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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*

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

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

III NUCLEAR SSC DESIGN AND ANALYSIS REQUIREMENTS

A. This Section provides requirements for the design and analysis of hazard category 1, 2, and 3 (HC 1–3) nuclear facility structures, systems and components (SSCs) to include non-building structures. General requirements of ESM Chapter 5, Section I also apply.

1. Requirements are also provided for facilities with biological hazards, or significant chemical or toxicological hazards.

B. Per DOE Order 420.1C*, Facility Safety*, the design of HC 1–3 nuclear facilities is required to comply with DOE-STD-1189, *Integration of Safety into the Design Process*, and DOE-STD-1020, *Natural Phenomena Hazards Analysis and Design Criteria for DOE Facilities*.

C. DOE O 420.1C applies to the design and construction of new HC 1–3 nuclear facilities and major modifications to them. With regard to the design and construction of "non-major modifications" (to existing nuclear facilities), DOE-STD-1020 includes associated requirements and guidance.

D. DOE-STD-1189, Appendix A provides criteria for specification of the seismic design basis of SSCs. These criteria relate to radiological hazards only.

1. Appendix A invokes the use of ANSI/ANS 2.26, and ASCE/SEI 43.[1]

2. A seismic design basis consists of both a seismic design category (SDC, which is not the same as IBC SDC) and a limit state (LS) in order establish performance expectations for SSCs subjected to seismic hazards.

E. Each SSC in a HC 1–3 nuclear facility must be assigned an NPH Design Category (NDC) for each NPH, which includes seismic (S), wind (W), flood (F), precipitation (P), and volcanic eruption (V).

1. Each NPH has three design categories (1 – 3). For example, for seismic, the NDCs are SDC-1 – SDC-3; for wind, WDC-1 – WDC-3; etc.[2]

2. Category 1 is associated with the smallest /lowest consequence of SSC failure, while category 3 is associated with the largest/highest consequence.

F. The determination of which of the three categories applies (in a given design/evaluation, and for each NPH) shall be based on analysis of the severity of unmitigated consequences of SSC failure—to both the collocated worker and the public—using the

[1] ANS Standard 2.26 was adopted by DOE for purposes of seismic design basis specification. The seismic design classifications of ANS 2.26 are to be used in association with DOE radiological criteria provided in DOE-STD-1189 Appendix A. It is intended that the requirements of Section 5 of ANS 2.26 and Appendix A of ANS 2.26 be used for the selection of the appropriate Limit States for SSCs performing the safety functions specified. The resulting combination of Seismic Design Category (SDC) and Limit State (LS) selection provides the seismic design basis for SSCs to be implemented in design through ASCE 43.

[2] Use of NDC-1-3 only is also consistent with DOE-STD-1189. Use of NDC-4 and -5 categories is not expected at LANL so not addressed in this Chapter

categorization methodology given in DOE-STD-1189, App. A performed by the safety analyst. The NDC for LANL nuclear[3] facilities shall per Table III-1.

Table III-1. NDC Determination

	Unmitigated Consequence of SSC Failure from a Seismic Event	
Category[4]	**Collocated Worker**	**Public**
NDC-1	dose < 5 rem	not applicable
NDC-2	5 rem < dose < 100 rem	5 rem < dose < 25 rem
NDC-3	dose > 100 rem	dose > 25 rem

The following supplemental direction to ANS 2.26 shall be used when selecting SDCs and LSs:

1. To ensure that SSC Limit State selected for determining the permissible maximum stress, strain, deformation, or displacement is consistent with the safety function(s) of the SSC as determined from hazard and accident analyses, the safety analyst (responsible for hazard and accident analysis), the NPH Design Engineer (responsible for seismic design, and for coordinating the selection of SSC Limit State and SSC Seismic Design Category; likely to be the structural Engineer of Record, or SEOR), and the Equipment Expert (responsible for the mechanical or electrical design of the equipment, likely to be the Cognizant System Engineer, or CSE) must work together and evaluate the functional requirements of the SSC and its subcomponents in relation to their modes of failure.[5]

   a. There are four limit states to be considered:

Limit State	An SSC designed to this Limit State…
A	…may sustain large permanent distortion short of collapse and instability (i.e., uncontrolled deformation under minimal incremental load) but shall still perform its safety function and not impact the safety performance of other SSCs.
B	…may sustain moderate permanent distortion but shall still perform its safety function. The acceptability of moderate distortion may include consideration of both structural integrity and leak-tightness.
C	…may sustain minor permanent distortion but shall still perform its safety function. An SSC that is expected to undergo minimal damage during and following an earthquake such that no post-earthquake repair is necessary may be assigned this Limit State. An SSC in this Limit State may perform its confinement function during and following an earthquake.
D	…shall maintain its elastic behavior. An SSC in this Limit State shall perform its safety function during and following an earthquake. Gaseous, particulate, and liquid confinement by SSCs is maintained. The component sustains no damage that would reduce its capability to perform its safety function.

[3] Per DOE-STD-1020 2.1.5 & 2.3.9 (and indicated subsequently herein), Table III-1 could also apply to non-nuclear facilities with significant biological, chemical, or toxicological hazards.
[4] In accordance with DOE-STD-1189 Appendix A and DOE-STD-1020 para. 2.2.2.5, Table III-1 includes only NDC-1, 2 and 3. However, ANS 2.26 includes SDC-4 and 5. Per 1020 (by way of reference to 1189 App. A for the assignment of NDCs), NDC-4 and NDC-5 will rarely occur at DOE sites.
[5] From DOE-STD-1020-2012

b. In conceptual design, if there are no bases for defining seismic-related design basis accidents (DBAs), then Hazard Category 2 facility structural designs must default to ANSI/ANS 2.26 SDC-3, Limit State D.

2. If the safety functions of an SSC include confinement and/or leak tightness, irrespective of the Seismic Design Category (SDC) of the SSC, a Limit State C or D must be selected, unless the SSC functional requirements can be described as given in Limit State B column for SSC Type "confinement barriers and…" in ANS 2.26, Appendix B.[6]

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

G. NPH design of facilities with biological hazards, or significant chemical or toxicological hazards, shall be based on an appropriate NDC level.[7] Specifically, the NDC for SSCs that provide protection from biological, chemical or toxicological hazards shall be determined based on the unmitigated consequences of SSC failure from an NPH event. The methodology for this characterization should be consistent with DOE-STD-1189 and direction from the responsible program office. In the case of seismic NPH, the LS shall be chosen to be consistent with safety and containment criteria.

1. If the SSC in question is also for nuclear safety then the higher of the NDCs determined from the application of the radiation dose criteria and the criteria for biological/chemical/toxicological dose shall be used.

2. If biological, chemical, or toxicological hazards in a non-nuclear facility are not significant then NPH design can follow the IBC (i.e., starting with assignment of Risk Category in accordance with paragraph 1604.5).

H. NPH Design Category (NDC) -1 and -2 SSCs:

1. SDC-1 and SDC-2 SSCs shall be designed following the IBC as shown Table III-2 herein.

a. Nonstructural elements attached to SDC-1 and SDC-2 structures shall be designed in a manner that allows for seismic deformations of the structure without excessive damage to the structure.

2. WDC-1 and WDC-2 SSCs shall be designed using the criteria in IBC for Risk Category II and Risk Category IV facilities, respectively.

3. FDC-1 and FDC-2 SSCs[8] shall be designed such that the following requirements are met:

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[6] DOE-STD-1189-2008, App. A

[7] Treating facilities with bio hazards in this manner comes from a recommendation in the draft 1020 HDBK. And the same goes for G.2 (i.e., IBC-design for insignificant bio, chem, toxic hazards).

[8] Pertains to such SSCs in/on FDC-1 and FDC-2 facilities *only*. In other words, this provision (H.3) does not 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, see Flood Loads under "NDC-3 SSCs" 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 DOE-STD-1020 Tables 5-2A and 5-2B, and DOE-STD-1088 para.5.1.6, for more detail.

a. The design basis flood loads, F_a, resulting from flood hazards shall be based on IBC requirements for Risk Category II and Risk Category IV facilities, respectively.[9]

b. For SSCs that cannot be located above the Design Basis Flood Level (DBFL), the applicable provisions of DOE-STD-1020 paras. 5.5.5.1 and/or 5.5.5.2 shall be met.

4. PDC-1 and PDC-2 SSCs shall be designed using the criteria in IBC for Risk Category II and Risk Category IV facilities, respectively, using the applicable Importance Factor.

5. Lightning design, specifically SSC categorization for lightning hazards, and designing SSCs for lightning protection is addressed in ESM Ch. 7, Electrical.

6. VDC-1 and VDC-2 SSCs[10] shall be designed such that both of the following requirements are met:

a. The load combinations in IBC shall be used substituting the ash load, V, for the snow load, S, in the load combination equations.

b. Design shall consider ash density and the impact of precipitation combined with ashfall. Refer to DOE-STD-1020, para. 8.4.4 for additional detail guidance.

