SEISMIC DESIGN AND ASSESSMENT OF POST INSTALLED REBAR CONNECTIONS FOR RETROFITTING OF STRUCTURES

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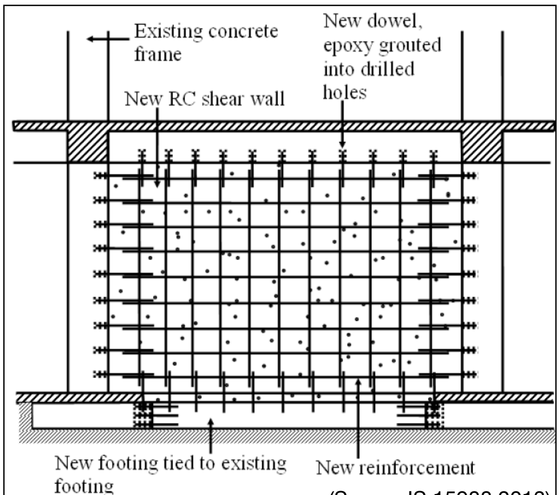
INTRODUCTION

Earthquake, for sure, is one of nature's most unpredictable hazard. Over the past so many years, we have witnessed a staggering rise in seismic activity all over the globe. In India, we are sitting on a seismically active zone, with more than 59% of the land being susceptible to earthquakes (as per the Vulnerability Atlas of India [1]). In India and its neighboring countries (within 300 km), there have been more than 2700 incidents of earthquakes of magnitude 4 and above which have recorded over the last 10 years, making it an average of 22 earthquakes per month. While standards are being upgraded continuously to take into consideration higher seismic performance of structures, there is also a need to strengthen many existing structures to meet the current seismic demand. Also, it may be required to address the deficiency of multiple structures arising out of several reasons like ageing, change of usage, increase in the load, construction errors, etc.

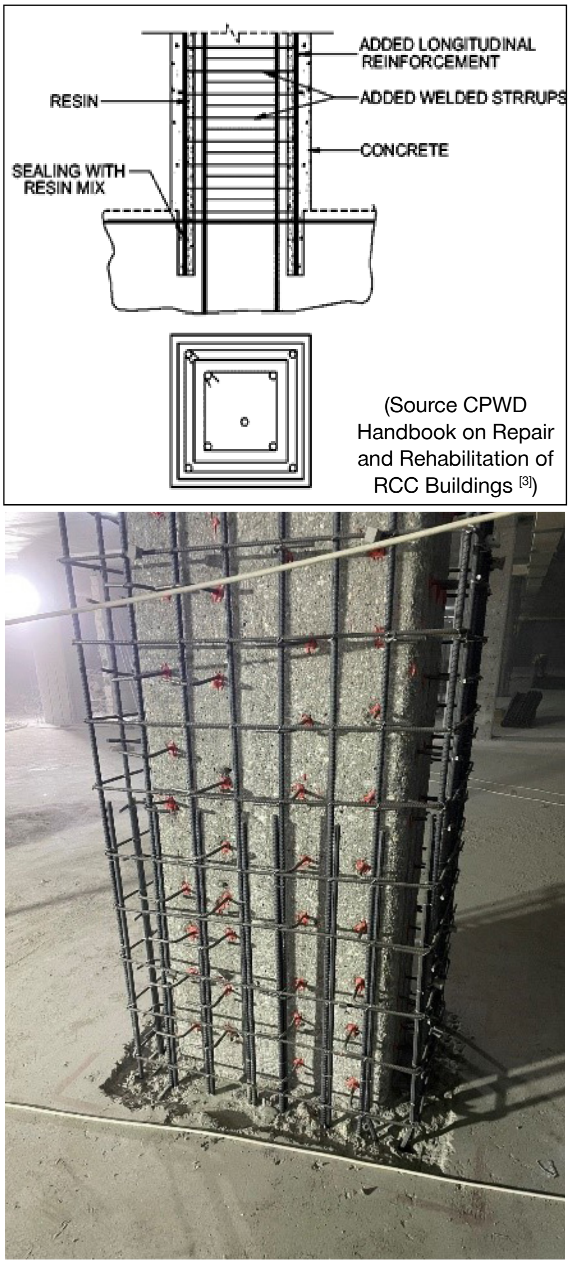
The seismic retrofitting of structures can be done either by increasing the seismic capacity (e.g., stiffening existing structures, strengthening the members, enhancing ductility, reducing irregularity) and/or by improving the seismic ductility of the structure (i.e., strengthening vs. brittle failure mechanisms). Other advances techniques aim the reduction of the seismic demand on the building (i.e., isolating the structure or introducing damping elements). Strengthening techniques may include interventions at a global level (e.g., addition of shear walls or bracing, thickening of walls, base isolation, etc.) or at member level (strengthening of deficient members like jacketing of columns or beams, strengthening of foundations, etc.) (Fig. 1 and Fig. 2).



(Source IS 15988:2013)

Fig. 1: Retrofitting of Structure using Shear Wall

When existing reinforced concrete member need to be connected to new elements or additional concrete is needed to increase sections of existing members the use of post installed reinforcing bars (rebars) becomes an integral part of the application. The application being critical to ensure desired seismic performance of the structure, a basic requirement is also to ensure that the connection to the existing concrete member also has adequate seismic performance during earthquake events.



(Source CPWD Handbook on Repair and Rehabilitation of RCC Buildings [3])

Fig. 2: Strengthening of structures using concrete jacketing

Post-installed rebar connections involve installation of deformed reinforcing bars in holes drilled in concrete filled with injectable mortars. The reinforcing bars are embedded in adhesives in holes drilled into existing concrete member and are cast in new concrete on the other side (Fig. 3a). In concrete-to-concrete connections using post-installed technology, the bars are typically embedded as required to develop the tension yield strength of the reinforcing steel. The fundamental principle of any post installed concrete to concrete connection is that it should at least behave as a cast-in connection.

To this end the performance of the mortar used and its interaction with the reinforcing bars and the concrete is of key importance (Fig. 3b).

