containing detailed aspects relevant for construction products such as design, execution and evaluation of tests, and express the common understanding of existing knowledge and experience of the Technical Assessment Bodies in EOTA at a particular point in time.

3.2.6 Eurocodes

Eurocode, or EC or EN, are harmonized technical rules specifying how structural design should be conducted within the European Union. These codes have been developed by the European Committee for Standardisation upon the request of the European Commission.

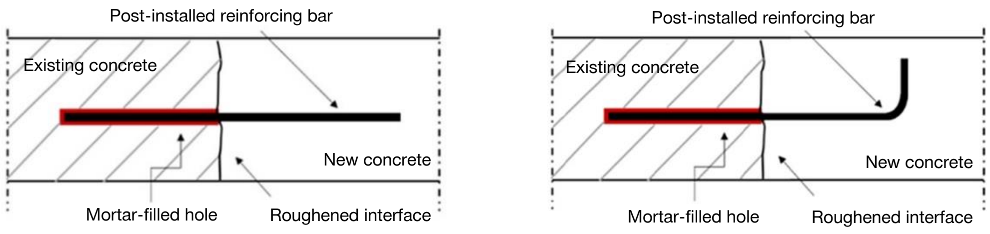
4. CONCRETE EXTENSIONS

4.1 What they are

A post-installed rebar (PIR) connection is the installation of deformed reinforcing bars (rebar) in holes drilled in concrete to emulate the behavior of cast-in-place reinforcing bars, for example a bridge deck extension or a post-installed corbel. 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 on one side of the interface and are usually cast into new concrete on the other side of the interface (Figure 4.1). 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 post-installed anchors in baseplate applications, are often installed with small concrete cover ($3\phi > c > 2\phi$), where ϕ is the reinforcement bar diameter and c is the concrete cover. This geometrical boundary condition is in general given by the individual geometry of the bridge's element. 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 in baseplate applications or concrete overlay applications (shear dowels).
- Post-installed reinforcing bars are generally embedded as required to "develop" their design stress σ_{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 4.1
Post-installed reinforced bar straight or hooked



4.2 Design principles

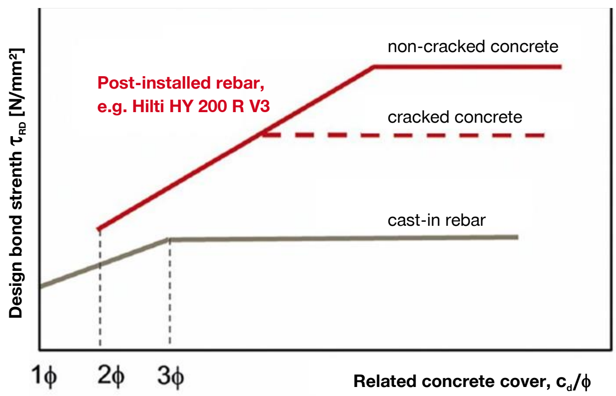
Until 2018 the technical report TR023 [7] “Assessment of Post-installed Reinforcing Bar Connections” provided guidance for verifying that post-installed reinforcing bar connections, built with a specific mortar system, exhibit comparable behavior to cast-in-place reinforcing bar connections in terms of load and displacement behavior under several environmental conditions (since 2018 TR 023 is replaced by EAD 330087-00-0601 [8]). As a result, a given post-installed reinforcing bar system assessed by EAD 33087-00-0601 results in at least similar bond strength and similar displacement behavior as cast-in-place reinforcing bars when considering the influencing factors stated in the related EAD. Due to this core philosophy, the design of post-installed reinforcing bar connections employing that system can follow the provisions for cast-in-place reinforcing bars following EN 1992-1-1 (in this document also named EC2-1) [9]. However, the application range of post-installed rebar is limited by EAD 33087-00-0601 to:

- 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.
- Simply supported elements.

To overcome this limitation and to provide a supplementary design solution a technical report TR 069 [10] “Design method for anchorages of post-installed reinforcing bars (rebars) with improved bond-splitting behavior as compared to EN 1992-1-1“ was published in 2020.

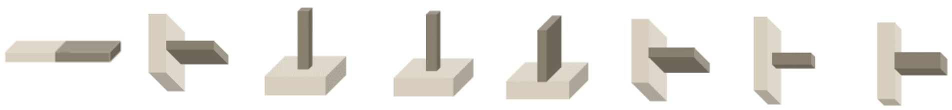
This design guideline allows the design of moment-resisting post-installed rebar connections without the execution as a splice on European level. TR 069 utilizes the real bond-splitting behavior of post-installed rebar systems taking into account the concrete cover in the design equations. According to Figure 4.2, when the value of minimum concrete cover is greater than 2ϕ (where ϕ is the bar diameter), post-installed rebar systems exhibit significantly higher bond-splitting behavior than cast-in-place bars of equivalent bar diameter and anchorage length. This behavior is qualified and assessed according to EAD 332402-00-0601 [11].

Figure 4.2
Schematic design bond strength as a function of the related concrete cover



The allowable post-installed concrete extensions taking into account connection types, allowable forces, design methods, required EAD and covered load cases are given in Table 1 and will be further discussed in the next section.

Table 1
Allowable concrete to concrete connection taking account of connection type, allowable forces, design method, required EAD and covered load cases



	1	2	3	4	5	6	7	8
	Splice	End-Anchorage	End-Anchorage	End-Anchorage				
Forces and Moments	Yes	Only Shear	Only Compression	Yes				
Frame examples	All configuration connected via splices	Simply supported beam/ slab	Wall/ column to foundation	Column to foundation	Wall to foundation	Slab to wall	Beam to wall	Beal to column
Required EAD	EAD 330087-00-0601/ EAD 331522			EAD 332402-00-0601				
Design Method	EC 2/ EC 8			TR 069				
Load cases	Static, sustained loading, fire, 50 years, 100 years, Seismic			Static, sustained loading, 50 years				

4.3 Static design of structural post-installed rebar connections

As represented in Table 1, the design can follow EC2-1 or TR 069 depending on the frame and actions insisting on the structure.

4.3.1 Static design anchorage length of post-installed rebar as per Eurocode 2

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

$$f_{bd} = 2.25\,\eta_1\eta_2 f_{ctd} \qquad \text{(EC2-1, 8.2)}$$

where:

2,25 basic value of the design bond strength

η_1 coefficient related to the quality of the bond condition and the position of the bar during concrete pouring. η_1 =1.0 stands for good bond conditions and η_1 = 0.7 is taken for all other cases. Note for post-installed rebar η_1 = 1.0 should be taken

η_2 coefficient related to the rebar diameter ϕ [mm]:

$\eta_2 = (132-\phi)/100 \leq 1.0$ for $\phi > 32$mm

$\eta_2 = 1.0$ for $\phi \leq 32$mm

f_{ctd} the design tensile strength of the concrete

For post-installed anchor f_{bd} can be taken from the relevant product's ETA.

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

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

where:

ϕ reinforcing bar diameter

σ_{sd} design steel stress at the beginning of the anchorage

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 (α_1 to α_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:

Under tension: $l_{bd} = \alpha_1\, \alpha_2\, \alpha_3\, \alpha_4\, \alpha_5\, l_{b,rqd} \geq l_{b,min}$ (EC2-1,8.4) (e.g., case 2 in Table 1)

Under compression: $l_{bd} = \alpha_4\, l_{b,rqd} \geq l_{b,min}$ (e.g., case 3 in Table 1)

where:

α_1 considers the form of the bar; for post-installed rebar fixed with chemical anchors the value α1 is equal to 1.0 being the reinforcing bar always straight.

α_2 takes into account the concrete cover: $0.7 \leq \alpha_2 = 1\text{-}0.15(c_d\text{-}k\phi)/\phi \leq 1.0$ where, for straight bars, c_d is the smallest of the concrete cover and half of the clear spacing of bars and k=1 for bars without.

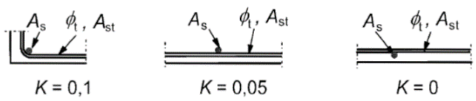
α_2 takes into account passive confinement provided by the surrounded concrete.

α_3 takes into account the effect of transverse reinforcement where $0{,}7 \leq \alpha_3 = 1\text{-}K\lambda \leq 1.0$ with $\lambda = (\Sigma A_{st} - \Sigma A_{st,min}) / A_s$.

ΣA_{st} = cross-sectional area of the minimum transverse reinforcement along the design anchorage length l_{bd}

$\Sigma A_{st,min} = 0.25A_s$ for beams and $\Sigma A_{st,min}$=0 for slabs with A_s = area of a single anchored bar with maximum bar diameter (mm²)

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



α_3 takes into account 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 α_3 =1.0 maybe assumed.