7. General Design Criterion: Any SSC that may fail to perform its safety functions when subjected to water intrusion or submergence (resulting from flooding), flooding (resulting from local precipitation), or wind-borne missiles (resulting from wind-related hazards), shall be protected, either by barriers designed to withstand these effects, or by placing the SSCs in a location that precludes the occurrence of them. For requirements and guidance on such protection refer to the following paragraphs of DOE-STD-1020:

a. Wind: 4.4.2.3

b. Flood: 5.5.4.1, 5.5.4.7, 5.5.5.1, 5.5.5.2, and 5.5.5.7

c. Precipitation: 7.5.4.3, and 7.5.3.5

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[9] IBC requirements are predicated on the establishment of flood hazard areas (FHAs). LANL has not completed a probabilistic hazard flood assessment of the site; however, LA-14165 Section 2.6 provides sufficient information to allow for design to this hazard, subject to the following changes: DOE-STD-1020 and DOE-STD-1023 will be taken to be (=) DOE-STD-1020-2012, 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 DOE-STD-1020 Table 5-3 (i.e., add Case 3 for both Levee/Dam Failure and Local Precipitation; and delete Snow as a source of flooding). Finally, note that IBC requires compliance with both ASCE 24 and ASCE 7, and that ASCE 7 uses symbols other than F_a to define the flood loads that must be designed for (which are stipulated in DOE-STD-1020-2012, para. 5.5.1.5, under its definition of F_a).

[10] A formal probabilistic hazard assessment for volcanic activity at the site has not been performed. However, in LA-14426, "*Preliminary LANL Volcanic Hazards Evaluation*" indicates that the annual probability of volcanic eruption somewhere within the LANL region in on the order of $1x10^{-5}$. Based on this low probability of occurrence, design specifically for the volcanic hazard is not considered.

Table III-2. Effect of Limit State on SDC-1/SDC-2 Design

SDC	IBC Risk Category	I(1)	Limit State A	Limit State B	Limit State C	Limit State D
1	II	1.0	$R_a = R^{(2)}$	$R_a = 0.8$ R, but $R_a \geq 1.2$	$R_a = 0.67$ R, but $R_a \geq 1.2$	$R_a \geq 1.0$
2	IV	1.5	N/A	$R_a = R$	$R_a = 0.833$ R, but $R_a \geq 1.2$	$R_a \geq 1.0$

(1): I = Importance Factor from Table 1.5-2 of ASCE 7
(2): R = Response Modification Coefficient given in ASCE 7; R_a = Actual (reduced) R to be used in the design. Refer to DOE-STD-1020-2012 Table 3-1, footnote 1 for more details.

With the exception of the requirements specified above (i.e., Para. H), the design/analysis of NDC-1 and NDC-2 SSCs shall be in accordance with Section II of this Chapter. The design/analysis of NDC-3 SSCs shall be per this Section (III).

1.0 NDC-3 STRUCTURES AND STRUCTURAL SYSTEMS

1.1 Acceptance Criteria

1.1.1 General

A. The structural demands (member forces, displacements, etc.) shall be calculated using the loads of paragraph 1.2, the load combinations of paragraph 1.3, and the analysis procedures of paragraph 1.4.

B. Linear analyses shall meet the strength acceptance criteria in paragraph 1.1.2(A) and the displacement criteria in paragraph 1.1.3.

C. Nonlinear analyses shall meet the displacement acceptance criteria in paragraph 1.1.3. The capacity of yielding elements in nonlinear analyses shall meet the strength design criteria in paragraph 1.1.2(B).

D. The following materials may be used for the design of NDC-3 structures and structural systems provided that they are designed in accordance with the material design code/standard, as amended by these criteria.

Material	**Material Design Code/Standard**[11]
Reinforced Concrete	ACI 349*****
Structural Steel	ANSI/AISC N690*****
Stainless Steel	ANSI/ASCE 8
Cold Formed Steel	AISI Standard S100
Reinforced Masonry	TMS 402/ACI 530/ASCE 5 (ACI 530)

[11] 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.

* NOTE: Per DOE O 420.1C Chg 1, 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 collocated worker; at LANL, SS SSCs that do so at the Category NDC-3 level must be designed to ACI 349 and ANSI/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.

E. Adoption of a material design code/standard in paragraph 1.1.1(D) includes all provisions in that code/standard, including seismic detailing provisions, as amended by this criteria.

F. Seismic

1. In designing a new SDC-3 facility that has a direct-confinement safety function, and/or contains SSCs that do, the LS shall be D unless LS C is proven (by analysis) to be adequate.

G. Wind

1. Barriers and other SSCs that are provided for wind or missile protection of SSCs with safety functions, shall be placed in a WDC category equal to, or higher than, the category of the SSC to be protected. These protective SSCs, or barriers, shall be designed using stress, strain, or deformation limits appropriate for the protective function and the failure mode of the barrier.

H. Flood

1. For a facility in a flood-prone area which has SSCs of FDC-3, dikes, barriers, and/or enclosures shall be built for providing flood protection to the SSCs. These dikes, barriers, and enclosures shall be placed in an FDC equal to the category of the SSC(s) to be protected.

2. Protective SSCs (like those mentioned in 1.1.1.H.1) shall be designed using stress, strain, deformation limit, or leak tightness criteria appropriate for the protective function and the failure mode of the barrier. DOE-STD-1020, Section 5.5.4.1 provides design criteria for protective SSCs.

I. Precipitation

1. The failure of the safety function of some SSCs, when subjected to a precipitation-related hazard, can occur not only because of excessive deformation or distortion (e.g., roof loading), but also as a result of inundation of the SSC or intrusion of precipitation water runoff into or onto the SSC. In selecting an SSC PDC and SSC design methods and criteria, such inundation and water intrusion-related failure modes (e.g., shorting or malfunctioning of an electric circuit or equipment) shall also be considered in determining unmitigated SSC failure consequences.

2. Barriers, enclosures, dikes, and other SSCs, that are provided for precipitation protection of SSCs, with safety functions, shall be placed in a PDC equal to the SSC to be protected.

3. Protective SSCs (like those mentioned in 1.1.1.I.2) shall be designed using stress, strain, deformation limit, or leak tightness criteria appropriate for the

protective function and the failure mode of the barrier following the requirements in DOE-STD-1020 Section 5.5.4.1.

4. Since LANL is a large site with varying topography, the precipitation runoff levels may vary from facility to facility.

J. Volcanic Eruption

1. Until such time as a credible and quantifiable risk is determined, this risk shall be taken as zero[12]. Note that LANL is undertaking new natural phenomena hazards assessments and new criteria may exist for ash fall loads due to volcanism; check with the Chapter 5 POC for criteria on ashfall loads prior to undertaking new nuclear design activities.

1.1.2 Strength Acceptance Criteria

A. For linear analyses, the total demand acting on an element shall be less than or equal to the element's code capacity:

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

where the code capacity is developed in accordance with paragraph 1.5.

B. For nonlinear analyses, the capacity of all elements, including yielding elements, shall be limited to code capacities (i.e., design strengths). 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…

where ϕM_n ϕV_n and ϕP_n are the design strengths in flexure, shear, and axial force, respectively which are developed in accordance with paragraph 1.5.

1.1.3 Deformation Acceptance Criteria

A. The deformation limits, serviceability requirements and deformation acceptance criteria for impulsive and impact loads of the material codes/standards identified in 1.1.1(D) shall apply.

B. The deformation acceptance criteria for seismic loads shall be in accordance with Section 5.2.3 of ASCE 43.

[12] LA-14426, *Preliminary Volcanic Hazards Evaluation for [LANL] Facilities and Operations*, Sept. 2010, concluded "…Volcanism in the vicinity of the Laboratory is unlikely within the lifetime of the facility (ca. 50 – 100 years) but cannot be ruled out..."

1.2 Loads[13]

1.2.1 Dead Load (D)

A. 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.

1. For evaluation of seismic response, not only must the contribution of nonstructural components to D be considered, but their contribution to stiffness must be as well.

B. Unit weights of 150 pcf for reinforced concrete and 490 pcf for rolled steel shall be used in the derivation of D.

C. The value of D used in the evaluation of existing facilities shall be based on the best estimate of existing dead loads and shall include consideration of/allowance for future use.

1.2.2 Live Load, Experiment Blast Load (L_{EB})

A. Determine L_{EB} based on project-specific requirements. 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.

1.2.3 Crane Loads (C or C_{cr}[14])

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

1.2.4 Fluid Loads (F)

A. 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.

1.2.5 Lateral Soil Pressure Loads (H)

A. 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 LANL. A default value of 0.5 for at-rest lateral soil pressure coefficient, K_o, may be used to determine H[15].