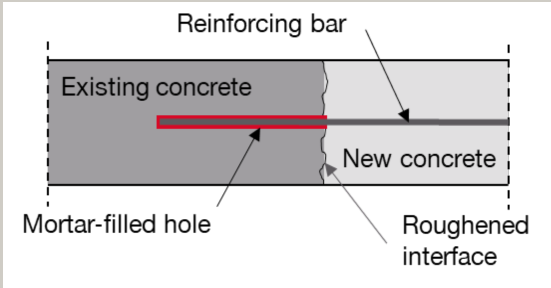


Fig. 3a - Post-installed rebar application (typ.)

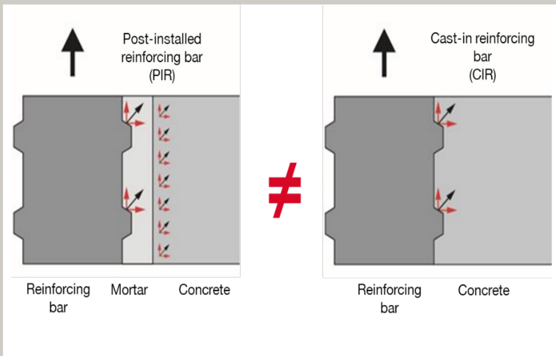


Fig. 3b - Difference between cast-in and post-installed rebars in terms of loading transfer mechanism

Fig. 3: Post-installed reinforcing bars embedded in concrete

FUNDAMENTAL OF QUALIFICATION AND DESIGN POST-INSTALLED REBARS

There are always three imperatives to ensure safety of any post installed connection – product assessment/qualification, correct design, and proper installation.

As a result of extensive research and development over the last 3 decades, we have seen a lot of progress in terms of parallel evolution of qualifications and design provisions (Fig. 4).

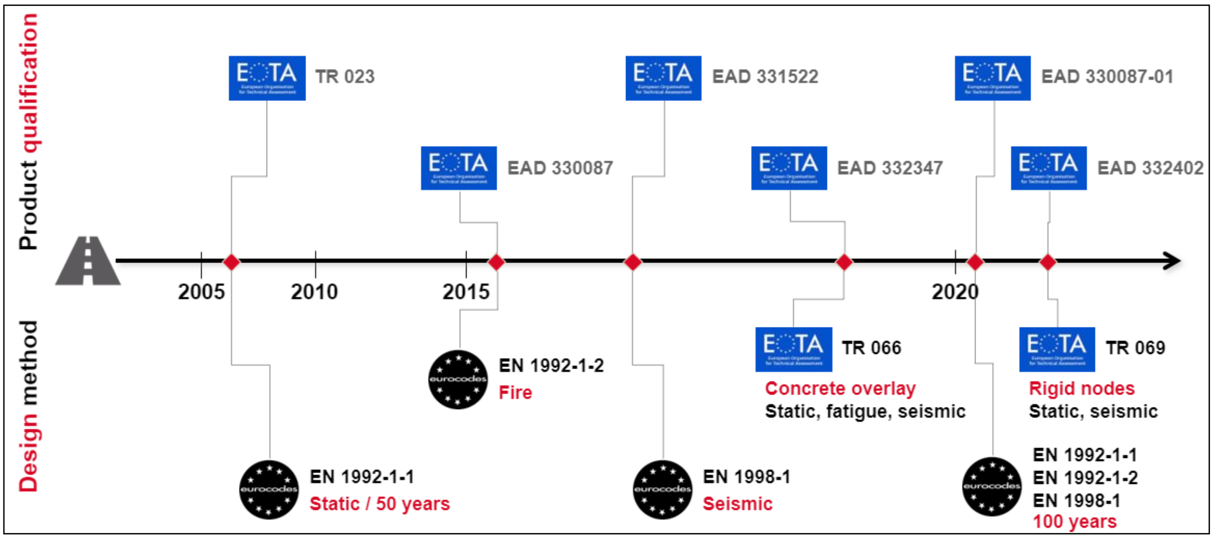


Fig. 4: Development of design standards for post installed rebar connections

A post installed rebar connection can be broadly classified as end anchorage (Fig. 5), splice connection (Fig. 6) and shear connectors or concrete overlays (Fig. 7)