α_4 is for the influence of one or more welded transverse bars (diameter of transverse bar > 0.6 diameter of post-installed reinforcing bar) along the design anchorage length. Parameter equals 0.7 if transverse reinforcement is welded to the reinforcement to be anchored, otherwise α_4 = 1.0.

α_5 takes into account transverse pressure while $\alpha_5 = 1 - 0.04p \geq 0.7$ and ≤ 1.0 where p is the transverse pressure along the anchorage length (active confinement). The confining pressure that is applied to pre-stress the concrete element laterally prior to loading exerts an initial volumetric strain due to compaction. 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 lb,min is given as follows:
$l_{b,min} \geq \max(0.3 l_{b,rqd}; 10\phi; 100mm)$ for bars under tension (EC2-1, 8.6)
$l_{b,min} \geq \max(0.6 l_{b,rqd}; 10\phi; 100mm)$ for bars under compression (EC2-1, 8.7)

4.3.2 Static design splice length of post-installed rebar as per Eurocode 2

The splice length l_0, as per the anchorage length l_{bd}, is calculated from the basic anchorage length $l_{b,rqd}$ with:

$l_0 = \alpha_1\, \alpha_2\, \alpha_3\, \alpha_5\, \alpha_6\, l_{b,rqd} \geq l_{0,min}$ (EC2-1, 8.10) (e.g., case 1 in Table 1)

where:

α_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

For bars in compression (usually considered only in highly loaded columns), all α-factors except α_6 are the same as for anchorage, see above.

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

The minimum splice length $l_{o,min}$ can be calculated as follows:
$l_{0,min} = \max(0.3 \cdot \alpha_6 \cdot l_{b,rqd}; 15\phi; 200mm)$ (EC2-1, 8.11)

4.3.3 Static design embedment depth of post-installed rebar as per Technical Report TR 069

The EOTA TR069 combines reinforced concrete design principles (EN 1992-1-1, in this document named EC2-1) with anchoring to concrete principles (EN 1992-4, in this document named EC2-4). The individual failure modes of the system connection are rebar steel yielding, concrete cone, and bond/splitting. The design is based on the hierarchy of strength design principle, i.e. the lowest resistance of the individual failures model is decisive. The requirements of EC2-1 in terms of minimum anchorage length, as discussed above, must be fulfilled.

$R_d \leq \min\,(N_{Rd,y}; N_{Rd,c}\,; N_{Rd,sp})$ (TR 069, 4.1)

where:

R_d design value of resistance, $R_d = \frac{R_k}{\gamma_M}$ with R_k characteristic resistance and γ_M according do EN 1990

$N_{Rd,y}$ design resistance to yielding

$N_{Rd,c}$ design concrete cone break-out resistance

$N_{Rd,sp}$ design bond splitting resistance

Design resistance to yielding

The resistance to yielding is a function of rebar diameter and steel strength and can be obtained from the following equation:

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

with:

$N_{Rk,y} = A_s f_{yk}$

where:

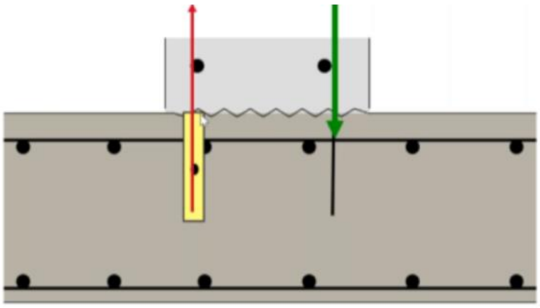
A_s cross sectional area of all tensioned post-installed rebars within the connection

f_{yk} yield strength

Design concrete cone break out resistance

The embedment depth is calculated from the interface between the old and the new concrete. This is where the concrete cone can occur, Figure 4.3.

Figure 4.3 Schematic representation from where the embedment depth is considered to start (yellow rectangle)



The characteristic resistance for the group of reinforcement under tension action resulting from the moment resisting mechanism shall be obtained as per the equation below:

$N_{Rk,c} = N^0_{Rk,c} \frac{A_{c,N}}{A^0_{c,N}} \psi_{s,N} \psi_{re,N} \psi_{ec,N} \psi_{M,N}$ (TR 069, 4.3)

where:

$N^0_{Rk,c}$ Characteristic concrete cone break-out resistance for a single post-installed rebar not influenced by any adjacent post-installed rebar or edge

$A_{c,N}$ Actual projected area of the group of tensioned rebars

$A^0_{c,N}$ Reference projected area of a single reinforcement post-installed in the concrete with large spacing and edge distance with the concrete cone idealized as a pyramid of height l_b and base length $s_{cr,N} = 3l_b$

$\psi_{s,N}$ Factor considering the disturbance of the distribution of stresses in the concrete due to the proximity of an edge in the concrete member in case of concrete cone failure

$\psi_{re,N}$ Shell spalling factor accounts for the reduced strength of rebars with an anchorage length l_b < 100mm inserted in concrete elements with closely spaced reinforcement

$\psi_{ec,N}$ Factor considering the effect of eccentricity between the point of application of the axial force and the center of gravity of the tensioned rebar (e.g. in the case or more layers of tensioned reinforcement)

$\psi_{M,N}$ Factor considering the effect of the compression zone of the cross section of the attached reinforced concrete element in case of bending moment

Design bond splitting resistance

The characteristic resistance for the group of reinforcement under tension shall be obtained as given below. If the load on the tensioned bars is applied eccentrically and/or the values c_{min} and c_{max} are different for each tensioned bar, the resistance $N_{Rk,sp}$ shall be calculated separately for each rebar.

$N_{Rk,sp} = \tau_{Rk,sp}\, l_b\, \phi\, \pi$ (TR 069, 4.10) with:

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

$$\leq \tau_{Rk,ucr} \frac{\Omega_{cr}}{\Omega_{p,tr}} \psi_{sus} \qquad \text{for} \qquad 7\phi \leq l_b \leq 20\phi$$

$$\leq \tau_{Rk,ucr} \left(\frac{20_{lb}}{\phi}\right)^{lb1} \frac{\Omega_{cr}}{\Omega_{p,tr}} \psi_{sus} \qquad \text{for} \qquad l_b \geq 20\phi$$

where:

sp1, sp2, sp3, sp4 and lb1: Fitting exponents according to relevant ETA

A_k Fitting factor from relevant product ETA

$\tau_{Rk,sp}$ bond resistance in uncracked concrete (upper value)

η_1 coefficient related to the quality of the bond condition and the position of the bar during concrete pouring. η_1 = 1.0 stands for good bond conditions and η_1 = 0.7 is taken for all other cases. Note for post-installed rebar η_1 = 1.0 should be taken

c_d minimum between clear concrete cover in all directions and half of the clear spacing from the closest neighboring reinforcing bar

c_{max} maximum between clear concrete cover in x direction (rebar disposition's direction) and half of the clear spacing from the closest neighboring reinforcing bar

k_m Factor for the effectiveness of transverse reinforcement. Equal to:

- 12, where rebars are confined inside a bend of links passing round the bar of at least 90°.
- 6, where a rebar is more than 125mm and more than 5 bar diameters from the nearest vertical leg of a link crossing the splitting plane in an approximately perpendicular direction.
- 0, if a splitting crack would not intersect transverse reinforcement and hence a crack through the plane of rebars would for without intersecting transverse reinforcement.

k_{tr} Normalized ratio to consider the amount of transverse reinforcement crossing a potential splitting surface in accordance with fib Model Code 2010

Φ Diameter of the rebar

Ω_{cr} Factor to account for the influence of cracked concrete on resistance to combine pull-out and concrete failure taken from relevant ETA

$\Omega_{p,tr}$ Factor to account for transverse pressure in concrete

l_b Embedment length of the post-installed rebar

ψ_{sus} Factor to account for the effect of sustained loads as per EN 1992-4 [18] and dependent from the product factor $\psi_{sus}{}^0$ taken from the relevant European Technical Specification

4.4 Seismic design of structural post-installed rebar connections

Seismic design as per Eurocode

Until 2018, qualification guidelines for post-installed rebar included methods and criteria to assess the performance in static and fire conditions, but not seismic.

From 2018, the EAD 330087 [12] was endorsed by EOTA and Eurocode. This EAD includes methods and criteria to assess the performance of post-installed rebar systems when subjected to seismic conditions.

To specify post-installed rebar systems in compliance with Eurocode 8, the products must now be assessed in accordance with the EAD 330087. Qualification ensures that:

- Product performance under earthquake simulated cyclic loading is tested and assessed
- The product is compliant with European CPR
- The design of anchorage lengths and splices can be performed in accordance with EN 1992-1-1

Based on the EAD 331522, two types of tests must be performed with the aim of verifying the equivalence of post-installed and cast-in rebar systems:

- Confined cyclic push-pull tests without influence of edge distance and spacing – Comparison between the bond strength degradation of post-installed and cast-in rebar systems (pull out failure mode)
- Cyclic tests at minimum allowed edge distance (c_d = 2d) – Comparison between the splitting strength degradation and energy dissipation of post-installed and cast-in rebar systems (splitting failure mode)

As a result of the assessment process, the seismic European Technical Assessment (ETA) of a product includes the bond strength values and concrete covers that can be used for design of post-installed rebar connections under seismic loading. Those values can be used in accordance to the Eurocode 8 prescription in section 5.6.3 for seismic actions.

