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Abstract: This paper presents the technical background for the seismic qualification procedures for post-installed anchors in the European Technical Approval Guideline (ETAG 001) seismic annex issued in 2013. We discuss requirements for a comprehensive guideline and reference supporting documentation. Numerical studies to generate new simulated seismic protocols for anchors are summarized with focus on their application to Europe. To reduce the time and cost of anchor product qualification testing while fulfilling the requirement of European building codes to assess two performance categories, we combine the results of our numerical studies to generate novel testing protocols that allow for the assessment of anchor behavior at multiple levels in a unified protocol. Validation tests demonstrate that the unified protocol results in anchor performance comparable with that achieved in multiple, single-performance-level tests.

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Development and validation of European guidelines for seismic qualification of post-installed anchors

Highlights

A. Technical background for the seismic amendment of the European Technical Approval Guideline (ETAG 001 Annex E) are given.

B. Numerical studies to generate new simulated seismic protocols for anchors are summarized.

C. Experimental tests demonstrate that the proposed protocol is capable to evaluate seismic anchor performance.

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Development and validation of European guidelines for seismic qualification of post-installed anchors

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1. Introduction

Many parts of the world are subject to earthquakes and Europe is no exception. In addition to direct losses from damaged infrastructure and the unfortunate human losses, indirect losses resulting from disruption of operations can have lasting impacts on a community following an earthquake. This risk depends not only on the magnitude of the seismic hazard, but also on the vulnerability of the built environment. Historically, inadequate anchorage to concrete, in particular of nonstructural components and systems, has been identified as a significant contributor to direct and indirect losses during earthquakes [e.g., [1]-[5]]. Proper seismic anchorage requires (1) the availably of and adherence to sound seismic anchorage design provisions and (2) anchor products qualified to remain functioning under seismic conditions. In this paper, the focus is on the latter of these requirements.

In the course of European harmonization of building codes and standards, national level documents for post-installed anchors were replaced by a European Technical Approval Guideline (ETAG) beginning in 1997 [6]. However, prior to the release of ETAG 001 Annex E in 2013 [7], the scope of the guideline did not include seismic applications. This new Annex E includes two performance categories for anchors (denoted as C1 and C2). Performance category C1 provides anchor capacities in terms of strength (force), while performance category C2 provides anchor capacities in terms of both strength and deformation (displacement). The category C1 testing procedures and assessment criteria closely resemble those currently used in the United States [American Concrete Institute (ACI) 355.2 [8], ACI 355.4 [9]]. While the C2 anchor performance category, which is more demanding, is required for applications in safety-critical infrastructure (higher building importance class) or when increased seismic demands are anticipated.

This paper outlines the technical basis for the test procedures in the C2 performance category. We briefly summarize the history of developments in seismic anchorage qualification guidelines and the technical requirements for a comprehensive seismic anchorage qualification. We then synthesize our numerical and experimental investigations to develop a new seismic anchorage testing protocol and discuss its applicability to Europe, as well as the unification of multi-level demands (*serviceability* and *suitably*) for use in ETAG 001 Annex E [7]. Finally, we present the results of exploratory tests to validate the equivalence of post-installed anchor performance tested using the unified protocol and multiple, single-performance-level protocols.

2. Brief history of seismic anchorage qualification

In the United States, prior to 1997, qualification of post-installed anchors for seismic performance was not common practice outside of the nuclear and telecommunications industries [10]. At that time, post-installed anchors were routinely listed by the International Conference of Building Officials Evaluation Service (ICBO ES) as suitable for seismic conditions based on static tests in uncracked concrete. Anchor connection failures observed during the 1994 Northridge Earthquake in California prompted a review of this practice and between 1995 and 1997, mechanical post-installed anchors were not permitted for use in seismic applications in the United States. Testing and acceptance criteria [AC01 [11], AC58 [12]] based loosely on the Canadian standard CAN/CSA-N287.2 [13] were adopted by the ICBO ES in 1997 and listing of mechanical anchors for seismic loading resumed in 1998 [10]. As an alternate means of qualification for seismic loading, the ICBO ES adopted load cycling tests developed by the Structural Engineers Association of Southern California (SEAOSC) [14]. Both these tests (CAN/CSA-N287.2 and SEAOSC) are performed in uncracked concrete, which are less demanding then tests in cracked concrete.

Test programs and evaluation requirements for post-installed mechanical anchors in cracked concrete were introduced in the United States in 2001 in ACI 355.2 [15]. Shortly thereafter, the ICC-ES[2] developed new acceptance criteria AC193 [16] for mechanical anchors based on ACI 355.2. Subsequently, ACI and ICC-ES extended these criteria to include adhesive anchors in cracked concrete [ACI 355.4 [9], AC308 [17]]. It is worth noting that ACI 355.2 and ACI 355.4 are based on ETAG 001 [6] with the exception of the *simulated seismic tests*, which did not exist in the European standard prior to 2013. The testing procedures in ACI 355, AC193 and AC308 are now largely harmonized. The seismic testing procedures and acceptance criteria in these documents include tension and shear load cycling in a static crack. They are based on the state-of-the-art as practiced in 2001 and continue today to serve as the basis for issuing post-installed anchor approvals in the United States.

Parallel to developments in the United States, in Germany, the *Deutsches Institut für Bautechnik* (DIBt) issued a guideline for the use and testing of post-installed anchors in German nuclear facilities (DIBt KKW Guideline [18]). This guideline is applicable to anchors used to attach safety-relevant components under extreme loading conditions such as an earthquake, explosion or aircraft impact. The guideline requires tension and shear load cycling in a static crack as well as tests of the anchor in large crack opening and closing cycles, which in this guideline is called *crack movement tests*.

Important load cycling parameters and assessment criteria for the above-mentioned seismic anchorage qualifications are summarized in Table 1. It is notable that in current standards used in the United States, anchor performance is evaluated in a crack width (w) of 0.5 mm. This crack width is also used to evaluate anchor performance

[2] In 2002, the three major model code bodies the Unites States - including the ICBO - merged under the umbrella of the International Code Council (ICC). For this reason, in this paper ICBO documents reaffirmed subsequent to 2002 are referenced hereafter as ICC-ES documents.

for non-seismic applications; i.e., the crack width represents service conditions, rather than seismic conditions. In addition to requiring that anchor performance be verified in 0.5 mm wide cracks, the DIBt KKW Guideline [18] also verifies performance in 1.0 mm and 1.5 mm wide cracks. These large crack widths assume that anchors are located where the reinforcement in the concrete has undergone a strain of 0.5 % (yielding). Load cycling may be at a constant load level followed by monotonic loading to failure, stepwise-decreasing load amplitudes followed by monotonic loading to failure or stepwise-increasing load amplitudes until failure occurs. The number of load cycles varies significantly between the standards (from 15 to 340 total cycles) as do the target load values (load factors) applied in tension (N) or shear (V) at each level of cycling.

Table 1. Simulated seismic test parameters and assessment criteria.

Standard	Load pattern	Load type[a]	Crack width, mm	No. of load cycles at each step	Load factors at each step	Key assessment criteria
CAN/CSA-N287.2 [13]	Decreasing	PT	-	30/30/80/200	0.53/0.45/0.30/0.15[b]	$N_u \geq N_{s,y}$ [c]
		AS	-	30/30/80/200	±0.16/0.12/0.08/0.04[b]	$V_u \geq V_{s,y}$ [c]
SEAOSC [14]	Increasing	PT	-	5/5/5/5 …	0.25/0.50/0.75/1.0 …[d]	$N_u \geq N_{u,m(bolt)}$ [e]
		AS	-	5/5/5/5 …	±0.25/0.50/0.75/1.0 …[d]	$V_u \geq V_{u,m(bolt)}$ [e]
AC01 [11], *AC58* [12]	Decreasing	PT	-	10/30/100	~ 0.50/0.375/0.25[f]	$N_u \geq 0.8 \cdot N_{u,m}$ [c,g]
		AS	-	10/30/100	~ ±0.50/0.375/0.25[f]	$V_u \geq 0.8 \cdot V_{u,m}$ [c,g]
ACI 355 [[8], [9]], *AC193* [16], *AC308* [17]	Decreasing	PT	0.5	10/30/100	0.50/0.375/0.25[h]	$N_u \geq 0.8 \cdot N_{u,m}$ [c,h]
		AS	0.5	10/30/100	±0.50/0.375/0.25[h]	$V_u \geq 0.8 \cdot V_{u,m}$ [c,h]
DIBt [18]	Constant	PT	1.5	15	~ 0.45[i]	$N_u \geq 0.7 \cdot N_{u,m}$ [c,i]
		AS	1.0	15	~ 0.45[i]	$V_u \geq 0.9 \cdot V_{u,m}$ [c,i]

[a] PT = Pulsating Tension; AS = Alternating Shear.

