
An Overview of Post-Installed Rebars in Moment Resisting RC Connections from International Research: Performance, Design Implications and Applicability in NZ

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ABSTRACT

Post-installed rebar (PIR) systems are increasingly used for the seismic retrofit and extension of reinforced concrete (RC) structures. Despite their widespread deployment, questions remain around their seismic performance relative to cast-in reinforcement, and the conditions required to ensure ductile and reliable behaviour under cyclic loading. This paper synthesises evidence on the performance of PIR in moment-resisting RC connections, focusing on its anchorage behaviour and failure mechanisms. Findings indicate that PIR consistently provides higher bond strength than straight cast-in-place bars, potentially allowing for a reduction in required anchorage length. The governing failure mode of PIR is shown to be sensitive to detailing conditions, with potential shifts between bond, splitting, and steel failure. Furthermore, the importance of adequate embedment, confinement and sufficient edge distance is emphasized to ensure ductile capacity, particularly during seismic loads and crack cycling. In addition, the paper examines the potential application of PIR within New Zealand for seismic retrofit and structural modification, with reference to prevailing regulatory frameworks and design philosophies, emphasising the need for methodologies such as EOTA TR069 to preserve ductile behaviour and alignment with NZS 3101 objectives.

1 INTRODUCTION

Post-installed rebars (PIR) have become a critical solution for connecting new concrete to existing structures, particularly in seismic retrofitting, repair, and structural extensions where traditional cast-in methods are not feasible.

Despite their growing importance, NZS 3101 currently does not provide explicit guidance on PIR applications, leaving designers to rely on international standards. AS 5216:2021 was among the first standards to formally address PIR, setting a precedent in the Australasian region. Globally, the fib Model Code 2010 has served as a key reference, incorporating provisions from EN 1992-1-1 for designing post-

installed rebars as if they were cast-in. However, European standards are undergoing revision, and it is anticipated that PIR will be explicitly included in future updates.

The design approach of treating PIR as cast-in reinforcement has been widely adopted for simply supported connections, but the fib Model Code 2010 does not endorse this method for moment-resisting connections. Historically, the design and prequalification of bonding agents for concrete-to-concrete (C2C) rigid connections were limited to static load conditions, which posed challenges for applications in seismic regions. Recently, research has shifted toward understanding their performance under cyclic and seismic loading, particularly in moment-resisting reinforced concrete (RC) connections, where bond reliability and ductility are critical. These were followed by the development of new design methods, such as the EOTA TR069.

This paper synthesizes international findings, with a particular focus on TR069, and examines the applicability of these systems within the New Zealand context, considering local seismic demands and regulatory gaps. The discussion aims to bridge the current knowledge gap and provide insights into future design practices for PIR in structural applications, particularly in the future revisions of NZS3101.

2 PIR PERFORMANCE VS CAST-IN REINFORCEMENT

Post-installed rebars (PIR) and cast-in reinforcement share a fundamental principle: both rely on bond strength to transfer loads along the intended load path. The core difference between the bond mechanisms of cast-in (CIR) and post-installed (PIR) rebar lies in how the tensile load is transferred to the surrounding concrete: cast-in rebar relies on mechanical interlock of ribs with concrete (Fig 1a), while post-installed rebar relies on adhesive micro-interlocking with the concrete surface, as well as mechanical interlock and potential adhesion between the bar and the chemical (Fig 1b). Modern, high-performance adhesive mortars used for post-installed rebar can actually achieve higher bond strengths than conventional concrete, allowing for significantly reduced development lengths.



Figure 1: Working principles of Cast-in vs Post-installed reinforcing bars. Source: Hilti Corporation.

From a design and execution perspective, cast-in reinforcement can incorporate hooks or other anchorage details to reduce embedment depth, while PIR exclusively relies on straight anchorage, which may require greater embedment lengths if designed using cast-in assumptions. Cast-in bars are placed within formwork before concrete pouring, creating a monolithic bond during curing, whereas post-installed rebars are inserted into hardened concrete through drilled holes filled with injection mortars, forming a combined mechanical and adhesive bond. This approach is essential for retrofitting applications where new elements must be anchored to existing structures.

Installing PIR inherently require additional equipment such as concrete scanners, rotary hammers, and thorough hole-cleaning tools. Surface preparation is critical, as roughening the concrete interface enhances shear transfer through friction. PIR systems also demand strict compliance with adhesive curing protocols and rigorous quality control measures, including, in some cases, on-site proof load testing.

