ENGINEERING CALCULATIONS AND ANALYSIS REPORT

ECAR No.: 418 ECAR Rev. No.: 0 Project File No.: N/A Date: 10/23/2008

Title: ANALYSIS OF TRA-670 UTILITY BATTERY ROOM WALL ANCHORAGES

1. Index Codes

Building/Type: 670 SSC ID: TRA-670-E-58/59 Site Area: TRA

2. Quality Level: 1

3. Objective/Purpose

The purpose of this calculation is to perform a structural evaluation of the bracing necessary to attach the tops of the reinforced concrete block walls enclosing the TRA-670-E58/E59 battery room to the ceiling above the walls. This is based upon recommendations in a recent seismic evaluation of these walls performed by others (Ref. 1). The block walls are not themselves a seismic category 1 SSC, but are considered a potential seismic interaction hazard to the safety related, seismic category 1 battery banks. The objective of the anchorage modification is therefore to provide adequate anchorage to preclude adverse interaction with the battery banks during a safe shutdown earthquake (SSE).

4. Conclusions/Recommendations

The battery room walls, as modified by the proposed design, would be seismically adequate to preclude adverse interaction with the batteries or with other equipment in the event of a safe shutdown earthquake.

The proposed bracing design satisfied the applicable structural design codes.

It is recommended the bracing design installed consistent with the design documented in Drawing 600847.

5. Review (R) and Approval (A) and Acceptance (Ac)[1]:

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DOC. CONTROL		B. R. PIKE	Billie R. Pike 10/30/08

1 Review and Approval are required. See LWP-10200 for definitions and responsibilities.
2 An Electronic Change Request (ECR) indicating final review and concurrence by the listed individuals can be used in lieu of signatures.
3 If Required, per LWP-10200.

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6. Additional Distribution: (Name and Mail Stop)	Document Control:

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Scope and Brief Description

The purpose of this calculation is to perform a structural evaluation of the bracing necessary to attach the tops of the reinforced concrete block walls enclosing the TRA-670-E58/E59 battery room to the ceiling above the walls. This is based upon recommendations in a recent seismic evaluation of these walls performed by others (Ref. 1). The block walls are not themselves a seismic category 1 SSC, but are considered a potential seismic interaction hazard to the safety related, seismic category 1 battery banks. The objective of the anchorage modification is therefore to provide adequate anchorage to preclude adverse interaction with the battery banks during a safe shutdown earthquake (SSE). The anchorage design evaluated here is documented in Reference 6.

Design Inputs and Sources

a) Quality Level 1 is defined per QLD RTC-000633, "Seismic Interaction Structural Anchorage"
b) ATR SC-1 structures, systems and components (SSC) are analyzed for PC-4 loading, which is the project direction of the ATR Seismic Assessment Project, per PLN-588 (Ref. 2).
c) Seismic inputs are taken from the probabilistic soil-structure interaction analysis (PSSI) (Ref 3), as applied in the ARES calculation (Ref. 1).

Results of Literature Searches and Other Background Data

TRA-670-E58/E59 battery room walls were evaluated in ARES calculation number 0602301.01-S-007 (called "group 8" walls in Ref. 1, see page 61). This calculation recommended that the battery room walls be anchored at their tops with supplemental anchorage having an out of plane (to the walls) capacity of at least 400 pounds per linear foot. This recommendation was based upon uncertainty about the attachment of the vertical reinforcement to the ceiling above the walls.

A subsequent drawing search discovered an inactive drawing showing details of the battery room walls which indicated there was no connection of the vertical reinforcement to the ceiling (Ref. 4). Field investigation with re-bar finding equipment to confirm this drawing also revealed that in some places the tops of the vertical bars were as low as 32 inches below the ceiling, leaving the top three courses of concrete block without vertical reinforcement.

In light of this information, it was decided that the out of plane anchorage recommended in reference 1 should take the form of lateral bracing acting at an elevation on the wall where vertical reinforcement was present. A design for such bracing is documented in Reference 6. Evaluation of this design is the subject of this calculation report.

Assumptions

Except where field verification is noted, it is assumed that physical materials and configurations conform to the cited drawings and specifications.

Computer Code Validation

Mathcad version 13 is used. Validation of the calculations is by manual checking performed by the technical checker.

Analysis

The first step in the evaluation of the wall is to analyze the impacts to the dynamic analysis of Ref. 1 created by the proposed bracing modification. The unmodified wall has an effective height of ten feet, which will be lowered to seven feet by the installation of lateral bracing acting three feet below the ceiling. This shortening of the wall is expected to increase the frequency of the wall's response, and move the response closer to the peak of the in-structure response spectra. This will tend to increase the acceleration demands upon the wall. However, a shorter effective height of the wall will also decrease the bending demand for a given acceleration level, which will tend to counteract the frequency effect.

The evaluation method of Reference 1 is reproduced in Appendix A, with the wall height modified and resulting changes in the numerical values propagated through the analysis. The result of this re-evaluation confirms the expected increase in response frequency and acceleration demand, and an overall reduction (improvement) in the demand to capacity ratio of the wall.

The proposed bracing design is then evaluated, with calculations as documented in Appendix A. The conclusion of the bracing design is that the proposed design is adequate.

Recommendations

It is recommended the bracing design be installed consistent with the design documented in Reference 6.

Conclusions

The battery room walls, as modified by the proposed design, would be seismically adequate to preclude adverse interaction with the batteries or with other equipment in the event of a safe shutdown earthquake.

The proposed bracing design satisfies the applicable structural design codes.

References

1. ARES calculation number 0602301.01-S-007, "Seismic Evaluation of Concrete Block and Shielding Walls in TRA-670," Revision 0, July 2008.

2. PLN-588, "ATR Seismic Assessment Plan," Rev. 3, May 2007.

3. "Probabilistic Soil-Structure Interaction of TRA-670," Report No. CJC-INL-C-003 Revision 1, Carl J. Costantino and Associates, Spring Valley NY, November 2007.

4. Drawing 155667, "ATR Battery Bank Enclosure Partial Floor Plan and Details," Revision 3, 1975.

5. AISC, Manual of Steel Construction, Allowable Stress Design, Ninth Edition, 1989.

6. Drawing 600847, "ATR First Basement Battery Area Wall Anchoring Installation and Detail," Revision 0, October 2008.

Appendix A: Calculations

The demand and capacity for the battery room wall are calculated in ARES calculation number 0602301.01-S-007 (referred to as "Group 8" wall in the ARES calculation). The present modifcation to the wall will alter the effective height of the wall, by adding stiff bracing 36 inches down from the top of the wall. This change will affect the capacity, response frequency and demand for this wall. The magnitude and acceptability of these changes need to be evaluated.

The following calcuation is based upon ARES 0602301.01-S-007, to which the cited page numbers will refer. Only those parts of the ARES calculation which affect the final demand to capacity evaluation of the wall are modified here.

Wall parameters from the ARES calculation:

$L := 120 \cdot in - 36 \cdot in$ — Height of the wall (also referred to as H) page 62, minus the depth of the new bracing

$d := 3.81 \cdot in$ — Effective depth to centroid of reinforcement, page 63

$c := 0.25 \cdot in$ — Distance to neutral axis, page 63

$\phi M_n := 10020 \cdot lbf \cdot \frac{in}{ft}$ — Moment capacity, page 63

$w := 52 \cdot \frac{lbf}{ft^2}$ — Weight per unit area, page 64

Calcluate permissible drift, per method of EPRI NP-6041-SL, following ARES page 65

$$F_{CLd} := \frac{\frac{L}{d}}{30}$$

This parameter is now less than 1.0, so take the calculated value

$$F_{CLd} = 0.735$$

$$\Delta mL := \min\left(0.04, \frac{0.005}{\frac{c}{d}} \cdot F_{CLd}\right)$$

Calculated value remains greater than 0.04 radians, sc take as 0.04 radians

$$\Delta mL = 0.04$$

Permissible drift limit, radians

$\Delta_u := \Delta mL \cdot L$

$\Delta_u = 3.36\,in$

Permissible drift limit, adjusted for modified wall height. This value has decreased, and so is conservative relative to the clearance evaluations in the ARES calculation.

$$S_{AC} := g \cdot \left[\frac{8(\phi M_n)}{w \cdot L^2} - 4 \cdot \frac{\Delta_u}{L} \right]$$

$S_{AC} = 2.462\,g$

Seismic capacity, page 66. This has increased (from 1.126g) as expected for the shorter wall.

$$f_s := \left(\frac{1}{2 \cdot \pi} \right) \cdot \sqrt{\frac{1.5 \cdot S_{AC}}{\Delta_u}}$$

$f_s = 3.278\,Hz$

New secant frequency, page 66. This has also increased as expected.

