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FUTURE DEVELOPMENTS IN PERFORMANCE-BASED SEISMIC DESIGN AND RELATED QUALIFICATION OF POST-INSTALLED ANCHORS IN NEW ZEALAND

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ABSTRACT

Performance-based seismic design of post-installed anchors needs the development of a new framework that can provide tools for designers to anticipate a realistic concrete-anchor system damage in seismic design scenarios relevant for New Zealand. Seismic capacity of anchors is not available for performance-based seismic design from anchor qualification methods considered currently as state-of-the-art. An outlook is provided in this article for the potential first steps in future developments based on a comprehensive assessment of the current state-of-the-art design and qualification approaches, incorporating a novel holistic framework proposed for post-installed anchor seismic performance.

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INTRODUCTION

Performance-based seismic design of post-installed anchors is an open question. It can be demonstrated that performance-based seismic design of post-installed anchors is not possible with internationally available design codes and using the anchor capacities available from seismic qualification methods considered currently as state-of-the-art [1-3]. The seismic qualification and assessment methods of post-installed anchors in international practice [4] were developed before 2013 and have been extensively used since then, in e.g. European Technical Product Specifications. The core ideas of these methods have not been further developed since their first publication. The seismic design methods and the seismic qualification methods are currently disconnected since the assessed anchor capacities are not related to actual building responses in seismic design scenarios [5]. The pass-or-fail criterion used in the state-of-the-art seismic qualification [4] does not provide fastener capacities that could be used for performance-based seismic design.

The New Zealand Code Environment

The New Zealand concrete structures standard, NZS 3101 [6] was first published in 1982, and as a basis (with the exception of the provisions for seismic loading), ACI 318-77 was used with minor modification.

After its first revision, NZS 3101 was re-issued in 1995, and minor changes have also been made to facilitate a planned future harmonization with the Australian concrete structures code. In particular, new sections covering the design for durability and fire have been based on the corresponding sections of AS 3600, modified as appropriate for New Zealand conditions, materials and regulations. The other non-seismic sections of NZS 3101 were still based largely on the provisions of the building code of the American Concrete Institute, with some of the new provisions of ACI 318-89 being incorporated in NZS 3101:1995.

After a second revision, NZS 3101 was re-issued in 2006 and is in force at the date of submitting this article for publication. During the second revision of NZS 3101 various technical advancements and improvements have been incorporated that have been developed since 1995. The non-seismic sections of NZS 3101 are largely based on ACI 318-02.

The current requirements for post-installed mechanical anchors and post-installed adhesive anchors (Clause 17.5.5 in NZS 3101:2006 [6]) have been added to the standard in 2006 and have been substantially modified by Amendment 3 in 2017, and read as follows: “Post-installed mechanical anchors and post-installed adhesive anchors shall pass the prequalification testing stipulated in ETAG 001, Annex E and be designed in accordance with EOTA TR045”. The rest of Chapter 17 in NZS 3101:2006 [6] provides design rules generally based upon ACI 318, but only covering cast-in-place ductile steel headed studs, headed bolts, hooked bolts and hooked steel plates with diameters less than 50 mm and embedment lengths shorter than 635 mm.

The European Code Environment

The referenced documents in Clause 17.5.5 of NZS 3101:2006 [6] has been superseded. ETAG 001 “Metal Anchors for Use in Concrete”, Annex E: “Assessment of Metal Anchors under Seismic Action” [7] was published in 2013 and has been superseded by EOTA TR 049 [4] in 2016. EOTA TR 045 “Design of Metal Anchors for Use in Concrete under Seismic Actions” [8] was published in 2013 and has been superseded by EN 1992-4 [9] in 2018.

As part of the Eurocode 2 series, EN 1992-4 [9] was published in July 2018 for the design of fastenings for use in concrete. The design methods in EN 1992-4 [9] are based on a combination of tests and numerical analyses consistent with EN 1990 [10] and rely on characteristic resistances given in European Technical Product Specifications. The seismic characteristic resistance of a post-installed anchor is determined in accordance with Annex C of EN 1992-4 [9], following the selection of the relevant recommended seismic performance category in accordance with EOTA TR 049 [4]. The second generation of Eurocode 2 (EN 1992-1-1:2023 [11]) also delegates the design of fastenings to EN 1992-4 [9]. It is noted that the second generation of Eurocode 8 (EN 1998-1-1:2024 [12]) contains Annex G (Normative) that complements the

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seismic design provisions of EN 1992-4 [9]. It is claimed in Annex G of EN 1998-1-1 [12] that it is intended that seismic design provisions in EN 1992-4 [9] are removed in the future revision of EN 1992-4 [9]. It is noted that EN 1992-4 [9] is in the process for a revision in CEN/TC 250 at the date of submitting this article for publication, but no details of these activities are publicly available.

Both EN 1992-4 [9] and Annex G of EN 1998-1-1 [12] introduce two seismic performance categories, but the selection criteria are different. The selection in EN 1992-4 [9] is based on seismicity level defined by the design peak ground acceleration at the actual design location together with the Importance Class as per EN 1998-1 [13]. The selection in EN 1998-1-1 [12] is based on seismicity level defined by the reference spectral acceleration corresponding to the constant acceleration range of the horizontal 5%-damped elastic response spectrum for the actual design location together with the ductility class of the main structure based on its deformation capacity and cumulative energy dissipation capacity. In EN 1998-1-1 [12], the selection of a seismic performance category is detailed for different combinations of connection types (structural or non-structural) and design options (OPs). The three design options are elastic design (OP1), capacity design (OP2) or ductile design (OP3). It is noted that the exact same design options are used in EN 1992-4 [9] too, but with a different nomenclature. The term Importance Class as per EN 1998-1 [13] is not used in EN 1998-1-1 [12].