While NZS 3101 provides detailed guidance for the design of cast-in reinforcement, it does not currently include explicit provisions for post-installed systems. As a result, compliance in New Zealand often relies on

international standards such as EADs (European Assessment Documents) and prequalified systems with European Technical Assessments (ETAs). When equivalency to cast-in performance is demonstrated, PIR can be designed using NZS 3101 development and lap length provisions, ensuring safe and effective integration in retrofit scenarios.

Moment-resisting connections introduce additional complexity for post-installed systems. Historically, design and qualification of bonding agents in this type of connection focused on static loads, limiting their application in seismic regions. Recent advancements, however, have addressed this gap. EOTA TR069 now represents the state-of-the-art for verifying post-installed rebars under cyclic and seismic loading, setting performance criteria for adhesive systems in moment-resisting concrete-to-concrete connections. This development marks a significant step toward enabling PIR solutions in structural applications where ductility and energy dissipation are critical, aligning with modern seismic design requirements.

2.1 Bond Strength and Anchorage Behaviour

The design of PIR as cast-in usually disregards the full potential of the bonding agent and limits the bond strength to the concrete bond strength. In Eurocode 2 (EN 1992-1-1), referenced by fib Model Code 2010, the design bond strength f_{bd} for cast-in ribbed reinforcing bars is defined as:

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \quad (1)$$

where η_1 = bond condition factor (1.0 for good, 0.7 otherwise); η_2 = bar diameter factor (1.0 for $\phi \leq 32$ mm; $(132 - \phi)/100$ for $\phi > 32$ mm); and f_{ctd} = concrete design tensile strength.

Rewriting f_{ctd} as a function of f_{ck} and simplifying the equation, it is possible to define f_{bd} as below, assuming good bond and rebar diameter less than 32mm:

$$f_{bd} = 0.315 \cdot f_{ck}^{2/3} \quad (2)$$

where f_{ck} = characteristic compressive strength of concrete (MPa).

This formulation links bond strength solely to concrete properties, which may be appropriate for cast-in bars but fails to account for the potentially enhanced performance of high-strength bonding agents used in PIR - some of which exhibit effective bond strengths exceeding those of concrete with $f_{ck} > 100$ MPa.

To overcome this limitation, a “beyond cast-in” design methodology has been introduced for PIR. This approach involves product specific testing and qualification of bond performance under bond-splitting failure modes, as defined in EAD 332402-00-0601.

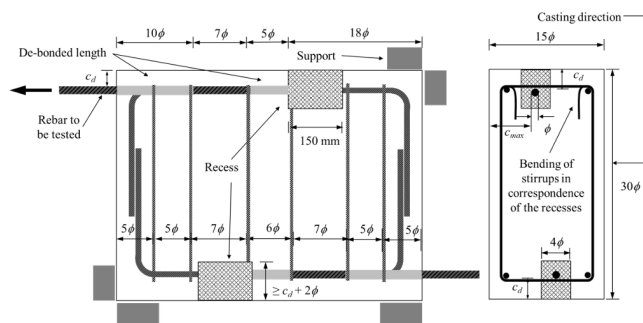


Figure 2: Typical beam-end test specimen used in EAD 332402-00-0601. Source: EOTA (2018).

In EAD 332402-00-0601, the performance of PIR is established through a structured programme of monotonic and cyclic tension tests designed to characterise both ultimate resistance and degradation under repeated loading. Under static conditions, specimens are subjected to displacement-controlled monotonic pull-out tests in accordance with the Beam-End Test (BET) configuration (Fig 2), in which load and slip are recorded continuously until failure. Peak loads typically occur within 1–3 minutes, and where peak slip exceeds the allowable limit, the failure load is

taken at a prescribed displacement (e.g., $\delta_1 = 1.5$ mm for $\phi \leq 25$ mm, increasing to 3.0 mm for large-diameter bars). The resulting failure loads (N_u) are normalised to nominal concrete strength and converted to

characteristic bond resistance; additional metrics such as displacement at 50% of N_u and coefficients of variation are used to check ductility and statistical reliability.

