HALF-DAY WEBINAR

Recent Advances in Design Methodology for Post-Installed Reinforcements

14 August 2020, Hong Kong

香 港 大 學
THE UNIVERSITY OF HONG KONG

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(This programme might be subject to minor modifications without further notice.)

WELCOME MESSAGE

Ir Prof Francis Au

Department of Civil Egnineering
The University of Hong Kong, Hong Kong, China

On behalf of the Department of Civil Engineering, The University of Hong Kong, I would like to welcome you all to the Half-Day Webinar on Recent Advances in Design Methodology for Post-installed Reinforcements.

Post-installed reinforcements (PIR) use adhesive or cementitious grout to bond the reinforcements and concrete together. They are widely used to connect new structural components to old concrete structures. However, design methodology for PIR is not available for Hong Kong. This half-day webinar offers a timely opportunity for engineering professionals and academics to share their ideas and experiences in the recent advancement in the structural design of post-installed reinforcement. Close interaction and knowledge transfer between these parties are crucial to the practical and economical applications of post-installed reinforcements.

Lastly, I would like to extend our gratitude to the Organizing Committee, under the able leadership of Dr. Ray Su, for their dedication and contributions. I wish all participants an inspiring and fruitful experience.

Post-installed reinforcing bar technology – State of the art and future developments

Dr. Giovacchino Genesio

Hilti Corporation

Biography

Giovacchino Genesio graduated in structural engineering at the University of Florence (Italy) and obtained his PhD at the University of Stuttgart (Germany) on seismic assessment and retrofitting of reinforced concrete structures. He worked as a consultant for 5 years dealing with qualification, design, training and quality inspection of fastening systems. He is currently Code and Approval Engineer for Hilti. His field of interest includes concrete to concrete connections, seismic retrofitting and fastening technology. He is author or co-author of several publications on these topics.

Abstract

The use of post-installed reinforcing bars has been continuously gaining importance in reinforced concrete construction in the past decades. The main fields of application are the strengthening and modification of existing structures as well the optimization and increase of flexibility in construction processes for new buildings. The growing of interest for this technology has boosted the development of dedicated products and systematic investigations have led to the establishment of third-party qualification procedures usually based on the principle of equivalency of the load-displacement behavior of post-installed and cast-in reinforcing bars. Key issues include verifiable bond strength, viability of the adhesive delivery system, robustness to different environmental and loading conditions. This approach allows to apply the state-of-the-art provisions of reinforced concrete design in the main applications, where post-installed reinforcing bars are usually involved, namely lap splicing, starter bars and doweling in shear-friction interfaces. In more recent times the development of high-performance systems and the research dedicated to specific applications are opening the door to a new generation of products that jointly with innovative design approaches enable optimized solutions. A brief overview over this journey is given in this contribution.

1. Introduction

Post-installed reinforcing bars consist of deformed reinforcing bar in holes drilled in hardened concrete and filled with injectable mortars. They are typically used in concrete-to-concrete connections where new concrete is placed against existing concrete with the surface of latter roughened (Figure 1). Their performance is strictly related to the overall system including drilling machine as well as tools to clean the borehole and inject the mortar to avoid formation of air voids.

The use of post-installed reinforcing bars is getting widespread in the practice of reinforced concrete (r.c.) constructions. Mortar with validated performance and proper design allow the embedded reinforcing bars to develop full tension yield strength, making it an ideal solution to replace misplaced or missing starter bars at interfaces of various types of member joints (Figure 2a) due to movement of bars as a result of casting and vibrating of wet concrete. Post-installed reinforcing bars are also gaining popularity in structural retrofitting, seismic strengthening as well as building extension works, for which new starter bars or surface dowels

(Figure 2b) can be added to existing structure with minimal disturbance, as oppose to laborious partial hacking of existing members to allow a new cast-in connection.

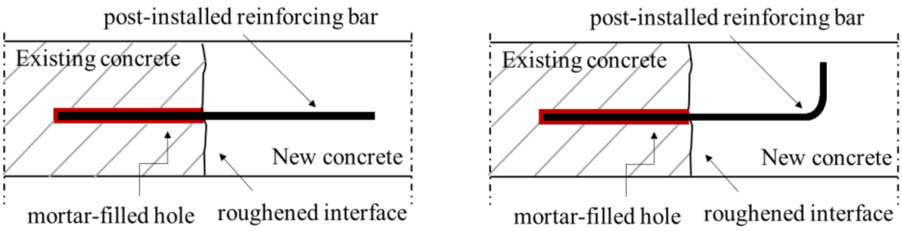


Figure 1. Post-installed reinforcing straight or hooked bar (typ.)

a) Application of post-installed rebars to replace misaligned couplers of basement diaphragm walls (left) and missing starter bars of concrete bored piles (right)

b) Post-installation of surface dowels for basement skin walls (left) and new starter bars to existing structure (right)

Figure 2. Examples of applications with post-installed reinforcing bars

In this paper the use of post-installed reinforcing bar systems in the practice of r.c. constructions is discussed. The need and principles of qualification procedure is highlighted considering the main assessment criteria available in the international landscape. A rational classification of applications is given and their design is briefly addressed with focus the main aspects that need to be considered in addition to conventional r.c. design. Furthermore, recent advancements are changing the way of designing concrete-to-concrete connections with post-installed reinforcing bar systems making it strictly linked to specific product performance characteristics.

The concepts discussed in this paper mainly refers to standards environment of Eurocodes. A transfer of these principles to the standards established by the American Concrete Institute (ACI) is possible but requires specific adjustments. They cannot be handled here due to the limited length of this contribution.

2. Qualification of post-installed reinforcing bar systems

In Europe the use of mortar systems for the realization of connections with post-installed reinforcing bars is limited to products evaluated according to the provisions established by the European Organization for Technical Assessment (EOTA), namely European Assessment Document (EAD) 330087 [6] for static loading and under fire exposure (which has recently replaced the former Technical Report (TR) 23), and EAD 331522 [7] for seismic loading conditions. A product assessed in accordance with these EADs holds an European Technical Assessment (ETA).

The assessment of post-installed rebar systems is based on the verification of comparable performance with cast-in reinforcing bars with respect to the failure modes expected in reinforcing bar design. These failure modes are: “pullout” (i.e., extraction of the bars from the concrete without significant damage of the surrounding concrete) and “splitting” (i.e., cracking and spalling of the concrete cover or formation of radial cracks between closely spaced bars). Pullout failure occurs when enough confinement (e.g., large concrete cover) is provided to the bar, while splitting failure occurs when the concrete tensile resistance associated with the cover thickness is not sufficient to reach the load corresponding to bond failure.

The performance of post-installed reinforcing bars is strongly linked to the performance of the mortar and its robustness in different installation conditions (e.g., temperature, humidity) and is usually significantly sensitive to jobsite/installation conditions (e.g., improper or incomplete borehole cleaning or/and injection, corrosive environment), loading conditions (e.g., freeze-thaw cycles, sustained loading at high temperature, cyclic seismic loading), borehole drilling method, quality and type of equipment used for installation, and depth and diameter of the reinforcing bar. All these aspects are taken into account in the product assessment requirements in accordance with the EAD 330087 [6] and EAD 331522 [7]. More recently a procedure to verify the suitability of a post-installed reinforcing bar system to be employed in application with design working life up to 100 years have been developed (EAD 330087-v01). This allows not to cover the entire of range of the working life categories described in the EN 1990 [1].

A system assessed in accordance to these EADs can be used for lap splices and anchorages designed following the provisions of EN 1992-1-1 [2] (static loading), EN 1992-1-2 [3] (under fire exposure) and EN 1998-1 [5] (seismic loading), when a straight bar is allowed.

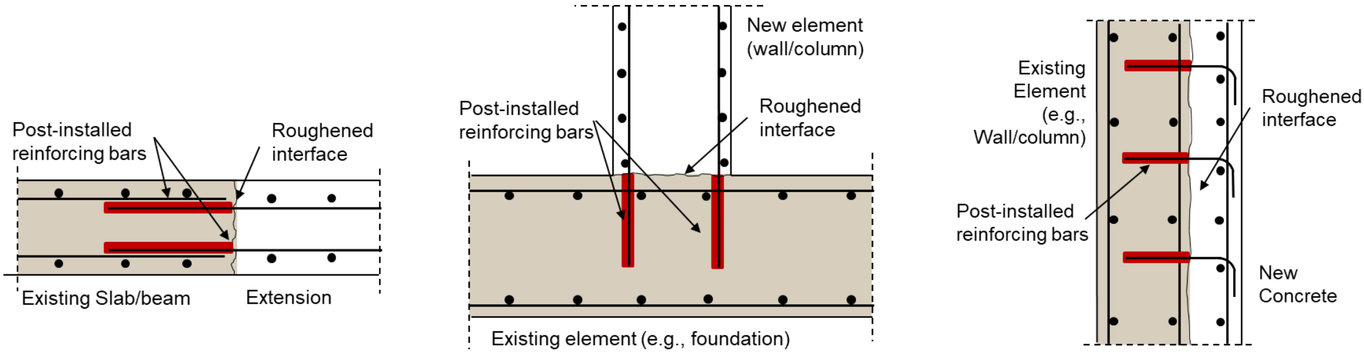
The research effort of the last decades has also shown that post-installed reinforcing bar systems, may exhibit a superior load carrying behavior than cast-in bars under specific condition and types of concrete to concrete connections. To provide clear rules and guidance in the assessment of the real bond-splitting performance of different product the EOTA has adopted the EAD 332402 [8]. A product with an ETA in accordance with this EAD can be designed for anchorages in moment resisting connections following the provisions of the TR 069 [12]. Furthermore, assessment and design provisions to address the connection of connection of concrete elements with shear dowels have been developed (EAD 332347 [8] and TR 066[11]).

3. Design of Applications with post-installed reinforcing bar systems

Post-installed reinforcing bars are typically used to realize monolithic connections between an existing r.c. member and a new element, e.g., to extend or strengthen an existing structure. In some cases this technology is used also to optimize and speed up the construction process of new buildings. The types of connections between r.c. members can be grouped in three categories as displayed in Figure 3. The realization of lap splices (Figure 3a) is the most common method to transfer tension forces between adjacent reinforcing bars by mean of local struts. This type of connection is the most common for cast-in connections, but it is often not feasible with post-installed reinforcing bars, because

(i) Reinforcement in the existing member to build up a lap splice is often not available, where needed for the new connection; and
(ii) the efficiency of the connection is highly influenced by the tolerances in the position of the existing reinforcement and of the installation of the post-installed reinforcing bars.

For these reasons, starter bars (Figure 3b) might be needed to realize new moment resisting connections. Due to the fact that these bars cannot obviously terminate with hooks, as in the common practice for cast-in bars, particular attention should be made in the assessment of the force flow through the structural node. In connections, where the dominant action is not a bending moment, and consequent tension in one layer of post-installed reinforcing bars and compression in the other, but, a shear force, different design considerations are made to ensure the transfer of the shear forces at an interface (Figure 3c). In the following sections relevant design aspects of each application are discussed.



a) Lap splices b) Anchorage for structural joints c) Shear dowels

Figure 3. Typical concrete to concrete connections with post-installed reinforcing bars

Anchorages for structural joints

The anchorage length, l_{bd}, of reinforcing starter bars post-installed with polymeric mortar qualified under EAD 330087 [6] can be calculated using the same design principle and formula 8.4 per EN 1992-1-1 [2] for cast-in bars, which can be rearranged into the following form

$$l_{bd} = (\varnothing/4 \cdot \sigma_{sd}/f_{bd,PIR}) \cdot \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 > l_{b,min} \quad (1)$$

where,

l_{bd}	=	design anchorage length
$\varnothing$	=	nominal diameter of the reinforcing bar
σ_{sd}	=	design stress in the reinforcing bar associated with the considered design action
$f_{bd,PIR}$	=	design bond strength of post-installed reinforcing bar system taken from the relevant ETA
α_1	=	coefficient of form of bar (0.7 for bent bar, 1.0 for straight bar)
α_2	=	coefficient of concrete cover (between 0.7 for lager cover and 1.0 for small cover)
α_3	=	coefficient of confinement effect of non-welded transverse reinforcement (between 0.7 and 1.0)
α_4	=	coefficient of confinement of welded transverse reinforcement (0.7 if present, else 1.0)
α_5	=	coefficient of confinement effect of transverse pressure (between 0.7 and 1.0)
$l_{b,min}$	=	minimum anchorage length if no other limitation is applied, maximum of {100mm, 10 × bar diameter, 30% (or 60%) of basic anchorage length for tension (or compression) case}

In the context of post-installed reinforcing bar it should be observed that:

(i) due to installation constraint, post-installed rebar is always straight when installed to existing concrete, hence α_1 is always taken as 1.0, and

(ii) it is obviously not feasible to weld post-installed rebar to existing transverse reinforcement, hence α_4 is always taken as 1.0.

