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Design and installation of post-installed reinforcements: A state-of-the-art review

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ABSTRACT

Post-installed reinforcement (PIR) helps facilitate retrofitting works, mitigate misplaced reinforcement problems, as well as support newly cast concrete members such as modular integrated construction. However, it has not been holistically addressed in major international reinforced concrete (RC) design codes. Nonetheless, based on established design philosophy and associated failure modes, the cast-in reinforcement design method in RC can be extended to design qualified PIR systems. The qualification of PIR system can be referenced to Acceptance Criteria (AC) 308 (2016), European Assessment Document (EAD) 330087 (2018) and EAD 330499 (2017) in the US and Europe, respectively. In Hong Kong, PIR is conservatively limited to shear connections. Its assumption of pinned connection is less justifiable for some deep sections of beams, which may induce hogging moments, causing tension at the top reinforcement of the supports. In some cases of cantilever slabs, moment connections are necessary to maintain equilibrium. Hence, this paper reviews an up-to-date design methodology and installation guide to complement the Hong Kong Code of Practice for Structural Use of Concrete (HKBD) by referring to the recently published international design codes and documents. The proposal is useful to promote economical, sustainable and technically sound use of PIR system.

KEYWORDS

Bonded anchor (BA); modular integrated construction; post-installed reinforcement (PIR); reinforcement anchorage (RA); strut-and-tie model (STM).

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1. Introduction

Post-installed reinforcement (PIR) system is different from post-installed anchor bolt system, where PIR is used for concrete-to-concrete connections and anchor bolts are used for steel-to-concrete connections. In the application of PIR, holes drill into one side of existing concrete and the reinforcements are inserted into the drilled holes together with adhesive. The protruding bars on the other side of the interface are cast into new concrete at a later stage. Hence, PIR often serves as starter-bars or lap splicing-bars in modern construction~~s~~ method such as modular integrated construction and it can eliminate the problem of misplacement of reinforcements or defective couplers by allowing existing concrete structures to support newly cast components. PIR bars are versatile and can be applied in almost any location on concrete for rehabilitation and strengthening projects, such as horizontal, vertical and overhead applications. Some typical application examples of PIR are shown in Figure 1, which include (i) end connection of new beam/ slab into walls, (ii) lap splice of new slab to existing slab, (iii) connection of new column onto foundation, and (iv) new concrete overlays (e.g. for wall strengthening, column jacketing or slab thickening).

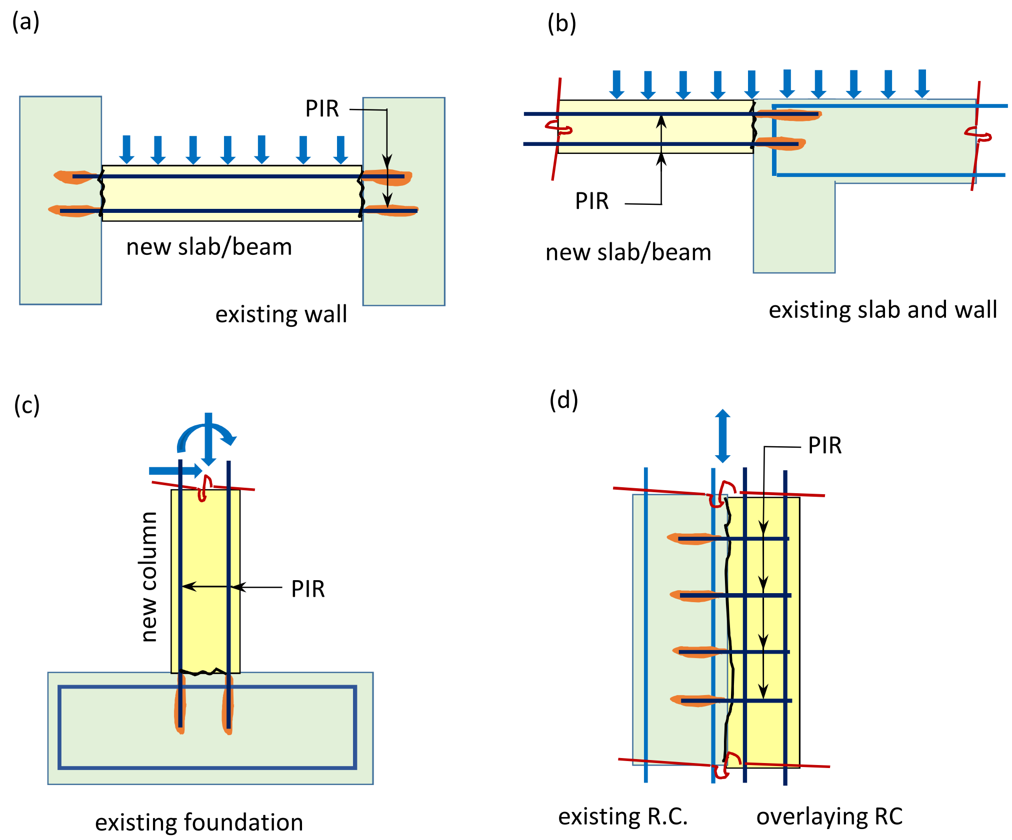

Figure 1. Typical PIR applications (a) end anchor of new beam/slab into walls; (b) lap splice of new slab to existing slab; (c) end anchor of new column onto foundation; and (d) new overlays.

2. PIR installation guide

Figure 2 shows the typical six-step installation procedures of PIR, where the first step is to detect existing reinforcements, followed by surface roughening, drilling, hole cleaning, adhesive injection, and finally, rebar insertion. Up-to-date PIR installation guides governed by the Hong Kong, European and American standards are briefly discussed in this paper on selected entities, i.e. materials, qualification of installers and guidance on surface roughening. Other details on reinforcements detection, hole drilling, hole cleaning, adhesive injection and rebar insertion should refer to the relevant Manufacturer's Published Installation Instructions (MPII).

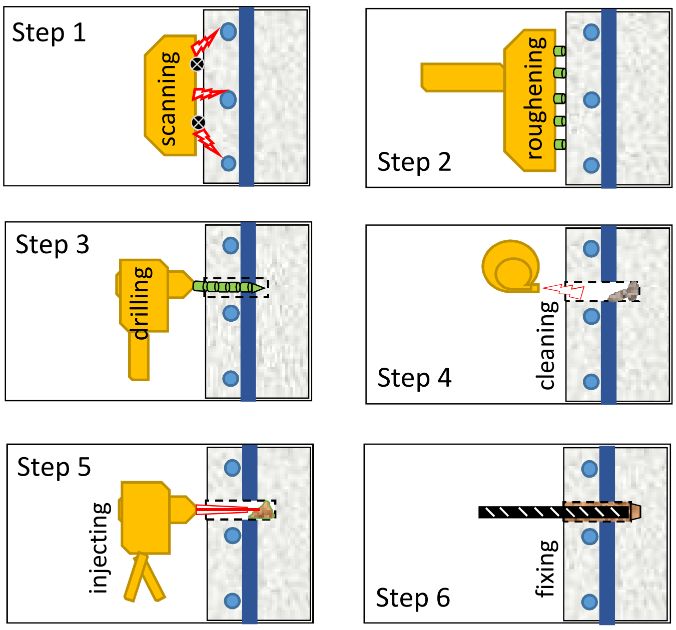

Figure 2. Typical installation sequential procedure of PIR.