4.5 100 years' service life requirement

As public infrastructure continues to deteriorate, ever-increasing scrutiny is placed on structural reliability throughout the service life of structures by officials, owners, and engineers. In bridges, tunnels, and other civil structures, a 100-year service life requirement is placed on the structure in Eurocode 1990 [Table 2]. Anchored connections in structures with Design Working Life Category 5 must also satisfy the 100-year requirement.

Table 2
Adapted from EN 1990 Table 2.1 "Indicative design working life"

Design Working Life Category	Indicative design working life (years)	Examples
1	10	Temporary structures
2	10–25	Replaceable structural parts
3	15–30	Agricultural and similar structures
4	50	Buildings and other common structures
5	100	Monumental buildings, bridges, and other civil structures

Requirement for a service life and/or design life of 100 years is based on the goal of minimizing maintenance requirements and to ensure that the investment is spent in a rational way.

Nowadays the EAD 332402-00-0601 [13] is providing the answer for such a request as it is written on the assumption that the estimated design life of the post-installed rebar systems for the intended use is at least 50 years or 100 years. Hilti may also provide engineering judgment with 120 years due to the limitations of the discussed EAD. The biggest differences during the assessment process of a product between the two different design life ranges is that specifically the long-term tests that are connected to 50 years are modified to a 100-year range.

However, it is important to note that this design life assessment is related to the bond between mortar and concrete (bond strength) by providing bond strength values for 50 years and 100 years, while the steel of the rebar and the concrete is not considered within the scope of the EAD. Consequently, the EAD assumes that concrete quality is not deflecting during the design life, meaning that defining the exposure classes in the bridge projects, structural classes and consequently the required nominal concrete cover is fundamental for applying the logic of EAD.

4.6 Installation

The proper installation of post-installed rebar is key for the structural monolithic behavior as per the design. The step-by-step process is as follows:

1. Localization of existing reinforcement and/or other embedded elements
2. Roughening of the exiting concrete surface
3. Installation of post-installed reinforcing system:
    a) Drilling method as per engineer's specification
    b) Concrete hole cleaning. No additional bore hole cleaning necessary if Hilti SafeSet system with automatic cleaning is adopted as per product's ETA indication
    c) Injection of the mortar system as per engineer's specification
    d) Reinforcing bar installation
4. If required from the structural engineer: site testing of post-installer reinforcing system through pull-out test
5. Pouring of the new concrete element

Localization of existing rebar elements is important both in respect to the indicated overlapping between new and existing rebar if the design has been performed according to EC2-1, and to avoid hitting rebar during the drilling phase. Two types of technologies can be used to detect rebar:

(1) Ferrous scanner that locates rebar through usage of magnetic fields. Recommended Hilti solution is the scanner PS 300, which allows measurements up to 200mm deep when minimum distance between rebar is not lower than 30mm.

(2) Ground penetrating radar scanner that locates rebar and not ferrous elements on multiple layers. Recommended Hilti solution is the scanner PS 1000, which allows measurement up to 300mm. If detecting only ferrous elements, a ferrous scanner is recommended.

Surface roughening prior to casting new concrete against existing concrete increases not only adhesion but also 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: the surface should have at least 3mm roughness at about 40mm spacing. If 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.

When it comes to the rebar installation, frequently deep embedment depth is required. Three drilling methods are possible:

- Rotary-impact drilling equipped with standard or cruciform carbide bits or with a Hilti SafeSet system if an integrated concrete cleaning phase is wanted alongside drilling
- Compressed air drills
- Diamond coring that utilizes either wet coring technology or dry coring (to avoid on concrete structure and typically used for lower embedment depth).

The Hilti SafeSet system consists of Hollow Drill Bits (HDB) used in combination with Hilti Vacuum Cleaners (as given in the product relevant ETA). 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.

Hilti SafeSet system

For deep embedment depths or when vibrations must be avoided, diamond coring usually represents the preferred solution. After diamond coring drilling, the hole surface will be perfectly smooth. To keep original performance while using diamond coring, concrete holes can be roughened after diamond coring through the roughening tool TE-Y RT before the injection phase (Figure 4.4).

Figure 4.4
Roughening tool system enables original performances after diamond coring execution

When the drilling method has not been predetermined, it is advisable to use an adhesive that is suitable for all drilling methods (e.g., Hilti HIT-RE 500 V4). However, bond strength when diamond drilling without a roughening tool can be much lower than when hammer drilling. If diamond core drilling is allowed, it is stated in the relevant ETA.

4.7 Hilti recommended solutions

Table 3
Recommended Hilti solutions for concrete extensions

Hilti System	**RE 500 V4**	**HY 200 R V3**
		
When to use	High performance and extreme conditions When it is the only solution: • Submerged and water-filled conditions • High embedment depths with high temperatures • Diamond coring holes with no roughening • Large diameters (f > 32mm)	High performance for everyday applications When recommended: • Everyday applications where it's needed to design in static/ seismic/fire or fatigue • High productivity through short curing time
Design software	PROFIS Engineering rebar module	
Embedment range*	Up to 3.2m	Up to 2m
Rebar range diameter	Static ϕ8 – ϕ40 Seismic ϕ10 – ϕ40	Static ϕ8 – ϕ32 Seismic ϕ10 – ϕ32
100 years certified requirement	Yes	Yes
Working time @20°C	30 minutes	9 minutes
Curing time @20°C	7 hours	1 hour
Base material T range	-5°C ≤ T ≤ 40°C	-10°C ≤ T ≤ 40°C
Comment for installation	Hammer-drilled holes: Yes Diamond coring holes: Yes Underwater and water-filled: Yes Hilti SafeSet system with cleaning: Yes Hilti SafeSet systemwith roughening tool: Yes	Hammer-drilled holes: Yes Diamond coring holes: With roughening tool Underwater and water-filled: No Hilti SafeSet system with cleaning: Yes Hilti SafeSet system with roughening tool: No

*** temperature and dispenser dependent**
Simplified overview, detail can be found in the relevant product ETA

5. CONCRETE OVERLAYS

5.1 What are they

When a new layer of concrete is applied to existing concrete with the aim of strengthening or repairing a structure, the result is referred to as an overlay. The overlay (here the term overlay is used even for concrete jacketing applications) is usually cast directly or applied as shotcrete. Its function is to augment the flexural compression, flexural tension, shear strength and ductility, depending on the position of placement. Some typical applications involve the strengthening of structural elements such as vaults, pillars, beams and foundations (Figure 5.1).

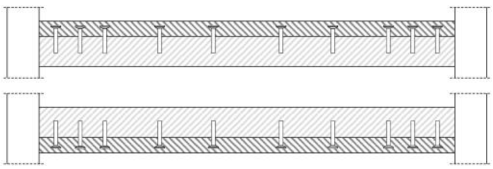
Prior to placement of the overlay, the surface of the old concrete member is prepared by suitable means and pre-wetted.

Shrinkage of the new concrete overlay can be reduced by careful selection of the concrete mix. However, the constraint forces caused by differential shrinkage and, in certain cases, by differential temperature gradients, cannot be avoided. Initially, stresses in the bond interface result from a combination of peripheral loads and internal constraint forces. It must be kept in mind that stresses due to shrinkage and temperature gradients in the new concrete typically reach their maximum at the perimeter (peeling forces). The combination of peripheral and internal stresses often exceeds the capacity of the initial bond, thus requiring the designer to allow for a de-bonded interface. This is particularly true in the case of bridge overlays, which are subject to fatigue stresses resulting from traffic loads.

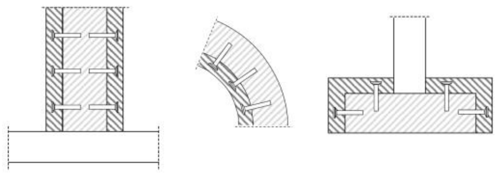
Furthermore, these stresses vary with time, and bond failure can take place years after installing the overlay. When this happens, the tensile forces set up must be taken up by connectors positioned across the interface.