It is assumed that the design in accordance with EN 1992-4 [9] is a performance-based approach that may be acknowledged for design scenarios under static and quasi-static loading. The same assumption cannot be confirmed for seismic design scenarios and the consequent seismic design of post-installed anchors in accordance with EN 1992-4 [9] can be either overly conservative or grossly unsafe in New Zealand. The calibration of the crack cycling protocol and crack widths for EOTA TR 049 [4] is introduced later in this article.

The Need for a Different Approach

It has been demonstrated [1-3,5,14-16] that performance-based seismic design of post-installed anchor connections is not possible in New Zealand, based on the current seismic qualification and seismic design methods recommended by NZS 3101 [6]. As a possible resolution for this challenge, a performance-based framework has been proposed [1] for the seismic behaviour of post-installed anchors in which the seismic damage of the concrete-anchor system is the key driving factor. The proposed framework anticipates major future developments both in the seismic qualification and the seismic design methods currently considered as state-of-the-art. An outlook for these future developments is given in the followings.

HOLISTIC ASSESSMENT OF THE STATUS QUO

The largest stock of post-installed anchors is used in buildings for non-structural parts and components. Current approaches in the seismic design for connections of non-structural parts and components, including those recommended by NZS 1170.5 [17], are overly simplified, and in the New Zealand code environment, also hold an inherent risk by addressing the connection design to 'appropriate material Standards'.

For steel-to-concrete connections with post-installed anchors, NZS 3101 [6] is deemed to be an appropriate material Standard. The definition of an appropriate material Standard is clear in NZS 1170.5 [17] and it requires that the appropriate material Standard must have been developed for use with NZS 1170.5 [17]. As it was shown, seismic design of post-installed anchors in accordance with NZS 3101 [6] is currently delegated to EN 1992-4 [9]. However, EN 1992-4 [9] does not fulfil the requirements of NZS 1170.5 [17] as an appropriate material Standard. Being part of the Eurocode 2 series, EN 1992-4 [9] is based on Eurocodes and not on AS/NZS & NZS standards. This situation generates an apparent compliance loophole and a challenge for New Zealand practitioners, where the values of actions in New Zealand Building Code (NZBC) compliant post-installed anchor design must be obtained from the relevant parts of the EN 1991 (Eurocode 1) series and EN 1998 (Eurocode 8) series for seismic actions, but the Eurocodes are not part of the NZBC B1 Verification Method [18] pathway.

The challenge is not only a formal code compliance gap. On one hand, it was shown [1,3,5] that the seismic qualification methods outlined in EOTA TR 049 [4] cannot be demonstrated to be conform with the reliability management assumptions of the Eurocodes. This creates a reliability disconnection between the EOTA and EN approaches. On the other hand, the New Zealand relevant characteristic seismic capacity of an anchor in accordance with the EOTA TR 049 [4] assessment is defined as its characteristic static capacity in cracked concrete (with a static crack width of w = 0.5 mm) or its subjectively reduced value, which does not provide usable information for performance-based seismic design of post-installed anchors [1,15]. The assessed characteristic seismic capacity of anchors is not related to actual earthquake responses in seismic design scenarios.

Seismic Design of Connections with Post-Installed Fasteners

The design of connections for non-structural parts and components with post-installed anchors cannot be considered as a 'trivial' design task [19].

The designer of connections for non-structural parts and components with post-installed anchors should understand the building response at the member level to be able to carry out performance-based connection design. The building response depends on the ground motion acting at the base of the building and the dynamic characteristics of the building (mass, stiffness, damping) together with its strength and ductility.

The designer should understand and determine the response of the floor which the component is mounted to or suspended from. This includes estimating not only the acceleration amplification of the ground motions at the actual floor level but also the understanding of the actual moment-curvature responses of the primary structural members in bending (beams and columns) at the actual floor level.

The designer should understand and determine the stresses and deformations at the section of the concrete member where the post-installed anchor connection is to be installed. Structural cracking depends fundamentally on the stresses and deformations developed, and as a simplification, may be characterised by the maximum crack width considered in the concrete member for the actual earthquake event.

The designer should understand the seismic response of the component. This depends on the mass of the component and the connection system (braces, attachment type, anchor type etc.). The designer should be able to anticipate if there is amplification or attenuation in the acceleration response of the component with regard to the floor acceleration and the designer should be able to estimate the oscillation period of the component that may be independent from that of the floor motion.

It is noted that it was observed for anchors with different working principles (expansion and undercut) that the anchor displacements can be very different, but the maximum anchor loads are not, for the same spectral/input acceleration [20]. The different load-displacement characteristics of various anchor types were not found to influence the earthquake driven oscillating behaviour of suspended non-structural components, and the same oscillating period seemed to develop for the same component connected with anchors of different working principles [21].

It is also noted that the most demanding phenomenon in the seismic capacity of post-installed anchors is the cyclically opening and closing cracks at the anchors, which latter in turn, are cyclically loaded by tension and compression loads due to the oscillating movement of the component. To address this demand, the state-of-the-art seismic qualification procedure outlined in EOTA TR 049 [4] also incorporates a specific testing protocol that mimics this phenomenon in a simplified way.

The real seismic capacity of post-installed anchors depends on the actual damage of the concrete-anchor system due to the earthquake. This damage is the result of a complex interaction between the actual loads, the general and local structural response of the member where the connection is installed to, the mechanical properties of the concrete, and the specific working principles of the anchor.

The designer should understand the working principle of the selected post-installed anchor and its sensitivity to seismic actions. Although the current state-of-the-art qualification and assessment methods for post-installed anchors [4] cannot provide useable capacity data for performance-based seismic design, the available information may help the designer to perceive the sensitivity of the anchor to a hypothetical seismic loading [1]. The designer may compare the published anchor capacities determined for static loading in cracked concrete and the published seismic capacities claimed in the assessment documents. Based on this comparison, designers may make their own engineering judgements.