[b] Factor(s) applied to specified steel yield strength ($N_{s,y}$, $V_{s,y}$).

[c] No failure allowed during load cycling.

[d] Factor(s) applied to "First Major Event", e.g., a stiffness change during reference tests in uncracked concrete.

[e] $N_{u,m(bolt)}$ and $V_{u,m(bolt)}$ are obtained from tests with cast-in bolts.

[f] Factor(s) applied to mean ultimate load ($N_{u,m}$, $V_{u,m}$) in reference tests in uncracked concrete.

[g] Anchor displacement limits during load cycling are imposed.

[h] Factor(s) applied to mean ultimate load ($N_{u,m}$, $V_{u,m}$) in reference tests in cracked concrete (w = 0.3 mm).

[i] Factor(s) applied to mean ultimate load ($N_{u,m}$, $V_{u,m}$) in reference tests in cracked concrete (w = 1.0 mm).

In crack movement tests, an anchor is installed in a closed hairline crack and loaded by a sustained tension load (N_w) that is a fraction of the ultimate strength of the anchor. Crack opening (w_1) and closing (w_2) is typically achieved by applying an external load to the reinforced concrete specimen, which is serving as the anchorage component. In guidelines developed prior to ETAG 001 Annex E, a pulsating tension load is applied to the anchorage component and the initial crack closing width is allowed to increase as cycling progresses (due to the splitting force developed by the anchor and degradation of the reinforcement bond) provided a minimum specified difference w_1 - w_2 is maintained. After completion of the crack cycles, the anchor is loaded in tension to failure in an open crack to establish the residual strength of the anchor. Key crack movement test parameters and assessment criteria are summarized in Table 2. It is important to note that the small crack widths (smaller than 0.3 mm) and large number of cycles (1000) used in the U.S. guidelines are not intended to represent seismic conditions. Only the DIBt KKW Guideline [18] attempts to simulate conditions during an earthquake, however, since the guidelines allows for yielding of the reinforcement steel, which is outside the scope of design provisions in most building codes, they are overly stringent in many cases. Additional details regarding the historical seismic anchorage qualification guidelines can be found in [19].

Table 2. Crack movement test parameters and assessment criteria of guidelines containing seismic provisions.

Standard	Crack width w_1, mm	Crack width w_2, mm	No. of crack cycles	Sustained tension load N_w	Key assessment criteria
ACI 355 [[8], [9]], *AC193* [16], *AC308* [17]	0.3	0.1	1000	$0.23 \cdot N_{u,m}{}^{a}$	$N_u \geq 0.9 \cdot N_{u,m}{}^{a,c,d}$
DIBt [18]	1.5	1.0	10	$0.50 \cdot N_{u,m}{}^{b}$	$N_u \geq 0.7 \cdot N_{u,m}{}^{b,c}$

[a] Mean ultimate tension load ($N_{u,m}$) based on reference tests in cracked concrete (w = 0.3 mm).

[b] Mean ultimate tension load ($N_{u,m}$) based on reference tests in cracked concrete (w = 1.0 mm).

[c] No failure allowed during crack cycling.

[d] Anchor displacement limits of 2.0 mm after 20 cycles and 3.0 mm after 1000 crack cycles are imposed.

3. Requirements for a comprehensive seismic anchorage qualification

Post-installed and cast-in-place anchors are commonly used in construction to secure nonstructural components and systems (NCSs), such as mechanical, electrical and plumbing systems, as well as to connect structural members to concrete. In reinforced concrete structures, the deformation induced during an earthquake result in cracking of concrete beams, columns, walls and floors. These cracks open and close depending on the amplitude, frequency content and duration of the earthquake motion, the efficiency of the soil-structure interface and the dynamic characteristics of the structure. Since the structural system often serves as the base material for concrete anchorages, these cracks will influence anchor performance. Additionally, the structural system filters, and typically amplifies, the earthquake input motion, influencing acceleration demands throughout the structure. Anchors used to attach nonstructural components are loaded according to the component's dynamic characteristics and the acceleration at the point of attachment, resulting in simultaneous cyclic tension and shear forces. In contrast, the loads on anchors used in structural connections are directly governed by the response of the primary structure that is the columns, beams and walls that comprise the gravity and lateral load resistance systems. Although considerations in this section pertain to both cast-in-place and post-installed anchors used in any structural system, the focus is on post-installed anchors used to secure nonstructural components within buildings, because they represent the largest volume of safety-critical anchorage applications and, as discussed later, are conservative with respect to the number of load cycles.

A brute-force approach to qualify anchors for seismic applications would be to require dynamic testing of all anchored systems prior to use. While this is not practical or economically feasible, we mention it because with the increasing prevalence of seismic certification by shake table testing of nonstructural components in many countries, e.g., AC156 [20], the authors encourage consideration of the use of representative concrete anchorage conditions and monitoring of anchorage demands as part of any nonstructural component testing and certification process. This would lead to more reliable nonstructural component installations and increase knowledge about seismic anchorage demands. For typical situations, however, where the application in which the anchor will be used is unknown at the time the anchor undergoes seismic qualification, generic testing provisions that cover a range of service conditions are required. These testing provisions may either be (1) representative of relevant and statistically acceptable characterizations of the conditions to which the anchors will likely be subjected, or (2) surrogate tests that have been deemed to provide acceptable performance. In this paper, and in the provisions adopted for the ETAG 001 C2 performance category, the former approach is used.

As discussed in Section 2 of this paper, previous seismic anchorage qualification guidelines have applied various combinations of tension load cycling, shear load cycling and crack movement tests with diverse values for key parameters. In actuality, during a seismic event, anchors in concrete will simultaneously be subjected to tension and shear load cycling at dynamic rates, while experiencing cyclic crack opening and closing in the anchorage material. Although studies that conclusively demonstrate the equivalence of results from separate tension, shear and crack cycling tests with results obtained from simultaneous combinations of these actions are lacking, there is mounting evidence from full-scale dynamic experiments that separation of actions leads to indicative seismic performance for anchors [[21]-[25]]. In the remainder of this section, we synthesize evidence from numerous investigators that informed the development of the new seismic anchorage testing protocols presented in Section 4.

3.1 Dynamic effects

Although earthquakes are dynamic processes involving inertial effects, none of the existing anchor qualification standards summarized in Section 2 require verification of anchor performance at rapid loading or crack cycling rates. Typical anchor loading rates during seismic events originally proposed by Klingner [26] were experimentally verified by Watkins in shake table tests of anchored nonstructural components [23]. In addition, Hoehler and co-authors [27] evaluated available research on anchors loaded at accelerated rates, together with the results of new investigations, to assess the validity of excluding dynamic loading in anchor qualification. They concluded that loading rates associated with earthquakes, for which rise times to peak load of less than 0.025 seconds may be assumed [19], do not reduce the load capacity of cast-in-place or post-installed anchors.

While increased loading rates on anchors do not negatively affect the strength of anchors, they typically reduce anchor displacements during load cycling [19]. Therefore, restricting the load cycling frequency in anchor prequalification tests to preclude inertial effects, for example, to below 0.5 Hz, produces more consistent displacement results. A similar conclusion and recommendation for restricting load and crack cycling frequencies was proposed by Mahrenholtz [28]. In summary, dynamic effects may conservatively be excluded from anchor qualification testing.