Seismic qualification is performed through cyclic displacement-controlled testing using the same specimen geometry, following a prescribed sequence of three half-cycles at each displacement level (e.g., 0.1, 0.25, 0.5, 0.75, 1.0, 1.25, 1.5, 2.0, 2.5, and 3.0 mm), up to a maximum amplitude defined by the corresponding monotonic test ($s_{max} \approx 3$ mm for typical systems). Peak cyclic loads, residual loads after the final cycle, and the envelope-based hysteretic energy are calculated to quantify cyclic degradation (Fig 3). Reduction factors (e.g., $\alpha_{eq,sp}$, typically between 0.4–1.0; and crack-width-dependent Ω -factors for 0.5 mm and 0.8 mm cracking) are derived by comparing cyclic peak and residual strengths to monotonic reference values. Together, these static and cyclic procedures provide a rigorous, quantitative basis for determining both ultimate and seismic bond performance, enabling TR069-compliant design checks that reflect realistic behaviour of PIR systems under seismic demands.

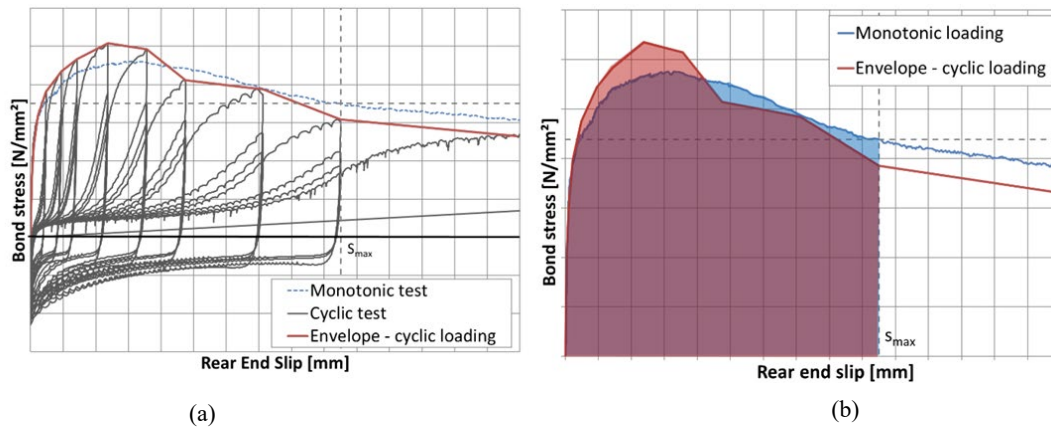


Figure 3: Load-displacement curves. a) monotonic curve and envelope derived from cyclic; b) area under monotonic curve and under the envelope curve. Source: EOTA (2018).

Bond strengths are derived from a systematic series of Beam-End Tests in which PIR are loaded monotonically and cyclically to failure, and the measured peak loads N_u are normalised to nominal concrete strength and converted into bond stress using the generic formula below, while more specific bond strengths may be adopted for different failure modes. The tests vary concrete cover, side cover, rebar diameter, embedment length, crack widths and concrete strength, enabling regression of the model parameters in the TR069 bond-splitting equations.

$$\tau_u = \frac{N_u}{\pi \phi l_b} \quad (3)$$

where N_u is the failure load; ϕ is the rebar diameter; and l_b is the embedment depth.

2.2 Anchorage Failure Modes

While traditional cast-in reinforced concrete design does not explicitly verify bond or concrete failure modes for each individual rebar - relying instead on conservative embedment and detailing rules tied to concrete strength - EOTA TR 069 introduces a more explicit, performance-based methodology for post-installed reinforcement. This approach combines principles from conventional reinforced concrete design with anchor design concepts and requires that all three governing failure modes - steel yielding (Fig 4a), bond-splitting failure (Fig 4b), and concrete cone breakout (Fig 4c) - be checked individually in a hierarchy-of-strength framework, with the lowest resistance dictating the design.