2.1. Temperature effect on adhesive materials

One of the important components in PIR is the adhesive material. An adhesive material with higher bond strength may result in shorter embedment depth design. Hence, understanding the requirement of the adhesive is crucial to ensure its bond performance. It is known that adhesives are sensitive to temperature in three stages (Gamache, 2017):

(i) The storage temperature of the adhesive can influence the shelf life;

(ii) Installation temperature of concrete and adhesive affects the gel, cure time and adhesive viscosity; and

(iii) Service life temperature of the concrete structures, which when elevated can remarkably affect the bond strength of PIR.

The suggested storage temperature of the adhesive before installation is provided in MPII. A non-universal but effective solution is to keep the temperature of adhesive at approximately 20°C before installation to allow an optimal injectability. The design of PIR including the selection of adhesives should cater to the service temperature in the base material. In general, a high base material temperature reduces the bond strength of the adhesives. On the contrary, low temperature has little negative impact on the bond strength, and this has been verified in freeze-thaw tests (Gamache, 2017). For PIR subject to high temperature or fire, specialist advice and data should be sought from the product manufacturer. The variation of adhesive bond strength with temperature obtained through tests (see European Organisation for Technical Assessment (EOTA) European Assessment Document (EAD) 330087, 2018 or other European National Approvals) should be provided by the product manufacturer. It is noted that the temperature at a depth within concrete will often be much lower than that at the concrete surface. Therefore, a longer embedment depth to compensate for the loss of bond strength close to the surface of the concrete appears to be beneficial in mitigating the effects of fire (Interim Advice Note (IAN) 104/15, 2015 and British Standard (BS) 8539, 2012). The choice of adhesive depends on the use, loading direction, environmental considerations, anchorage length, reinforcement diameter, drilling method and on-site conditions. It should be noted that some adhesives for anchor systems may not be used for PIR. Only those approved adhesives for PIR by EOTA EAD 330087, 2013 (replacing EOTA Technical Reports (TR) 023, 2006) or Acceptance Criteria (AC) 308, 2016 Table 3.8 are suitable.

2.2. Concrete base materials

Several base materials are suitable to adopt the PIR system, for example, masonry, normal weight (cracked/uncracked), lightweight and prestressed concrete. In this paper, only the normal weight reinforced concrete (RC) structure is elaborated as it is commonly used in Hong Kong. PIR can be used in concrete grade ranging from C12/C15 to C50/C60 (characteristic cylinder/cube strength in MPa), conforming to EOTA EAD 330087, 2018 and AC 308, 2016. For higher grade concrete (> 70 MPa) as allowed in the Hong Kong Code of Practice for Structural Use of Concrete (HKBD), 2013, the bond strength of PIR is capped at the limit of C60, unless justified by special technical data from the manufacturer. The minimum thickness of the concrete members in which reinforcement will be installed should be greater or equal to the sum of the minimum anchorage length of the PIR and the minimum cover thickness.

2.3. Reinforcement materials

Similar to concrete, reinforcement should conform to the HKBD, 2013, with the exception that plain steel reinforcement of grade 250 is forbidden to be applied in the PIR system. Deformed carbon steel reinforcement in grade 500B and 500C with surface geometry (i.e. rib parameters, relative rib areas, longitudinal and transverse ribs) complying with Construction Standard (CS) 2, 2012 shall be used. It is noted that these requirements for reinforcement in terms of geometry are the same as Eurocode (EN) 1992-1-1, 2004 Annex C.

2.4. Installers and supervision

PIR shall be handled by qualified installers in accordance with the construction documents and, where applicable, MPII (see Annex F.2(b) and (c) in EN 1992-4, 2004 and American Concrete Institute (ACI) 318, 2014 Cl. 17.8.1). The manufacturer's recommendations for the specified adhesive system should take precedence over all other guidance. Qualification of the installer should be acquired through certification programs. The commentary in ACI 318, 2014 Cl. R17.8.2.3 states that "An equivalent certified installer program should test the installer's knowledge and skill by an objectively fair and unbiased administration and grading of a written and performance exam. Programs should reflect the knowledge and skill required to install available commercial adhesive systems. The effectiveness of a written exam should be verified through statistical analysis of the questions and answers. An equivalent program should provide a responsive and accurate mechanism to verify credentials, which are renewed on a periodic basis."

Making reference to BS 8539, 2012, the installer is required to be trained by a competent trainer including (i) on-site training on the installation of PIR for different adhesives; (ii) knowledge of PIR function, material safety data sheets and consequence of improper installation; and (iii) closer supervision should be provided for installer has limited experience.

2.5. Roughening of concrete surface

Roughening on existing concrete surfaces before the casting of adjoining fresh concrete can increase both the adhesion and joint friction. The carbonated layer should be removed in the areas that are to receive PIR. A rule of thumb is to remove the carbonated concrete over a circular area with a diameter (d_{rough}) given by bar diameter plus 60 mm (i.e., $d_{\text{rough}} = \phi + 60\text{mm}$). The requirement of roughening in HKBD, 2013 and EN 1992-1-1, 2004 are briefly introduced.

(i) Roughening according to HKBD

Similar to construction joints, HKBD, 2013 Cl. 10.3.10 suggests that roughening can be done through fine spraying of water, stiff brushing, sandblasting or by scale hammer. The joint must be clean and free from loose particles. Roughening can be done by water spraying and/or brushing for approximately two to four hours after the concrete placing. Damage or dislodge the coarse aggregate particles should be avoided.

(ii) Roughening to EN 1992-1-1

Compared to the deemed-to-comply qualitative approach in HKBD, 2013, EN 1992-1-1, 2004 Cl. 6.2.5 provides a quantifiable roughening calculation (shear friction) for PIR design. The demand of shear stress ($\tau_{\text{Edi}} = \beta\ V_{\text{Ed}} / (z\ b_{\text{i}})$) at interface should be less than shear capacity ($\tau_{\text{Rdi}} = \text{c}\ f_{\text{ctd}} + \mu\ \sigma_{\text{n}} + \rho\ f_{\text{yd}}\ (\mu \sin\alpha + \cos\alpha) \leq 0.5\ \nu\ f_{\text{cd}}$) with relevant parameters. β is the area ratio of longitudinal force in new concrete to total longitudinal force either in compression or tension zone; V_{Ed} is the transverse shear force; z is the lever arm of the section; b_{i} is the interface width; c and μ are the factors which depend on the roughness of the interface; f_{ctd} is the concrete design tensile capacity; σ_n is the stress caused by the minimum external normal force across the interface that can act simultaneously with shear force.

