Performance-based seismic design of post-installed anchors in New Zealand

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ABSTRACT

Performance-based seismic design of connections is not possible in international practice, based on the state-of-the-art seismic qualification methods available for post-installed anchors. This situation needs revision since the current seismic design of post-installed anchors in New Zealand in accordance with NZS 3101 can be either overly conservative or grossly unsafe. The pass-or-fail criterion used in current state-of-the-art seismic qualification of post-installed anchors (EOTA TR 049, which is the superseding document of ETAG 001 Annex E; referenced in NZS 3101) provides a single value for anchor seismic capacity relevant for New Zealand, which does not allow for performance-based seismic design. In turn, both the seismic demand on anchors and the seismic capacity of anchors are continuums and must be approached accordingly. A performance-based framework is proposed for the basis of future developments in seismic design and qualification of post-installed anchors in New Zealand. It is demonstrated that the approach is fully transparent and provides the possibility to identify key driving parameters that may need further investigation. The proposed approach is derived from fundamental design input information and creates a holistic framework in which the concrete-anchor system damage is in focus, represented by a complex performance parameter that may drive the seismic design and qualification process. This paper gives further insight to the proposed holistic approach to promote its development and future practical application in New Zealand.

1 INTRODUCTION

Post-installed anchors as proprietary products are widely used in New Zealand. Structural and non-structural connections are both designed. Structural connections are understood as systems of fasteners (e.g. post-installed mechanical anchors or post-installed adhesive anchors etc.) and fixtures (e.g. base plates etc.) that transmit actions to and/or between primary earthquake resisting parts of a structure. Structural connections may be elements of critical load paths for lateral forces generated by horizontal ground movements in seismic restraining or strengthening systems. Non-structural connections are understood as systems of fasteners and fixtures that transmit actions from components of engineering systems to the earthquake resisting parts of a structure. Non-structural connections can therefore also be elements of load paths for lateral forces generated by horizontal ground movements in seismic restraining systems.

1.1 The current code environment

In NZS 1170.5:2004 Section 8.7, the post-installed mechanical or adhesive anchors are generally considered to be non-ductile. Connection design is addressed to 'appropriate material Standards' in NZS 1170.5:2004. For steel-to-concrete connections with post-installed anchors, NZS 3101:2006 is deemed to be an appropriate material Standard. NZS 3101:2006 is a primary reference document in the New Zealand Building Code (NZBC) B1 AS/VM (1st Ed., Amd 21, Ver. 2023-11-02), and EN 1992-4:2018 is the superseding document in effect for a secondary reference document in it (see NZS 3101:2006 Clause 17.5.5). Consequently, EN 1992-4:2018 is currently an NZBC B1 Verification Method, and therefore designers in New Zealand are expected to perform the seismic design of post-installed anchors accordingly. The definition of an appropriate material Standard is clear in NZS 1170.5:2004 and it requires that the appropriate material Standard must have been developed for use with NZS 1170.5:2004. However, EN 1992-4:2018 does not fulfil this requirement. EN 1992-4:2018 is based on Eurocodes and not on AS/NZS & NZS standards, and if anchor loads are calculated based on AS/NZS & NZS standards then EN 1992-4:2018 cannot be directly used for design. This situation generates an apparent compliance loophole and a challenge for New Zealand practitioners, since the values of actions in 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 pathway.

The EN 1992-4:2018 design (together with the relevant parts of EN 1990:2002 and EN 1992-1-1:2004) leads to a connection ULS safety associated with Reliability Class RC2 (i.e. Consequence Class CC2), with a reliability index of $\beta_{50} = 3.8$ for a 50-year reference period. It is noted that Table C.1 in EN 1992-4:2018 is in contradiction with the claim given in Clause 4.1(2) of EN 1992-4:2018 and, therefore, the recommendations given in columns I, III and IV in Table C.1 of EN 1992-4:2018 must be omitted. Reliability Class RC2 would cover applications in Importance Class II (as per EN 1998-1:2004). The New Zealand equivalent may be Importance Level 2, in accordance with AS/NZS 1170.0:2002 and NZS 1170.5:2004, with the ULS based on 500-year return period earthquake events (10% probability during a 50-year design working life). It can be realized that post-installed anchor design in accordance with EN 1992-4:2018 in a New Zealand context offers the possibility of designing connections only in IL2 buildings (as per AS/NZS 1170.0:2002).

It is noted that the second generation of Eurocode 8 (EN 1998-1-1:2024) has been published recently, and it contains Annex G (Normative) that complements the seismic design provisions of EN 1992-4:2018. It is claimed in Annex G of EN 1998-1-1:2024 that it is intended that seismic design provisions in EN 1992-4 are removed in the future revision of EN 1992-4. In accordance with EN 1998-1-1:2024, seismic actions should be specified in terms of their return periods, depending on the specified Limit State (LS) and Consequence Class (CC) of structures. EN 1998-1-1:2024 defines four LSs (near collapse, NC; significant damage, SD; damage limitation, DL; fully operational, OP) and EN 1990:2023 defines five CCs (CC0 to CC4; but provisions in Eurocodes cover design rules only for structures classified as CC1 to CC3). The term Importance Class as per EN 1998-1:2004 is not used any further in EN 1998-1-1:2024. The return periods corresponding to specific combinations of LSs and CCs are Nationally Determined Parameters (NDPs) and are to be given in National Annexes of the member states of the EU, however the default values related to the 475-years return period selected for LS SD + CC2 are indicated for buildings in Table 1 (EN 1998-1-2:2023).

