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SECTION III - NUCLEAR SSCS DESIGN AND ANALYSIS REQUIREMENTS

New in this revision (older revisions addressed in 6.0 Record of Revisions)

Added Hilti KB-TZ2 to App. A; new App. H for design for same. Crane guidance expanded in App E.

Contact the Structural Engineering Standards POC
for upkeep, interpretation, and variance issues

Ch. 5 Section III	**Structural POC/Committee**

A. This Section provides the minimum requirements for the structural design and analysis of new and existing nuclear structures, systems, and components (SSCs).[1]

B. Acronyms, notations, and references not defined herein are included in Subsections 2.0—4.0 of Section I of this Chapter.

C. 1020: Per DOE Order 420.1C Change 3*, Facility Safety*, the design of new facilities and "major modifications (a defined term)" must comply with DOE-STD-1020-2016, *Natural Phenomena Hazards Analysis and Design Criteria for DOE Facilities*.

1. The remainder of this Section refers to DOE-STD-1020-2016 as "1020."

2. The vast majority of the content of "1020" (applicable to nuclear SSCs) isn't repeated herein (in this Section)[2].

3. ASCE 4 and ASCE 43: LANL amends all "1020" references to ASCE 4-98 and ASCE 43-05 to ASCE 4-16 and ASCE 43-19, respectively; see Appendix F herein for details.

4. For the design and construction of "non-major modifications," either follow "1020" or contact the Chapter POC for direction (Requirement 5-0129). *Guidance: In such cases, following "1020" is the preferred option; however, in some circumstances a well-considered and documented determination that following the code of record (COR), or an enhanced COR, is appropriate. COR is discussed in general terms in ESM Ch 1 Z10, and in ESM Ch 16 IBC Program for those applications.*

5. *The civil engineering systems referred to in the "1020" chapters on flood and precipitation hazards are addressed in ESM Chapter 3, Civil.*

6. *The requirements associated with lightning hazards in "1020" are addressed in ESM Chapter 7, Electrical.*

[1] Per DOE-STD-1020-2016, and as indicated in Section I Subsection 3.0, the nuclear facilities considered herein are DOE Hazard Category 1, 2, and 3 nuclear facilities. In addition to having nuclear materials above DOE HC 3 thresholds, these facilities can also include chemical and/ or toxicological hazards.

[2] While this Section does include some of the requirements of "1020" that are applicable to LANL, as well as some related explanations, commentary, etc., users are highly encouraged to consult DOE-HDBK-1220-2017 to gain a better understanding of "1020."

D. Table 2-1 in "1020" contains the Natural Phenomena Hazard Design Categories (NDCs) for NDCs 1–3 based on the unmitigated consequence thresholds associated with radiation and chemical exposure (i.e., Total Effective Dose [TED] and Protective Action Criteria [PAC] respectively) by co-located workers and the public.[3,4]

Using Table 2-1, for each natural phenomena hazard (NPH)—seismic (S), wind (W), flood (F), precipitation (P), and volcanic eruption (V)—safety and other SSCs shall be categorized by assigning to them one of three NPH Design Categories (NDCs): NDC-1, NDC-2, or NDC-3 (Requirement 5-0130).

1. Although NDC-4 and NDC-5 exist, the likelihood of occurrence is small enough that they are not addressed herein (Requirement 5-0131).

2. Each NDC is applicable across the NPHs (Requirement 5-0132).

    a. For example, NDC-1 applies to seismic design category (SDC) SDC-1, wind design category (WDC) WDC-1, etc.

E. Limit states (LSs): The LS of an SSC for seismic (i.e., A, B, C, or D), and the limiting acceptable deformation, displacement, stress, strain, etc. for other NPHs, shall be determined in accordance with "1020" paragraph 2.4 (Requirement 5-0133).

1. For convenience, ANS 2.26 Appendix B, *Examples of Application of LSs to SSCs*, is adapted for LANL and included herein as Appendix D.

F. The project records, quality control, quality assurance, and construction of nuclear SSCs shall comply with the provisions of Section I of this Chapter; ESM Chapter 1, *General*; ESM Chapter 16, *IBC Program*; and the provisions contained herein (i.e., Section III) (Requirement 5-0134).

1.0 NDC-1 AND NDC-2 SSCS

1.1 General

A. As indicated previously, NPH-specific design categorization is required by "1020." Therefore, in order for this portion of this Section to be applicable, the design categorization required by the respective portion of "1020" has to have been performed (Requirement 5-0135).

B. For example, in the "Seismic" article (immediately below), it is assumed that the "safety SSCs" and the "other SSCs" applicable to the design-basis seismic event have been categorized in accordance with "1020" paragraph 3.1.1 and the categorization indicates that SDC-1 and/or SDC-2 SSCs exist/are applicable.

1.2 Seismic

A. SDC-1 and SDC-2 SSCs—Building Structural

1. As applicable/ necessary, adjust the LS (determined per "1020" paragraph 2.4) in accordance with "1020" paragraph 3.1.3 (Requirement 5-0136).

2. Using "1020" Table 3-1, SDC-1 and SDC-2 SSCs shall be designed as Risk Category II and Risk Category IV facilities, respectively, and according to the criteria of Section II of this Chapter (Requirement 5-0137).

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[3] Regarding chemical hazards and the associated PACs in Table 2-1, refer to "1020" para. 2.3.3.6.
[4] Despite not being mentioned here, facility workers/building occupants are "accounted for." Refer to Table 2-1 Note 2, and "1020" para. 2.3.3.3.

B. SDC-1 and SDC-2 Non-Facility Equipment (e.g., Programmatic, Utilities, Infrastructure, Environmental Remediation, etc.) shall be restrained in accordance with Appendix G, *Restraint of Non-Facility Components*, herein (Requirement 5-0138).

1.3 Wind

A. WDC-1 and WDC-2 SSCs

1. WDC-1 and WDC-2 SSCs shall be designed as Risk Category (RC) II and RC IV facilities, respectively, and according to the criteria of Section II of this Chapter (Requirement 5-0139).

1.4 Flood

A. FDC-1 and FDC-2 SSCs

1. FDC-1 and FDC-2 SSCs[5] shall be designed such that the following requirements are met[6]:

a. They shall be able to withstand flood loads resulting from Design Basis Flood Levels (DBFLs)[7] using the load combinations specified in the IBC for Risk Category II and Risk Category IV, respectively; excluding the load factor for the flood load (Requirement 5-0140).

b. When SSCs that may fail due to water intrusion or submergence cannot be located above the DBFL, consider the other mitigation strategies in "1020" paragraph. 5.5.4 (Requirement 5-0141).

1.5 Precipitation

A. PDC-1 and PDC-2 SSCs[8]

1. PDC-1 and PDC-2 SSCs shall be designed as Risk Category II and Risk Category IV facilities, respectively, and according to the criteria of Section II of this Chapter subject to the following changes (Requirement 5-0142):

a. Ground Snow Load, p_g , = 19.2 psf (Requirement 5-0143).

b. PDC-2 design rainfall intensity, I, = 2.3 inches/hour (Requirement 5-0144).

1.6 Volcanic Eruption

A. **VDC-1 and VDC-2 SSC** design is not required (Requirement 5-0145).

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[5] Pertains to FDC-1 and FDC-2 SSCs in/on FDC-1 and FDC-2 facilities *only*. In other words, this provision doesn't pertain to FDC-1 and/or FDC-2 SSCs within/on facilities with FDC-3 SSCs. For FDC-1 and FDC-2 SSCs in a facility that also has FDC-3 ones, see Flood Loads under "NDC-3 Structures and Structural Systems" herein. On a related note, while the FDC of a facility is likely the same as the highest FDC of the SSCs in/on it, the DBFL for a facility and its SSCs might differ. See "1020" para. 5.4.2 for more detail.

[6] Flood design per "1020" requires PFHA, FSA and, perhaps, a CFHA; and the IBC requires establishment of FHAs. Until such time that these exist, LA-14165 Para. 2.6.2 is all that's available for determining Design Basis Flood Levels (DBFLs) and flood loads. Included therein are some of what's required for design for this NPH subject to the following changes: "DOE-STD-1020" and "DOE-STD-1023" be taken as (=) DOE-STD-1020-2016, PC-1 = FDC-1, PC-2 = FDC-2, ASCE 24 = ASCE 24-13, ASCE 7 = ASCE 7-10, IBC 2003 = 2015 IBC, and the Source of flooding in Table 2-19 will be edited to read in accordance with "1020" Table 5-4 (i.e., add "1020" Case Nos. 3 and 4 to Levee/Dam Failure in Table 2-19, and delete Case No. 3 from Local Precipitation in Table 2-19).

[7] A DBFL can be influenced by local precipitation. See "Precipitation" herein, and "1020" fig. 5-1.

[8] There are two (2) precip.-related hazards included in "1020" Ch. 7: localized flooding and roof ponding/ (over-) loading. Only the latter is addressed herein. To address localized flooding, which can result in water intrusion or submergence of SSCs and/ or structural failure from hydrostatic or hydrodynamic loads, the Design Basis Precipitation Level (DBPL) must be determined from a Probabilistic Precipitation Hazard Assessment (PPHA). Finally, the localized flood hazard in "1020" Ch. 7 is related to the flood hazard in Ch. 5. Refer to "1020" paras. 5.2.2.1, 5.5.3.1, 7.1.1(d) and 7.3.1.

2.0 NDC-3 STRUCTURES AND STRUCTURAL SYSTEMS

2.1 Acceptance Criteria

A. General

1. The structural demands (member forces, displacements, etc.) shall be determined using the loads, load combinations, and analysis procedures indicated subsequently herein (Requirement 5-0146).
2. Linear analyses shall demonstrate compliance with the subsequent paragraphs on strength and deformation acceptance criteria (Requirement 5-0147).
3. Nonlinear analyses shall demonstrate compliance with the subsequent paragraph on deformation acceptance criteria (Requirement 5-0148), and yielding elements (in nonlinear analyses) shall demonstrate compliance with the subsequent paragraph on strength acceptance criteria (Requirement 5-0149).
4. The following materials are permitted for use (Requirement 5-0150). Design, detailing and determination of capacities shall be in accordance with ASCE 43, and the indicated material design standards, subject to any/all applicable amendments herein (Requirement 5-0151):

Material	Material Design Code/Standard[9]
Reinforced Concrete	ACI 349*, **
Structural Steel	ANSI/AISC N690*
Stainless Steel	ANSI/ASCE 8
Cold Formed Steel	AISI S100
Reinforced Masonry	TMS 402/ACI 530/ASCE 5 (ACI 530)

* Per DOE O 420.1C Chg 3, in addition to being applicable to the design of Safety Class (SC) SSCs, both ACI 349 and AISC N690 "…must be evaluated for applicability..."—along with ACI 318 and AISC 360—for Safety Significant (SS) SSCs. SS SSCs protect the co-located worker; at LANL, SS SSCs that do so at the Category NDC-3 level must be designed to ACI 349 and AISC N690. For those SS SSCs that fall into the NDC-1 or NDC-2 categories because of the unmitigated consequences, they must be designed, at a minimum, to ACI 318 and AISC 360, consistent with the requirements in Section II of this Chapter.

While both AISC 360 and AISC 325 (the Steel Construction Manual) are indicated in DOE O 420.1C as being applicable to SS SSCs, the inclusion of the latter is likely an error (since the only portion of "325" that is enforceable is "360").

** *Use of NRC RG 1.142 with ACI 349 is recommended. The RG includes revisions to the content of ACI 349, and the associated rationale/commentary. Finally, the RG is a 2020 document, whereas ACI 349 is 2013 document that's based on a 2008 document (i.e., ACI 318-08).*

B. Strength Acceptance Criteria

1. For linear analyses, the demand acting on an element shall be less than or equal to its capacity (as stipulated previously, and per subsequent amendments, herein) (Requirement 5-0152):

$$\text{Demand} \leq \text{Capacity} \qquad \text{(Eq. III-1)}$$

[9] Use the most recent version of the applicable material codes in force in agreement with the established code of record. Project design documents shall contain the dates for the code of records. Where no code of record is established for the modification of existing facilities, the engineer in responsible charge shall document the revision of the code used for design.

2. For nonlinear analyses, the capacity of all elements, including yielding elements, shall be limited to the respective design strength (i.e., strength reduction factor x nominal strength) (Requirement 5-0153). Some of the more common design strengths are as follows:

ϕM_n for bending,
ϕV_n for shear, (Eq. III-2)
ϕP_n for axial loads, etc.,

3. where ϕM_n ϕV_n and ϕP_n are the design strengths in flexure, shear, and axial force, respectively, which are determined as stipulated previously, and per subsequent amendments, herein.

C. Deformation Acceptance Criteria

1. As applicable, deformation limits, serviceability requirements and deformation acceptance criteria stipulated in the aforementioned material design standards shall be complied with (Requirement 5-0154).

2. The deformation acceptance criteria for seismic loads shall be in accordance with Section 5.2.3 of ASCE 43 (Requirement 5-0155).

2.2 Loads

A. Dead Load (D)

1. The value of D shall include the weight of all materials of construction incorporated into the building including, but not limited to, walls, floors, roofs, ceilings, stairways, nonstructural components (e.g., ceilings, built-in partitions, finishes, cladding, fixed service equipment, piping, etc.) and other similarly incorporated items, including the weight of cranes (Requirement 5-0156).

   a. For evaluation of seismic response, not only must the contribution of nonstructural components to D be considered, but their contribution to stiffness must be too (Requirement 5-0157).

2. Unit weights of 150 pcf for reinforced concrete and 490 pcf for rolled steel shall be used in the derivation of D (Requirement 5-0158).

3. The value of D used in the design of new facilities shall be based on best estimates of the actual weights comprising it (Requirement 5-0159).

B. Live Load, Experiment Blast Load (L_{EB})

1. Determine L_{EB} based on project-specific requirements (Requirement 5-0160). As necessary/ applicable, L_{EB} shall include the blast loading on the outside, or inside, of a building from experimental explosions, and/or the loads that must be resisted by a building from blasts/ explosions occurring within experimental-explosion containment structures (Requirement 5-0161).

C. Crane Loads (C or C_{cr})

1. Determine C (or C_{cr}) in accordance with Chapter 4.0, Section 4.9 of ASCE 7, and based on project-specific requirements (Requirement 5-0162).

D. Fluid Loads (F)

1. Determine F – the load due to fluids with well-defined pressures and maximum heights (e.g., fluid in tanks, etc.) – based on project-specific requirements (Requirement 5-0163).

E. Lateral Soil Pressure Loads (H)[10]

1. Subterranean structural walls shall be designed to resist H, assuming steady-state/at-rest conditions, the magnitude of which shall be specified by a geotechnical report approved by a LANL subject matter expert (Requirement 5-0164). A default value of 0.5 for at-rest lateral soil pressure coefficient, K_o, may be used to determine H[11].

2. In determining H, the density of the backfill or native soil, whichever is greater, shall be used (Requirement 5-0165).

3. The value of H shall be increased if soils with expansion potential are present at the site (based on criteria provided in the project geotechnical report) (Requirement 5-0166).

F. Precipitation Loads (S, R, and D_i)

1. Snow Loads (S): Determine S using the procedure prescribed in Chapter 7.0 of ASCE 7 as amended by the following (Requirement 5-0167):

   a. ASCE 7 Section 7.2 Ground Snow Loads, p_g, substitute the following text: p_g = 32 psf (Requirement 5-0168).

   b. ASCE 7 Section 7.3.3 *Importance Factor, I_s*, substitute the following text: I_s = 1.0 (Requirement 5-0169).

2. Rain Loads (R): Determine R using the procedure prescribed in Chapter 8.0 of ASCE 7 as amended by the following (Requirement 5-0170):

   a. ASCE 7 Section 8.3 Design Rain Loads, add the following text: Use 4.6 inches/24 hours as the rainfall rate used to determine R (Requirement 5-0171).

   b. In addition, all of the provisions of ASCE 7 Chapter 8 shall be complied with/evaluated (Requirement 5-0172).

   c. Finally, roofs shall have positive drainage, and shall be equipped with secondary drains or scuppers (Requirement 5-0173).

3. Ice Loads (D_i, Atmospheric Ice Weight): D_i shall be considered for ice-sensitive structures (i.e., structures of relatively small weight and large exterior surface areas, such as lattice structures, guyed masts, open catwalks and platforms, signs, etc.) (Requirement 5-0174). Thus, D_i need not be considered for buildings and building-like structures.

4. Determine D_i using the procedure prescribed in Chapter 10.0 of ASCE 7 as amended by the following (Requirement 5-0175):

   a. Use 0.25 inch for t, the nominal ice thickness due to freezing rain (Requirement 5-0176).

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[10] Refer to Para. E in Appendix E herein for lateral soil pressure loads that result from earthquake.

[11] Average value for K_0 for cohesionless soils per Bowles (Foundation Analysis and Design, 3rd Ed.; Fig. 11-2), and a reasonable value for normally consolidated clays per Winterkorn & Fang (Foundation Engineering Handbook, pp. 488).

b. Use 2.5 for I_i, the importance factor or multiplier on ice thickness (Requirement 5-0177).

c. The wind load concurrent with ice loading, W_i, shall be based on a 40-mph 3-second-gust wind speed (Requirement 5-0178).

d. Use 1.0 for I_w, the importance factor for wind on ice-covered SSCs (Requirement 5-0179).