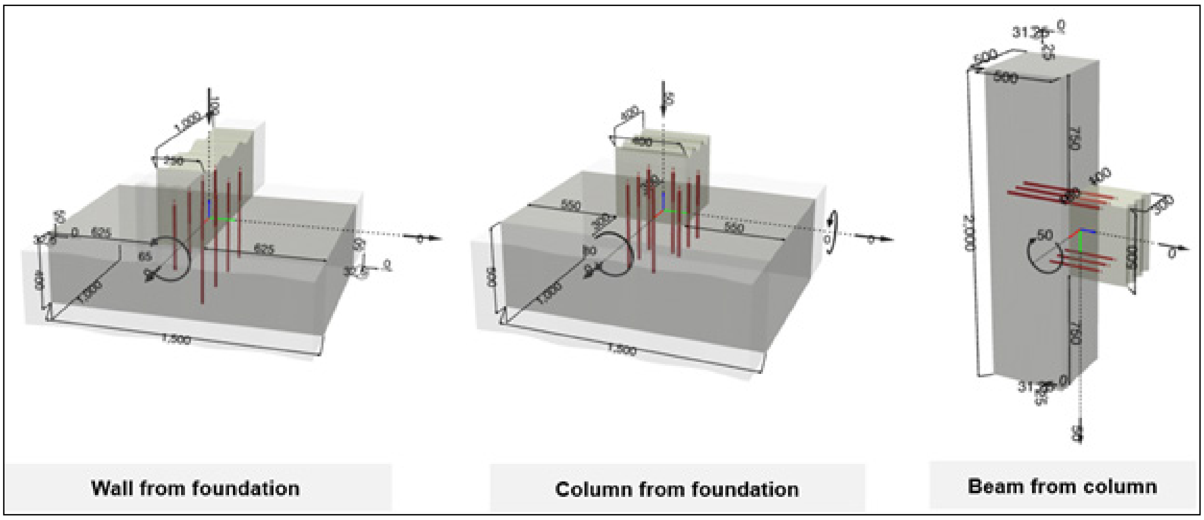


Fig. 5: Examples of end anchorage for post installed rebar connection

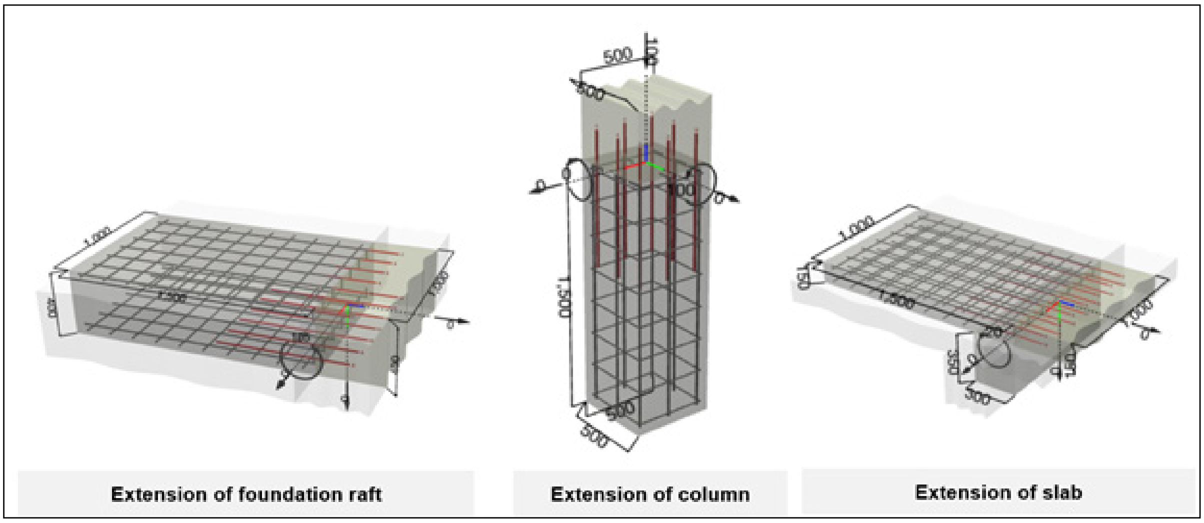


Fig. 6: Examples of splice connections for post installed rebars

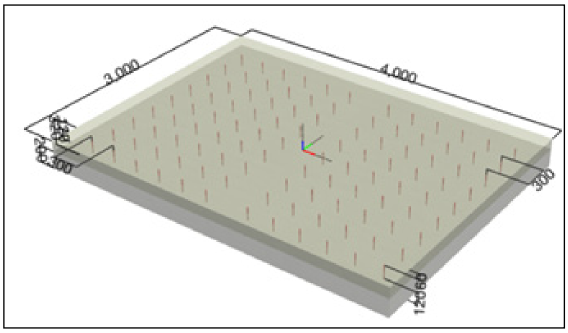


Fig. 7: Examples of shear-friction applications

REGULATORY FRAMEWORK FOR ASSESSMENT/ QUALIFICATION OF POST INSTALLED REBAR CONNECTIONS –

Assessment of post-installed rebars for equivalency to cast-in bars

To allow the use of post-installed reinforcing bar systems, verification of the compatibility of the post-installed bars with existing and neighboring cast-in bars in terms of strength, stiffness, and serviceability is required. Refer to Spieth, 2002 [4] and Genesio et al. (2017) [5] for more details and the scientific background. Furthermore, the performance of post-installed reinforcing bars is strongly linked to the mortar performance and its robustness in different installation conditions (e.g., temperature, humidity) as well as being sensitive to jobsite conditions (e.g., improper hole cleaning or/ and injection, corrosive environment), loading conditions (e.g., freeze-thaw cycles, sustained loading at high temperature, cyclic seismic loading), quality and type of equipment used for installation, and depth and diameter of the application. All these considerations point to the necessity for appropriate product qualification requirements aiming at ensuring that the behavior and performance of a post-installed reinforcement connection is similar to that of cast-in one.

Over the past three decades, extensive research work has led to the development of qualification procedures for the post installed connections, to prove their equivalence to cast-in rebars in terms of load vs. displacement behavior, bond-splitting resistance and robustness as related to installation, environmental, and loading conditions. The European Assessment Document (EAD) 330087 [6] issued by the European Organization for Technical Assessment (EOTA) provides comprehensive guideline in terms of performance assessment under static loading, fire exposure and seismic loading.

A post-installed rebar system assessed according to EAD 330087 [6] can be used following the principles of the reinforced concrete design standards EN 1992-1-1 [7] and EN 1998-1 [8] for the calculation of lap splices (Fig. 6) and anchorage lengths (Fig. 5) of longitudinal reinforcement as well as shear-friction applications, when rebars are used as dowel (Fig. 7).

Typically, the same equations are valid for both static and seismic design (refer to EN 1998-1 [8], sect. 5.6). However, for seismic design, additional requirements for reinforcement detailing are usually provided. These include increase of anchorage length to account for steel yielding and strain penetration at the onset of potential plastic hinges. Also, it takes into consideration the inclusion of seismic hooks at the end of anchorage bars to improve the confinement of the nodal zone as well as to guarantee a more stable cyclic behavior, where a sufficient straight anchorage length cannot be provided. These requirements are mainly motivated by the need to avoid a possible pullout failure.

In regions where enough confinement of the tensioned bar(s) cannot be provided, radial stresses may induce splitting cracks in the cover and/or between bars located on the same splitting plane and reduce their pullout resistance. The assessment of post-installed

> “The behavior and performance of a post-installed reinforcement connection is similar to that of cast-in one”

reinforcing bars under cyclic (seismic) loading is conducted by comparing the performance of the system in the near (splitting failure) and far (pullout) edge conditions The adhesive used must ensure that under cyclic loading the performance of the system is still equivalent to a cast-in bar.

EAD 330087[6] provides a comprehensive test protocol for seismic testing of post installed rebars which include bond strength under seismic loading and test for minimum concrete cover. Technical details and scientific background are provided by Simons (2007) [9] and Genesio et al. (2019) [10].