It is noted that in the first generation of Eurocode 8 (EN 1998-1:2004), there was no clear target reliability. The second generation of Eurocode 8 (EN 1998-1-1:2024) sets a reliability target for seismic design in terms of annual probability of exceedance of collapse (which is close to the formal Limit State NC) as $p_1 = 2 \cdot 10^{-4}$ (see in Table 1 indicated in bold characters), corresponding to $\beta_1 = 3.5$ that falls between those of CC3 ($\beta_1 = 3.3$) and CC4 ($\beta_1 = 3.7$) as per ISO 2394:2015 Table G.4 for large relative cost of safety measure. It is noted that such target reliabilities should be seen as indicative values for supporting economic optimization and may not be acceptable for life safety risks. Also, these target reliabilities are meant for system level failures, whereas EN 1992-4:2018 design offers member level analysis.

Table 1: Return period, reliability index and annual probability of exceedance for buildings, after EN 1998-1-1:2024 and EN 1998-1-2:2023.

Limit State	**Consequence Class**			
	CC1	**CC2**	**CC3-a**	**CC3-b**
	[Return period, T (years) / Reliability index, β_{50} (-) / Annual probability of exceedance, p_1 (-)]			
NC	600 / 1.75 / $8.2 \cdot 10^{-4}$	1600 / 2.33 / **$2.0 \cdot 10^{-4}$**	2500 / 2.56 / $1.0 \cdot 10^{-4}$	5000 / 2.91 / $0.4 \cdot 10^{-4}$
SD	275 / 1.20 / $24.4 \cdot 10^{-4}$	475 / 1.60 / $11.3 \cdot 10^{-4}$	600 / 1.76 / $8.0 \cdot 10^{-4}$	900 / 2.0 / $4.6 \cdot 10^{-4}$
DL	100 / 0.38 / $87.0 \cdot 10^{-4}$	115 / 0.50 / $73.5 \cdot 10^{-4}$	125 / 0.55 / $68.6 \cdot 10^{-4}$	140 / 0.63 / $61.7 \cdot 10^{-4}$

Note: The relationship between the annual probability of exceedance, p_1 and the probability of exceedance for a 50-year reference period, p_{50} can be expressed as $p_{50} = 1 - (1 - p_1)^{50}$ and the relationship between the reliability indices for 50-year reference period (β_{50}) and for 1-year reference period (β_1) is indicated in Figure 1. The reliability index (β), in general, is a substitute for the probability of failure (p_f), and can be expressed as $\beta = -\Phi^{-1}(p_f)$, where Φ^{-1} is the inverse standardized normal distribution.

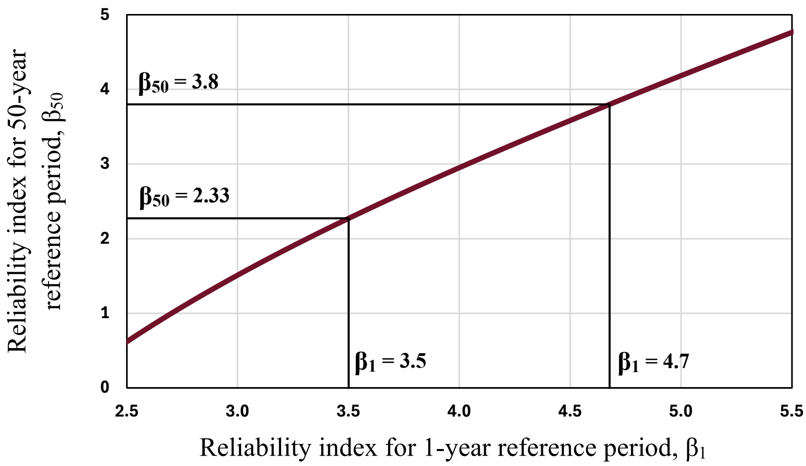

Figure 1: Relationship between reliability indices for 50-year reference period (β_{50}) and for 1-year reference period (β_1)

It is recalled that NZS 1170.5:2004 Section 8.7 generally considers post-installed mechanical or adhesive anchors to be non-ductile. Brittle failures on the member level have in many situations more serious consequences for the structural system integrity than ductile failure. This might need the use of a higher Consequence Class and higher reliability targets for brittle failure modes when the structure cannot be forced into ductile modes such as that in capacity design for seismic design situations. Target reliabilities are used for designing individual members (including connections) for separate failure modes, in most practical applications. The consequences, on the other hand, and as mentioned above, are usually related to system failures and the system failure probability may be the result of contribution of a set of member failure modes. Apparently, engineering judgement is needed in the seismic design of post-installed anchors in accordance with the Eurocodes since the safety factors set by EN 1992-4:2018 may lead to a connection ULS reliability of β_{50} = 3.8 for a 50-year reference period and safety factors for any other reliability level are not given in the Eurocodes. It is noted that the reliability level can be specified using different reference periods, which may not necessarily coincide with the design working life, especially in the case of post-installed connection design in existing structures. Eurocodes generally provide provisions for the design of new structures. When reliability is verified using a shorter reference period than the design working life in the case of existing structures, then the partial factor γ_R for a material property R should generally be greater than in the case when the reference period is taken equal to the design working life. It is also noted that the partial factor γ_R for a material property R generally increases with increasing reliability index. Annex C (Informative) of EN 1990:2023 provides guidance for determining partial factors by the so-called 'design value format method' for subjectively chosen reliability indices. It is noted that the design value format method should include model uncertainties too, which are not widely known for post-installed anchor failure models.