3.2 Load pattern

Structural response during an earthquake produces transient, reversed cyclic response. A large body of research is available on how cyclic time-history response can be represented in loading sequences for experimentation [see for example: [29]-[32]]. Specifically, for anchors, Silva [10] found that headed bolt anchors and undercut anchors tested in tension and shear according to the DIBt, ACI 355, and SEAOSC methods yielded similar allowable design load capacities although the load patterns are quite different (refer to Table 1). Nevertheless, Silva argues that a stepwise-increasing load is preferable because it provides additional information about the anchor stiffness throughout the entire anticipated loading range. Silva’s results were subsequently extended to other anchor types and failure modes for tension and shear load cycling [[33], [34]] and for crack cycling [[35], [36]]. The results of these studies corroborate using stepwise-increasing protocol for seismic anchor qualification.

3.3 Crack widths

The width of a crack in concrete coincident with an anchor can have a profound detrimental influence on the anchor behavior [37]. The variation of crack widths during a seismic event can accelerate this strength degradation [[35], [36]].

Prediction of crack widths in reinforced concrete elements in a structure subjected to seismic loads is complex. Crack widths are a function of numerous parameters; most significantly, the strain at the location of interest, the type of cracking (tension, flexural or shear), as well as the geometric and material parameters of the reinforced concrete elements near the crack location. Over the past decade there has been extensive discussion in the European concrete anchorage community regarding the applicable crack width for seismic qualification tests [[19], [38], [39]]. Although the discussion has been limited to tension and flexural cracking, 0.8 mm is generally accepted as the upper-bound crack width considered to occur at the onset of yielding of the reinforcement just outside the plastic hinge zone. Anchors installed within the expected plastic hinge regions are outside the scope of ETAG 001 and ACI 355. In addition to the technical justifications, the 0.8 mm crack width can be explained as follows. In ETAG 001 and ACI 355 the maximum considered crack width under service load is taken to be 0.5 mm. Crack width increases approximately linearly with increasing reinforcement strain up to the onset of yielding in the reinforcing steel. According to Eurocode 2 [40] and ACI 318 [41], the ratio between yield load and allowable service load is about 1.6, resulting in a crack width of 0.5 mm · 1.6 = 0.8 mm at the onset of yielding.

As a structure responds to an earthquake, cracks in reinforced concrete will not only open, but also can be closed during moment reversals in structural members. Crack closure can significantly affect anchor performance and must be accounted for in representative simulated seismic tests [35]. This behavior can be simulated in seismic anchor qualification tests by applying compressive load to the anchorage component thereby forcing crack widths to near-zero values during cycling. Consideration of this influence is a significant difference to the existing crack movement tests described in Section 2.

3.4 Tension-shear load interaction

Guillet [42] investigates post-installed anchor behavior under combined cyclic tension and shear loads that might occur during an earthquake. He concludes that the tri-linear or quadratic interaction curves typically used for anchors under combined tension and shear static loads may lead to unconservative designs for cycling. Linear interaction curves are recommended to be used for seismic anchor design, in the absence of product-specific cyclic interaction tests.

4. Development of new testing protocols

As presented in Section 3, a comprehensive seismic anchor qualification procedure should include quasi-static tests with stepwise-increasing anchor tension load cycling and shear load cycling in cracked concrete, as well as stepwise-increasing crack cycling tests with representative seismic crack widths. Although existing seismic anchor qualifications include aspects of these features (see Section 2), in this section we present new protocols that address the requirements in a comprehensive way. Our approach, illustrated in Figure 1, involved extensive numerical simulation and experimental testing of installed anchors. In this paper, we provide an overview of the process with details specific to its implementation in ETAG 001 Annex E.

We first designed a suite of seven reinforced concrete buildings with a broad range of dynamic response characteristics and analyzed their response to suite of ground motions intended to envelope design events in high seismic regions of the United States and Europe. To generate protocols, we use Rainflow counting (an algorithm to reduce an arbitrarily varying time series to a set of simple reversals) to reduce the time history response of the buildings into a series of constant amplitude cycles (bins). For crack width protocols, curvature histories of the beam ends are extracted just outside of the plastic hinge zones, where a linear relationship exists between curvature and crack width. These curvature histories are combined on a per building basis and then each building combination binned in ten equal steps normalized by the maximum crack width. Additional details about the crack protocols can be found in [43]. To develop the load protocols, elastic single-degree-of-freedom (SDOF) models with frequencies ranging between 5 Hz and 20 Hz, which covers the majority of mechanical and electrical components in commercial buildings [44], are subjected to the floor level time histories from the building analyses. The acceleration time history responses of the elastic SDOF models are then used to develop the load protocol again using the Rainflow counting algorithm. It is noted that using single-degree-of-freedom system response in lieu of floor motions yields larger cycle amplitudes and different cycle count distributions than the floor motions themselves due to the resonance associated with the frequency ratio of the elastic SDOF to that of the input floor acceleration. Therefore the obtained tension and shear loading protocol developed for nonstructural components are conservative (higher cycle count) for anchors used in structural anchorage applications. Additional details about the load protocol can be found in [45]. In what follows, we expand on the details of the numerical investigation and ensuing data processing steps articulated in Figure 1.

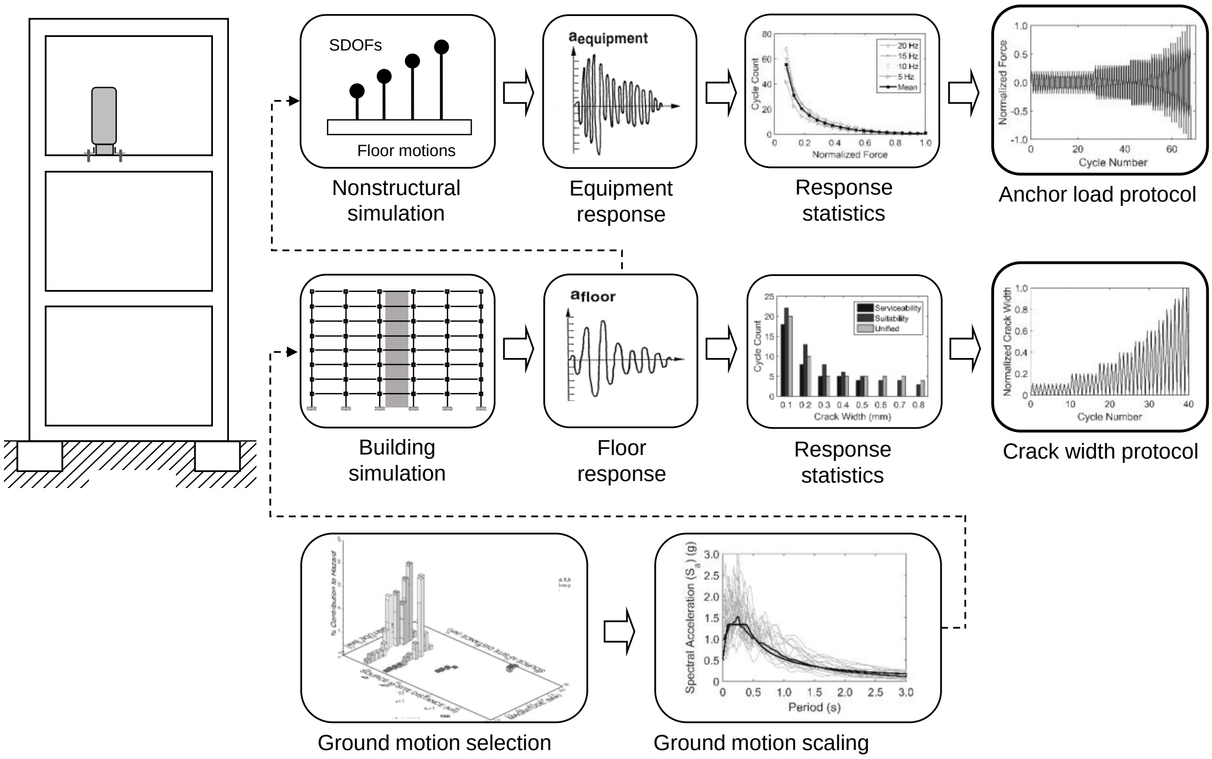


Figure 1. Methodology to generate simulated seismic crack cycling and anchor load cycling protocols.