Based on equation (1), the design yield embedment length (l_{bd} for $\sigma_s = f_{yd}$) of a specific post-installed reinforcing bar can be calculated by taking the design bond strength ($f_{bd,PIR}$) a system qualified in accordance with relevant ETA in accordance with EAD 330087 [6] or EAD 331522 [7]. The value $f_{bd,PIR}$ cannot be larger than the bond strength value $f_{bd,EC2}$ calculated following the equation (8.2) of EN 1992-1-1 [2] and it is influenced by several factors, i.e., concrete strength, bar diameter and drilling method.

It is noted that the use of the anchorage length equations discussed above for structural joints needs to be followed together with specific requirements of EN 1992-1-1 [2]. Structural joints may be classified as "simply supported", i.e., where no bending moment in the connection is assumed and only shear needs to be transferred at the interface (e.g., end supports of continuous beams or slabs) or "rigid nodes", where the forces to be anchored comes mainly from the bending action at the interface between the connected elements (e.g., structural joints in moment resisting frame structures).

If a simply supported connection is assumed, the requirements of section 9.2 (for beams) or 9.3 (for slabs) of EN 1992-1-1 [2] shall be followed to calculate the forces to be anchored considering:

(i) Tension in the bottom reinforcement due to the shear in the new element as per the assumed strut-and-tie mechanism (i.e., curtailment of longitudinal tension reinforcement);
(ii) Tension force due to the assumed partial fixity in the top reinforcement; and
(iii) The assumed partial fixity shall comply with the result given by the structural analysis of the connection.

The design of rigid nodes is mainly ruled by EN 1992-1-1 [2] considering an appropriate Strut-and-Tie model following the provisions of section 6.5. This design approach is quite straight forward, if the anchorage ends with a bent, which is the regular case for cast-in bars, where the "node", intended as equilibrium point between the concrete strut and the tension tie (reinforcing bar), see EN 1992-1-1 [2], Figure 6.28. However, this type of detailing is obviously not feasible with post-installed reinforcing bars. The provision of EN 1992-1-1 [2], Figure 6.27 to define a compression-tension node in the case of a straight anchorage lead to very long and often unfeasible anchorage length. Procedures have been developed in the past years to reduce the conservativism of such design approach taking into account product dependent performance characteristic, see e.g., Kupfer et al. (2003) [15]. Therefore, their applicability is limited to the tested products.

Other methods provide a rational way to conservatively take into account the tensile strength of concrete to anchorage tensioned bars in rigid nodes (e.g., Mahrenholtz, 2014 [16]). More recently EOTA has introduced an assessment procedure of the realistic bond-splitting performance of a post-installed reinforcing bar system (EAD 332402 [8]) that allow the design of rigid nodes following the TR 069 [12]. The design approach to calculate the anchorage length of PIR in moment resisting connections in accordance with the EOTA TR 069 [12] is based on the establishment of a hierarchy of strengths between the following resistances:

- Steel yielding ($N_{Rd,y}$) in accordance to EN 1992-1-1 [2]
- Bond-splitting resistance ($N_{Rd,sp}$)
- Concrete breakout ($N_{Rd,c}$)

For the calculation of the characteristic concrete breakout resistance ($N_{Rk,c}$), the provisions of EN 1992-4 [4] are followed with a few exceptions (e.g., no limitation on the maximum number of bars in the connection covered by the design model).

$$N_{Rk,c} = k_1 \cdot f_{ck}^{0,5} \cdot l_b^{1,5} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{ec,N} \cdot \psi_{re,N} \cdot \psi_{M,N} \qquad (2)$$

Where: k_1 = 7.7 or 11.0 for cracked or uncracked concrete, respectively, f_{ck} = concrete compressive strength; l_b = anchorage length of the reinforcing bar; $\psi_{A,N} = A_{c,N} / A^0_{c,N}$ = factor for geometric effect of axial spacing and edge distance; $\psi_{s,N}$ = factor for the disturbance of the distribution of stresses in the concrete due to the proximity of an edge of the concrete member; $\psi_{re,N}$ = factor for the effect of dense reinforcement; $\psi_{ec,N}$ = for the load eccentricity; $\psi_{M,N}$ = the positive effect of a compression force be-tween fixture and concrete in cases of bending moments, with or without axial force.

The characteristic bond-splitting resistance ($N_{Rk,sp}$) is calculated using the analytical formulation included in the fib Model Code 2010 and considering the influence of different parameters (concrete strength f_{ck}, bar diameter $\varnothing$, minimum cover c_d as defined in the EN 1992-1-1 [2], maximum cover c_{max} as defined in the fib Model Code 2010 [13] and the anchorage length l_b). The factor $k_m \cdot K_{tr}$ takes into account the positive influence of transverse reinforcement following the provisions of Model Code 2010 [13].

$$N_{Rk,sp} = \tau_{Rk,sp} \cdot \varnothing \cdot \pi \cdot l_b \tag{3}$$

$$\tau_{Rk,sp} = A_k \cdot (f_{ck}/25)^{sp1} \cdot (25/\varnothing)^{sp2} \cdot [((c_d/\varnothing)^{sp3}) \cdot (c_{max}/c_d)^{sp4}) + k_m \cdot K_{tr}] \cdot (7\varnothing/l_b)^{lb1} \leq \tau_{Rk,ucr} \tag{4}$$

where: the factor A_k and the exponents sp1, sp2, sp3, sp4, and lb1 are product dependent parameters to be taken from the ETA in accordance with the EAD 332402 [8]. The upper limit of the splitting resistance is the pullout resistance in uncracked concrete ($\tau_{Rk,ucr}$) as given in the same ETA. The value $\tau_{Rk,ucr}$ is multiplied by the fac-tor $\Omega_{cr} < 1,0$ if cracked concrete conditions apply. The factor Ω_{cr} depends on the sensitivity of the PIR system to cracks in concrete running along the bar axis. More details can be found in the TR 069 [12].

Lap splices

In the case of lap splicing, per EN 1992-1-1 [2] Equation 8.10, an additional coefficient α_6 ranging 1.0 to 1.5 will have to be introduced to consider the influence of percentage of area of reinforcing bars that are lapped, hence Equation (1) becomes

$$l_0 = (\varnothing/4 \cdot \sigma_{sd}/f_{bd,PIR}) \cdot \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot \alpha_6 > l_{0,min} \tag{5}$$

Where:

$l_{0,min}$ = minimum anchorage length if no other limitation is applied, maximum of {200mm, 15 × bar diameter, 30% of basic anchorage length multiplied by the factor α_6}

In case of replacing series of missing couplers or starter bars, α_6 likely be taken as 1.5 (i.e. 100% lapped), hence the value calculated using equation (5) shall be multiplied by 1.5.

Shear dowels

In a large variety of structural applications particular attention must be paid to the shear transfer between interfaces of r.c. members cast at different times. A few examples are shown in Figure 4.

a) Bridge deck strengthening b) Column jacketing c) Foundation strengthening

Figure 4. Use of post-installed reinforcing bars in typical shear-friction applications

The design of the shear transfer at interfaces follows in most of the state-of-the-art standards based on the shear-friction theory according to which shear is transferred through a combination of friction acting over the interface surface area and dowel action mechanisms. Design equations are available e.g., in EN 1992-1-1 [2], Section 6.2.5. More detailed provisions can be found in the fib Model Code 2010 [13] and [14], but without any reference to seismic loading. The available design provisions cover cases where reinforcing bars serve as shear dowels crossing the concrete interfaces and, usually, their full anchorage (i.e., length required to reach steel yielding as per the relevant reinforced concrete design standard) is assumed in design, but in there is general lack of provision about how a full anchorage of the dowels can be design. The recently introduced EOTA TR 066 [11] offers a solution for the design of "short" dowels under static and fatigue load. The design bond strengths assumed for cast-in reinforcing bars is usually rather limited to avoid splitting failure in conditions of small concrete cover and/or close bar spacing (e.g., f_{bd} = 2.3 MPa in C20/25 as per EN 1992-1-1 [2]). These conditions are not necessarily applicable to rebar elements used for shear friction applications due to:

(i) usually larger spacing and edge distances than in other connections; and
(ii) the type of loading is different, i.e., dowel action instead of pure tension

Furthermore, the limitations in terms of thickness of the r.c. overlays given by the nature of some applications make the design in some cases unfeasible, e.g., in a floor strengthening the thickness of the overlay is typically less than 10 cm to avoid not acceptable reduction of the usable height between two floors and an excessive increase of weight of the structure.

The shear strength of the interface is given by the addition of several contributions, namely mechanical interlock (first part of the equation) of the effects of friction (second part of the equation) and dowel action (third part of the equation) and it is limited by the concrete strut failure.

$$\tau_{Rk} = c_r \cdot f_{ck}^{1/3} + \mu \cdot \sigma_n + \mu \cdot \kappa_1 \cdot \alpha_{k1} \cdot \rho \cdot \sigma_s + k_2 \cdot \alpha_{k2} \cdot \rho \ (f_{yk} \cdot f_{ck})^{0,5} \leq \beta_c \cdot \nu \cdot f_{ck} \qquad (6)$$

With:

τ_{Rk}	=	characteristic shear strength of the interface
f_{ck}	=	characteristic compressive strength of the weaker concrete of the two layers
μ	=	friction coefficient
σ_n	=	(lowest expected) compressive stress resulting from an eventual normal force acting on the interface
k_1	=	contribution factor for the friction mechanism
ρ	=	reinforcement ratio of the reinforcing steel crossing the interface
σ_s	=	tensile stress of reinforcing steel crossing the interface associate the relevant failure mode calculated as per EN 1992-4 [4] ($\sigma_s \leq f_{yd}$)
k_2	=	contribution factor for the dowel mechanism
f_{yd}	=	design yield strength of the reinforcing steel crossing the interface
β_c	=	coefficient for the strength of the compression strut according to fib MC2010 [13], [14]
ν	=	effectiveness factor for the concrete according to fib MC2010 [13], [14]

The factors k_1 and k_2 as well as the friction coefficient μ are function of the interface roughness and are tabled in the TR 066 [11]. α_{k1} and α_{k2} are product dependent factors to be taken from the relevant ETA in accordance with the EAD 332347 [8].

A design approach to address in a more rational and realistic manner the shear-friction resistance of interfaces taking into account the performance of the post-installed reinforcing bars used as dowels as a function of their embedment in the new and old r.c. member has been developed by Vintzileou et al. (2018) [17] and it covers both static and seismic loading.

4. Conclusions

This paper gives an overview on the field of application of connections with post-installed reinforcing bars in r. c. constructions. After explaining the principles of qualification of such systems for structural use, the main applications are explained. Some specific aspects of the design connections using post-installed reinforcing bars in combination with the provision of EN 1992-1-1 [2]. Furthermore, state-of-the-art application-based design models for anchorage of PIR in moment resisting connections and post-installed dowels in shear-friction interfaces are briefly introduced.

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Design methods for anchorage of post-installed rebars: Rebar end anchorage design, introduction to anchor theory and the improved bond splitting provision

Dr. Daniel Looi

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Biography

Dr Daniel Looi is currently a lecturer and course coordinator for the civil engineering programme at Swinburne University of Technology, Sarawak campus, Malaysia. He is a chartered professional engineer (structural) of Engineers Australia and a member of the Earthquake Committee of the Institution of Engineers Malaysia. He obtained his bachelor's degree in civil engineering from The University of Malaya and his PhD in structural engineering from The University of Hong Kong (HKU). He was a postdoctoral fellow in the Department of Civil Engineering of HKU. Daniel has published research works in seismic engineering, concrete mechanics, modular buildings, and fastening technologies, which include post-installed reinforcements. He is the recipient of the HKIE Outstanding Paper Award for Young Researcher/Engineer in 2015. In his earlier career, Daniel worked as a structural application engineer in a multinational company, specialised in structural analysis and design computation for buildings and plants.

Abstract

Post-installed rebar (PIR) is one of the technologies used to connect new reinforced concrete elements with existing members. PIR is drilled and installed into cured concrete, bonded by a qualified adhesive system in the existing concrete, and usually served as starter-bars and/or to create lap splicing with the reinforcements in new concrete structures on the other side of the interface. A guidebook entitled "Guide for Design, Installation, and Assessment of Post-installed Reinforcements" authored by Su, Looi and Zhang is in-press to facilitate the industry in designing of connections with PIR. In this seminar, the PIR design method suitable for Hong Kong, using rebar end anchorage design with aid of strut-end-tie method (written in the guidebook) will be shared. Additionally, a design method introduced with the TR 069 (2020), which harmonises the rebar anchorage design method and anchor theory, considering the realistic bond-splitting behaviour of a PIR system under static loading (assessed in accordance with the EAD 332402, 2020) will be briefly elaborated.