B. In determining H, the density of the backfill or native soil, whichever is greater, shall be used.

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[13] The load criteria given are consistent with the requirements of DOE-STD-1020, many of which are consistent with the minimum load requirements in ASCE 7. Regarding NPH-load criteria, by and large, DOE-STD-1020 requires that they be developed through the use of site-specific characterization and probabilistic hazard assessment.
[14] AISC N690 uses "C," while ACI 349 uses C_{cr}
[15] Average value 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).

C. 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).

1.2.6 Precipitation Loads (S, R, & D_i for building roofs[16])

A. Snow Loads (S): Determine S using the procedure prescribed in Chapter 7.0 of ASCE 7 as amended by the following:

1. ASCE 7 Section 7.2 Ground Snow Loads, *p_g,* substitute the following text: Use 32 psf for p_g, the ground snow load used in determining S for roofs of PDC-3 structures.[17]

2. ASCE 7 Section 7.3.3 *Importance Factor, I*, substitute the following text: Use 1.2 for the value of I for PDC-3 structures.

B. Rain Loads (R): Determine R using the procedure prescribed in Chapter 8.0 of ASCE 7 as amended by the following:

1. 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.[18]

2. In addition, all of the provisions of ASCE 7 Chapter 8 shall be complied with/evaluated.

3. Finally, roofs shall have positive drainage, and shall be equipped with secondary drains or scuppers.

C. 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.).[19] Thus, D_i need not be considered for buildings and building-like structures.

Determine D_i using the procedure prescribed in Chapter 10.0 of ASCE 7 as amended by the following:

1. Use 0.25 inch for t, the nominal ice thickness due to freezing rain.[20]

2. Use 2.5 for I_i, the importance factor or multiplier on ice thickness.[21]

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[16] DOE-STD-1020 stipulates two precipitation hazards: 1. Site flooding from extreme local precip events (vs. flooding from extreme regional events, rivers, levee/dam failure, etc.), and 2. Building roof loading. Only the latter hazard is addressed herein. The former hazard is addressed in ESM Ch. 3, Civil.

[17] This load comes from LA-UR-06-6329: *Site-Specific Extreme Rainfall and Snow Hazard Curves at Los Alamos National Laboratory, Los Alamos, New Mexico*; E. Lawrence, 10/15/06. Table 4 of this document lists snow loads for many return periods (RPs), one of which is not the 2,500-yr RP required by DOE-STD-1020 for PDC-3 structures. Given the 31.55 psf in Table 4 for a 2,000-yr. RP, and the loads associated with other RPs, it seems reasonable to round the 31.55 up to 32 to arrive at the 2,500-yr value.

[18] Rate from LA-UR-06-6329. Table 1 (which was used to develop Table 4 referred to in footnote above) lists the amounts (in inches) of daily rainfall for many RPs, one of which is not the 2,500-yr RP required by DOE-STD-1020 for PDC-3 structures. Given the 4.59 inches for 2,000 yrs., and the amounts associated with other RPs, it seems reasonable to round the 4.59 up to 4.6 to arrive at the 2,500-yr value.

[19] LA-14165 indicates that, while ice storms don't occur at LANL, icing can still occur on ice-sensitive SSCs ("…in low spots where cold air settles…") at LANL

[20] LA-14165

3. The wind load concurrent with ice loading, W_i, shall be based on a 40-mph 3-second-gust wind speed.[22]

4. Use 1.0 for I_w, the importance factor for wind on ice-covered SSCs.[23]

1.2.7 Thermal Loads (T_o and T_a)

A. 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.

1. 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).

B. 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.

1. 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*.[24]

1.2.8 Pipe/Equipment Reactions (R_o and R_a)

A. 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.

1. The use of "loads" herein applies to piping and equipment reactions, or related internal moments and forces.

1.2.9 Operating Basis Earthquake (E_o)

A. 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.*

1.2.10 Wind Loading (W)

A. Determine W using the procedure prescribed in Chapter 26.0 of ASCE 7 as amended by the following:

1. 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 for WDC-3 SSCs.[25]

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[21] Ibid. Values in this report for t and I_i were chosen from a range of values such that they roughly correlated with research performed by Mertz. Also, ASCE 7 Table C10-1 indicates I_i = 2.5 for 1,400-yr RP, and 1,400 yrs is as high as the table goes.
[22] From LA-14165 and ASCE 7
[23] From ASCE 7 Table C10-1, which indicates 1.0 applies to all RPs (i.e., 25–1,400 yrs).
[24] 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.

2. ASCE 7 Section 26.7.3 Exposure Categories*,* substitute the following text: Exposure C shall apply for all cases.[26]

3. Replace Section 26.10.3 Protection of Openings, with the following*:*

26.10.3 Protection Against Wind-Borne Missiles

WDC-3 SSCs that have the potential to be adversely affected by the impact of missiles resulting from the design basis wind shall be evaluated using the missile characteristics given in 26.10.3.1.[27]

26.10.3.1 Wind-borne Missile Characteristics[28]

A 2″×4″ timber plank, weighing 15 pounds, with a 50-mph horizontal velocity, at a height $\leq$ 30 ft above grade.

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

Refer to Appendix D of this document for DBE loads and related requirements.

1.2.12 Tornado Loading (W_t)

A. Tornado hazard need not be considered at LANL; hence, W_t shall be taken as zero.[29]

1.2.13 Differential Pressure (P_a)

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

   1. The use of "load" herein applies to maximum differential pressure load, or related internal moments and forces.

1.2.14 Impulsive and Impactive Loads (Y_r, Y_j, Y_m)

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

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[25] DOE-STD-1020 allows for use of ANS-2.3-2011 to determine V. Using the 2,500-yr. RP required by 1020 for WDC-3, and ANS-2.3 Figs. 1 and 4 [Fig. 1 indicates LANL is barely in Region III (i.e., it's almost in Region II), which results in use of Fig. 4], and Table 3, V = 118 mph; however, since LANL is located in special wind region (SWR, per ASCE 7 Sect. 26.5.2), ANS-2.3 recommends use of a site-specific study (SSS) to determine V. In lieu of a SSS, since ASCE 7 Fig. 26.5-1B (RC III and IV) indicates V = 120 mph for LANL (i.e., disregarding its location in SWR; ref. ESM Ch. 5, Sect. II, para. 1.3.B), use of V = 120 mph is more reasonable than use of 118 mph.

[26] Based on ASCE 7 26.7.2 and 26.7.3, and the fact that Exposure D at LANL is extremely rare, if not nonexistent

[27] It's important to note the difference between missiles generated by straight-line winds and those generated by tornado winds. DOE-STD-1020, Sect. 4.3.3.3 (applicable since ANS-2.3 is being used for wind-related hazard design parameters), and ANS-2.3 Table 4 require consideration of missile impacts generated by tornado wind speeds > 100 mph. Using the figures and tables of ANS-2.3 referred to in previous footnotes, the WDC-3 tornado wind speed at LANL is < 100 mph; thus, missile impact resulting from such is not applicable. However, missiles impacts can result from straight-line winds. See footnote 26 for more detail.

[28] Ref. DOE-STD-1020-2002, Table 3-1; and LANL ESM Ch. 5 Sect. III, rev. 5, para. 1.2.12.4. ANS-2.3 Sect. 4, Wind-generated missiles, indicates that wind speeds > 110 mph can generate missiles that are large enough to cause structural damage. Given this, LANL's 120-mph WDC-3 wind speed, and DOE-STD-1020-2002 inclusion of missile criteria for wind (and tornado), it is prudent to require consideration of impact by this missile.

[29] Ref. DOE-STD-1020 Table 4-1 and para. 1.2.10.1 herein. This is consistent with LA-14165 Sects. 2.3.2 and 2.3.3.

1. The use of "loads" herein applies to the specific type of load as follows:

   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).

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

1.2.15 Self-Straining Forces

A. 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.

1. 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.

B. 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 combinations in paragraph 1.3. Estimation of these effects shall be based on a realistic assessment of such effects occurring in service.

1.2.16 Flood Loads

A. New FDC-3 SSCs shall be located on mesa tops since they are not susceptible to the flood-related hazards that are otherwise applicable at LANL.

B. Hazards from localized flooding resulting from extreme precipitation (i.e., snow, rain, or ice), and the design of site drainage systems to mitigate these hazards, are addressed in DOE-STD-1020 Sections 7.3–7.5.

1. Per 1020 Section 7.2.2.8, new safety-related structures shall not be built in areas where flooding from site precipitation can occur unless flood mitigation measures are provided.

1.3 Load Combinations

1.3.1 General

A. The load-combination equations indicated below (i.e., paragraphs 1.3.2 and 1.3.3) shall be augmented in accordance with DOE-STD-1020-specific requirements (i.e., those indicated in the following two sub-paragraphs; 1.3.1.B and 1.3.1.C).