Determine the seismic demand associated with this frequency. Use the tables provided with the ARES calculation, taken from the probabilistic soil-structure interaction analysis. The input will be the north-south (y-direction) average spectral acceleration of the ceiling (node 1234) and floor (node 576), interpolated for 6% damping, as was done in the ARES calculation. Conservatively take the provided frequency of 3.286 Hz rather than interpolate to 3.275 Hz for this small difference.

$Sa5_{1234} := 0.6009 \cdot g$

Spectral acceleration for 5% damped

$Sa10_{1234} := 0.4333 \cdot g$

Spectral acceleration for 10% damped

$$Sa6_{1234} := \left(\frac{6-5}{10-5} \right) \cdot \left(Sa10_{1234} - Sa5_{1234} \right) + Sa5_{1234}$$

$Sa6_{1234} = 0.567\,g$

6% damped spectral acceleration at ceiling

$Sa5_{576} := 0.5095 \cdot g$

Spectral acceleration for 5% damped

$Sa10_{576} := 0.3700 \cdot g$

Spectral acceleration for 10% damped

$$Sa6_{576} := \left(\frac{6-5}{10-5} \right) \cdot \left(Sa10_{576} - Sa5_{576} \right) + Sa5_{576}$$

$Sa6_{576} = 0.482\,g$ — 6% damped spectral acceleration at floor

$Ave_Sa := 0.5 \cdot (Sa6_{1234} + Sa6_{576})$

$Ave_Sa = 0.524\,g$ — Average spectral acceleration applicable to the wall. This has increased from the previous value of 0.4335g from ARES page 66

$\frac{Ave_Sa}{S_{AC}} = 0.213$ — New demand to capacity ratio

The new demand to capacity ratio is less than unity and therefore acceptable, and is lower than the ARES calculated value of 0.39. The net affect of effectively shortining the wall is an improvement in the demand to capacity ratio. The modified wall is acceptable. END OF WALL EVALUATION PER ARES METHOD.

The ARES calculation recommends that top anchorage with a capacity of 400 lbf / ft be provided (ARES page 117). This is based upon the peak of the response spectra, and is therefore not affected by the frequency and seismic demand changes calculated above. However, the distribution of load between the top and the bottom was assumed in ARES to be 50/50, and that will be affected. Calculate the horizontal demand.

$a := 36 \cdot in$ — height of wall above bottom bracing attachment

$SA_{TR} := 0.6974 \cdot g$ — ZPA acceleration at the top of the wall - node 1234

$f_{10} := 16.38 \cdot Hz$ — Page 65, for wall height of 10ft

$f_7 := f_{10} \cdot \left(\frac{L + a}{L}\right)^2$ — Adjust wall frequency for a 7-foot height

$f_7 = 33.429\,Hz$ — Frequency for an 7-foot unbraced length

Look up spectral acceleration values for f7 at the top and bottom of the wall. Use values for 33.24Hz rather than interpolating between frequencies for this small difference. Note that damping interpolation is not required, because at this high frequency, 5% and 10% damped responses are essentially identical.

$SA_B := 0.246 \cdot g$ — Acceleration response at bottom (node 576)

$SA_T := 0.717 \cdot g$ — Acceleration response at top (node 1234)

$SA_A := 0.5 \cdot (SA_B + SA_T)$

Average wall response

$SA_A = 0.481\ g$

$R_B := \dfrac{SA_A \cdot \frac{w}{g} \cdot L}{2}$

The lateral reaction force for the wall section below bracing will be divided between the floor and the bracing.

$R_B = 87.633 \dfrac{lbf}{ft}$

Lateral reaction at the bottom of the wall

$COF := \dfrac{R_B}{w \cdot 10 \cdot ft}$

Check the coefficient of friction that would be required to laterally restrain the bottom of the wall, based upon the bottom lateral reaction and the full wall weight.

$COF = 0.169$

Friction of 0.17 is easilty justified -- wall does not require anchorage at the bottom

$R_T := \dfrac{w}{g} \cdot \left[(SA_A) \cdot \left(\dfrac{5}{8}\right) \cdot 7 \cdot ft + SA_{TR} \cdot 3 \cdot ft \right]$

Calculate a lateral reaction force at the bracing, taking 5/8 of the unbraced reaction at the top (moment end) for a moment-pinned condition, and all of the wall above the bracing is also reacted at the top

$R_T = 218.336 \dfrac{lbf}{ft}$

This evaluation shows that the 400 pound per linear foot specified by ARES is a conservative load fo the braced design. Use 400 lb/ft.

The capacity of the bracing needs to be evaluated. The bracing is shown in drawing 600847. Future references to sheet numbers will be for this drawing unless otherwise stated.

$F_r := 400 \cdot \dfrac{lbf}{ft}$

Horizontal demand, ARES page 117

$L_{door} := 3 \cdot ft + 4 \cdot in$

Door length from drawing 155667grid B-4

$L_{spacing} := 9.75 \cdot in + 12 \cdot in$

Spacing around north door frame, sheet 3

$L_{tol} := 6 \cdot in$

Brace location tolerance (x2 braces), note 3 sheet 1

$$L_{trib} := \frac{L_{door} + L_{spacing} + L_{tol} + \left(23 + \frac{7}{8}\right)\cdot in}{2}$$

Bracing spacing is irregular; calculate the largest tributary length for a single brace per the draft drawings.

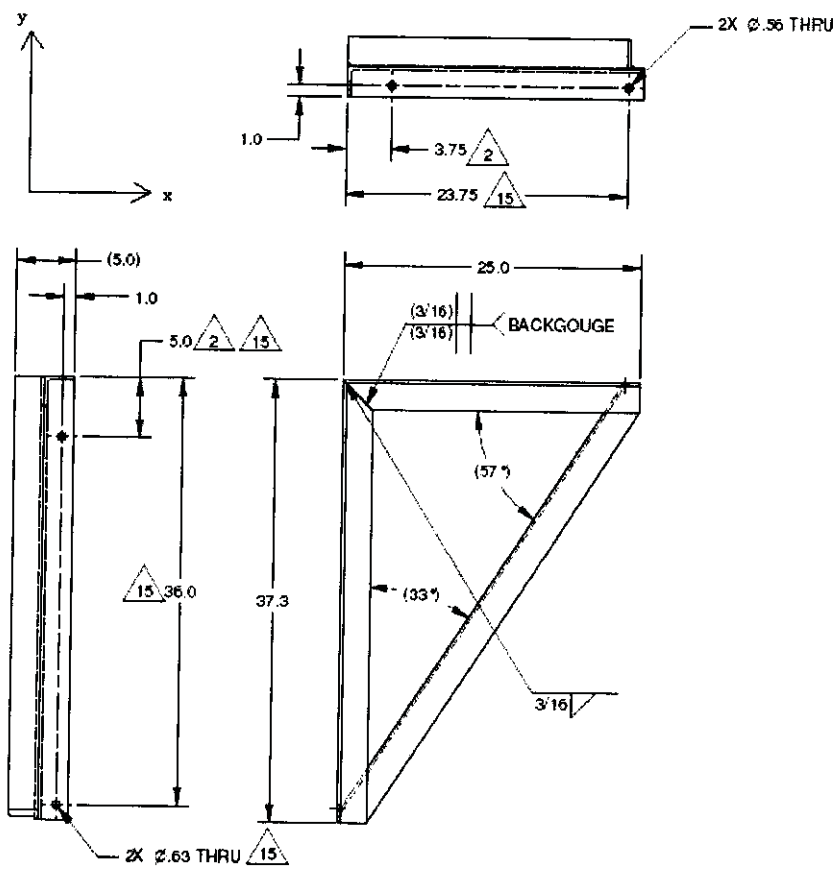
$$L_{trib} = 3.818\ ft$$

$$P_r := F_r \cdot L_{trib}$$

The required load on the maximally loaded brace is the linear demand times the tributary length.

$$P_r = 1.527 \times 10^3\ lbf$$

The force in the brace diagonal will be calculated based upon the distance from the toe of the brace to the outermost bolting hole in both the horizontal and vertical directions. y is up; x is perpendicular to the braced wall.