If σ_n is in tension, c is equal to zero; ρ is the area ratio of reinforcement (A_s) crossing the interface, including ordinary shear reinforcement to the joint area (A_i); f_{yd} is the value of the PIR design yield capacity; α is the inclination angle formed by the longitudinal axis of reinforcement with contact interface, and limited between 45° - 90°; v is the shear strength reduction factor for cracked concrete; f_{cd} is the concrete design cylinder strength. A similar formulation with more detailed variance is given in the Model Code for Concrete Structures (fib), 2010 Cl. 7.3.3.6. An interaction coefficient (κ_1) for tensile force activated in the dowel steel and interaction coefficient (κ_2) for flexural resistance are introduced in this code. Engineers should be cautious with the use of the shear capacity equation listed above according to EN 1992-1-1, 2004 Cl. 6.2.5, where reinforcements are assumed to yield. It is noted that in the PIR system, steel yielding failure mode can only be achieved with longer anchorage length.

3. PIR design guides

The starter bars of cast-in reinforcement (CIR) that use to provide connections are designed in accordance with EN 1992-1-1, 2004 and ACI 318, 2014 Chapter 25 in Europe and the US, respectively. However, PIR bars are addressed differently in these major design codes. Despite the popularity of using PIR all over the world, a holistic design provision is not explicitly documented in these codes. However, based on ~~the~~ associated failure modes, relevant design philosophies such as the design of anchorage length and lap splicing length can be rationally traced.

3.1. International design standards for PIR

Theoretically, PIR design can be divided into two major categories:

(i) Anchorage

Anchorage is the use of the conventional method of reinforcement anchorage (RA) design as equivalent to CIR (EN 1992-1-1, 2004 or ACI 318, 2014 Chapter 12) or more recently the bonded anchor (BA) design (EN 1992-4, 2018 or ACI 318, 2014 Chapter 17). Detailed discussions can be referred to Charney et al., 2013, Morgan, 2015 and Mahrenholtz et al., 2014-2015.

(ii) Strut-and-tie model (STM)

STM can be designed based on the procedure in Kupfer et al., 2003, EN 1992-1-1, 2004 Cl. 6.5 or ACI 318, 2014 Chapter 23.

Extensive research carried out on static and seismic behaviour, showed that the load-slip performance of PIR installed with a qualified system can be similar or even more superior to that of CIR. Thus, the design provisions of end anchorage for CIR can be extended to PIR with qualified products. Specific guidelines are published to qualify PIR designed by using RA theory, for example, EAD 330087, 2018 and AC 308, 2014. On the contrary, documents such as EOTA EAD 330499, 2017 and AC 308, 2014 in Europe and the US, respectively, allow PIR to be designed to BA theory. Table 1(a) and Table 1(b) summarise the list of major international documents for qualification and the design of PIR with relevant documents on post-installed anchors, respectively.

Table 1(a). List of major international documents for qualification of PIR.

Qualification			
Document	Organisation	Roles and functions	Remarks
EOTA TR 023 (2006)	EOTA	Qualification of PIR in Europe under static load.	Withdrawn in 2018.
EOTA EAD 330087 (2018)	EOTA	Qualification of PIR in Europe under static load and fire exposure.	Replacing EOTA TR 023, publication expected 2019. Design according to EN 1992-1-1 and EN 1992-1-2
EOTA EAD 331522 (endorsed draft 2018)	EOTA	PIR with mortar under seismic action.	Publication expected 2019. Design to EN 1998-1.
AC 308 (2016)	ICC-ES	Qualification of PIR and adhesive anchors under static and seismic load.	With test criterion to supplement ACI 355.4. Design according to ACI 318, PIR in Chapter 18 (for seismic) and Chapter 25, and anchors in Chapter 17.
ACI 355.4	ACI	Qualification of	Design according to ACI 318, Ch.

(2011)		post-installed adhesive anchors in under static and seismic load.	17
EOTA EAD 330499 (2017)	EOTA	Qualification of post-installed anchors in Europe under static load.	Design according to EN 1992-4 or CEN/TS 1992-4-5
EOTA TR 049 (2016)	EOTA	Qualification of post-installed anchors in Europe under seismic load.	Design to EN 1992-4 or EOTA TR 045

Table 1(b). List of major international documents for design of PIR.

DESIGN			
Document	Organisation	Roles and functions	Remarks
EN 1992-1-1 (2004)	CEN	General RC design in Europe.	Design provisions for anchorage and splice length in Chapter 8.
ACI 318 (2014)	ACI	General RC design in US.	Design provisions for development length (reinforcement theory) in Chapter 25, and anchor theory in Chapter 17.
EOTA TR 029 (2007)	EOTA	Guideline for design of post-installed anchors for use in Europe.	Superseded by EN 1992-4
EOTA TR 045 (2013)	EOTA	Guideline for design of post-installed anchors for use (seismic region) in Europe.	Superseded by EN 1992-4
EN 1992-4 (2018)	CEN	Standard for design of post-installed anchor theory design in Europe.	

BS 8539 (2012)	BSI (The British Standard Institutions)	Selection and installation of post-installed anchors in UK.	Recommendations for anchors without European Technical Approvals (ETAs) qualification.

3.2. Hong Kong design guide on using PIR

In Hong Kong, the Code of Practice for Structural Use of Concrete administrated by the Buildings Department, the HKSAR Government, 2013 is used for the design, construction and quality control of RC structures. As it is mainly referenced to the withdrawn British design code BS 8110 Part 1, 1997, the requirement on anchorage bond length is also based on the yield strength of reinforcement. These clauses can be found in HKBD, 2013 Cl. 8.4 and BS 8110, 1997 Cl. 3.12.8.3. There is no specific calculation provided for lap length in HKBD, 2013, but provisions are given based on some deemed-to-comply practices, commonly as a length of the multiple of bar size. Compared to either the requirement in EN 1992-1-1, 2004 (i.e., anchorage length in Cl. 8.4.4 and splicing length in Cl. 8.7.3) or the splicing development length calculation in ACI 318, 2014 Cl. 25.4.2.3, the anchorage or splicing length in the Hong Kong design has not accounted for various coefficients such as the shapes/sizes of reinforcement, minimum cover, confinement effects, casting position and types of adhesive grout used. Hence, the HKBD, 2013 may not be directly applicable to the design of PIR. Similar challenges were encountered in using generic RC design codes such as EN 1992-1-1, 2004 and ACI 318, 2014. However, with the recent publications of EOTA EAD 330087, 2018 for static loading (repealed EOTA TR 023, 2006), EOTA EAD 331522, 2018 for seismic actions and AC 308, 2014, the use of PIR is qualified in these documents and the design of PIR are permitted as per RA theory in EN-1992-1-1, 2004 and ACI 318, 2014. Hence, a similar reform is called for in order to incorporate the latest design formulation of PIR into the HKBD, 2013 which could benefit the Hong Kong construction industry.

4. PIR Design Methodology

Different PIR design methodologies are developed in Europe and the US. Reference was made to the withdrawn BS 8110, 1997 when compiling HKBD, 2013 which is expected to be compatible with the Eurocode rather than the US. Hence, it is used as the support documents of a suitable design method for Hong Kong. The design of PIR connection requires engineers to determine the type, size, spacing, quantity, anchorage length and splice length of the reinforcement. The key parameters of the existing structure, site constraints and the arrangements of the connection that would affect the connection design are summarised in Table 2.