For example, the designer may consider the anchor capacities published in assessment documents as it is illustrated in Figure 1, where European Technical Product Specification documents (ETA, European Technical Assessment) listed by EOTA have been collected and analysed for torque-controlled, wedge-type expansion anchors in Figure 1.a and for bonded injection anchors in Figure 1.b. The illustrated information is the ratio of the published characteristic capacities for static loading in cracked concrete and for the seismic performance category "C2" in accordance with EOTA TR 049 [4], expressed as $N_{Rk,seis,C2,ETA}/N_{Rk,p,cr,ETA}$ or $\tau_{Rk,seis,C2,ETA}/\tau_{Rk,p,cr,ETA}$. When the illustrated ratio is low, the designer may judge that the specific post-installed anchor product is particularly sensitive to the prescribed crack cycling at increased crack widths. When the illustrated ratio equals unity, the designer may judge that the specific post-installed anchor product is not particularly sensitive to the prescribed crack cycling at increased crack widths. Such engineering judgements, however, cannot be generalised since the EOTA TR 049 [4] approach represents only one hypothetical earthquake demand and cannot be related to general seismic design scenarios.

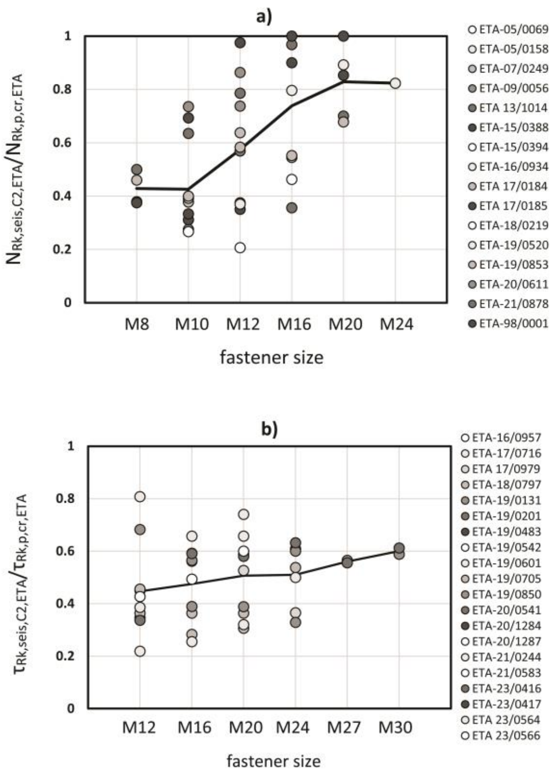

Figure 1: (a) Ratios of $N_{Rk,seis,C2,ETA}/N_{Rk,p,cr,ETA}$ in European Technical Assessment documents for torque-controlled, wedge-type expansion anchors and (b) Ratios of $\tau_{Rk,seis,C2,ETA}/\tau_{Rk,p,cr,ETA}$ in European Technical Assessment documents for bonded injection anchors.

Consideration of the Seismic Damage of the Concrete-Anchor System

A holistic approach has been proposed for the seismic behaviour of post-installed anchors [1], which hypothesises that seismic design scenarios may be characterized with a single parameter, where the simplified parameter considers the actual damage of the concrete-anchor system and the load demand on the connection, and such a parameter may be the basis of connection design and anchor qualification in the future. The most appropriate parameter for this purpose is yet to be determined. It may be assumed that, in a simplified way, the concrete-anchor system damage can be characterized by the actual width of the crack formed at the location of the anchor and the actual number of crack opening and closing cycles acting on the anchor together with the anchor load that reflects the actual earthquake scenario [1].

It is the current consensus in practice that the concrete at the location of a post-installed anchor is assumed to be cracked during an earthquake. Concrete has low tensile strength, and cracks are expected to form in service conditions too, without any earthquake motions. The width and the movement of cracks have a profound negative influence on the anchor response. The capacity is reduced in cracked concrete and depending on the type of the force transfer mechanism (undercut, expansion, bonding) this strength loss can be significant (20 to 80 percent) for large crack widths [22]. Earthquake motions are expected to result in the cracking of concrete coinciding with the anchors. Cracks do not only open but also close during earthquakes which can significantly affect the performance and the capacity of the anchor.

The designer should be able to anticipate the extent of the concrete-anchor system damage for the seismic design scenario considered. Currently, there are no tools available for designers to make such assumptions. Borosnyoi-Crawley [1] proposed the Accumulated Damage Potential (ADP) defined by Mahrenholtz [20] as a practical tool, simple but meaningful enough, to describe the seismic damage of the concrete-anchor system.

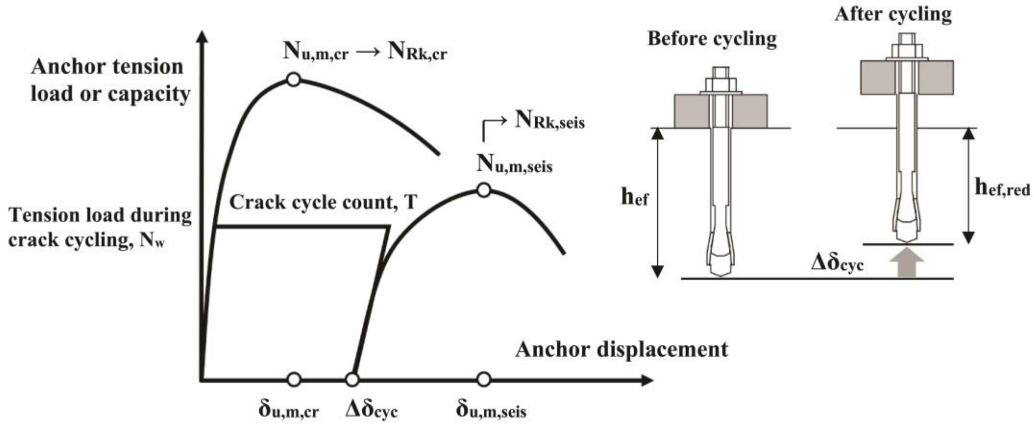

Figure 2: Schematic load-displacement curves indicating the change in the static pullout behaviour and tension capacity together with the permanent displacement after crack cycling tests for a combination of crack cycling number T, anchor tension load during crack cycling N_w and maximum considered crack width w for a constant f'c specified concrete strength. Note: $N_{u,m}$ and $\delta_{u,m}$ denotes mean ultimate tension capacity and displacement, respectively. [2].