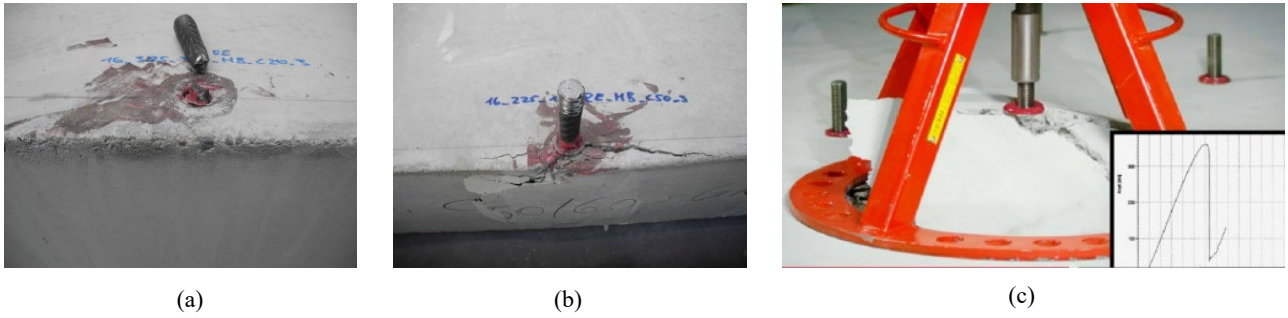


Figure 4: Anchorage failure modes from EOTA TR069. a) steel failure; b) bond-splitting failure; c) concrete cone breakout. Source: Hilti Corporation.

The failure modes and summary of calculations for static conditions are described below.

Steel yielding, a desirable ductile mode, is determined by bar yield strength and cross-sectional area.

$$N_{Rk,y} = A_s \cdot f_{yk} \quad (4)$$

where A_s is the cross-sectional area of all tension post-installed rebars within the connection.

Bond-splitting failure, which may occur through adhesive bond loss or longitudinal splitting of the concrete, is governed by embedment length, adhesive/concrete interface quality, bar diameter, and critically, concrete cover and confinement conditions.

$$N_{Rk,sp} = \tau_{Rk,sp} \cdot l_b \cdot \phi \cdot \pi \quad (5)$$

$$\tau_{Rk,sp} = \eta_1 \cdot A_k \cdot \left(\frac{f_{ck}}{25}\right)^{sp1} \cdot \left(\frac{25}{\phi}\right)^{sp2} \cdot \left[\left(\frac{C_d}{\phi}\right)^{sp3} \cdot \left(\frac{C_{max}}{C_d}\right)^{sp4} + k_m \cdot K_{tr}\right] \left(\frac{7\phi}{l_b}\right)^{lb1} \cdot \Omega_{p,tr} \quad (6.1)$$

$$\leq \tau_{Rk,ucr} \cdot \Omega_{cr,03} (or \Omega_{p,tr}) \cdot \psi_{sus} \quad \text{for } 7\phi \leq l_b \leq 20\phi \quad (6.2)$$

$$\leq \tau_{Rk,ucr} \cdot \left(\frac{20\phi}{l_b}\right)^{lb1} \cdot \Omega_{cr,03} (or \Omega_{p,tr}) \cdot \psi_{sus} \quad \text{for } l_b > 20\phi \quad (6.3)$$

where $\tau_{Rk,sp}$ is the characteristic bond splitting resistance; l_b is the embedment length of the PIR; ϕ is the nominal diameter of the rebar; A_k is the characteristic fitting factor (available in the ETA); $sp1, sp2, sp3, sp4$ and $lb1$ are the curve fitting exponents (taken from ETA); $C_d = \min$ (clear concrete cover, half of clear rebar spacing); $C_{max} = \max$ (clear concrete cover, half of clear rebar spacing); $\Omega_{p,tr}$ = transverse pressure in concrete factor; $\tau_{Rk,ucr}$ is the characteristic bond resistance in uncracked concrete (50y design life); $\Omega_{cr,03}$ is the cracked concrete (0.3mm) factor (from ETA); ψ_{sus} is the sustained load factor.

Concrete cone breakout, analogous to anchor bolt pullout, may govern when embedment depth or confinement is limited.

$$N_{Rk,c} = N_{Rk,c}^0 \cdot \frac{A_{C,N}}{A_{C,N}^0} \cdot \Psi_{s,N} \cdot \Psi_{eC,N} \cdot \Psi_{re,N} \cdot \Psi_{M,N} \quad (7)$$

$$N_{Rk,c}^0 = k_1 \cdot \sqrt{f_{ck}} \cdot l_b^{1.5} \quad (8)$$

where $N_{Rk,c}^0$ is the characteristic concrete cone break-out resistance for a single PIR not influenced by geometric conditions; $A_{C,N}$ is the actual projected area of the group of tension rebars; $A_{C,N}^0$ is the reference projected area; $\Psi_{s,N}$ = factor considering the disturbance of the distribution of stresses in the concrete due to the proximity of an edge; $\Psi_{eC,N}$ = factor considering the effect of eccentricity; $\Psi_{re,N}$ = factor considering the effect of dense reinforcement between which the rebar is installed; $\Psi_{M,N}$ = factor considering the effect of the compression zone of the cross section of the attached reinforced concrete element in case of bending moments; k_1 = factor for resistance to concrete failure.