4.1 Parametric buildings and analytical models

Seven prototypical reinforced concrete buildings were considered in these studies: five special moment resisting frame (SMRF) buildings of 2, 4, 8, 12 and 20 stories and two dual lateral system buildings of 4 and 8 stories comprised of an ordinary moment resisting frame (OMRF) and a structural shear wall. These buildings envelope the period characteristics of common building stock used in high seismic regions of the United States and other countries with well-developed building codes. The prevalence of shorter buildings (fewer than 10 stories) was intended to weight the result statistics toward prevalent building stock in the United States and Europe. Each building had a footprint of 45.7 m by 36.6 m with five bays in each direction, with a longitudinal bay width of 9.1 m, transverse bay width of 7.3 m and story height of 3.7 m. The building designs were in accordance with governing codes in the United States at the time the studies were performed [[41] , [46], and [47]].

Two-dimensional numerical models of the buildings were constructed in the nonlinear finite element software OpenSees [48]. Both constitutive and geometric nonlinearities were considered for all analyses. For the frame elements, inelasticity was concentrated (lumped) at the beam and column ends. This was done to obtain objective curvature responses [49], since the curvature time histories are vital for the crack width protocol. The structural shear walls, in the dual systems, were modeled using spread plasticity elements. Eigenvalue and pushover analyses were first conducted to characterize the numerical models, then nonlinear time history analyses were performed to develop the protocols. Additional details about the design buildings and analytical models can be found in [50].

4.2 Seismic hazard

The suite of buildings was located within the Los Angeles basin (California, USA) and in close proximity to a number of known faults of high seismic activity. A probabilistic seismic hazard analysis (PSHA) was undertaken to estimate the magnitude and source-to-site distance (M, R) bins associated with a hazard representing a probability of exceedance of 10 % in 50 years (design event) [43] at the selected site. Hazard deaggregation, based on the 2002 edition of the National Seismic Hazards Mapping Project models [51], indicated that 98 % of the hazard is associated with seismic sources within 20 km or less and approximately 60 % of the hazard is within the near field (distance of 10 km or less). The spectral acceleration was conservatively estimated at short periods (S_s) and at one second (S_1) as 2.01 g and 0.61 g [51], respectively; for an assumed site class of C (dense soil) as defined by ASCE 7 [47]. Subsequently, the hazard deaggregation is used to guide the selection of ground motion records. Details on the suite of 21 selected motions obtained from the Pacific Earthquake Engineering Research Center Next Generation of Ground-Motion Attenuation Models (PEER-NGA) strong motion database [52], and their subsequent scaling to the design acceleration spectrum over a range of periods, can be found in [45]. We note however that the magnitudes and distance pairs of the selected ground motions represent 94 % of the deaggregated contributions with ground motions of a peak ground acceleration above 0.51 g.

While it is recognized that a variety of seismic conditions exist throughout the world, the significant seismic hazard of the selected site is anticipated to conservatively represent demands at many locations. The applicability of this site to another site of high seismicity in Europe is explored using a second site at Cittanova, Italy. In the latter case, we conduct probabilistic seismic hazard analysis and deaggregation at the site using the Interactive Seismic Hazard Map tools, developed by the *Istituto Nazionale di Geofisica e Vulcanologia and the Dipartimento della Protezione Civile* [53], for a return period of 475 years; which closely represents the design event used in our study. The deaggregation produced distance-magnitude (M_w) bins in intervals of 10 km and 0.5 (unitless). Figure 2 shows that the distance-magnitude pairs and percentage hazard distributions for the two sites are similar for a given distance bin (compare cumulative percentages across bottom) and for magnitudes greater than 6.0 (compare cumulate percentages up left axis). The greater contribution from lower magnitude events at the Cittanova site will produce a conservative result for cycle count (more cycles) compared to the Los Angles site used to develop the protocol due to a reduced count of higher amplitude acceleration excursions. Consequently, one may infer that the protocols developed herein can be applied to sites of similar deaggregated hazard elsewhere in the world.

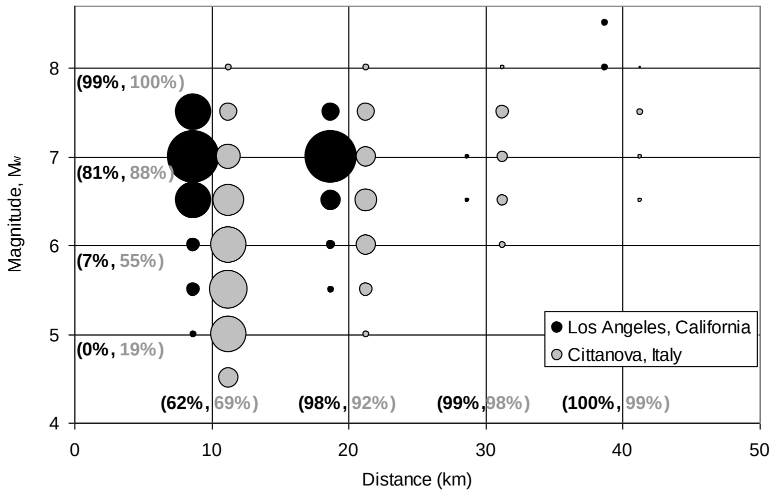


Figure 2. Deaggregation for site locations in Los Angeles, USA and Cittanova, Italy [50].

4.3 Protocols based on the numerical and experimental studies

Table 3 provides the load and crack cycling protocols extracted from the numerical analyses. The cycle count and distribution for both tension and shear load cycling are identical and all bin amplitudes are normalized to the maximum considered value. The arithmetic mean (μ) and the mean plus 1.28 times the standard deviation (σ) represent the 50 % fractile (P50) and 90 % fractile (P90) of the cyclic demand, respectively. The crack cycle counts are extracted for each building and, similarly, the load cycles for each oscillator.

During the development of the P50 and P90 protocols, an extensive suite of experiments with anchors representing most commercially-available mechanical and chemical anchor types was conducted to explore parameter sensitivities. These tests highlight that the crack width and crack cycling is more critical for anchor performance under tension loads than the cycling of the tension load itself. Furthermore, the number of crack cycles in a crack movement test (cumulative cyclic damage), and not just the crack width amplitude, must be accurately represented. Results from these tests are reported in [34] and [36].

Table 3. Load and crack cycle protocols indicating the number of cycles per amplitude step.

Step	P50 (μ)		P90 (μ + 1.28σ)	
	Load	Crack	Load	Crack
0.1	50	10	84	17
0.2	27	7	48	12
0.3	15	5	27	8
0.4	9	4	16	6
0.5	5	3	11	5
0.6	3	3	7	5
0.7	3	2	5	4
0.8	2	2	5	3
0.9	2	2	5	3
1.0	2	2	4	3
Sum	118	40	212	66

4.4 Development of test protocol for ETAG 001

4.4.1 Serviceability and suitability test protocols

ETAG 001 Annex E [7] is linked to European seismic design standards through Eurocode 8 [54]. Eurocode 8 follows a limit state design philosophy, also known as load and resistance factor design, that mandates that for Damage Limitation States (DLS) "an adequate degree of reliability against unacceptable damage shall be ensured by satisfying deformation limits". It also specifies that for Ultimate Limit States (ULS) the behavior of structural or nonstructural elements does not present risks to persons. For anchors, this philosophy translates into testing provisions and assessment criteria at two performance levels. In this paper we use the terms "serviceability" and "suitability" to describe these levels. Serviceability requirements are intended to establish displacement behavior under service conditions; i.e., moderate concrete crack widths, design level anchor strength utilization and typical cycling demand. Suitability requirements establish whether an anchor exhibits reliable behavior and adequate residual load capacity in extreme events; i.e., maximum considered concrete crack widths, anchor strength utilization and cycling demand. These requirements are imposed via target crack widths and loading levels applied to the P50 (mean) and P90 (mean + 1.28 standard deviation) protocol presented above.

