Design of Anchors In Accordance with the 2003 IBC

By John F. Silva and Norbert Randl

The 2003 *International Building Code®* (IBC) is the first widely adopted model building code to contain specific provisions for the design of post-installed anchors in concrete. (*Note: The NFPA 5000 Building Code likewise references ACI 318-02 for reinforced concrete design.*) The provisions are contained in Appendix D of ACI 318-02, and the reference to this document is contained in IBC Section 1913, Anchorage to Concrete – Strength Design as follows:

> ***1913.1 Scope****. The provisions of this section shall govern the strength design of anchors installed in concrete for purposes of transmitting structural loads from one connected element to the other. Headed bolts, headed studs and hooked (J- or L-) bolts cast in concrete and* <u>*expansion anchors and undercut anchors installed in hardened concrete*</u> *shall be designed in accordance with Appendix D of ACI 318, provided they are within the scope of Appendix D.*

Furthermore, the qualification of post-installed anchors that may be designed using the provisions of Appendix D is addressed in ACI 318-02 Section D.2.3 as follows:

> *Post-installed anchors that meet the assessment requirements of ACI 355.2 are included. The suitability of the post-installed anchor for use in concrete shall have been demonstrated by the ACI 355.2 qualification tests.*

The significance of these provisions is best illustrated by a brief review of the history of anchorage design in the U.S.

For well over half a century, the design of anchors in concrete has been approached using tabulated resistance values (allowables) developed for use with Allowable Stress Design (ASD) procedures. Since early codes were silent on the subject of anchors installed in hardened concrete (i.e., post-installed anchors), proprietary products were assessed through criteria developed by the various model code organizations in the U.S. (ICBO, BOCA, SBCCI) that likewise presumed the use of ASD design procedures and assigned a global safety factor to the mean result of anchor tests conducted in plain concrete. Code provisions for the strength design of cast-in anchors first became available with the publication of the 1988 *Uniform Building Code®* (UBC), whereby the strength predictor equations were based on engineering models that had been adopted in nuclear and precast concrete design. With the issuance of the 2000 IBC, however, the strength design of cast-in anchors was revised to follow the procedures developed by ACI and commonly referred to as the Concrete Capacity Design (CCD) method. The CCD method relies heavily on a body of anchor research developed both in the U.S. and Europe over the past quarter century and includes models for the prediction of anchor failure modes not previously considered. Nevertheless, the 2000 IBC provisions specifically excluded anchors in hardened concrete owing to the lack of suitable qualification procedures at the time of publication. The issuance of ACI 355.2 *Evaluating the Performance of Post-Installed Mechanical Anchors in Concrete* in 2001 and its adoption into the acceptance criteria AC193 by the ICC Evaluation Service enabled the inclusion of design provisions for post-installed expansion and undercut anchors in the 2003 IBC.