B. DOE-STD-1020 Section 7.5.4.2.2 includes unique load-combination equations for the roof precipitation loads (i.e., S, R, and D_i). The equations are unique in that they treat S, R and D_i as extreme loads.[30]

C. Although the DOE-STD-1020 load-combination equations for wind loads (W), found in Section 4.4.2.2.2, are not unique in treatment of W as an extreme load[31]; they are unique in that both the live load (L) and the roof live load (L_r) occur in the same equation and receive the same emphasis.

D. DOE-STD-1020 Section 8.4.2 includes unique load-combination equations for ash load (i.e., V). The equations are unique in that they treat L_r as an extreme load.

1.3.2 Reinforced Concrete Members and Reinforced Masonry Members

A. The load combinations of ACI 349 shall be used to combine demands for reinforced concrete and masonry structures/members as amended by the following:

1. 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 in paragraph 1.3.4.[32]

B. The load combinations in ACI 530 are omitted.

1.3.3 Structural Steel Members, Cold-Formed Steel Members and Stainless Steel Members

A. The load combinations of AISC N690 shall be used to combine demands for structural steel, cold-formed steel and stainless steel structures/members as amended by the following:

1. 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 factor in Section 1.3.4.[33]

2. The AISC N690 load combination equations that shall be used for consideration of concurrently-acting D_i and W_i are NB2-4 and NB2-13 (i.e., severe equations, which are not to be confused with 'extreme ones' referred to in 1.3.1.B herein) subject to the following modifications: D_i shall be added to D, and W_i shall be taken to be W.[34]

B. The load combinations in ASCE 8 are omitted.

C. The load combinations in AISI Standard S100 are omitted.

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[30] The authoritative nuclear standards—ACI 349 and AISC N690—treat R and S as normal loads in their load-combo equations. Ref. 349 equations 9-2 and 9-3, and N690 equations NB2-2 and NB2-3.
[31] Same as what's done in ACI 349 and AISC N690. Ref. load-combo equations 9-7 and NB2-7, respectively.
[32] Refer to ACI 349 Definitions, safe shutdown earthquake (SSE). In short, in DOE space, E_{ss} = E (i.e., DBE per para. 1.2.11 herein).
[33] Refer to AISC N690 Glossary, Safe shutdown earthquake (SSE). In short, in DOE space, E_s = E (i.e., DBE per para. 1.2.11 herein).
[34] Similar is done in ASCE 7 Sects. 2.3.4 and 2.4.3 (for commercial/non-nuclear).

1.3.4 Inelastic Energy Absorption Factor (F_μ)

A. The factor F_μ is defined in ASCE 43 for bending moment, shears and axial loads, except as amended below:[35]

1. Use 1.0 for F_μ for non-compact members or members subject to local buckling.

B. For F_μ for masonry and cold-formed steel members use 1.0.[36]

C. 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 3.6.4 are met.[37]

1.4 Analysis Procedures

A. The structural analysis shall be consistent with the requirements in the material codes identified in paragraph 1.1.1(D).

B. Seismic modeling and analyses shall be consistent with the requirements of ASCE 43 and ASCE 4.

C. Time histories shall meet the requirements of ASCE 43.

1.5 Capacities

A. ASCE 43 Section 4.2 shall be used for determining capacities as amended by the following:

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

   a. ACI 349 Appendix C, Alternative Load and Strength-Reduction Factors*,* shall not be used in new design.[38]

   b. Delete ACI 349 Section 9.2.10.[39]

   c. Design of anchorage to concrete shall be in accordance with Appendix A of this Section.[40]

2. 4.2.4, Structural Steel, replace all text with the following text:

   a. Design of structural steel structures shall be performed using 1.3.3.A herein and AISC N690.[41]

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[35] ASCE 43 para. 5.1.2.1 defines F_μ as indicated here
[36] Ref. ASCE 43 Sections 4.1.2 and 5.1.2.1
[37] Ibid
[38] In order for the 349 capacities to be compatible with the ASCE-7 demands, 349 App. C can't be used. See Record of Revisions, Rev. 2.
[39] The reduction in 'E' in 9.2.10 is only applicable to SDC-5 SSCs; thus, use at LANL is not expected. Furthermore, per R9.2.10 -- "…this provision assumes shear controls the design. Where that cannot be established, the licensed design professional is cautioned in the use of this provision." – it's safest to simply omit the provision.
[40] Criteria provided in Appendix A for anchorage to concrete come from a variety of sources (e.g., DOE-O-420.1C, ACI 349, ASCE 7, LANL-specific requirements, etc.).

3. 4.2.5, Reinforced Masonry, replace all text with the following text:

   a. Design of reinforced masonry structures shall be performed using 1.3.2 herein and ACI 530.[42]

   b. Add the following sentence to ACI 530 Section 6.2: Post-installed (PI) anchor bolts shall not be used.[43]

   c. Delete ACI 530 Chapter A, Empirical Design of Masonry.[44]

4. Add 4.2.6, Cold-Formed Steel:[45]

   a. Design of cold-formed steel (CFS) structures shall be performed using 1.3.3 herein and AISI S100.

5. Add 4.2.7, Stainless Steel:[46]

   a. Design of stainless steel structures shall be performed using 1.3.3 herein and ASCE 8.

1.6 Detailing Requirements

A. Follow ASCE 43 Section 6, Ductile Detailing Requirements.

B. Structural steel designs shall allow for/ensure the following:[47]

   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.

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[41] Only N690 is referred to by DOE O 420.1C and DOE-STD-1020 for NDC-3 – NDC-5. And N690 has changed significantly (to include inclusion of strength design/LRFD, and reference to AISC 341 for seismic detailing) since ASCE 43 was published.
[42] Since the publication of ASCE 43, the IBC merely refers to ACI 530 for design capacity (due to significant change in 530).
[43] LANL-specific requirement. 1st, DOE directives and standards are silent on nuclear design of masonry. 2nd, there is no nuclear design standard for masonry (like ACI 349 or AISC N690). 3rd, per 530 C8.1.3 and C9.1.6, design of PI anchors is not explicitly addressed (i.e., as is done in ACI 349 App. D).
[44] Per 530 App. A, para. A.1.2.2, Empirical design is prohibited at LANL
[45] LANL-specific requirement; DOE and ASCE 43 are silent on nuclear design of CFS
[46] LANL-specific requirement; DOE and ASCE 43 are silent on nuclear design of stainless steel
[47] OSHA requirements. For additional discussion, see ESM Chapter 5 Section II, paragraph 7.1.1, and LANL Master Specification Section 05 1000.

1.7 Additional Structural Design Considerations

1.7.1 Foundation Design

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

1. Minimum embedment depth of foundations is 36 inches unless the foundation bears directly on welded tuff.[48]

2. Interconnect all SDC-3 spread footing type foundations using tie beams[49]. 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. 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.[50]

1.7.2 System Interaction

A. Potential interactions between SSCs (i.e., two-over-one phenomenon) shall be considered in the facility safety analysis and in the assignment of NDC (and LS for the seismic hazard) to SSCs. See ANS 2.26, Section 6.3.2.4 for more detail. For existing SSCs, following the guidance contained in DOE/EH-0545 Chapter 7, Seismic Interaction, may suffice, or at least be of use in addressing interaction(s).

1.7.3 Progressive Collapse

A. Refer to Section II (of this Chapter) paragraph 1.6.

1.7.4 Permanent Explosive Facilities and Facilities Containing Explosives, or Those that can be Affected by Such

A. Refer to Section II (of this Chapter), paragraph 1.5.

1.7.5 Additional Seismic Analysis/Design Considerations

A. Follow ASCE 43 Chapter 7, Special Considerations.

2.0 NDC-3 NONSTRUCTURAL SYSTEMS AND COMPONENTS

A. The structural design of NDC-3 nonstructural systems and components shall conform to paragraph 1.0 herein in addition to the following requirements.

B. The design and documentation requirements for nonstructural systems and components vary, depending upon the functional requirements. For NDC-3 projects, the functionality requirements of nonstructural systems and components (that are SC, SS, or OEITS) are

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[48] LA-14165 Sect. 3.3.1
[49] The tie-beam provision was added to address the uncertainties associated with distributed faulting on the Pajarito Plateau.
[50] ASCE/SEI 43 Sect. 7.4.3 requires compliance with the IBC SDC-D provisions for foundation design and detailing. See the footnote associated with ESM Ch. 5 Sect. II para. 3.3.2 for more detail.

normally specified in preliminary hazards assessment documents, or documented safety analysis.

C. The seismic design/qualification of nonstructural systems and components shall be in accordance with the requirements of ASCE 43 and paragraphs 1.1.1.F.1 and 1.1.1.F.2 herein.