The crack widths used for cyclic tension load tests (static crack opening width w) and for cyclic crack tests (target crack width w_{max}) were taken as 0.5 mm for serviceability and 0.8 mm for suitability testing. The crack width of 0.5 mm corresponds to the maximum crack width for anchor qualification currently used for the simulated seismic tests according to ACI 355 and 0.8 mm is the maximum crack width considered for seismic anchorage (see Section 3.3). Due to the small influence of crack width on anchor shear test results, the crack width was chosen conservatively as 0.8 mm throughout the shear cyclic load test [34].

The anchor load levels used for cyclic load tests (target anchor load N_{max} for tension and V_{max} for shear) and for cyclic crack tests (sustained anchor tension load N_w) were defined relative to the mean ultimate capacities in cracked concrete $N_{u,cr,m}$ and $V_{u,cr,m}$ as determined from monotonic reference tests in 0.8 mm cracks. The characteristic strength was calculated as the 5 % fractile of the monotonic capacities assuming a 15 % coefficient of variation (v) and a statistical k-factor of k_s = 1.645. Thus $N_{max} = (1 - k_s \cdot v) \cdot N_{u,m,cr} = (1 - 1.645 \cdot 15\ \%) \cdot N_{u,m,cr} = 0.75 \cdot N_{u,m,cr}$ for cyclic tension load. Reversed cyclic shear loading typically results in anchor steel failure. The coefficient of variation for anchor steel failure is generally less than 10 % [34], so the characteristic shear load was calculated as $V_{max} = (1 - 1.645 \cdot 10\ \%)\ V_{u,m,cr} = 0.85 \cdot V_{u,m,cr}$. The values $0.75 \cdot N_{u,m,cr}$ and $0.85 \cdot V_{u,m,cr}$ were selected as the maximum considered loads at the suitability level demand. For the serviceability level demand, these target loads were reduced to a design level by applying a partial safety factor for load (γ_F = 1.4) and for material (γ_M = 1.5) resulting in $1 / (\gamma_F \cdot \gamma_M) = 1 / (1.4 \cdot 1.5) = 0.48$; i.e., an approximately 50 % reduction in the target loads [55]. For cyclic crack tests, the sustained anchor tension load (N_w) was derived from the characteristic monotonic strength, again estimated by $0.75 \cdot N_{u,m,cr}$, divided by the material safety factor $\gamma_M = 1.5$. At the suitability level this results in $N_w = 0.75 \cdot N_{u,m,cr} / 1.5 = 0.5 \cdot N_{u,m,cr}$ [36]. To determine serviceability demands, the sustained anchor tension load must be reduced. Following the rationale that during an earthquake both the load on the anchor and the crack width will be cycling and because the most critical condition for anchor performance is when the anchor load and crack width reach their maxima at the same instant in time, but the off-peak portion of the cycles represents reduced demand, the sustained load on serviceability level may be reduced to 80 % of the original value [56], i.e. $0.8 \cdot 0.5 \cdot N_{u,m,\mathrm{cr}} = 0.4 \cdot N_{u,m,cr}$. The test parameters are summarized in Table 4.

Table 4. Crack width and anchor load parameters for simulated seismic tests and reference tests.

Test type	Serviceability		Suitability	
	Crack width	Anchor load	Crack width	Anchor load
Monotonic tension[a]	w = 0.8 mm	-	w = 0.8 mm	-
Monotonic shear[b]	w = 0.8 mm	-	w = 0.8 mm	-
Cyclic tension	w = 0.5 mm	$N_{max} = 0.375\ N_{u,cr,m}$	w = 0.8 mm	$N_{max} = 0.75\ N_{u,cr,m}$
Cyclic shear	w = 0.8 mm	$V_{max} = 0.425\ V_{u,cr,m}$	w = 0.8 mm	$V_{max} = 0.85\ V_{u,cr,m}$
Cyclic crack	w_{max} = 0.5 mm	$N_w = 0.4\ N_{u,cr,m}$	w_{max} = 0.8 mm	$N_w = 0.5\ N_{u,cr,m}$

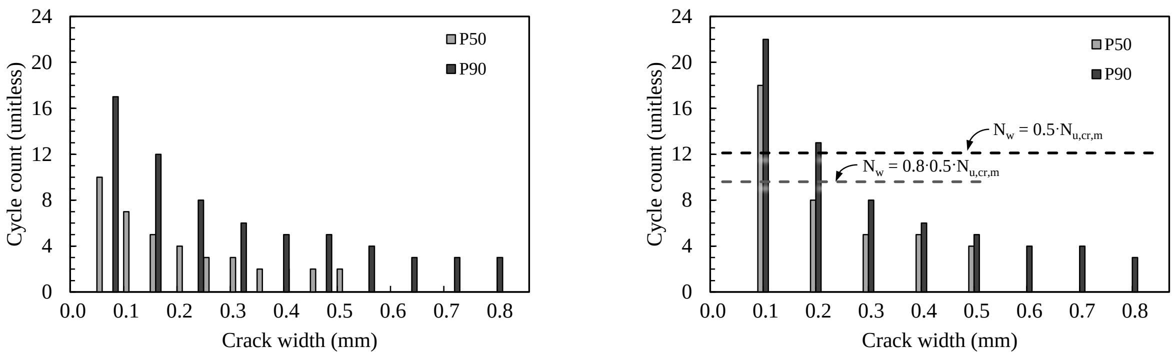
[a)] *To establish $N_{u,cr,m}$.*

[b)] *To establish $V_{u,cr,m}$.*

4.4.2 Unifying serviceability and suitability level protocols

Seismic qualification is time-intensive and expensive. Thus, the requirement to perform simulated seismic tests at two levels (*serviceability* and *suitability*) within a single anchor performance category, i.e., the ETAG 001 C2 performance category, poses a significant and potentially unnecessary burden for manufacturers seeking product approval. Therefore, it was desired to combine the serviceability and suitability level demands into a single unified protocol. This requires reorganization of the cycles in the P50 and P90 protocols considering the demand levels summarized in Table 4 in a way that still allows for evaluation of serviceability and suitability behavior.

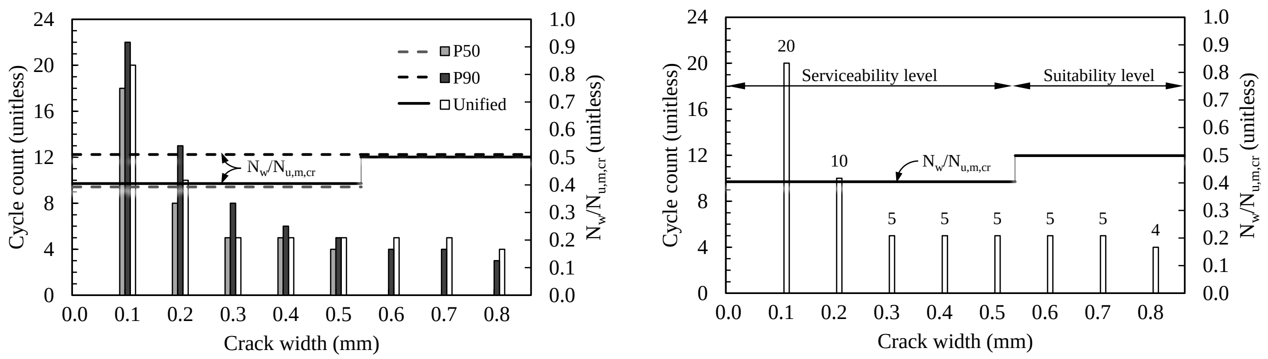
This process is first described for cyclic crack protocols. Since the P50 and the P90 protocols have different crack width steps in an absolute scale (Figure 3a), the two protocols were first re-binned. The P50 and P90 protocols then consisted of five bins ranging from w = 0.1 mm to 0.5 mm and eight bins ranging from w = 0.1 mm to 0.8 mm, respectively (Figure 3b). The permanent anchor load N_w is for the suitability (P90) level test higher than for the serviceability (P50) level test.



(a) (b)

Figure 3. Re-binning serviceability and suitability level protocols (example: cyclic crack width): (a) original cycle counts; (b) re-binned cycle counts with indicated load level.