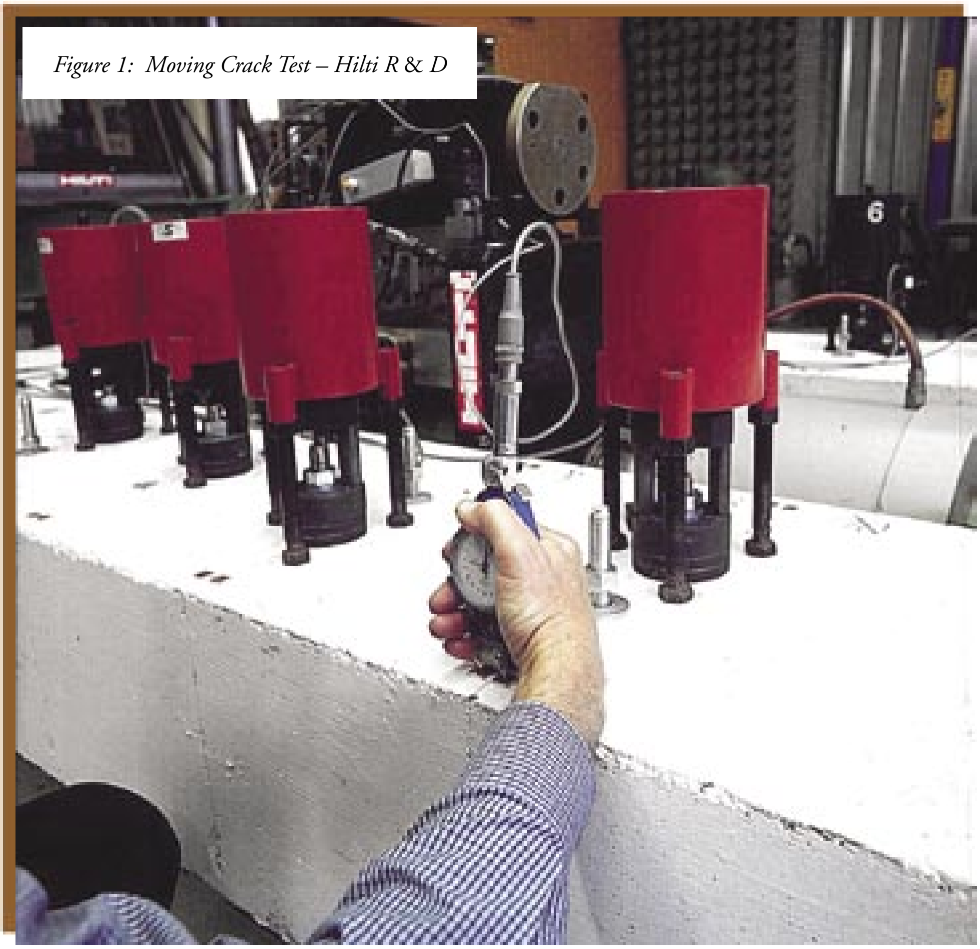

Figure 1: Moving Crack Test – Hilti R & D

Aside from the introduction of the CCD method, the 2003 IBC represents significant changes to the manner in which post-installed anchors are qualified and designed.

Cracked concrete: The influence of concrete cracking on anchor performance has been under study for several decades. A key finding is that cracks that develop in the neighboring concrete subsequent to anchor installation have a high likelihood of passing through the anchor location, due to tensile stresses introduced by the anchor into the concrete. Previously, the effects of concrete cracking on anchor performance were specifically considered only in the context of load and resistance factor design (LRFD), whereby additional load factors were included for anchors installed in the "tension zone" of a member. Evaluation reports for post-installed anchors issued since 1991 have limited their use to

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concrete that is expected to remain uncracked for the life of the anchorage ("uncracked" or "non-cracked" concrete). Misunderstandings regarding the basis for the restriction, the intent of the language contained in evaluation reports, and the lack of suitable alternatives have led to widespread disregard of this restriction in design. For the first time, ACI 355.2 provides specific procedures for the qualification of anchors that are to be used in cracked and uncracked concrete conditions as distinct from those products that are to be used only in uncracked concrete.

Seismic design: Seismic qualification testing in the form of simulated seismic loading protocols has been a part of anchor acceptance criteria since the Northridge earthquake of 1994. ACI 355.2 requires that the simulated seismic loading tests be conducted on anchors installed in tension cracks having a width 0.5 mm (0.020 in.). Furthermore, all anchors used to resist seismic loads must generally be qualified for use in cracked concrete. The 2003 IBC specifically prohibits the application of the ASD provisions of the code (Section 1912) to anchors in hardened concrete and to anchors that are used to resist seismic loads.

Reliability testing: Reliability tests are intended to demonstrate the suitability of products for job-site use in terms of their ability to function properly under less than ideal conditions. Such tests have been a part of acceptance criteria for post-installed anchors for many years. Nevertheless, ACI 355.2 introduces two new concepts that have the potential to significantly change the assessment outcome. The first is the use of a "reference expansion" for displacement controlled anchor systems (the most common of these is the "drop-in" anchor). Reference expansion refers to the degree of anchor expansion achieved through the use of a representative amount of driving energy. A special setting tool outfitted with a standard weight falling through a specified distance is used to establish the reference expansion, and this degree of anchor expansion is then used in the specified reliability tests. Displacement controlled anchors that require an inordinate degree of setting energy will likely be penalized by this provision.

The second change is the introduction of the "moving crack test" for anchors to be qualified for use in cracked concrete. In this test, the anchor is placed in a hairline crack and subjected to a constant tension load while the crack width is cycled. This test is intended to assess the anchor response to changes in crack width over time, as caused by changes in structure loading, temperature, etc. Limits on the anchor displacement over a specified number of crack cycles are established in the criteria, and the residual tension capacity of the anchor is measured at the conclusion of the test. The degree to which the anchor satisfies the displacement requirements while maintaining a sufficient margin against failure is contributory to establishment of the anchor reliability rating, expressed in terms of an anchor category, as well as the tension capacity as governed by pullout.