Table 2. Factors affecting PIR connection design.

Pre-qualification	(1) Adhesive assessment standard: RA theory: EOTA EAD 330087 or; BA theory: EOTA EAD 330499.
Base materials and reinforcement	(1) Strength grade of concrete; (2) Condition of concrete (cracked or uncracked, carbonated or non-carbonated, etc.); (3) Maximum chloride content in concrete; (4) Ultimate bond strength and design bond strength of adhesive; (5) Minimum thickness of base material.
Job site constrains	(1) The minimum and maximum concrete temperatures at time of installation and during the whole design life; (2) Access and geometrical constrains on job site;
Installation	(1) Requirements for preparation/roughening of existing concrete surface; (2) Requirements for hole drilling (hammer, core, or compressed air drill); (3) Hole diameter; (4) Orientation of connection (downward, horizontal, or overhead); (5) Environmental condition of concrete (dry, water-saturated, water filled or flooded); (6) Existing reinforcement layout and size as given in drawing and confirmed on site using detection equipment; (7) Requirements on training/certification of installers and supervisor as required.
Design requirements	(1) Design code (RA theory: EN 1992-1-1 and HKBD or BA theory: EN 1992-4; (2) Design life; (3) Load type (sustained, static, quasi-static, seismic, shock, wind); (4) Fire requirements; (5) Corrosion resistance; (6) Creep; (7) Fatigue; (8) Seismic.

4.1 Design philosophy of PIR as anchorages

PIR can be rationally designed as anchorages either by the RA theory (equivalent to CIR) or the BA theory (steel anchors), with the differences in the assumptions and limitations. For the qualification of PIR using the RA theory, the EOTA EAD 330087, 2018 is used to determine the suitability of an adhesive system. Once this adhesive system is suitably qualified, PIR can be designed using the RA theory based on EN 1992-1-1, 2004 Chapter 8. On the contrary, for the qualification of bonded anchors according to the BA theory, the EOTA EAD 330499, 2017 provides provisions to determine the suitability of mortars or adhesive for anchors. Once an adhesive system is suitably qualified, anchors can then be designed according to the newly released EN 1992-4, 2018. Table 3 provides a general comparison of both design methods. In view of the complexity of the BA theory and the fact that engineers in Hong Kong are well versed with the RA theory, the design provisions for RA theory are highlighted in this paper.

Table 3. Comparison of RA theory and BA theory as per relevant European Standards.

Main difference	RA theory	BA theory
Adhesive assessment qualification documents	EOTA EAD 330087	EOTA EAD 330499
Design standard	Chapter 8, EN 1992-1-1	EN 1992-4
Load direction	Tension	Tension, shear, combination of both
Load transfer mechanism	Equilibrium with local or global concrete struts, may require the supplement of transverse reinforcement in lapping splices.	Utilisation of tensile concrete strength
Failure modes	Tension: steel failure, pull-out, splitting (near to the edge)	Tension: steel failure, concrete breakout (cone failure), bond failure (pull-out failure), splitting (near to the edge); Shear: steel failure, concrete breakout and concrete pryout
Provision to base material	Uncracked concrete*	Cracked and uncracked concrete

Safety factor	Lower than bonded anchor theory	Higher than RA theory (to preclude steel yielding)
Basic design value of reinforcement	Yield strength	Ultimate strength for anchor and yield strength
Basic design value of bond strength	Deduced by calculation (associated with concrete tensile strength)	Tested and approved (associated with bond strength)
Design steps	a) Calculation of required sectional of reinforcement; b) Calculation of required embedment length.	a) Calculation of all characteristic capacities; b) Determination of min. capacity controlling failure anchorage.
Design results	Reinforcement length	Strength capacity
l_b Allowable embedment length	max {0.3 $l_{b,rqd}$; 10ϕ; 100 mm} $\leq l_b \leq 60\phi$	$6\phi \leq l_b \leq 20\phi$

*The equivalence in terms of pullout resistance in cracked concrete between a PIR and CIR is checked in the qualification as per EOTA EAD 330087.

Design provisions for RA theory in HKBD

The anchorage detailing provisions for CIR in HKBD, 2013 adapting the RA theory may also be used for the design of PIR. Relevant clauses are discussed as follows.

(i) Straight bar anchorage (Cl. 8.4)

Equation (1) shows the derivation of the basic anchorage length (l_b) with the assumption of force equilibrium. The derivation of Equation (1) can be proven by considering the resistance of anchorage bond (F_{bond}) is greater than the compressive or tensile force experiences in the reinforcement (F_{rebar}), yields $f_{bu}A_{s,surface} \geqslant f_{rebar}A_s$ and thus $f_{bu}(\pi\phi)l_b \geqslant f_{rebar}(\pi\phi^2/4)$, where f_{bu} is the factored bond stress capacity; $A_{s,surface}$ is the lateral surface area of reinforcement bonded with concrete; f_{rebar} is the stress and A_s is the reinforcement sectional area. According to HKBD, 2013 Cl. 8.4.5, the reinforcement is assumed fully stressed to its design yield strength ($0.87f_{yk}$) at the start of the anchorage length which incorporated with the material safety factor ($\gamma_s = 1.15$) for design as shown in Equation (2). The factored bond stress capacity (f_{bu}) according to Cl. 8.4.4, is a function of the concrete characteristic cube strength ($f_{cu,k}$). Equation (3) shows the bond stress estimation with the coefficient β which is implicitly accountable for reinforcement type and force action. For common ribbed bars, β can be 0.50 and 0.63 for tensile and compressive action, respectively. This value includes a partial safety factor for bond stress (γ_m) of 1.4.

$$l_b \geq \frac{f_{rebar}}{f_{bu}} \frac{\phi}{4}, \tag{1}$$

$$l_b \geq \frac{0.87 f_{yk}}{f_{bu}} \frac{\phi}{4}, \tag{2}$$

$$f_{bu} = \beta \sqrt{f_{cu,k}}. \tag{3}$$

(ii) Lapped splice (Cl. 8.7.3)

There are situations where PIR is used with lapped splice. Equation (4) summarises the requirement for a minimum lap length ($l_{o,min}$). There are some special requirements for splice lapping in tension, which location (top or bottom section) and concrete cover are to be considered, to decide on a factor of 1.4 or 2.0 times of the minimum lap length. For splice lapping in compression, the factor is 1.25 times of the minimum lap length.

$$l_{o,min} \geq \max\{15\phi, 300 \text{ mm}\}. \tag{4}$$

(iii) Simplified rules for simply supported beam (Cl. 9.2.1.5 and Cl. 9.2.1.7)

Reinforcement to resist at least 15% of maximum mid-span moment is to be provided as top bars for partial fixity of negative moment at support despite the assumption of simply support. 50% of the calculated mid-span bottom reinforcement is to be provided as bottom bars at the support of simply supported beams. Equation (5(a)) shows the detailing requirement of straight anchorage length for simply supported beams with effective depth d. Bend and hook are not addressed as it is irrelevant to PIR.