Figure 5.1.1
Concrete overlay common scheme object of this paragraph

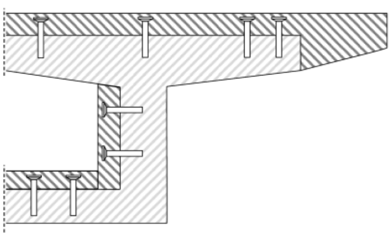
Beam, slabs



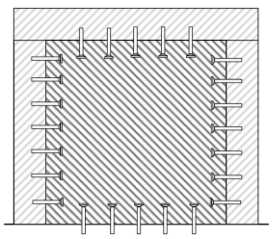
Columns, walls, aches, shells, tunnels, foundations



Bridges



Shear wall in reinforced concrete frame



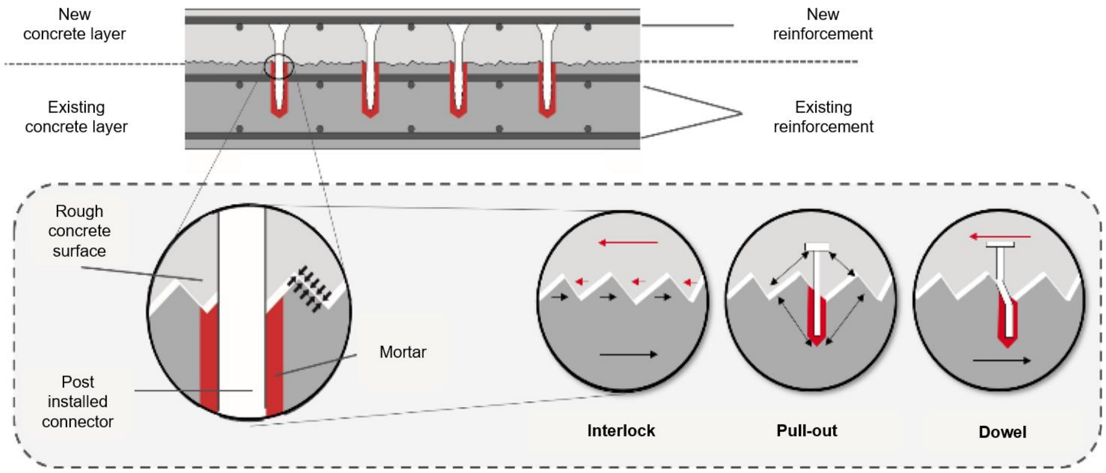
= Existing concrete

= New concrete / overlay

5.2 Design principles

Forces at the interface between the new and existing concrete are determined from the external and internal forces acting on the building component. When designing the interface, it must normally be assumed that the interface is de-bonded. The shear connectors crossing the interface must be placed in such a way that shear forces ("shear flow") at the interface are transmitted at design level (Figure 5.2).

Figure 5.2
Load contribution of the different components



Because of separation at the interface, the shear connectors are subject to a tensile force and simultaneously to a bending moment, both of which depend on the roughness of the interface surfaces. If the surfaces are roughened, additional interlocking effects and cohesion can take up part of the shear force at the interface.

Together with external forces, the structure will be subject even to other forces resulting from constraint at the perimeter.

Though subdividing the interface into zones to contribute to different shear stress is allowed, re-distributing the stress for rough and very rough surfaces is not, so the maximum value of each zone is decisive. When designing concrete overlay applications, three verifications have to be taken into account:

1. Verification of the shear interface
2. Verification of fastening in existing concrete
3. Verification of fastening in the new concrete overlay

The EAD that identify the qualification process for connectors that connect two layers of concrete cast at different times is the EAD 332347-00-0601 [14].

The following paragraph will concern point 1 considering both the European design report TR 066 and the method based on Hilti expertise. Points 2 and 3 will be addressed in the baseplate fastening section as referring to the same verifications required for post-installed anchor as per Eurocode 2 part 4 for baseplate applications.

5.3 Design shear interface resistance following TR 066

The transmission of shear forces at the interface between the new and existing concrete is determined by aggregate interlock, shear friction and dowel action. In general, the following equation applies:

$\tau_{Rd} \geq \tau_{Ed}$

where:

τ_{Rd} Design resistance of the shear force per meter (“shear flow”) at the interface

τ_{Ed} Design value of the shear flow acting at the interface

5.3.1 Static design resistance following TR 066

To evaluate design resistance TR 066 recommends the following equation:

$$\tau_{Rd} = \underbrace{c_r \cdot f_{ck}^{1/3}}_{\text{Interlock}} + \underbrace{\mu \cdot \sigma_n + \mu \cdot \kappa_1 \cdot \alpha_{k1} \cdot \rho \cdot \sigma_S}_{\text{pull-out}} + \underbrace{\kappa_2 \cdot \alpha_{k2} \cdot \rho \cdot \sqrt{\frac{f_{yk}}{\gamma_s} \cdot \frac{0{,}85 \cdot f_{ck}}{\gamma_c}}}_{\text{dowel}} \leq \underbrace{\beta_c \cdot \nu_e \cdot \frac{0{,}85 \cdot f_{ck}}{\gamma_c}}_{\text{concrete strut}} \quad \text{(TR066 2.9, 2.11)}$$

where:

c_r Coefficient for adhesive bond resistance in a reinforced interface (Table 3)

c_a Coefficient for adhesive bond resistance in an unreinforced interface (Table 3)

f_{ck} minimum value of concrete compressive strength of the two concrete layers, measured on cylinders

f_{yk} Characteristic yield strength of the shear connector

μ Friction coefficient (Table 3)

σ_n Lowest expected compressive stress resulting from an eventual normal force acting on the interface (compression has a positive sign)

κ_1 Interaction coefficient for tensile force activated in the shear connector (Table 3)

κ_2 Interaction coefficient for flexural resistance in the shear connector (Table 3)

k_1 Modification factor for material properties of the connector from product ETA

α_{k2} Modification factor for geometry of the connector from product ETA

ρ Reinforcement ratio of the steel of the shear connector crossing the interface

σ_S Steel stress associated to the relevant failure mode

γ_c Safety factor for concrete; 1.50 as given in EN 1992-4 for strengthening of existing structures

γ_s Safety factor for steel; 1.15 as given in EN 1992-4 for supplementary reinforcement

b_i Width of the interface of the composed section

ν_e Coefficient for reduction of concrete strength - $\nu_e = \left(0{,}55 \cdot \left(\frac{30}{f_{ck}}\right)^{1/3} < 0{,}55\right.$

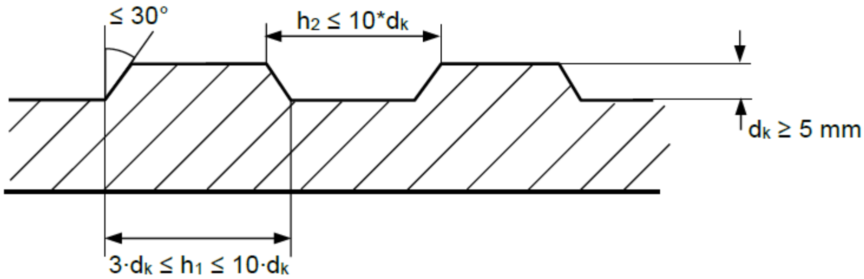
β_c Coefficient for the strength of the compression strut (Table 4)

Table 4
Coefficients and parameters for different surface roughness

Surface characteristics of interface	c_a	c_r	κ_1	κ_2	β_c	μ
Very rough, (including shear keys[1]) **R_t ≥ 3,0 mm**	0,5	0,2	0,5	0,9	0,5	0,8 (f_{ck} ≥ 20) 1,0 (f_{ck} ≥ 35)
Rough, R_t ≥ 1,5 mm	0,4	0,1	0,5	0,9	0,5	0,7
Smooth (concrete surface without treatment after vibration or slightly roughened when cast against formwork)	0,2	0	0,5	1,1	0,4	0,6
Very smooth (steel, plastic, timber formwork)	0,025	0	0	1,5	0,3	0,5

1) Shear keys should satisfy the geometrical requirements given in Figure 5.3

Figure 5.3
Geometry of shear keys



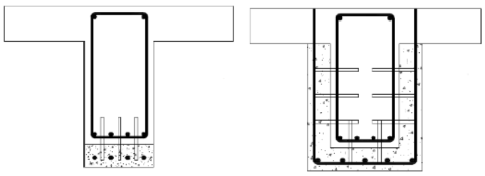
where: d_k is the height of a shear key, h_1 is the base length of a shear key.

5.3.2 Design under seismic

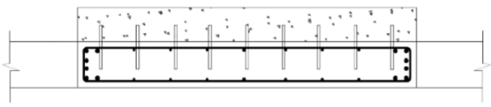
Design of shear interface under seismic cyclic loading is covered by TR 066. Seismic force acting on the structural element activate tensile forces perpendicular to the interface, which are carried by connector and transferred to the two concrete layers.