The decisive design resistance - defined as the characteristic resistance divided by the concrete partial safety factor (typically 1.5) - corresponds to the lowest design resistance among the three failure modes. This value must exceed the required design action, determined using the appropriate load combinations from the relevant standard (AS/NZS 1170.0 in the New Zealand context).

Under seismic loading conditions, the decisive design capacity is determined such that yielding of the reinforcement governs, enforced through an overstrength factor γ_{Rd} (typically 1.0 or 1.2 depending on the ductility class of the structure as defined in EN 1998-1), before any brittle failure mode can occur. This requirement is expressed as:

$$R_{d,eq} = N_{Rd,y,eq} \leq \min(N_{Rd,c,eq}; N_{Rd,sp,eq}) \quad (9)$$

where $N_{Rd,y,eq}$ is the seismic design resistance to yielding ($= \gamma_{Rd} \cdot N_{Rd,y}$); $N_{Rd,c,eq}$ is the seismic design concrete cone break-out resistance; $N_{Rd,sp,eq}$ is the seismic design bond-splitting resistance.

In addition, TR 069 introduces further seismic modification factors (α - and Ω -factors) applied to the characteristic resistances. For concrete cone breakout:

$$N_{Rk,c,eq} = \alpha_{eq} \cdot N_{Rk,c} \quad (10)$$

where $\alpha_{eq} = 1.0$ if crack is up to 0.3mm, or 0.85 if more than 0.3mm.

Bond-splitting resistance under seismic loading is expressed as:

$$\tau_{Rk,sp} = \eta_1 \cdot \alpha_{eq,sp} \cdot A_k \cdot \left(\frac{f_{ck}}{25}\right)^{sp1} \cdot \left(\frac{25}{\phi}\right)^{sp2} \cdot \left[\left(\frac{c_d}{\phi}\right)^{sp3} \cdot \left(\frac{c_{max}}{c_d}\right)^{sp4} + k_m \cdot K_{tr}\right] \left(\frac{7\phi}{l_b}\right)^{lb1} \quad (11.1)$$

$$\leq \tau_{Rk,ucr} \cdot \Omega_{cr,eq} \cdot \alpha_{eq,p} \quad \text{for } 7\phi \leq l_b \leq 20\phi \quad (11.2)$$

$$\leq \tau_{Rk,ucr} \cdot \left(\frac{20\phi}{l_b}\right)^{lb1} \cdot \Omega_{cr,eq} \cdot \alpha_{eq,p} \quad \text{for } l_b > 20\phi \quad (11.3)$$

Here, $\Omega_{cr,eq}$ takes values $\Omega_{cr,03}$ or $\Omega_{cr,05}$ or $\Omega_{cr,08}$ depending on the assumed seismic crack width; $\alpha_{eq,sp}$ is the seismic reduction factor for splitting; and $\alpha_{eq,p}$ is the seismic reduction factor for pullout. All seismic safety factors and coefficients are provided in the product-specific ETA.

2.3 Experimental Findings and Cyclic Seismic Loading Behaviour

New Zealand is one of the most seismically active countries in the world, and the structural reliability of post-installed reinforcement (PIR) under earthquake loading is therefore a critical consideration for both retrofit and new construction. Because many existing buildings require strengthening and new designs must satisfy stringent performance objectives, it is essential to verify that PIR can sustain cyclic demands and develop ductile behaviour comparable to cast-in reinforcement. Based on numerous experimental studies, the performance of qualified PIR systems has been demonstrated to be efficient and dependable in seismic conditions, providing confidence that - when properly designed, detailed, and installed - they can achieve cast-in equivalence under earthquake demands.

A broad experimental record shows that properly qualified post-installed rebars (PIR) can match, and in some cases exceed, the cyclic and seismic performance of conventional cast-in reinforcement. Simons (2007) demonstrated that under alternating push-pull protocols, PIR systems maintained bond strength across multiple large-amplitude cycles, with hysteretic bond-slip behaviour essentially identical to cast-in bars when static pull-out capacity was equivalent, fulfilling the “no adverse seismic effect” criterion (Fig 5).