(iv) Simplified rules for simply supported or end supports of a continuous solid slab (Cl. 9.3.1.3)

A general detailing rule recommended for simply supported solid slabs stipulated in HKBD, 2013 is to provide 50% of the maximum mid-span moment and 50% of the calculated maximum span reinforcement, for the top and bottom bars, respectively. The reinforcement is to be anchored into the support conforming to Equations (2) and (5(b)). If the design ultimate shear stress at the face of support is less than half of the appropriate value of concrete shear stress capacity (v_c), HKBD, 2013 Cl. 6.1.2.5 recommends a straight length of bar beyond the centreline of the support equal to either one-third of support width or 30 mm, whichever greater may be considered as the effective anchorage. An effective full tensile anchorage is assumed by providing the following simplified detailing rules where l_{span} is the slab span length:

$$l_{beam,sr} = \max\{12\,\phi \text{ after support centreline, } 12\,\phi + d/2 \text{ from support face}\}. \quad (5(a))$$

$$l_{slab,sr} = \max\{0.15 l_{span}, 45\,\phi\}. \quad (5(b))$$

Design provisions for RA theory in EN 1992-1-1

This section reviews the anchorage detailing provisions for CIR in EN 1992-1-1, 2004 as per European Technical Assessment (ETA) which uses the RA theory for the design of PIR.

(i) Straight bar anchorage (Cl. 8.4)

EN 1992-1-1, 2004 uses the design stress (σ_{sd}) rather than the characteristic yield stress (f_{yk}) with material safety factor (γ_s). In fact, the assumption of fully stressed to its yield strength is rarely the case, as good detailing principles put lapped splice at low-stress location and the provided area of steel is greater than the required area (Concrete Design Guide No. 5 (CDG-5, 2015). Design stress (σ_{sd}) can be rationally determined using the steel area ratio of required ($A_{s,rqd}$) to provided ($A_{s,prov}$), multiplied by the design yield strength (i.e., $A_{s,rqd}/A_{s,prov} \cdot f_{yk}/\gamma_s$). Engineers should be cautioned that a shorter anchorage length may induce other failure mechanisms associated with anchors, i.e. concrete cone (breakout) or bond (pullout). The design bond stress (f_{bd}) according to Cl. 8.4.2 (2), is a function of concrete design tensile strength (f_{ctd}) according to Cl. 3.1.6 (2). Equation (6) shows the bond stress estimation with coefficients η_1 and η_2 that are implicitly accountable for bond condition, position and diameter of reinforcement. To be consistent with HKBD, 2013, Equation (6(b)) shows the factored bond stress capacity (f_{bu}) with the inclusion of material partial safety factor (γ_m).

$$f_{bd} = 2.25\ \eta_1\ \eta_2 f_{ctd}, \quad (6(a))$$

$$f_{bu} = 2.25\ \eta_1\ \eta_2 f_{ctk,0.05} / \gamma_{m}. \quad (6(b))$$

Where η_1 is t coefficient for bond condition and related to reinforcement position during concreting (1.0 for good and 0.7 for others); η_2 is a coefficient for influence of diameter (1.0 for $\phi \leq 32$ mm and $(132 - \phi)/100$ for $\phi > 32$ mm); f_{ctd} is taken as the characteristic tensile strength at 5% fractile ($f_{ctk,0.05}$) with consideration of a partial safety factor $\gamma_m = 1.5$ for concrete. Hence, the basic derivation in Equation (1) is analogical but the stress experienced by reinforcement is the design stress rather than the characteristic yield stress. Equation (7) shows the basic required anchorage length ($l_{b,rqd}$). Interestingly, EN 1992-1-1, 2004 introduced further checking procedure on the design anchorage length (l_{bd}) and imposed a minimum anchorage length ($l_{b,min}$) which are not required in HKBD, 2013. Equations (8) and (12) are the expressions for the design and minimum anchorage length, respectively.

$$l_{b,rqd} \geq \frac{\sigma_{sd}}{f_{bu}} \frac{\phi}{4}, \tag{7}$$

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_5 l_{b,rqd} \geq l_{\min}. \tag{8}$$

Where α_1 is a coefficient for the effect of the form of reinforcement, assuming adequate cover (for straight bars, α_1 is 1.0) and α_2 is a coefficient for the effect of concrete minimum cover to consider splitting failure and is stated in Equations (9(a)) and (9(b)) for straight bars.

$$0.7 \leqslant \alpha_2 = 1 - \frac{0.15(c_d - \phi)}{\phi} \leqslant 1.0 \text{ (Tension)}, \tag{9(a)}$$

$$\alpha_2 = 1.0 \text{ (Compression)}. \tag{9(b)}$$

Where $c_d = \min\{s/2, c_1, c\}$ for straight bars, s is the clear spacing of bars, c_1 is the side cover and c is the top or bottom cover. Although the other coefficients present challenges to achieve for PIR system, they are nonetheless included for discussions. Coefficient α_3 in Equation (10) accounts for the effect of the confinement by transverse reinforcement and coefficient α_5 in Equation (11) is the effect of the pressure transverse to the plane of splitting along the design anchorage length.

$$0.7 \leq \alpha_3 = 1 - K\lambda \leq 1.0 \text{ (Tension)}, \tag{10(a)}$$

$$\alpha_3 = 1.0 \text{ (Compression).} \tag{10(b)}$$

Where K is defined in Figure 3 and λ is the ratio of excess transverse reinforcement area to longitudinal reinforcement area, $(\Sigma A_{st} - \Sigma A_{st,min})/ A_s$.

$$0.7 \leqslant \alpha_5 = 1 - 0.04p \leqslant 1.0 \text{ (Tension).} \tag{11}$$

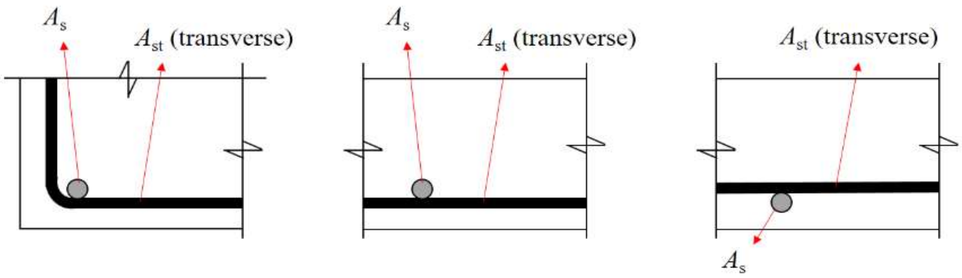

Figure 3. Values of K for beams and slab in EN 1992-1-1.