Typical seismic repair/ strengthening applications involving shear interfaces

Layer over/ under/ lateral/ closed jacket



Closed or partial jacketing thickening of sides/ thickening of the confined boundary elements



Under seismic conditions, resistance of the connectors and the decisive failure mode shall be calculated assuming performance category C1 or C2, depending on the application and the design assumption (see Table 4 as reference). As for static, guidance in this regard is provided by EN 1992-4 [18]. Seismic design resistance at the interface is calculated as follows:

$$\tau_{Rd} = \alpha_{seis}\left[c_r \cdot f_{ck}^{\frac{1}{3}} + \mu \cdot \sigma_n + \mu \cdot \kappa_1 \cdot \alpha_{k1} \cdot \rho \cdot \sigma_{S,eq} + \kappa_2 \cdot \alpha_{k2} \cdot \rho \cdot \sqrt{\frac{f_{yk}}{\gamma_s} \cdot \frac{f_{ck}}{\gamma_c}}\right] \leq \beta_c \cdot \nu_e \cdot \frac{0{,}85 \cdot f_{ck}}{\gamma_c} \qquad \text{(TR066, 3.2)}$$

where:

$\sigma_{A,eq}$ Steel stress associated with the relevant failure mode under seismic conditions

α_{seis} given in the relevant product ETA in accordance to EAD 332347

Other parameter as per the table 5.

Table 5
Coefficients and parameters for different surface roughness for seismic cycling loading

Surface characteristics of interface	c_r	κ_1	κ_2	β_c	μ_v ($f_{ck} \geq 20$)	μ_v ($f_{ck} \geq 35$)
Rough, $R_t \geq 1{,}5$ mm	0	0,5	0,9	0,5	$0{,}4\sqrt[3]{\left(\frac{f_{cd}}{\sigma_c+\sigma_n}\right)^2}$	$0{,}27\sqrt[3]{\left(\frac{f_{cd}}{\sigma_c+\sigma_n}\right)^2}$
Smooth (concrete surface without treatment after vibration or slightly roughened when cast against formwork)	0	0,5	1,1	0,4	$0{,}27\sqrt[3]{\left(\frac{f_{cd}}{\sigma_c+\sigma_n}\right)^2}$	$0{,}135\sqrt[3]{\left(\frac{f_{cd}}{\sigma_c+\sigma_n}\right)^2}$

5.3.3 Design under fatigue

When an interface is subject to substantial changes in stress, i.e. not predominantly static forces, it must be designed to withstand fatigue. In these circumstances the interface between the two concrete layers must always be very roughened.

According to EOTA TR066, fatigue is taken into account by means of a reduction coefficient η_{sc}:

$\Delta v_{Ed} \leq \eta_{sc} \cdot v_{Rd}$ (TR066 2.13)

Without the effect of static loadings:

$\Delta v_{Ed} = v_{Ed,\max}$ (TR066 2.14)

$\eta_{sc} = 0{,}4$ or otherwise given in the relevant connector's ETA

where:

Δv_{Ed} Shear stress acting as fatigue relevant loading

η_{sc} Factor for fatigue loading

$v_{Ed,max}$ Upper shear stress acting as fatigue relevant loading

Δv_{Ed} Shear stress acting as fatigue relevant loading

5.4 Design shear interface resistance following Hilti Method

As an alternative to TR 066 when an ETA is not available, Hilti supports structural engineers by providing a design method based on Hilti expertise both for static and seismic design. Design resistance follows the equation:

$$v_{Rd} = \underbrace{\mu_h \cdot (\sigma_n + \kappa_{1h} \cdot \rho \cdot \sigma_s)}_{\text{Pull-out}} + \underbrace{\kappa_{2h} \cdot \rho \cdot \sqrt{\frac{f_{yk}}{\gamma_s} \cdot \frac{\alpha_{cc} \cdot f_{ck}}{\gamma_c}}}_{\text{dowel}} \leq \underbrace{\beta_c \cdot \nu \cdot \frac{\alpha_{cc} \cdot f_{ck}}{\gamma_c}}_{\text{concrete strut}}$$

where:

μ_h coefficient of friction

σ_n lowest expected compressive stress resulting from an eventual normal force acting on the interface (perpendicular to the joint surface)

ρ reinforcement ratio of the steel of the shear connector crossing the interface (total cross-section area of connectors / joint surface area)

σ_s steel stress in the shear connector associated to the relevant failure mode

κ_{1h} contribution factor for the friction mechanism according to table below

κ_{2h} contribution factor for the dowel mechanism according to table below

f_{yk} characteristic steel yield strength of the shear connector

γ_s Partial factor for steel

γ_c Partial factor for concrete

α_{cc} is the coefficient taking account of long-term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied

f_{ck} Characteristic compressive cylinder strength of concrete at 28 days

β_c coefficient for the strength of the compression strut depending on the surface roughness category acc. to TR 066 table 2.2

v $0{,}55 \cdot \left(\frac{30}{f_{ck}}\right)^{\frac{1}{3}} < 0{,}55$ coefficient for reduction of concrete strength according to Fib MC2010 7.3-51

In the Hilti Method, only mechanically roughened and smooth Interface are taken in account.

Table 6 gives the parameters needed for the application of the Hilti method.

Tables 6
Parameters needed for the application of the Hilti method

Parameter	Interface characteristics	K_{1h} (6d < h_{eff} < 20d)
Parameter K_{1h} (Static Loading)	Mechanically roughened (≥ 1.5mm)	0.60
	Smooth Interface (< 1,5mm)	0.40
Parameter K_{1h} (Seismic Loading)	**Interface characteristics**	**K_{1h} (10d < h_{eff} < 20d)**
	Mechanically roughened (≥ 1.5mm)	$0.02\frac{h_{eff}}{d} + 0.2$
	Smooth Interface (< 1,5mm)	0.20
Parameter K_{2h}	**Normalized embedment depth**	**K_{2h}**
	$\frac{h_{ef}}{d} > 8$	0.70
	$6 \leq \frac{h_{ef}}{d} \leq 8$	$0.1\frac{h_{eff}}{d} - 0.1$
	$\frac{h_{ef}}{d} = 6$	0.5

5.5 Installation

Proper surface preparation and connector installation is key to fulfill design requirements. Main phases are the following:

1. Removal of damaged concrete/concrete covering
2. Surface preparation through roughening
3. Installation of post-installed connectors as per manufacturer instructions
4. Inspection and on-site pull-out testing if required from the structural engineer
5. Placement of new reinforcement, pre-treatment and pouring of new concrete layer

Surface roughness has a decisive influence on the shear forces that can be transmitted. For design purposes, the characteristic dimension is the mean depth of roughness, R_t, measured according to the sand-patch method [17]. Three commonly used technology to roughen surface are water jetting, sand blasting and grating (Figure 5.4).

Figure 5.4 Methodology to roughen concrete surface

Water jetting **Sand blasting** **Grating**

It is recommended that a mean roughness, R_t, is stipulated when specifying the surface treatment. Prior to approving the treatment, a sample surface must be created and checked using the sand-patch method. An indication of surface roughness is reported in table 7.

Table 7 Indication of surface roughness

Category	Methods/ Situation	Application: Static & quasi-static	Application: Fatigue cyclic loading	Application: Seismic cyclic loading	Peak to mean roughness R_t* [mm]
Very rough	Water jetting, indented	Yes	Yes	Yes (to handle as Rough)	≥ 3,0
Rough	Sand-blasted	Yes	No	Yes (to handle as Rough)	≥ 1,5
Smooth	Untreated, slightly roughened	Yes	No	Yes	< 3,0
Very smooth	Existing concrete cast against steel formwork	Yes	No	No	Not measurable

***Parameter for surface roughness based on volumetric measurement according sand patch method.**
Rt is the mean height based on this measurement. Other methods for determination of the surface roughness maybe used.
Equivalence with the given values of R_t need to be provided.

The connectors must be positioned in the load-bearing direction of the building component with respect to the distribution of the applied shear force in such a way that the shear force at the interface can be constrained and de-bonding of the new concrete overlay prevented.

Pre-treatment is usually done with primer consisting of thick cement mortar.

Before the cement mortar primer is applied, the old concrete should be adequately wetted 24 hours in advance, and thereafter at suitable intervals. Before applying the primer, the concrete surface should be allowed to dry to such an extent that it has only a dull moist appearance.

The mortar used as a primer should consist of water and equal parts by weight of Portland cement and sand of 0/2 mm particle size. This is applied to the prepared concrete surface and brushed in.

The concrete mix for the overlay should be formulated to ensure low-shrinkage (Water-Cement Ratio 0.40). The overlay must be placed on the still fresh primer, i.e., wet on wet.

Careful follow-up is necessary to ensure an overlay of adequate durability. Immediately after placement, the concrete overlay must be protected for a sufficiently long period (at least five days) against drying out and excessive cooling.