1. Introduction

Post-installed rebar (PIR) has been extensively used in concrete-to-concrete connection. Common applications of PIR in Hong Kong (and other parts of Asia) are typically to form starter bars at existing reinforced concrete (RC) element to connect to new RC member. Although PIR is common, there is no specific guide for the use of PIR in Hong Kong. Hence, a guidebook entitled "Guide for Design, Installation, and Assessment of Post-installed Reinforcements" authored by Su, Looi and Zhang is in-press [1] to facilitate the industry in designing for PIR. This short paper discusses about design methods for PIR that is suitable for Hong Kong, by harmonising the Code of Practice for Structural Use of Concrete 2013 (HK

CoP2013, 2013) [2] and Eurocode 2 (EN 1992-1-1, 2004) [3]. Important concepts in the guidebook are replicated here with a reproduced design example of simple PIR connection.

2. Failure modes of PIR

PIR is designed for axial actions (tension and compression) and it is essential for engineers to understand its load transfer mechanism (see Figure 1) and associated failure modes (schematically shown in Figure 2). Unlike anchor design [4] which is subject to shear force resistance calculation, shear action in PIR is accounted for at the roughened interface between new and old concrete. The shear resistance is attributed by mechanical interlocking, friction and dowel actions, and are limited by the concrete strut failure (refer to Equation (6) in the short paper presented by the first speaker Dr. Genesio). In Figure 1, strut forces are mobilised by external tension action, forming mechanical interlocking and bond strength of rebar to adhesive and adhesive to concrete material. These struts act across the adhesive and convert into micro-struts at the concrete base material. The micro-struts are then acting onto the adjacent cast-in rebars (i.e. lapping), forming equilibrium of tie forces in those rebars.

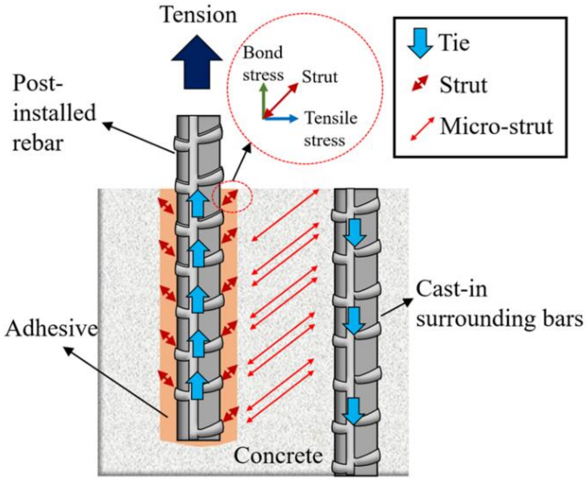
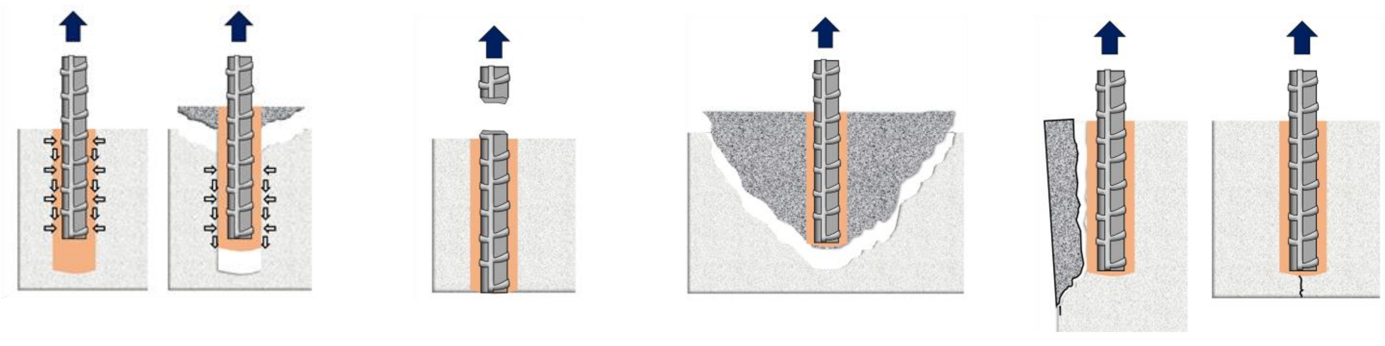


Figure 1: Load transfer mechanism of PIR.

It is worth to mention that bond strengths between the rebar-adhesive and adhesive-concrete are equally important and should not fail in pull-out (see Figure 2(a)). Other failure modes (i.e. rebar yielding, concrete cone and concrete splitting in Figures 2(b) to 2(d), respectively) should be carefully catered for in PIR design.



(a) Pull-out (bond failure) (b) Rebar yielding (deep embedment) (c) Cone (short embedment) (d) Splitting (small cover thickness)

Figure 2: Failure modes of PIR.

Table 1: Summary of anchorage length requirements for cast-in rebars in HK CoP2013 and EN 1992-1-1 (2004)

HK CoP2013 [2]		**EN 1992-1-1 (2004) [3]**	
Basic anchorage length: $$l_b \geq \frac{0.87 f_{yk}}{f_{bu}} \frac{\phi}{4}$$ where f_{yk} is characteristic yield strength of rebar; ϕ is rebar diameter; f_{bu} is factored bond stress capacity as a function of characteristic concrete cube strength ($f_{cu,k}$) according to Cl. 8.4.4 and is defined by $\beta\sqrt{f_{cu,k}}$. β is 0.50 and 0.63 for tension and compression stresses, respectively. This value includes a partial safety factor for bond stress (γ_m) of 1.4.	Eq. (1a)	Required anchorage length: $$l_{b,rqd} \geq \frac{\sigma_{sd}}{f_{bd}} \frac{\phi}{4}$$ where σ_{sd} is design stress in rebar, ϕ is rebar diameter; f_{bd} is design value of ultimate bond stress as a function of concrete design tensile strength (f_{ctd}) according to Cl. 8.4.2(2) and is defined by 2.25 η_1 η_2. η_1 and η_1 are to implicitly account for bond condition, position of rebar and rebar diameter, f_{ctd} is the concrete design tensile strength taken as 5% fractile with partial safety factor (γ_m) of 1.5	Eq. (1b)
(No further provision for splitting and minimum anchorage length)		Design anchorage length: $$l_{bd} = \alpha_2\, l_{b,rqd} \geq l_{b,min}$$ where α_2 is coefficient for the effect of concrete minimum cover to consider splitting failure for straight bars as defined in Eq. (3). Other coefficients such as α_1 is always unity for straight bars, α_3 for confinement effects and α_4 for welded rebars are not considered in this paper.	Eq. (2)
		α_2 coefficient for splitting: $$0.7 \leq \alpha_2 = 1 - \frac{0.15(c_d - \phi)}{\phi} \leq 1.0 \text{ (Tension)}$$ $$\alpha_2 = 1 \text{ (Compression)}$$ where c_d is taken as min{a/2, c_1, c} for straight bars, s is the clear spacing of bars, c_1 is the side cover and c is the top or bottom cover.	Eq. (3)
		Minimum anchorage length: $$l_{b,min} \geq \max\{0.3\, l_{b,rqd}, 10\phi, 100 \text{ mm}\} \text{ (Tension)}$$ $$l_{b,min} \geq \max\{0.6\, l_{b,rqd}, 10\phi, 100 \text{ mm}\} \text{ (Compression)}$$	Eq. (4a) Eq. (4b)
Special consideration is required for splice lapping in tension, as the location (i.e., top, bottom or corner of a section) and concrete cover have to be taken into account, to decide on a factor of 1.4 or 2.0 times the minimum lap length. For compression lap length, the factor is 1.25 times the minimum lap length.		Design lap length: $$l_o = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{o,min}$$ where α_6 is a coefficient of the percentage of lapped bar (p_1) relative to the total cross-section area within 0.65 l_o from the centre of the lap length. Readers are advised to refer to Figure 8.8 in EN 1992-1-1 (2014) [3]	Eq. (5)
Minimum lap length: $$l_{o,min} \geq \max\{15\phi, 300 \text{ mm}\}$$	Eq. (6a)	Minimum lap length: $$l_{o,min} \geq \max\{0.3\alpha_6\, l_{b,rqd}, 15\phi, 200 \text{ mm}\}$$	Eq. (6b)

3. Design provisions for rebar anchorage design in HK CoP2013 and EN 1992-1-1 (2004)

To facilitate discussion of end anchorage design in conventional cast-in rebar, Table 1 is prepared to summarise design provisions in HK CoP2013 [2] and EN 1992-1-1 (2004) [3].

Based on the summary in Table 1, some important observations are noted as follows:

1. Eq. (1a) is derived as per conventional British codes practice, assuming anchorage bond force of rebar is greater than design yield strength (0.87 f_{yk}) of rebar.
2. Eq. (1b) in EN 1992-1-1 (2004) [3] uses design stress in rebar (σ_{sd}), which can be subject to wider interpretation and not restricted to design yield strength of rebar.
3. EN 1992-1-1 (2004) [3] is more comprehensive and requires more steps to arrive at design anchorage length with additional provision of splitting control using α_2 coefficient.
4. For lap length design, EN 1992-1-1 (2004) [3] is also more comprehensive and requires more steps to arrive at design lap length with an α_6 coefficient.

It may be argued that PIR can be designed according to cast-in rebars provisions. Nonetheless, challenges may arise, and they are pointed out as follows:

1. There is no provision for design bond strength of chemical adhesives for PIR in the codes [2, 3].
2. The use of rebar yield strength may result in very long anchorage length simply because of installation feasibility of PIR, i.e. only applies to straight rebars without bend or hook. Furthermore, there is no explicit guidance to arrive at the design stress in rebar.

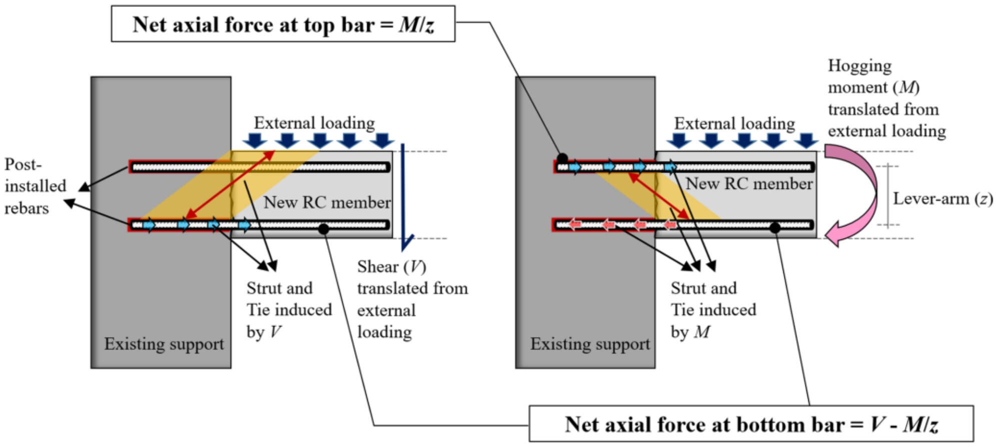
4. Proposed design provisions for simple PIR connection

From the discussion above, it is apparent that engineers in Hong Kong need more than HK CoP 2013 [2] and EN 1992-1-1 (2004) [3] to design for PIR. This short paper is written to fill the gap by proposing harmonised design provisions of the codes [2, 3], consistent with the guidebook to be published [1]. The recommendations feature alternative ways to circumvent the challenges in calculating anchorage length of PIR. There are six proposals, of which five are written for simple connections and are summarised in Table 2. For the sixth proposal on moment connection (without lapping), readers are advised to refer to the short paper presented by the third speaker Ir. Augustus Lee.