-2 WELDMENT 2

SCALE: 1/8

$l_h := 25 \cdot in$ — length of horizontal member

$l_v := 37.3 \cdot in$ — length of vertical member

$l_{anchors} := 23.75 \cdot in - 3.75 \cdot in - 4 \cdot in$ — minimum spacing between ceiling anchors, see sheet 5 -2 weldment and sheet 1 note 2

$l_{anchors} = 16\,in$

$\alpha := atan\left(\frac{l_h}{l_v}\right)$ — angle between vertical and diagonal

$\alpha = 33.832\,deg$

$P_{diag} := \frac{P_r}{\sin(\alpha)}$ — force in the diagonal member when maximum bracing force is applied at the bolt hole of the vertical member. This may be either compressive or tensile.

$P_{diag} = 2.743 \times 10^3\,lbf$

Calculate the allowable compressive stress in the diagonal. Neglect permissible increases in ASD based capacities.

$A_{diag} := 1.19 \cdot in^2$ — Area of the 2.5 x 2.5 x.0.25 angle, per AISC property table

$r := 0.491 \cdot in$ — Min radius of gyration per AISC property table

$f_a := \frac{P_{diag}}{A_{diag}}$ — compressive stress in the diagonal

$f_a = 2.305 \times 10^3\,psi$

$l := \sqrt{l_h^2 + l_v^2}$ — length of the diagonal

$l = 44.903\,in$

$K := 1.0$ — conservatively take K=1.0 (pinned-pinned)

$F_y := 36000 \cdot psi$ — assumed yield strength of steel

$E := 29000000 \cdot psi$ assumed elastic modulus of steel

$$C_c := \sqrt{\frac{2 \cdot \pi^2 \cdot E}{F_y}}$$ ASD Eqn E2-1

$C_c = 126.099$

$$Fa := \frac{\left[1 - \frac{\left(\frac{K \cdot l}{r}\right)^2}{2 \cdot C_c^2}\right] \cdot F_y}{\frac{5}{3} + \frac{3 \cdot \left(\frac{K \cdot l}{r}\right)}{8 \cdot C_c} - \frac{\left(\frac{K \cdot l}{r}\right)^3}{8 \cdot C_c^3}}$$ ASD Eqn E2-1 (gives lower capacity than Eqn E2-2)

$Fa = 1.403 \times 10^4 psi$ Allowable compressive stress

$\frac{f_a}{Fa} = 0.164$ Demand to capacity ratio for compression.

There is some small eccentricty of the angle due to the offset of the centroid from the plane of loading (the outside of the angle leg) which will result in a small and negligible bending moment in the diagonal. The compression demand is low. No need to evaluate interaction with bending due to eccentricity or flexural/torsional buckling. The diagonal brace is acceptable as a compression member.

Tensile allowable loads will be higher than compressive; therefore the diagonal brace is also acceptable as a tensile member.

Calcluate the length of weld on the end of the diagonal. Consider only the depth of the vertical or horizontal member (neglecting the weld on the other side of the diagonal leg and the crossing angle). This calculation also constitutes a bounding evaluation of the weld at the square corner.

$l_{weld} := 2.5 \cdot in$ conservatively take length of the weld equal to the depth of the connection

$$t_{weld} := \frac{\frac{3}{16} \cdot in}{\sqrt{2}}$$ fillet weld is called out as 3/16 inch

$a_{weld} := l_{weld} \cdot t_{weld}$ area of weld

$a_{weld} = 0.331\ in^2$

Conservatively assume 60ksi ultimate strength filler metal

$Pweld_{allow} := a_{weld} \cdot 60000 \cdot psi \cdot 0.3$ Allowable shear load in the weld, ASD Table J2.5

$Pweld_{allow} = 5.966 \times 10^3\ lbf$

$\frac{P_{diag}}{Pweld_{allow}} = 0.46$ Demand to capacity ratio of weld in shear.

Bending in the diagaonal is negligible. Only one of two welds joining each end of the diagonal to the brace weldment is considered -- capacity is adequate based upon this conservative approximate evaluation.

Analyze the anchor bolt loads for the brace using an ASD approach. Conservatively assume all forces on the brace are transmitted through the bolts (no contact with the ceiling or wall other than at bolts). Neglect permissible increases to adjust allowable stress design to strength design.

$F_t := 5457 \cdot lbf$

$F_s := 2889 \cdot lbf$

Hilti technical data allowable tensile and shear capacity for HILTI HDA-P 20 M12x125/30 in 3000 psi concrete. Slab is 36 inches thick, so base material thickness is acceptable.

$M := P_r \cdot l_v$ Moment applied to the bolt pattern in the ceiling

$f_t := \frac{M}{l_{anchors}}$ The tensile bolt could be either the nearest or farthest from the corner, depending upon direction of loading.

$f_t = 3.56 \times 10^3\ lbf$

$\frac{f_t}{F_t} = 0.652$ Tensile D/C ratio

$f_s := \frac{P_r}{2}$ Shear demand, per bolt

$f_s = 763.542\ lbf$

$\frac{f_s}{F_s} = 0.264$ Shear D/C ratio

$$\left(\frac{f_t}{F_t}\right)^1 + \left(\frac{f_s}{F_s}\right)^1 = 0.917$$

interaction < 1.2 therefore acceptable

Evaluate the bending moment in the top piece of the brace based upon hole locations and allowable stress.

$S := 0.394 \cdot in^3$

Section modulus of 2.5 x 2.5 x 0.25 angle, per AISC property table for x-x section (assume angle is held in plane of ceiling or wall)

$F_b := F_y \cdot 0.6 \cdot 1.7$

Bending allowable stress. Credit adjustment of 1.7 for strength design from DOE-STD-1020-02 Section 2.3.2

$F_b = 3.672 \times 10^4 \, psi$

$F_v := P_{diag} \cdot \cos(\alpha)$

Calculate axial force in the vertical member

$F_v = 2.278 \times 10^3 \, lbf$

$\delta_{bolthole} := 5.75 \cdot in$

maximum distance from brace vertex to bolt hole, sheet 5 -2 weldment, with +2 inches from note 2

$M_{max} := F_v \cdot \delta_{bolthole}$

maximum moment occurs where the bolt is farthest from a corner, which is at the square corner of the brace

$\sigma_b := \frac{M_{max}}{S}$

$\sigma_b = 3.325 \times 10^4 \, psi$

maximum bending stress in the brace, based upon smallest bolt spacing within tolerance

$\frac{\sigma_b}{F_b} = 0.906$

bending D/C ratio < 1 acceptable

The design tolerance of plus or minus 2.0 inches on bolt hole location is acceptable.

OTHER CONSIDERATIONS

When the wall is pulling away from the brace, the bracing force will be transmitted to the wall by the 1/2 inch through bolted threaded rod. This rod is acceptable by inspection for the approximately 1.5 kip load. There is a 4 x 12 inch by 0.25 inch thick back plate on the wall at each anchor point. This plate distributes the load to the reinforced block, and is acceptable by inpsection.

The relevant features of the brace have been considered. The brace is acceptable for the specified seismic load of 400 pounds per linear foot of wall.

Appendix B: Hilti Product Data

Attached are page(s) from the 2008 Hilti North American Product Technical Guide. For complete details on this product, including data development, product specifications, general suitability, installation, corrosion, and spacing & edge distance guidelines, please refer to the Technical Guide, or contact Hilti.

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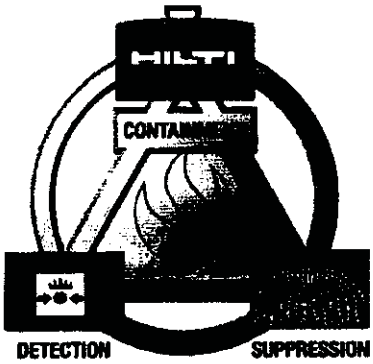

Firestop Systems

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- Firestop Systems Guides
  - Through Penetrations
  - Joint Penetrations
- FACT Program
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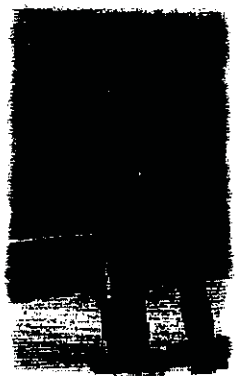
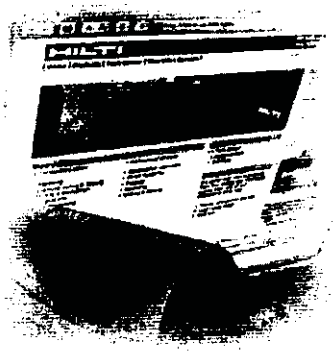


Hilti Diaphragm Deck Design

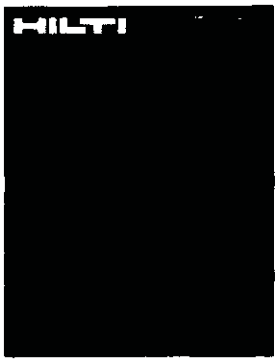
The Hilti Diaphragm Deck Design Program allows designers to quickly and accurately design roof deck and composite floor deck diaphragms.