G. Thermal Loads (T_o and T_a)

1. Determine T_o (i.e., normal loads encountered during normal operating, start-up, or shutdown conditions) and T_a (i.e., abnormal loads generated by a postulated accident, including T_o), based on the most critical time-dependent and /or position-dependent temperature variations that are applicable to the project-specific requirements (Requirement 5-0180).

2. The use of "loads" herein applies to internal moments and forces caused by temperature distributions within the structure (and other temperature-induced loads in the case of T_o) (Requirement 5-0181).

3. In the design/analysis of concrete structures, T_o and T_a shall be considered in accordance with/as required by ACI 349 Appendix E (Requirement 5-0182).

   a. An acceptable alternative for considering the effects of T_o and T_a is ACI 349.1R, *Reinforced Concrete Design for Thermal Effects on Nuclear Power Plant Structures*.[12]

H. Pipe/Equipment Reactions (R_o and R_a)

1. Determine R_o (i.e., normal loads encountered during normal operating, start-up, or shutdown conditions, excluding D and earthquake reactions) and R_a (i.e., abnormal loads generated by a postulated accident and including R_o), based on the most critical transient or steady-state condition that is applicable to the project-specific requirements (Requirement 5-0183).

   a. The use of "loads" herein applies to piping and equipment reactions, or related internal moments and forces (Requirement 5-0184).

I. Operating Basis Earthquake (E_o)

1. The operating basis earthquake is applicable to nuclear power plants (i.e., it's not applicable at LANL). *Refer to ACI 349 Definitions and AISC N690 Glossary for more detail.*

J. Wind Loading (W)

1. Determine W using the procedure prescribed in Chapter 26.0 of ASCE 7 as amended by the following (Requirement 5-0185):

   a. ASCE 7 Section 26.5.1 Basic Wind Speed: Substitute the following text: Use 120 mph for the basic wind speed, V, used in the determination of W on buildings and other structures (Requirement 5-0186).

   b. ASCE 7 Section 26.7.3 Exposure Categories: Substitute the following text: Exposure C shall apply for all cases (Requirement 5-0187).

[12] The reason for this allowance is, as indicated at the outset of 349.1R, 349 App. E is intended for nuclear power plant structures, while 349.1R is not restricted to such.

c. Modify ASCE 7 Section 26.10.2, Openings, by adding the following sentence (Requirement 5-0188):

Any opening in the building envelope created by a missile breach(s) that exceeds one (1) square foot per 1,000 cubic feet of volume shall be included in the determination of the amount of openings (Requirement 5-0189).

d. Replace ASCE 7 Section 26.10.3 Protection of Openings, with the following (Requirement 5-0190):

26.10.3 Straight-Line-Wind Missiles
Structures and structural systems shall be designed to prevent breaching, or else protected from impact, by the following two (2) types of missiles (Requirement 5-0191):

26.10.3.1 Small/ Lightweight Missile
One-inch-diameter, solid steel sphere, weighing 0.147 lb, 12-mph horizontal velocity, impact height unlimited (Requirement 5-0192).

26.10.3.2 Medium/ Medium-Weight Missile
ASTM A53 Gr B Pipe 6 Std, 15-feet long, 48-mph horizontal velocity, impact height $\leq$ 30 ft above grade (Requirement 5-0193).

K. Design Basis Earthquake (E, E_s, or E_{ss})

1. Refer to Appendix E of this document for DBE loads and related requirements (Requirement 5-0193a).

L. Tornado Loading (W_t)

1. W_t shall be considered for structures that are to be sealed; however, only with regard to the combination of W_t acting simultaneously with atmospheric pressure change (APC) as described subsequently (Requirement 5-0194).

2. W_t = maximum external pressure acting away from the structure determined using the procedure described in the previous paragraph, Wind Loading (W), as amended by the following:

   a. Use V = 90 mph (Requirement 5-0195).

3. APC = internal pressure acting outward on the structure shall be taken as 0.2 psi (Requirement 5-0196).

4. Ensure that the structure is designed to withstand whichever of the following two (2) load combinations produces the largest demand (Requirement 5-0197):

   a. $\frac{1}{2}APC + W_t$

   b. $APC + \frac{1}{2}W_t$

M. Differential Pressure (P_a)

1. Determine P_a (i.e., abnormal load generated by a postulated accident) based on the project-specific requirements (Requirement 5-0198).

a. The use of "load" herein applies to maximum differential pressure load, or related internal moments and forces (Requirement 5-0199).

N. Impulsive and Impactive Loads (Y_r, Y_j, Y_m)

1. Determine Y_j, Y_m, and Y_r (i.e., abnormal loads generated by a postulated accident) based on the project-specific requirements (Requirement 5-0200).

   a. The use of "loads" herein applies to the specific type of load as follows (Requirement 5-0201):

      Y_j = Jet impingement load, or related internal moments and forces, on the structure.

      Y_m= Missile impact load, or related internal moments and forces, on the structure.

      Y_r = Loads, or related internal moments and forces, on the structure generated by the reaction of the broken high-energy pipe (during the postulated accident).

2. For more detail on these loads, specific examples of them, how to design for them, etc., refer to ACI 349 and AISC N690.

O. Self-Straining Forces

1. In the design for normal loads, consideration shall be given to the forces due to such effects as ambient temperature change, prestressing, vibration, impact, shrinkage, creep, unequal settlement of supports, construction, and testing (Requirement 5-0202).

   a. Unless specifically addressed through analysis, the effects of self-straining forces shall be accommodated by placement of relief joints, suitable framing systems, or other details (Requirement 5-0203).

2. Where the structural effects of differential settlement, creep, shrinkage, or expansion of shrinkage-compensating concrete are significant, they shall be included with the dead load, D, in the load-combination equations (Requirement 5-0204). Estimation of these effects shall be based on a realistic assessment of such effects occurring in service (Requirement 5-0205).

P. Flood Loads

1. FDC-3 structures and structural systems shall be designed such that the following requirements are met[13]:

   a. They shall be able to withstand flood loads resulting from Design Basis Flood Levels (DBFLs) using the load combinations in ACI 349 and AISC N690 wherein each applicable flood load shall be considered an "Extreme Load" (Requirement 5-0206).

   b. When SSCs that may fail due to water intrusion or submergence cannot be located above the Design Basis Flood Level (DBFL), consider the other mitigation strategies in "1020" paragraph 5.5.4 (Requirement 5-0207).

[13] Flood design per "1020" requires PFHA, FSA and, perhaps, a CFHA. Until such time that these exist, LA-14165 Para. 2.6.2 is all that's available for determining Design Basis Flood Levels (DBFLs) and flood loads. Included therein are some of what's required for design for this NPH subject to the following changes: "DOE-STD-1020," "DOE-STD-1023," "ASCE 24-98," ASCE 4-98," "ASCE 7-98," and "IBC 2003" be taken as (=) DOE-STD-1020-2016; PC-3 = FDC-3; and the Source of flooding in Table 2-19 will be edited to read in accordance with "1020" Table 5-4 (i.e., add "1020" Case Nos. 3 and 4 to Levee/Dam Failure in Table 2-19, and delete Case No. 3 from Local Precipitation in Table 2-19).

Q. Volcanic Eruption Loads

1. VDC-3 SSC design is not required (Requirement 5-0208).

2.3 Load Combinations

A. Reinforced Concrete and Reinforced Masonry

1. The load combinations of ACI 349 Chapter 9 shall be used to combine demands for reinforced concrete and masonry structures and structural systems as amended by the following (Requirement 5-0209):

   a. The term E_{ss} in ACI 349 Equations 9-6 and 9-9 shall be replaced by E_{ss}/F_{μ}, where F_{μ} is the inelastic energy absorption factor (addressed subsequently herein) (Requirement 5-0210).

   b. NPH loads shall be considered "extreme environmental loads" (Requirement 5-0211).

2. The load combinations in ACI 530 are omitted (Requirement 5-0212).

B. Structural Steel Members, Cold-Formed Steel and Stainless Steel

1. The load combinations of AISC N690 shall be used to combine demands for structural steel, cold-formed steel and stainless steel structure and structural systems as amended by the following (Requirement 5-0213):

   a. The term E_s in AISC N690 Equations NB2-6, NB2-9, NB2-15 and NB2-18 shall be replaced by E_{ss}/F_{μ}, where F_{μ} is the inelastic energy absorption (addressed subsequently herein) (Requirement 5-0214).

   b. NPH loads shall be considered "extreme environmental loads" (Requirement 5-0215).

   c. The AISC N690 "severe environmental load combinations" shall be used for consideration of concurrently-acting D_i and W_i subject to the following modifications: D_i shall be added to D, and W_i shall be taken to be W (Requirement 5-0216).

2. The load combinations in ASCE 8 are omitted (Requirement 5-0217).

3. The load combinations in AISI Standard S100 are omitted (Requirement 5-0218).

C. Inelastic Energy Absorption Factor (F_{μ})

1. The factor F_{μ} selected for use shall be as stipulated in ASCE 43 (Requirement 5-0219).

2. For F_{μ} for masonry and cold-formed steel members use 1.0 (Requirement 5-0220).

3. F_{μ} for stainless steel members shall be the same as that used for structural steel members in ASCE 43 provided that the ductility provisions of DOE-STD-1020 Section 3.6.1 are met (Requirement 5-0221).

2.4 Analysis Procedures

A. The structural analysis shall comply with the requirements in the material design standards identified previously (Requirement 5-0222).

B. Seismic modeling and analyses shall comply with the requirements of ASCE 43 and ASCE 4 (Requirement 5-0223).

C. Time histories shall comply with the requirements of ASCE 43 (Requirement 5-0224).

2.5 Capacities

A. ASCE 43 Section 4.2 shall be used for determining capacities as amended by the following (Requirement 5-0225):

1. 4.2.2, Reinforced Concrete: Add the following text:

   a. Delete ACI 349 Section 9.2.10 (Requirement 5-0226).

   b. Design of anchorage to concrete shall be in accordance with Appendix A of this Section (Requirement 5-0227).

2. 4.2.5, Reinforced Masonry: Add the following text:

   a. Post-installed (PI) anchor bolts shall not be used (Requirement 5-0228).

2.6 Detailing Requirements

A. Comply with any/ all applicable portions of ASCE 43 Section 6, *Ductile Detailing Requirements* (Requirement 5-0229).

B. Structural steel designs shall allow for/ensure the following (Requirement 5-0230):

1. Each column anchored with a minimum of four (4) anchor rods.

2. Each column base plate assembly, including the column-to-base plate weld and the column foundation, designed to resist a minimum eccentric gravity load of 300 pounds located 18 inches from the extreme outer face of the column in each direction at the top of the column shaft.

2.7 Additional Structural Design Considerations

A. **Foundation Design**

1. Detailing of NDC-3 building foundations shall meet the following requirements:

   a. Minimum embedment depth of foundations is 36 inches unless the foundation bears directly on welded tuff (Requirement 5-0231).

   b. Interconnect all SDC-3 spread footing type foundations using tie beams (Requirement 5-0232). The tie beam shall be capable of resisting, in tension or compression, a minimum horizontal force equal to 10% of the larger column vertical load (Requirement 5-0233). The tie beams shall also be capable of resisting bending due to prescribed differential settlements of the interconnected footings as stipulated by the project geotechnical report and to eccentric positioning of columns and corresponding column loads on spread footings, simultaneously with the horizontal force (Requirement 5-0234).

B. **Common-Cause Failure and System Interaction**

1. In addition to being addressed in “1020” paragraph 2.3.2(b), requirements and information on common-cause failure and system interaction (more casually referred to as “two-over-one”) are also found therein in paragraph

3.6.3; Section I of this Chapter in the paragraphs "DOE Natural Phenomena Hazard Mitigation Requirements" and "Applicability;" ASCE 43 Section C8.3; and in "1020" companion document DOE-HDBK-1220 (*Appendices A and C in 2017*).

C. **Progressive Collapse**

1. Refer to Section II of this Chapter in paragraph "Add Section 1617 Minimum Antiterrorism Structural Design Measures" (Requirement 5-0235).

D. **Permanent Explosive Facilities and Facilities Containing Explosives, or Those that can be Affected by Such**

1. Refer to Section II of this Chapter in paragraph "Add Section 1616 Accidental Blast Loads" (Requirement 5-0236).

E. **Additional Seismic Analysis/ Design Considerations**

1. Comply with any/all applicable portions of ASCE 43 Chapter 7, *Special Considerations* (Requirement 5-0237).

3.0 NDC-3 NONSTRUCTURAL SYSTEMS AND COMPONENTS

A. In addition to complying with the applicable portions of "1020," also comply with the applicable portions of the paragraph "Designated Seismic Systems" in Section I of this Chapter (Requirement 5-0238).

B. NDC-3 Non-Facility Equipment (e.g., Programmatic, Utilities, Infrastructure, Environmental Remediation, etc.) shall be restrained in accordance with Appendix G, Restraint of Non-Facility Components, herein (Requirement 5-0239).

C. As applicable, the following might be helpful in ensuring compliance with DOE-STD-1020:

1. ASCE 4 content: Buried Pipes and Conduits, Vertical Liquid Storage Tanks, and Distribution Systems.

2. ASCE 43 content: Rocking and Sliding of Unanchored Bodies, and Unreinforced Masonry Walls Used for Partitions, Barriers or Radiation Shielding.

4.0 EXISTING HC 1 – 3 NUCLEAR FACILITIES WITH SSCS IN NDC-3

A. Refer to Section I of this Chapter, "DOE Natural Phenomena Hazard Mitigation Requirements" paragraph (Requirement 5-0240).

B. DOE-STD-1020 Paragraph 3.5.3 requires the use of in-structure response spectra (ISRS) for determining the seismic demand on equipment. The ISRS for LANL's TA-55-0004 (or "PF4") are provided in SB-DO:CALC-10-014 (UCNI; available in EDRMS).

5.0 QUALITY CONTROL, QUALITY ASSURANCE AND PEER REVIEW

A. Refer to Section I of this Chapter, "Project Quality Assurance" paragraph (Requirement 5-0241).

6.0 RECORD OF REVISIONS

Rev	Date	Description	POC	OIC
0	6/28/99	Initial issue in Facility Eng Manual.	Doug Volkman, *PM-2*	Dennis McLain, *FWO-FE*
1	2/09/04	Incorporated IBC & ASCE 7 in place of UBC 97; incorporated DOE-STD-1020-2002 versus 1994; incorporated concepts from DOE O 420.1A; FEM became ESM, an OST.	Mike Salmon, *FWO-DECS*	Gurinder Grewal, *FWO-DO*
2	5/17/06	Revised load combos since ACI 349 is specified for PC-3 R/C structures and the load factors in the current version of ASCE 7 (used for load combos) are inconsistent with the strength reduction factors in the current version of ACI 349, including new section 1.1.13 on crane and pipe restraint loads, companion table to go with former Table III-6 (i.e., Table III-6 became Tables III-6 and -7. OST became ISD.	Mike Salmon, *D-5*	Mitch Harris, *ENG-DO*
3	10/27/06	Administrative changes only. Organization and contract reference updates from LANS transition, 420.1A became 1B. Master Spec number/title updates. Became ISD, other administrative changes.	Mike Salmon, *D-5*	Kirk Christensen, *CENG*
4	6/19/07	Incorporated new seismic hazard analysis results into the DBE Response Spectra. Added Appendices A & B for concrete anchor design.	Mike Salmon, *D-5*	Kirk Christensen, *CENG*
5	6/20/11	Major revision. Added 1189 requirements; removed PC-4 requirements; new response spectra.	Mike Salmon, *D-5*	Larry Goen, *CENG*
6	3/27/15	Major revision. Incorporated DOE O 420.1C Chg 1 and DOE-STD-1020-2012 versus 2002. Eliminated historical 10-psf future-floor-DL and, for roofs, 30-psf min roof LL (Lr) and prohibition on LL reduction. Created Apps C & D for LS and DBE loads.	Mike Salmon, *AET-2*	Larry Goen, *ES-DO*
7	08/28/18	Replaced Drillco Maxi-Bolt with Hilti HDA in App A and B.	Mike Salmon, *AET-2*	Larry Goen, *ES-DO*
8	08/05/19	Added Hilti KB-TZ and HIT-RE 500 V3 to App A, KB-TZ as App. C. App C–D became D–E.	Mike Salmon, *E-1*	James Streit, *ES-DO*
9	03/24/21	Incorporated DOE O 420.1C Chg 3, DOE-STD-1020-2016, and ASCE 4-16 and ASCE 43-19 (adding App F); new App G for programmatic restraint, other minor changes. Many basis footnotes moved to requirements ID document.	Mike Salmon, *ALDFO*	James Streit, *ES-DO*
10	07/19/21	Added Hilti KB-TZ2 to App. A; new App. H for design for same. Crane guidance expanded in App E.	Mike Salmon, *ALDFO*	James Streit, *ES-DO*

APPENDIX A: DESIGN OF ANCHORING TO CONCRETE

A.1 Description

A. This Appendix establishes the technical requirements for design/analysis for structural embedments (used to transmit structural loads) in concrete for NDC-3 SSCs.