Table 2: Design proposals for simple PIR connection

Proposal	Design commentary	
1. Provide detailed option to determine bond stress capacity (f_{bu}) of adhesives in PIR systems.	General method: $$f_{\mathrm{bu}} = \beta\sqrt{f_{\mathrm{cu,k}}}$$ where β is 0.50 and 0.63 for tension and compression stresses, respectively. This value includes a partial safety factor for bond stress (γ_m) of 1.4. Detailed method: $$f_{\mathrm{bu}} = \frac{f_{\mathrm{bd}}}{\alpha_2} = 2.25\,\eta_1\eta_2 f_{\mathrm{ctd}}/\alpha_2 = 2.25\,\eta_1\eta_2 f_{\mathrm{ctk,0.05}}/\gamma_{\mathrm{m}}/\alpha_2$$ or European Technical Assessment (ETA) and manufacturer's technical data. All parameters have been defined in Eqs. (1b) and (3). Table 3.1 in EN 1992-1-1 (2004) [3] is useful to determine the 5% fractile of tensile strength ($f_{ctk,0.05}$).	Eq. (7a) Eq. (7b)
2. Replace the design yield stress (f_{yd} = 0.87f_{yk}) in Eq. (1a) with an actual design stress (f_{sd}) from simple	Cl. 9.2.1.4(2) of EN 1992-1-1 (2004) [3] allows the use of an STM to calculate the axial forces (F_{Ed}) in the reinforcement, which is well suited for estimating the design stress (f_{sd}).	

strut-and-tie (STM) equilibrium model (see Figure 3).	$$f_{\mathrm{sd}} = F_{\mathrm{Ed}}/A_{\mathrm{s}} = [\lvert V_{\mathrm{Ed}}\rvert\,{}^{a}/_{z} \pm M_{\mathrm{Ed}}/z]\;/\;A_{\mathrm{s}}$$ where A_s is total rebar cross-sectional area, V_{Ed} is design shear force, M_{Ed} is design moment at support for moment connection, a is the shear span, lever arm z is assumed to be 0.9 d and d is the effective depth of the section. Figure 3(a) shows the idealised STM which assumes a 45-degree truss model; hence a/z is equal to unity.	Eq. (8)



(a) Shear action at simply supported and fixed connection

(b) Hogging moment action at fixed connection

Figure 3: Simple STM equilibrium model to arrive at actual design stress

3. Impose a minimum anchorage length ($l_{b,min}$).	A minimum anchorage length ($l_{b,min}$) is required to prevent the initiation of cone failure (see Figure 2(c) due to short embedded length). Note that there are no provisions for minimum anchorage length in HK CoP2013 [2], hence it is proposed based on Eq. (4) as per EN 1992-1-1 (2004) [3]. The lapping requirement is also proposed based on Eq. (6b) [3] for consistency. Tension: $l_{\mathrm{b,min}} \geq \alpha_{\mathrm{lb}}\ \max\{0.3 l_{\mathrm{b,rqd}}, 10\phi, 100\ \mathrm{mm}\}$ Compression: $l_{\mathrm{b,min}} \geq \alpha_{\mathrm{lb}}\ \max\{0.6 l_{\mathrm{b,rqd}}, 10\phi, 100\ \mathrm{mm}\}$ Lapping: $l_{\mathrm{o,min}} \geq \alpha_{\mathrm{lb}}\ \max\{0.3\alpha_6\, l_{\mathrm{b,rqd}}, 15\phi, 200\ \mathrm{mm}\}$ where $l_{b,rqd}$ has been defined in Eq. (1b) and α_{lb} is an amplification factor equals to 1.5 if no testing is carried out on PIR in cracked concrete in accordance with EAD 330087-00-0601 (2018) [5].	Eq. (9a) Eq. (9b) Eq. (9c)
4. Check shear induced web crushing (see Figure 3, where compression strut is a band highlighted in darker yellow).	The minimum anchorage length may inhibit possibility of cone failure. The strut force induced by the external loading provide additional confinement effect in precluding cone failure. Two options to check for web crushing are recommended: Method A: Enhanced shear strength close to support: $$v_{\mathrm{c,enhanced}} = \left(\frac{2d}{a}\right) v_{\mathrm{c}} = 2(0.79)\left(\frac{100A_{\mathrm{s}}}{b_{\mathrm{v}}d}\right)^{\frac{1}{3}}\left(\frac{400}{d}\right)^{\frac{1}{4}}\left(\frac{1}{\gamma_{\mathrm{m}}}\right)\left(\frac{f_{\mathrm{cu}}}{25}\right)^{\frac{1}{3}} \geq v_{\mathrm{applied}};\ \leq \min\{0.8\sqrt{f_{cu}}, 8\ \mathrm{MPa}\}$$ where all the parameters should be based on Table 6.3 and Cl. 6.1.2.5(g) of HK CoP2013 [2]. Method B: Checking struts by using an STM	Eq. (10a)

	$$f_{\mathrm{strut}} = \frac{V_{\mathrm{Ed}}/\sin 45°}{b\, w_{\mathrm{strut}}} \leq \beta_{\mathrm{strut}}\, f_{\mathrm{cu,k}}/\gamma_{\mathrm{c}}$$ where w_{strut} is the strut band width estimated using $d/\sqrt{2}$; b is the width of the connecting member, β_{strut} is the recommended strut efficiency factor equal to $\beta_{\mathrm{strut}} = 0.5$ for cube strength or 0.6 for cylinder strength according to Su and Looi (2016) [6] and γ_{c} which is the material partial safety factor for concrete = 1.5.	Eq. (10b)
5. Impose a minimum edge distance.	EN 1992-1-1 (2004) [3] states that the maximum boundary is reached when α_2 is equal to 1.0, and c_{d} corresponds to 1 ϕ. It is noted that such a small cover of *1* ϕ may present challenges with drilling holes in PIR systems. Hence, EAD 330087-00-0601 (2018) [5] proposes a minimum cover as a function of drilling method, reinforcement size and with or without the use of a drilling aid, to account for possible deviations during the drilling process (see embedded Table 2.1).	Eq. (11)

Table 2.1 Minimum concrete cover (c_{min}) proposed in EAD 330087-00-0601 (2018) [5]

Use of drilling aid	**Drilling method**	**Bar diameter ϕ**	c_{min}
No	Hammer or diamond	< 25 mm	30 mm + 0.06 $l_{\mathrm{v}} \geq 2\phi$
		≥ 25 mm	40 mm + 0.06 $l_{\mathrm{v}} \geq 2\phi$
	Compressed air	< 25 mm	50 mm + 0.08 l_{v}
		≥ 25 mm	60 mm + 0.08 $l_{\mathrm{v}} \geq 2\phi$
Yes	Hammer or diamond	< 25 mm	30 mm + 0.02 $l_{\mathrm{v}} \geq 2\phi$
		≥ 25 mm	40 mm + 0.02 $l_{\mathrm{v}} \geq 2\phi$
	Compressed air	< 25 mm	50 mm + 0.02 l_{v}
		≥ 25 mm	60 mm + 0.02 $l_{\mathrm{v}} \geq 2\phi$

where l_{v} is the setting anchorage depth of rebars (in unit mm).

5. Example of simple PIR connection design

A simply supported RC slab is planned to be cast after the construction of the RC shear wall using PIR. Figure 4 shows details of the concrete-to-concrete connection and design steps are summarised in Table 3.

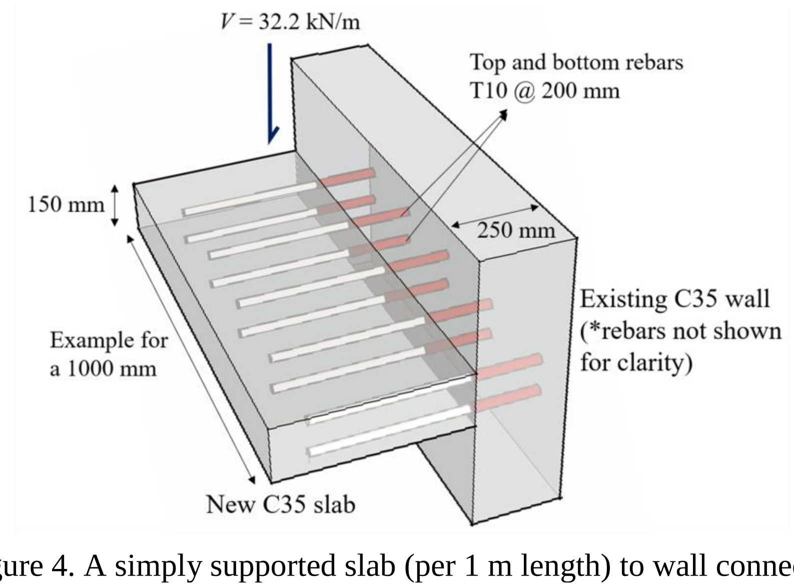


Figure 4. A simply supported slab (per 1 m length) to wall connection

Table 3. Design steps for a simply supported slab-to-wall connection.

Step	Design calculations	Remarks
1	Calculate the bond strength of PIR. General method: Using Eq. (7a) for tension, $f_{bu} = \beta\sqrt{f_{bu}} = 0.5\sqrt{35} = 3.0$ MPa Detailed method: Using Eq. (7b), assuming $c_d/\phi \geq 5$ (see Step 5 to be checked later), hence $\alpha_2 = 0.7$, $$f_{bu} = \frac{f_{bd}}{\alpha_2} = 2.25\ \eta_1\eta_2 f_{ctd}/\alpha_2 = 2.25\ \eta_1\eta_2 f_{ctk,0.05}/\gamma_m/\alpha_2$$ $$= 2.25\ (1)(1)(1.94)/1.5/0.7 = 4.1\ \text{MPa}$$	Proposal 1
2	Refer to STM in Figure 3 and details in Figure 4, calculate for actual design stresses. As it is a simple connection, axial stress for top rebar = 0 MPa. Using Eq. (8) for bottom rebar, $f_{sd} = 32200 / (5\ \pi\ 10^2/4) = 82$ MPa	Proposal 2
3	Using Eq. (1b), taking $f_{bd} = f_{bu}$, calculate for required anchorage lengths for bottom rebar. General method: $$l_{b,rqd} \geq \frac{\sigma_{sd}}{f_{bd}}\frac{\phi}{4}$$ $$= 82/3 \times 10/4 = 68\ \text{mm}$$ Detailed method: $l_{b,rqd} = 82/4.1 \times 10/4 = 50$ mm Assuming cracked concrete with $\alpha_{lb} = 1.5$, the minimum tension anchorage length is given in Eq. (9a) and the calculations are: General method: $$l_{b,min} = \alpha_{lb} \max\{0.3\ l_{b,rqd}, 10\ \phi, 100\ \text{mm}\}$$ $$= 1.5 \max\{0.3(68), 10(10), 100\ \text{mm}\} = 150\ \text{mm}$$ Detailed method: $$l_{b,min} = 1.5 \max\{0.3(50), 10(10), 100\ \text{mm}\} = 150\ \text{mm}$$	Proposal 3
4	Check for web crushing. Method A: The applied shear stress is $v_{applied} = V_{Ed}/(0.8bd) = 32200/(0.8\times1000\times150)$ $v_{applied} = 0.27$ MPa From Eq. (10a), $$v_{c,enhanced} = \left(\frac{2d}{a}\right)v_c = 2(0.79)\left(\frac{100A_s}{b_v d}\right)^{\frac{1}{3}}\left(\frac{400}{d}\right)^{\frac{1}{4}}\left(\frac{1}{\gamma_m}\right)\left(\frac{f_{cu}}{25}\right)^{\frac{1}{3}} \geq v_{applied}$$ $$= 2(0.79)(0.33)^{1/3}(400/150)^{1/4}(1/1.25)(35/25)^{1/3}$$ $$= 1.25\ \text{MPa} \geq 0.27\ \text{MPa}$$ $$\leq \min\{0.8\sqrt{35}, 8\ \text{MPa}\} = 4.7\ \text{MPa}$$ Method B: From Eq. (10b), $$f_{strut} = \frac{V_{Ed}/\sin 45°}{b\ w_{strut}} \leq \beta_{strut}\ f_{cu,k}/\gamma_c$$ $$w_{strut} = d/\sqrt{2} = 0.9 \times 150/\sqrt{2} = 95\ \text{mm}$$ $$f_{strut} = (V_{Ed}/\sin 45°)/(b\ w_{strut}) = 46/(1000 \times 95) = 0.49\ \text{MPa}$$ $$\beta_{strut}\ f_{cu,k}/\gamma_c = 0.5\ (35)/1.5 = 11.7\ \text{MPa} \geq 0.49\ \text{MPa}$$	Proposal 4 (OK) (OK)
5	Check for minimum concrete cover	Proposal 5

From Eq. (3),

$$c_d = \min\{a/2, c_1, c\} = \min\{100/2, 50, 50\} = 50 \text{ mm}$$

From Table 2.1 and corresponding Eq. (11), apply drilling aid, compressed air drilled, ϕ = 10, hence

$$c_{min} = 50 \text{ mm} + 0.02\, l_v = 50 \text{ mm} + 0.02\,(150) = 53 \text{ mm}$$

Summary — Provide bottom bars with 5 T10 @ 200 mm per 1 m length with anchorage length, l_b = 150 mm and a minimum cover of 55 mm. It should be noted that top rebars need to be designed according to simplified detailing rules [1].

6. Recent advances in concrete splitting design provisions

During drafting of the PIR Guidebook [1], it has not included more refined discussions on concrete splitting. Hence, it is worth to share recent advances in concrete splitting design provisions in this paper. The three methods for rebar end anchorage, anchor design and an improved bond-splitting method are briefly discussed.