Where p is the transverse pressure (in MPa) at the ultimate limit state along l_{bd}. As this paper concerns only with PIR, unrelated coefficient α_1 and α_4 are excluded. The minimum anchorage length can be calculated by using the Equations (12(a)) and (12(b)).

$$l_{b,min} \geqslant \max\{0.3l_{b,rqd}, 10\phi, 100 \text{ mm}\} \text{ (Tension),} \tag{12(a)}$$

$$l_{b,min} \geqslant \max\{0.6l_{b,rqd}, 10\phi, 100 \text{ mm}\} \text{ (Compression).} \tag{12(b)}$$

It should be noted that the minimum anchorage length (l_{min}) shall be multiplied by an amplification factor (α_{lb}) to account for the difference of CIR and PIR in cracked concrete. In general, if there is no test carried out to PIR in cracked concrete in accordance with EOTA EAD 330087, 2018, α_{lb} is taken as 1.5.

(ii) Lapped splice (Cl. 8.7.3)

The designed lap length for EN 1992-1-1, 2004 is shown in Equation (13) with α_1, α_2, α_3 and α_5 are previously defined (Equations (9) to (11)). α_6 is a coefficient of the percentage of the lapped bar (p_1) relative to the total sectional area within $0.65l_o$ from the centre of lap length (Equation (14)). The minimum lap length can be calculated by using Equation (15). Similar to the minimum anchorage length for PIR, the minimum lap length ($l_{o,min}$) shall be multiplied by an amplification factor (α_{lb}) to account for the difference of CIR and PIR in cracked concrete. In general, if there is no test carried out to PIR in cracked concrete in accordance with EOTA EAD 330087, 2018, α_{lb} is taken as 1.5.

$$l_o = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b,rqd} \geqslant l_{o,min}, \tag{13}$$

$$1.0 \leqslant \alpha_6 = (\rho_1/25)^{0.5} \leqslant 1.5\,, \tag{14}$$

$$l_{o,min} \geqslant \max\left\{0.3\,\alpha_6 l_{b,rqd},\ 15\phi,\ 200\text{ mm}\right\}. \tag{15}$$

(iii) Simplified rules for simply supported beams (Cl. 9.2.1.4)

Value of 15% of maximum bending moment in the span and 25% (National Annex dependent, in contrast, it is 50% in HKBD, 2013) of the steel area provided in the span is recommended for the top and bottom bars, respectively, at the support of simply supported beams. Both top and bottom bars are to be anchored with l_{bd} measured from the face of support. It is interesting to note that Cl. 9.2.1.4(2) allows a STM equivalent model to calculate the axial forces in reinforcement, which appears to be more suitable for the design stress (σ_{sd}) estimation in Equation (7).

(iv) Simplified rules for simply supported solid slabs (Cl. 9.3.1.2)

In simply supported slabs, 15% - 25% of maximum bending moment in the span and 50% of the calculated span reinforcement should be provided for the top and bottom bars at the support of solid slabs, respectively (as opposed to the 50% provision in HKBD, 2013). Both top and bottom bars are anchored with l_{bd} measured from the face of support. Similar to the simply supported beams, Cl. 9.2.1.4(2) of STM is allowed.

Comparisons of bond strength according to the RA theory are shown in Table 4. The case is assumed for a ribbed bar with $\phi \leq 32$ where the reinforcement is at a good position during concreting. The material safety factors for bond stress (1.4 for HKBD, 2013 and 1.5 for ETA) are excluded. In general, the HKBD, 2013 presents more conservative values (except for lower strength concrete with 20 MPa cube strength with $\alpha_2 = 1.0$) compared to the EN 1992-1-1, 2004 as per ETA.

Table 4. Summary of bond strength in accordance with RA theory in HKBD and EN 1992-1-1 as per ETA (exclude material safety factor for bond stress, i.e., 1.4 for HKBD and 1.5 for EN 1992-1-1).

Concrete characteristic cube strength, $f_{cu,k}$ (MPa)	Concrete characteristic tensile strength at 5% fractile, $f_{ctk,0.05}$ (MPa)	Bond strength (Tension)				Bond strength (Compression)	
		HKBD $\beta = 0.5$	ETA normalised by $\alpha_2 = 0.7$*	ETA normalised by $\alpha_2 = 0.85$*	ETA normalised by $\alpha_2 = 1.0$*	HKBD $\beta = 0.63$	ETA normalised by $\alpha_2 = 1.0$*
20	1.3	3.1	4.2	3.4	2.9	3.9	2.9
30	1.8	3.8	5.8	4.8	4.1	4.8	4.1
40	2.1	4.4	6.7	5.5	4.7	5.6	4.7
50	2.5	4.9	8.0	6.6	5.6	6.2	5.6
60	2.9	5.4	9.3	7.7	6.5	6.8	6.5

* α_2 is a coefficient for the effect of concrete min. cover to consider splitting failure and is stated in Equations (9(a)) and (9(b)) for straight bars. In this example, the limit boundary was taken as 0.7 and 1.0, where the case of $\alpha_2 = 1.0$ is more susceptible to splitting failure due to insufficient edge cover. It should be noted that the case of $\alpha_2 = 1.0$ corresponds to a concrete cover of 1ϕ, which present challenges in hole drilling. The minimum concrete cover to account for possible deviation in drilling is found in Table 5, with a minimum concrete cover of 2ϕ, corresponds to $\alpha_2 = 0.85$.

Table 5. Minimum concrete cover (c_{min}) proposed in EOTA EAD 330087.

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

Yes	Hammer or diamond	< 25 mm	30 mm + 0.02 $l_v \geq 2\phi$
		≥ 25 mm	40 mm + 0.02 $l_v \geq 2\phi$
	Compressed air	< 25 mm	50 mm + 0.02 l_v
		≥ 25 mm	60 mm + 0.02 $l_v \geq 2\phi$

Where l_v is setting anchorage depth of reinforcement (in unit mm).

4.2 Design philosophy of PIR as STM (state-of-the-art moment connection)

For moment connections designed with CIR, they require bent bars rather than straight bars that are not achievable on PIR. As a result, the compressive strut is anchored in the bonding area of straight reinforcement rather than in the bend area. If the anchorage depth is approaching 15ϕ, an STM is more suitable (Lee et. al., 2019). According to the BA theory, the concrete breakout will form at a horizontal angle of 30°. However, among the moment connection cases, due to the beneficial effect offered by the compressive strut of STM, the forming of a cone crack will be hindered at a steeper angle. In lieu of propagation of cone cracking, one of the STM failure modes may occur e.g. compressive strut failure or splitting failure of concrete.