B. Included: anchors, embedded plates, shear lugs, grouted embedments, and specialty inserts.

1. Cast-in-place (CIP) anchors include headed bolts, threaded and nutted bolts, headed studs, and hooked bolts. CIP anchors are of ASTM A36, A193, A354 Gr BD, A449, A572, A588, or F1554 material (Requirement 5-0242). ASTM F1554 is the preferred material specification in AISC N690. Welding and mechanical properties of headed studs shall comply with AWS D1.1 (per AISC N690) and ESM Chapter 13, *Welding, Joining, and NDE* (Requirement 5-0243).

2. Allowed post-installed (PI) anchors are the Hilti HDA undercut anchor (i.e., HDA) (Requirement 5-0244), KB-TZ expansion anchor (i.e., KB-TZ) (Requirement 5-0245), KB-TZ2 expansion anchor (i.e., KB-TZ2) (Requirement 5-0245a), and HIT-RE 500 V3 Adhesive anchor (i.e., V3) (Requirement 5-0246).

    a. Caution: Prior to specifying the KB-TZ, check onsite or suppler stocking levels since Hilti has announced that production will cease as of July 21, 2021.

Table III.A-1 NDC-3 PI Anchor Key Attributes

Model	Positives	Negatives
HDA	• undercut designs are often preferred in nuclear industry • stainless is available for harsher environments • no derating required for LANL use	• concrete depth of 7 inches for smallest size
KB-TZ	• simplest design & installation	• weaker than HDA • must derate by 25% for LANL use • cannot use stainless version since not nuclear qualified[1] • availability is constrained (Caution above)
KB-TZ2	• replacement for KB-TZ • stainless is available for harsher environments • no derating required for LANL use	• weaker than HDA
V3	• most flexibility with anchor steel ○ many more types and sizes, non-proprietary, and threaded rod or rebar.	• weaker than HDA • onerous to design & install

3. HDA

    a. Lengths: As indicated in Appendix B herein, with the exception of the smallest-diameter anchor (i.e., "M10"), the HDA is available in two lengths to accommodate various fixture/baseplate thicknesses.

[1] So use limited to indoor, dry environment

b. HDA material and set-types availability is as follows:

Table III.A-2 Off-the-Shelf Material and Set-Type Combinations

Material	Pre-set (P) (set prior to fixture)	Through-set (T) (set after/with fixture)
high-strength carbon steel	X	X
stainless steel ("R")	X	X
normal-strength carbon steel ("DUC")	X	See Note 1

Table Note 1: While the HDA DUC is not "off-the-shelf available" in the "T" configuration, Hilti will produce such anchors provided the order size is large enough (e.g., 2000–4500 pieces). The production lead-time for the HDA-T DUC is approximately 10 weeks.

4. KB-TZ
   a. Lengths: As indicated in Appendix C herein.
   b. Material type is high-strength carbon steel.

5. KB-TZ2
   a. Lengths: As indicated in Appendix H herein.
   b. Material types are high-strength carbon steel and stainless steel.

6. V3
   a. Lengths: As indicated in ICC-ES ESR-3814.
   b. Material types: As indicated in ICC-ES ESR-3814.

7. Purchase, installation, and testing requirements for NDC-3 anchors:
   a. For the HDA, KB-TZ, and KB-TZ2, see LANL Master Specification Section 05 0521 (Requirement 5-0247).
   b. For the V3, in coordination with the Chapter POC and LANL IQPA, develop a project-specific construction specification section that combines, as appropriate, portions of LANL Master Specification Sections 05 0520 and 05 0521 (Requirement 5-0248).

C. Not Included: Through bolts; multiple anchors connected to a single steel plate at the embedded end of the anchors; direct anchors such as powder or pneumatic actuated nails or bolts; and PI anchors for commercial, and NDC-1 and NDC-2 applications. The latter anchors are included in Section II, Appendix A of this Chapter.

A.2 Definitions

A. Definitions of anchors per ACI 355.2 and ACI 355.4 apply (Requirement 5-0249).

B. Unless noted otherwise, all variable notations (and their definitions) in this appendix follow ACI 349, Appendix D (Requirement 5-0250).

A.3 Applicable Codes and Standards (Requirement 5-0251)

ACI 318 Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary

ACI 355.2	Qualification of Post-Installed Mechanical Anchors in Concrete (ACI 355.2) and Commentary
ACI 355.4	Qualification of Post-Installed Adhesive Anchors in Concrete (ACI 355.4) and Commentary
ACI 349	Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-13) and Commentary

A.4 Applicable Industry Standards

ASME NQA-1	Quality Assurance Requirements for Nuclear Facility Applications, American Society of Mechanical Engineers (Requirement 5-0252)

A.5 LANL Documents

LANL Master Spec Section 03 6021	Grouting–High Confidence (Requirement 5-0253)
LANL Master Spec Section 05 0521	Post-Installed Concrete Anchors Purchase–Nuclear Safety (Requirement 5-0254)
ESM Chapter 13	Welding, Joining, and NDE (Requirement 5-0255)
LANL Memo ES-DO-18-015 Rev.1	LANL Building Official Rescinded Approval of Drillco Maxi-Bolt Post-Installed Concrete Anchors (Requirement 5-0256)

A.6 Prerequisites for Determining Anchor Design Loads

A. The SEOR shall coordinate with appropriate personnel to determine and document the NPH Design Category (i.e., NDC) of the anchorage and, where NDC = SDC, the Limit State (LS) as well (Requirement 5-0257). The NDC (or SDC+LS) shall be determined by considering both of the following:

1. The safety classification of the item or system being anchored (i.e., its required performance/functionality) (Requirement 5-0258), and

2. System interaction: refer to Section II, Appendix A of this chapter (in the sub*paragraph on the value of I_p within the paragraph, Prerequisites for Determining Anchor Design Loads*) (Requirement 5-0259).

B. Engineering drawings shall indicate the assigned NDC (or SDC+LS) used to design/analyze the anchorage of an SSC(s) (Requirement 5-0260).

A.7 Determining Anchor Design Loads

A. Loads on anchors shall account for baseplate flexibility, eccentricity of connections, and the dynamic (strain rate and low-cycle fatigue) effects of loads and forces (Requirement 5-0261).

A.8 Environmental Conditions

A. Anchors for indoor use in non-aggressive chemical environments may be carbon steel with a zinc electroplating. Anchors for use outdoors, or in aggressive environments, shall be galvanized, or made of stainless steel (Requirement 5-0262).

B. Adhesive anchors intended for use in elevated temperature and/or radiation environments shall be qualified in those environments (Requirement 5-0263).

A.9 Requirements for Transfer of Shear Load to Foundations

A. For structures where the eave height exceeds 20 feet, provide shear lugs on base plates of columns in the SLRS (Requirement 5-0264). Design the lugs in accordance with ACI 349 Sections D.10 and D.11; and, where there is no conflict, per the recommendations of the most current version of the *AISC Steel Design Guide 1, Base Plate and Anchor Rod Design* (Requirement 5-0265). Where shear lugs are required, they shall be designed to transmit the entirety of shear load in elements of the SLRS to foundation elements (Requirement 5-0266).

A.10 General Design Requirements

A. *CIP embedments (ref. A.1.B herein) should be used in lieu of PI embedments whenever possible, and particularly for resisting heavy loads; and for anchoring rotating, reciprocating, or vibrating equipment such as fans, pumps, and motors. In the event that concrete is already placed due to construction sequencing, contact the Chapter POC for guidance.*

B. PI embedments are used to attach SSCs to hardened concrete where CIP embedments do not exist, or where it is determined to be most effective and efficient to use PI embedments. Cases exist where PI embedments will be specified prior to concrete placement to allow for release of construction documents (such as where fixture/baseplate details, or locations are not known at the time of drawing issue). The decision to specify PI embedments prior to concrete placement should be weighed carefully, considering the amount of construction cost and time required to drill holes, field modify plates, and to avoid cutting rebar.

C. PI embedments and surface-mounted plates are recommended for applications where support requirements are added or modified after concrete placement. PI embedments may be specified on design drawings prior to concrete placement for lightly-loaded items where a CIP embedment is not economical.

D. Pretensioning of CIP anchors: Some examples of applications that might require pretension include structures that cantilever from concrete foundations, moment-resisting column bases with significant tensile forces in the anchor rods, or where load reversal/vibration might result in the progressive loosening of the nuts on the anchor rods. Refer to the AISC Steel Construction Manual, "Anchor Rod Nut Installation;" and/or equipment manufacturers' recommendations (i.e., whenever pretensioning is specified for anchors used for rotating or vibrating equipment), for more detail/guidance.

1. The majority of anchorage applications do not require pretension. In these instances, anchor rod nuts are merely tightened to a snug-tight condition. Snug-tight is defined as tightness attained by a few impacts of an impact wrench, or the full effort of an ironworker with an ordinary spud wrench.

2. When nuts are subject to possible loosening, and prestensioning is not possible/desirable, a locking method should be provided. Acceptable locking methods include: double nuts or jam nuts, interrupted rod threads, and tack welds (complying with the welding requirements of AISC N690).

E. Use of PI anchors shall comply with the following:

1. PI anchors should not be installed through liner plate. Contact the Chapter POC for guidance.

2. PI-anchor design shall provide for at least ± 1 inch fixture/baseplate and anchor relocation (within fixture/baseplate) to facilitate anchor installation (Requirement 5-0267). Providing for the latter might require over-sizing the fixture (to allow for field drilling of holes). Due consideration shall be given to the location tolerances of the anchors to avoid interferences with rebar.

3. Welding to PI anchors is not permitted (Requirement 5-0268).

4. PI anchors shall not be used in masonry walls (Requirement 5-0269). Through-bolting may be an acceptable alternative.

5. A minimum of two (2) PI anchors shall be used at each connection (Requirement 5-0270)[2]. One (1) anchor may be use used for connecting conduit clamps or for similar installations. *Note: equipment, glove boxes, etc. mounted on four-legged-type frames may have one anchor bolt per leg.*

6. PI anchors shall not be located in the bottom of precast and pre/post-tensioned T-beams stems (Requirement 5-0271). PI anchors into the sides of the T-beam stems shall be designed (prior to installation), and the design must be approved by the SEOR (Requirement 5-0272). And, in such designs, anchors shall not be located closer than 6 inches to pre-/post-tensioning steel (Requirement 5-0273).

A.11 Design Requirements

A. For the HDA and KB-TZ2, anchor design strength shall be determined in accordance with ACI 349-13, Appendix D (Requirement 5-0274).

Note: ACI 349, Appendix D, allows for a choice in the selection of strength-reduction factors: Those in paragraph D.4.4 must be used with the load combinations of Section 9.2, while those in paragraph D.4.5 must be used with the load combinations of Section C.9.2.

B. For the KB-TZ, anchor design strength shall be determined in accordance with ACI 349, Appendix D, except subparagraph D.4.1.1 shall be modified to read as follows (Requirement 5-0275):

For the design of anchors

$0.75\varphi N_n \geq N_{ua}$

$0.75\varphi V_n \geq V_{ua}$

Notes: The note at the bottom of the "HDA paragraph" (above) applies here too. And the 0.75 factor here is applicable to nonductile design (i.e., ACI 349 Para. D.3.6.3).

C. For the V3, anchor design strength shall be determined in accordance with ACI 318-14 Chapter 17 and ICC-ES ESR-3814 (Requirement 5-0276).

D. All new PI-anchor design shall be based on "cracked concrete" unless it is analytically proven (and documented) that the concrete remains uncracked (i.e., tensile stresses in concrete do not exceed $7.5\sqrt{f_c'}$) under service loads, including wind and seismic forces (Requirement 5-0277).

E. Appendices B, C, and H (herein), as well as the following, shall apply to the design of HDA and KB-TZ anchors respectively (Requirement 5-0278). Unless noted otherwise, the sections and paragraphs listed in what follows are from ACI 349-13, Appendix D:

D.4.2.1 The effect of reinforcement to restrain concrete breakout shall not be considered (i.e., Condition B in D.4.4 shall be used) (Requirement 5-0279).

D.5.2.6 Use the following $\psi_{c,N}$ value: 1.0 (unless "uncracked" is proven in accordance with paragraph A.11.D above) (Requirement 5-0280).

[2] Per the "4-anchor-minimum provision" in the previous paragraph herein, Detailing Requirements, the use of only two (2) PI anchors doesn't apply to the installation of new structural steel columns.

D.5.3.5 Use the following $\psi_{c,P}$ value: 1.0 (unless "uncracked" is proven in accordance with paragraph A.11.D above) (Requirement 5-0281).

D.6.2.7 Use the following $\psi_{c,V}$ value: 1.0 (unless "uncracked" is proven in accordance with paragraph A.11.D above) (Requirement 5-0282).

D.7 In lieu of Sections D.7.1, D.7.2, and D.7.3, the shear-tension interaction expression given in Section RD.7 may be used with $\alpha = 5/3$.

APPENDIX B: HILTI HDA UNDERCUT ANCHOR DESIGN FIGURES AND TABLES

Figure III.B-1 Pre-setting HDA-P and HDA-PR Anchors (Pre-setting)

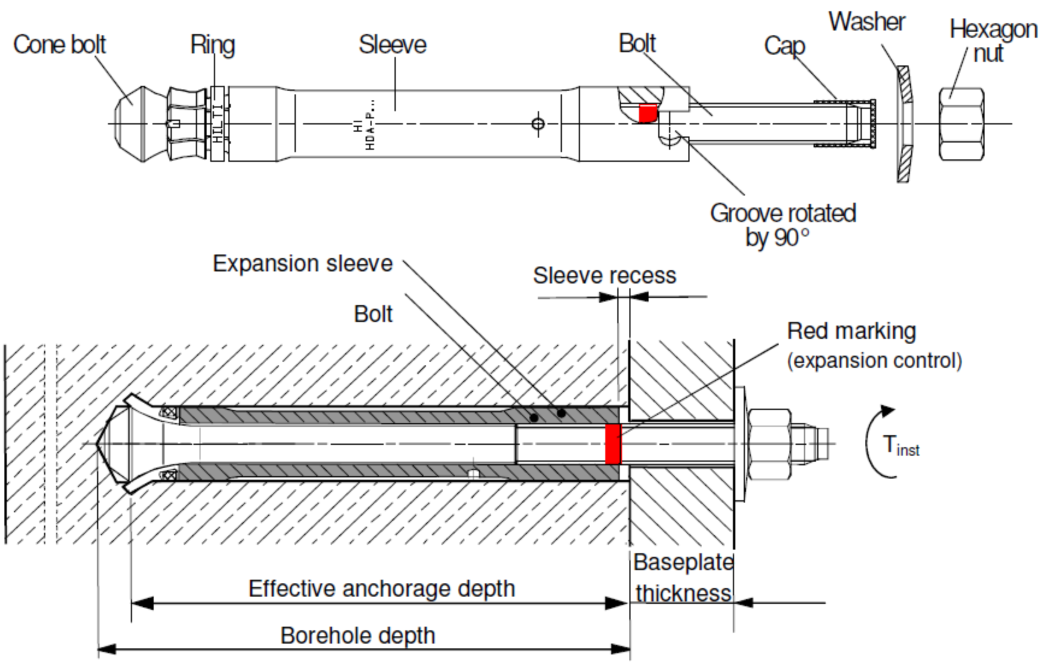

Figure III.B-2 Through-fastening HDA-T and HDA-TR Anchors (Through-fastening)