The ADP can be interpreted as the energy consumption of the concrete-anchor system during seismic damage manifested by crack cycling, and it has the physical unit of Nm. The ADP is considered to be related to the axial displacement of an anchor during crack opening and closing cycles [23]. The ADP theory is based on the idea that for a given number of crack cycles, the incremental axial displacement of an anchor during a crack cycling experiment is a function of the tension load and the crack width, and the correlation of these two variables. Anchor displacement is considered as a result of the damage potential accumulated over the crack cycles. Assuming a linear influence of the tension load (N) and the crack width (w), the ADP was expressed as an integral [20, 23]:

$$ADP = \int \big(N(t) \cdot w(t)\big) dt \qquad (1)$$

The linearised version of the ADP [23] includes a constant integration coefficient and the ADP is simply calculated as the sum of the multiplication of the tension load (N), crack width (w), crack cycle count (T) and the integration coefficient (a) at each crack width level, expressed as:

$$ADP = \sum a \cdot N_i \cdot w_i \cdot T_i \qquad (2)$$

As it seems, the ADP is higher when the tension load on the anchor during the earthquake event (N) is higher (e.g., heavier suspended non-structural components), or the maximum crack width of the concrete substrate at the location of the anchor (w) is higher (e.g., areas where high tensile stress is developed in the reinforcement at a closer location to high deformation zones, or plastic hinges of a member), or the number of the crack opening-closing cycle (T) is higher (that is defined by the intensity and duration of the earthquake and the dynamic response of the member where the connection is installed in the building).

The ADP can provide a useful tool for the designer to select an appropriate embedment depth for the anchors since the residual load capacity of anchors after an earthquake event primarily depends on the remaining embedment depth ($h_{ef,red} = h_{ef} - \Delta\delta_{cyc}$) after the load and crack cycling has resulted in a permanent displacement ($\Delta\delta_{cyc}$) reducing the original embedment depth (h_{ef}; see Figure 2). This permanent displacement ($\Delta\delta_{cyc}$) can be roughly estimated by the ADP, as it was demonstrated by Mahrenholtz [20].

It is noted that future research is needed to understand the real physical meaning of the ADP and to determine if the linear assumption can be justified or not. Borosnyoi-Crawley [1] compared the accumulation of ADP per cycles with actual fastener displacement accumulation observed during laboratory experiments performed on either torque-controlled, wedge-type expansion anchors [21] or on cast-in headed studs [20]. It was found that the tendency of damage accumulation represented by the linearized ADP is different from the displacement accumulation observed experimentally. The linearized ADP tends to overestimate the damage accumulation tendency for cycling small crack widths and tends to underestimate the damage accumulation tendency for cycling large crack widths [1].

The Significance of Crack Widths

The maximum considered crack width during an earthquake event has major impact on the anchor behaviour that is clearly reflected in the ADP. Structural cracking during earthquakes is a result of the interaction of several parameters, such as the intensity and duration of the actual ground motion, the actual building response with all of its non-linearities, the actual seismic weight (or seismic mass) incorporated with the anchor or group of anchors, the exact actual location of the connection within a structural member, and the geometry, detailing and structural response of the structural member (beam, slab, column, wall) at the exact actual location of the connection. The available crack width prediction models in codes, including the NZS 3101 [6] crack width prediction model based on Frosch [24], are valid only for Serviceability Limit State (SLS) conditions, when the stress developed in the reinforcement is relatively low. Prediction of the maximum considered crack widths for seismic design scenarios based on crack width prediction models in codes, especially when the stress in the reinforcement could reach the yield level, may result in misleading assumptions.

It has been demonstrated [14,25] that empirical crack width estimating proposals (including the NZS 3101 [6] crack width prediction model) are not robust and have the tendency to underestimate the experimentally observed large crack widths and to overestimate the experimentally observed small crack widths. To demonstrate this, Figure 3 illustrates calibration examples for the crack width prediction model of NZS 3101 [6]. The experimental crack width database of Sakalauskas et al [26] was used, compiling 1352 data points from 274 flexural members from 21 literature sources. It is noted that the analysis in [14,25] was based on the experimental crack width database of McLeod [27], compiling 204 data points. In Figure 3 the ratios of the experimental and calculated maximum crack width values (w_{exp}/w_{cal}) are indicated over the observed experimental maximum crack widths (w_{exp}), grouped by the level of the tensile stress in the reinforcement. It can be seen that the crack