- Ability to design with innovative Hilti fasteners for frame and sidelap connection
- Creates easy to use load tables with span ranges based on user input
- Allows for different safety factors depending on load type, building code and field quality control
- Direct link to Hilti website



Hilti Online

- Technical Library
- Design Centers
- Interactive Product Advisors
- Full-line Product Catalog
- Online Ordering
- Maps to Hilti locations
- "Contact Us" program to answer your questions



MI – Industrial Pipe Support Technical Guide

A guide to specifying the Hilti modular pipe support system for medium to heavy loads without welding.

- MI System is the ideal solution for pipes up to 24 in. diameter
- Reliable fastenings without welds
- Easily installed

HDA Undercut Anchor 4.3.1

4.3.1.1	Product Description
4.3.1.2	Material Specifications
4.3.1.3	Technical Data
4.3.1.4	Installation Instructions
4.3.1.5	Ordering Information
4.3.1.6	HDA Removal Tool
4.3.1.7	Sample Calculations

4.3.1.1 Product Description

The Hilti HDA Undercut Anchor is a heavy duty mechanical undercut anchor whose undercut segments incorporate carbide tips so as to perform a self-undercutting process designed to develop a ductile steel failure. The HDA system includes either preset (HDA-P) or through-set (HDA-T) style anchors, stop drill bits, setting tool, and roto-hammer drill for four metric bolt sizes: M10 (3/8"), M12 (1/2"), M16 (5/8") and M20 (3/4"). The HDA is available in a sherardized and 316 stainless steel versions for outdoor environments. Each size/style is offered in two lengths to accommodate various material thicknesses to be fastened (except M10).

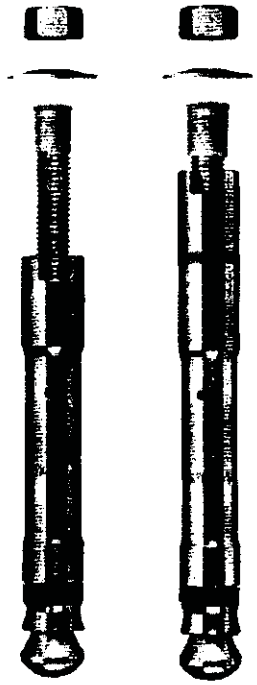
Product Features

- Undercut segments provide cast-in-place like performance with limited expansion stresses
- Bolt meets ductility requirements of ACI 318-05 Section D1
- Self-undercutting wedges provide an easy, fast and reliable anchor installation
- Excellent performance in cracked concrete (tension zones, earthquake-resisting structures)
- Undercut keying load transfer allows for reduced edge distances and anchor spacings
- Through-set style provides increased shear capacity
- Fully removable
- Sherardized (53 µm zinc) and 316 stainless steel for corrosive environments
- Sherardized zinc coating has equivalent corrosion resistance to hot dipped galvanizing

Guide Specifications

Undercut Anchors Undercut anchors shall be of an undercut style with brazed tungsten carbides on the embedded end that perform the self-undercutting process. Undercut portion of anchor shall have a minimum projected bearing area equal to or greater than 2.5 times the nominal bolt area. The bolt shall conform to ISO 898 class 8.8 strength requirements. Anchors dimensioned and supplied by Hilti.

Installation Refer to 4.3.1.3 and 4.3.1.4.



HDA-P Undercut Anchor (Pre-Set Type) | HDA-T Undercut Anchor (Through-Set Type)

Listings/Approvals

ICC-ES (International Code Council)
ESR-1546

City of Los Angeles
Research Report No. 25422

European Technical Approval (ETA)
ETA-99/0009
ETA-99/0016

Qualified under NQA-1 Nuclear Quality Program

Building Code Compliance

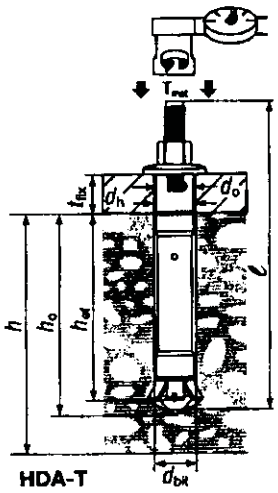
IBC 2006
UBC 1997

4.3.1.2 Material Specifications

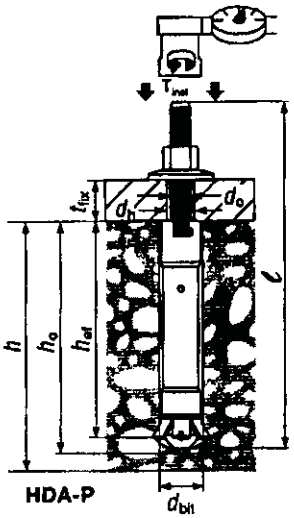
	Mechanical Properties f_y ksi (MPa)	min. f_u ksi (MPa)
HDA-T/-TF/-P/-PF carbon steel cone bolt; M10, M12, M16 and M20 meets strength requirements of ISO 898, class 8.8	92.8 (640)	116 (800)
HDA-T/-TF/-P/-PF carbon steel sleeve; M10 & M12 conforms to European Standard No. 25CrMoS4	–	123.3 (850)
HDA-T/-TF/-P/-PF carbon steel sleeve; M16 conforms to European Standard No. 25CrMoS4	–	101.5 (700)
HDA-T/-TF/-P/-PF carbon steel sleeve; M20 conforms to European Standard No. 25CrMoS4	–	79.8 (550)
HDA-TR/-PR stainless steel cone bolt; M10, M12 and M16 conforms to AISI 316 or 316 Ti	87 (600)	116 (800)
HDA-TR/-PR stainless steel sleeve; M10 and M12 conforms to AISI 316 or 316 Ti	–	123.3 (850)
HDA-TR/-PR stainless steel sleeve; M16 conforms to AISI 316 or 316 Ti	–	101.5 (700)
HDA-T/-TF/-P/-PF nut conforms to DIN 934, grade 8		
HDA-TR/-PR nut conforms to DIN 934, grade A4-80		
HDA-T/-TF/-TR/-P/-PF/-PR washer conforms to DIN 6796		
HDA-T/-P components are electroplated min. 5 µm zinc		
HDA-TF/-PF sherardized components have average 53 µm zinc		