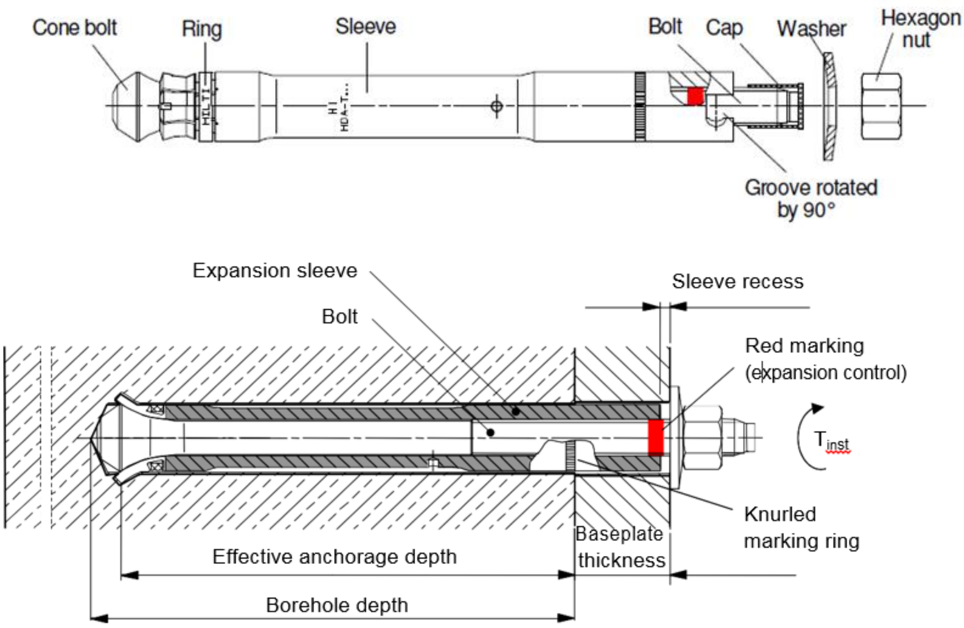

Table III.B-1 Design Information for HDA and HDA-R

Design parameter	Symbol	Units	Nominal anchor diameter						
			M10		M12		M16		M20
			HDA	HDA-R	HDA	HDA-R	HDA	HDA-R	HDA
Anchor diameter	d_a	mm (in.)	19 (0.75)		21 (0.83)		29 (1.14)		35 (1.38)
Effective minimum embedment depth[1, 2]	$h_{ef,min}$	mm (in.)	100 (3.94)		125 (4.92)		190 (7.48)		250 (9.84)
Minimum edge distance[3]	c_{min}	mm (in.)	80 (3-1/8)		100 (4)		150 (5-7/8)		200 (7-7/8)
Minimum anchor spacing	s_{min}	mm (in.)	100 (4)		125 (5)		190 (7-1/2)		250 (9-7/8)
Minimum member thickness	h_{min}	-	See Table III.B-4a for HDA-P and Table III.B-4b for HDA-T						
Strength reduction factor for tension, steel failure modes[4]	ϕ	-	0.75						
Strength reduction factor for shear, steel failure modes[4]	ϕ	-	0.65						
Strength reduction factor for concrete breakout, side-face blowout, pullout or pryout strength[4]	ϕ	-	0.65 for tension loads 0.70 for shear loads						
Yield strength of anchor steel	f_{ya}	lb/in²	92,800 for HDA; 87,000 for HDA-R						
Ultimate strength of anchor steel	f_{uta}	lb/in²	116,000						
Tensile stress area	A_{se}	in²	0.090		0.131		0.243		0.380
Steel strength in tension	N_{sa}	lb	10,431		15,152		28,236		44,063
Effectiveness factor cracked concrete[5]	k_c	-	24						
Modification factor for uncracked concrete[6]	$\psi_{c,N}$	-	1.25						
Pullout strength cracked concrete, static and seismic[7]	$N_{p,cr}$	lb	8,992	8,992	11,240	11,240	22,481	22,481	33,721
Steel strength in shear, static HDA-P/PR[8]	V_{sa}	lb	5,013	6,070	7,284	8,992	13,556	16,861	20,772
Steel strength in shear, seismic[8] HDA-P/PR	$V_{sa,seismic}$	lb	4,496	5,620	6,519	8,093	12,140	15,062	18,659
Axial stiffness in service load range in cracked/uncracked concrete[9]	-	10³ lb/in	80 / 100						

Table III.B-2 Notes

1. Actual h_{ef} HDA-T is given by $H_{ef,\ min} + (t_{fix,max}-t_{fix})$ where $t_{fix,max}$ is given in Table III.B-4b and t_{fix} is the thickness of the part(s) being fastened.
2. To calculate the basic concrete breakout strength in shear, V_b, l equals h_{ef}. In no case shall l exceed $8d_a$. See ACI 349-13 Appendix D, paragraph D.6.2.2.
3. No values for the critical edge distance (c_{ac}) are provided since tension tests deemed to be compliant with ACI 355.2-07 indicated that splitting failure under external load does not affect the capacity of the HDA. On a related note, given this, the value of $\psi_{cp,N}$ (from ACI 349-13 paragraph D.5.2.7) shall be taken as 1.0 in all instances.
4. See ACI 349-13 Appendix D, paragraph D.4.4. For use with the load combinations of ACI 349-13, Section 9.2.
5. See ACI 349-13 Appendix D, paragraphs D.5.2.2 and D.5.2.9. The value of k_c (i.e., 24) is based on testing and assessment deemed to be compliant with ACI 355.2-07.
6. See ACI 349-13 Appendix D, paragraphs D.5.2.6 and D.5.2.9. The value of $\psi_{c,N}$ (i.e., 1.25) was derived from k_{uncr}/k_{cr} = 30/24, and these k-factor values are based on testing and assessment deemed to be compliant with ACI 355.2-07.
7. The pullout strength of the anchor in cracked concrete is governed by anchor displacement under conditions with crack width cycling. In uncracked concrete, pullout does not govern.
8. For HDA-T see Table III.B-2 which follows.
9. Minimum axial stiffness values. Maximum values may be 3 times larger if high-strength concrete is used.

Table III.B-3 Design Information for Steel Strength in Shear for HDA-T and HDA-TR

Anchor Designation		Thickness of base plate(s) t_{fix} mm	Thickness of base plate(s) t_{fix} in.	Steel Strength in Shear, Static V_{sa} lb	Steel Strength in Shear, Seismic $V_{sa,\ seismic}$ lb
Carbon Steel Anchors	HDA-T 20-M10x100	$15 \le t_{fix} < 20$	$^5/_8 \le t_{fix} < ^{13}/_{16}$	13,938	12,589
	HDA-T 22-M12x125	$15 \le t_{fix} \le 20$	$5/8 \le t_{fix} \le 13/16$	16,636	15,062
		$20 \le t_{fix} \le 50$	$13/16 \le t_{fix} \le 2$	18,659	16,636
	HDA-T 30-M16x190	$20 \le t_{fix} \le 25$	$13/16 \le t_{fix} \le 1$	30,574	27,427
		$25 \le t_{fix} \le 30$	$1 \le t_{fix} \le 1\text{-}3/16$	34,621	31,248
		$30 \le t_{fix} \le 35$	$1\text{-}3/16 \le t_{fix} \le 1\text{-}3/8$	38,218	34,396
		$35 \le t_{fix} \le 60$	$1\text{-}3/8 \le t_{fix} \le 2\text{-}3/8$	41,365	37,093
	HDA-T 37-M20x250	$25 \le t_{fix} \le 40$	$1 \le t_{fix} \le 1\text{-}9/16$	45,187	40,690
		$40 \le t_{fix} \le 55$	$1\text{-}9/16 \le t_{fix} \le 2\text{-}1/8$	50,807	45,636
		$55 \le t_{fix} \le 100$	$2\text{-}1/8 \le t_{fix} \le 4$	54,629	49,233
Stainless Steel Anchors	HDA-TR 20-M10x100	$15 \le t_{fix} < 20$	$^5/_8 \le t_{fix} < ^{13}/_{16}$	15,512	13,938
	HDA-TR 22-M12x125	$15 \le t_{fix} \le 20$	$5/8 \le t_{fix} \le 13/16$	20,233	17,985
		$20 \le t_{fix} \le 50$	$13/16 \le t_{fix} \le 2$	22,256	20,008
	HDA-TR 30-M16x190	$20 \le t_{fix} \le 25$	$13/16 \le t_{fix} \le 1$	35,745	32,148
		$25 \le t_{fix} \le 30$	$1 \le t_{fix} \le 1\text{-}3/16$	37,768	33,946
		$30 \le t_{fix} \le 35$	$1\text{-}3/16 \le t_{fix} \le 1\text{-}3/8$	39,566	35,520
		$35 \le t_{fix} \le 60$	$1\text{-}3/8 \le t_{fix} \le 2\text{-}3/8$	40,915	36,869

For pound-inch units: 1 mm = 0.03937 inch, 1 lb_f = 4.45 N.

Table III.B-4 Base Plate Hole Diameter and Minimum Thickness for HDA and HDA-R

P = pre-set (prior to fixture); T = through-set (through after fixture)

HDA M10 to M20 and HDA-R M10 to M16			M10		M12		M16		M20	
			P	T	P	T	P	T	P	T
Hole diameter in base plates	d_h	mm	12	21	14	23	18	32	22	40
		in.	0.47	0.83	0.55	0.91	0.71	1.26	0.87	1.57
Min. thickness of base plates	$t_{fix,min}$	mm	0	15	0	15	0	20	0	25
		in.	0	0.59	0	0.59	0	0.79	0	0.98

For inch units: 1mm = 0.03937 inches

Table III.B-5a Base Plate Maximum Thickness and Concrete Minimum Thickness for HDA-P and HDA-PR

Anchor type			HDA-P M10 HDA-PR M10	HDA-P M12 HDA-PR M12		HDA-P M16 HDA-PR M16		HDA-P M20	
Maximum thickness of base plate(s)	$t_{fix,max}$	mm	20	30	50	40	60	50	100
		in.	0.79	1.18	1.97	1.57	2.36	1.97	3.94
Minimum thickness of concrete member	h_{min}	mm	180	200		270		350	
		in.	7.1	7.9		10.6		13.8	

For inch units: 1 mm = 0.03937 inches

Table III.B-4b Base Plate Maximum Thickness and Concrete Minimum Thickness for HDA-T and HDA-TR

Anchor type			HDA-T M10 HDA-TR M10	HDA-T M12 HDA-TR M12		HDA-T M16 HDA-TR M16		HDA-T M20	
Maximum thickness of base plate(s)	$t_{fix,max}$	mm	20	30	50	40	60	50	100
		in.	0.79	1.18	1.97	1.57	2.36	1.97	3.94
Minimum thickness of concrete member[1]	h_{min}	mm	200 - t_{fix}	230 - t_{fix}	250 - t_{fix}	310 - t_{fix}	330 - t_{fix}	400 - t_{fix}	450 - t_{fix}
		in.	7.9 - t_{fix}	9.1 - t_{fix}	9.8 - t_{fix}	12.2 - t_{fix}	13.0 -t_{fix}	15.7-t_{fix}	17.7 - t_{fix}

For inch units: 1 mm = 0.03937 inches

[1] h_{min} is dependent on the actual thickness of base plate(s) t_{fix}

e.g., HDA-T M12x125/50: t_{fix} = 20 mm ➔ h_{min} = 250 - 20 = 230 mm

t_{fix} = 50 mm ➔ h_{min} = 250 - 50 = 200 mm

Figure III.B-3 Pre-setting HDA-P DUC Anchors (Pre-setting)

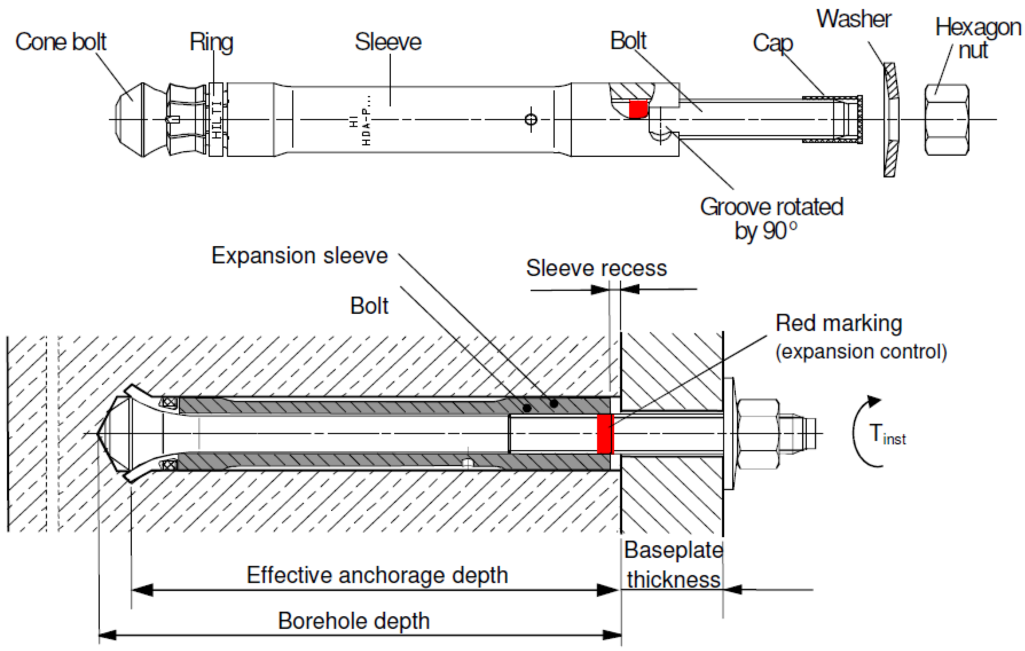

Figure III.B-4 Through-fastening HDA-T DUC Anchors (Through-fastening)

Refer to Appendix A Section A.1 for details on production lead time and minimum-order size.

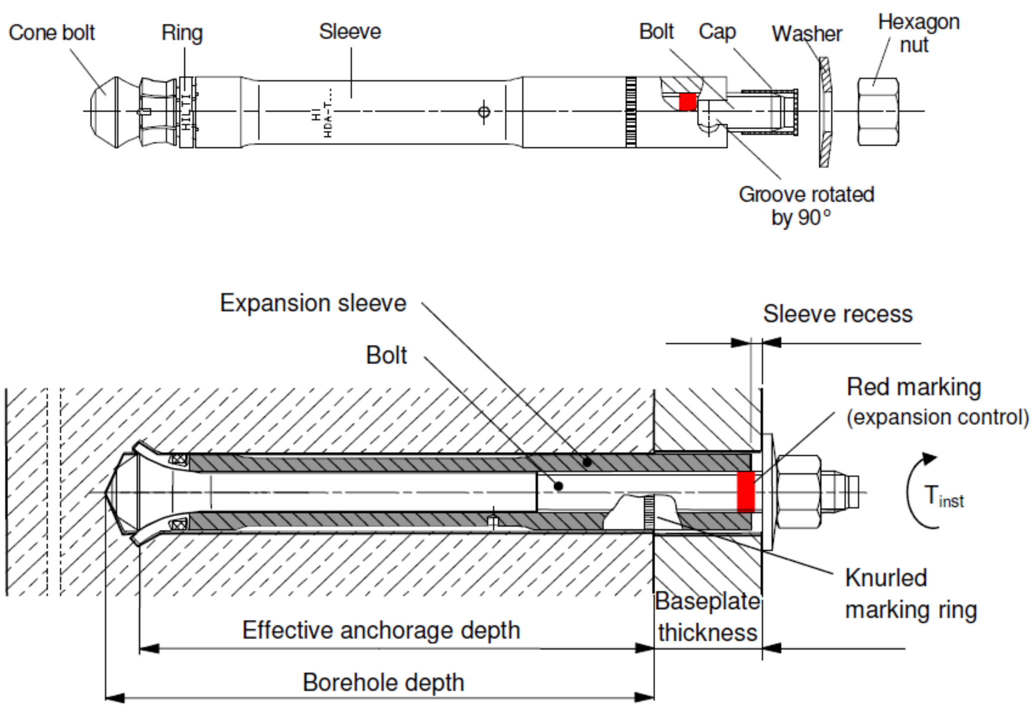

Table III.B-5 Design Information for HDA DUC

Design parameter	Symbol	Units	Nominal anchor diameter		
			M10	M12	M16
			HDA DUC	HDA DUC	HDA DUC
Anchor O.D.	d_a	mm (in.)	19 (0.75)	21 (0.83)	29 (1.14)
Effective min. embedment depth[1,2]	$h_{ef,\ min}$	mm (in.)	100 (3.94)	125 (4.92)	190 (7.48)
Minimum edge distance[3]	c_{min}	mm (in.)	80 (3-1/8)	100 (4)	150 (5-7/8)
Minimum anchor spacing	s_{min}	mm (in.)	100 (4)	125 (5)	190 (7-1/2)
Minimum member thickness	h_{min}	mm (in.)	See Table III.B-8a for HDA-P and Table III.B-8b for HDA-T		
Strength reduction factor for tension, steel failure modes[4]	ϕ	-	0.75		
Strength reduction factor for shear, steel failure modes[4]	ϕ	-	0.65		
Strength reduction factor for concrete breakout, side-face blowout, pullout or pryout strength[4]	ϕ	-	0.65 for tension loads 0.70 for shear loads		
Yield strength of anchor steel[5]	f_{ya}	lb/in^2	34,810		
Ultimate strength of anchor steel[5]	f_{uta}	lb/in^2	58,020		
Tensile stress area	A_{se}	in^2	0.090	0.131	0.243
Steel strength In tension	N_{sa}	lb	5,222	7,600	14,099
Effectiveness factor cracked concrete[6]	k_c	-	24	24	24
Modification factor for uncracked concrete[7]	$\psi_{c,N}$	-	1.25	1.25	1.25
Pullout strength cracked concrete, static and seismic[8]	N_p	lb	Not Applicable		
Steel strength in shear, static HDA-P DUC[9]	V_{sa}	lb	2,958	4,560	8,459
Steel strength in shear, seismic HDA-P DUC[9]	$V_{sa,\ seismic}$	lb	2,426	4,560	8,459
Axial stiffness in service load range in cracked/uncracked concrete[10]	β	10^3 lb/in.	80 / 100		

Table III.B-5 Notes

1. Actual h_{ef} for HDA-T is given by $h_{ef,min}$ + ($t_{fix,max}$ – t_{fix}) where $t_{fix,max}$ is given in Table III.B-8b and t_{fix} is the thickness of the part(s) being fastened.
2. To calculate the basic concrete breakout strength in shear, V_b, l equals h_{ef}. In no case shall l exceed 8d_a. See ACI 349-13 Appendix D, paragraph D.6.2.2.
3. No values for the critical edge distance (c_{ac}) are provided since tension tests deemed to be compliant with ACI 355.2-07 indicated that splitting failure under external load does not affect the capacity of the HDA DUC. On a related note, given this, the value of $\psi_{cp,N}$ (from ACI 349-13 paragraph D.5.2.7) shall be taken as 1.0 in all instances.
4. See ACI 349-13 Appendix D, paragraph D.4.4. For use with the load combinations of ACI 349-13, Section 9.2.
5. Specified minimums.
6. See ACI 349-13 Appendix D, paragraphs D.5.2.2 and D.5.2.9. The value of k_c (i.e., 24) is based on testing and assessment deemed to be compliant with ACI 355.2-07
7. See ACI 349-13 Appendix D, paragraphs D.5.2.6 and D.5.2.9. The value of $\psi_{c,N}$ (i.e., 1.25) was derived from k_{uncr}/k_{cr} = 30/24, and these k-factor values are based on testing and assessment deemed to be compliant with ACI 355.2-07.
8. Pullout strength of the anchor is not applicable since pullout failures did not occur in the qualification tests. Typical failure mode in tension was failure of the steel cone bolt.
9. For HDA-T see Table III.B-6.
10. Minimum axial stiffness values. Maximum values may be 3 times larger if high-strength concrete is used.