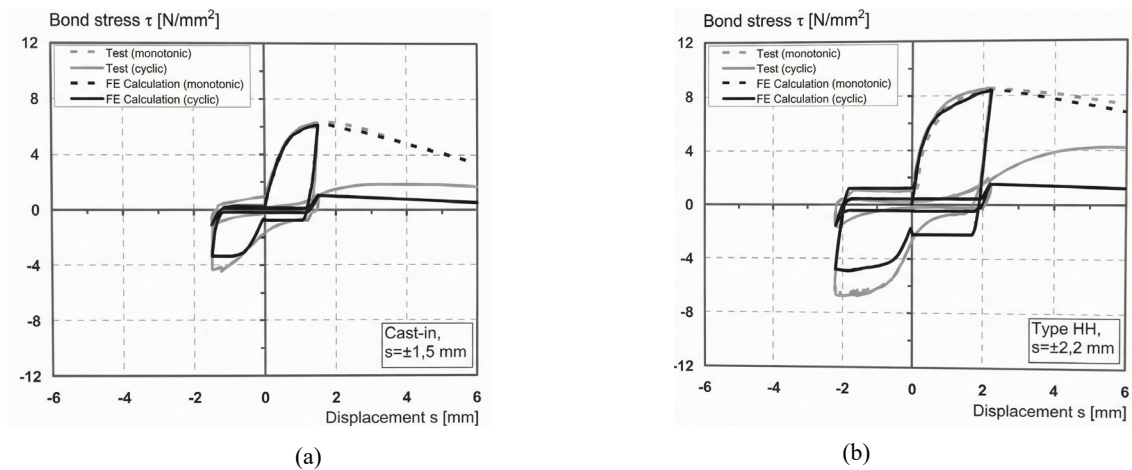


Figure 5: Comparison between bond stress and displacement in cast-in (a) and PIR systems (b). Obs: Original text in German, translated to English. Source: Simons (2007).

Confinement and embedment strongly influence these outcomes. Randl & Kunz (2012) showed that at low concrete cover both cast-in and PIR systems reach similar splitting limits; however, when cover or transverse reinforcement is increased, PIR can mobilise significantly higher bond stresses - typically 1.5-2 times those of cast-in bars - until limited by the adhesive rather than concrete splitting. These findings support design methodologies that explicitly incorporate cover and confinement when determining anchorage length.

Full-scale cyclic joint testing further reinforces these trends. In exterior beam-column joints, Mahadik et al. (2020) observed that epoxy-grouted PIR achieved peak strength, drift capacity, and energy dissipation just slightly lower to cast-in hooked bars but outperforming straight cast-in bars that exhibited early slip and pinched hysteresis loops (Fig 6). Column–foundation tests by Mahrenholtz et al. (2014) likewise showed ductile plastic hinge formation in PIR connections, with strength, stiffness degradation, and energy dissipation matching monolithic cast-in behaviour - provided that embedment length and installation quality met qualification requirements.