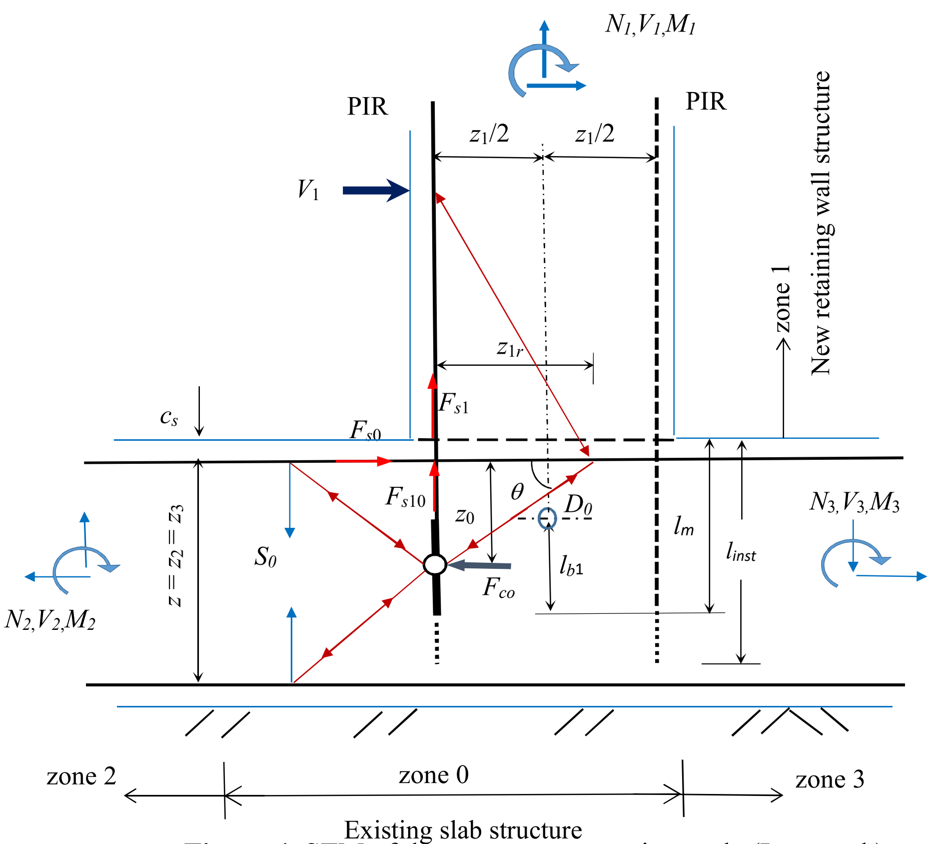

Figure 4. STM of the moment connecting node (Lee et. al.).

Furthering the pioneer STM work by Schlaich et al., 1987, a detailed STM used on PIR was proposed by Kuper, 2003 and Muenger et al., 2002, and validated by Hamad et al., 2006. STM complies to the RC theory and DIN 1045-1, 2008 in that the tensile forces cannot be transferred directly to the concrete. Take a retaining wall structure as an example, the STM analysis can be divided into four zones (Figure 4). Zone 1 is the newly cast slab while the other slabs are on an existing wall. The connection of the PIR node is in Zone 0 which is between Zones 2 and 3. The state-of-the-art moment connection by STM design procedure is given as below:

(1) Formulate the force equilibrium at node;
(2) Check anchorage length;
(3) Check tension in existing reinforcement;
(4) Check concrete compressive strut;
(5) Check splitting force in transition zone.

Together with the horizontal angle θ and the required bond length subject to adhesive strength, the anchorage length is obtained. By maintaining the force equilibrium at the anchorage node, the STM theory and equations are formulated, (Lee et. al., 2019).

(i) Anchorage check

From the cracked pattern observation of moment connections, the slab effective inner lever arm (z_{1r}) of the PIR may be reduced by a factor of k to the distance between the top and bottom reinforcement of slab (z_1) (Equation (16)). In the closing moment case, k is taken as 1.0. In opening moment case, k is ranging from 0.85 to 1.0 (Kupfer, 2003). Alternatively, z_{1r} can also be obtained by the typical flexural analysis from the effective depth d (HKBD, 2013 and EN 1992-1-1, 2004). The PIR tensile force F_{s1} is obtained from applied moment M_1, as shown in Equation (17).

$$z_{1r} = z_1 \cdot k\,, \tag{16}$$

$$F_{s1} = M_1 / z_{1r}. \tag{17}$$

For $F_{s10} = F_{s1}$, with bar perimeter (u), the anchorage bond length is:

$$l_{b1} = \frac{F_{s10}}{f_{bm} \cdot \Sigma u}. \tag{18}$$

Where the f_{bm} is taken as mean bond strength of the used adhesive system. This is justified only when there is sufficient large spacing of the PIR in the considered tests. In the case of closer spacing and/or small cover, splitting might become decisive and a reduced value of f_{bm} should be used for design purposes. In the node, the lever arm of internal forces z_0 is obtained from the subtraction of the installed anchorage depth (l_{inst}) by the concrete cover (c_s) and half of the anchorage bond length (l_{b1}) (Equation (19(a))). However, if l_{inst} is much longer than required, a more realistic z_0 shall be calculated according to Equation (19(b)) from the compressive strut inclination angle θ:

$$z_0 = l_{inst} - c_s - l_{b1}/2, \tag{19(a)}$$

$$z0 = z1r. \tan \theta, \tag{19(b)}$$

$$30^o < \theta < 63^o. \tag{20}$$

However, this angle is limited by Equation (20). The position of the strut failure crack is t (= $c_s + z_0$). Re-arranging the basic anchorage length (l_b) becomes:

$$l_b = t + l_{b1}/2 \tag{21}$$

(ii) Wall Near Face reinforcement check

The near face (NF) reinforcement force (F_{s0}) at the node is the sum of the moment force outside the node area with the internal forces lever arm (z_0) and reinforcement lever arm (z). The reinforcement area required is $A_{s0,rqd} = F_{s0}/(f_{yk}/\gamma_s)$.

$$F_{s0} = M_1 \cdot (1/z_0 - 1/z). \tag{22}$$

(iii) Wall Far Face reinforcement check

Similarly, in closing moment case, the far face (FF) reinforcement tensile force F_{s3} is given in Equation (23(a)). In fact, this formulation is the same as the bending check in the conventional RC design. In the open moment case, the reinforcement tensile force (F_{s3}) is obtained from the sum of tension in existing reinforcement outside node area (F_{s2}) and the tension in node due to reduced lever arm (H_{s2}) as shown in Equations (23(b)) to (25). The reinforcement area required is $A_{s3,rqd}$ = $F_{s3}/(f_{yk}/\gamma_s)$.

$$F_{s3} = M_1/z, \tag{23(a)}$$

$$F_{s3} = F_{s2} + H_{s2}, \tag{23(b)}$$

$$F_{s2} = M_1/z + N_2/2, \tag{24}$$

$$H_{s2} = \left(M_1 + (V_2+V_3)\frac{z_1}{2}\right).\left(\frac{1}{z_0} - \frac{1}{z_2}\right) + V_1\left(\frac{z_{1R}}{z_0} - 1\right). \tag{25}$$