Table III.B-6 Design Information for Steel Strength in Shear for HDA-T DUC

Refer to Appendix A Table III.A-1 for details on production lead time and minimum-order size.

Anchor Designation		Thickness of base plate(s) t_{fix}		Steel Strength in Shear, Static V_{sa}	Steel Strength in Shear, Seismic $V_{sa, seismic}$
		mm	in.	lb	lb
Carbon Steel Anchors	HDA-T DUC M10x100	$15 \le t_{fix} < 20$	$5/8 \le t_{fix} < 13/16$	12,092	9,150
	HDA-TOUC M12x125	$15 \le t_{fix} < 20$	$5/8 \le t_{fix} \le 13/16$	12,092	10,883
		$20 \le t_{fix} \le 50$	$13/16 \le t_{fix} \le 2$	13,633	12,269
	HDA-T DUC M16x190	$20 \le t_{fix} < 25$	$13/16 \le t_{fix} \le 1$	25,164	17,615
		$25 \le t_{fix} \le 30$	$1 \le t_{fix} \le 1\text{-}3/16$	28,603	20,022
		$30 \le t_{fix} < 35$	$1\text{-}3/16 \le t_{fix} \le 1\text{-}3/8$	31,497	22,048
		$35 \le t_{fix} \le 60$	$1\text{-}3/8 \le t_{fix} \le 2\text{-}3/8$	34,051	23,836

For pound-inch units: 1 mm = 0.03937 inch, 1 lbf = 4.45 N.

Table III.B-7 Base Plate Hole Diameter and Minimum Thickness for HDA DUC

For "T," refer to Appendix A Table III.A-1 for details on production lead time and minimum-order size

HDA DUC M10 to M16			M10		M12		M16	
			P	T	P	T	P	T
Hole diameter in base plate(s)	d_h	mm	12	21	14	23	18	32
		(in.)	(0.47)	(0.83)	(0.55)	(0.91)	(0.71)	(1.26)
Min. thickness of base plate(s)	$t_{fix, min}$	mm	0	15	0	15	0	20
		(In.)	0	(0.59)	0	(0.59)	0	(0.79)

For in-lb units: 1mm = 0.03937 inches

Table III.B-8a Base Plate Maximum Thickness and Concrete Minimum Thickness for HDA-P DUC

Anchor type			HDA-P DUC M10	HDA-P DUC M12		HDA-P DUC M16	
Maximum thickness of base plate(s)	$t_{fix, max}$	mm	20	30	50	40	60
		in.	0.79	1.18	1.97	1.57	2.36
Minimum thickness of concrete member	h_{min}	mm	180	200		270	
		in.	7.1	7.9		10.6	

For inch units: 1 mm = 0.03937 Inches

NOTE: Additional lead time is associated with M12/50, and even more so with M16/60.

Table III.B-8b Base Plate Maximum Thickness and Concrete Minimum Thickness for HDA-T DUC

Refer to Appendix A Table III.A-1 for details on production lead time and minimum-order size

Anchor type			HDA-T DUC M10	HDA-T DUC M12		HDA-T DUC M16	
Maximum thickness of base plate(s)	$t_{fix, max}$	mm	20	30	50	40	60
		in.	0.79	1.18	1.97	1.57	2.36
Minimum thickness of concrete member[1]	h_{min}	mm	200- t_{fix}	230- t_{fix}	250- t_{fix}	310 - t_{fix}	330- t_{fix}
		in.	7.9- t_{fix}	9.1 - t_{fix}	9.8- t_{fix}	12.2- t_{fix}	13.0 - t_{fix}

For inch units: 1 mm = 0.03937 inches

[1] h_{min} is dependent on the actual thickness of base plate(s) t_{fix}

e.g.,HDA-T M12x125/50: $t_{fix.} = 20$ mm → $h_{min} = 250 - 20 = 230$ mm

$t_{fix} = 50$ mm→ $h_{min} = 250 - 50 = 200$ mm

APPENDIX C: HILTI KB-TZ EXPANSION ANCHOR DESIGN FIGURES AND TABLES

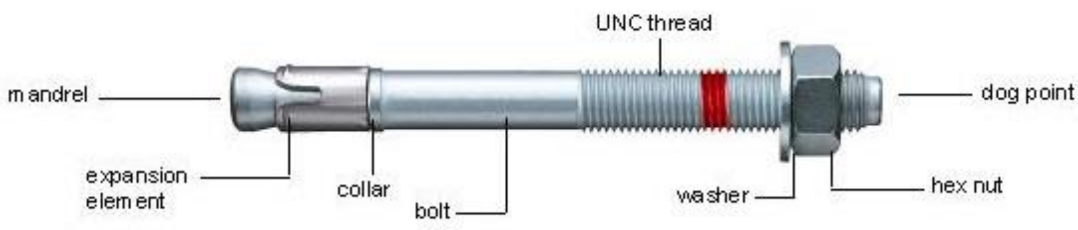

Figure III.C-1 KB-TZ Anchor

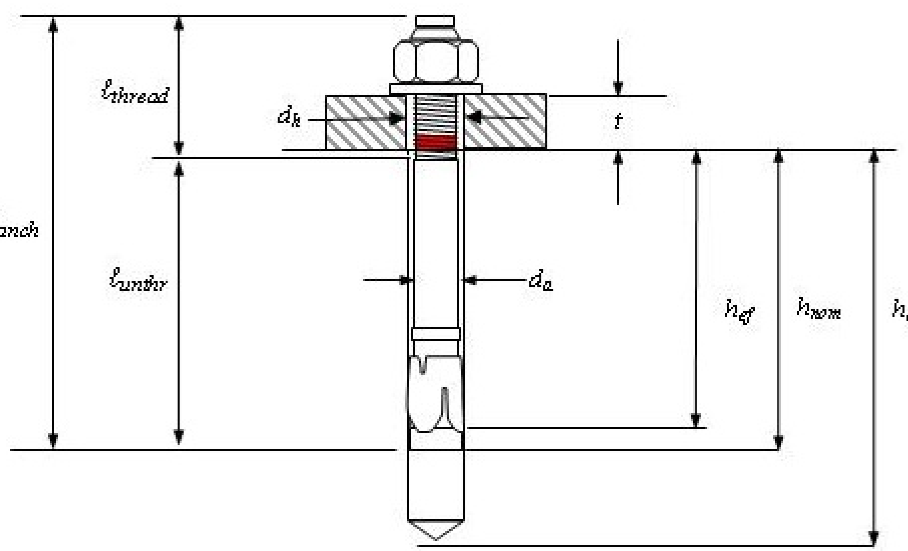

Figure III.C-2 KB-TZ Anchor Installed

Table III.C-1 KB-TZ Design Information

DESIGN INFORMATION	**Symbol**	**Units**	**Nominal anchor diameter**											
			3/8		**1/2**				**5/8**			**3/4**		
Anchor Diameter	d_a	in. (mm)	0.375 (9.5)		0.5 (12.7)				0.625 (15.9)			0.75 (19.1)		
Effective min. embedment depth	$h_{ef,\ min}$	in. (mm)	2 (51)		2 (51)		3-1/4 (83)		3-1/8 (79)	4 (102)		3-3/4 (95)		4-3/4 (121)
Min. member thickness	h_{min}	in. (mm)	4 (102)	5 (127)	4 (102)	6 (152)	6 (152)	8 (203)	5 (127)	6 (152)	8 (203)	6 (152)	8 (203)	8 (203)
Critical edge distance	c_{ac}	in. (mm)	4-3/8 (111)	4 (102)	5-1/2 (140)	4-1/2 (114)	7-1/2 (191)	6 (152)	6-1/2 (165)	8-3/4 (222)	6-3/4 (171)	10 (254)	8 (203)	9 (229)
Min. edge distance[1]	c_{min}	in. (mm)	2-1/2 (64)		2-3/4 (70)		2-3/8 (60)		3-5/8 (92)	3-1/4 (83)		4-3/4 (121)		4-1/8 (105)
	for s ≥	in. (mm)	5 (127)		5-3/4 (146)		5-3/4 (146)		6-1/8 (156)	5-7/8 (149)		10-1/2 (267)		8-7/8 (225)
Min. anchor spacing[1]	s_{min}	in. (mm)	2-1/2 (64)		2-3/4 (70)		2-3/8 (60)		3-1/2 (89)	3 (76)		5 (127)		4 (102)
	for c ≥	in. (mm)	3-5/8 (92)		4-1/8 (105)		3-1/2 (89)		4-3/4 (121)	4-1/4 (108)		9-1/2 (241)		7-3/4 (197)
Min. hole depth in concrete	h_a	in. (mm)	2-5/8 (67)		2-5/8 (67)		4 (102)		3-7/8 (98)	4-3/4 (121)		4-1/2 (117)		5-3/4 (146)
Yield strength of anchor steel	f_{ya}	lb/in^2 (N/mm^2)	100,000 (690)		84,800 (585)				84,800 (585)			84,800 (585)		
Ult. Strength of anchor steel	f_{uta}	lb/in^2 (N/mm^2)	125,000 (862)		106,000 (731)				106,000 (731)			106,000 (731)		
Tensile stress area	A_{se}	in^2 (mm^2)	0.052 (33.6)		0.101 (65.0)				0.162 (104.6)			0.237 (152.8)		
Steel strength in tension	N_{sa}	lb (kN)	6,500 (28.9)		10,705 (47.6)				17,170 (76.4)			25,120 (111.8)		
Steel strength in shear	V_{sa}	lb (kN)	3,595 (16.0)		5,495 (24.4)				8,090 (36.0)			13,675 (60.8)		
Steel strength in shear, seismic	$V_{sa,seis}$	lb (kN)	2,255 (10.0)		5,495 (24.4)				7,600 (33.8)			11,745 (52.2)		
Pullout strength uncracked concrete[2]	$N_{p,\ uncr}$	lb (kN)	2,515 (11.2)		-		5,515 (24.5)		-	9,145 (40.7)		8,280 (36.8)		10,680 (47.5)
Pullout strength cracked concrete[2]	$N_{p,cr}$	lb (kN)	2,270 (10.1)		-		4,915 (21.9)		-	-		-		-
Effectiveness factor uncracked concrete[3]	k_{uncr}		24											
Effectiveness factor cracked concrete[4]	k_c		17											
Modification factor for uncracked concrete[5]	$\Psi_{c,N}$		1.41											
Strength reduction factor for tension, steel failure modes[6]			0.75											
Strength reduction factor for shear, steel failure modes[6]			0.65											
Strength reduction factor for concrete breakout, side-face blowout, pullout, or pryout failure modes[6]			0.65 for tension loads 0.75 for shear loads											

For SI: 1 inch = 25.4 mm, 1 lbf =4.45N, 1 psi=0.006895 MPa. For pound-inch units: 1mm=0.03937 inches

[1]Interpolation in accordance with Figure III. C-3 herein is permitted.

[2]See ACI 349-13 Appendix D, paragraph D.5.3.2.

[3]See ACI 349-13 Appendix D, paragraphs D.5.2.2 and D.5.2.9. The value of k_{uncr} (i.e., 24) is based on testing and assessment deemed to be compliant with ACI 355.2-01.

[4]See ACI 349-13 Appendix D, paragraphs D.5.2.2 and D.5.2.9. The value of k_c (i.e., 17) is based on testing and assessment deemed to be compliant with ACI 355.2-01.

[5]See ACI 349-13 Appendix D, paragraphs D.5.2.6 and D.5.2.9. The value of $\Psi_{c,N}$ (i.e., 1.41) was derived from k_{uncr}/k_{cr}= 24/17, and these k-factor values are based on testing and assessment deemed to be compliant with ACI 355.2-01.

[6]See ACI 349-13 Appendix D, paragraph D.4.4. For use with the load combinations of ACI 349-13, Section 9.2.

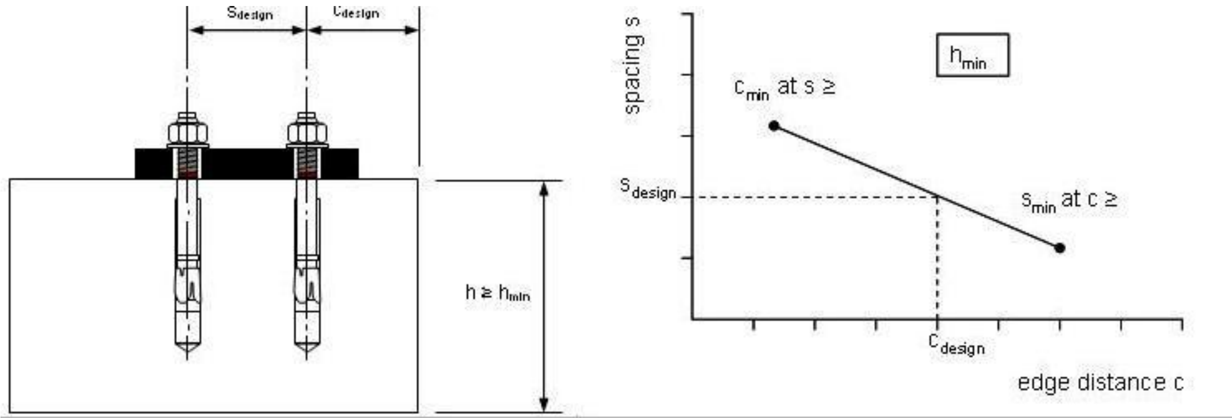

Figure III.C-3 Interpolation of Minimum Edge Distance and Minimum Spacing for KB-TZ

Table III.C-2 Anchor Length and Fastened Part Information for KB-TZ

SETTING INFORMATION	Symbol	Units	Nominal anchor diameter (in.)														
			3/8			1/2				5/8				3/4			
Effective min. embedment	h_{ef}	in. (mm)	2 (51)			2 (51)		3 ¼ (83)		3 1/8 (79)		4 (102)		3 ¾ (95)		4 ¾ (121)	
Min. thickness of fastened part[1]	t_{min}	in. (mm)	0 (0)			¾ (19)		¼ (6)		3/8 (9)		¾ (19)		0 (0)		7/8 (23)	
Min. dia. of hole in fastened part	d_h	in. (mm)	7/16 (11.1)			9/16 (14.3)				11/16 (17.5)				13/16 (20.6)			
Standard anchor lengths	ℓ_{anch}	in. (mm)	3 (76)	3 ¾ (95)	5 (127)	3 ¾ (95)	4 ½ (114)	5 ½ (140)	7 (178)	4 ¾ (121)	6 (152)	8 ½ (216)	10 (254)	5 ½ (140)	7 (178)	8 (203)	10 (254)
Threaded length (incl. dog point)	ℓ_{thread}	in. (mm)	1 ½ (38)	2 ¼ (57)	3 ½ (93)	1 5/8 (41)	2 3/8 (60)	3 3/8 (86)	4 7/8 (124)	1 ½ (38)	2 ¾ (70)	5 ¼ (133)	6 ¾ (171)	2 ½ (63)	4 (103)	5 (128)	7 (179)
Unthreaded length	ℓ_{unthr}	in. (mm)	1 ½ (39)			2 1/8 (54)				3 ¼ (83)				3 (77)			

[1]The minimum thickness of the fastened part is based on use of the anchor at minimum embedment and is controlled by the length of thread. If a thinner fastening thickness is required, increase the anchor embedment to suit.

APPENDIX D: LIMIT STATE APPLICATION TO SCC EXAMPLES

Adapted for LANL from ANSI/ANS-2.26-2004 (R2010) Appendix B

The selection of a Limit State for structures, systems, and components (SSCs) will depend on SSC component type and the safety function it performs. This appendix provides guidance for selection of a Limit State through use of examples. The examples should not be interpreted as requirements. The selection of the Limit State should be based on the specific safety analysis and the safety function of the SSC.