Target reliability indices vary from country to country and can be defined for individual structural members or components. The acceptable value of target reliability indices depends on the type of failure, the cost of failure, the cost of enhanced safety and the existing safety level. Clause C2.3.2.2 of NZS 3101:2006 explains that the design strength of a member, as used in NZS 3101, is the nominal strength calculated in accordance with the provisions and assumptions stipulated in NZS 3101 multiplied by a strength reduction factor, ϕ as detailed in Clause 2.3.2.2 of NZS 3101:2006 for the ultimate limit state (ULS) and 2.6.3.2 for serviceability limit state (SLS) load combinations involving seismic forces. The rules for computing the nominal strength of a member are based on chosen limits of stress, strain, cracking or crushing, and conform to research data for each type of structural action and to established structural engineering practice. Clause C2.3.2.2 of NZS 3101:2006 further explains that the basis for the selected values of strength reduction factors are detailed in the study by MacGregor (1983), which ascertained that for the values of ϕ similar to those in Clause C2.3.2.2 of NZS 3101:2006 and load factors corresponding to AS/NZS 1170:2002, the target values of the reliability index (that is called as safety index (β) in NZS 3101:2006) of 3.0 for dead and live load, 2.5 for dead and live and wind forces and 2.0 for dead and live and earthquake forces applied. It is added that these values for the safety index (β) are within the range implicit in AS/NZS 1170:2002. The length of reference period corresponding to the given target reliability indices is not detailed in NZS 3101:2006, however, it is noted that the reliability indices in those early studies (e.g. the paper by MacGregor (1983), based on the NBS Special Publication 577, June 1980) were determined for structural members based on a service period of 50 years. It can be concluded that the safety level of reinforced concrete design in accordance with NZS 3101:2006 and the AS/NZS 1170 series is different from that of the Eurocodes.

Both EN 1992-4:2018 and Annex G of EN 1998-1-1:2024 defines two types of connections: type A) connections between primary and/or secondary seismic members, and type B) connections for attachments of non-structural elements. The selection of the relevant seismic performance category of anchors, "C1" or "C2", in EN 1992-4:2018 is based on seismicity level defined by the design peak ground acceleration of the actual design location together with the aforementioned Importance Classes as per EN 1998-1:2004. The selection of the relevant seismic performance category of anchors, "SPR1" (= "C1") or "SPR2" (= "C2"), in EN 1998-1-1:2024 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 1998-1-1:2024, the selection of seismic performance category is detailed for different combinations of connection types (A or B) and design options. 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 1992-4:2018 too, but with a different nomenclature.

The seismic capacity of post-installed anchors is determined after the selection of the relevant seismic performance category, and the characteristic seismic resistances of anchors must be taken from a European Technical Product Specification, ETPS (e.g. a European Technical Assessment document, ETA, listed by the

European Organisation for Technical Assessment, EOTA). EN 1992-4:2018 recognizes the two different seismic performance categories for post-installed anchors, referred as "C1" and "C2", respectively ("SPR1" and "SPR2", respectively, in the parlance of EN 1998-1-1:2024; see above), based on the nomenclature followed in EOTA documents (e.g. European Assessment Document EAD 330232 for mechanical post-installed anchors or EAD 330499 for adhesive post-installed anchors). Performance category "C1" currently does not apply to New Zealand in accordance with EN 1992-4:2018. It must be emphasised that the "C1" performance category test methods have been proven to be inferior for seismic qualification and assessment, demonstrated by multiple scholars in the literature (Silva, 2001; Hoehler and Eligehausen, 2008; Mahrenholtz, 2012; Mahrenholtz and Wood, 2020), therefore, it is not discussed any further in this paper. It is noted that in April 2023, ACI Committee 355 decided to replace the existing simulated seismic testing protocols (known in New Zealand as "C1") with the more realistic tests described in EOTA TR 049, known in New Zealand as "C2". These changes have been implemented in the ACI CODE-355.2-24 and ACI CODE-355.4-24. It is also noted that ACI 318 is not expected to cite these changes earlier than its 2028 revision. ACI CODE-318-25 currently cites ACI CODE-355.2-19 and ACI CODE-355.4-11.

As of 2025, NZS 3101:2006 Clause 17.5.5 references documents for seismic prequalification and seismic design of post-installed anchors, which are not state-of-the-art. The upcoming revision of NZS 3101 should address this situation.