1. Seismic input to nonstructural systems and components that are in/on structures shall consist of in-structure response spectra per the requirements stipulated in ASCE 4 (only to the extent these are consistent with the requirements of ASCE 43).

2. Seismic input to nonstructural systems and components that are not in/on structures shall consist of the DBE stipulated in paragraph 1.2.11, modified to account for the response of adjacent structures per ASCE 4 Section 3.4.1.3, Subsystem Input Away from Reference Location.

3. Seismic analysis/design of buried systems and components shall be in accordance with ASCE 4 Section 3.5, Special Structures.

D. 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 43 Section 8.3, and DOE/EH-0545.

E. The design/qualification of systems and components for non-seismic NPH loads shall include the loads and load combinations in paragraphs 1.2 and 1.3.2, respectively, herein.

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

A. Refer to Section I of this Chapter (*paragraph 1.3.C.*)

4.0 QUALITY ASSURANCE AND PEER REVIEW

A. Refer to Section I of this Chapter (*paragraph 1.6.F*).

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 Gr B7, A354 Gr BD, A449, A572, A588, A687, or F1554 material. ASTM F1554 Gr. 55 is the preferred material specification in AISC N690. Welding and mechanical properties of headed studs shall comply with AWS D1.1 (per AISC 360) and ESM Chapter 13, *Welding, Joining, and NDE*.

2. Allowed post-installed (PI) anchors are the Hilti HDA Undercut anchor (i.e., HDA)[1], KB-TZ Expansion anchor (i.e., KB-TZ), and HIT-RE 500 V3 Adhesive anchor with threaded rod and post-installed reinforcing bar connection (i.e., V3).[2]

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% • cannot use stainless version since not nuclear qualified[3]
V3	• most flexibility with anchor steel ○ many more types and sizes, non-proprietary, and threaded rod or rebar.	• weaker than HDA • must derate by 25% • onerous to design & install

3. HDA

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

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

[1] Ref LBO memos under "LANL and Other Documents."
[2] KB-TZ and V3 originally approved by VAR-10314; this document supersedes.
[3] So use limited to indoor, dry environment

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 isn't "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. V3
   a. Lengths: As indicated in ICC-ES ESR-3814.
   b. Material types: As indicated in ICC-ES ESR-3814.
6. Purchase, installation, and testing requirements for NDC-3 anchors:
   a. For the HDA and KB-TZ, see the LANL Master Specification Section 05 0521.
   b. For the NDC-3 V3, a LANL specification section is in development; coordinate with the Chapter POC.

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.

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

C. Ductile Embedment Design: Applies when the strength of all 'concrete failure modes' is higher than the strength of embedment steel (i.e., failure is in the steel). See ACI 349, D.3.6.1 for more information.

D. Nonductile Embedment Design: Applies when the strength of embedment steel is higher than the strength of the concrete failure modes (i.e., failure is in the concrete). See ACI 349, D.3.6.3 for more information.

A.3 Applicable Codes and Standards

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

A.5 LANL Documents

LANL Master Spec Section 05 0521	Post-Installed Concrete Anchors Purchase – Nuclear Safety
ESM Chapter 13	Welding, Joining, and NDE
LANL Memo ES-DO-18-015 Rev.1	LANL Building Official Rescinded Approval of Drillco Maxi-Bolt Post-Installed Concrete Anchors[4]
LANL Memo ES-DO-18-016	LANL Building Official Approval of Hilti HDA Post-Installed Concrete Anchors for Use in NDC-3 Applications

A.6 Prerequisites for Determining Anchor Design Loads

A. The SEOR shall coordinate with appropriate personnel (i.e., refer to paragraph III.F) to determine and document the NPH Design Category (i.e., NDC) of the anchorage and, where NDC = SDC, the Limit State (LS) as well. 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), and

2. System interaction: refer to Section II, Appendix A of this chapter (*paragraph A.5.C*).

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

A.7 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.

A.8 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. Design the lugs in accordance with the recommendations of the most current version of the AISC *Steel Design Guide 1 - Base Plates and Anchor Rod Design*. Do not rely on anchor bolts to transmit shear load in elements of the SLRS to foundation elements.

[4] Projects underway may continue to use the Maxi-Bolt.

A.9 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 ESM Structural 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 ESM Structural 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. 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.[5]

   3. Welding to PI anchors is not permitted.

[5] Spec section 05 0521 requires existing concrete to be GPR-scanned (to preclude anchors from damaging rebar), the results of which might require fixture/anchor relocation. And, even when avoidance of rebar (based on existing drawings) is attempted during anchor design, it's still possible for GPR-scan results to require relocation.

4. PI anchors shall not be used in masonry walls. Through-bolting may be an acceptable alternative.[6]

5. A minimum of two (2) PI anchors shall be used at each connection[7]. 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. 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. And, in such designs, anchors shall not be located closer than 6 inches to pre-/post-tensioning steel.

A.10 Design Requirements

A. For the HDA, anchor design strength shall be determined in accordance with ACI 349-13, Appendix D.

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, paragraph D.4.1 except subparagraph D.4.1.1 shall be modified to read as follows:[8]

For the design of anchors

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

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

Note: The note at the bottom of the "HDA paragraph" (above) applies here too.

C. For the V3, anchor design strength shall be determined in accordance with ACI 318-14 Chapter 17 (except subparagraph 17.3.1.1 shall be modified to read as follows[8]) and ICC-ES ESR-3814:

The design of anchors shall be in accordance with Table 17.3.1.1... [paragraph] 17.2.3 and [subparagraph] 17.3.1.2… Finally, after complying with the aforementioned requirements, a 0.75 factor shall be applied to each applicable tensile and shear strength and those factored strengths shall meet or exceed the applicable, respective tensile and shear loads (e.g., $0.75\varphi N_n \geq N_{ua}$, $(0.75)(0.55)\varphi N_{ba} \geq N_{ua,s}$, $0.75\varphi V_n \geq V_{ua}$, etc.).

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

[6] None of the nuclear codes or standards address or include post-installed anchorage to masonry.
[7] This requirement isn't applicable to the installation of new steel columns. One of OSHA provisions 29 CFR 1926, Subpart R, Section 755(a) is that new steel columns be erected using at least four (4) anchors (for stability/construction safety).
[8] KB-TZ and V3 derating: ACI uses a factor of 0.75 (25% reduction) on elements that fail in a brittle manner (i.e., shear) to express the reliability of brittle failure. By LANL treating all installations with the additional conservatism of an assumed brittle failure mechanism, LANL is confident that demand/capacity ratios are within the intent of the current ACI 349 without unequivocal documentation of same. In addition, both anchors are qualified for use in cracked concrete and for seismic loads. Finally, the HIT-RE 500 V3 will not creep under such reduced sustained tension loads and its acceptance by ASCE 43 was expected in 2019.

in concrete do not exceed $7.5\sqrt{f_c'}$) under service loads, including wind and seismic forces.

E. Appendices B and C (herein), as well as the following, shall apply to the design of HDA and KB-TZ anchors respectively. 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)[9].

D.5.2.6 Use the following $\psi_{c,N}$ value: 1.0 (unless "uncracked" is proven in accordance with paragraph A.10.B above).

D.5.3.5 Use the following $\psi_{c,P}$ value: 1.0 (unless "uncracked" is proven in accordance with paragraph A.10.B above).

D.6.2.7 Use the following $\psi_{c,V}$ value: 1.0 (unless "uncracked" is proven in accordance with paragraph A.10.B above).

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 α = 5/3.

[9] Since PI anchors are used in hardened concrete, it is highly unlikely that supplementary reinforcement (as defined in ACI 349 App. D) exists in a prospective anchor location.