SSC Type	Limit State A	Limit State B	Limit State C	Limit State D
Generic	Refer to ESM Ch. 5 Section III body (or ANS 2.26 Section 5) for the definitions of the four Limit States addressed in this table.			
Building structural components	Substantial loss of SSC stiffness and some strength loss may occur, but some margin against collapse is retained so that egress is not impaired; building needs major repair and may not be safe for occupancy until repaired.	Some loss of SSC stiffness and strength may occur, but SSC retains substantial margin against collapse; building may need some repair for operations and occupancy to continue.	The SSC retains nearly full stiffness and retains full strength, and the passive component it is supporting will perform its normal and safety functions during and following an earthquake.	SSC damage is negligible; structure retains full strength and stiffness capacities; building is safe to occupy and retains normal function.
Structures or vessels for containing hazardous material	Applicable to vessels and tanks that contain material that is either not very hazardous or leakage is contained or confined by another SSC to a local area with no immediate impact to the worker. Recovery from a spill may be completed with little risk, but the vessel is not likely to be repairable. Most likely applicable to vessels containing low hazard solids or liquids.	Applicable to vessels and tanks whose contents if released slowly over time through small cracks will either be contained by another SSC or acceptably dispersed with no consequence to worker, public, or environment. Cleanup and repair may be completed expediently. Most likely applicable to moderate-hazard liquids or solids or low-hazard low-pressure gases.	Applicable to low-pressure vessels and tanks with contents sufficiently hazardous that release may potentially injure workers. Damage will be sufficiently minor to usually not require repair.	Content and location of item is such that even the smallest amount of leakage is sufficiently hazardous to workers or the public that leak-tightness must be assured. Most likely applicable to moderate and highly hazardous pressurized gases but may be required for high-hazard liquids. Post-earthquake recovery is assured.

SSC Type	Limit State A	Limit State B	Limit State C	Limit State D
Confinement barriers and systems containing hazardous material (e.g., glove boxes, building rooms, and ducts)	No SSC of this type should be designed to this Limit State.	Barriers could be designed to this Limit State if exhaust equipment is capable of maintaining negative pressures with many small cracks in barriers and is also designed to Limit State D for long-term loads. Safety-related electrical power instrumentation and control if required must also be assured including the loss of off-site power. Localized impact and impulse loads may be considered in this Limit State.	Barriers could be designed to this Limit State if exhaust equipment is capable of maintaining negative pressures with few small cracks in barriers and is also designed to Limit State D for long-term loads. Safety-related electrical power instrumentation and control if required must also be assured including the loss of off-site power. Adequate confinement without exhaust equipment may be demonstrable for some hazardous materials.	Systems with barriers designed to this Limit State may not require active exhaust depending on the contained hazardous inventory and the potential for development of positive pressure. Safety related electrical power, instrumentation, and control, if required, must also be assured including the loss of off-site power.
Equipment support structures, including support structures for pressure vessels and piping, fire suppression systems, cable trays, heating ventilation and air-conditioning ducts, battery racks, etc.	The SSC may undergo substantial loss of stiffness and some loss of strength, and yet the equipment it is supporting may perform its safety functions (normal function may be impaired) following exposure to specified seismic loads; the SSC retains some margin against such failures that may cause systems interactions.	The SSC may undergo some loss of stiffness and strength, and yet the equipment it is supporting may perform its safety functions (normal function may be impaired) following exposure to specified seismic loads; the SSC retains substantial margin against such failures that cause systems interactions.	The SSC retains nearly full stiffness and retains full strength, and the passive equipment it is supporting may perform its normal and safety functions during and following exposure to specified seismic loads.	No SSC of this type should be designed to this Limit State.

SSC Type	Limit State A	Limit State B	Limit State C	Limit State D
Mechanical or electrical SSCs	The SSC must maintain its structural integrity. It may undergo large permanent distortion and yet perform its safety functions; no assurance that the SSC will retain its normal function or will remain repairable.	The SSC must remain anchored, and if designed as a pressure-retaining SSC, it must maintain its leak-tightness and structural integrity. It may undergo moderate permanent distortion and yet perform its safety functions; there is some assurance that the SSC will retain its normal function and will remain repairable.	The SSC must remain anchored, and if designed as a pressure-retaining SSC, it must maintain its leak-tightness and structural integrity. It may undergo very limited permanent distortion and yet perform its normal functions (with little or no repair) and safety functions after exposure to its specified seismic loads.	The SSC remains essentially elastic and may perform its normal and safety functions during and after exposure to its specified seismic loads.
High-efficiency particulate absorber filter assemblies and housings	Assemblies designed to this level should have no nuclear or toxic hazard safety functions.	Assemblies designed to this level should have no nuclear or toxic hazard safety functions.	This Limit State may be expected to be applied to systems categorized as SDC-4 or lower.	N/A at LANL (this Limit State may be expected to be applied to systems classified as SDC-5 and possibly some in SDC-4).
Electrical raceways (cable trays, conduits, raceway channels)	The electrical raceways may undergo substantial distortion, displacement, and loss of stiffness, but the connections (e.g., at the penetrations or at the junction boxes) are very flexible or are such that the cables may still perform their function during and following exposure to specified seismic loads.	The electrical raceways may undergo some distortion, displacement, and loss of stiffness, but the connections (e.g., at the penetrations or at the junction boxes) have some flexibility or are such that the cables may still perform their function during and following exposure to specified seismic loads.	Cable connection (e.g., at the penetrations or at the junction boxes) are rigid or brittle or are such that the electrical raceways may undergo only very limited distortion, displacement, and loss of stiffness during exposure to specified seismic loads before the cable functions are impaired.	Cable connections (e.g., at the penetrations or at the junction boxes) are very rigid or brittle or are such that the electrical raceways may undergo essentially no distortion or loss of stiffness during exposure to specified seismic loads before the cable functions are impaired.
Deformation sensitive SSCs[16]	These types of SSCs should not be designed to this Limit State.	These types of SSCs should not be designed to this Limit State.	Functional evaluation is required when designing to this Limit State. Component testing may be required.	This type of SSC should typically be designed to this Limit State, and testing may be required.

[16] Deformation-sensitive SSCs are defined as those whose safety functions may be impaired if these SSCs undergo deformations within the elastic limit during an earthquake (e.g., a valve operator, a relay, etc.).

SSC Type	Limit State A	Limit State B	Limit State C	Limit State D
Anchors and anchor bolts for equipment and equipment support structures	To ensure that system interactions do not occur during an earthquake, no anchors or anchor bolts should be designed to this Limit State. [17]	The anchors or anchor bolts may undergo only moderate permanent distortion without impairing the safety function of the equipment (normal function may be impaired) following exposure to the specified seismic loads.	The anchors or anchor bolts may undergo very limited permanent distortion without impairing the normal and safety functions of the equipment following exposure to the specified seismic loads.	The anchors or anchor bolts need to remain essentially elastic so as not to impair the normal and safety functions of the equipment during and following exposure to the specified seismic loads.
Pressure vessels and piping[18]	Tanks, pressure vessels, and piping systems that do not contain or carry any hazardous fluid, have no safety functions, and whose gross leakage during and following an earthquake will not impact safety. Repair may require replacement of vessel and piping.	Tanks, pressure vessels, and piping systems that can perform their safety function even if they develop small leaks as a result of moderate permanent distortion caused by a design-basis earthquake. In situ repair of vessel may be possible. The safety function of the SSC may include confinement if the radiological release is within prescribed limits.	Tanks, pressure vessels, and piping systems that may have no significant spill and leakage during and following and earthquake. Includes vessels and piping systems that have confinement as a safety function.	Tanks, pressure vessels, and piping systems that are required to have very high confidence of no spills and leakage during and following an earthquake. Includes vessels and piping systems that have containment as a safety function.

[17] Anchor bolts designed to code-allowables generally will exceed this Limit State because of conservatism inherent in the standard design procedures (e.g., factor of safety of 4 for expansion anchors). This assumes that appropriate over strength factors of the attached members are considered.

[18] Pressure vessels and piping systems designed to ASME Boiler and Pressure Vessel Code (B&PVC), Section III, Division 1, Service Level D are capable of providing containment function (i.e., Limit State D), even though the code permits stress levels beyond the yield stress. Thus, pressure vessels and piping systems that have confinement as a safety function are permitted to be designed to ASME B&PVC, Section III, Service Level D.

APPENDIX E: DESIGN BASIS EARTHQUAKE LOADS

A. The LANL design basis earthquake response spectra are defined in the free-field at the ground surface.

B. The design basis earthquake (DBE) response spectra in Tables III.D-1 – 4 and Figures III.D-1 – 4 below are the Design Response Spectra (DRS) defined in ASCE 43 (2005[19]) Equation 2-1, and include the effects of the Design Factor (DF).

1. The LANL DBE response spectra for site-wide use, excluding TA-55 site, is specified in Tables III.D-1 and III.D-2, and Figures III.D-1 and III.D-2.[20]

2. The LANL DBE response spectra for TA-55 site are specified in Tables III.D-3 and III.D-4, and Figures III.D-3 and III.D-4.

3. Response spectra at intermediate frequencies shall be obtained by log-log interpolation (Requirement 5-0283).

C. Lateral soil pressure, H, on embedded structures resulting from earthquake ground shaking shall be calculated using ASCE 4 and shall be included in E (or E_s, or E_{ss}) (Requirement 5-0284). The paragraph, Lateral Soil Pressure Loads (H) herein (in the main body) applies (Requirement 5-0285).

D. Site Limitation: Structures shall not be located within 50 feet of known active faults (Requirement 5-0286). Hazardous waste treatment, storage and disposal facilities must not be located within 200 feet of a fault that has had displacement within the last 11,000 years (Requirement 5-0287).

E. The potential for seismic-induced displacement hazards (i.e., fault rupture) must be assessed on a project-specific basis (Requirement 5-0288). The project-specific plan for addressing fault displacement hazards associated with new construction shall be submitted to the Chapter 5 Point of Contact, or designee, for review and approval prior to the start of design (Requirement 5-0289).

1. A minimum fault displacement of 2 cm (0.8 in.) at the surface shall be used for design (Requirement 5-0290).

F. *Cranes and Seismic Loads (Guidance): ESM Chapter 6 Section D10+E10 paragraph D1090 requires the use of both ASME NOG-1 and ASME NUM-1 for the deisgn of ML-1 and ML-2 cranes, and both of these standards include seismic requirements. ASCE 43 Paragraph 1.4 indicates CMAA has a standard(s) on the seismic design of cranes. At the time of writing, CMAA's Standards Director said the organization does not have standards, publications, or guidance on any aspect of "seismic and cranes." Although AISC Design Guide 7 pertains to industrial buildings, paragraph 13.6 therein could be of use more broadly; it discusses the contribution of the crane to the seismic mass of the building, interaction between the crane and building, design of cranes that must serviceable after the DBE, etc. Finally, CISC-ICCA's "Guide for the Design of Crane-Supporting Steel Structures" offers suggestions on how to*

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[19] Spectra is based on standards that predate the current ones.
[20] And TA-50, with approved variance addressing geotechnical consistency

account for interaction of the mass of the crane and the mass of the structure, and for the crane acting as a tie between the crane rails.

Table III.D-1 Site-Wide Free-Field Surface SDC-3 DBE Horizontal DRS

	Spectral Acceleration (g)		
Frequency (Hz)	**2% Damping**	**5% Damping**	**10% Damping**
0.2	0.107	0.094	0.073
0.498	0.662	0.583	0.456
0.925	1.374	1.083	0.862
9	1.374	1.083	0.862
33	0.482	0.482	0.482
100	0.482	0.482	0.482

Table III.D-2 Site-Wide Free-Field Surface SDC-3 DBE Vertical DRS

	Spectral Acceleration (g)		
Frequency (Hz)	**2% Damping**	**5% Damping**	**10% Damping**
0.2	0.048	0.039	0.032
0.469	0.262	0.213	0.175
1	0.585	0.455	0.354
8	2.499	1.885	1.454
12	2.499	1.885	1.454
50	0.564	0.564	0.564
100	0.564	0.564	0.564

Table III.D-3 TA-55 Free-Field Surface SDC-3 DBE Horizontal DRS

		Spectral Acceleration (g)		
Period (s)	**Frequency (Hz)**	**2% Damping**	**5% Damping**	**10% Damping**
10	**0.100**	0.025	0.021	0.018
5	**0.200**	0.125	0.106	0.090
1.5	**0.667**	0.690	0.552	0.444
1	**1.000**	0.915	0.724	0.578
0.5	**2.000**	1.022	0.806	0.640
0.15	**6.667**	1.115	0.888	0.712
0.1	**10.000**	0.933	0.770	0.640
0.85	**11.765**	0.826	0.695	0.589
0.075	**13.333**	0.752	0.643	0.552
0.06	**16.667**	0.644	0.566	0.498
0.05	**20.000**	0.578	0.519	0.466
0.03	**33.333**	0.439	0.419	0.419
0.02	**50.000**	0.419	0.419	0.419
0.01	**100.000**	0.419	0.419	0.419

Table III.D-4 TA-55 Free-Field Surface SDC-3 DBE Vertical DRS

		Spectral Acceleration (g)		
Period (s)	**Frequency (Hz)**	**2% Damping**	**5% Damping**	**10% Damping**
10	**0.100**	0.015	0.012	0.010
5	**0.200**	0.074	0.062	0.052
1.5	**0.667**	0.403	0.312	0.246
1	**1.000**	0.538	0.412	0.322
0.5	**2.000**	0.682	0.563	0.404
0.15	**6.667**	1.302	0.971	0.741
0.1	**10.000**	1.524	1.173	0.922
0.85	**11.765**	1.623	1.266	1.006
0.075	**13.333**	1.623	1.266	1.006
0.06	**16.667**	1.431	1.141	0.925
0.05	**20.000**	1.216	0.989	0.816
0.03	**33.333**	0.741	0.655	0.585
0.02	**50.000**	0.474	0.474	0.474
0.01	**100.000**	0.474	0.474	0.474