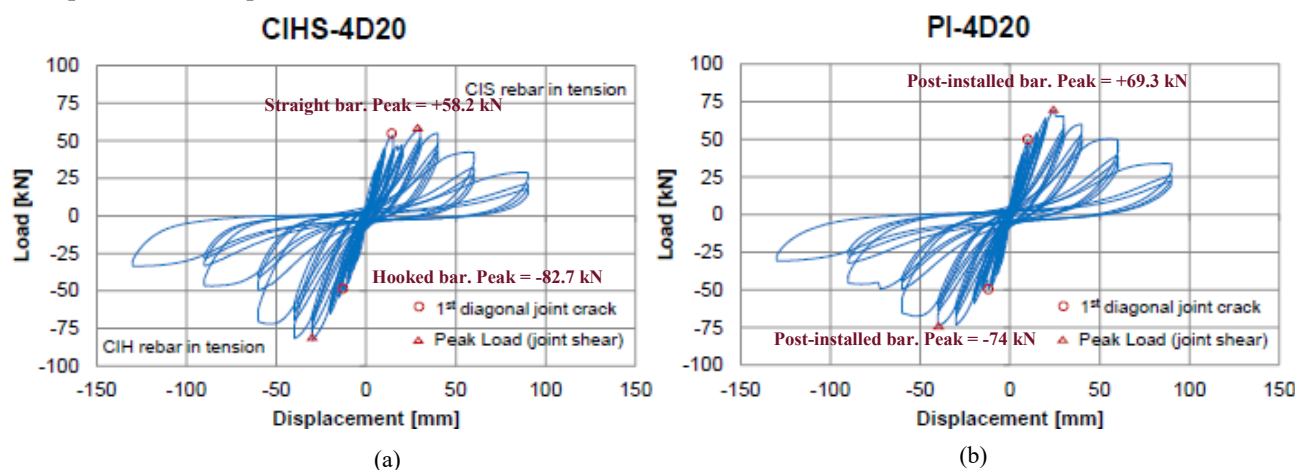


Figure 6: Hysteresis loop for Cast-in Hooked and Straight (CIHS) and Post-installed (PI) rebars of beam-column joints. Source: Simons (2007).

Behaviour near edges and at shorter embedment lengths highlights the need to explicitly check concrete-related failure modes. Using a beam-end test methodology, Rex et al. (2018) found that high-bond PIR frequently reached bar yield where cast-in straight bars failed prematurely by splitting or pull-out failure (Fig 7).

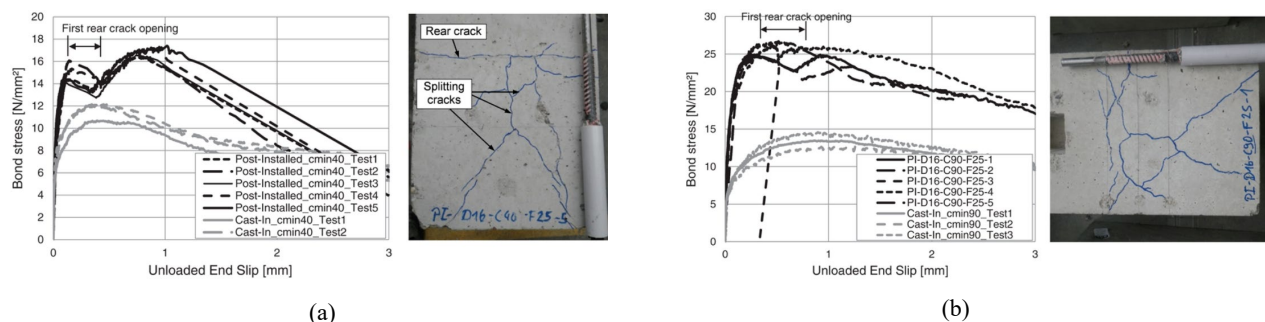


Figure 7: Bond stress vs unloaded end slip of a Post-installed rebars and Cast-in rebars for 40mm (a) and 90mm covers (b). Source: Rex et al (2018).

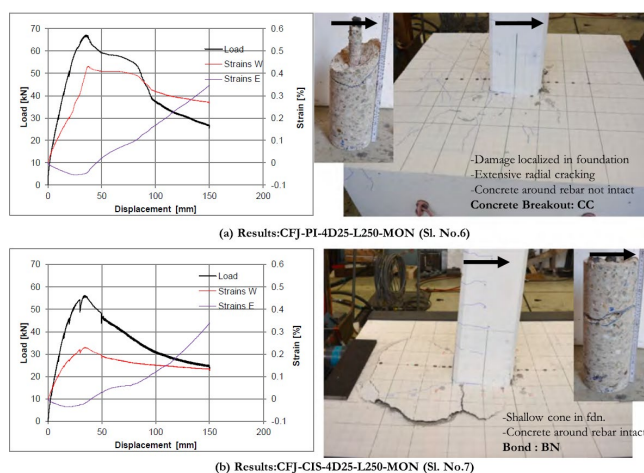


Figure 8: Results from monotonically loaded specimens (PI (a) & CIS (b)) with 250mm embedment. Source: Mahadik et al (2021).

methodologies such as EAD 330087 and EOTA TR 069. These frameworks align with international guidance aimed at ensuring cast-in equivalence for safety and serviceability, providing a robust foundation for PIR use in seismic regions such as New Zealand.