To reduce differences between the serviceability and suitability test demands for the low amplitude bins, some of the suitability cycles were moved from the lower to the higher level bins (Figure 4a). This relocation was based on Miner's rule assuming a linear influence of anchor load, crack width and number of cycles on the damage expressed by anchor displacement. The sustained load $N_w = 0.4 \cdot N_{u,cr,m}$ for the serviceability test portion and is increased to $0.5 \cdot N_{u,cr,m}$ after completing cycling up through $w = 0.5$ mm (Figure 4b).



(a) (b)

Figure 4. Unifying serviceability and suitability level protocols (example: cyclic crack width): (a) relocated cycles for unified protocol taking crack width and anchor load into account; (b) final unified protocol.

A similar approach was applied for the cyclic load protocols. However, the target anchor load F_{max} (N_{max} or V_{max}), given as a normalized fraction of $N_{u,cr,m}$ and $V_{u,cr,m}$ for tension and shear, respectively, allows for a direct re-binning of the cycles from the 10 % bins. The transition from serviceability to suitability level is conducted after completing the $F/F_{max} = 0.5$ load level. For cyclic tension tests, the transition is accompanied with an increase of the static crack opening width w from 0.5 mm to 0.8 mm. Cycles at the $F/F_{max} = 0.1$ amplitude were truncated as they have negligible impact on anchor performance and elongate the protocol unnecessarily. Table 5 shows the resulting cycle distribution for the unified cyclic load and the crack protocols.

Table 5. Anchor load amplitudes and crack width amplitudes of unified test protocol.

Unified anchor load protocol				Unified crack width protocol				
Anchor load F/F_{max},[a]	Number of cycles	Crack width, mm Tension	Crack width, mm Shear	Crack width w/w_{max}	Crack width w_1, mm	Crack width w_2,[b] mm	Number of cycles	$N_w / N_{u,cr,m}$
0.1	-	0.5	0.8	0.125	0.1	0.0	20	0.4
0.2	25	0.5	0.8	0.250	0.2	0.0	10	0.4
0.3	15	0.5	0.8	0.375	0.3	0.0	5	0.4
0.4	5	0.5	0.8	0.500	0.4	0.0	5	0.4
0.5	5	0.5	0.8	0.625	0.5	0.0	5	0.5
0.6	5	0.8	0.8	0.750	0.6	0.0	5	0.5
0.7	5	0.8	0.8	0.875	0.7	0.0	5	0.5
0.8	5	0.8	0.8	1.000	0.8	0.0	4	0.5
0.9	5	0.8	0.8				59	
1.0	5	0.8	0.8					
	75							

[a] $F_{max} = 0.75 \cdot N_{u,cr,m}$ (tension), $F_{max} = 0.85 \cdot V_{u,cr,m}$ (shear).

[b] Crack closure achieved by applying compression force to the test specimen.

5. Validation tests

The unified protocols in Section 4 fulfil the requirements imposed by Eurocode 8, namely, they provide anchor displacement at a serviceability level as well as residual load capacity and corresponding displacement at a suitability level. To verify the unified protocols experimentally and to check the impact of cycle re-binning, exploratory tests were conducted. Only cyclic shear and cyclic crack tests were performed as they typically govern anchor performance.

5.1 Test program and setups

The P50, P90, and unified protocols are investigated by comparing the results (loads and displacements) of cyclic shear tests with an undercut anchor (size M10; h_{ef} = 90 mm) and cyclic crack tests with a headed bolt (19 mm shaft diameter; h_{ef} = 100 mm). In addition, cyclic shear tests and cyclic crack tests were performed using a bolt-type expansion anchor (size M12, effective embedment depth h_{ef} = 83 mm) using the unified test protocols. These anchor types and sizes were selected based on previous experience to produce potentially critical results. A total of 42 tests were conducted.

Normal weight concrete with a nominal 28-day compressive strength of 20 MPa served as the anchorage material. For cyclic shear tests, wedge-split reinforced concrete slabs 1635 mm × 1550 mm × 260 mm designed in accordance with ETAG 001 [6] were used. For cyclic crack tests, we used reinforced concrete specimens 700 mm × 420 mm × 270 mm conforming to ETAG 001 that allow for reliable generation and control of cracks. Post-installed anchor installation was in accordance with ETAG 001, while the headed bolts were cast into the concrete. Immediately prior to testing, the anchor installation torque was reduced by 50 % to account for stress relaxation over time typically observed in practice and in accordance with ETAG 001. For cyclic shear load tests, a 630 kN servo-controlled hydraulic actuator was used to load the anchor by means of a fixture that was mechanically held down to avoid uplift during testing (Figure 5a,b). For the cyclic crack tests, the concrete specimen was loaded axially by a 630 kN actuator (Figure 5c), while a 250 kN servo-hydraulic actuator was used to load the anchor axially (Figure 5c,d). Additional details can be found in [34] and [36].

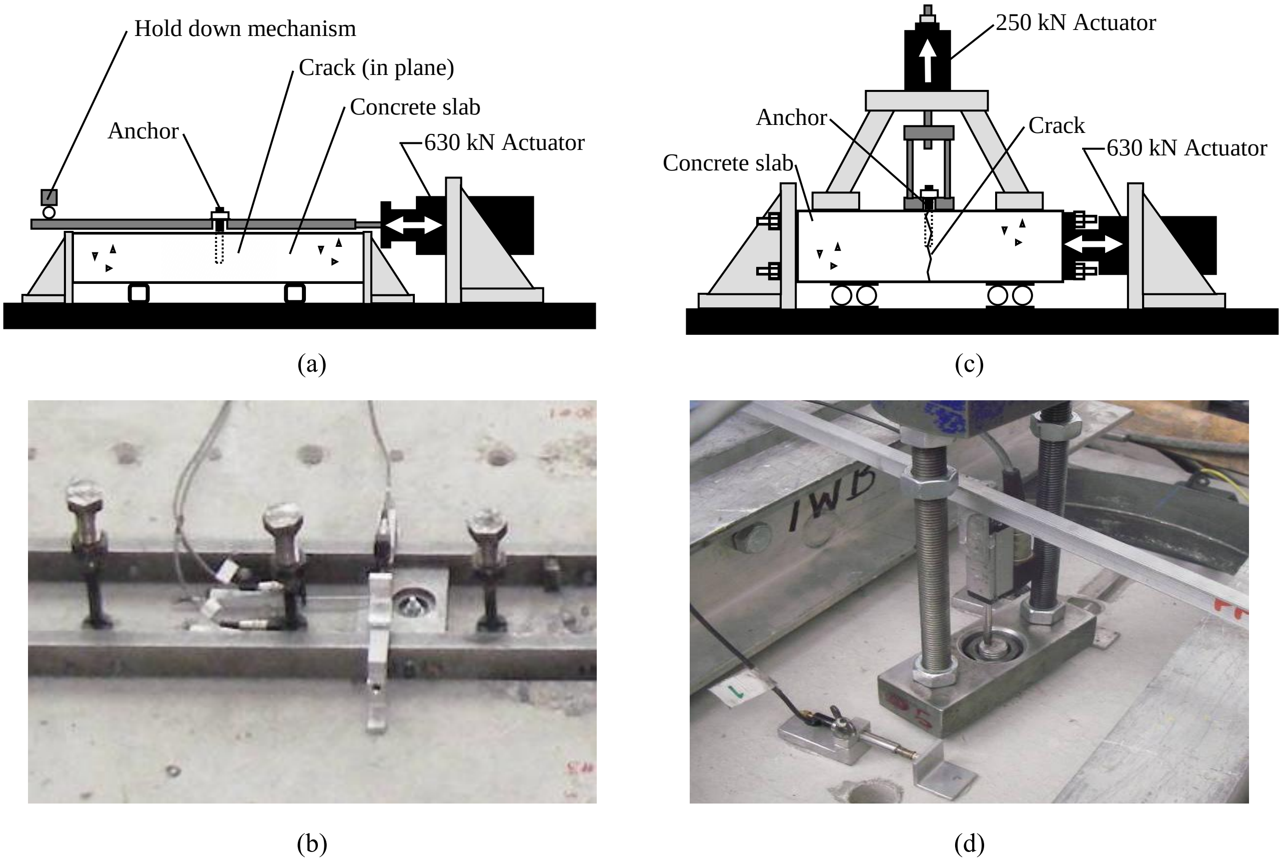
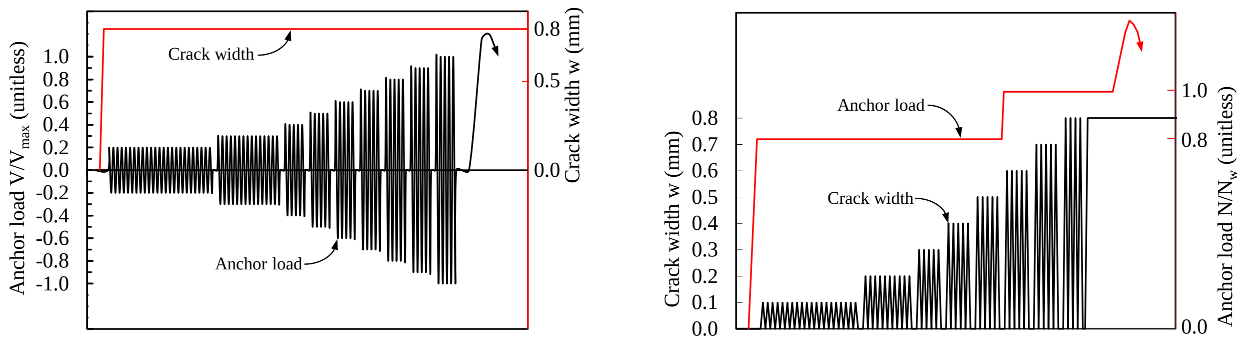