(i) *extended EN 1992-1-1 rebar end anchorage method*

A higher bond stress of f_{bu} as per ETA or manufacturer's technical data for PIR system qualified by EAD 330087 [5] can be used with an extended EN 1992-1-1 method using Eq. (12), proposed by Tepfers [7]. This extended method extrapolated α_2 linearly for $c_d \geq 3$ and allowing a new α_2' of less than 0.7.

$$\alpha_2' = \frac{1}{\frac{1}{0.7} + \delta \frac{c_d - 3\phi}{\phi}} \geq 0.25 \qquad \text{Eq. (12)}$$

where δ is a factor calibrated by test, if linearly continues with the same slope, δ = 0.15. c_d has been defined in Eq. (3) and ϕ is the rebar diameter.

(ii) *EN 1992-4 [4] anchor design method*

In anchor design, capacity is given by specifying the embedment depth. Eqs. (13a) and (13b) show examples of concrete splitting capacity and combined concrete cone and pull-out capacity, respectively, according to anchor theory in EN 1992-4 (2018) [4].

concrete splitting capacity:

$$N_{Rk,sp} = N^0_{Rk,sp}\left(\frac{A_{c,N}}{A^0_{c,N}}\right)(\varphi_{s,N})(\varphi_{re,N})(\varphi_{ec,N})(\varphi_{h,sp}) \qquad \text{Eq. (13a)}$$

combined concrete cone and pull-out capacity:

$$N_{Rk,p} = N^0_{Rk,p}\left(\frac{A_{p,N}}{A^0_{p,N}}\right)(\varphi_{g,Np})(\varphi_{s,Np})(\varphi_{re,Np})(\varphi_{ec,Np}) \qquad \text{Eq. (13b)}$$

where $N^0_{Rk,p} = (\pi)(D)(h_{ef})(\tau_{Rk})(\varphi_{sus})$ and τ_{Rk} is product dependent. It is not the intention of this short paper to go rigorously into the equations and their rather complex definitions, hence readers are advised to refer to the code [4]. It is important to mention that the range of application of anchor design is 4ϕ to 20ϕ. Taking the anchor design equations directly to design for PIR can be conservative due to the influence of concrete member thickness in EN 1992-4 (2018) [4].

(iii) *TR069 [8] improved bond-splitting method*

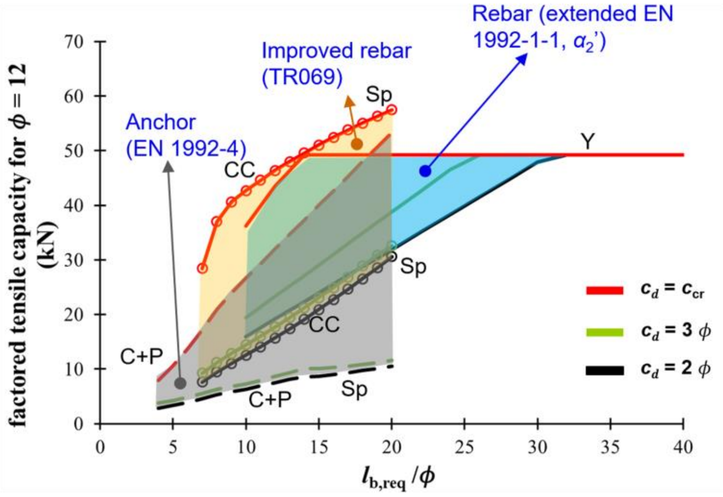
TR069 (2020) [8] is written based on the new EAD 332402 (2020) [9], specifically to cater for moment connections. The formulations in TR069 (2020) [8] are analogous to EN 1992-4 (2018) [4], where the combined pull-out and concrete cone resistance in Eq. (13b) is replaced by bond-splitting resistance in Eq. (14) to allow for geometric parameters, i.e. small edge distances and/or spacing between rebars as well as anchorage length higher than 20ϕ.

$$N_{Rk,sp} = (\pi)(D)(l_b)(\tau_{Rk,sp}) \qquad \text{Eq. (14a)}$$

where $$\tau_{Rk,sp} = \eta_1(A_k)\left(\frac{f_{ck}}{25}\right)^{sp_1}\left(\frac{25}{D}\right)^{sp_2} \times \left[\left(\frac{c_d}{D}\right)^{sp_3}\left(\frac{c_{max}}{c_d}\right)^{sp_4} + (k_m)(K_{tr})\right] \times \left(\frac{7D}{h_{ef}}\right)^{lb1}(\Omega_{p,tr}) \qquad \text{Eq. (14b)}$$

where the factor A_k and the exponents sp_1, sp_2, sp_3, sp_4, and lb_1 are product dependent parameters to be taken from the ETA in accordance with EAD 332402 (2020) [9]. More details can be found in TR069 (2020) [8]. This method is new and can be useful as an alternative in checking for cone and bond-splitting failure of top rebars at moment joints (without lapping).

An example of solution with varying embedment depth of PIR comparing the three methods (to better account for concrete splitting) for a 500 MPa T12 rebar or Grade 8.8 anchor installed using an ETA (adhesive with f_{bu} = 8.8 MPa) in C30/C37 300 mm thick RC member is presented in Figure 5. It is observed that the light grey shaded area of using anchor design EN 1992-4 [4] can be applied to a maximum of embedment length of 20ϕ. Combined concrete and pull-out failure and splitting failure are the governing failure modes of anchors (not steel yielding). The light yellow (TR069 [8]) and light blue shaded area (extended EN 1992-1-1 α_2' method) are conforming well to each other, which demonstrated the feasible use of these two methods in more comprehensive PIR design to account for splitting using a higher bond strength.



C+P = combined concrete and pull-out failure
CC = cone failure
Sp = splitting failure
Y = rebar yielding

*It should be noted that 2ϕ cover is taken as a minimum example, but specific anchors need more cover thickness.

Figure 5. An example of PIR solution using rebar end anchorage (extended EN 1992-1-1 α_2' method), anchor (EN 1992-4) and improved rebar (TR069) for a 500 MPa T12 rebar or 8.8 anchor installed with an adhesive of f_{bu} = 8.8 MPa in C30/37 300 mm thick RC member.

7. Conclusions

This short paper summarised the five proposals of design provisions in a Guidebook [1] for simple PIR connection, after reviewing the HK CoP 2013 [2] and EN 1992-1-1 (2004) [3]. Load transfer mechanism and failure modes of PIR were initially presented. A reproduced example of RC slab-to-wall connection was demonstrated. Finally, recent advances in concrete splitting provisions were reflected to give an update to the readers. While waiting for the Guidebook [1] to be published, readers are also encouraged to refer to a state-of-the-art review of PIR paper recently published in HKIE Transactions [10].

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Experimental and analytical study of moment connections with post-installed reinforcements

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Biography

Ir Lee is a RPE and a certified CIC BIM manager. He obtained his BEng (1st Hons), LLB and MPhil in the U.K. He also has a MSc (Distinct) awarded by the HKUST. His PhD research was on "Structural Behaviour of Post-installed Reinforcements in Wall-slab Moment Connections" in the RMIT University. He was a BIM panel and Safety, Health and Wellbeing panel corresponding member of the IStructE. Currently, he serves as the Structures expert panel in the ICE. He is a consultant of the RMIT University. His research interests are finite element analysis, modular integrated construction, retrofitting technology and GIS application.

Abstract

Post-installed reinforcements (PIR) technology has wide engineering applications from A&A works to new works for later on construction stage such as core-wall and beam/slab connections. Hence, it is especially useful as a remedy for defective or misplaced couplers and for the hot topic of Modular Integrated Construction. Despite the popularity, the design and use of PIR has not been fully addressed in major international design codes including Hong Kong's. Based on the associated failure modes, qualified PIR systems can be designed as the cast-in reinforcements subject to various modified analytical theories. A research collaboration on the topic of PIR between the RMIT University and the University of Hong Kong has been launched since 2017. A series of full-scale experimental studies were conducted to validate the proposed modified design methods of Rebar Anchorage, Bond Anchors and the Strut and Tie Model (STM) in wall-slab moment connections. Brief discussions on experimentations and structural analyses have been given in the Webinar. Focus has been stressed on STM – a state-of-the-art design.

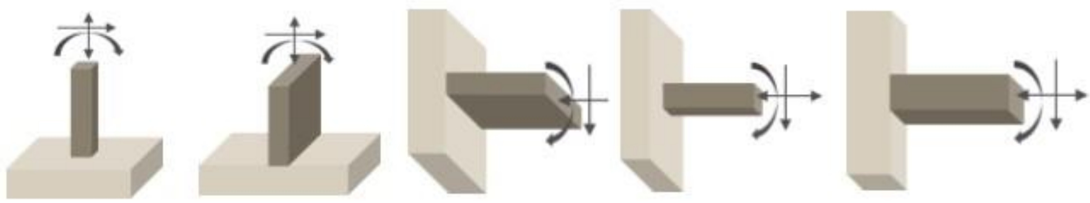
1. Introduction

Post-installed reinforcements (PIR) technology has wide engineering applications from new works to alteration and addition works. Not only does it help to support newly cast additions such as modular units in modular integration construction (MiC), but it also facilitates structural rehabilitation as functioning as remedy for defective or misplaced couplers. However, the use of PIR has not been fully addressed in major reinforced concrete design codes worldwide including Hong Kong's. Based on the design philosophy and the associated failure modes, the design of cast-in reinforcement can be extended to the qualified PIR systems with reference to the latest editions of AC 308 [1] and EOTA EAD 330087/330499 [6,7], in the USA and Europe, respectively. Hence, a review to the international codes such as ACI 318 [2] and EN 1992-1-1 [4] is envisaged. Although the traditional bonded anchor (BA) theory for fastening may apply to PIR, this design is only valid for short anchorages. The design is codified in the recently published EN 1992-

4 [5] with the introduction of a beneficial coefficient which accounts for moment connections. Last year, a technical report EOTA TR 069 [9] further enriched the content of BA design by introducing an improved bond-splitting behaviour (with the intended working life stated in the ETA according to EAD 332402 [8]).

For longer design lengths or other preferred failure modes, the Rebar Anchorage (RA) theory becomes more suitable. EN 1992-1-1 [4] allows for bond strength improvement by large concrete cover. Further concrete confinement may also help to push the bond strength to an extreme as if cast-in-situ reinforcements (CIR) are used with qualified adhesives. Nevertheless, the structural behaviour of these long-embedded anchors should be totally different to short bonded anchors. Situations are even complicated in moment-resisting connections. Muenger *et. al*. [19] and Kupfer *et. al*. [12] proposed the strut and tie model (STM) to identify the force flow and the related failure modes. Later on was validated by Hamad *et. al*. [11]. This STM theory can be employed together with the RA theory.

Various research has been done on the column-foundation connections of sub-structures [17]. However, a wider category of superstructures has been overlooked e.g. wall-slab and beam-column (Fig. 1). Hence, a collaborative research by RMIT and the University of Hong Kong has been conducted on the topic of “the Structural Behaviour of PIR in Wall-Slab Moment Connections. A series of full-scale experimental studies were conducted to explore the structural behaviour of applying PIR which connected wall and slab. Validations and recommendations on these governing theories of BA, RA and STM have been proposed in this article.



CFJ *WFJ* *SWJ* *BWJ* *BCJ*

Fig. 1: PIR moment connections for both sub-structures and super-structures.
(source: EOTA TR 069 [9])

2. Experimental study

Wall-slab connections on top floor of buildings were investigated in laboratory (Fig. 2). They were subject to higher closing moments than intermediate floors. The experimental setup is shown in Fig. 2.2. To facilitate the application of the load, samples were rotated anticlockwise with slab upright and wall mounted horizontally on a testing bay. A reinforcement cage for the wall was fixed and welded to a steel angle which would prevent the crushing and displacement of the concrete edge. Wall elements (500 W × 1200 L with varying depths) were cast first. Seven days later, the contact surface for slab connection was mechanically roughened based on the intended use in accordance with EN 1992-1-1 [4]. The mechanical tool provided interface roughness factors c and μ (for determining the capacity of the shear stress, τ_{Rdi}) of 0.45 and 0.7, respectively. Holes were drilled by the rotary-impact method. They were repeatedly cleaned and dried by flushing with water and wire brushing, according to EAD 330087 [6] and the relevant manufacturer’s installation instructions. A two-component resin was injected using the piston plug provided by the manufacturer in order to assure a void-free installation. Three days later, slabs (500 W × 200 D) were cast.