1.2 The challenge

A research collaboration was started in the 2000's between the University of California San Diego and the University of Stuttgart, initiated and funded by an anchor manufacturer. Results of the research (Wood et al, 2010) provided input for the development of the unified protocols (Mahrenholtz, 2012), which have been adopted in the early 2010's in EOTA documents known as the "C2" seismic performance qualification and assessment. The research was targeted to studying the experimental performance of post-installed anchors and generating more realistic recommendations for seismic qualification than those available in the 2000's. The "C2" seismic performance qualification and assessment procedure was not developed with the aim of providing anchor capacity information for general seismic design and, therefore, it cannot offer a basis for performance-based seismic design of post-installed anchors.

It is demonstrated in the literature (Borosnyoi-Crawley, 2024a; 2024b; 2024c; 2024d; 2024e) that, as a consequence of the above, the seismic qualification methods in EOTA documents, i.e. TR 049, EAD 330232 or EAD 330499 and the seismic design methods in EN 1992-4:2018 are not connected to each other. The challenge is two-fold.

On one hand, the work conducted by Wood et al (2010) and the unified protocols developed from its findings by Mahrenholtz (2012), which are the backbone of the EOTA assessments, cannot be formally demonstrated to conform with the reliability management assumptions of the Eurocodes, particularly with the claim of EN 1992-4:2018 Clause 4.1(2) about the β-value of 3.8 for connection design (Borosnyoi-Crawley, 2024a; 2024c). The theoretical considerations in the development of the unified protocols were established for non-structural components installed just outside the critical zones in beams of 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 (Wood et al, 2010). Also, the 0.30 mm, 0.50 mm and 0.80 mm crack widths in the unified protocols have been arbitrarily chosen without experimental evidence or well-established theoretical considerations (Borosnyoi-Crawley, 2024a; 2024b; 2024c).

On the other hand, the "C2" characteristic seismic capacity of an anchor in accordance with the EOTA assessment is defined as its characteristic *static capacity* in cracked concrete (with static crack width of w = 0.5 mm), in some cases multiplied with the ratio of an achievable maximum testing load related to the original maximum load defined in the unified protocols (Borosnyoi-Crawley, 2024c; 2024d). Such pass-or-fail criterion in the post-installed anchor seismic qualification and assessment does not provide usable information for performance-based seismic design of post-installed anchors. The "C2" characteristic seismic capacity of anchors is not related to actual earthquake responses in seismic design scenarios (Borosnyoi-Crawley, 2024c; 2024d). Seismic events may impose a wide range of actual crack widths and a wide range of actual number of crack opening-closing cycles acting on post-installed anchors. Consequently, anchor seismic capacity needed for performance-based design cannot be a single number as it is claimed by the EOTA "C2" qualification and assessment method.

1.3 Some words about ductile connection design

The vast majority of the seismic "C2" approved post-installed anchors fail by pull-out failure during the assessment testing, or in some rare cases by concrete cone breakout failure, which are both brittle failure modes. Certain, high bond adhesive anchors may reach steel failure at unrealistically deep embedment depths or with the use of low strength threaded inserts that may provide a less brittle failure, nevertheless, such a failure mode also does not satisfy the EN 1992-4:2018 requirements for the ductility of post-installed anchors.

In accordance with EN 1992-4:2018 Clause 9.2(3)b, the design option with requirements on the ductility of the post-installed anchors (that is applicable only for the tension component of the load) should not be used for connecting primary seismic members (as defined in EN 1998-1:2004) due to the possible large non-recoverable displacements of the anchors that may be expected. The anchors should not be accounted for energy dissipation in the global structural analysis or in the analysis of a non-structural element. The contribution of the anchors to the energy dissipation capacity of the structure (EN 1998-1:2004 Clause 4.2.2) is not addressed in EN 1992-4:2018. It is noted however, that the anchors may be accounted for energy dissipation if proper justification is provided e.g. by a nonlinear time history (dynamic) analysis (according to EN 1998-1:2004) and the hysteretic behaviour of the anchor is taken from an appropriate assessment document. Currently, no such post-installed anchor product is available on the market since the EAD 330232 and EAD 330499 test protocols cannot provide the required anchor performance information.

Although the seismic design philosophy for reinforced concrete structures relies heavily on the ductility and nonlinear performance of the structure, the current seismic assessment and design of post-installed anchors is primarily force-based, with only an indirect check of displacements performed in certain cases (Sharma, 2019). Once the structure goes beyond the elastic response, forces acting on the connections (and on the anchors) become less relevant while the displacement demand and the hysteretic behaviour become more important. As the anchor itself goes beyond the elastic response, a force-based assessment method and design becomes less informative than desirable (Stehle and Sharma, 2019). The EAD 330232 and EAD 330499 load cycling "C2" protocol was critically analysed in the literature and it was demonstrated that the initial 75 cycles only cover a rather small part of the load-displacement curve of the anchors, and in fact, the obtained envelope curves are almost identical to the static load-displacement curves, consequently, no additional information can be obtained from the unified load cycling test protocol compared to the static reference tests in cracked concrete (Stehle and Sharma, 2020). The authors in Stehle and Sharma (2020) proposed a displacement-controlled cycling protocol as an extension or rather a replacement for the "C2" tension load cycling protocol of EAD 330232 and EAD 330499 that is apparently able to capture the hysteretic behaviour over the entire range, stiffness and strength degradation and residual displacement at each cycle. This alternative cycling protocol has great potential in providing post-elastic response characteristics and more refinement of the proposal would be worthwhile. Further research is needed in this field, to be able to develop appropriate testing methods to satisfy the seismic design provisions of EN 1992-4:2018 Clause 9.2(3)b for ductile anchors in the future.