5.6 Hilti recommended solutions

Threaded rod HAS-U + RE 500 V4 or HY 200 R V3

Table 8
Recommended solutions for concrete overlays

Hilti System	HUS3-H	Hilti HCC-B + RE 500 V4	Threaded rod HAS-U + RE 500 V4 or HY 200 R V3	Bended rebar + RE 500 V4 or HY 200 R V3
				
Applications recommended	Deck, vault, wall, and beam strengthening and refurbishment	Deck strengthening	Deck strengthening, pillar, foundation strengthening	
Advantages	First solution for jobsite productivity and easiness to install	No need to prepare anchor rods and high performance	High flexibility in embedment depth both in existing and new concrete	
Certification and available designs	Static Seismic	Static Fatigue	Static	Method Static Seismic
Range of diameters	Static [mm] 8 – 10 – 14 Seismic [mm] 10 – 14	Static [mm] 16 Fatigue [mm] 16	Static M10 – M12 – M16	Static from ϕ8 to ϕ20 Seismic From ϕ10 to ϕ20
Design software	PROFIS Engineering concrete overlay module			
Installation phases	1. Surface roughening 2. Hole drilling 3. Drill hole cleaning 4. HUS3-H installation 5. Pour new concrete	1. Surface roughening 2. Hole drilling 3. Drill hole cleaning 4. Connector installation 5. Mortar injection 6. Mortar curing time 7. Pour new concrete		
Comment for installation	No mortar needed and no hole cleaning requirement if concrete hole is ventilated 3 times. HUS3-H installed with impact screwdriver	Hilti SafeSet system with hollow drill bit helps to increase installation productivity		

Simplified overview, detail can be found in the relevant product ETA

6. BASEPLATE FASTENINGS

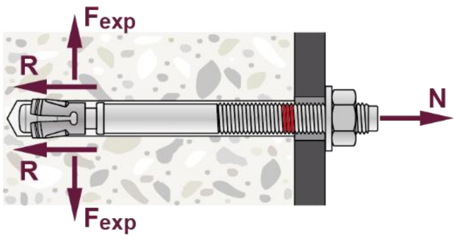
6.1 What are they

Post-installed baseplate fastening applications occur every time a steel element must be fastened to the base material, typically concrete for bridge's applications. Fastening systems cover a wide range of applications where design requirements can be strongly different. Some applications like sound barriers shall consider aspects such as fatigue loads. Others for retrofitting, like the installation of seismic viscous dampers or seismic base isolators or structural baseplates for external post-tensioning, shall take in consideration seismic as a key prerequisite.

There are three basic working principles that make anchor hold in the concrete structure and that identify anchor typology:

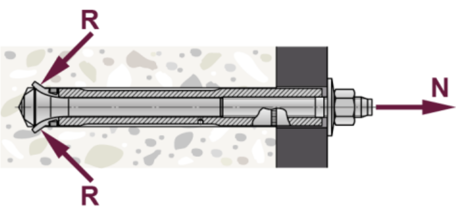
Basic working principles for baseplate fastening

Expansion



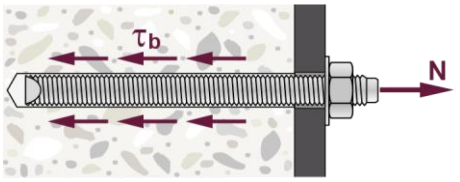
In the case of torque-controlled fasteners a hole is drilled, and the fastener is inserted into the drill hole and anchored by tightening the screw or nut with a calibrated torque wrench. A tensile force is produced in the bolt, the cone at the tip of the anchor is drawn into the expansion sleeve and forced against the sides of the drilled hole. Deformation-controlled anchors comprise an expansion sleeve and cone. They are set in place by expanding the sleeve through controlled deformation. This is achieved either by driving the cone into the sleeve or the sleeve over the cone.

Mechanical interlock



As with cast-in-place systems, undercut anchors develop a mechanical interlock between anchor and the base material. To do this, a cylindrically drilled hole is modified to create a notch, or undercut, of a specific dimension at a defined location either by means of a special drilling apparatus, or by the undercutting action of the anchor itself. In case of self-undercutting the undercut is generated using the expansion element inserted into the pre-drilled hole. Use of rotary-impact action permits the expansion element to simultaneously undercut the concrete and widen to their fully installed position. The cone bolt provides at its end space for the drilling dust which accumulate during formation of the undercut. This process results in a precise match between the undercut form and the anchor geometry.

Bonding



Bonded anchors are available in various systems. A distinction is made between anchors in which the mortar is contained in plastic or glass capsules and injection systems in which the mortar is delivered in cartridges. Irrespective of the system, forces are applied from the threaded rod to the mortar via mechanical interlocking and to the anchor base via micro-interlock, friction and bonding between the mortar and hole wall.

A fourth innovative principle for screw anchors, resulting from expansion principle distributed along the entire anchor length, is gaining popularity thanks to its high performance and installation productivity. Screw anchors are typically hardened to permit the thread to engage the base material during installation. They are installed in drilled holes. They may be driven by means of special impact drivers, or in other systems with a conventional drill equipped with an adapter. The diameter of the drilled hole is matched to the geometry of the screw so that the thread cuts into the concrete and an external force can be transferred to the concrete through this positive interlocking connection.

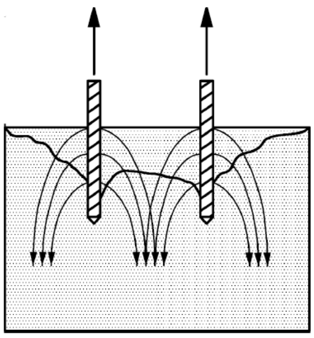
Regardless of the application type and principle, anchor corrosion always affects baseplate fastenings in bridges; for this reason, corrosion is addressed in a self-standing paragraph.

6.2 Design principles

Until 2018, regulatory framework was subject to many updates with a high level of fragmentation among Technical Reports and European Technical Assessment Guidelines. Since 2018, post-installed anchor theory has been consolidated and adopted in the new Eurocode 2 part 4 (EC2-4) "Design of fastening for use in concrete" [18].

EC2-4 provides a design method for fastening that is used to transmit actions to the concrete and refers to the anchor design theory where the concrete tensile capacity is directly used to transfer loads into the existing structure (Figure 6.1).

Figure 6.1
Fastening design theory principle



As a principle [18] base material should be assumed in cracked conditions. Uncracked concrete may be assumed only if it is proven that under the characteristic combination of loading at serviceability limit state, the fastener with its entire embedment depth is located in uncracked concrete. This verification can be performed through the following equation:

$$\sigma_L + \sigma_R \leq \sigma_{adm} \qquad \text{(EC2-4, 4.4)}$$

where:

σ_L is the stress in the concrete induced by external loads including fastener loads.

σ_R is the stress in the concrete due to restraint of intrinsic imposed deformations (e.g. shrinkage of concrete) or extrinsic imposed deformations (e.g. due to displacement of support or temperature variations). If no detailed analysis is conducted, then σ_R = 3N/mm should be assumed.

σ_{adm} the admissible tensile stress for the definition of uncracked concrete.

The design concept for anchors is the same as for any other structural design: design actions must be lower or equal the design resistances. Characteristic values are multiplied by partial safety factors that can be found in the according EC2-4 and in the ETA of the concerning anchor.

6.3 Design under static

The design in accordance with EC2-4 can be applied to both new and existing bridges and viaducts that are covered by EN 1992 (Eurocode 2, concrete structures) and EN 1994 (Eurocode 4, composite structures).

Fastenings can be designed as both single fasteners and groups of fasteners for anchoring in normal concrete, whereby it is assumed that only fasteners of the same type, manufacturer, diameter and anchoring depth are used within a group. For a group of fastenings, the loads are transferred to the individual anchors by means of a common fixture – usually a steel plate. Although the design of the fixture itself is not considered in EC2-4, the design must, nevertheless, correspond to the standard to be applied.

Verification needs to be performed for the two following states:

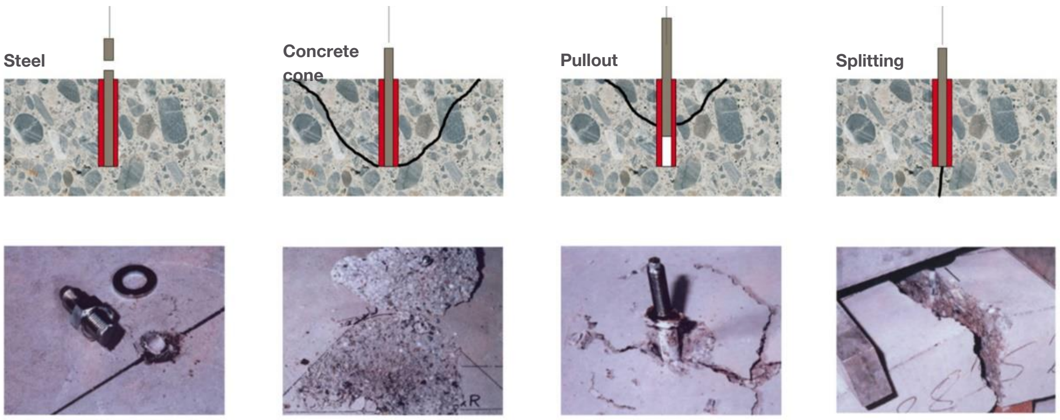
- Serviceability limit state: it shall be shown that the displacement occurring under characteristic actions is not larger than the admissible displacement. The admissible displacement depends on the item to be fastened and must be specified by the structural engineer. The functionality of the component being fastened also needs be observed when subjected to displacement. The characteristic displacements as given in the approval/assessment can generally be interpolated linearly, but in the case of combined tension and shear loads they should be added vectorially.
- Ultimate limit state: it must be shown that the value of the design actions does not exceed the value of the design resistance, whereby the failure mode with the mathematically lowest resistance value is decisive for the design.