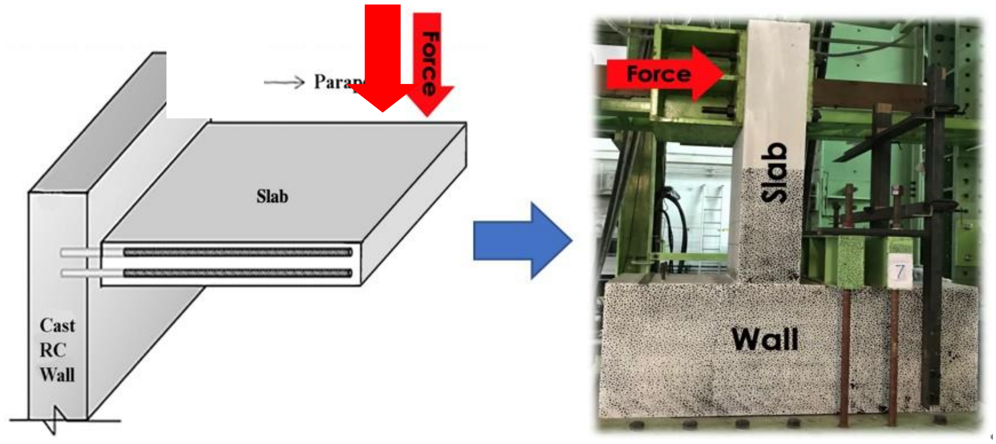


Fig. 2: Laboratory test sample (wall-slab connection) for closing moment cases.

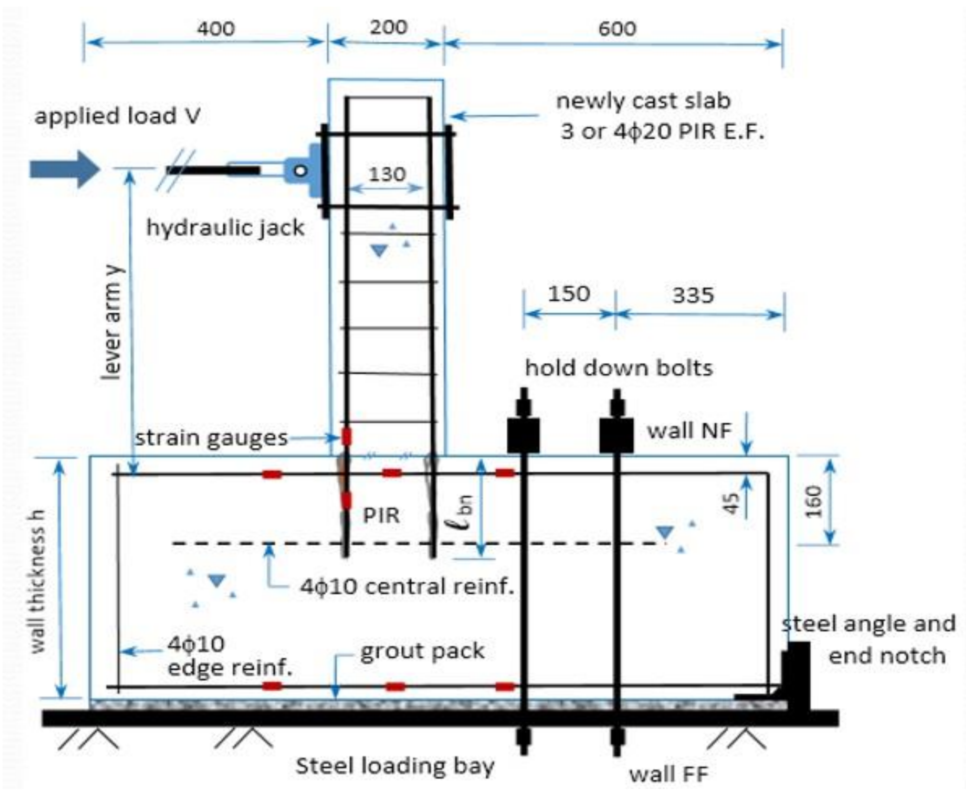
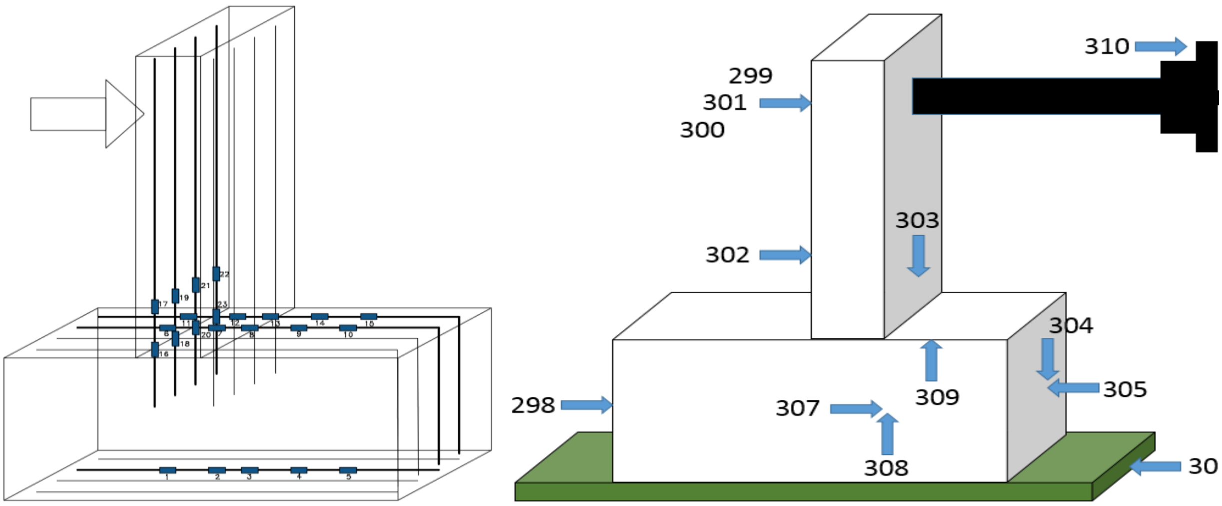


Fig. 3: Experimental setup. *(source: Lee et al. [14])*

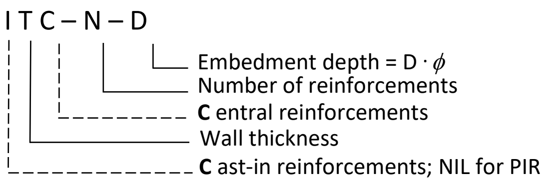
Samples were hauled to the testing bay with cement grouting under the wall and bay for a flat contact and correct alignment. The right side of sample was restricted from moving in both the horizontal and vertical directions by using end notches and holding down bolts, respectively (Fig. 3). A hydraulic jack was used to monotonically apply horizontal load on the slab from left to right at a rate of 0.06 mm/s. The test stopped when 20% softening of the ultimate load was attained. The whole deformation process was monitored by using linear variable differential transformers (LVDTs), strain gauges and digital image correlation (DIC) equipment. The arrangement of LVDTs and strain gauges are shown in Fig. 4. DIC was carried out by taking digital images every 6 seconds. A layer of lime plaster was applied to the reinforced concrete surface around the connection and randomly sprayed with black paint to form the speckles for DIC analysis.



Front view Rear view

Fig. 4: Strain gauges and LVDT layouts.

Labelling scheme of a 3-field alphanumeric code is used. The fields are defined in Fig. 5. For example, 500C-3-23 means that the thickness of the wall is 500 mm with a layer of reinforcements in the centre (C) of the wall, along with the PIR bar (ϕ 20 T&B) number (3) and the embedment depth (23ϕ). In this case, 'C' denotes the presence of 4ϕ16 longitudinal reinforcements. As control, CIR samples were also cast and tested. An alphabet 'C" in front of the code denotes for CIR rather than PIR such as C500-4-20.



(Note: all in mm; ϕ is reinforcement diameter)

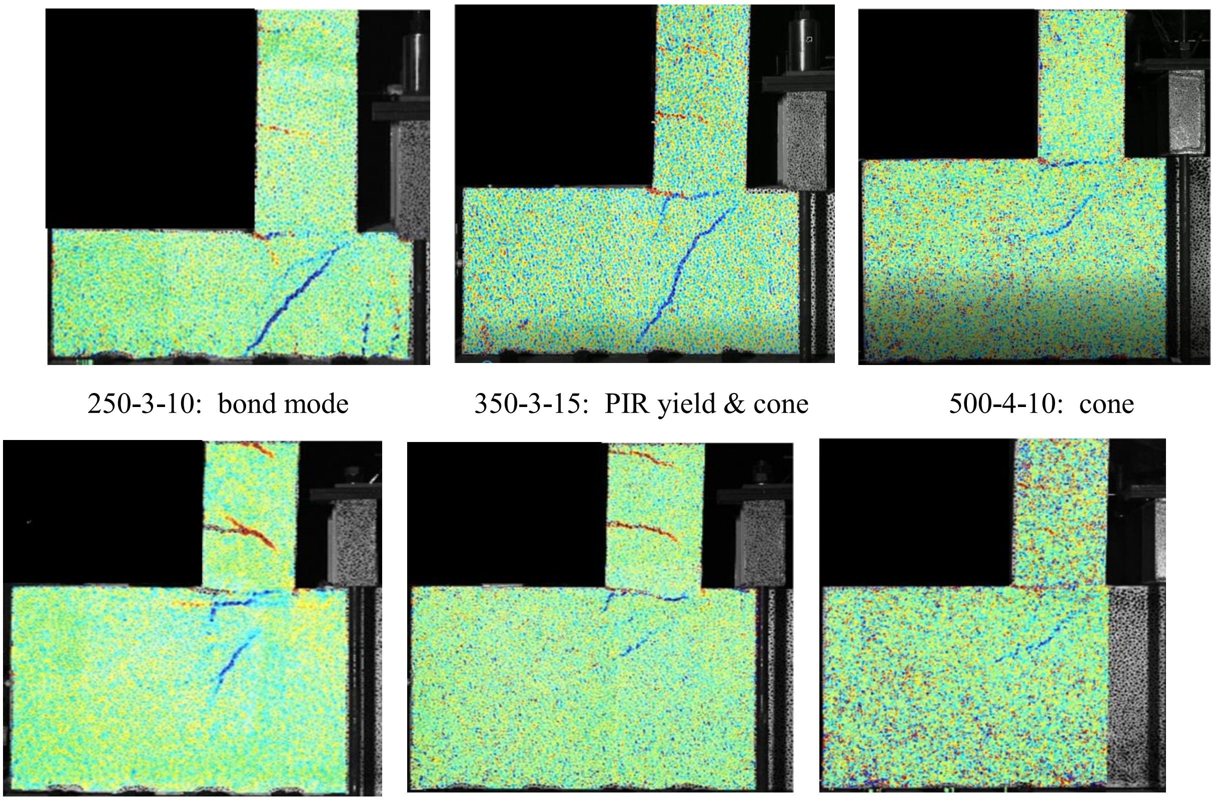
Fig. 5: Labelling scheme of test samples.

3. Experimental Results

Six PIR and two CIR samples were tested with various anchorage depths and wall thicknesses (Table 1). Details may be referred to the article Structural behaviour of post-installed reinforcement bars in moment connections of wall-slabs [14]. The 1st Major crack propagated in wall structures and the corresponding failure mode are given in Fig. 6.

Table 1: Testing configurations of samples (in mm).

Sample	225-3-10	350-3-15	500-4-10	500-3-23	500C-3-23	500-4-20	C500-4-15	C500-4-20
Wall thick	225	350	500	500	500	500	500	500
PIR	3φ20T&B	3φ20T&B	4φ20T&B	3φ20T&B	3φ20T&B	4φ20T&B	4φ20T&B	4φ20T&B
Anchorage	200 (10φ)	300 (15φ)	200 (10φ)	460 (23φ)	460 (23φ)	400 (20φ)	300 (15φ)	400 (20φ)
Lever arm, *y*	1160	1035	885	885	885	885	885	885
Wall rebars	4φ20NF;4φ12FF	4φ20EF	4φ20EF	4φ20EF	4φ20EF	4φ16NF;4φ20FF	4φ20EF	4φ20EF



250-3-10: bond mode   350-3-15: PIR yield & cone   500-4-10: cone

500-3-23: PIR yield & strut   500C-3-23: PIR yield & strut   500-4-20: strut

Fig. 6: 1st Major crack propagated in wall structures. *(source: Lee et.al. [14])*

Multiple cracking propagations were normally observed in all testings. 1st major crack caused a local softening to structures which may have been due to cone or strut failure. Forces were soon distributed within the R.C. until another major cracks propagated and then the load peaked except in short anchorage cases where cone/bond failure dominated (225-3-10 and 500-4-10). All load defection curves are plotted in Fig. 8. It is observed that the structural behaviour of PIR and CIR are similar. Hence, the anchorage design of PIR is very conservative under the current standards.