To evaluate the behavior under cyclic loading in far-edge conditions tests are performed in displacement controlled set-up using constant slip protocol, which consists of application of ten displacement cycles between a specific value of push and pull, followed by residual tension load test. The limiting value of displacements for the load cycles shall be 1.5 mm for diameter of rebar less than 25 mm, 2.0 mm for rebar diameter between 25 mm and 40 mm and 3.0 mm beyond 40 mm rebar diameter.

The cyclic behavior of post-installed reinforcing bars has been extensively investigated by Simons (2007) [9] using the same testing and assessment procedures developed by Eligehausen et al. (1983) [11] to investigate the bond strength of cast-in bars degradation under cyclic loading. The reference bond degradation curve (i.e., bond strength measured at the cycle n vs. bond strength at first cycle) for cast-in bars is the black dashed line shown Fig. 8b), which is valid for ten "push/pull" cycles between $\pm s_u$, where su corresponds to the displacement at peak load measured in reference monotonic pullout test with confined setup. This curve fits rather well the test results of Eligehausen et al. (1983) [11] as well (Fig. 8a).

It is worth mentioning that this loading protocol does not reflect the real seismic demand on a reinforcing bar, but a well reproducible and idealized condition under which the bond strength degradation of a post-installed bar system can be conservatively assessed and compared with the performance of cast-in bars.

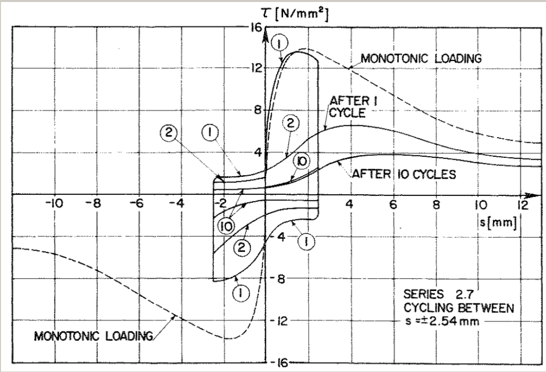


Fig. 8a - hysteretic behavior of cast-in reinforcing bars (typ), Eligehausen et al. (1983) [8]

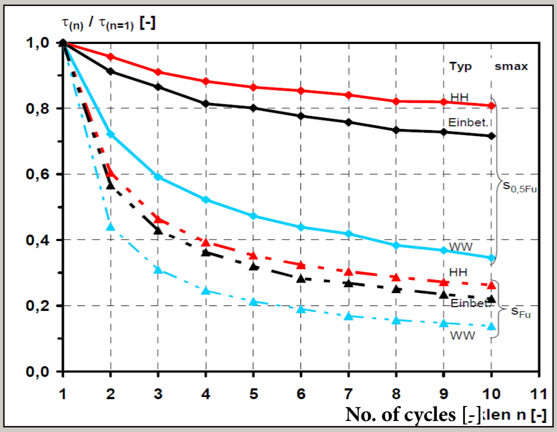


Fig. 8b - comparison of bond strength degradation of cast-in vs. post-installed reinforcing bars, Simons (2007) [9]

Fig. 8: Cyclic load protocol and assessment for post-installed reinforcing bar seismic qualification

Further, for seismic loading, cyclic tests are conducted to determine the splitting resistance of post installed rebars, as this failure mode is likely to be decisive in near edge conditions and in presence of dense reinforcement. The tests are performed using the Beam End Test set-up (BET) and unconfined set-up (Fig. 9). Details and validation of this specimen and setup are discussed by Rex et al. (2018) [12]. The test shall be performed in displacement control with increasing slip protocol (ISP) (see Fig. 8), which consists of application of three displacement cycles between 0 and maximum axial displacement (i.e., at pull-out) followed by a residual tension test. The maximum axial displacement shall be derived from monotonic tests with cast-in rebar. The assessment is based on the comparison between the cyclic performance of the post-installed reinforcing bars and the monotonic load-displacement behavior of cast-in bars as related to peak strength, dissipated energy calculated as the area below the cast-in bar monotonic curve and the envelope of the hysteretic curves obtained with post-installed reinforcing bars and residual resistance at maximum axial displacement.

Fig. 9a - Typical BET specimen

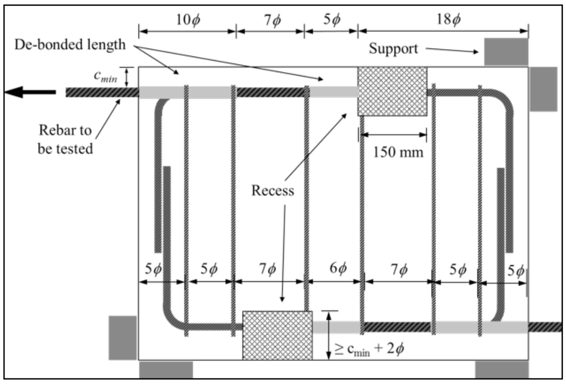


Fig. 9b - Schematic of BET specimen suitable for testing of post-installed rebars

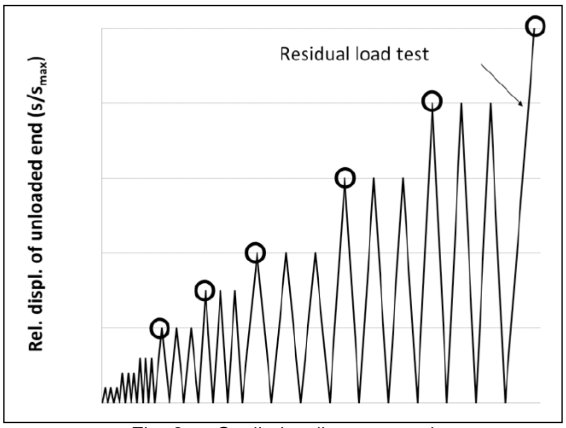


Fig. 9c - Cyclic loading protocol

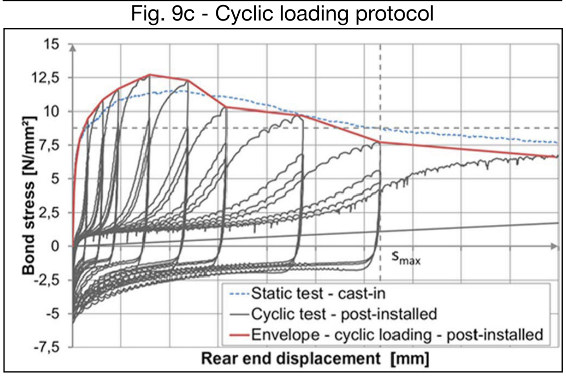


Fig. 9d -Typical cyclic response of post-installed rebars compared to cast-in bar