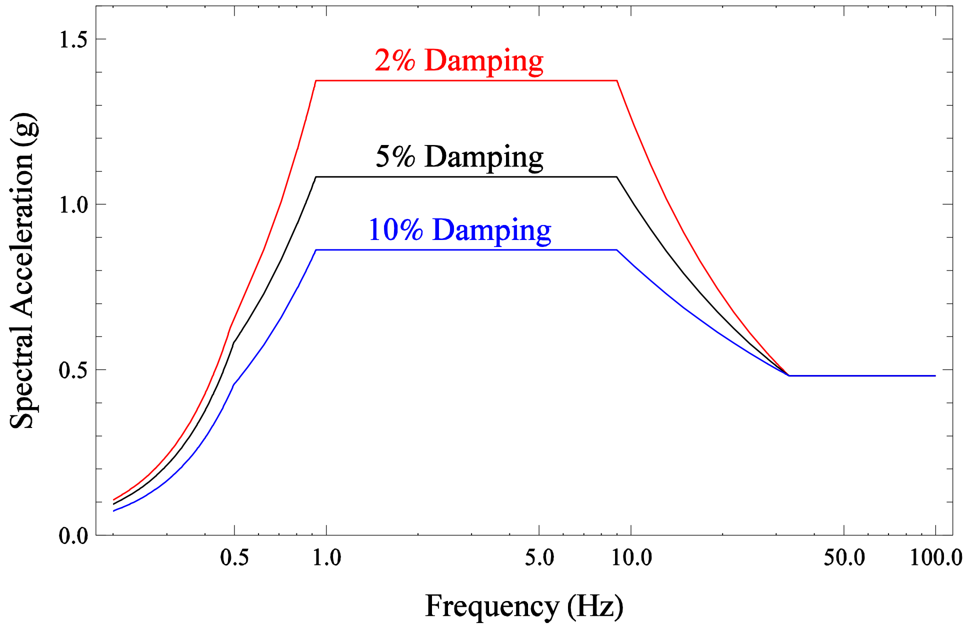

Figure III.D-1 Site Wide Free-Field Surface SDC-3 DBE Horizontal DRS

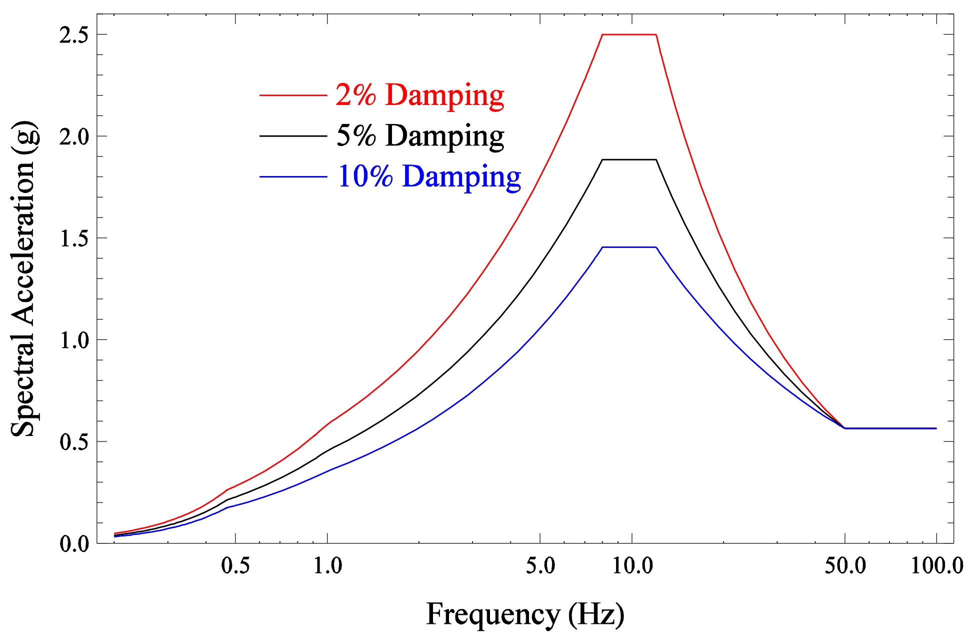

Figure III.D-2 Site Wide Free-Field Surface SDC-3 DBE Vertical DRS

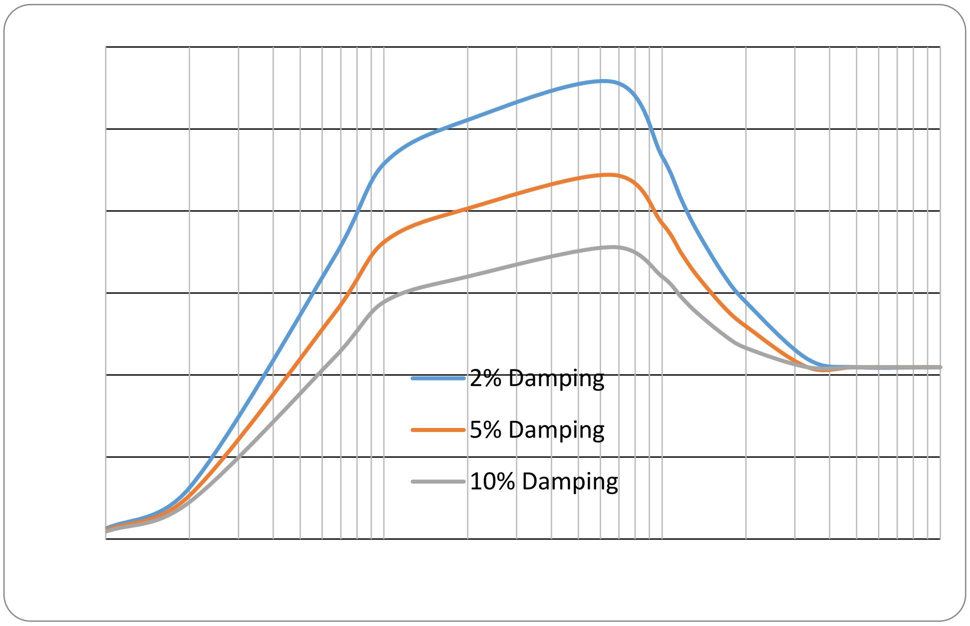

Figure III.D-3 TA-55 Free-Field Surface SDC-3 DBE Horizontal DRS [acceleration (g) on vertical versus frequency (Hz) on horizontal]

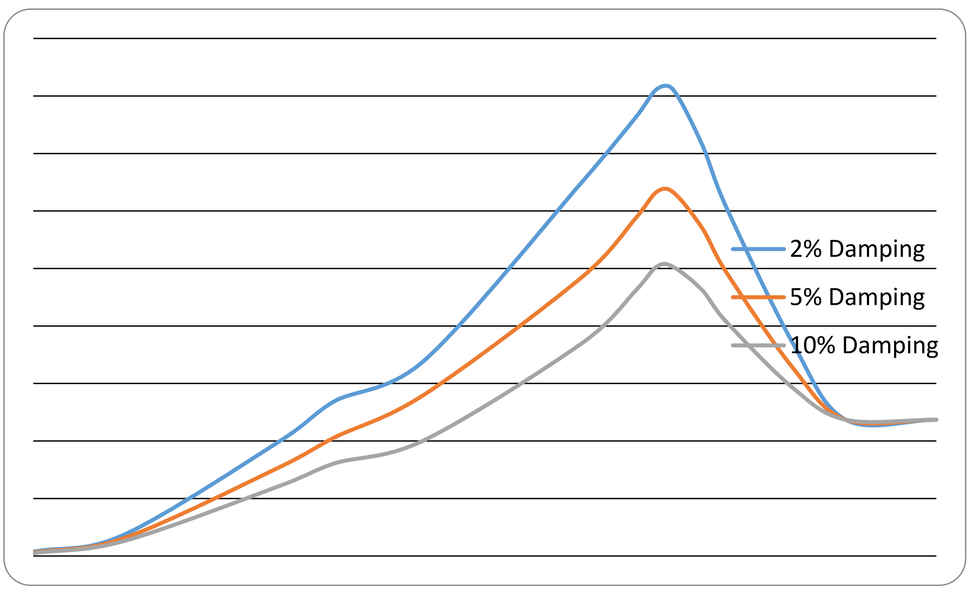

Figure III.D-4 TA-55 Free-Field Surface SDC-3 DBE Vertical DRS [acceleration (g) on vertical versus frequency (Hz) on horizontal]

APPENDIX F: "1020" AMENDMENTS FOR ASCE 4 AND ASCE 43

As noted early in this document, LANL amends all "1020" ASCE 4 and ASCE 43 edition references from 1998 to 2016 and 2005 to 2019, respectively (Requirement 5-0291). In doing so, a number of minor wording changes to "1020" treatment of ASCE 4 and ASCE 43 are also necessary; these are reflected in the amended "1020" paragraphs below.

1. Replace "4-98" with "4-16," and "43-05" with "43-19," with the following result:

   3.1.2 SDC-3, SDC-4, and SDC-5 SSCs shall be designed according to the criteria of ASCE/SEI 43-19 ~~43-05~~…and ASCE 4-16 ~~4-98~~…in this Standard.

2. Replace "43-05" with "43-19" with the following result:

   Table 3-1 Note 2 The IBC invokes…as defined in…ASCE/SEI 43-19 ~~43-05~~…ASCE/SEI 7-10).

3. Replace "43-05" with "43-19," and revise associated verbiage, with the following result:

   3.2.1 For SDC-3 through SDC-5 SSCs, appropriate Target Performance Goals ~~DBE return periods (hazard exceedance probabilities)~~ given in ASCE/SEI 43-19 (Table 1-1), and ~~appropriate~~ scale ~~design~~ factors given in ASCE/SEI 43-19 ~~43-05~~ (Chapter 2 ~~1-2~~), shall be used to determine the DBE response spectrum/acceleration time series ~~seismic ground motion~~ applicable for the facility site.

4. Replace "43-05" with "43-19," and revise associated verbiage, with the following result:

   3.3.1 Site characterization…qualifications:

   • Table 1 of ANSI/ANS-2.27-2008 states…characterized.

   • Column 2 of Table 1 of ANSI/ANS-2.27-2008 is labeled "Maximum Considered Earthquake Spectral Response Acceleration." It provides a range of spectral response accelerations for low, moderate and high seismic environments. For DOE purposes, and for consistency with terminology in ASCE/SEI 43-19 ~~43-05~~, this column heading means "Spectral response acceleration at H_D = 1 E-4 annual frequency ~~probability~~ of exceedance."

5. Replace "4-98" with "4-16," and strike the associated unnecessary verbiage, with the following result:

   3.5.1 For SDC-3 through SDC-5 SSCs, soil-structure interaction analyses may rely on the applicable provisions of ASCE 4-16 ~~4-98, with this exception: the wave incoherence provision (Section 3.3.1.10) shall not be used in meeting ASCE 4-98 Section 3.3~~. The DOE/EM Memo...analysis.

6. Replace "4-98" with "4-16," and "43-05" with "43-19," and strike the associated unnecessary verbiage, with the following result:

   3.5.2 For new facilities, the method of determining design basis seismic ground motion shall satisfy the requirements of Section 2 of ASCE/SEI 43-19 ~~43-05~~ and Section 2 of ASCE 4-16 ~~4-98, provided these requirements are consistent with Section 2 of ASCE/SEI 43-05~~.

7. Replace "4-98" with "4-16," and strike the associated unnecessary verbiage, with the following result:

   3.5.3 The requirements in ASCE 4-16 ~~4-98~~ shall be met in performing dynamic response analyses and generating in-structure response spectra~~, provided such requirements are consistent with the requirements of ASCE/SEI 43-05.~~

8. Replace "43-05" with "43-19" with the following result:

   3.6.4 Seismic qualification of equipment may be performed by testing and/or by using actual earthquake experience or generic shake table test data, subject to the criteria and limitations given in ASCE/SEI 43-19 ~~43-05~~, ASME QME-1…*Energy Facilities*.

APPENDIX G: RESTRAINT OF NON-FACILITY (E.G., PROGRAMMATIC) EQUIPMENT

Note: This material supersedes any similar material in ESM Ch 16 IBC-GEN.

Restraint (i.e., anchorage and/or bracing) of non-facility (e.g., programmatic, utilities) equipment is required in either of the following two circumstances (Requirement 5-0292)**:**

1. When required by the **manufacturer** for normal operations.[21]

a. If this is the sole reason restraint is required (i.e., seismic restraint per criterion 2 below not required), then the design need not be per ESM Ch. 5 if the manufacturer provides alternative design requirements (Requirement 5-0293).

2. **In a nuclear facility**, restraint is always required (Requirement 5-0294). The design must be per Section III, the installation and quality control per the appropriate LANL Master Spec Sections, and the QA per ESM Ch.16 (as a minimum) (Requirement 5-0295). There are two exceptions:

a. **SDC-1 and SDC-2 SSCs.** Subject to the provisions in Sect. III, NDC-1 AND NDC-2 SSCs, Seismic, seismically-exempt equipment (ref. Section II, 1613.5 Amendments to ASCE 7, Add 1613.5.4 Nonstructural Components Exempt from Seismic Design) need not be restrained provided that adverse interactions won't result.***** Non-seismically-exempt equipment might not require restraint provided that adverse interactions won't result*** and** if an alternative method is accepted by the LANL Building Official wherein:

i. An analysis that indicates the interaction effects of the unrestrained equipment[22] is acceptable at the DBE, or

ii. The equipment will be located in an essentially unoccupied[23] area and is protected as such through administrative or engineering control.

***** If equipment is to be unrestrained, adverse interaction with safety SSCs (i.e., Safety Class, Safety Significant, Other Hazard Control[24]) must be considered, documented as part of the design, and, if necessary, prevented (Requirement 5-0296). It is expected that determining the significance of potential interaction effects will require interaction between a structural subject matter expert (SME) and a safety basis SME. For more detail on interactions, refer to Sect. III (*Common-Cause Failure and System Interaction; and App. A, Prerequisites for Determining Anchor Design Loads, System Interaction).*

b. **SDC-3 SSCs.** If the equipment is to be installed on a floor structure and the provisions of ASCE/SEI 43 Section 7.1, *Rocking and Sliding of Unanchored Rigid Bodies* are met to the satisfaction of the LBO, then seismic restraint is not required.

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[21] Reasons related to performance, functionality, operability, etc. (e.g., motor, centrifuge, certain suspended items/systems, etc.).
[22] E.g., unrestrained from rocking, swaying, overturning, sliding, impact, etc. Also: In all cases, equipment must be anchored if it is permanently attached to utility services (electricity, gas, and water). (ASCE 7 C13.1.1).
[23] I.e., incidental occupancy: occupied for a total of less than 2 hours/day, yearly average (RP 8 exemption 1.3.d utilized herein). Not the same as incidental use (IBC sec. 509 term relating to certain adjunct uses).
[24] Formerly "Other Equipment Important to Safety."

NOTE for Equipment Outside: Both of the above criteria also apply outside of a building; if the above require an item to be restrained for wind and/or seismic were it inside, then restraint must either be provided or shown/proven unwarranted (Requirement 5-0297). Ref ASCE-7-10 Section 15.1.1, *Nonbuilding Structures*, and Ch. 29, *Wind Loads on Other Structures and Building Appurtenances*.

APPENDIX H: HILTI KB-TZ2 (NEW) EXPANSION ANCHOR DESIGN FIGURES AND TABLES

TABLE 1 Setting Information
(Carbon Steel and Stainless Steel Anchors)

Setting information	Sym	Units	Nominal anchor diameter (in.)												
			3/8			1/2				5/8			3/4		
Nominal bit diameter	*do*	in.	3/8			1/2				5/8			3/4		
Effective min. embedment	*hef*	in. (mm)	1-1/2 (38)	2 (51)	2-1/2 (64)	1-1/2 [1] (38)	2 (51)	2-1/2 (64)	3-1/4 (83)	2-3/4 (70)	3-1/4 (83)	4 (102)	3-1/4 (83)	3-3/4 (95)	4-3/4 (121)
Nominal embedment	*hnom*	in. (mm)	1-7/8 (48)	2-1/2 (64)	3 (76)	2 [1] (51)	2-1/2 (64)	3 (76)	3-3/4 (95)	3-1/4 (83)	3-3/4 (95)	4-1/2 (114)	4 (102)	4-1/2 (114)	5-1/2 (140)
Min. hole depth	*ho*	in. (mm)	2 (51)	2-3/4 (70)	3-1/4 (83)	2-1/4 [1] (57)	2-3/4 (70)	3-1/4 (83)	4-1/4 (108)	3-3/4 (95)	4-1/4 (108)	4-3/4 (121)	4-1/4 (108)	4-3/4 (121)	5-3/4 (146)
Installation torque carbon steel[1]	*Tinst*	ft-lb (Nm)	30 (41)			50 (68)				40 (54)			110 (149)		
Installation torque stainless steel[1]	*Tinst*	ft-lb (Nm)	30 (41)			40 (54)				60 (81)			125 (169)		
Fixture hole diameter	*dh*	in. (mm)	7/16 (11.1)			9/16 (14.3)				11/16 (17.5)			13/16 (20.6)		

[1] Design information for hef = 1-1/2 is only applicable to carbon steel (CS) KB-TZ2 bolts (stainless version in this embedment unavailable)

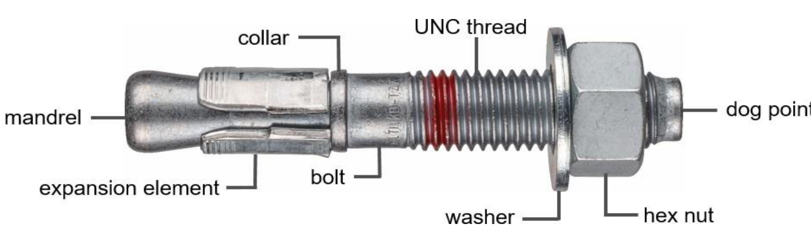

Figure 1 Kwik Bolt TZ2 (KB-TZ2)

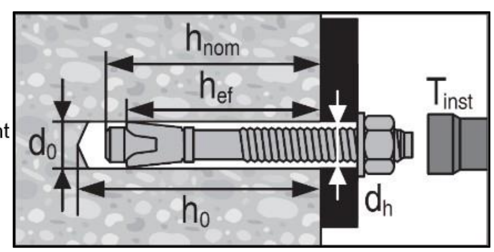

Figure 2 KB-TZ2 Installation Parameters

Table 2 Length Identification System
(Carbon Steel and Stainless Steel Anchors)

Length ID marking on bolt head		**A**	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**I**	**J**	**K**	**L**	**M**	**N**	**O**	**P**	**Q**	**R**	**S**	**T**	**U**	**V**	**W**
Length of anchor, ℓ_{anch} (inches)	from	1½	2	2½	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10	11	12	13	14	15
	up to but not including	2	2½	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10	11	12	13	14	15	16

For **SI:** 1 inch = 25.4 mm.