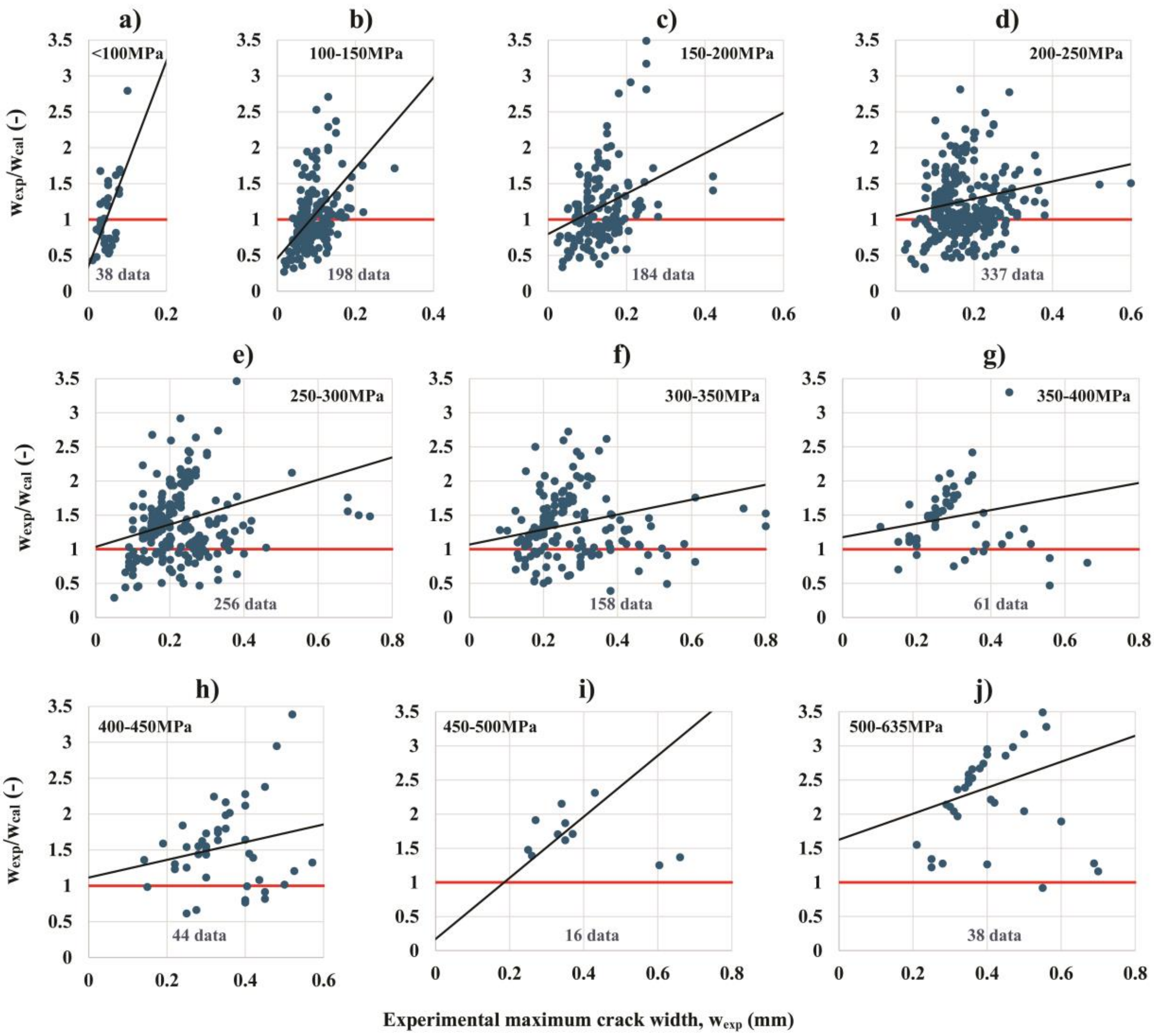

Figure 3: Ratios of the experimental and calculated maximum crack widths (w_{exp}/w_{cal}) for the crack width prediction model of NZS 3101 [6] calibrated against the experimental crack width database of Sakalauskas et al [26], represented as groups corresponding to different levels of the tensile stress in the reinforcement.

width prediction model of NZS 3101 [6] tends to generally underestimate the crack widths (w_{exp}/w_{cal} > 1.0), especially when the tensile stress in the reinforcement exceeds 200 MPa.

Ideally, designers should make their own engineering judgements in anticipating the maximum considered crack width during an earthquake event. Currently, there are no tools available for designers to make such assumptions. Borosnyoi-Crawley [1] proposed an estimation for the maximum considered crack widths in seismic design scenarios that creates a relationship between the tensile stress calculated for the steel reinforcement in accordance with NZS 3101 and the maximum crack widths considered, based on the findings of [14,25]. It is noted that any such a proposal needs future verification since the results depend on the crack width database used for the calibration. It is also noted that the proposal in [1] is limited to beams and slabs and cannot be extended to walls, columns and diaphragms since it considers the compensation for the model error of the empirical crack width prediction of Frosch [24] that is the basis of Clause 2.4.4 in NZS 3101:2006 [6]. Future work is needed in this field.

It is noted that the impact of the crack opening and closing cycles near the targeted maximum crack width may be disproportionately high on the response of post-installed anchors, as it was reported in [20], therefore any inaccuracy in the prediction of the maximum crack width considered in an actual seismic design scenario may have significant influence on the result of the anchor capacity verification.

The Significance of the Number of Crack Opening and Closing Cycles

Seismic events may impose a wide range of actual crack widths and a wide range of actual number of crack opening and closing cycles acting on post-installed anchors. The number of the crack opening and closing cycles is paramount in the residual load capacity of anchors after an earthquake event. The designer should be able to estimate the number of the crack opening and closing cycles to perceive the level of damage of the concrete substrate in the seismic design scenario.