Fig. 9: Beam End Test (BET) Set-up (Source EAD 330087 [6])

Assessment of product specific performance of post-installed rebars

Research has shown that post installed rebars, in end anchorages, can behave better than cast-in rebars if high strength mortar is used, but this could never be leveraged owing to the design limitation where the designer was restricted to use the bond strength of cast-in bars. Experimental evidence (Rex et al., 2018 [12]) has demonstrated that the bond strength of high performing mortar system allow the increase of splitting dominant field beyond the ratio $c_d/\phi > 3$ (Fig. 10a). At the same time, it is important to highlight that the difference in bond strength between post-installed and cast-in bars decreases with increasing anchorage length (Fig. 10b) due to the shear lap effect. On this research basis, the EOTA has developed the EAD 332402 [13], [14] and [15] that establishes the rules for the assessment of enhanced bond-splitting performance of post installed systems, following the principles explained in the fib Model Code 2010 [16]. It covers both static and seismic loading, for a design working life up to 100 years.

The basic assessment mainly consists of:

1. derivation of bond-splitting equation and assessment of all relevant products parameters as function of: concrete strength f_{ck}, bar diameter ∅, minimum cover c_d, maximum cover c_{max} as defined in the fib Model Code 2010 [16] with a beam-end-tests similar to the one shown in Fig. 10
2. Assessment of the bond strength degradation with increasing anchorage length.
3. Pullout strength assessed according to the EAD 330499 [17] as upper limit of the splitting resistance (Fig. 11) including the sensitivity to cracked concrete, temperature, sustained load as well as other environmental and loading influencing factors.

The seismic assessment follows the principles explained in the previous section of this paper with the difference that the benchmark behavior is not the cast-in bar anymore, but the static performance of the post-installed rebar system under consideration (refer also to Cattaneo et al., 2023 [18]).

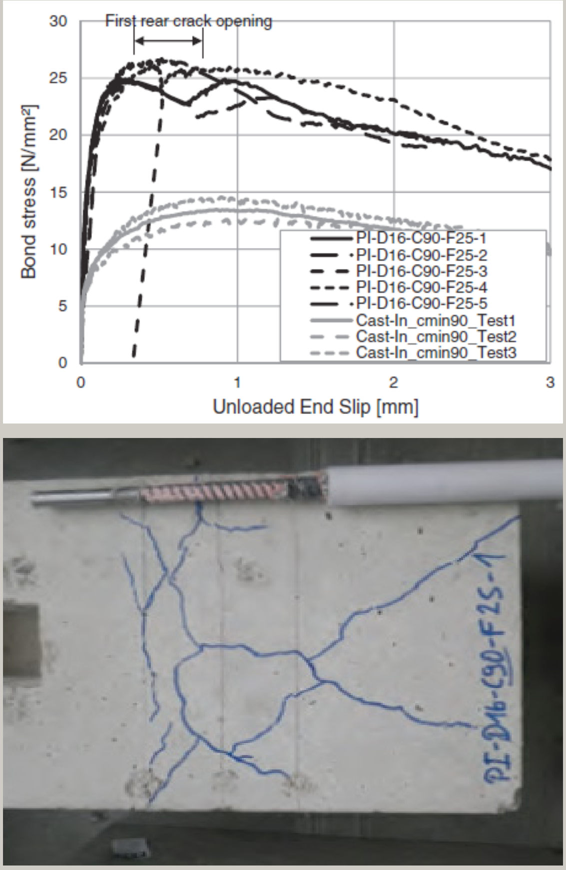


Fig. 10a - BET with small anchorage length *(7∅) and $c_d / \emptyset \approx 5.6$)*

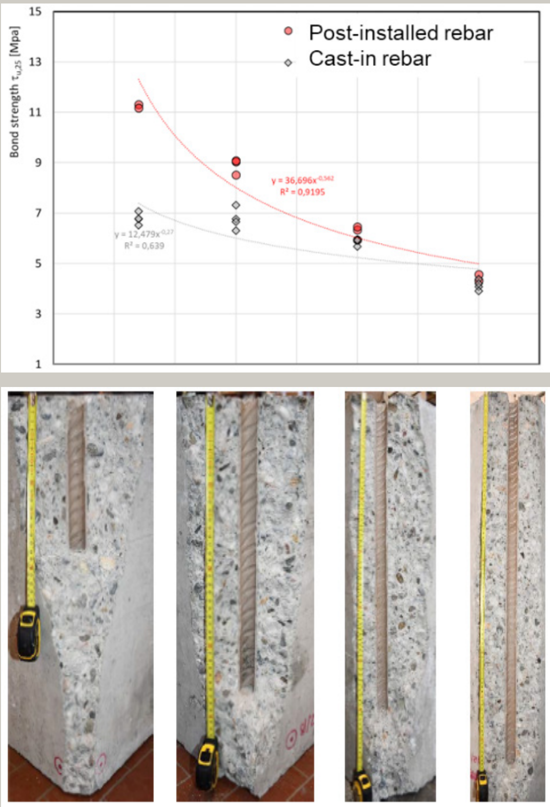


Fig. 10b - Influence of anchorage length on bond strength

Fig. 10: Experimental evidence of superior bond strength of post-installed vs. cast-in rebars

The bond-strength of a post-installed rebar system assessed according to the EAD 332402 [13], [14] and [15] is schematically shown in Fig. 12.