The use of LRFD has traditionally been seen as a method to increase both the reliability and efficiency of structures. Its application to the area of anchorage to concrete via the CCD method achieves this end by explicitly considering a number of possible failure modes (steel failure, concrete breakout, anchor pullout, pryout, etc.), and assigning capacities to each on the basis of simplified engineering models. To illustrate how this approach differs from the ASD paradigm, consider an anchor that when loaded in tension in normal strength concrete fails consistently by yielding and fracture of the steel bolt. Under ASD assessment procedures, the mean result of tension tests conducted with this anchor would be divided by a global safety factor (typically 4 if special inspection is provided, 8 if it is not), regardless of failure mode. Using the CCD method, steel failure is assessed independently of concrete failure, appropriate strength reduction factors are assigned to each, and the controlling design strength is compared to the factored load. Moreover, the

CCD method explicitly considers probable scatter associated with each failure mode by calibrating the design equations to the 5% fractile of the expected strength and requiring that the 5% fractile of tests results, not the mean, be used for the assessment. (*The 5% fractile is defined as that value that will be exceeded by 95% of the population with a 90% confidence. It is determined on the basis of a non-central Student's t-distribution. The reader is referred to Hahn, Gerald J. and Meeker, William Q., Statistical intervals: a guide for practitioners, John Wiley* & *Sons, Inc., 1991.*)

Efficiency increases associated with application of LRFD vs. ASD will largely depend on the consistency of the anchor performance and the extent of testing conducted in the qualification program.

A number of issues have arisen in connection with the use of Appendix D for the design of anchors since the issuance of ACI 318-02. Some of these are addressed below.

Q Can I use existing ASD design data developed for post-installed anchors in conjunction with the provisions of Appendix D?

A Generally speaking, no. While it is theoretically possible to convert ASD design values back to an LRFD format with appropriately conservative assumptions for failure mode, coefficient of variation, etc., the assignment of the correct anchor category is difficult given the lack of appropriate reliability test data. Note that anchors that do not satisfy a minimum degree of reliability are excluded from use with Appendix D.

Q How are the seismic provisions of Appendix D different from previous criteria?

A Anchor approvals issued under ASD criteria commonly address seismic design through reference to the Alternate Allowable Load Combinations of the code, wherein a ⅓ increase in the design resistance is often permitted for cases involving wind or seismic loading. In Appendix D, two basic provisions apply to the design of anchors to resist seismic loads: 1) The anchor design must either be controlled by the strength of a ductile steel element (D.3.3.4) or the anchor design must be adequate to force yielding of the (steel) member attached by the anchor to the structure (D.3.3.5); 2) The anchor capacity for resisting seismic loads is reduced to 75% of the controlling design strength (D.3.3.3). Taken together, these provisions are intended to avoid non-ductile anchor failure modes under seismic loading conditions. They represent a substantial departure from past practice. (*Note: The parallel provisions for anchor design contained in Appendix B of ACI 349-01 permit non-ductile failure modes provided that an additional degree of over-capacity is provided.*)

Q Are the provisions of Appendix D more conservative than the strength design provisions contained in previous codes?

A Previous formulations of strength design provisions for anchors were based on the "45 degree cone" model, which was also incorporated in earlier editions of ACI 349. This model was compared with the CCD method by ACI committee 349 through use of an extensive database of anchor test results. The correlation of both models with test results at small embedments was good; however, the 45 degree cone model is believed to over-predict the capacity of deep embedments. More importantly, the predictions of concrete breakout capacity for anchor groups and anchors near free edges associated with the 45 degree cone model were shown to be unconservative. (*Additional information on these comparisons may be found in the commentary to ACI 349-01 and in two papers authored by Shirvani, Klingner and Graves that appear in Vol. 101, No. 6 of the ACI Structural Journal, Nov.-Dec. 2004*)

Q Do the provisions of Appendix D apply to adhesive anchors?

A No. Specific provisions for the design and assessment of adhesive anchor systems are under development. In particular, the tension resistance of adhesive anchors is driven by concepts of bond that also affect the performance of anchor groups and anchors in near-edge conditions. The assessment of adhesive anchors should consider additional reliability criteria, as well as questions of sensitivity to temperature extremes and environmental exposure.

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Global Safety Factor

A direct comparison of the global safety factor associated with anchor designs according to ASD and LRFD under the 2003 IBC is possible only for specific cases since it depends on the anchor category as assessed from reliability tests, the degree of scatter associated with the test results, and the controlling failure mode for a given design condition. The following examples are provided for illustration.