4.3.1 HDA Undercut Anchor

4.3.1.3 Technical Data



HDA-T



HDA-P

Anchor Nomenclature

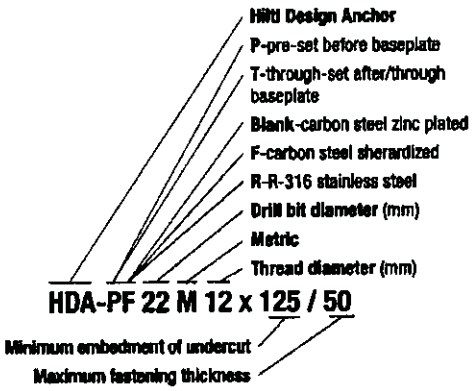


Table 1 - HDA Specifications

Anchor Size		HDA-T/HDA-P	M10 x 100/20	M12 x 125/30	M12 x 125/50	M16 x 190/40	M16 x 190/60	M20 x 250/50	M20 x 250/100
h	Thickness of base material, min.[1]	mm (in.)	170 (6-3/4)	190 (7-1/2)	190 (7-1/2)	270 (10-5/8)	270 (10-5/8)	350 (13-3/4)	350 (13-3/4)
ℓ	Total anchor length	mm (in.)	150 (5.90)	190 (7.48)	210 (8.27)	275 (10.83)	295 (11.61)	360 (14.17)	410 (16.14)
	length I.D. code[2]	letter	I	L	N	R	S	V	W
t_{fix}	Fastening thickness								
	HDA-T, min.[3]	mm (in.)	10 (0.39)	10 (0.39)	10 (0.39)	15 (0.59)	15 (0.59)	20 (0.79)	20 (0.79)
	HDA-T, max.	mm (in.)	20 (0.79)	30 (1.18)	50 (1.97)	40 (1.58)	60 (2.36)	50 (1.97)	100 (3.94)
	HDA-P, max.	mm (in.)	20 (0.79)	30 (1.18)	50 (1.97)	40 (1.58)	60 (2.36)	50 (1.97)	100 (3.94)
d_{bit}	Nom. dia. of drill bit[4]	mm	20	22	22	30	30	37	37
h_o	Min. depth of drill hole	mm (in.)	107 (4.21)	134.5 (5.30)	134.5 (5.30)	203 (7.99)	203 (7.99)	266 (10.47)	266 (10.47)
h_{ef}	Effective anchoring depth	mm (in.)	100 (3.94)	125 (4.92)	125 (4.92)	190 (7.48)	190 (7.48)	250 (9.84)	250 (9.847)
d_h	Recommended clearance hole (min.) HDA-T	mm (in.)	21 (7/8)	23 (15/16)	23 (15/16)	32 (1-1/4)	32 (1-1/4)	40 (1-9/16)	40 (1-9/16)
	HDA-P	mm (in.)	12 (1/2)	14 (9/16)	14 (9/16)	18 (3/4)	18 (3/4)	22 (7/8)	22 (7/8)
d_o	Anchor Diameter, HDA-T	mm (in.)	19 (0.748)	21 (0.827)	21 (0.827)	29 (1.142)	29 (1.142)	36 (1.42)	36 (1.42)
	HDA-P	mm (in.)	10 (0.394)	12 (0.472)	12 (0.472)	16 (0.630)	16 (0.630)	20 (0.78)	20 (0.78)
d_w	Washer diameter	mm (in.)	27.5 (1.08)	33.5 (1.32)	33.5 (1.32)	45.5 (1.79)	45.5 (1.79)	50 (1.97)	50 (1.97)
S_w	Width across flats	mm (in.)	17	19	19	24	24	30	30
T_{max}	Max. tightening torque[5]	Nm (ft-lb)	50 (37)	80 (59)	80 (59)	120 (88)	120 (88)	300 (221)	300 (221)
Sleeve properties									
A_{st}	Cross sectional area	mm^2 (in^2)	196 (0.304)	223 (0.346)	223 (0.346)	445 (0.690)	445 (0.690)	675.6 (1.047)	675.6 (1.047)
S_{st}	Elastic section modulus	mm^3 (in^3)	596 (0.0364)	779 (0.0475)	779 (0.0475)	2110 (0.1288)	2110 (0.1288)	3950 (0.241)	3950 (0.241)
Bolt properties									
A_b	Bolt nominal area	mm^2 (in^2)	78.5 (0.122)	113 (0.175)	113 (0.175)	201 (0.312)	201 (0.312)	314.16 (0.487)	314.16 (0.487)
A_t	Bolt tension area	mm^2 (in^2)	58 (0.090)	84.3 (0.131)	84.3 (0.131)	157 (0.243)	157 (0.243)	245 (0.380)	245 (0.380)
S_b	Elastic section modulus	mm^3 (in^3)	67 (0.0041)	117 (0.0071)	117 (0.0071)	293 (0.0179)	293 (0.0179)	541.3 (0.033)	541.3 (0.033)
A_{brg}	Undercut bearing area[6]	mm^2 (in^2)	260 (0.403)	354 (0.549)	354 (0.549)	624 (0.967)	624 (0.967)	707.1 (1.096)	707.1 (1.096)

1 Base material thickness as required to minimize splitting as per the CCD design method.

2 Length code in accordance with ICC-ES Acceptance Criteria AC193.

3 Minimum thickness of fastened part as required to ensure engagement of full sleeve cross section in shear.

4 Metric stop drill bit must be used. See Section 4.3.1.4 for use of matched tolerance diamond core bits.

5 Torque tightening of the anchor is not required for proper set. Torque tightening may reduce initial slip under load and can improve fatigue performance.

6 Recommended area for calculating maximum bearing stress in accordance with the CCD method.

4.3.1.3.1 Design Information - Undercut Anchors

Undercut anchors represent the state of the art in post-installed anchor technology. When properly designed and proportioned, they transfer tension loads to the concrete in much the same way as cast-in-place headed bolts, that is, via bearing. Since friction is less critical in developing tension capacity, lower expansion forces are transmitted to the concrete. This reduces the overall stress state in the concrete prior to and during loading. The Hilti HDA Undercut Anchor System is the result of extensive research to determine the optimum geometry for load transfer at the bearing surface. Besides allowing for easy installation, the self-undercutting system automatically results in an excellent fit between the anchor bearing surface and the undercut, critical for limiting initial displacements. Maximum bearing stresses corresponding to steel failure are limited to approximately 12 f_c, sufficiently low to preclude crushing in the bearing region. This limits displacements at both service and ultimate loads.

The HDA is equipped with a shear sleeve machined from high grade carbon steel. When used in the pre-set configuration (HDA-P), shear loads are transferred through the threaded bolt to the sleeve and subsequently to the concrete in bearing. In through-set applications (HDA-T), the sleeve engages the part to be fastened, thus substantially increasing the ultimate shear capacity of the anchorage. At ultimate, the sleeve and bolt act in concert to develop the full shear capacity of the anchor.

The HDA Undercut Anchor is proportioned to consistently develop the bolt strength in tension at critical edge distances and spacings. At spacings and edge distances less than critical, concrete cone failure will generally limit the ultimate load. The reduction of expansion forces allows for designed installations at minimum edge distances and spacings significantly less than those typically used for other types of mechanical expansion anchors. It is virtually impossible for the HDA Undercut Anchor to fail by pullout or pull-through. The predictability of the failure modes associated with the HDA Undercut Anchor allow for increased repeatability in determining ultimate capacities for a particular design condition.

The HDA Undercut Anchor was extensively tested prior to market introduction. Testing included static tension, shear, and oblique loading of both single anchors and groups, shock, seismic groups, seismic and shock loading. Exhaustive testing of the HDA performance in cracks confirms it's suitability for installation in tension zones. Design data is provided in three formats.

4.3.1.3.2 Design Method

4.3.2.3.2.1 Strength Design (LRFD)

ACI 318 Appendix D replaces the strength design provisions of the IBC 2006 and provides a comprehensive and rational framework for calculating anchor capacity. The applicability of the method to the HDA Undercut Anchor is based on the similarity of performance and failure modes established for the HDA with those associated for cast-in-place headed bolts. This method can also be used for design in Canada according to CSA A23.3-94 providing the appropriate *f* factors for steel and concrete. See Section 4.1.6.

4.3.2.3.2.2 Allowable Stress Design (ASD)

Compatible with existing Hilti design methods. Test data to develop the average ultimate load capacity, and evaluating the data using the 5% fractile method to determine the allowable working load. See ESR-1546 Section 4.2.

4.3.1 HDA Undercut Anchor

Table 2 — HDA Strength Design Information

Design parameter	Symbol	Units	Nominal anchor diameter						
			M10		M12		M16		M20
			HDA HDA-F	HDA-R	HDA HDA-F	HDA-R	HDA HDA-F	HDA-R	HDA
Anchor O.D.	d_o	mm (in.)	19 (0.75)		21 (0.83)		29 (1.14)		35 (1.38)
Effective min. embedment depth[1]	$h_{ef,min}$	mm (in.)	100 (3.94)		125 (4.92)		190 (7.48)		250 (9.84)
Minimum edge distance	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}	mm (in.)	170 (6-3/4)		190 (7-1/2)		270 (10-5/8)		350 (13-3/4)
Anchor category[2]	1,2 or 3	-	1						
Strength reduction factor for tension, steel failure modes[3]	ϕ	-	0.75						
Strength reduction factor for shear steel failure modes[3]	ϕ	-	0.65						
Strength reduction factor for tension, concrete failure modes[3]	ϕ	Cond. A Cond. B	0.75 0.65						
Strength reduction factor for shear, concrete failure modes[3]	ϕ	Cond. A Cond. B	0.75 0.70						
Yield strength of anchor steel	f_y	lb/in²	92,800	87,000	92,800	87,000	92,800	87,000	92,800
Ultimate strength of anchor steel	f_{ut}	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	10,440		15,196		28,188		44,080
Effectiveness factor uncracked concrete	k_{uncr}	-	30						30
Effectiveness factor cracked concrete[4]	k_{cr}	-	24						24
k_{uncr}/k_{cr}[5]	$\Psi_{c,N}$	-	1.25						1.25
Pullout strength cracked concrete[6]	$N_{pn,cr}$	lb	8,992		11,240		22,481		33,721
Steel strength in shear static[7] HDA-P/PF/PR	V_{sa}	lb	5,013	6,070	7,284	8,992	13,556	16,861	20,772
Steel strength in shear, seismic[7,8] HDA-P/PF/PR	$V_{sa,seis}$	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[10]	β	1000 lb/in.	80 / 100						

1 Actual h_{ef} for HDA-T is given by $h_{ef,min} + (t_{in} - t_{actual})$ where t_{in} is given in Table 1 and t_{actual} is the thickness of the part(s) being fastened.