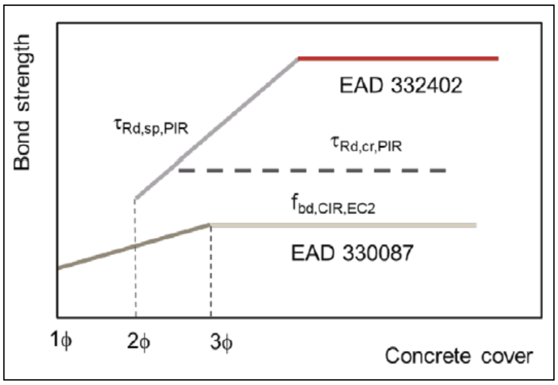


Fig. 12a - Bond strength as function of the concrete cover

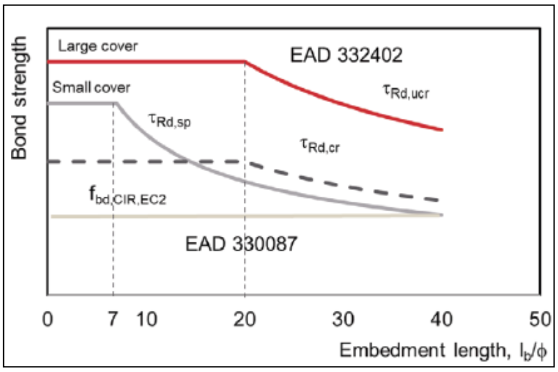


Fig. 12b -Bond strength as function of the anchorage depth

Fig. 12: Influence of confinement and anchorage depth on bond splitting resistance of post-installed rebars assessed according to the EAD 330087 and EAD 332402

Design as equal to cast-in

EAD 330087[6] covers post installed connections designed in accordance with EN 1992-1-1 [7] for design of concrete structures. The standard covers the design provision for calculation of anchorage and lap splices lengths for connections with cast-in rebars. With the basic assumption that post-installed connections should behave at least as cast-in connections, the provisions of EN 1992-1-1 [7] can be extended for design of post installed connections with a few modifications. Fundamentally, the design of anchorages and lap splices as per Eurocode, has the following formulation

$$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,reqd} \geq \alpha_{lb} \cdot l_{b,min}$$

Where,

- α_1 is for the effect of the form of the bars assuming adequate cover
- α_2 is for the effect of concrete minimum cover

- α_3 is for the effect of confinement by transverse reinforcement
- α_4 is for the influence of one or more welded transverse bars along the design anchorage length l_{bd}
- α_5 is for the effect of the pressure transverse to the plane of splitting along the design anchorage length
- α_6 is for the percentage of lapped reinforcement (not applicable for end anchorages)
- $l_{b,min}$ is the minimum anchorage length which is multiplied with the factor

$1.0 \leq \alpha_{lb} \leq 1.5$ that takes into account the product dependent sensitivity to cracked concrete of a post-installed rebar system as reported in the European Technical Assessment (ETA).

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd,PIR}}$$

σ_{sd} is the tension stress to be anchored. In seismic applications this value is usually taken as the yield strength (f_{yd}) multiplied by an overstrength factor $\gamma_{Rd} \geq 1.0$ according to EN 1998-1.

$f_{bd,PIR}=k_b \cdot f_{bd} \leq f_{bd}$ (f_{bd} according to EN 1992-1-1 [7] and $0.7 \leq k_b \leq 1.0$ according to the relevant ETA) is the bond strength of the post-installed rebar system. Note that this value must be replaced by $f_{bd,seis}$ in seismic applications. Both $f_{bd,PIR}$ and $f_{bd,seis} \leq f_{bd,PIR}$ are reported from the relevant ETA for a specific rebar diameter, concrete strength class and drilling method.

Summarizing, according to EN 1992-1-1 [7], increasing concrete confinement results in utilizing the higher bond strength resulting in decrease in anchorage length until pull-out is reached (confinement of $c_d = 3\phi$) (Fig. 11). According to this approach, the design adequacy is checked for splitting (formation of radial cracks due to exceeding of tensile strength of the concrete around the rebar due to small cover or spacing), pull-out of rebar via shearing off concrete between the ribs and yield strength of the reinforcing bar (limiting the capacity of the connection).

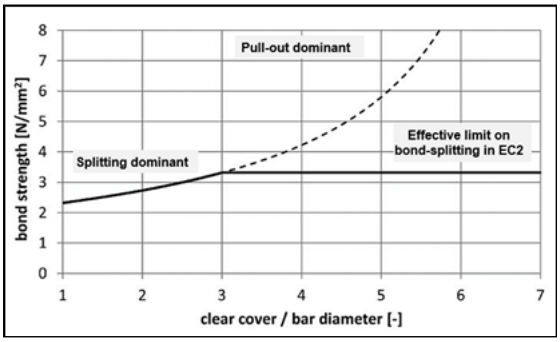


Fig. 11 Effective limit on bond splitting resistance as per EN 1992-1-1 [7]

Design accounting for specific post-installed rebar product performance

The introduction of EAD 332402 [13], [14] and [15] has established a comprehensive assessment of the product dependent bond-strength of post-installed rebars. A product with an ETA according to this EAD can be used for a design of end anchorages according to the EOTA Technical Report (TR) 069 [19]. The TR 069 [19]

Table 1. Values of coefficients α_1 to α_6 for cast-in as per EN 1992-1-1 [7] and post-installed rebars qualified as per EAD 330087 [6]