APPENDIX B: DESIGN FIGURES AND TABLES FOR HILTI HDA UNDERCUT ANCHORS

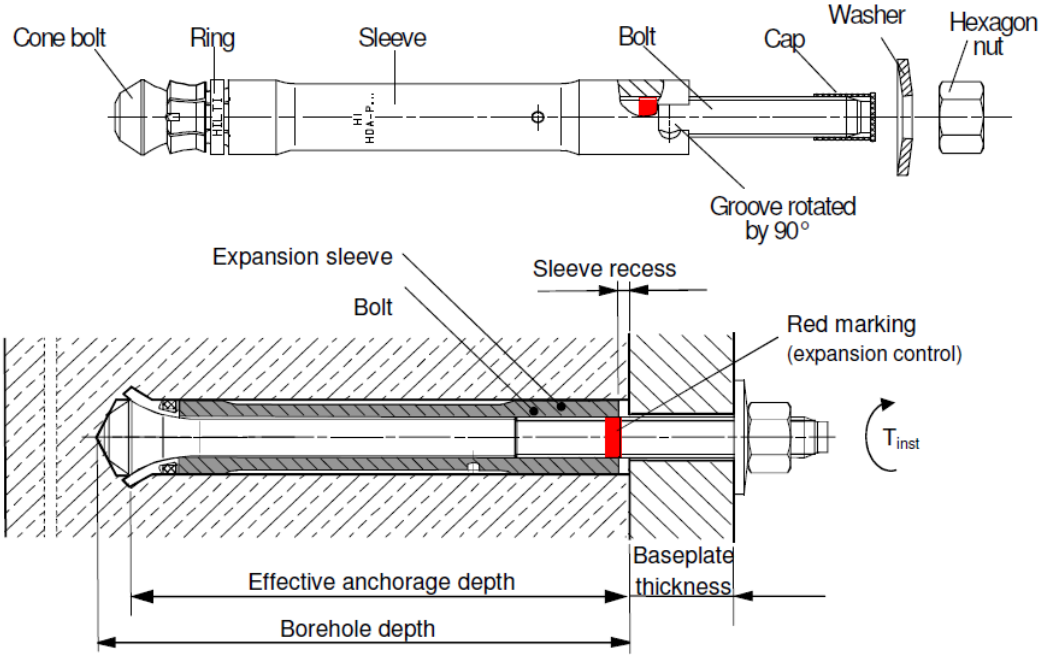

Figure III.B-1 Pre-setting HDA-P and HDA-PR Anchors (Pre-setting)

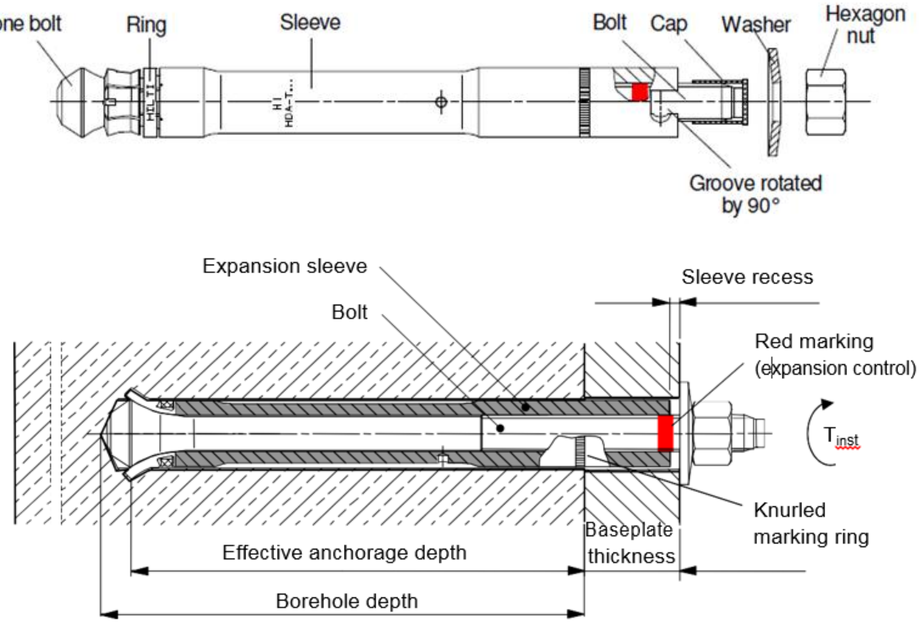

Figure III.B-2 Through-fastening HDA-T and HDA-TR Anchors (Through-fastening)

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]	β	10³ lb/in	80 / 100						

Table III.B-2 Notes

1. Actual h_{ef} HDA-T is given by $H_{ef,\,min} + (t_{fix,max}\text{-}t_{fix})$ 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	$15 \le t_{fix} < 20$	$^{5}/_{8} \le tfix < ^{13}/_{16}$	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	$15 \le t_{fix} < 20$	$^{5}/_{8} \le tfix < ^{13}/_{16}$	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 (set 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

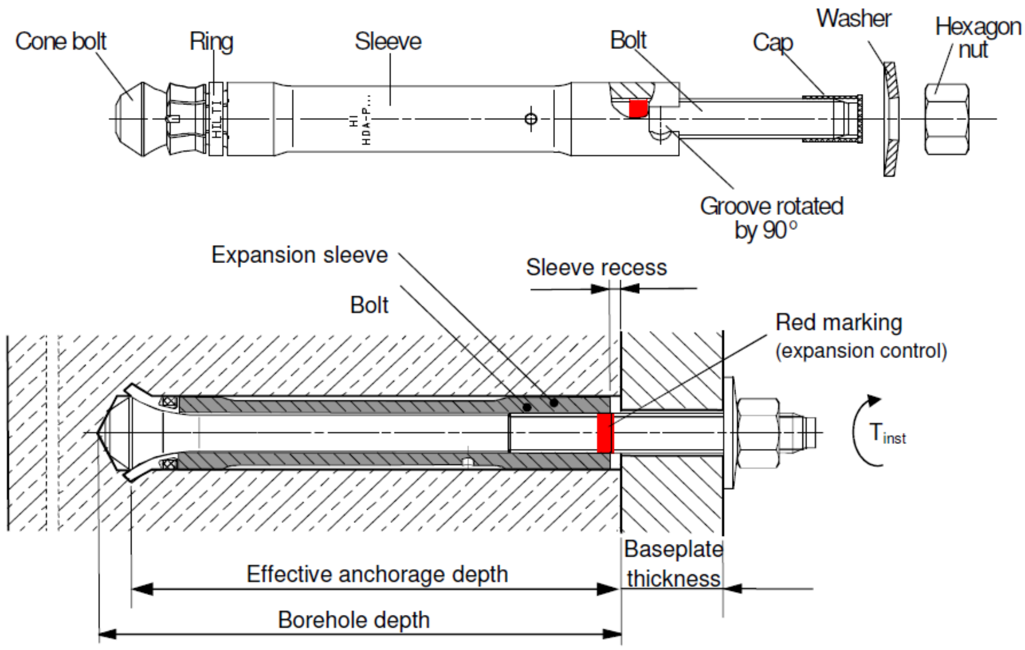

Figure III.B-3 Pre-setting HDA-P DUC Anchors (Pre-setting)

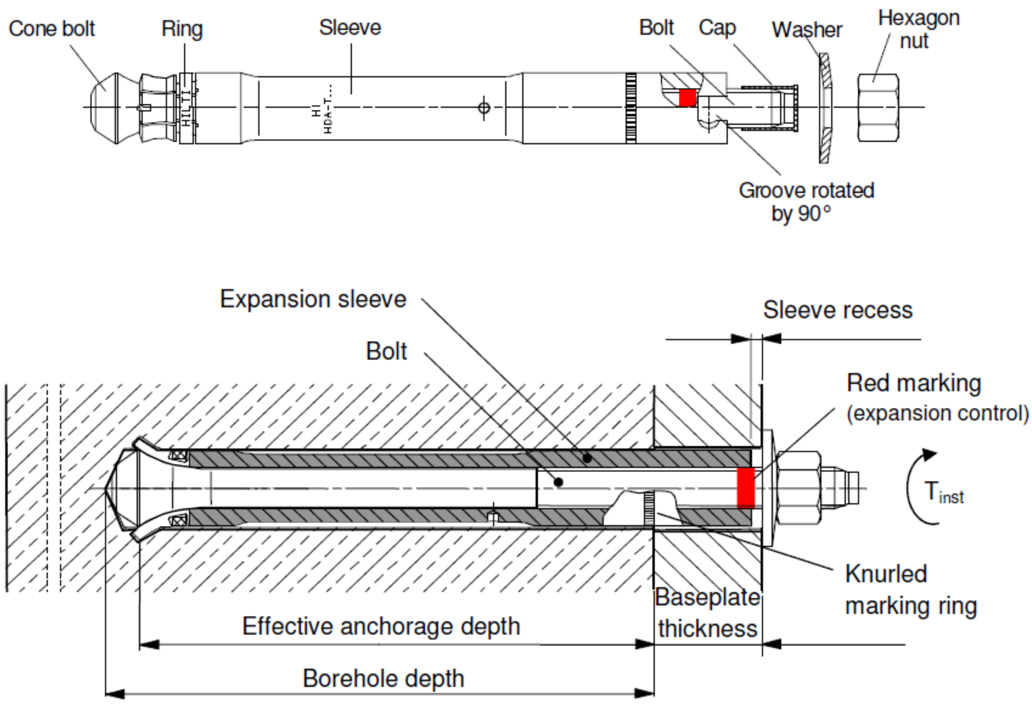

Figure III.B-4 Through-fastening HDA-T DUC Anchors (Through-fastening)

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-2 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-2 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-2 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: DESIGN FIGURES AND TABLES FOR HILTI KB-TZ EXPANSION ANCHORS