It is noted that Annex C of EN 1992-4:2018 stipulates further requirements for the design of post-installed anchors according to Clause 9.2(3)b. Inter alia there is a requirement for a free stretch length of 8d that cannot be met by any seismic approved post-installed anchor product currently available on the market. It is noted that research and product development is ongoing with ductile undercut anchors that meet the requirement for a free stretch length of 8d (Mahrenholtz and Borosnyoi-Crawley, 2024).

The behaviour of post-installed anchors under crack cycling is especially important for non-structural connections both for design without requirements on the ductility of the anchors (e.g. EN 1992-4:2018 Clause 9.2(3)a) and design with requirements on the ductility of the anchors (e.g. EN 1992-4:2018 Clause 9.2(3)b) (Muciaccia, 2017). Currently, EN 1992-4:2018 requires that post-installed anchors for ductile design in accordance with Clause 9.2(3)b shall have a European Technical Product Specification that includes a qualification for performance category "C2". This requirement needs revision. It is noted that it is intended to provide loading protocols for cyclic testing and acceptance criteria for structural members and components in the future in CEN/TS 1998-1-101 that is an unpublished Draft as of September 2024, waiting for approval and public commenting.

2 THE PROPOSED HOLISTIC APPROACH

The seismic capacity of post-installed anchors depends on the actual damage of the concrete-anchor system due to an earthquake event. 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; however, in a simplified way, this damage of the concrete-anchor system may be characterized by the actual width of the crack formed at the location of the anchor and the actual number of crack opening-closing cycles acting on the anchor together with the anchor load that reflects the actual earthquake scenario. The crack anatomy depends on several factors, 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.

A holistic approach has been proposed for the seismic behaviour of post-installed anchors (Borosnyoi-Crawley, 2024c), which hypothesises that seismic design scenarios may be characterized with a single, simplified parameter that considers the actual damage of the concrete-anchor system and the load demand on the connection, which parameter may be the basis of connection design and anchor qualification and assessment. The most appropriate parameter for such a purpose is yet to be determined. As a potential candidate, the Accumulated Damage Potential (ADP) is suggested that was proposed by Mahrenholtz (2012) and by Mahrenholtz and Eligehausen (2016). 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 (Mahrenholtz, 2012; Mahrenholtz and Eligehausen, 2016), noted that additional tests should be performed in the future to validate the theory:

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

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 (2024c) compared the accumulation of ADP per cycles with actual fastener displacement accumulation observed during laboratory experiments when the EOTA unified crack cycling protocol was applied on either bolt-type expansion anchors (Mahrenholtz et al, 2012) or on cast-in headed studs (Mahrenholtz, 2012). 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 (Borosnyoi-Crawley, 2024c). Further research is needed in this field, however, at the current stage of the development of the holistic approach, the ADP is provisionally proposed as the single parameter representing seismic design scenarios for connection design with post-installed anchors.

The general structure of the holistic approach for the connection of non-structural components in buildings is illustrated in Figure 2. Five different design inputs are considered: 1) The assumed consequences of failure, represented by e.g. the Consequence Class of a building; 2) The geographic location of the building; 3) The structural characteristics of the building; 4) The location of the non-structural component in a structural member; 5) The seismic mass and period of the non-structural component.

The flowchart of the holistic approach reveals in a visual perception the shortcomings of the EOTA "C2" seismic qualification and assessment method for performance-based design and highlights the elements of design input necessary for an anchor qualification framework that may fit to the purposes of performance-based design. It can be observed that any consequence of failure can be considered resulting in any return period (in agreement with the ideology of Table 1), and the assumptions should not be fixed to a 475-year return period as those in the EOTA "C2" approach after Wood et al (2010). It can be observed that any geographic location can be considered, and the assumptions should not be fixed to Charter Oak, LA County, California, USA, as those in the EOTA "C2" approach after Wood et al (2010). From a structural perspective, earthquakes are rapidly varying support motions, rather than applied loads. The motion of the buildings (their response) is influenced by the dynamic characteristics, e.g. mass and stiffness, together with strength and ductility. The relationship between the building response and its natural period of vibration and damping ratio can be described by the acceleration spectrum and the displacement spectrum, for each earthquake event. Building responses can be broken down to element level responses, such as floor acceleration response, beam-column joint curvature response or inter-storey drift response. The crack anatomy depends on the exact location of the non-structural component in the actual structural element (beam, wall, slab, column) and the assumptions should not be fixed to locations just outside the critical region in a beam of specific moment-resisting frames as those in the EOTA "C2" approach after Wood et al (2010). Earthquake events may impose a wide range of actual crack widths and a wide range of actual number of crack opening-closing cycles acting on the anchors. The crack cycles depend on the dominant period(s) of the building response to the earthquake (Watkins and Hutchinson, 2011; Mahrenholtz et al, 2012). A set of crack cycling protocols (as being the most demanding seismic performance test) is needed to cover an acceptable range of applications and the assumptions should not be fixed to one single crack cycling protocol that considers a limited set of design scenarios (prototype buildings) as those in the EOTA "C2" approach after Wood et al (2010). The fundamental period and the seismic mass of the non-structural component together with the type of the post-installed anchor determines if the component acceleration is an amplification or an attenuation of the floor acceleration. The load cycles depend on the oscillating behaviour of the non-structural component that is independent from the floor motion (Watkins and Hutchinson, 2011; Mahrenholtz et al, 2012). The load demands must reflect the component characteristics more realistically than those set in the EOTA "C2" approach based on Mahrenholtz (2012), since it was found experimentally that anchor loads in non-structural component tests are considerably lower than the loads applied in the EOTA "C2" protocols (Mahrenholtz et al, 2012).