2 See ACI 318-05 D.4.4.

3 For use with the load combinations of ACI 318-05 9.2. Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.

4 See ACI 318-05 D.5.2.2.

5 See ACI 318-05 D.5.2.6.

6 See ESR-1546, Section 4.1.3.

7 For HDA-T see Table 3.

8 See ESR-1546, Section 4.1.6.

9 Splitting failure under external load does not govern the resistance of the HDA. Therefore, no values for the critical edge distance c_{cr} are provided since this calculation is not required for design.

10 Minimum axial stiffness values, maximum values may be 3 times larger (e.g. due to high strength concrete)

Table 3 - Steel Strength in Shear, HDA-T (lb)

Anchor Designation		Thickness of fastened part(s)		Steel Strength in Shear, Static	Steel Strength in Shear, Seismic[1]
		mm	in.	V_{sa}	$V_{sa,seis}$
HDA-T HDA-TF	HDA-T 20-M10x100	10 ≤ t < 15	3/8 ≤ t < 5/8	13,938	12,589
		15 ≤ t < 20	5/8 ≤ t < 13/16	15,737	14,163
	HDA-T 22-M12x125	10 ≤ t < 15	3/8 ≤ t < 5/8	16,636	15,062
		15 ≤ t ≤ 50	5/8 ≤ t < 2	18,659	16,636
	HDA-T 30-M16x190	15 ≤ t < 20	5/8 ≤ t < 13/16	30,574	27,427
		20 ≤ t < 25	13/16 ≤ t < 1	34,621	31,248
		25 ≤ t < 30	1 ≤ t < 1-3/16	38,218	34,396
		30 ≤ t ≤ 60	1-3/16 ≤ t < 2-3/8	41,365	37,093
	HDA-T 37-M20x250	20 ≤ t < 35	13/16 ≤ t < 1-3/8	45,187	40,690
		35 ≤ t < 50	1 ≤ t < 2	50,807	45,636
		50 ≤ t ≤ 100	2 ≤ t < 4	54,629	49,233
HDA-TR	HDA-TR 20-M10x100	10 ≤ t < 15	3/8 ≤ t < 5/8	15,512	13,938
		15 ≤ t < 20	5/8 ≤ t < 13/16	16,186	14,613
	HDA-TR 22-M12x125	10 ≤ t < 15	3/8 ≤ t < 5/8	20,233	17,985
		15 ≤ t ≤ 50	5/8 ≤ t < 2	22,256	20,008
	HDA-TR 30-M16x190	15 ≤ t < 20	5/8 ≤ t < 13/16	35,745	32,148
		20 ≤ t < 25	13/16 ≤ t < 1	37,768	33,946
		25 ≤ t < 30	1 ≤ t < 1-3/16	39,566	35,520
		30 ≤ t ≤ 60	1-3/16 ≤ t < 2-3/8	40,915	36,869

1 The nominal steel strength $V_{sa,seis}$ for the HDA-P shall be taken from Table 2.

Table 4 - HDA-P/T, HDA-PF/TF & HDA-PR/TR Allowable Static Tension (ASD), Normal Weight Uncracked Concrete (lb)[1,3,4]

Nominal Anchor Diameter	Embedment Depth h_{ef}		Concrete Compressive Strength[2]							
			f'_c= 2500 psi		f'_c= 3000 psi		f'_c= 4000 psi		f'_c= 6000 psi	
	mm	in.	Condition A	Condition B	Condition A	Condition B	Condition A	Condition B	Condition A	Condition B
M10	100	3.94	5,593	4,872	5,593	5,593	5,593	5,593	5,593	5,593
M12	125	4.92	7,844	6,798	8,141	8,141	8,141	8,141	8,141	8,141
M16	190	7.48	14,704	12,743	15,101	15,101	15,101	15,101	15,101	15,101
M20	250	9.84	22,185	19,227	23,614	23,548	23,614	23,614	23,614	23,614

1 Values are for single anchors with no edge distance or spacing reduction. For other cases, see ICC ESR-1546, Section 4.2.

2 Values are for normal weight concrete. For sand-lightweight concrete, multiply values by 0.85. For all-lightweight concrete, multiply values by 0.75. See ACI 318-05 D.3.4.

3 Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs. See ACI 318-05 D.4.4.

4 Allowable static tension loads for 2,500 psi are calculated by multiplying the concrete breakout strength N_b by the strength reduction ϕ factor of 0.65 and dividing by an α of 1.4 according to ICC ESR-1546 Section 4.2. N_b is calculated as per ACI 318-05 D.5.2.2. This load may be adjusted for other concrete strengths according to ICC ESR-1546 Section 4.1.3 by using the following equation.

$$N_{b,f'c} = N_b \sqrt{\frac{f'c}{2500}}$$

Table 5 - HDA-P/T, HDA-PF/TF & HDA-PR/TR Allowable Static Tension (ASD), Normal Weight Cracked Concrete (lb)[1,3,4]

Nominal Anchor Diameter	Embedment Depth h_{ef}		Concrete Compressive Strength[2]			
	mm	in.	f'_c= 2000 psi	f'_c= 3000 psi	f'_c= 4000 psi	f'_c= 6000 psi
M10	100	3.94	3,734	4,573	5,281	5,593
M12	125	4.92	4,668	5,717	6,601	8,085
M16	190	7.48	9,336	11,434	13,203	15,101
M20	250	9.84	14,003	17,150	19,804	23,614

1 Values are for single anchors with no edge distance or spacing reduction. For other cases, see ICC ESR-1546, Section 4.2.

2 Values are for normal weight concrete. For sand-lightweight concrete, multiply values by 0.85. For all-lightweight concrete, multiply values by 0.75. See ACI 318-05 D.3.4.

3 All values applicable to either Condition A or Condition B (pullout or steel failure control). See ACI 318-05 D.4.4.

4 Allowable static tension loads for 2,500 are calculated by multiplying the pullout strength N_{pn} by the strength reduction ϕ factor of 0.65 and dividing by an α of 1.4 according to ICC ESR-1546 Section 4.2. See Table 2 for N_{pn}. This load may be adjusted for other concrete strengths according to ICC ESR-1546 Section 4.1.3 by using the following equation.

$$N_{pn,\alpha,f'c} = N_{pn,\alpha} \sqrt{\frac{f'c}{2500}}$$

4.3.1 HDA Undercut Anchor

Table 6 - HDA-P/T, HDA-PF/TF & HDA-PR/TR Allowable Seismic Tension (ASD), Normal Weight Concrete (lb)[1,3,4]

Nominal Anchor Diameter	Embedment Depth h_{ef}		Concrete Compressive Strength[2]			
	mm	in.	f'_c= 2000 psi	f'_c= 3000 psi	f'_c= 4000 psi	f'_c= 6000 psi
M10	100	3.94	3,564	4,365	5,041	5,339
M12	125	4.92	4,455	5,457	6,301	7,717
M16	190	7.48	8,911	10,914	12,603	14,414
M20	250	9.84	13,367	16,371	18,904	22,541

1 Values are for single anchors with no edge distance or spacing reduction. For other cases, see ICC ESR-1546, Section 4.2.

2 Values are for normal weight concrete. For sand-lightweight concrete, multiply values by 0.85. For all-lightweight concrete, multiply values by 0.75. See ACI 318-05 D.3.4.