3 DESIGN IMPLICATIONS, EOTA TR069, AND CURRENT ADOPTION IN NZ

The growing body of international research on post-installed reinforcement (PIR) has direct implications for design practice in New Zealand, where seismic demands require anchorage systems capable of sustaining cyclic degradation, maintaining ductility, and avoiding brittle failure modes. Historically, the absence of PIR-specific provision in NZS 3101 has meant that New Zealand practitioners have relied on cast-in reinforcement theory, supplemented by international guidance such as the fib Model Code 2010, EN 1992-1-1, EN 1992-4, and European Technical Assessments (ETAs). As a result, current practice in New Zealand typically adopts a cast-in equivalence approach, whereby PIR may be designed using NZS 3101 development-length rules provided that equivalence to cast-in performance is demonstrated through supplier testing and product qualification. This approach has been widely accepted for simply supported connections and retrofit applications, but its extension to moment-resisting connections remains limited by the standard's scope.

In practice, New Zealand engineers increasingly reference European guidance - specifically EAD 332402 for qualification and EOTA TR069 for design - when PIR is used in structural applications beyond simple anchorage, such as beam-column joints, column-foundation interfaces, etc. This reflects a slow but increasing industry recognition that NZS 3101's generic development-length model does not explicitly check the governing anchorage failure mechanisms and therefore may under - or over-estimate safety in scenarios

involving short embedments, reduced cover, edge effects, or cyclic cracking. The forthcoming Technical Statement by ConcreteNZ on PIR (currently awaiting publication), is expected to clarify adoption of PIR in the country and recommend prequalification for seismic applications, but in its current form fails to recognise TR069-based design as a valid pathway for demonstrating compliance with the Building Code. This is expected to be addressed in a later revision of the Technical Statement. Nevertheless, PIR adoption in NZ is effectively expanding through practice-based acceptance despite the lack of direct references in the local code framework.

EOTA TR069 provides the rational, performance-based methodology needed to support this evolving practice. It explicitly verifies steel yielding, bond-splitting, and concrete cone breakout, prompting the designer to make ductile steel behaviour govern ahead of brittle modes. Importantly, TR069 permits the use of system-specific, experimentally derived bond strengths from ETAs rather than limiting design to the assumptions unique to cast-in rebar theory. This allows PIR to take advantage of the realistic bond stresses demonstrated in full-scale laboratory tests - often exceeding those predicted by concrete bond theory - thereby reducing embedment lengths, improving constructability when adequate confinement is present, maximising the potential of the connections through value engineering

For seismic design, TR069 extends this framework by requiring the application of overstrength factors, α -factors, and Ω -factors, ensuring that bond degradation and concrete damage under cyclic demands are explicitly accounted for. These checks reinforce the hierarchy-of-strength needed for ductile response and align closely with the seismic philosophy embedded in NZS 3101.

4 CONCLUSIONS

In the current New Zealand context, the use of post-installed reinforcing bars (PIR) is often unavoidable, particularly in seismic retrofit and remedial works. As a result, practitioners frequently rely on European guidance to address aspects not explicitly covered by NZS 3101. Supplier-endorsed qualification processes established regulatory acceptance pathways, and the development of a forthcoming ConcreteNZ Technical Statement have contributed to a workable interim framework for assessing and specifying PIR in selected seismic applications; however, these measures may not fully address all design and performance considerations.

The comprehensive set of experimental results reviewed in this paper demonstrates that qualified PIR systems consistently achieve ductile, repeatable, and cast-in-equivalent, or better behaviour under cyclic and seismic loading for straight bars. Studies across multiple specimen types - including beam-column joints, column-foundation interfaces, and elements near edges - show that PIR can mobilise high bond stresses, maintain stable hysteresis loops, and develop bar yielding without premature splitting or concrete breakout when designed properly.

These findings highlight three key parameters essential for seismic reliability: a fully qualified adhesive system, adequate confinement and the avoidance of concrete breakout via sufficient embedment. When these conditions are met, PIR can equal or exceed the anchorage performance of straight cast-in reinforcement, including large cyclic deformation demands typical of New Zealand earthquakes.

The design methodology provided by EOTA TR069 reflects this experimental evidence, offering explicit checks for steel yielding, bond-splitting, and concrete cone breakout, along with seismic-specific modification factors for cracked concrete, cyclic degradation, and overstrength requirements. By permitting the use of system-specific ETA bond strengths, TR069 enables more accurate and economical designs that better represent actual PIR behaviour. This transitional period in New Zealand practice is therefore expected to inform the evaluation and potential incorporation of PIR provisions in future revisions of NZS 3101.

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