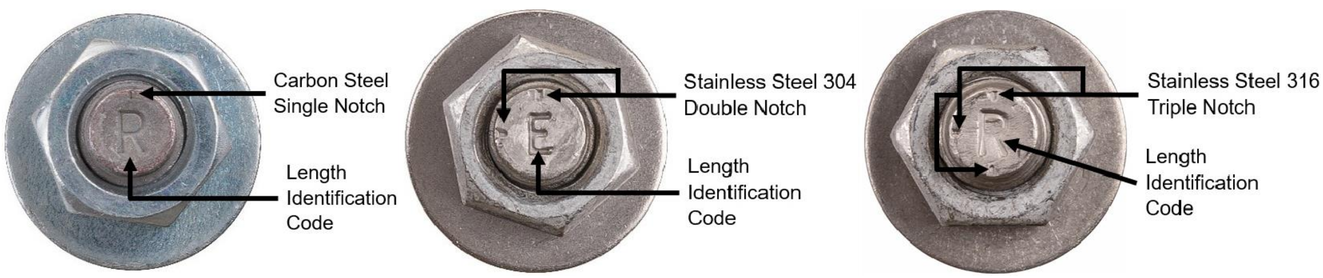

Figure 3 KB-TZ2 Bolt Head with Length Identification Code and Material Head Notch Embossment

Table 3 Carbon Steel KB-TZ2 Design Information, Tension

Design parameter	Symbol	Units	Nominal Anchor Diameter (in)												
			3/8			1/2				5/8			3/4		
Effective min. embedment[1]	h_{ef}	in. (mm)	1-1/2 (38)	2 (51)	2-1/2 (64)	1-1/2 (38)	2 (51)	2-1/2 (64)	3-1/4 (83)	2-3/4 (70)	3-1/4 (83)	4 (102)	3-1/4 (83)	3-3/4 (95)	4-3/4 (121)
Tension, steel failure modes															
Strength reduction factor for steel in tension[2]	Φ	-	0.75			0.75				0.75			0.75		
Min. specified yield strength	f_{ya}	lb/in² (N/mm²)	100,900 (696)			96,300 (664)				87,000 (600)			84,700 (584)		
Min. specified ult. strength	f_{uta}	lb/in² (N/mm²)	126,200 (870)			114,000 (786)				106,700 (736)			105,900 (730)		
Effective tensile stress area	$A_{se,N}$	in² (mm²)	0.051 (33.2)			0.099 (63.6)				0.164 (106.0)			0.239 (154.4)		
Steel strength in tension	N_{sa}	lb (kN)	6,490 (28.9)			11,240 (50.0)				17,535 (78.0)			25,335 (112.7)		
Tension, concrete failure modes															
Critical edge distance	c_{ac}	in. (mm)	5 (127)	4-3/8 (111)	5-1/2 (140)	8 (203)	5-1/2 (140)	6-3/4 (171)	10 (254)	10 (254)	11-1/2 (292)	8-3/4 (222)	12 (305)	10 (254)	9 (229)
Strength, reduction factor for concrete in tension	Φ	-	0.65			0.65				0.65			0.65		
Effectiveness factor for uncracked concrete	k_{uncr}	-	24			27		24		24			27		24
Effectiveness factor for cracked concrete	k_c	-	21		17	24	21		17	21		17	21		
Modification factor for anchor resistance, tension[3]	$\psi_{c,N}$	-	1.0			1.0				1.0			1.0		
Pullout strength uncracked conc.[4]	$N_{p,uncr}$	lb (kN)	N/A	N/A	4,180 (18.6)	N/A	N/A	N/A	N/A	5,380 (23.9)	N/A	8,995 (40.0)	N/A	N/A	N/A
Pullout strength cracked conc.[4]	$N_{p,cr}$	lb (kN)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	8,835 (39.3)
Pullout strength seismic[4]	$N_{p,seis}$	lb (kN)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	8,700 (38.7)
Normalization factor, uncracked concrete	n_{uncr}	-	N/A	N/A	0.35	N/A	N/A	N/A	N/A	0.50	N/A	0.50	N/A	N/A	0.39
Normalization factor, cracked concrete, seismic	n_{cr}	-	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.29
Tension, axial stiffness															
Axial stiffness in service load range	β_{uncr}	lb/in.	131,572			158,583				290,360			412,333		
	β_{cr}	lb/in.	91,336			113,514				167,367			62,179		

For SI: 1 inch = 25.4 mm, 1 lbf =4.45N, 1 psi=0.006895 MPa. For pound-inch units: 1mm=0.03937 inches

[1] Figure 2 illustrates the installation parameters
[2] The KB-TZ2 is considered a ductile steel element in accordance with ACI 349-13 Appendix D.
[3] For all design cases, $\psi_{c,N} = 1.0$. The appropriate effectiveness factor for cracked concrete (k_{cr}) or uncracked concrete (k_{uncr}) shall be used.
[4] For all design cases, $\psi_{c,P} = 1.0$. Tabular value for pullout strength is for a concrete compressive strength of 2,500 psi. Pullout strength for concrete compressive strength greater than 2,500 psi may be increased by multiplying the tabular pullout strength by $(f'c/2{,}500)^n$ for psi, or $(f'c/17.2)^n$ for MPa, where n is given as n_{uncr} for uncracked concrete and n_{cr} for cracked concrete and seismic. NA (not applicable) denotes that pullout strength does not need to be considered for design.

Table 4 Stainless Steel KB-TZ2 Design Information, Tension

Design parameter	Symbol	Units	Nominal Anchor Diameter (in)											
			3/8			1/2			5/8			3/4		
Effective min. embedment[1]	h_{ef}	in. (mm)	1-1/2 (38)	2 (51)	2-1/2 (64)	2 (51)	2-1/2 (64)	3-1/4 (83)	2-3/4 (70)	3-1/4 (83)	4 (102)	3-1/4 (83)	3-3/4 (95)	4-3/4 (121)
Tension, steel failure modes														
Strength reduction factor for steel in tension[2]	Φ	-	0.75			0.75			0.75			0.75		
Min. specified yield strength	f_{ya}	lb/in^2 (N/mm^2)	96,300 (664)			96,300 (664)			91,600 (632)			84,100 (580)		
Min. specified ult. strength	f_{uta}	lb/in^2 (N/mm^2)	120,100 (828)			120,400 (830)			114,600 (790)			100,500 (693)		
Effective tensile stress area	$A_{se,N}$	in^2 (mm^2)	0.051 (33.2)			0.099 (63.6)			0.164 (106.0)			0.239 (154.4)		
Steel strength in tension	N_{sa}	lb (kN)	6,180 (27.5)			11,870 (52.8)			18,835 (83.8)			24,045 (107.0)		
Tension, concrete failure modes														
Critical edge distance	c_{ac}	in. (mm)	4-1/2 (114)	5-1/2 (140)	4-1/8 (105)	5-1/2 (140)	6-1/4 (159)	7-1/2 (191)	10 (254)	6-1/2 (165)	8-3/4 (222)	12 (305)	10 (254)	10 (254)
Strength, reduction factor for concrete in tension	Φ	-	0.65			0.65			0.65			0.65		
Effectiveness factor for uncracked concrete	k_{uncr}	-	24			24			24			24	27	24
Effectiveness factor for cracked concrete	k_c	-	21		17	17	21	17	21		17	21		
Modification factor for anchor resistance, tension, uncracked concrete[3]	$\psi_{c,N}$	-	1.0			1.0			1.0			1.0		
Pullout strength uncracked concrete[4]	$N_{p,uncr}$	lb (kN)	N/A	N/A	4,185 (18.6)	3,380 (15.0)	4,010 (17.8)	5,500 (24.5)	4,085 (18.2)	6,015 (26.8)	8,050 (35.8)	N/A	N/A	N/A
Pullout strength cracked concrete[4]	$N_{p,cr}$	lb (kN)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	8,835 (39.3)
Pullout strength seismic[4]	$N_{p,seis}$	lb (kN)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	8,795 (39.1)
Normalization factor, uncracked concrete	n_{uncr}	-	N/A	N/A	0.37	0.46	0.50	0.50	0.50	0.42	0.47	N/A	N/A	N/A
Normalization factor, cracked concrete, seismic	n_{cr}	-	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.50
Tension, axial stiffness														
Axial stiffness in service load range	β_{uncr}	lb/in.	175,802			137,147			153,923			342,679		
	β_{cr}	lb/in.	79,861			97,983			69,627			75,715		

For SI: 1 inch = 25.4 mm, 1 lbf =4.45N, 1 psi=0.006895 MPa. For pound-inch units: 1mm=0.03937 inches

[1] Figure 2 illustrates the installation parameters
[2] The KB-TZ2 is considered a ductile steel element in accordance with ACI 349-13 Appendix D.
[3] For all design cases, $\psi_{c,N} = 1.0$. The appropriate effectiveness factor for cracked concrete (k_{cr}) or uncracked concrete (k_{uncr}) shall be used.
[4] For all design cases, $\psi_{c,P} = 1.0$. Tabular value for pullout strength is for a concrete compressive strength of 2,500 psi. Pullout strength for concrete compressive strength greater than 2,500 psi may be increased by multiplying the tabular pullout strength by $(f'c/2,500)^n$ for psi, or $(f'c/17.2)^n$ for MPa, where n is given as n_{uncr} for uncracked concrete and n_{cr} for cracked concrete and seismic. NA (not applicable) denotes that pullout strength does not need to be considered for design.

Table 5 Carbon Steel KB-TZ2 Design Information, Shear

Design parameter	Symbol	Units	Nominal Anchor Diameter (in)												
			3/8			1/2				5/8			3/4		
Anchor O.D.	d_a	in. (mm)	0.375 (9.5)			0.500 (12.7)				0.625 (15.9)			0.750 (19.1)		
Effective min. embedment[1]	h_{ef}	in. (mm)	1-1/2 (38)	2 (51)	2-1/2 (64)	1-1/2 (38)	2 (51)	2-1/2 (64)	3-1/4 (83)	2-3/4 (70)	3-1/4 (83)	4 (102)	3-1/4 (83)	3-3/4 (95)	4-3/4 (121)
Shear, steel failure modes															
Strength reduction factor for steel in shear[2]	Φ	-	0.65			0.65				0.65			0.65		
Steel strength in shear	V_{sa}	lb (kN)	3,225 (14.4)	3,385 (15.1)		5,535 (24.6)		6,875 (30.6)		10,255 (45.6)			13,805 (61.4)		
Steel strength in shear, seismic	$V_{sa, seis}$	lb (kN)	3,225 (14.4)	3,385 (15.1)		5,535 (24.6)		6,875 (30.6)		10,255 (45.6)			13,805 (61.4)		
Shear, concrete failure modes															
Strength reduction factor for concrete in shear	Φ	-	0.70			0.70				0.70			0.70		
Load bearing length of anchor in shear	l	in. (mm)	1-1/2 (38)	2 (51)	2-1/2 (64)	1-1/2 (38)	2 (51)	2-1/2 (64)	3-1/4 (83)	2-3/4 (70)	3-1/4 (83)	4 (102)	3-1/4 (83)	3-3/4 (95)	4-3/4 (121)
Coefficient for pryout strength	k_{cp}	-	1	1	2	1	1	2	2	2	2	2	2	2	2

For SI: 1 inch = 25.4 mm, 1 lbf =4.45N, 1 psi=0.006895 MPa. For pound-inch units: 1mm=0.03937 inches

[1] Figure 2 illustrates the installation parameters
[2] The KB-TZ2 is considered a ductile steel element in accordance with ACI 349-13 Appendix D.

Table 6 Stainless Steel KB-TZ2 Design Information, Shear

Design parameter	Symbol	Units	Nominal Anchor Diameter (in)											
			3/8			1/2			5/8			3/4		
Anchor O.D.	d_a	in. (mm)	0.375 (9.5)			0.500 (12.7)			0.625 (15.9)			0.750 (19.1)		
Effective min. embedment[1]	h_{ef}	in. (mm)	1-1/2 (38)	2 (51)	2-1/2 (64)	2 (51)	2-1/2 (64)	3-1/4 (83)	2-3/4 (70)	3-1/4 (83)	4 (102)	3-1/4 (83)	3-3/4 (95)	4-3/4 (121)
Shear, steel failure modes														
Strength reduction factor for steel in shear[2]	Φ	-	0.65			0.65			0.65			0.65		
Steel strength in shear	V_{sa}	lb (kN)	4,615 (20.5)	4,885 (21.7)		8,345 (37.1)			12,355 (55.0)			16,560 (73.7)		
Steel strength in shear, seismic	$V_{sa, seis}$	lb (kN)	4,615 (20.5)	4,885 (21.7)		8,345 (37.1)			12,355 (55.0)			13,470 (59.9)		
Shear, concrete failure modes														
Strength reduction factor for concrete in shear	Φ	-	0.70			0.70			0.70			0.70		
Load bearing length of anchor in shear	l	in. (mm)	1-1/2 (38)	2 (51)	2-1/2 (64)	2 (51)	2-1/2 (64)	3-1/4 (83)	2-3/4 (70)	3-1/4 (83)	4 (102)	3-1/4 (83)	3-3/4 (95)	4-3/4 (121)
Coefficient for pryout strength	k_{cp}	-	1	1	2	1	2	2	2	2	2	2	2	2

For SI: 1 inch = 25.4 mm, 1 lbf =4.45N, 1 psi=0.006895 MPa. For pound-inch units: 1mm=0.03937 inches

[1] Figure 2 illustrates the installation parameters
[2] The KB-TZ2 is considered a ductile steel element in accordance with ACI 349-13 Appendix D.

Table 7 Minimum Edge Distance, Spacing, and Concrete Thickness for KB-TZ2

Design parameter	Symbol	Units	Nominal Anchor Diameter (in)												
			3/8			1/2				5/8			3/4		
Effective min. embedment	h_{ef}	in. (mm)	1-1/2 (38)	2 (51)	2-1/2 (64)	1-1/2 (38)	2 (51)	2-1/2 (64)	3-1/4 (83)	2-3/4 (70)	3-1/4 (83)	4 (102)	3-1/4 (83)	3-3/4 (95)	4-3/4 (121)
Min. Member thickness	h_{min}	in. (mm)	3-1/4 (83)	4 (102)	5 (127)	3-1/2 (89)	4 (102)	5 (127)	5-1/2 (140)	5 (127)	5-1/2 (140)	6 (152)	5-1/2 (140)	6 (152)	8 (203)
Carbon Steel															
Min. edge distance	c_{min}	in. (mm)	5 (127)	2-1/2 (64)	2-1/2 (64)	8 (203)	2-3/4 (70)	2-3/4 (70)	2-1/4 (57)	4-1/2 (114)	3-1/2 (89)	2-3/4 (70)	5 (127)	4 (102)	3-1/2 (89)
	for s ≥	in. (mm)	8 (203)	6 (152)	5 (127)	12 (305)	5-1/2 (140)	9-3/4 (248)	5-1/4 (133)	6-1/2 (165)	5-1/2 (140)	7-1/4 (184)	10 (254)	5-3/4 (146)	5-1/2 (140)
Min. anchor spacing	c_{min}	in. (mm)	5 (127)	2-1/4 (57)	2 (51)	12 (305)	3-1/2 (89)	3 (76)	2 (51)	4-1/2 (114)	2-3/4 (70)	2-1/4 (57)	4-1/2 (114)	3-3/4 (95)	3-3/4 (95)
	for c ≥	in. (mm)	8 (203)	3-1/2 (89)	4 (102)	8 (203)	10 (254)	8 (203)	4-3/4 (121)	5-1/2 (140)	7 (178)	4-1/4 (8	6 (152)	7-1/2 (191)	4-3/4 (121)
Stainless Steel															
Min. edge distance	c_{min}	in. (mm)	5 (127)	2-1/2 (64)	2-1/2 (64)	*	2-3/4 (70)	2-1/2 (64)	2-1/4 (57)	4 (102)	3-1/4 (83)	2-1/4 (57)	5 (127)	4 (102)	3-3/4 (95)
	for s ≥	in. (mm)	8 (203)	5 (127)	5 (127)	*	5-1/2 (140)	4-1/2 (114)	5-1/4 (133)	7 (178)	5-1/2 (140)	7 (178)	11 (279)	7-1/2 (191)	5-3/4 (146)
Min. anchor spacing	s_{min}	in. (mm)	5 (127)	2-1/4 (57)	2-1/4 (57)	*	2-3/4 (70)	2-1/2 (64)	2 (51)	5-1/2 (140)	2-3/4 (70)	3 (76)	5 (127)	4 (102)	4 (102)
	for c ≥	in. (mm)	8 (203)	4 (102)	3-1/2 (89)	*	4-1/8 (105)	5 (127)	4-3/4 (121)	5-1/2 (140)	4 (102)	4-1/4 (108)	8 (203)	6 (152)	5-1/4 (133)

For SI: 1 inch=25.4mm * length not available

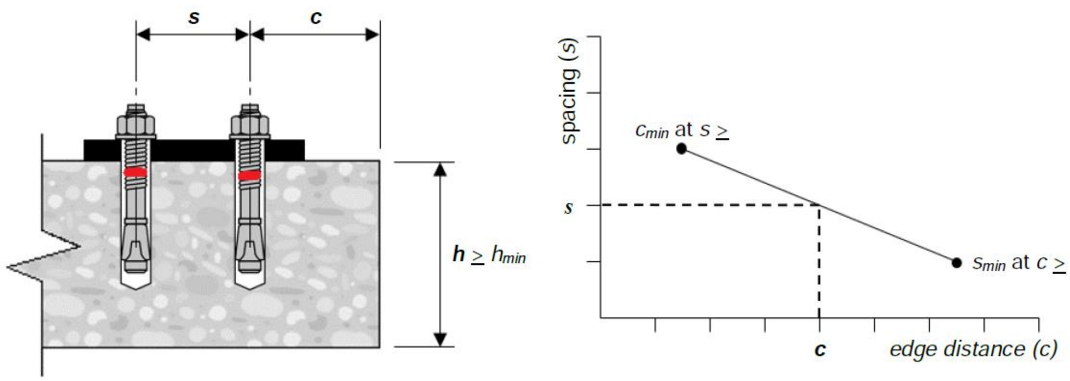

Figure 4 Parameters; Interpolation of Minimum Edge Distance and Anchor Spacing