3 All values applicable to either Condition A or Condition B (pullout or steel failure control). See ACI 318-05 D.4.4.

4 Allowable seismic tension loads for 2,500 psi are calculated by multiplying the pullout strength N_{pn} by the strength reduction ϕ factor of 0.65, then multiplying by a 0.75 factor describe in ACI 318-05 D.3.3.3, and dividing by an α of 1.1 according to ICC ESR-1546 Section 4.2. See Table 2 for N_{pn}. This load may be adjusted for other concrete strengths according to ICC ESR-1546 Section 4.1.3 by using the following equation.

$$N_{pn,cr\,f'c} = N_{pn,cr}\sqrt{\frac{f'c}{2500}}$$

Table 7 - HDA-P, HDA-PF & HDA-PR Allowable Static and Seismic Shear (ASD), Steel (lb)[1]

Design parameter	Units	Nominal anchor diameter						
		M10		M12		M16		M20
		HDA	HDA-R	HDA	HDA-R	HDA	HDA-R	HDA
Allowable static steel capacity[2] HDA-P/PR	lb	2,327	2,818	3,382	4,175	6,294	7,828	9,644
Allowable seismic steel capacity[3] HDA-P/PR	lb	1,993	2,491	2,889	3,587	5,380	6,675	8,269

1 Values are for single anchors with no edge distance or spacing reduction due to concrete failure.

2 Allowable static shear loads are calculated by multiplying V_{sa} by the strength reduction ϕ factor of 0.65 and dividing by an α of 1.4 according to ICC ESR-1546 Section 4.2. See Table 2 for V_{sa}.

3 Allowable seismic shear loads are calculated by multiplying V_{sa} by the strength reduction ϕ factor of 0.65, then multiply by 0.75 as per ACI 318-05 D.3.3.3, and dividing by an α of 1.1 according to ICC ESR-1546 Section 4.2. See Table 2 for V_{sa}.

Table 8 - HDA-T/TF/TR Allowable Static and Seismis Shear (ASD), Steel (lb)[1,2]

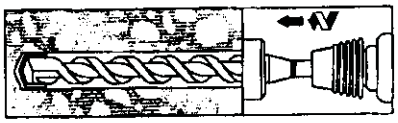
Anchor Designation		Thickness of fastened part(s)		Allowable Static Steel Capacity	Allowable Seismic Steel Capacity
		mm	in.	V_s	$V_{s,seismic}$
HDA-T HDA-TF	HDA-T 20-M10x100	10 ≤ t < 15	3/8 ≤ t < 5/8	6,471	5,579
		15 ≤ t < 20	5/8 ≤ t < 13/16	7,306	6,277
	HDA-T 22-M12x125	10 ≤ t < 15	3/8 ≤ t < 5/8	7,724	6,675
		15 ≤ t ≤ 50	5/8 ≤ t < 2	8,663	7,373
	HDA-T 30-M16x190	15 ≤ t < 20	5/8 ≤ t < 13/16	14,195	12,155
		20 ≤ t < 25	13/16 ≤ t < 1	16,074	13,849
		25 ≤ t < 30	1 ≤ t < 1-3/16	17,744	15,244
		30 ≤ t ≤ 60	1-3/16 ≤ t < 2-3/8	19,205	16,439
	HDA-T 37-M20x250	20 ≤ t < 35	13/16 ≤ t < 1-3/8	20,980	18,033
		35 ≤ t < 50	1 ≤ t < 2	23,589	20,225
		50 ≤ t ≤ 100	2 ≤ t < 4	25,363	21,819
HDA-TR	HDA-TR 20-M10x100	10 ≤ t < 15	3/8 ≤ t < 5/8	7,202	6,177
		15 ≤ t < 20	5/8 ≤ t < 13/16	7,515	6,476
	HDA-TR 22-M12x125	10 ≤ t < 15	3/8 ≤ t < 5/8	9,394	7,971
		15 ≤ t ≤ 50	5/8 ≤ t < 2	10,333	8,867
	HDA-TR 30-M16x190	15 ≤ t < 20	5/8 ≤ t < 13/16	16,596	14,247
		20 ≤ t < 25	13/16 ≤ t < 1	17,535	15,044
		25 ≤ t < 30	1 ≤ t < 1-3/16	18,370	15,742
		30 ≤ t ≤ 60	1-3/16 ≤ t < 2-3/8	18,996	16,340

1 Values are for single anchors with no edge distance or spacing reduction due to concrete failure.

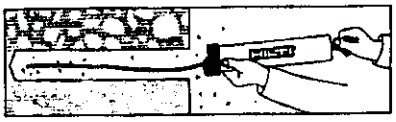
2 Allowable shear loads calculated using the steel strengths in shear from Table 3, multiplying by by the strength reduction ϕ factor of 0.65, and dividing by 1.4 according to ICC ESR-1546 Section 4.2.

4.3.1.4 Installation Instructions

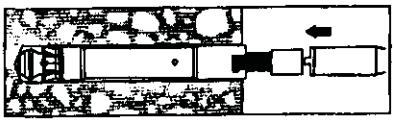
Setting Operation HDA-P/-PR/-PF (Pre-Set Style)



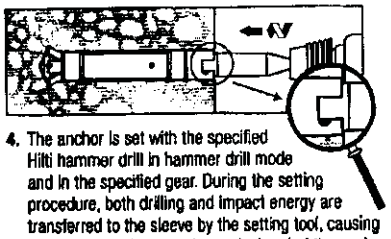
1. Drill a hole to the required depth using a stop drill bit matched to the anchor, (refer to specification table and ordering info.). If rebar is encountered, use a Hilti metric matched tolerance diamond core bit to drill through the rebar. Remove the concrete core and finish drilling the hole with the stop drill bit. Always consult with the Engineer of Record before cutting rebar.



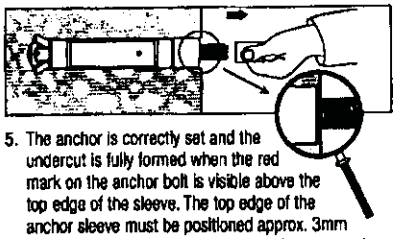
2. Clean hole with compressed air or a hand air pump such that drilling debris is evacuated.



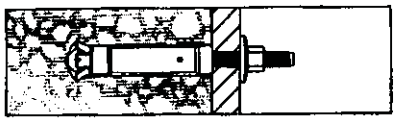
3. Insert the anchor into the hole by hand, so that the cone sits on the bottom of the drilled hole. Do not remove the plastic cap which protects the threaded rod. Using the assigned setting tool and Hilti hammer drill, the setting tool is guided over the anchor rod and engages the grooves in the sleeve. **It is critical to use the specified Hilti hammer drills.**



4. The anchor is set with the specified Hilti hammer drill in hammer drill mode and in the specified gear. During the setting procedure, both drilling and impact energy are transferred to the sleeve by the setting tool, causing the sleeve to slide over the conical end of the anchor bolt while forming the undercut in the base material. On the setting tool, the red ring indicates the progress of the setting operation. When this marking is flush with the concrete surface, check the anchor for proper setting (refer to step 5).

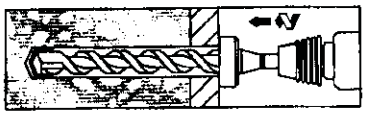


5. The anchor is correctly set and the undercut is fully formed when the red mark on the anchor bolt is visible above the top edge of the sleeve. The top edge of the anchor sleeve must be positioned approx. 3mm below the concrete surface. If the anchor setting time exceeds 60 seconds for M10, M12 or M16 anchors or 120 seconds for M20 anchors the installation failed and the anchors must not be loaded.

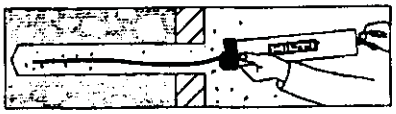


6. Remove the plastic thread protector cap. Secure the part to be fastened by using the conical spring washer and nut provided. Apply a torque not to exceed the maximum values given in the Specification Table. Torque is not required to set the anchor.

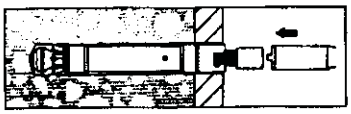
Setting Operation HDA-T/-TR/-TF (Through-Set Style)



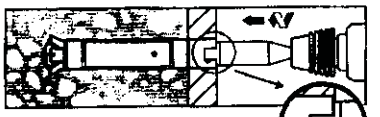
1. Drill a hole to the required depth using a stop drill bit matched to the anchor, (refer to specification table and ordering info.). If rebar is encountered, use a Hilti metric matched tolerance diamond core bit to drill through the rebar. Remove the concrete core and finish drilling the hole with the stop drill bit. Always consult with the Engineer of Record before cutting rebar.