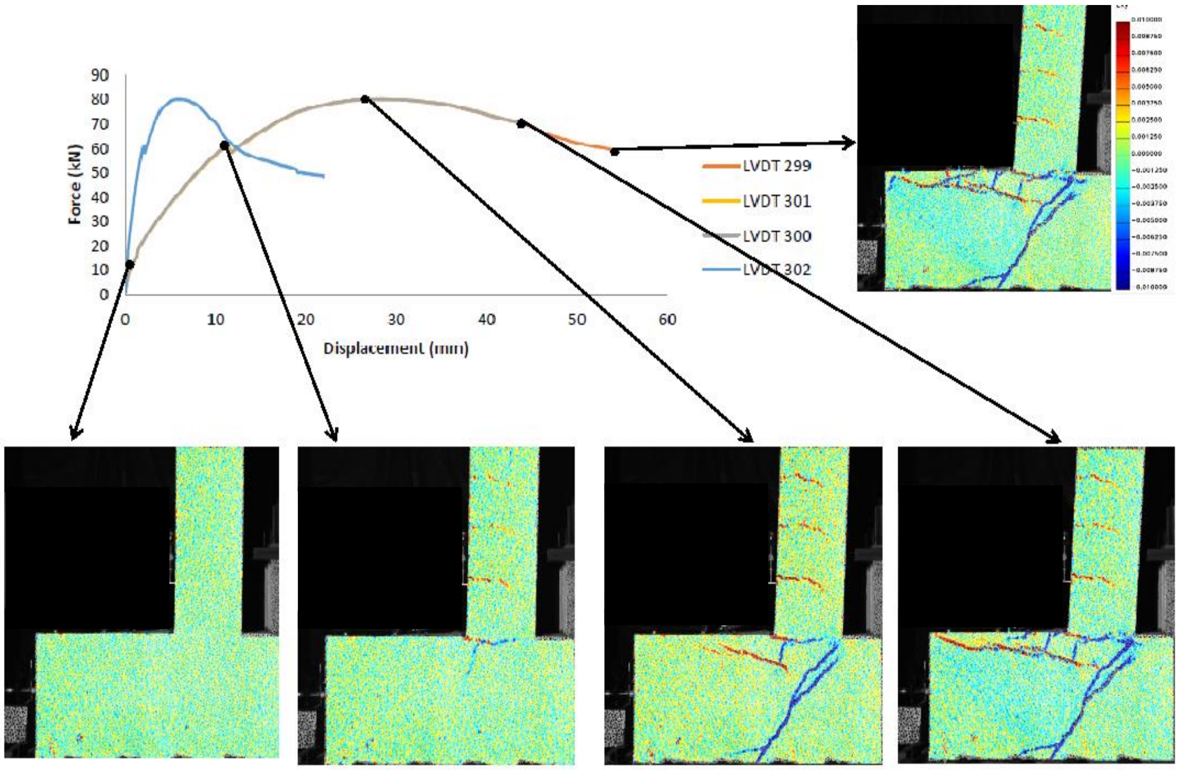


Fig. 7: Cracks propagation along the load deflection curve of sample 350-3-15.

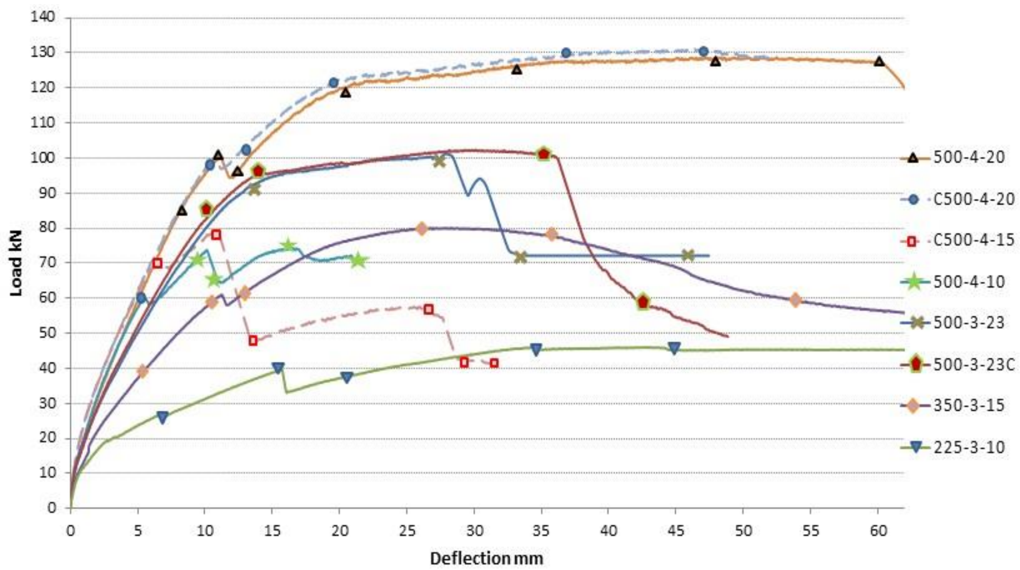


Fig. 8: Load Deflection Curves of PIR and CIR.

4. Predicted System Failure Modes

Fig. 9 shows various failure modes according to different theories:

1. RA - EN 1992-1-1 [4] [1]
2. BA - EN 1994 [5][2], EOTA TR 069 [9] [3]
3. STM - EN 1992-1-1 [4] [4]

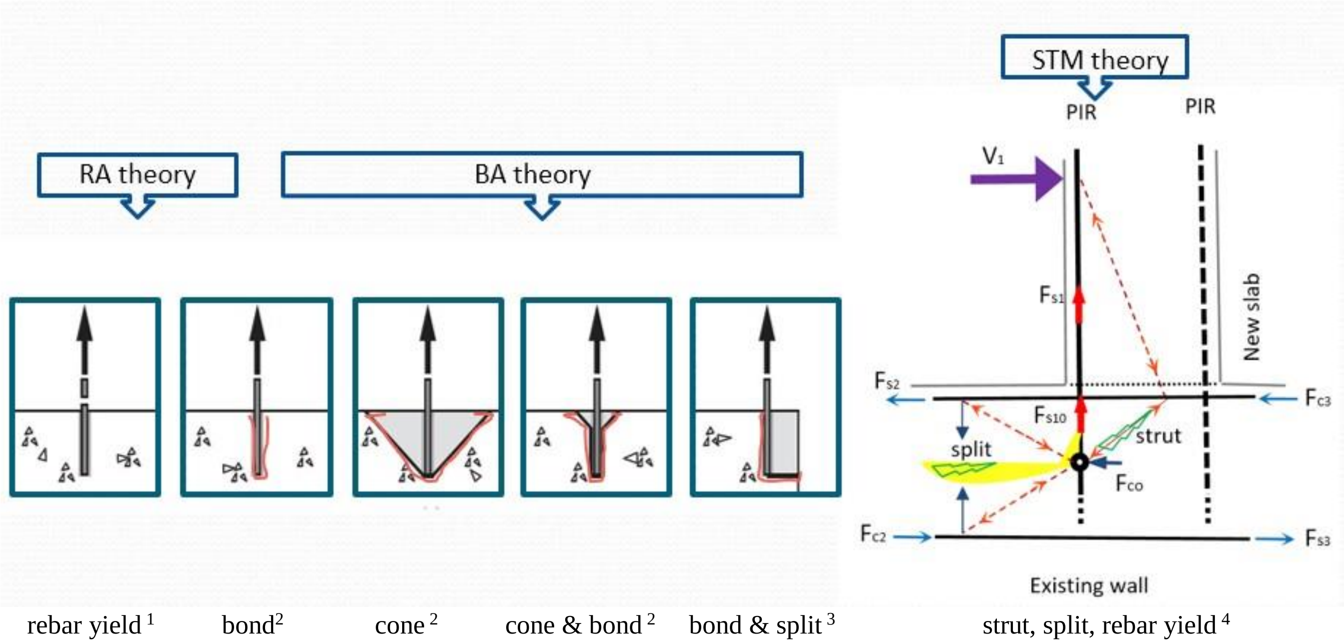


Fig. 9: Various failure modes according to different theories.

As observed from the experimental results, in order to increase the ductility of moment connections, longer anchorage lengths are preferred by Engineers i.e. to adopt the RA theory. Together using with the STM theory, the anchorage length may be shortened to fit the practical situations. Before introducing the BA and RA theories and the related experimental analysis, a fundamental knowledge of STM is illustrated.

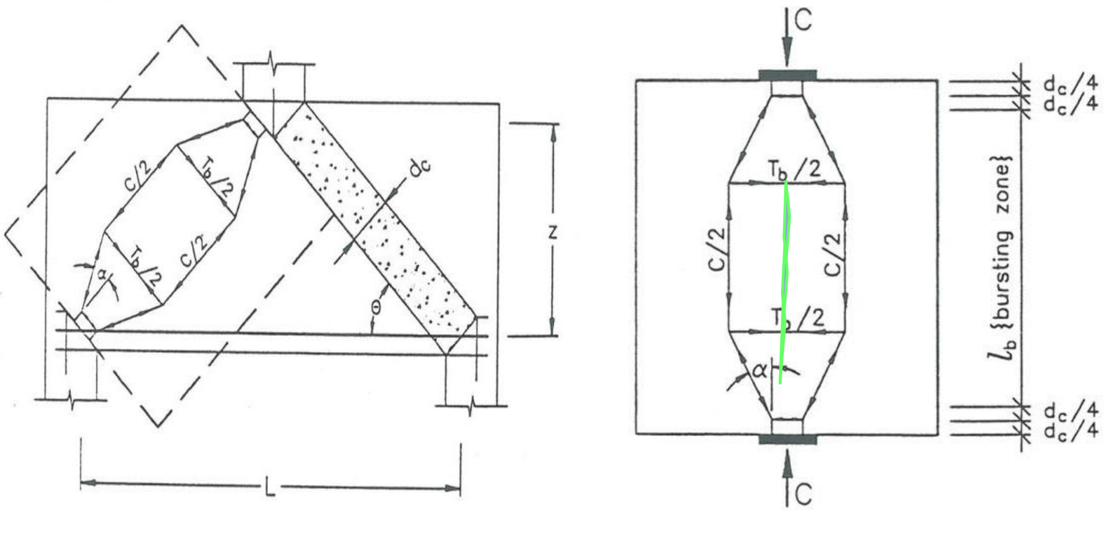


Fig. 10: Splitting failure mechanism formed in compressive strut. *(source: Foster and Gilbert [10])*

For linear material, continuity requires the compressive struts in deep beams and columns as given in Fig. 10 be bowed rather than being parallel sided. A maximum width of strut is formed prior to cracking. Once cracking occurs, the strut becomes narrower significantly. Deviation strut angle α = Atan (1/2) and strut angle θ = Atan (1/5) were proposed by Foster and Gilbert [10]. Hence, the applied bursting force $T_b = C \cdot \tan \alpha$ with splitting coeff. K = 0.6. Finally the critical bursting force arrives $T_{b,cr} = l_b \cdot b \cdot K \cdot f'^{1/2}$. The compressive strut failure happens if exceeding that force.

5. Bonded Anchors Theory

BA theory is recently updated in EN 1992-4 [5] and EOTA TR 069 [9] for cone, bond and splitting failures with the prescribed qualification requirements. Notations may be referred to these documents.

1. Concrete cone capacity

$$N_{Rk,c} = K \cdot f_{ck}^{\,0.5} \cdot h_{ef}^{\,1.5} \cdot A_{c,N} / A^0_{c,N} \cdot \Psi_{s,N} \cdot \Psi_{re,N} \cdot \Psi_{ec,N} \cdot \Psi_{M,N} / \Upsilon_{MC}$$

where
$K_{cr} = 7.7$ (for cracked concrete)
$K_{ucr} = K_{cr} / 0.7 = 11$ (for uncracked concrete)
$C_{cr,N} = 1.5\, l_b$ ($C_{cr,N}$ critical edge distance)
$A_{c,N}^{\,0} = (C_{cr,N})^2$
$\Psi_{s,N} = 0.7 + 0.3c/C_{cr,N} \leq 1.0$
$\Psi_{M,N} = 2.5 - z_1 / 1.5\, h_{ef}$ (from numerical result)

As observed from experiments, in moment connections, a compressive strut (a kind of STM) was formed even in cone failure. A new beneficial coefficient $\Psi_{M,N}$ was introduced by Mahrenholtz *et al.* [16] and put in EN 1992-4 [5]. But $\Psi_{M,N}$ is reduced to $2.0 - z_1 / 1.5\, h_{ef} \geq 1$. If this STM strut angle is effective in the range of $30^{\circ} < \theta < 63^{\circ}$, a limitation should be added to the code with $1.1 \geq \Psi_{M,N} \geq 2$.