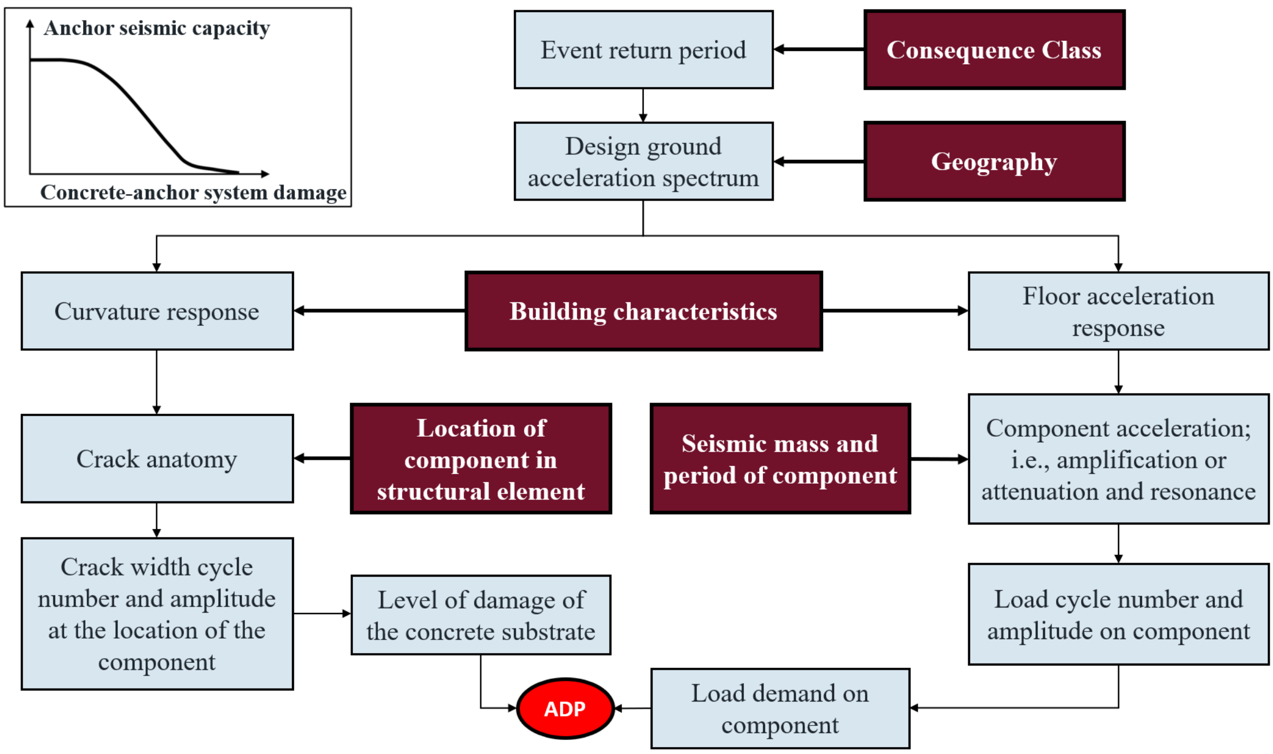

Figure 2: Holistic framework for post-installed anchor seismic performance (Borosnyoi-Crawley, 2024c)

Open questions: 1) It is yet to be determined what the appropriate concrete-anchor system damage parameter (other than the ADP) for the basis of connection seismic design and anchor qualification and assessment is; 2) It is yet to be determined what the appropriate excitation (crack or load) cycle numbers in the test protocols for New Zealand site locations and Importance Levels are, for building designs typical in New Zealand; 3) It is yet to be determined what the appropriate assumption of maximum design crack widths for different structural members and design situations are; 4) It is yet to be determined what the suitable load demand on the anchors for different design situations are.