Optimum utilization of the fastener is only possible if the design takes into account the loading direction as well as the type of action and differentiates the different modes of failure. For post-installed mechanical and chemical fasteners under tension loads (Figure 6.2), the EC2-4 [18] differentiates between steel failure, pull-out failure, concrete cone failure, and splitting under load and during installation as well as blow-out failures of headed studs near to an edge. For shear loads, the differentiated modes of failure include steel failure (bolt shearing or bending failure), concrete edge failure and pry-out failure. If existing reinforcement in the concrete member should be utilized in the design for the above-mentioned fasteners, such reinforcement also needs to be verified against steel and anchorage failure.

The EC2-4 optimally utilizes the performance capabilities for the given marginal conditions but, can also be considered as relatively complex as the load-bearing capacity of fasteners is described for all loading directions and all modes of failure. In the end, the lowest rated resistance in tension and shear shall be combined.

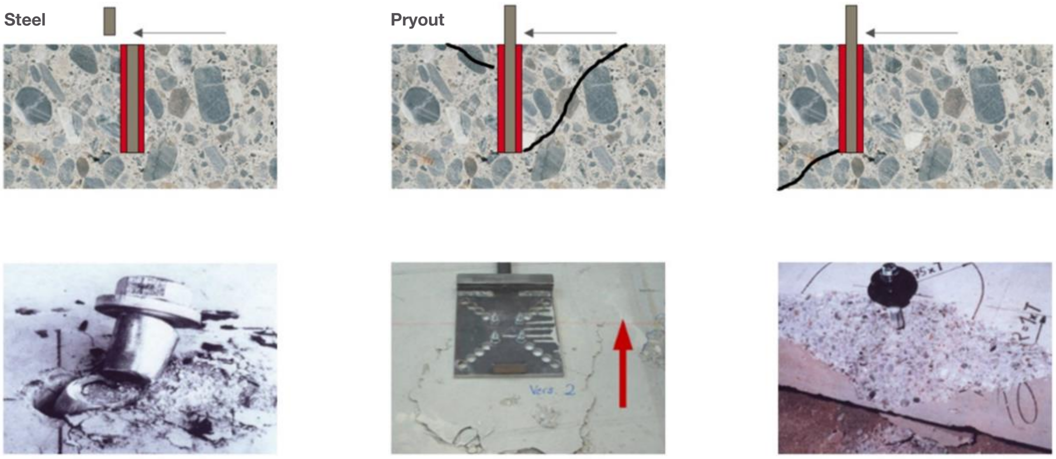
Figure 6.2
Tension and shear failure modes for post-installed anchors

Failure modes under tension loads for post-installed anchors



Failure modes under shear loads for post-installed anchors

Concrete edge



When there is not supplementary reinforcement in the base material, for combined tension and shear loads, verification for steel and concrete failure modes are carried out separately and both shall be fulfilled as follow:

For steel failure of fastener $\left(\frac{N_{Ed}}{N_{Rd,s}}\right)^2 + \left(\frac{V_{Ed}}{V_{Rd,s}}\right)^2 \leq 1$ (EC2-4, 7.54)

with:

N_{Ed}, V_{Ed} respectively resultant design tension and shear force of the fasteners

$N_{Rd,s}$, $V_{Rd,s}$ respectively design steel resistance for tension and shear of the fasteners

For concrete failure $\left(\frac{N_{Ed}}{N_{Rd,i}}\right)^{1,5} + \left(\frac{V_{Ed}}{V_{Rd,i}}\right)^{1,5} \leq 1$ or $\frac{N_{Ed}}{N_{Rd,i}} + \frac{V_{Ed}}{V_{Rd,i}} \leq 1{,}2$ (EC2-4, 7.55, 7.56)

with:

$N_{Rd,i}$, $V_{Rd,i}$ respectively design value of resistance for the most critical tension and shear concrete failure mode of the fasteners

The largest value of $\frac{N_{Ed}}{N_{Rd,i}}$ and $\frac{V_{Ed}}{V_{Rd,i}}$ shall be taken.

6.4 Design under seismic

EOTA TR045 [19] has provided a way to make seismic fastening point calculations since 2013. This technical report was superseded by EN 1992-4 Annex C: Design of fastenings under seismic actions in 2018. Seismic design of fasteners is not completely different from static design therefore changes for static design with EC2-4 are also valid for seismic design. This annex was published on top of EN 1992-4 to define the changes that distinguish seismic design from static.

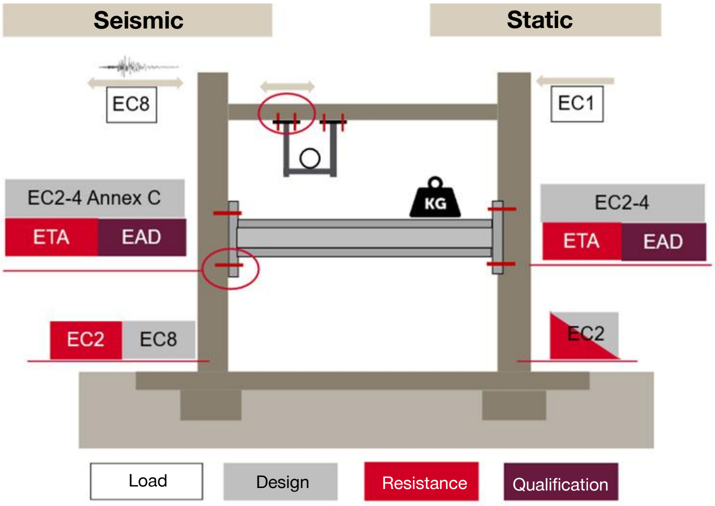


Figure 6.3
Overview of Eurocode system for concrete and anchors

EC2-4 Annex C classifies anchors suitable for use under seismic conditions in two categories: C1 and C2. According to this regulation, anchors without approval for seismic applications should be used only in low seismicity areas, while most seismic areas require use of anchors with seismic performance category C2. Seismic C1 can also be used when the application is confirmed to be a non-structural element without any safety relevance.

In summary, in order to make a seismic design according to C1 category, the following needed:

- Non-structural application in a building that belongs to importance class II or III
- Fastening in seismicity area ($a_g \times s$) between 0.05 and 0.1

If the application does not comply with the specification above, C2 seismic design must be chosen.

Table 9 gives the guideline to select the prerequisite in function of ground acceleration and application type together with building relevance.

Table 9
Guideline to select seismic category

$a_g \times s$	Structural applications: Building IV	Structural applications: Building II, III	Non-structural applications: Building IV	Non-structural applications: Building II, III
0,05 – 0,1 g		ETA C2		ETA C1
> 0,1 g			ETA C2	

EN 1992-4 Annex C [16] defines two additional coefficients to decrease the strength: α_{eq} and α_{gap}.

α_{eq} is the factor that takes into account the influence of seismic actions and associated cracking on (1) concrete cone resistance and bond strength of supplementary reinforcement, and (2) resistance of groups due to uneven load transfer to the individual fasteners in a group

α_{gap} is the reduction factor to consider inertia effects due to an annular gap between fastener and fixture in case of shear loading, given in the relevant European Technical Product Specification

The forces on the fasteners are amplified in presence of an annular gap under shear loading due to a hammer effect on the fastener. For reasons of simplicity this effect is considered only in the resistance of the fastening. In absence of information in the European Technical Product Specification the following values α_{gap} may be used: α_{gap} will be considered 1,0 in case there is no hole clearance between fastener and fixture; however, it will be considered 0,5 for connections with hole clearance. Designer should use seismic filling set to neglect the hammering effect under seismic shear loads.

6.5 100-year service life requirement

In bridges, tunnels, other civil structures, a 100-year service life requirement is placed on the structure in Eurocode 1990. Anchored connections in structures with Design Working Life Category 5 must also satisfy the 100-year requirement.