Determining the number of the crack opening and closing cycles is a time consuming task involving detailed numerical analyses that can be performed only in cases of sophisticated seismic design. One example of such analysis is found in the research work of Wood et al [28] for reinforced concrete frame building prototypes typical in California, USA. The number of crack opening and closing cycles depends not only on the characteristics of the ground motion (magnitude, duration etc.) but also on the building response with all of its nonlinearities and the exact location of interest within the building (floor

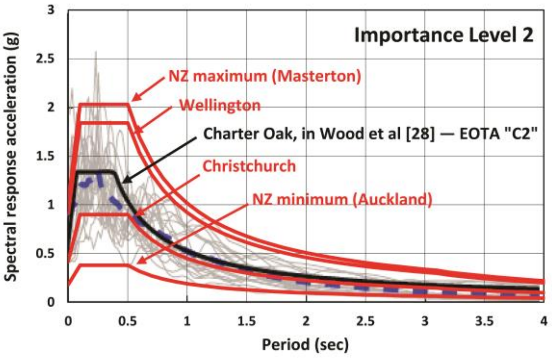

Figure 4: Design target acceleration spectra defined by Wood et al [28] for Charter Oak (Los Angeles County, California, USA), in comparison to four DZ TS 1170.5:2024 [29] spectra at New Zealand site locations for Importance Level 2 buildings.

level, structural member type and location of connection in the member). Currently, no simple tools are available for designers to determine the number of the crack opening and closing cycles in building types typical in New Zealand. This is one of the most fundamental barriers for the development of performance-based seismic design and qualification methods for post-installed anchors. It is noted that the number of the crack opening and closing cycles in a crack cycling test for anchor qualification must appropriately reflect the need of actual seismic design scenarios. It was demonstrated in [28] that the number of the crack opening and closing cycles may be substantially different for shorter and taller buildings, i.e. faster crack cycling was found for shorter buildings and slower crack cycling was found for taller buildings. Averaged approaches, however, like those recommended by [28] and adopted by EOTA TR 049 [4], are not suitable for performance-based design since the influence of the type of building and the influence of the number of storeys cannot be directly considered.

Limits for Performance-Based Design in the Current State-of-the-Art

The current pass-or-fail assessment criterion in the seismic qualification procedure of post-installed anchors relevant for New Zealand in EOTA TR 049 [4] considers the concrete-anchor system damage for a hypothetical earthquake demand that is based on a theoretical study performed on certain reinforced concrete moment resisting frame buildings designed and detailed by ACI 318-08, for 475-years return period design earthquake events in Occupancy Category II (as per ASCE 7-05) building prototypes, located in Charter Oak, LA County, California [28]. The studied building prototypes and the target design acceleration spectrum (see Figure 4) considered for the site location at Charter Oak in the hypothetical earthquake scenario resulted in a crack opening and closing cycle number of 59 [20]. The EOTA TR 049 [4] approach stipulates 0.5 mm and 0.8 mm maximum considered crack widths and $0.4N_{u,m,cr,0.8}$ and $0.5N_{u,m,cr,0.8}$ anchor loads for the first 45 cycles and the remaining 14 cycles, respectively; where $N_{u,m,cr,0.8}$ is the mean ultimate static tension capacity of the anchor in cracked concrete with 0.8 mm static crack width. This scenario corresponds to one value of ADP as it could be defined by Eqs. (1) and (2). If an anchor passes the demand for this hypothetical earthquake scenario, then the characteristic seismic capacity of the anchor in accordance with the EOTA TR 049 [4] assessment is defined as its characteristic static capacity in cracked concrete (with a static crack width of w = 0.5 mm).

As it seems, even if the designer is able to assume the concrete-anchor system damage for the seismic design scenario and calculate a design value of ADP for the connection of the component, there is no anchor seismic capacity information available for the design. The EOTA TR 049 [4] pass-or-fail assessment criterion could only tell if the anchor passed or failed the demand of a hypothetical earthquake scenario, but actual seismic capacity of the anchor is not provided.

FUNDAMENTALS OF THE PROPOSED APPROACH

The general structure of the holistic approach with the example of connections of non-structural components in buildings is illustrated in Figure 5.

Five different design inputs are considered:

- The assumed consequences of failure, represented by e.g., the Importance Level of a building.
- The geographic location of the building that includes the site soil condition information.
- The structural characteristics of the building implicit for its seismic response.
- The location of the non-structural component in a structural member.
- The seismic mass and period of the non-structural component.

The holistic approach provides a transparent, design-driven platform necessary for future developments of performance-based seismic design and related qualification of post-installed anchors. It reveals the tasks of the designer as (i) determining the relevant number of crack opening and closing cycles for the specific structural member in the building at the given location, for the given site soil conditions and for the given Importance Level of the building; (ii) determining the maximum considered crack width for the specific structural member in the building; (iii) determining the maximum load demand for the connection considered in the seismic design scenario; (iv) calculating the relevant ADP at the design level.

The design resistance of the anchor must reflect:

- The actual residual load capacity of the anchor that corresponds to the remaining embedment depth ($h_{ef,red} = h_{ef} - \Delta\delta_{cyc}$) after the load and crack cycling has resulted in a permanent displacement ($\Delta\delta_{cyc}$) reducing the original embedment depth (h_{ef}; see Figure 2).
- The reliability level that corresponds to the actual consequences of failure reflecting the brittle nature of the concrete related anchor failure in the seismic design scenario (i.e. pullout failure for most anchor types or concrete cone breakout in limited number of cases) and the Importance Level of the building, which are both implicit in the reliability index.
- The influence of the actual maximum crack width considered in the seismic design scenario that could be reflected in the partial safety factor considering the influence of crack width on the coefficient of variation of the mean ultimate capacity of post-installed anchors [2].
- The seismic performance objectives of the anchor other than the load capacity; related to either deformability (e.g. slip-type permanent displacement developed during the earthquake event in the form of pullout) or ductility [30] (e.g. plastic deformation developed in the steel components of the anchor during the earthquake event, if any).