2. Combined cone and bond capacity

$$N_{Rk,p} = \pi \cdot \varphi \cdot h_{ef} \cdot \tau_{Rk} \cdot A_{p,N} / A^0_{p,N} \cdot \Psi_{g,Np} \cdot \Psi_{s,Np} \cdot \Psi_{re,Np} \cdot \Psi_{ec,Np} / \Upsilon_{Mp} \cdot \Psi_{M,N}$$

where
$C_{cr,Np} = 10 \cdot \varphi \cdot (\tau_{Rk,ucr} / 7.5)^{0.5}$ (τ_{Rk} characteristic bond strength)
$A_{p,N}^{\,0} = (C_{cr,Np})^2$
$\Psi_{s,Np} = 0.7 + 0.3c/C_{cr,Np} \leq 1.0$

If $h_{ef} \geq 10\,\phi$, $\Psi_{M,N}$ can reflect the increase in the bond strength due to the compressive stress which is acting at the lower end of the anchorage (refer to Lee *et. al.* [14]). Hence, this beneficial coefficient should be introduced to EN 1992-4 [5] and EOTA TR 069 [9].

3. Bond-splitting capacity

$$N_{Rk,sp} = \tau_{Rk,sp} \cdot l_b\, \pi \cdot \phi$$

$$\tau_{Rk,sp} = \eta_1 \cdot A_k \cdot (f_{ck}/25)^{sp1} \cdot (2/\phi)^{sp2} \cdot [(C_d/\phi)^{sp3} \cdot (C_{max}/C_d)^{sp4} + k_m \cdot k_{tr}] \cdot (7\phi/l_b)^{lb1} \cdot \Omega_{p,tr}$$

$\tau_{Rk,ucr} \leq \Omega_{cr} / \Omega_{p,tr}\ \psi_{sys}$ for $7\,\phi \leq l_b \leq 20\phi$
$\tau_{Rk,ucr} \leq (20\,\phi / l_b)^{lb1}\ \Omega_{cr} / \Omega_{p,tr}\ \psi_{sys}$ for $l_b > 20\phi$

Where A_k, Ω_{cr}, sp1, sp2, sp3, sp4 and lb1 are provided in the ETA. Others are calculated according to EOTA TR 069.

6. Rebar Anchorage Theory

The RA theory is explicit given in EN 1992-1-1 [4]. The design bond strength of adhesive is

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd}\,.$$

Where η_1=1.0 or 0.7 for good or poor bond, respectively. $\eta_2 = (132 - \phi)/100 \geq 1$. If concrete confinement is taken into consideration, a full adhesive bond strength will be obtained. Splitting failure will then be replaced by bond failure. An adapted factor α'_2 is proposed to replace α_2 the coefficient for the effect of minimum cover (as given in EN 1992-1-1 [4]). Hence, the required anchorage length is $l_{b,rqd} \geq \sigma_{sd} \cdot \phi / (4 f_{bd})$. And the design anchorage length is

$$l_{bd} \geq \alpha_1 \cdot \alpha_2 \cdot l_{b,rqd} \geq l_{b,min}$$

where $\alpha_1 = 1.0$ for straight reinf., $0.7 \leq \alpha_2 = 1 - 0.15(C_d - \phi)/\phi \leq 1.0$ with $C_d \geq 3\phi$ and

$\alpha'_2 = [1/0.7 + \delta\,(C_d - 3)/\phi]^{-1} \geq 0.25$ with δ is increased in f_{bd}.

7. Strut and Tie Model Theory

Once the anchorage length of PIR is determined by the above modified RA theory, the whole supporting structures shall be checked against with the R.C. theory for bending and shear failures. Moreover, the reinforcements of the structure and concrete failure due to compressive strut and splitting shall be checked according to STM theory as well. Notations given by Kupfer *et al*. [12] are shown in Table 2.

Table 2: STM notations. *(source: Kupfer et al. [12])*

	Slab - zone 1 di _	Ex wall - zone 2&3	ing eas 2
Transverse force at the end-face section	V_1	V_2	V_3
Longitudinal force at the end-face section	N_1	N_2	N_3
Tensile force in the reinforcement	F_{s1}	F_{s2}	F_{s3}
Compressive force in the concrete	F_{c1}	F_{c2}	F_{c3}
Internal forces' lever arms	Z_1; Z_{1r}	$Z_2 = Z_3 = Z$; Z_{sp}	
		Node - zone 0 tinuity	
Tensile force in the existing top reinforcement		F_{s0}	
Compressive force (clamping force) at a right angle to the connecting reinforcement		F_{c0}	
Principal diagonal strut in the node		D_o	
Diagonal compression struts in the discontinuity region		D_{s0}, D_{0s}	
Splitting tensile force in the discontinuity region		S_0	
Splitting tensile stresses in the discontinuity region		σ_{sp}	
Net drilled hole depth		t_{bn}	
Anchorage length in the drilled hole		L_{b1}	
Internal forces' lever arm in the node		Z_0	
Support reaction in the new building component's axis		A	

1. <u>Anchorage check</u>

$F_{c0} = M / z_0$

$l_{b1} = F_{s10} / (f_{bd}\, \Sigma u)$ (f_{bd} is design bond strength; u is bar perimeter)

Demand to capacity ratio $R = l_{b1} / l_{bn} < 1$

2. <u>Base top rebar check</u>

$F_{s0} = V_1 \cdot (1 + y/z_0)$

$z_0 = l_{bn} - l_{b1}/2$

$\sigma_{s0} = F_{s0}/As_0 \leq f_y$

3. <u>Base bottom rebar check</u>

$F_{s3} = V_1 \cdot y \,/\, z_3$ (z = z_2 = z_3 as base T&B bars c/c)

$\sigma_{s3} = F_{s3}/As_3 \leq f_y$

From experiments, STM may apply if $l_{bn} > 10\ \phi$.

4. Strut compressive stress in direct node region

$D_{0d} = F_{co} \,/\, \cos\theta$

$D_{0Rd} = 0.75\,(30/\, f_{ck}\,)^{1/3}\, \alpha_{cc} \cdot f_{ck} \,/\gamma_c \cdot (b \cdot l_b \cos\theta)$ ($\tan\theta = z_0 \,/\, z_{1R}$ and $\alpha_{cc} = 1$)

Where θ is the strut horizontal angle and hyperbolic relation [18] is suggested rather than linear. This is agreed with Su *et. al*. [20] on strut efficiency factor.

5. Splitting tensile stress in discontinuity region

$F_{co} = V_1 \cdot y \,/\, z_0$

$M_{sp} = F_{co} \cdot z_0\,(1 - z_0\,/z_2)\,(1 - l_{b1}\,/2\, z_2)$

$W_{sp} = b \cdot\ z^2\,/2.14$

$\sigma_{sp} = M_{sp} \,/\, W_{sp} \leq f_{ctk,\,0.05} = \alpha_{cc}\, 0.7 \cdot 0.3\, f_{ck}^{\ 2/3}\,/\gamma_c$

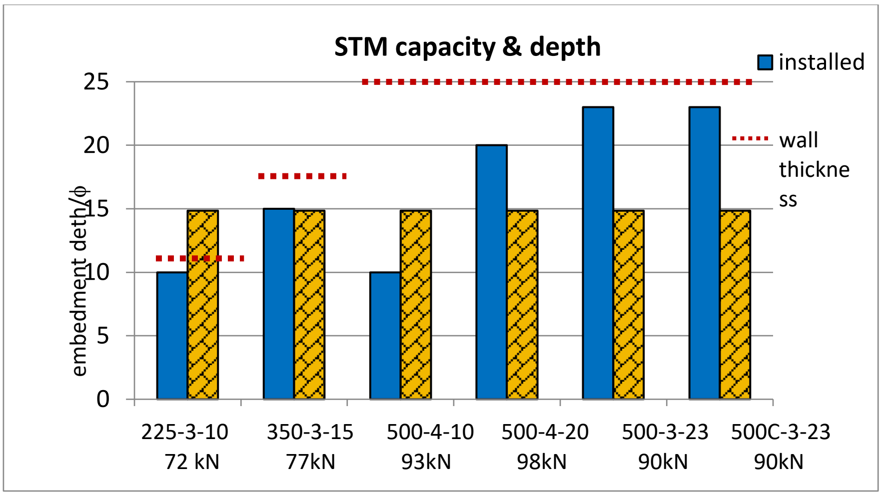


Fig. 11: Theoretical capacity of 15 ϕ embedment depth at strut angle 60°.

As mentioned earlier that the STM strut angle is effective in the range of $30^{\circ} < \theta < 63^{\circ}$, however, it is not a direct exercise to determine the strut angle. A lower bound of the strut capacity using $\theta = 60^{\circ}$ for determining the z_0 is, therefore, suggested. If an embedment depth of 15 ϕ is used, the theoretical PIR capacities are well matching with the experimental results as shown in Fig. 11.

8. Analytical Results

Analytical solutions according to the fore-mentioned proposed theories are attempted to explain the experimental behaviour and the modes of failure. The comparisons are given in Table 3. Taking the fact that PIR started to yield at 504 kN from the test of 500-3-23, different codes offer different design anchorage lengths l_{bd} according to the RA theory due to different design bond strength of adhesive f_{bd} allowed. However, if STM confinement is used, l_{bd} can be as short as 416 mm i.e. about 21ϕ < 23ϕ. If shorter anchorage is used and reinforcements are preventing from yield as sample 500-4-20 does, the 1st major crack is found to be the strut failure according the analytical STM. If the peak load is reached, the far face of wall rebars* will yield as well. On the other hand, BA theory estimates that the 2nd major crack is due to cone failure or even bond and splitting is possible (Table 3).

Table 3: Comparisons of experimental and analytical results

RA theory: sample 500-3-23			
Experimental PIR design load N	Analytical f_b/l_{bd} (N/mm²/mm)		
PIR Rebar yield	HK code [3]	EN 1992-1-1 [4]	STM confinement
504 kN (Y mode)	2.9 / 820	5.4 / 430	**<u>6.3 / 416</u>**
STM theory: sample 500-4-20			
Experimental applied load V	Analytical capacity ratio R (failed at $0.9 \leq R \leq 1.0$)		
1st major crack	Strut	Wall NF/FF rebars	Tension split
93 kN (S mode)	**<u>0.97</u>**	0.30 / 0.73*	0.48
BA theory: sample 500-4-20			
Experimental PIR load N	Analytical capacity ratio R (failed at $0.9 \leq R \leq 1.0$)		
2nd major crack	Cone $N_{R,c}$	Cone & bond $N_{R,c}$	Bond & split $N_{R,c}$
750 kN (C mode)	**<u>0.93</u>**	0.46	0.88

9. Conclusions

This article reviews the recent breakthrough on the adoption of design of PIR in moment resisting connections as post-installed anchors or post-installed reinforcement as given in EN 1992-4 [5] and EOTA TR 069 [9], respectively. In addition, some theoretical modifications are proposed according to experimental results. However, these design applications are limited to short anchorage which directly restrict the load capacity. If the ductility of systems is paramount or even no cracking is allowed on the supporting structures, RA theory with STM is an ideal option for engineers to oversee the structural behaviour and control the failure modes. The state-of-the-art of PIR design and installation have been codified as per the Hong Kong code [15]. In fact, in the design process, both the new and existing structures should take into consideration. The former must not be over-designed in order to protect the structural integrity of the latter. The effect of PIR system to the newly cast structures is another research area to supplement the design of PIR [13].

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CLOSING REMARKS

Ir Michael LEUNG

General Manager HILTI (HK) Limited

I would like to thank you all for joining this Half-Day Webinar on Recent Advances in Design Methodology for Post-installed Reinforcements.

The technology of post-installed reinforcement is growing increasingly important since reinforcing bars are being used frequently in horizontal, vertical and overhead applications on rehabilitation and strengthening of existing structures. Post-installed reinforcements are also used in specific situations in new construction to simplify construction procedures and provide flexibility in design and construction. This half-day webinar provided a platform for overseas prominent academics and eminent engineers to share their insights, experience and ideas on the latest development in new design methodology of post-installed reinforcement. I am sure what they have shared are very useful to practising engineers and construction professionals.

I would like to take this opportunity to thank all the speakers for their sharing and valuable time. I would also like to express my heartfelt thanks to the assistance provided by the supporting organisations, including The Hong Kong Institution of Engineers, The Institution of Civil Engineers Hong Kong, The Hong Kong Institute of Steel Construction, The Hong Kong Institute of Vocational Education and Hilti (HK) Limited.

Lastly, I would like to take this opportunity to give my sincere thanks to the organizing committee for their great efforts on making this event a big success. I hope you all have a pleasant day and find this webinar informative and enjoyable.

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