Until now, chemical anchors were assessed for only a 50-year service life, leaving 100-year requirements to satisfy engineering judgments. With the publication of EAD 330499 [20], anchors may now be assessed for a 100-year service life. Changes for the 100-year prerequisite in the new EAD result in the following considerations:

- **Decreased bond stresses** in some circumstances relating to sustained load, crack cycling, and temperature effects:
  - 0–10% reduction in uncracked concrete
  - 15–40% reduction in cracked concrete
- **Increased long-term displacements**
  - 5–20% increase in uncracked concrete
  - 5–40% increase in cracked concrete

How 100-year values are used in the design:

Design follows the new EC2-4 framework for anchorage in concrete.

Table 10 explains how design values for 100-year requirement are evaluated.

Table 10
Evaluation of design values for 100-year requirement

Service life	50 years	100 years
Design	EC2-4	EC2-4
Based on EAD	330499	330499
Sustained load testing	• 3+ months period under sustained loads • Displacements extrapolated to 50 years and verified against reference	• 6+ months period under sustained loads • Displacements extrapolated to 100 years and verified against reference • 3+ months of stabilized displacements • 100-year displacement increase factor determined if previous assessment per 330499
Crack movement testing	• 1000 crack movement cycles • Displacements must be less than 2 mm in the first 20 cycles, less than 3 mm throughout	• 2000 crack movement cycles • Displacements must be less than 2 mm in the first 20 cycles, less than 3 mm throughout
Freeze-thaw testing	• Cycles of freezing and thawing conducted until displacement is stabilized • Verification of stabilization of displacement	*No additional testing beyond EAD 330499 (Why? The freeze-thaw test was not developed to have an association with service life)*
Exposure to alkalinity and sulfur	Slice tests submerged in alkaline environment and exposed to Kesternich tests	*No additional testing beyond EAD 330499 (Why? These chemical exposure tests were not developed to have an association with service life)*

Other factors that should be considered for service life refer to corrosion:

- Exposure categories
- Steel requirements to prevent corrosion

6.6 Corrosion

Corrosion on steel components in bridges and viaducts exposed to chloride attack from exposure to marine environments or de-icing salts is a significant issue that decreases structural integrity and increases maintenance requirements. Exposure to these conditions require regular inspection, maintenance and rehabilitation, which drastically increases life-cycle costs.

Corrosion protection is the principle measure to mitigate these risks. Active corrosion protection comprises the measures that directly influence the corrosion reaction, e.g., galvanic separation, resistant materials or cathodic protection. Passive corrosion protection prevents or at least decelerates corrosion through the isolation of the metal material from the corrosive agent by the application of metallic or non-metallic protective layers of coating. For post-installed fastening systems there is usually no regular inspection or maintenance, so the use of resistant material or protective coating is the safer and more economical corrosion protection method.

In general, corrosion is expected to occur when the material, the protection or the structural design of a metallic component do not match the requirements by the surrounding environment. To evaluate the risk of corrosion, it is essential to assess the interaction between environmental conditions, material properties, material combinations and design characteristics. To understand this interaction, the following influencing factors to atmospheric corrosion must be considered:

- Humidity: Humidity is a requirement for all atmospheric corrosion reactions.
- Temperature: The higher the temperature, the higher the rate of corrosive attack.
- Salt: Salt-laden air near the seacoast and the salt used for de-icing in winter, typical for bridges and viaducts, accelerate corrosion.
- Industrial pollution: The high content of sulphur dioxide accelerates corrosive reactions.
- Galvanic (contact) corrosion: This form of corrosion is caused by the contact of dissimilar metals (where one metal is less noble than the other).

To help select the right corrosion protection for post-installed anchors, table 11 gives a general guideline for the most common applications for fastening elements.

Table 11
Guide to select corrosion protection for post-installed anchors

Environmental conditions	Fastened part	Carbon steel with or without phosphating	Electro-galvanized	Duplex-coated carbon steel	HDG/ sherardized 45–50 µm	A2 AISI 304	A4 AISI 316	HCR, e.g. 1.4529
Dry indoor	Steel (zinc-coated, painted), aluminium, stainless steel	■	■	■	■	■	■	■
Indoor with temporary condensation	Steel (zinc-coated, painted), aluminium	–	–	■	■	■	■	■
	Stainless steel			–	–			
Outdoor with low pollution	Steel (zinc-coated, painted), aluminium	–	–	□ 2)	□ 2)	■ 2)	■	■
	Stainless steel			–	–			
Outdoor with moderate concentration of pollutants	Steel (zinc-coated, painted), aluminium	–	–	□ 2)	□ 2)	■ 2)	■	■
	Stainless steel			–	–			
Coastal areas	Steel (zinc-coated, painted), aluminium, stainless steel	–	–	–	–	–	■	■
Outdoor, areas with heavy industrial pollution	Steel (zinc-coated, painted), aluminium, stainless steel	–	–	–	–	–	■	■
Close proximity to roads	Steel (zinc-coated, painted), aluminium, stainless steel	–	–	–	–	–	■	■
Special applications		Consult experts						■

■ expected lifetime of anchors made from this material is typically satisfactory in the specified environment based on the typically expected lifetime of a building. The assumed service life in European Technical Assessments is 50 years for concrete anchors, 25 years for power actuated fasteners, steel and sandwich panel screws, and 10 years for flat roof insulation screws.

□ a decrease in the expected lifetime of non-stainless fasteners in these atmospheres must be taken into account (≤ 25 years). Higher expected lifetime needs a specific assessment.

– fasteners made from this material are not suitable in the specified environment. Exceptions need a specific assessment.

1) Outdoor exposure for up to 6 months during construction is permissible for high-strength electro-galvanized siding and decking fasteners such as the X-ENP (see instructions for use for details).

2) From a technical point of view, HDG/duplex coatings and A2/304 material are suitable for outdoor environments with certain lifetime and application restrictions. This is based on long-term experience with these materials as reflected e.g. in the corrosion rates for Zn given in the ISO 9224:2012 (corrosivity categories, C-classes), the selection table for stainless steel grades given in the national technical approval issued by the DIBt Z.30.3-6 (April 2014) or the ICC-ES evaluation reports for our KB-TZ anchors for North America (e.g. ESR-1917, May 2013). The use of those materials in outdoor environments however is currently not covered by the European Technical Assessments (ETA) of anchors, where it is stated that anchors made of galvanized carbon steel or stainless steel grade A2 may only be used in structured subject to dry indoor conditions, based on an assumed working life of the anchors of 50 years.

To perform a corrosion assessment for stainless steel, it is possible to follow the Eurocode 3 (EN 1993-1-4) [21] where, through the definition of the Corrosion Resistance Factor, it is possible to evaluate the Corrosion Resistance Class for the stainless steel grade suitable for the application.

6.7 Installation

The Eurocode 2 part 4 Annex F [18] indicates clear instructions for anchor installation. The code emphasizes not only the anchor setting but also the concrete hole execution and cleaning phase. It is required that the instructions provided by the manufacturer are followed and reported in the relevant product ETA.

Main phases when installing post-installed anchors:

1. Concrete hole execution as per ETA indication. Possible methodologies to execute the hole:

   a) Perforation through hammer drill set in rotation-hammer mode. The hole surface will be rough.

Example of hammer drill bit

   b) Perforation through hollow hammer drill bit in rotation-hammer mode attached to vacuum cleaner. The bore hole surface will be rough. This drilling system automatically removes dust and cleans the hole while hammering. Additional cleaning phase won’t be necessary.

Example of hollow drill bit (Hilti SafeSet system)

   c) Perforation through diamond coring attached at diamond coring machine in rotation mode. The hole will be perfectly smooth. This system helps avoid vibration and execute large diameters with relevant embedment depth.

Example of diamond coring bit

   d) To increase performances, it is possible to roughen the hole after the diamond coring by using a specific drill bit attached to a hammer in rotation-hammer mode.

Example of roughening tool (Hilti SafeSet system)

2. Concrete hole cleaning as per relevant ETA. Cleaning phase is not required when:
   a) Hollow drill bit of Hilti SafeSet system is used
   b) Hilti HIT-Z special rods is used together with vinilestere mortar HY 200 A:

HIT-Z rod

HY 200 A

   c) Hilti HUS3-H mechanical screw anchor when 3x ventilations after drilling are executed and anchor is set in vertical upward orientation:

HUS3-H screw anchor

3. Mortar injection: only for chemical systems
4. Installation of the steel element
5. Curing time: only for chemical systems
6. Application of torque requirement. Torque not needed for mechanical screw anchor HUS3.

Reinforcement near the hole position should not be damaged during drilling. In pre-stressed concrete elements the distance between the drilling hole and the pre-stressed reinforcement shall be at least 50mm; for determination of the position of the pre-stressed reinforcement in the structure a suitable device, e.g., a reinforcement detector, may be used.

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