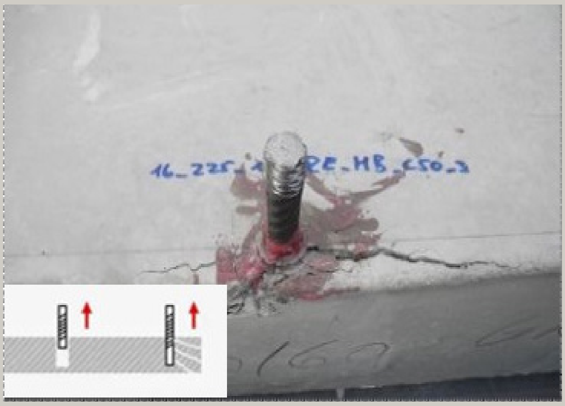
FACTOR	TYPE OF ANCHORAGE	CAST-IN REBAR		POST-INSTALLED REBAR	
		TENSION	COMPRESSION	TENSION	COMPRESSION
Shape of bar	Straight	$\alpha_1 = 1.0$	$\alpha_1=1.0$	$\alpha_1 = 1.0$	$\alpha_1=1.0$
	Hooked, bends	$\alpha_1 = 0.7$	$\alpha_1=1.0$	$\alpha_1 = 1.0$	$\alpha_1=1.0$
Concrete cover	All types	$0.7 \leq \alpha_2 \leq 1.0$	$\alpha_2=1.0$	$0.7 \leq \alpha_2 \leq 1.0$	$\alpha_2=1.0$
Confinement by transverse reinforcement	All types	$0.7 \leq \alpha_3 \leq 1.0$	$\alpha_3=1.0$	$0.7 \leq \alpha_3 \leq 1.0$	$\alpha_3=1.0$
Welded reinforcement	All types	$\alpha_4 = 0.7$	$\alpha_4=0.7$	$\alpha_4 = 1.0$	$\alpha_4=1.0$
Confinement by transverse pressure	All types	$0.7 \leq \alpha_5 \leq 1.0$	$\alpha_5=1.0$	$0.7 \leq \alpha_5 \leq 1.0$	$\alpha_5=1.0$
Percentage of lapped bars in the critical section	All types	$1.0 \leq \alpha_6 \leq 1.5$	$1.0 \leq \alpha_6 \leq 1.5$	$1.0 \leq \alpha_6 \leq 1.5$	$1.0 \leq \alpha_6 \leq 1.5$

Note: $\alpha_2 \cdot \alpha_3 \cdot \alpha_5 \geq 0.7$

is a guideline that includes provisions for the design of anchorages with post-installed rebars in moment resisting connections accounting for the product dependent bond-splitting performance.

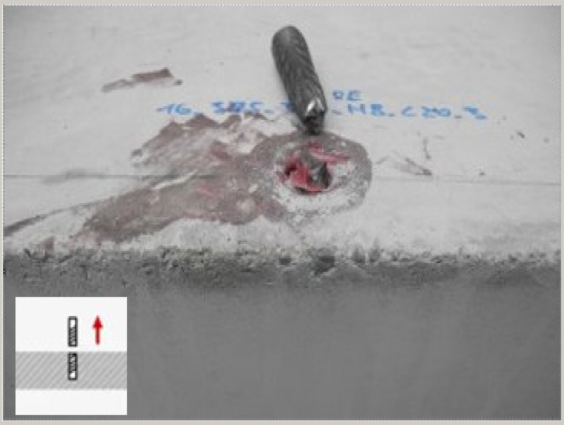
The design as per TR 069 [19] follows the logic of limit state design. The approach is based on the establishment of a hierarchy of strengths between steel yielding ($N_{rd,y}$), concrete breakout ($N_{Rd,c}$), and bond-splitting ($N_{Rd,sp}$) (Fig. 13).

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



Limit of bond-splitting

Limit of concrete cone failure



Limit of bar yeilding

Fig. 13: Failure modes as per TR 069 [19] design

The design yield resistance of the tension reinforcing bars ($N_{Rd,y}$) is calculated as follows –

$$N_{Rd,y} = f_{yk} \cdot A_s / \gamma_s$$

Where, A_s is the cross sectional area of tensioned reinforcing bars, f_{yk} is the characteristic steel yielding strength; γ_s is the steel partial factor.

For the calculation of the design concrete breakout resistance ($N_{Rd,c}$), the provisions of EN 1992-4 [20] are followed –

$$N_{Rd,c} = \frac{N^0_{Rk,c} \cdot \frac{A_{cN}}{A^0_{cN}} \cdot \psi_{s,N} \cdot \psi_{ec,N} \cdot \psi_{re,N} \cdot \psi_{MN}}{\gamma_{mc}}$$

$$N^0_{Rk,c} = k_1 \cdot f_{ck}^{0.5} \cdot l_b^{1.5}$$

where: k_1 =7.7 or 11.0 for cracked or uncracked concrete, respectively, f_{ck} is the characteristic concrete compressive strength; l_b is the anchorage length of the reinforcing bar.

A_{cN}/A^0_{cN} takes into account the geometric effect of axial spacing and edge distance, $\psi_{s,N}$ is the factor for the disturbance of the distribution of stresses in the concrete due to the proximity of an edge of the concrete member, $\psi_{re,N}$ is the factor for the effect of dense reinforcement, $\psi_{ec,N}$ considers the load eccentricity and ψ_{MN} is the positive effect of a compression force in case of bending moments, with or without axial force.

The design bond-splitting resistance ($N_{Rd,sp}$) is calculated by considering a uniform bond strength distribution and using the analytical formulation derived from the fib Model Code 2010 [16] and qualitatively shown in Fig. 12 to define the splitting strength $\tau_{Rk,sp}$ with its influencing parameters (concrete strength f_{ck}, bar diameter ∅, minimum cover c_d, maximum cover c_{max} as defined in the fib Model Code 2010 [16] and the anchorage length l_b)
The factor A_k and the exponents s_p1, s_p2, s_p3, s_p4 and lb1 are product dependent parameters to be taken from the relevant ETA.

$$\tau_{Rk,sp} = \eta_1 \cdot A_k \quad \left(\frac{f_{ck}}{25}\right)^{sp1} \cdot \left(\frac{25}{\phi}\right)^{sp2} \cdot$$

$$\left[\left(\frac{c_d}{\phi}\right)^{sp3} \cdot \left(\frac{c_{max}}{c_d}\right)^{sp4} + k_m K_{tr}\right] \cdot \left(\frac{7\phi}{l_b}\right)^{lb1}$$

$$\tau_{Rk,sp} \leq (\tau_{Rk,ucr} \cdot \Omega_{cr} \cdot \psi_{sus})\ for\ 7\phi \leq l_b \leq 20\phi$$

$$or\ \leq\ (\tau_{Rk,ucr} \cdot \left(\frac{20\phi}{l_b}\right)^{lb1} \cdot \Omega_{cr} . \psi_{sus})\ for\ l_b \geq 20\phi$$

$\tau_{Rk,ucr}$ is the pullout resistance of the system assessed according to the EAD 330499 [19]. The factors Ω_{cr} and ψ_{sus} quantify its sensitivity to cracked concrete (0.3 mm) and sustained load, respectively.