Example 1

Consider a post-installed anchor that has been assigned to anchor category 2 on the basis of reliability testing in accordance with ACI 355.2. The mean tension capacity associated with concrete breakout failure in uncracked concrete is determined from 10 tests whereby the coefficient of variation associated with the test results is 7%. According to ACI 318-02 Appendix D, the strength reduction factor for concrete breakout failure of a category 2 post-installed anchor is 0.55. (This assumes the use of the load combinations given in Chapter 9 of ACI 318-02.) For illustration purposes, the design resistance for this failure mode as a function of the measured mean capacity F_m can be approximated as:

$$\phi N_{cb} = \phi N_u = \phi F_m(1-Kv) = 0.55F_m(1-2.568\cdot 0.07) = 0.45F_m$$

(Note: The actual resistance according to Appendix D is a function of the coefficient for the basic concrete breakout strength in tension, k.)

For a design involving only dead and live loading where $D = L$, the global factor of safety on the anchor mean tension capacity associated with concrete failure may be expressed as:

$$SF = \frac{1.2+1.6}{2\times 0.45} = 3.1$$

Example 2

In this example, the same anchor considered above is calculated to have a nominal steel strength of 8,000 pounds while the concrete capacity under tension load as calculated in accordance with Appendix D is 12,000 pounds. Because the steel bolt used in the anchor has been determined to satisfy the ductility requirements of Appendix D, it is assigned a strength reduction factor of 0.75. Thus the controlling strength (all other failure modes having been determined to be non-applicable) is determined as follows:

$$\phi N_{cb} = 0.55\times 12{,}000 = 6{,}600 \text{ lb} > \phi N_s = 0.75\times 8{,}000 = 6{,}000 \text{ lb } \underline{\text{governs}}$$

For equal dead and live loads, the global factor of safety on the anchor tension capacity as governed by steel failure is given by:

$$SF = \frac{1.2+1.6}{2\times 0.75} = 1.9$$

whereby the global safety factor on concrete failure remains in excess of 3.1.▪

Q Why have so few approvals for post-installed anchors been issued under the 2003 IBC?

A Issuance of an Evaluation Services Report (ESR) by ICCES under the 2003 IBC requires testing in conformance with AC193, which in turn references ACI 355.2. This testing is more complex and, in some cases, more extensive than testing previously conducted. It is anticipated that the number of ESRs issued under the 2003 IBC will increase substantially in the next 12-24 months.▪

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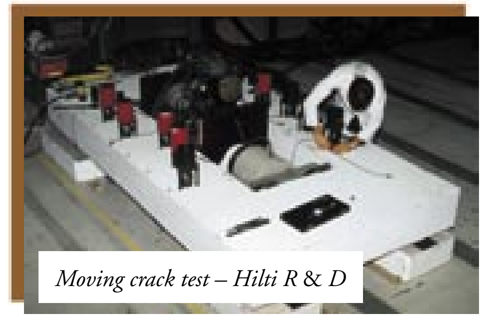
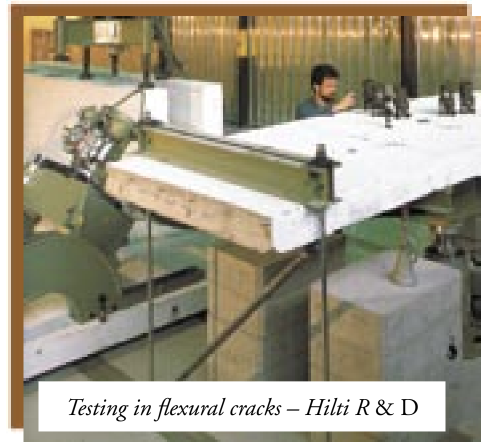
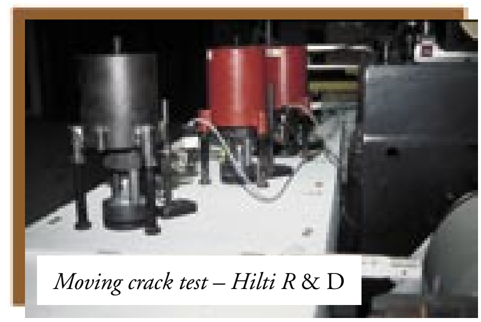


Moving crack test – Hilti R & D



Testing in flexural cracks – Hilti R & D



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