(iv) Compressive strut check

From Figure 4, in the nodal zone, the horizontal force (F_{c0}) is acting in the centre of effective anchorage length (l_b). At the nodal zone, the compressive strut (D_0) is anchored at the centre of effective anchorage length (l_b) and is balanced by the nodal horizontal force (F_{c0}) (Equation (26)). According to EN 1992-1-1, 2004 section 6.5, in the compression-tension node, the maximum strength for a concrete strut is given in Equation (27). Based on Equations 7.3-82 (1992-1-1, 2004), a strut efficiency factor α = $(0.75 \cdot \eta_{fc})$ must be used with $\eta_{fc} = (30/f_{ck})^{1/3} \leq 1$ to reduce concrete strength in the nodal zone. This is a hyperbolic rather than a linear reduction as per EN 1992-1-1, 2004 ($\alpha = k_2 \cdot v'$ with $k_2 = 0.85$ and $v' = 1\text{-}f_{ck}/250$)). Finally, the maximum concrete strut resistance (D_R) must be larger than the internal compressive strut force i.e. $D_R \geq D_0$ as given in Equation (28) where b is structural width and the strut width is $l_{b1} \cdot cos\ \theta$.

$$F_{c0} = \frac{M_1 + (V_2+V_3).\ z/2}{z_0}, \tag{26}$$

$$D_R = \alpha_{cc} \cdot \alpha \cdot f_{cd}, \tag{27}$$

$$D_0 = F_{c0}/cos\ \theta, \tag{28(a)}$$

$$D_R = \alpha_{cc} \cdot 0.75 \cdot (30/f_{ck})^{1/3} \cdot f_{ck}/\Upsilon_c \cdot (b \cdot l_{b1} \cdot cos\ \theta). \tag{28(b)}$$

(v) Splitting tensile stress in discontinuity zone

In the B-region, the vertical components of struts are taken up by tensile stress within concrete causing splitting failure. The maximum splitting moment (M_{sp}) is determined by the bursting transverse stress in anchorage zone. Having obtained the section modulus (W_{sp}), the splitting stress σ_{sp} (= M_{sp}/W_{sp}) is given by Equation (29). It checks against the concrete characteristic tensile strength at 5% fractile ($f_{ctk,0.05}$) by Equation (30) which can be determined by the experiment or the concrete compressive strength (f_{ck}) indirectly (EN 1992-1-1, 2004 3.1.6(2)).

$$\sigma_{sp} = F_{c0}.\ z_0.\left(1 - \frac{z_0}{z}\right).\left(1 - \frac{l_{b1}}{2z}\right) / \frac{b.\ z^2}{2.42}\ , \tag{29}$$

$$\sigma_{sp} \leq f_{\text{ctk},0.05}. \tag{30}$$

4.2.1. Design provisions for STM in HKBD

Similar to the RA theory in the HKBD, 2013, equations governing the RA design is also applied to STM. All the forces shall be factored to form designed forces and material safety factor shall be incorporated. The factored bond stress capacity (f_{bu}) obtained from concrete characteristic cube strength as given in Equation (3) is still valid and replaces the use of adhesive mean bond strength (f_{bm}).

(i) Straight bar anchorage

When determining the anchorage bond length (l_{b1}), the equation shall be modified according to Equation (31). This must be shorter than the design anchorage length (l_b) as obtained from the RA theory in Equation (2) with strut inclination horizontal angle being limited by Equation (20).

$$l_{b1} = \frac{F_{s10,d}}{f_{bu} \cdot \Sigma u} \tag{31}$$

For the reinforcement check, the design yield strength by using material safety factor (γ_s) 1.15 needs to be introduced to Equations (22) and (23). For the concrete strut resistance (D_R), the concrete material safety factor (γ_c) 1.5 shall be used. The parameters for α_{cc} and k_2 may be taken as 1.0 and 0.85, respectively as referring to EN 1992-1-1, 2004, 3.1.6(1) and 6.5.2(2).

(ii) Lapped splice

For lapped splice, the requirements for a minimum lap length (l_o), Equation (4) and the relevant factors for tension and compression cases are still valid.

4.2.2. Design Provisions for STM in EN 1992-1-1 as per EOTA

For STM design, the calculation of anchorage bond length (l_{b1}) can be obtained from the RA theory in accordance with EN 1992-1-1, 2004 section 8.4.4. Hence, the design bond stress (f_{bd}) and the factored bond stress capacity (f_{bu}) as given by Equation (6) are still valid in determining l_{b1}. The basic required anchorage length ($l_{b,rqd}$) is the maximum of the basic anchorage length (l_b) from Equation (21) and 15ϕ

as proposed by Lee et al., 2019. Finally, the design anchorage length (l_{bd}) can be taken as $l_{b,rqd}$ but subject to the minimum of the anchorage length ($l_{b,min}$) (Equations (7) to (12)). Of course, it must also be checked against the maximum installed hole length ($l_{inst,max}$) from the drilled hole diameter (ϕ_o) as per Equation (32). If the installed anchorage length (l_{inst}) is longer than required, the design anchorage length (l_{bd}) will then be shorter and the STM equilibrium will no longer be formed near to the tip region of PIR. A simplified approach may be adopted by assuming the horizontal strut angle θ at 60° (Lee el. al. 2019). Hence, the basic anchorage length becomes the same as stated in Equation (33). Standard reinforcement and compressive strut check for STM are applied. For the checking of splitting tensile stress (σ_{sp}), the concrete characteristic tensile strength ($f_{\mathrm{ctk},0.05}$) can be obtained from EN 1992-1-1, 2004 Table 3.1 or by Equation (34) with α_{ct} taken as 1.0 from EN 1992-1-1, 2004 3.1.6(2) and concrete material safety factor (γ_c) is 1.5.

$$l_{inst,max} \geqslant h\text{-}max\{2\phi_o,\ 30\ \mathrm{mm}\}, \tag{32}$$

$$l_b = c_s - z_{1R}\ .\ tan\ 60^{\circ} + l_{b1}/2\ , \tag{33}$$

$$\sigma_{sp} \leq f_{\mathrm{ctk},0.05} = \alpha_{ct} \cdot 0.7 \cdot (0.3 \cdot f_{ck}^{\ 2/3}/\gamma_c). \tag{34}$$

5. Summary, conclusion and outlook

As technology advances, the design of PIR has to be updated from time to time. This paper reviewed the latest requirements of various international documents on the temperature effect of adhesive materials, strength of RC base material, roughening of concrete surface and qualification of installers. Relevant provisions in the HKBD, 2013 have also been highlighted and referenced to the recent changes made in international standards (the European and American codes) specifically based on the RA theory. In view of the familiarity of Hong Kong engineers in using the HKBD, it is recommended that engineers may choose either the modified RA theory (from EN 1992-1-1, 2004) or a state-of-the-art STM theory for the use of PIR to meet the design assumptions and site constraints.

It should be noted that the performance of PIR system is highly affected by the product qualification, for example, qualification as per EOTA EAD 330087, 2018. Hence, a similar reform is called for to incorporate the latest design formulation of PIR into the HKBD, 2013 which will benefit the Hong Kong construction industry.

Detailed installation guide, design methodology and examples discussed in this paper will be recorded in a guidebook entitled "Guide for design, installation and assessment of post-installed reinforcement" which will be published by the Hong Kong University Press. Besides the requirement of qualification of PIR, engineers are recommended to refer to the Guide for details of the installation assessment under the qualified site supervision system in Hong Kong. This proposal is useful for the engineers to promote economical, sustainable and technically sound use of the PIR system especially to the hot topic of "modular integrated construction".

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