Seismic design considerations –

For seismic design, the verification follows

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

This means that yielding of steel should always be reached before any other (brittle) failure modes. However, in many cases, concrete breakout or splitting failures govern and thus in such cases, it is left to the designer to accept either splitting or concrete breakout as decisive failure mode if the predicted plastic mechanism of the structural system is ductile at a demand level at which the connection with post-installed rebars designed is still elastic.

For steel yielding, $N_{Rk,y,eq} = \gamma_{Rd} \cdot N_{Rk,y}$

Where γ_{Rd} is the overstrength factor related to the level of ductility for which the connection is designed according to EN 1998-1[8].

For concrete breakout, the following shall be considered – $N_{Rd,c,eq} = \alpha_{eq} \cdot N_{Rd,c}$

α_{eq} = 1 if the width of crack is equal to 0.3 mm
α_{eq} = 0.85 if the width of the crack is greater than 0.3 mm

The reduction factor α_{eq} = 0.85 is in line with the provision of EN 1992-4 [20] for single anchors and hence considering the effect of large crack width. No additional reduction for rebar groups, because is unlikely that tension rebars will experience different crack widths. For static loading conditions, a crack width of 0.3 mm can be assumed for designing. However, for seismic loading conditions, the expected crack widths can exceed the crack width limits given by EN 1992-1-1 [7] and reach crack widths of up to 0.8 mm. The maximum expected crack width in a connection is strongly affected by the overall behavior of the structure and is influenced by several factors such as the deformability of the existing member, the geometry of the connection, the design assumptions and the structural detailing of reinforcement bars. Generally, larger cracks are associated with connections that are designed to undergo larger deformations during a seismic event. Note that 0.8 mm is the upper limit of a flexural crack width prior to cross-section plasticization according to EN 1992-4 [20].

Furthermore, the ratio between anchorage length and thickness of the existing member is taken into account allowing the assumption of smaller crack widths for cases where the anchorage length is extended to approximately the entire thickness of the member. In such situations, practically, part of the anchorage is located in the compression zone and, therefore, the average crack width can be considered being smaller.

The resistance corresponding to pull-out and splitting failure is calculated as follows –

$$\tau_{Rk,sp,eq} = \alpha_{eq,sp} . \tau_{Rk,sp}$$

$$\tau_{Rk,sp,eq} \leq (\tau_{Rk,ucr} \cdot \Omega_{cr,eq} . \alpha_{eq,p})\ for\ \ 7\phi \leq l_b \leq 20\phi$$

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

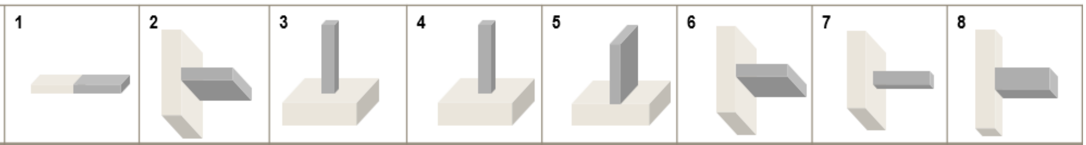
$\alpha_{eq,sp}$, $\Omega_{cr,eq}$ and $\alpha_{eq,p}$ are product dependent factors to be obtained from the ETA certificate of the post-installed rebar system assessed to resist seismic actions.

The splitting strength is reduced by the factor $\alpha_{eq,sp}$ due to seismic action accounting for the different energy dissipated in monotonic or cyclic loads. The factor $\alpha_{eq,p}$ accounts for the pull-out degradation due to cyclic loads and depends on the diameter. The parameter $\Omega_{cr,eq}$. varies with the rebar diameter and with the crack width.

CONCLUSION

There is a comprehensive set of guidelines available to design the post installed rebar connections, for static as well as seismic conditions. It is important to select the mortar whose performance has been assessed as per the relevant assessment documents and undertake a proper design of every connection.

A comprehensive overview is given in Fig. 14. Currently, in absence of adequate local design framework, there is inconsistency in the way these connections are treated. While a section of the designers exercise the right design practices, in many cases, the decision is made on prior experience, rule of thumb, generic specifications or random on-site pull-out tests. This leaves a lot of questions unanswered and compromises on the safety of the connection. It is pertinent to mention that the success of the entire retrofitting scheme, be it addition of shear walls or provision of concrete jackets, is to a great extent dependent on the performance of the connection and the ability of the post installed rebar to transfer the load as per design. An adequate anchorage depth based on the definite type of connection has to be determined for every connection and by no means, it can be generalized.



	1	2	3	4	5	6	7	8
Connection type	Splice	Simply supported (no moment)	Compression load only	Rigid	Rigid	Rigid	Rigid	Rigid
Members connected (examples)	Slab – Slab Wall - wall	Slab - wall	Column - foundation	Column to foundation	Wall to foundation	Slab to wall	Beam to wall	Beam to column
Design method	EC2			EOTA TR 069				

Fig. 14: Recommended design provisions for different type post-installed concrete-to-concrete connections

One may argue about the lack of local design guidelines, but this does not prevent us from adopting international design provisions which are well accepted and well researched. The need of the hour is make the structures safe against earthquake and efficiency of post installed rebars is a very crucial step in that.

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This webinar focused on steel buildings and structures that are rapidly gaining prominence in our country, particularly in various infrastructure segments. Even rooftop structures in existing buildings often utilize structural steel components. With the increasing frequency of earthquake tremors, understanding how to design these structures to be earthquake-resilient is paramount. Equally important is comprehending the behavior of connections in existing structures during earthquakes and addressing any issues that may arise.

The Seismic Academy, on behalf of Hilti, invited all civil engineering professionals and construction industry experts to an exclusive webinar. Participants gained insights into the following topics:

- Special design and detailing requirement for steel structures against earthquake
- Seismic design of connections using post installed anchors

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