Figure 5. Schematics and detailed views of the test setups: (a) and (b) cyclic shear; (c) and (d) cyclic crack tests.

Displacement transducers (stroke: 50 mm; accuracy: ±0.01 mm) were used to measure the displacement of the fixture in the direction of loading (cyclic shear tests) and axial displacement at the anchor head (cyclic crack tests), as well as the crack width (stroke: 5 mm; accuracy: ±0.005 mm) sensors was used near the anchor (Figure 5b,d). Force data was obtained using load cells placed in-line with the actuators (range: 50 kN / 630 kN; accuracy: ±1 %). All data were recorded with a 5 Hz sampling rate. For the cyclic shear tests, the cracks were opened to the specified static crack width of w = 0.8 mm by hammering steel wedges into sleeves placed in the concrete prior to anchor installation [57]. The cyclic shear load and cyclic crack test protocols were executed at quasi-static rates using linear ramp functions. For the cyclic crack tests, the anchor load was held constant at the specified sustained load N_w during cycling. After completion of the cycles, the anchors were unloaded and then loaded under displacement-control to failure to determine the residual capacity (Figure 6). The monotonic reference anchor capacities in shear ($V_{u,cr,m}$) and tension ($N_{u,cr,m}$), which are used to determine V_{max} and N_w, were taken from the tests reported in [34] and [36]. Key test parameters are summarized in Table 6.



(a) (b)

Figure 6. Unified test protocols: (a) cyclic shear load ($V_{max} = 0.85 \cdot V_{u,cr,m}$); (b) cyclic crack ($N_w = 0.5 \cdot N_{u,cr,m}$).

Table 6. Key test parameters for the unified protocols.

Test type	Anchor type[a]	Tested monotonic reference capacity (w = 0.8 mm)	Initial target load for cyclic test serviceability level (w = 0.5 mm)	Initial target load for cyclic test suitability level (w = 0.8 mm)
Cyclic load	UA	$V_{u,cr,m}$ = 89.2 kN	V_{max} = 0.425 · 89.2 = 37.9 kN	V_{max} = 0.85 · 89.2 = 75.8 kN
	EA	$V_{u,cr,m}$ = 32.4 kN	V_{max} = 0.425 · 32.4 = 13.8 kN	V_{max} = 0.85 · 32.4 = 27.5 kN
Cyclic crack	HB	$N_{u,cr,m}$ = 71.4 kN	N_w = 0.4 · 71.4 = 28.6 kN	N_w = 0.5 · 71.4 = 35.7 kN
	EA	$N_{u,cr,m}$ = 21.8 kN	N_w = 0.4 · 21.8 = 8.7 kN	N_w = 0.5 · 21.8 = 10.9 kN

[a] UA = Undercut anchor; EA = Expansion anchor; HB = Headed bolt

5.2 Results and discussion

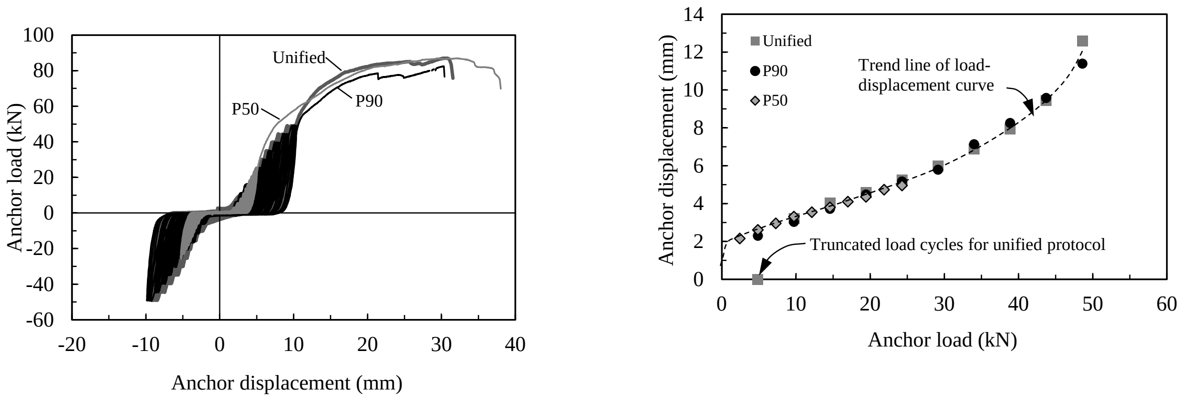
Table 7 summarizes key test results for the shear load cycling tests. The undercut anchors tested in cyclic shear failed by anchor steel rupture. In order to achieve five test replicates in which all of the cycles were completed without low-cycle fatigue of the anchor steel occurring, the target test load had to be reduced twice for the tests using the P90 protocol. After reduction of the target load V_{max} to 64 % (0.8·0.8) of the original value for the undercut anchor, all cycles were completed without effecting the residual load capacity of the anchor if compared to that measured for the unified protocol. The agreement between the mean displacements at the end of cycling ($s_{cyc,m}$), as well as the coefficients of variation of these displacements (ν), suggest that the unified protocol reasonably represents serviceability level cycling (P50)

Table 7. Results of validation tests for cyclic shear load.

Anchor type[a]	Protocol	Number of cycles	Crack width w	Target load $V_{max}/V_{u,cr,m}$	No. of tests	Serviceability Displacement $s_{cyc,m}$	Serviceability Displacement $v(s_{cyc,m})$	Suitability Displacement $s_{cyc,m}$	Suitability Displacement $v(s_{cyc,m})$	Suitability Residual Capacity $V_{u,m}$ ($V_{u,m}/V_{u,cr,m}$)	Suitability Residual Capacity $v(V_{u,m})$
			mm			mm	%	mm	%	kN (-)	%
UA	P90	212	0.8	0.85	1	Failure during cycle 201					
		212	0.8	0.8·0.85 = 0.68	1	Failure during cycle 204					
		212	0.8	0.8·0.8·0.85 = 0.54	5	-	-	9.95	13.3	93.0 (1.04)	11.7
	P50	118	0.8	0.5·0.8·0.8·0.85 = 0.27	5	4.90	4.2	-	-	91.9 (1.03)	9.8
	Unified	75	0.8	0.8·0.8·0.85 = 0.54	5	5.10	6.6	12.34	20.9	91.4 (1.02)	18.9
EA	Unified	75	0.8	0.85	1	Failure during cycle 69					
		75	0.8	0.8·0.85 = 0.68	1	Failure during cycle 70					
		75	0.8	0.9·0.8·0.85 = 0.61	5	3.66	9.3	6.31	9.9	39.6 (1.22)	4.7

[a] UA = Undercut anchor; EA = Expansion anchor.