Noticeably, the characteristic static load capacity of the anchor in cracked concrete (with a static crack width of w = 0.5 mm)

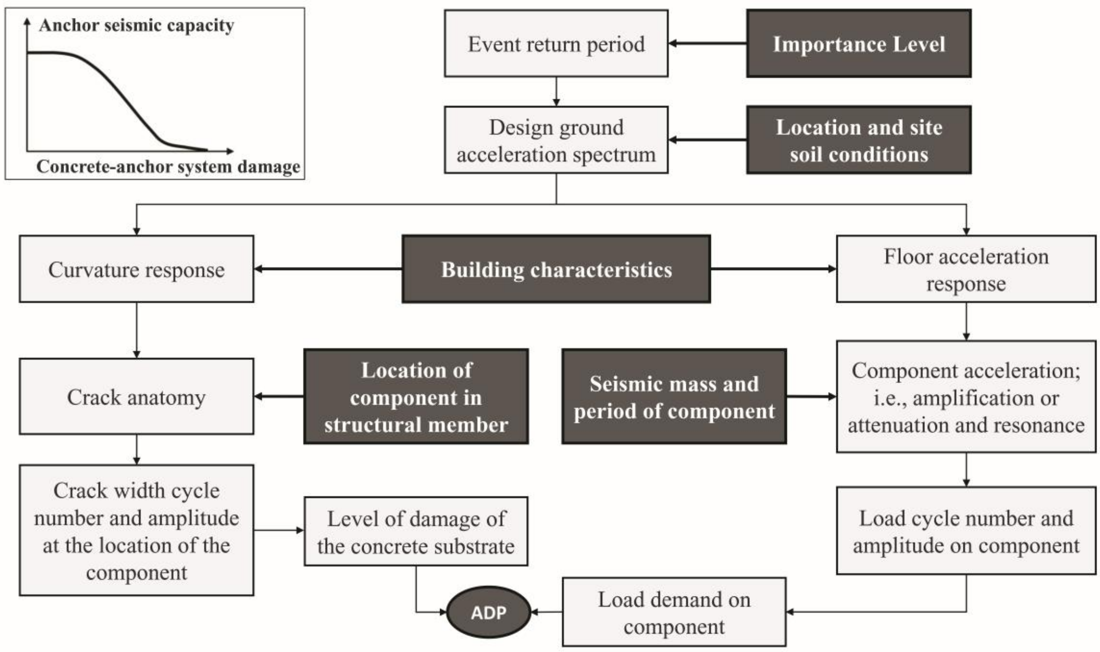

Figure 5: Holistic framework for post-installed anchor seismic performance [1].

that is the claimed maximum seismic load capacity of an anchor in accordance with the EOTA TR 049 [4] seismic qualification and assessment together with the partial safety factors currently set by EN 1992-4 [9] could not represent such information.

NEXT STEPS

Developments in the performance-based seismic design and qualification of post-installed anchors need to divert from the current state-of-the-art that relies on a conceptually incorrect qualification method, and the consideration of different approaches is needed.

As it has been highlighted in this article, multiple gaps need attention in the development of New Zealand specific design tools and methods to help designers anticipating the concrete-anchor system damage for seismic design scenarios and calculate the ADP at design level.

A detailed analysis should be performed for typical New Zealand building types in the future, similarly to the research work of Wood et al [28], to be able to generate base line data for a reasonable estimation of the number of crack opening and closing cycles for different seismic scenarios in New Zealand. It is noted that the number of crack opening and closing cycles depends not only on the magnitude and duration of the earthquake, but also on the structural type and the number of storeys of the building considered.

A detailed analysis should be performed for typical New Zealand building types in the future, to determine reasonable maximum crack widths considered for different structural members in different seismic scenarios in New Zealand. It is noted that the stress in the reinforcement, thus the expected crack width, can vary over a wide range, up until the yielding of the reinforcement, depending on the structural type of the building and the exact location of the component within a structural member.

It was demonstrated [20] that the seismic qualification test in accordance with EOTA TR 049 [4] is approximately three times as demanding as the shake table test in realistic earthquake simulations of non-structural component systems [21,31], provided that the same maximum anchor loads and crack widths are tested. In more realistic seismic design of connections with post-installed anchors, the anchor capacity (load) determined by qualification testing should reflect realistic anchor loads corresponding to actual seismic design situations. A detailed analysis should be performed for typical non-structural components and systems used in New Zealand, to determine more realistic anchor loads both for design and qualification purposes.

The anchor seismic qualification framework developed in the future should clearly reflect the design needs and should provide capacity information for the anticipated concrete-anchor system damage in seismic design scenarios. Anchor qualification procedures should provide a set of crack cycling test protocols for the range of ADP of interest (or other damage parameter yet to be defined). The crack cycling qualification tests should be performed for the range of maximum considered crack widths of interest. The currently used target values of 0.5 mm and 0.8 mm for the maximum considered crack width should be revised as they both lack scientific evidence in their validation [14].

The anchor characteristics available from seismic qualification and assessment should be relevant for performance-based seismic design. As a minimum, the characteristic residual seismic tension capacity, $N_{Rk,seis}$; the mean ultimate seismic displacement, $\delta_{u,m,seis}$; and the permanent displacement after crack cycling, $\Delta\delta_{cyc}$ (see Figure 2) should be reported for tension loads, based on crack cycling tests performed for the range of ADP, maximum considered crack widths and anchor loads of interest (it is noted that this article does not discuss the topic of shear loading). Anchor seismic capacity needed for performance-based design cannot be a single number as it is claimed by the EOTA TR 049 [4] qualification and assessment method relevant for New Zealand.

Figure 6 schematically illustrates the detailedness of anchor capacity information needed for performance-based seismic design to be provided by an appropriate seismic qualification and assessment framework developed in the future.

It was generally observed [20-21,31-36] that the load-displacement curve for post-installed anchors tested in crack cycling tests is bounded by the monotonic envelope from corresponding static tests performed with the same crack width.