Borosnyoi-Crawley (2024c) initiated an open discussion about the proposed holistic approach. It was demonstrated through a numerical exercise that different site locations across New Zealand (where Auckland, Christchurch, Wellington and Masterton were used as examples) would need different crack cycling test protocols for the same set of design scenarios, to cover the different seismic demands expressed by the design target acceleration spectra generated in accordance with DZ TS 1170.5:2024 for the selected four New Zealand locations, corresponding to the same Importance Level. The highest cycle number would correspond to Masterton, and the lowest cycle number would correspond to Auckland. Borosnyoi-Crawley (2024c) calculated the ADP values for different combinations of the selected New Zealand locations and maximum crack widths considered. The maximum load demand (constant tension load, N_w) applied on anchors in these hypothetical crack cycling tests was set as specific fractions of the mean ultimate static pull-out capacity of anchors, $N_{u,m,cr}$ (in accordance with the ideology of the EOTA unified crack cycling protocol) corresponding to the actual maximum crack width considered. It was found, as expected, that the ADP is not constant for a given geographic location if the targeted load demand is kept as a constant fraction of $N_{u,m,cr}$ for increasing maximum crack widths considered. Such scenarios would correspond to a wide range of concrete-anchor system damage, representing different locations of the components in different structural members in different structural systems, as selected by the designer. A representative illustration of the calculated ADP values is reproduced in Figure 3 (Borosnyoi-Crawley, 2024c). The maximum crack width considered was changed over the range of 0.3 mm to 1.5 mm, in accordance with the hypothesis for the compensation of the model error of the NZS 3101:2006 crack width prediction model, introduced in (Borosnyoi-Crawley, 2024c). It was demonstrated that NZS 3101:2006 is not robust in the estimation of crack widths, and the formulae given in NZS 3101:2006 Clause 2.4.4 underestimate the actual crack widths observed experimentally, when those are larger than w = 0.3 mm (Borosnyoi-Crawley, 2024b). Maximum crack widths considered for seismic scenarios in crack cycling protocols can be related to the tensile stress calculated for the steel reinforcement in accordance with NZS 3101:2006. Maximum crack width of 0.3 mm considered in crack cycling protocols would correspond to 150 MPa tensile stress calculated for the steel reinforcement, while 1.5 mm crack width would correspond to 500 MPa tensile stress calculated for the steel reinforcement in a seismic design scenario (Borosnyoi-Crawley, 2024c). Limitations of this paper does not allow a detailed discussion; readers are advised to familiarize themselves with the content of Borosnyoi-Crawley (2024c).

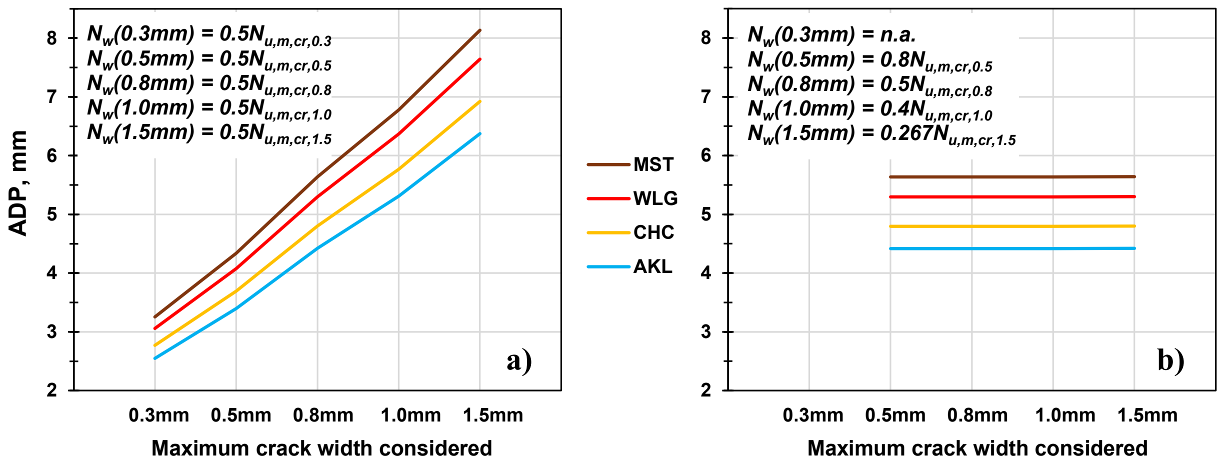

Figure 3: Calculated ADP values for ULS design scenarios at maximum crack widths considered as 0.3, 0.5, 0.8, 1.0 and 1.5 mm, where applicable (Borosnyoi-Crawley, 2024c)

a) Post-installed mechanical anchors, $N_{w,j} = 0.5 \cdot N_{u,m,cr,j}$

b) Post-installed mechanical anchors, constant ADP for each location

Note: AKL – Auckland, CHC – Christchurch, WLG – Wellington, MST – Masterton

It is noted that the ADP generates a displacement-like measure that is considered to be related to the axial displacement of an anchor during a crack cycling experiment (Mahrenholtz, 2012), as it is schematically illustrated in Figure 4. If deformations (displacements or rotations) are relevant for the design of the connection, it should be demonstrated that these deformations can be accommodated by the anchors (see Clause C.6 in EN 1992-4:2018 or Clause G.5 in EN 1998-1-1:2024). In a number of cases, the acceptable anchor displacement associated with a rigid support condition is considered to be in the range of 3 mm. It may be reasonable, therefore, to limit the level of the concrete-anchor system damage, i.e. the ADP to an agreed displacement limit. In such situations, as it is illustrated in Figure 3(b), the maximum load demand (constant tension load, N_w) applied on anchors in crack cycling tests cannot be kept as a constant fraction of $N_{u,m,cr}$ for all maximum crack widths considered, but need to be adjusted accordingly.

It is noted that anchor seismic displacements available from the EOTA "C2" qualification and assessment method (and published in e.g. the ETA documents) cannot be used for performance-based design, since those anchor displacements are not related to actual earthquake responses in seismic design scenarios but results of the incremental displacement accumulation during the crack cycling tests performed by applying the unified crack cycling protocol of the EOTA "C2" approach.