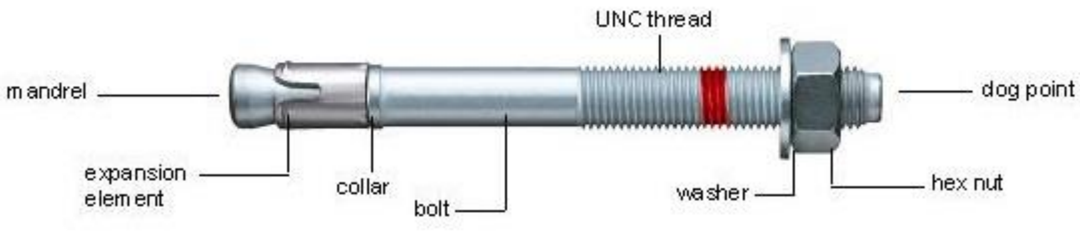

Figure III.C-1 KB-TZ Anchor

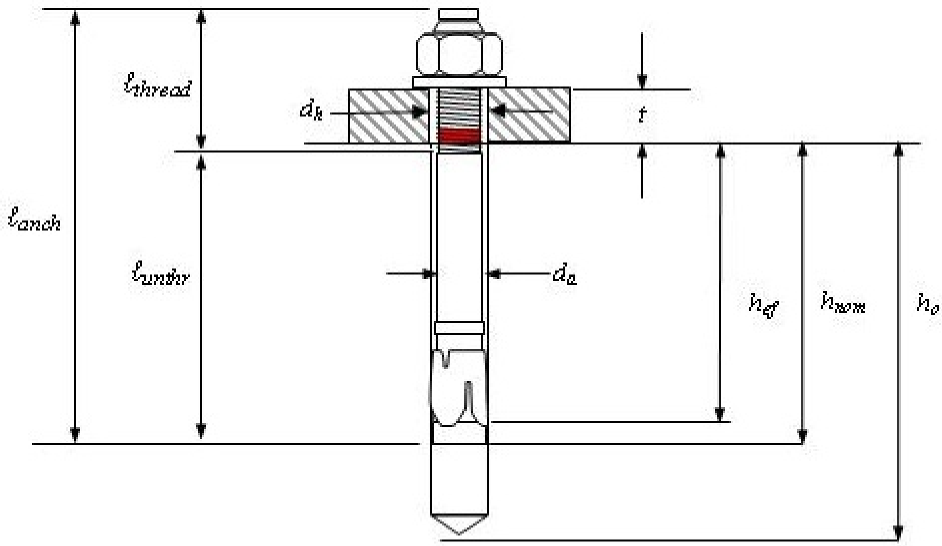

Figure III.C-2 KB-TZ Anchor Installed

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.

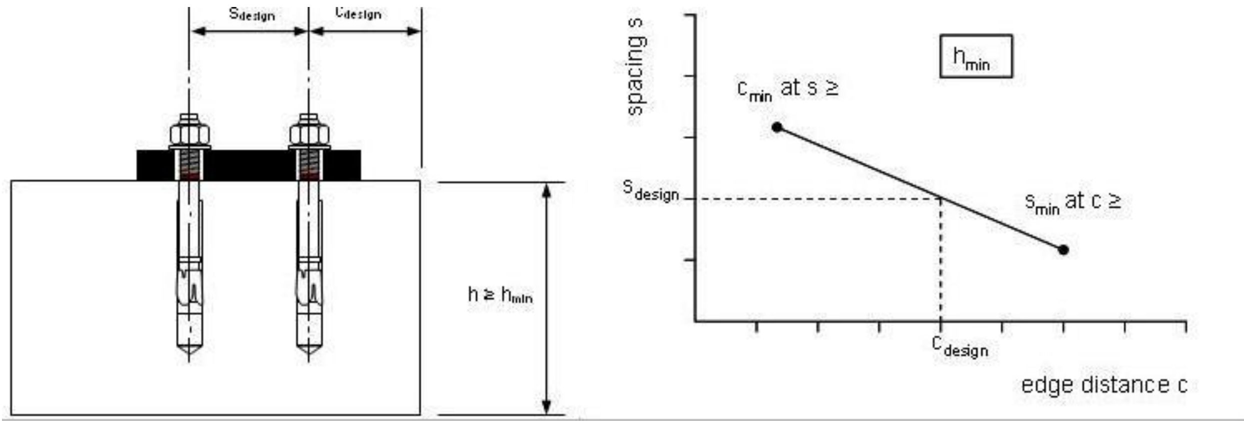

Figure III.C-3 Interpolation of Minimum Edge Distance and Minimum Spacing for KB-TZ

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

<table>
<tr><th rowspan="2">SETTING INFORMATION</th><th rowspan="2">Symbol</th><th rowspan="2">Units</th><th colspan="15">Nominal anchor diameter (in.)</th></tr>
<tr><th colspan="3">3/8</th><th colspan="4">1/2</th><th colspan="4">5/8</th><th colspan="4">3/4</th></tr>
<tr><td>Effective min. embedment</td><td>h_{ef}</td><td>in.
(mm)</td><td colspan="3">2
(51)</td><td colspan="2">2
(51)</td><td colspan="2">3 ¼
(83)</td><td colspan="2">3 1/8
(79)</td><td colspan="2">4
(102)</td><td colspan="2">3 ¾
(95)</td><td colspan="2">4 ¾
(121)</td></tr>
<tr><td>Min. thickness of fastened part[1]</td><td>t_{min}</td><td>in.
(mm)</td><td colspan="3">0
(0)</td><td colspan="2">¾
(19)</td><td colspan="2">¼
(6)</td><td colspan="2">3/8
(9)</td><td colspan="2">¾
(19)</td><td colspan="2">0
(0)</td><td colspan="2">7/8
(23)</td></tr>
<tr><td>Min. dia. Of hole in fastened part</td><td>d_h</td><td>in.
(mm)</td><td colspan="3">7/16
(11.1)</td><td colspan="4">9/16
(14.3)</td><td colspan="4">11/16
(17.5)</td><td colspan="4">13/16
(20.6)</td></tr>
<tr><td>Standard anchor lengths</td><td>ℓ_{anch}</td><td>in.
(mm)</td><td>3
(76)</td><td>3 ¾
(95)</td><td>5
(127)</td><td>3 ¾
(95)</td><td>4 ½
(114)</td><td>5 ½
(140)</td><td>7
(178)</td><td>4 ¾
(121)</td><td>6
(152)</td><td>8 ½
(216)</td><td>10
(254)</td><td>5 ½
(140)</td><td>7
(178)</td><td>8
(203)</td><td>10
(254)</td></tr>
<tr><td>Threaded length (incl. dog point)</td><td>ℓ_{thread}</td><td>in.
(mm)</td><td>1 ½
(38)</td><td>2 ¼
(57)</td><td>3 ½
(93)</td><td>1 5/8
(41)</td><td>2 3/8
(60)</td><td>3 3/8
(86)</td><td>4 7/8
(124)</td><td>1 ½
(38)</td><td>2 ¾
(70)</td><td>5 ¼
(133)</td><td>6 ¾
(171)</td><td>2 ½
(63)</td><td>4
(103)</td><td>5
(128)</td><td>7
(179)</td></tr>
<tr><td>Unthreaded length</td><td>ℓ_{unthr}</td><td>in.
(mm)</td><td colspan="3">1 ½
(39)</td><td colspan="4">2 1/8
(54)</td><td colspan="4">3 ¼
(83)</td><td colspan="4">3
(77)</td></tr>
</table>

[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 SSC EXAMPLES

Adapted for LANL from American National Standard ANSI/ANS-2.26-2004 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[1]	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.

[1] 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. [2]	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[3]	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.

[2] 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.

[3] 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 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.[1]

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.[2]

3. Response spectra at intermediate frequencies shall be obtained by log-log interpolation. Response spectra at intermediate damping shall be obtained using the interpolation procedure in ASCE 4 Section 2.2.1.

C. Soil-Structure Interaction. The requirements of ASCE 4 Section 3.3 shall be followed, except that the Wave Incoherence provision, given in Section 3.3.1.10 of ASCE 4, shall not be used.

D. 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}). Paragraph 1.2.5.B herein applies.

E. Site Limitation: Structures shall not be located within 50 feet of known active faults. 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 per 40 CFR 264.[3]

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

1. A minimum fault displacement of 2 cm (0.8 in.) at the surface shall be used for design[4].

[1] And TA-50, with approved variance addressing geotechnical consistency
[2] Except for TA-55, PC-3 (SDC-3) ground motion were derived following DOE-STD-1023 and the results of URS Corp, "Update of the Probabilistic Seismic Hazard Analysis and Development of Seismic Design Ground Motions at the Los Alamos National Laboratory," May, 2007. The derivation of this ground motion for the site can be found in LANL Calculation SB-DO: CALC 08-038, 10/8/08. For TA-55, the SDC-3 ground motion was derived from the results of URS Corp, "Update of the Probabilistic Seismic Hazard Analysis and Development of CMRR Design Ground Motions Los Alamos National Laboratory, New Mexico", October, 2009. See calc SB-DO: CALC-09-024, Rev. 0 for derivation.
[3] Modified version of ASCE 7 Sect. 11.8.1. Ref. ESM Ch. 5 Sect. II para. 1.4.2.A.
[4] From fault rupture studies.