Figure 7a depicts example load-displacement curves resulting from the P50, P90 and unified protocol tests on the undercut anchor. The load-displacement relation is nearly identical for all three protocols, resulting in a single backbone curve. Figure 7b depicts the maximum anchor shear displacements at each anchor load amplitude for the P50, P90 and unified protocol tested on the undercut anchor. The reported maximum displacement per load step is the average of all test repeats. The data points for all three tested protocols closely follow the same trend illustrating that the actual number of cycles per load step (which is different for each protocol) did not significantly influence the overall displacement response. Furthermore, it can be seen that truncating the cycles of the lowest load amplitude for the unified protocol had no discernable effect on the displacement at higher amplitudes.



(a) (b)

Figure 7. Cyclic shear tests on undercut anchor: (a) Example load displacement curves; (b) Anchor displacement as a function of anchor load (average of all test specimens).

The feasibility of the unified cyclic loading protocol was further confirmed by tests on an expansion anchor. As with the undercut anchor, the target test load was reduced twice (0.9·0.8 = 0.72) before five anchor samples completed all load cycles without failure. It is noted that the reduction factors were chosen ad-hoc to provide the largest factor for which all cycles were completed. One might expect that a 20 % load reduction would have a substantial impact on the cyclic fatigue; however, the number of cycles to failure were only marginally different (201 vs. 204 for the undercut anchor, and 69 vs. 70 for the expansion anchor). This highlights the importance of using a sufficiently large number of test repeats to obtain reproducible mean results.

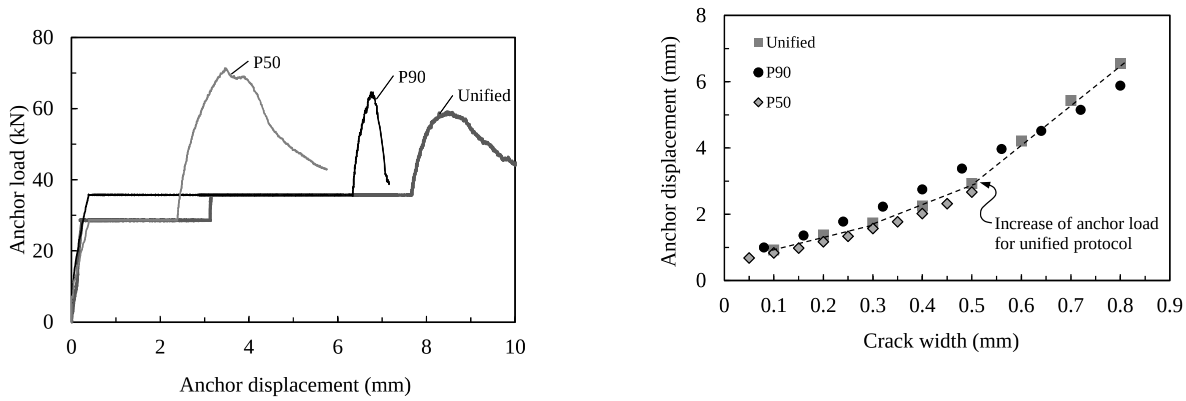
Table 8 summarizes key test results for the crack cycling tests. The headed bolt anchors completed all cycles and then failed in concrete cone breakout. The mean displacements at the end of serviceability cycling ($s_{cyc,m}$) of the unified protocol are in reasonable agreement with that of serviceability level cycling (P50). The residual load tested for the P90 protocol was 88 % of the monotonic capacity for the unified protocol, which is just below the threshold of 90 % specified by ETAG 001 Annex E for equivalence in crack cycling tests.

Table 8. Results of validation tests for cyclic crack protocol.

Anchor type[a]	Protocol	Number of cycles	Crack width w_{max}	Target load $N_w/N_{u,cr,m}$	No. of tests	Serviceability Displacement $s_{cyc,m}$		Suitability Displacement $s_{cyc,m}$		Suitability Residual Capacity $N_{u,m}$ ($N_{u,m}/N_{u,cr,m}$)	
						$s_{cyc,m}$	$v(s_{cyc,m})$	$s_{cyc,m}$	$v(s_{cyc,m})$	$N_{u,m}$ ($N_{u,m}/N_{u,cr,m}$)	$v(N_{u,m})$
			mm			mm	%	mm	%	kN (-)	%
HB	P90	66	0.8	0.5	4	-	-	5.90	6.6	59.9 (0.84)	13.7
	P50	40	0.5	0.4	4	2.74	10.8			71.8 (1.01)	5.0
	Unified	59	0.8	0.5	4	2.87	7.4	6.87	8.8	62.8 (0.88)	17.4
EA	Unified	59	0.8	0.5	1	Failure during cycle 57					
		59	0.8	0.875·0.5 = 0.44	5	8.52	9.1	23.18	22.5	24.3 (1.11)	26.7

[a] HB = Headed bolt; EA = Expansion anchor

Figure 8a plots typical load-displacement curves for the headed bolt anchors in the simulated seismic crack cycling test. The horizontal portions of the curves show the displacement that occurs while the anchor is loaded with a sustained tension load (N_w) and the crack cycles open and closed. The diagram in Figure 8b plots the maximum anchor displacements for each crack width amplitude for the P50, P90 and unified protocol tested on the headed bolt anchors. The reported maximum displacement per crack width is the average of all test repeats. At the serviceability level, the anchor displacement under the unified protocol largely follows the P50 protocol and the effect of the small shortfall for the 0.1 and 0.2 mm bins is marginal The effect of increasing the sustained tension load for the unified protocol after the crack width amplitude of 0.5 mm is clearly visible in the increased rate of anchor displacement.



(a) (b)

Figure 8. Cyclic crack test on cast-in headed bolts: (a) example load displacement curve; (b) anchor displacement as a function of crack width (average of all test specimens).

The expansion anchors failed by the anchor head pulling through the expansion segments (pull-through failure). The target load for the expansion anchors had to be reduced from the initial value to achieve four test repeats without failure occurring during crack cycling. The displacement after completing serviceability level cycles is $s_{cyc,m}$ = 8.52 mm, which is substantially more than for the headed bolt ($s_{cyc,m}$ = 2.87 mm) and more than what is acceptable according to ETAG 001. Furthermore, expansion anchors tend to show larger scatter in the test results particularly at larger crack widths or near ultimate load levels. The coefficient of variation of the residual capacity is $v(N_{u,m})$ = 26.7 %, which is greater than the 20 % currently allowed in ETAG 001. Both the displacement after crack cycling and the coefficient of variation of the residual capacity could be reduced by reducing the target load level (N_w); results not shown. These findings highlight the need to consider displacement assessment criteria, variable target load levels and the allowable coefficient of variation in seismic anchor qualification tests. All of these aspects have been adopted in test conditions and assessment criteria of ETAG 001 Annex E.

6. Summary and conclusions

This paper synthesizes research performed over the past fifteen years to develop new seismic qualification guidelines for post-installed anchors. The work resulted in forward-ordered, stepwise-increasing cyclic load and crack protocols for representative simulated seismic tests on post-installed anchors. Particular emphasis is given in this paper to applicability to and implementation in Europe, nonetheless, we relate the methodology to US practice. Nonlinear seismic analyses of seven prototypical reinforced concrete buildings resulted in protocols that reflect anticipated seismic demands in high seismic regions of the United States and Europe and allow evaluation of the anchor performance under serviceability and suitability test conditions. To reduce testing costs and effort, however, the protocols have been unified assuming linear damage accumulation. These unified protocols for cyclic crack widths and cyclic anchor loads allow assessment of serviceability and suitability level performance during a single qualification test. Particular focus was placed on the protocol’s ability to reliably quantify serviceability displacement demand and suitability residual load capacity as mandated by Eurocode 8. The applicability of the unified protocols was successfully validated experimentally for load and displacement at serviceability and suitability levels.

The proposed test protocols together with the specified test parameters were implemented in 2013 in the European anchor qualification guideline ETAG 001 Annex E for the C2 performance category. This category provides an additional level of safety for severe earthquake applications where the C1 qualification may be inadequate. Numerous anchor products have been successfully seismically qualified according to that standard since its introduction.

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