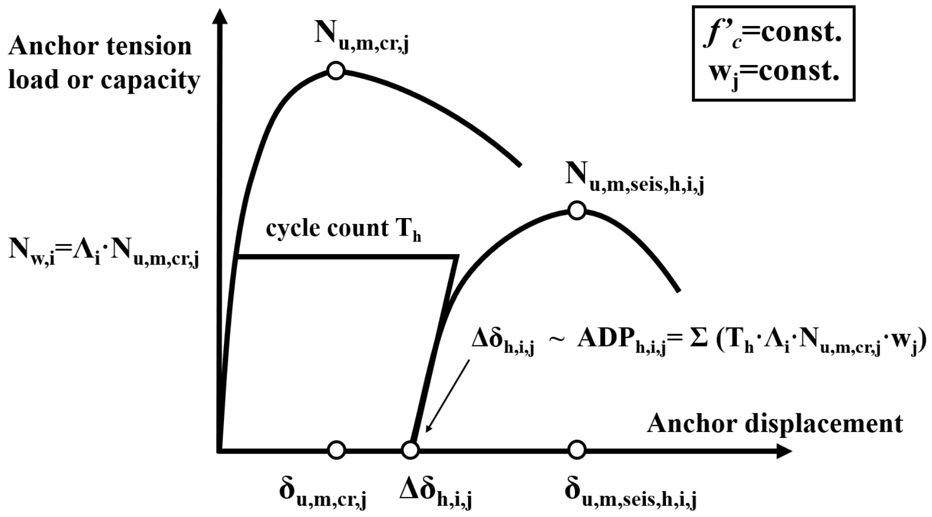

Figure 4: Schematic load-displacement curves indicating the change in the static pullout behaviour and tension capacity after crack cycling tests for a combination of crack cycling number T_h, anchor tension load during crack cycling $N_{w,i}$ and maximum considered crack width w_j for a constant f'_c specified concrete strength. Note: $N_{u,m}$ and $\delta_{u,m}$ denotes mean ultimate tension capacity and displacement, respectively

3 FUTURE WORK

Open questions have already been indicated in the previous section. Such developments call for urgency since the current state-of-the-art seismic anchor qualification method fails to provide anchor capacities needed for reliable, safe and economic performance-based seismic design of post-installed anchors. In a New Zealand context, the seismic design of post-installed anchors may be either overly conservative (e.g. in Auckland) or grossly unsafe (e.g. in Wellington) if the seismic capacity of anchors is taken from a European Technical Product Specification that is based on the EOTA "C2" qualification and assessment method (Borosnyoi-Crawley, 2024a; 2024c; 2024d). Performance-based seismic design would need anchor seismic capacities determined for a virtually infinite number of combinations of crack cycling numbers, anchor loads, maximum considered crack widths and concrete strengths for every single post-installed anchor product for all anchor diameters. Future work is necessary to find a practical way to simplify this need that does not lead to unreasonable sophistication and can result in a practically feasible anchor qualification framework. As it was demonstrated (Borosnyoi-Crawley, 2024a; 2024c; 2024d), the EOTA "C2" qualification and assessment method may need extension or complete re-definition, and in its current form cannot provide appropriate anchor capacity information for connection seismic design in New Zealand.

4 CONCLUSIONS

The seismic qualification and assessment methods of post-installed anchors in the current international state-of-the-art was developed before 2013 and have been extensively used since then, in e.g. European Technical Assessment (ETA) documents. These methods have not been further developed during the last ten years. Multiple gaps can be identified in these methods that create strong limitation for their universal use and makes their application in performance-based seismic design impossible. Seismic design of post-installed anchors is disconnected from the current state-of-the-art seismic qualification and assessment methods. In accordance with the New Zealand Building Code (NZBC) B1 AS/VM (1st Ed., Amd 21, Ver. 2023-11-02), these methods are currently considered as NZBC B1 Verification Method, and therefore designers in New Zealand are expected to perform the seismic design of post-installed anchors accordingly. Seismic design of post-installed anchors in New Zealand may be either overly conservative or grossly unsafe if the seismic capacity of anchors is taken from a European Technical Product Specification that is based on the EOTA "C2" qualification and assessment method and the EOTA "C1" seismic performance category does not apply to New Zealand. To overcome these challenges, a holistic approach has been proposed for the seismic behaviour of post-installed anchors, which hypothesises that seismic design scenarios may be characterized with a single, simplified parameter that considers the actual damage of the concrete-anchor system and the load demand on the connection, which parameter may be the basis of connection design and anchor qualification and assessment. It was demonstrated that anchor seismic demand and anchor seismic capacity as continuums may be better tackled within the proposed framework. It was demonstrated that the holistic approach is fully transparent and provides the possibility to identify key driving parameters for further verification. In the relationship between seismic design and seismic qualification of post-installed anchors, without a doubt, the design must be the driving force. Design determines the anchor demand, and the qualification procedures must provide appropriate proof for safe anchor capacity – in a transparent way and correctly addressing the design needs. It is urged that the New Zealand structural and earthquake engineering community acknowledges the current gap in the seismic design and seismic qualification of post-installed anchors and initiate discussions at the appropriate forums.

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