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

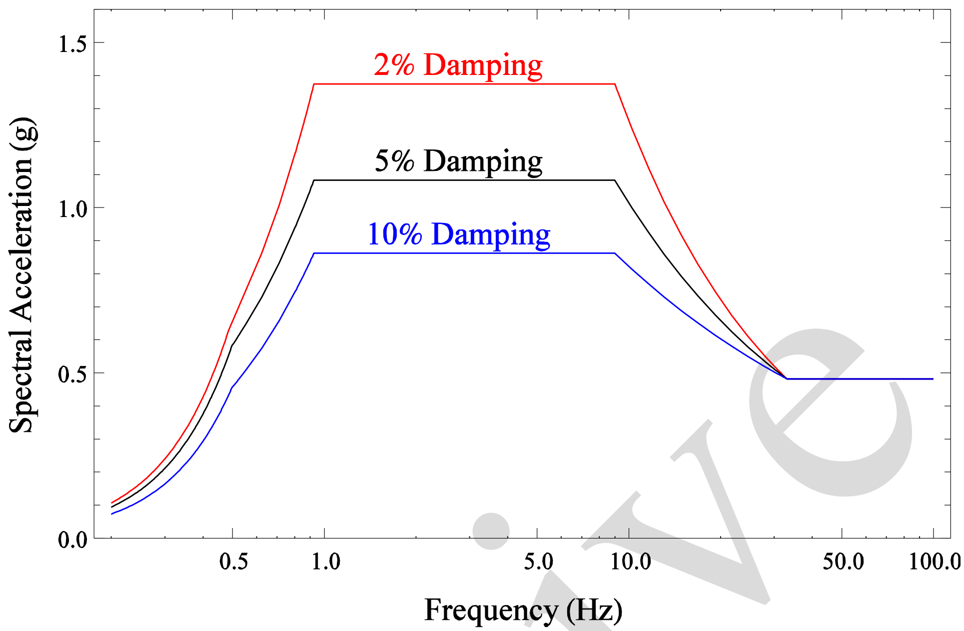

Figure III.D-1 Site Wide Free-Field Surface SDC-3 DBE Horizontal DRS

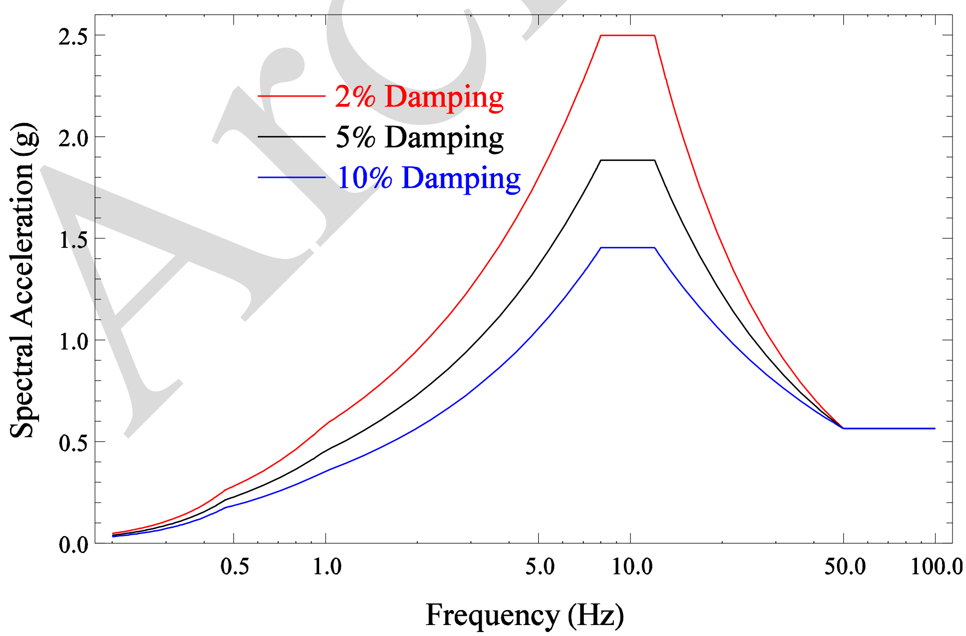

Figure III.D-2 Site Wide Free-Field Surface SDC-3 DBE Vertical DRS

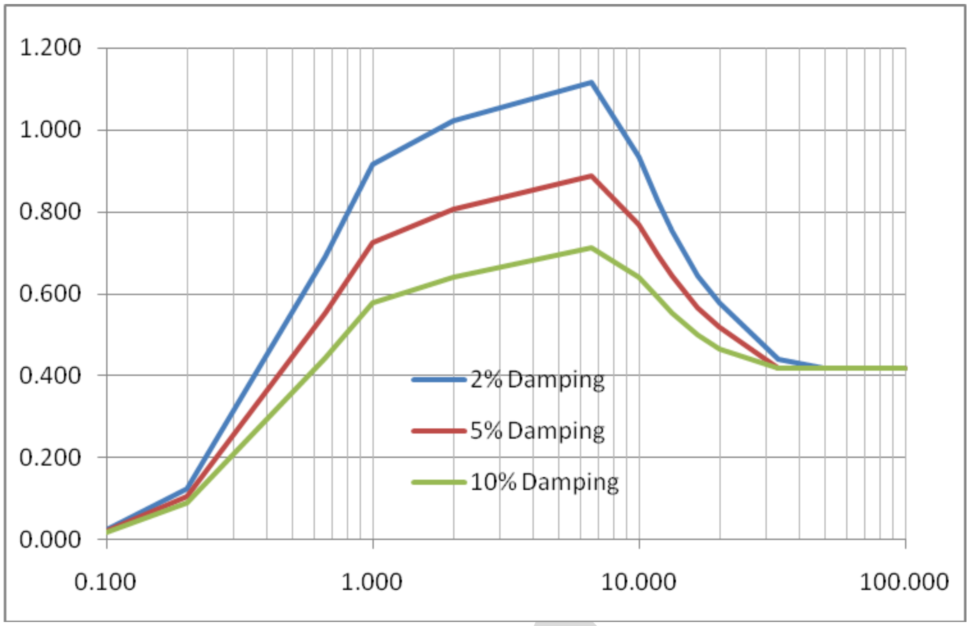

Figure III.D-3 TA-55 Free-Field Surface SDC-3 DBE Horizontal DRS [acceleration (g) on vertical versus frequency (Hz) on horizontal]

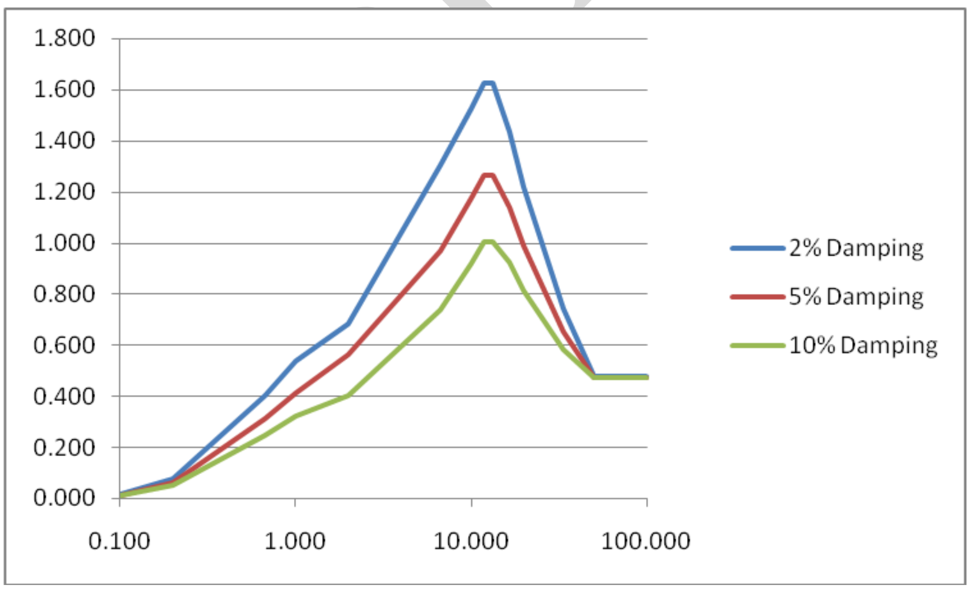

Figure III.D-4 TA-55 Free-Field Surface SDC-3 DBE Vertical DRS [acceleration (g) on vertical versus frequency (Hz) on horizontal]