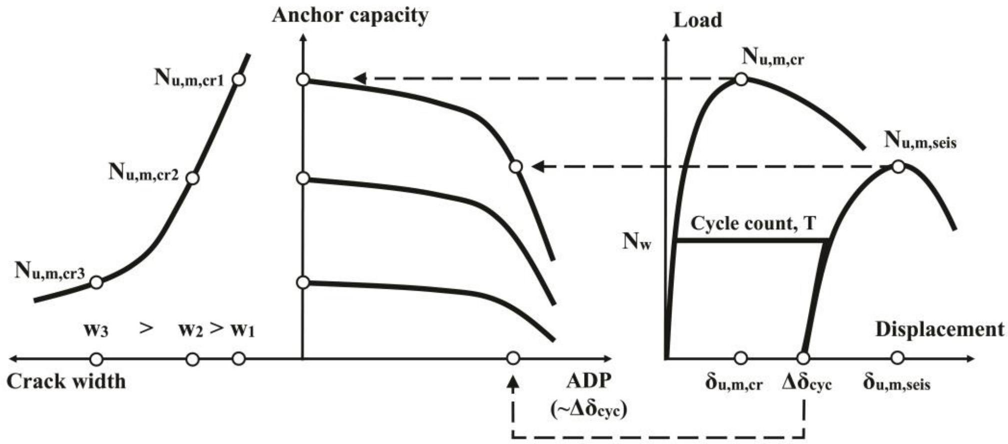

Figure 6: Schematic representation of the relationship of anchor performance parameters provided for static and seismic loading in future qualification and assessment framework.

In future qualification and assessment documents it would be reasonable to provide the characteristic static tension capacities of anchors corresponding to a complete range of crack widths, from the uncracked condition up to w = 1.5 mm static crack width, as it is schematically illustrated on the far left diagram in Figure 6. This information may provide designers with the impression of the maximum possible seismic capacities of the anchors.

It is noted that the coefficient of variation for the mean ultimate static tension capacity ($N_{u,m,cr}$ in Figures 2 and 6) of post-installed anchors strongly depends on and is generally increasing with increasing crack widths [37]. The magnitude of the change varies with the specific working principle of the anchor, e.g. in the case of mechanical post-installed anchors, it is more pronounced for screw anchors than for expansion anchors or undercut anchors [20]. It was also observed that this variation for a given crack width generally decreases with the increase in the compressive strength of the concrete [37]. Such observations confirm the need of publishing the characteristic values and not the mean values, to include the influence of the variation in the capacity.

It is also noted that the coefficient of variation for the displacement that corresponds to the mean ultimate static tension capacity ($\delta_{u,m,cr}$ in Figures 2 and 6) of post-installed anchors also strongly depends on and is generally increasing with increasing crack widths [37]. The magnitude of the change is larger than what could be observed for load capacity. In future qualification and assessment documents it would be reasonable to provide the coefficient of variation for the displacement performance data. Displacements corresponding to the mean ultimate static tension capacity ($\delta_{u,m,cr}$ in Figures 2 and 6), to the targeted ADP of the crack cycling part of the tests ($\Delta\delta_{cyc}$ in Figures 2 and 6), and to the mean ultimate residual seismic tension capacity ($\delta_{u,m,seis}$ in Figures 2 and 6) may be the minimum information provided in future qualification and assessment documents. It is noted that in the current practice of EOTA TR 049 [4] the scatter of the displacements is considered only as that of the fastener at 50% of the mean ultimate tension load from the residual capacity tests after performing the crack (or load) cycling tests and only the displacement in the residual capacity test is taken, i.e. the displacement that occurred during the crack (or load) cycling phase is neglected.

Anchor qualification for performance-based seismic design of connections with post-installed anchors would require a virtually infinite number of combinations of crack cycling numbers, anchor loads, maximum considered crack widths and concrete strengths for every single anchor product for all available diameters. Future work is necessary to simplify this need and develop a practical way that does not lead to unreasonable sophistication but result in a practically feasible qualification system that could provide reliable anchor capacities for performance-based seismic design.

The designer should be provided with the possibility of the selection of appropriate qualification procedures and the selection of appropriate seismic capacities of anchors that are based on the anticipated concrete-anchor system damage in actual seismic design scenarios.

The significant knowledge gaps highlighted in this article need urgent action. The engineering community in New Zealand should acknowledge the flaws in the methods currently used. The missing elements in the holistic framework of seismic design and qualification of post-installed anchors highlighted in this article should be addressed in open discussions at the appropriate engineering forums.

CONCLUSIONS

Performance-based seismic design of post-installed anchors is an open question. It can be demonstrated that performance-based seismic design of post-installed anchors is not possible with internationally available design codes and using the anchor seismic capacities available from qualification methods considered currently as state-of-the-art. From a New Zealand perspective, this can result in situations where the seismic design of post-installed anchors may be either overly conservative or grossly unsafe. As a possible resolution for this challenge, a performance-based framework was proposed for the seismic behaviour of post-installed anchors in which the seismic damage of the concrete-anchor system is the key driving factor. The proposed framework anticipates future developments both in the seismic qualification and the seismic design methods currently considered as state-of-the-art. The first steps for these future developments regarding the performance-based seismic design include (a) developing generic data for reasonable estimation of the number of crack opening and closing cycles for different seismic design scenarios in New Zealand; (b) developing generic data for reasonable estimation of the maximum crack widths considered for different structural members in different seismic design scenarios in New Zealand; (c) developing generic data for reasonable estimation of anchor loads for different seismic design scenarios in New Zealand; (d) developing simplified tools for designers to help them being able to estimate the concrete-anchor system damage for seismic design scenarios and calculate a design value of ADP (or other damage parameter yet to be defined) for the connection design of components. The first steps in the future developments regarding the seismic anchor qualification include (a) developing a framework for realistic seismic performance objectives of post-installed anchors; (b) developing a framework to simplify the needs of performance-based seismic design for reliable anchor seismic capacities; (c) developing a set of test protocols that does not lead to unreasonable sophistication but clearly reflects actual concrete-anchor system damage levels relevant for New Zealand seismic design scenarios. The future developments must be driven by designers with the inclusion of the wider engineering community, researchers and anchor manufacturers.

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