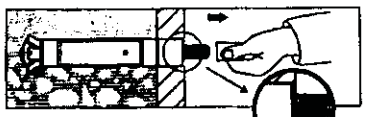
2. Clean hole with compressed air or a hand air pump such that drilling debris is evacuated.



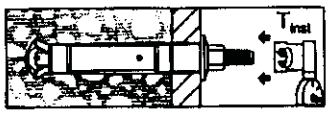
3. Insert the anchor into the hole by hand, so that the cone sits on the bottom of the drilled hole. Do not remove the plastic cap which protects the threaded rod. Using the assigned setting tool and Hilti hammer drill, the setting tool is guided over the anchor rod and engages the grooves in the sleeve. **It is critical to use the specified Hilti hammer drills.**



4. The anchor is set with the specified Hilti hammer drill in hammer drill mode and in the specified gear. During the setting procedure, both drilling and impact energy are transferred to the sleeve by the setting tool, causing the sleeve to slide over the conical end of the anchor bolt while forming the undercut in the base material. On the setting tool, the red ring indicates the progress of the setting operation. When this marking is flush with the connected part, check the anchor for proper setting (refer to step 5).



5. The anchor is set and the undercut is fully formed when the red marking on the anchor bolt is visible above the top edge of the sleeve. The top edge of the anchor sleeve must be positioned approx. 3mm below the surface of the fixture. If anchor setting time exceeds 60 seconds for M10, M12 or M16 anchors or 120 seconds for M20 anchors the installation failed and the anchor must not be loaded.



6. Remove the plastic thread protector cap. Secure the part to be fastened by using the conical spring washer and nut provided. Apply a torque not to exceed the maximum values given in the Specification Table. Torque is not required to set the anchor.

The HDA Undercut Anchor, designed to carry significant, safety-relevant loads, **must** be installed correctly with the prescribed tools to function properly. Carefully follow **all** instructions located inside the box. Installer training is also available upon request.

4.3.1 HDA Undercut Anchor

Table 9 - Equipment required for setting HDA Anchors

HDA carbon steel - galvanized

Anchor	Hilti Hammer Drill					Stop drill bit	Setting tool	Single impact energy for setting	RPM under load (1/min)
	TE 25 (1st gear)	TE 35	TE 56/ TE 56-ATC	TE 75	TE 76/ 76-ATC				
	max impact energy								
HDA-P 20-M10x100/20	•					TE-C-HDA-B 20x100	TE-C-HDA-ST 20 M10	3.7 - 4.7	250 - 500
			•			TE-Y-HDA-B 20x100	TE-Y-HDA-ST 20 M10	6.5 - 7.5	480 - 500
HDA-T 20-M10x100/20	•					TE-C-HDA-B 20x120	TE-C-HDA-ST 20 M10	3.7 - 4.7	250 - 500
			•			TE-Y-HDA-B 20x120	TE-Y-HDA-ST 20 M10	6.5 - 7.5	480 - 500
HDA-P 22-M12x125/30	•					TE-C-HDA-B 22x125	TE-C-HDA-ST 22 M12	3.7 - 4.7	250 - 500
			•			TE-Y-HDA-B 22x125	TE-Y-HDA-ST 22 M12	6.5 - 7.5	480 - 500
HDA-T 22-M12x125/30	•					TE-C-HDA-B 22x155	TE-C-HDA-ST 22 M12	3.7 - 4.7	250 - 500
			•			TE-Y-HDA-B 22x155	TE-Y-HDA-ST 22 M12	6.5 - 7.5	480 - 500
HDA-P 22-M12x125/50	•					TE-C-HDA-B 22x125	TE-C-HDA-ST 22 M12	3.7 - 4.7	250 - 500
			•			TE-Y-HDA-B 22x125	TE-Y-HDA-ST 22 M12	6.5 - 7.5	480 - 500
HDA-T 22-M12x125/50	•					TE-C-HDA-B 22x175	TE-C-HDA-ST 22 M12	3.7 - 4.7	250 - 500
			•			TE-Y-HDA-B 22x175	TE-Y-HDA-ST 22 M12	6.5 - 7.5	480 - 500
HDA-P 30-M16x190/40				•	•	TE-Y-HDA-B 30x190	TE-Y-HDA-ST 30 M16	7.0 - 9.0	150 - 350
HDA-T 30-M16x190/40				•	•	TE-Y-HDA-B 30x230			
HDA-P 30-M16x190/60				•	•	TE-Y-HDA-B 30x190			
HDA-T 30-M16x190/60				•	•	TE-Y-HDA-B 30x250			
HDA-P 37-M20x250/50					•	TE-Y-HDA-B 37x250	TE-Y-HDA-ST 37 M20	8.0 - 9.0	280
HDA-T 37-M20x250/50					•	TE-Y-HDA-B 37x300			
HDA-P 37-M20x250/100					•	TE-Y-HDA-B 37x250			
HDA-T 37-M20x250/100					•	TE-Y-HDA-B 37x300			

HDA-R carbon steel - galvanized

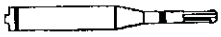
Anchor	Hilti Hammer Drill					Stop drill bit	Setting tool	Single impact energy for setting	RPM under load (1/min)
	TE 25 (1st gear)	TE 35	TE 56/ TE 56-ATC	TE 75	TE 76/ 76-ATC				
	max impact energy								
HDA-PR 20-M10x100/20	•	•				TE-C-HDA-B 20x100	TE-C-HDA-ST 20 M10	3.7 - 4.7	250 - 620
			•			TE-Y-HDA-B 20x100	TE-Y-HDA-ST 20 M10	6.5 - 7.5	480 - 500
HDA-TR 20-M10x100/20	•	•				TE-C-HDA-B 20x120	TE-C-HDA-ST 20 M10	3.7 - 4.7	250 - 620
			•			TE-Y-HDA-B 20x120	TE-Y-HDA-ST 20 M10	6.5 - 7.5	480 - 500
HDA-PR 22-M12x125/30	•	•				TE-C-HDA-B 22x125	TE-C-HDA-ST 22 M12	3.7 - 4.7	250 - 620
			•			TE-Y-HDA-B 22x125	TE-Y-HDA-ST 22 M12	6.5 - 7.5	480 - 500
HDA-TR 22-M12x125/30	•	•				TE-C-HDA-B 22x155	TE-C-HDA-ST 22 M12	3.7 - 4.7	250 - 620
			•			TE-Y-HDA-B 22x155	TE-Y-HDA-ST 22 M12	6.5 - 7.5	480 - 500
HDA-PR 22-M12x125/50	•	•				TE-C-HDA-B 22x125	TE-C-HDA-ST 22 M12	3.7 - 4.7	250 - 620
			•			TE-Y-HDA-B 22x125	TE-Y-HDA-ST 22 M12	6.5 - 7.5	480 - 500
HDA-TR 22-M12x125/50	•	•				TE-C-HDA-B 22x175	TE-C-HDA-ST 22 M12	3.7 - 4.7	250 - 620
			•			TE-Y-HDA-B 22x175	TE-Y-HDA-ST 22 M12	6.5 - 7.5	480 - 500
HDA-PR 30-M16x190/40				•	•	TE-Y-HDA-B 30x190	TE-Y-HDA-ST 30 M16	7.0 - 9.0	150 - 350
HDA-TR 30-M16x190/40				•	•	TE-Y-HDA-B 30x230			
HDA-PR 30-M16x190/60				•	•	TE-Y-HDA-B 30x190			
HDA-TR 30-M16x190/60				•	•	TE-Y-HDA-B 30x250			

HDA carbon steel - sheradized

Anchor	Hilti Hammer Drill					Stop drill bit	Setting tool	Single impact energy for setting	RPM under load (1/min)
	TE 25 (1st gear)	TE 35	TE 56/ TE 56-ATC	TE 75	TE 76/ 76-ATC				
	max impact energy								
HDA-PF 20-M10*x100/20		•				TE-C-HDA-B 20*x100	TE-C-HDA-ST 20 M10	3.5 - 4.0	610 - 630
HDA-TF 20-M10*x100/20		•				TE-C-HDA-B 20*x120			
HDA-PF 22-M12*x125/30		•				TE-C-HDA-B 22*x125	TE-C-HDA-ST 22 M12	3.5 - 4.0	610 - 630
HDA-TF 22-M12*x125/30		•				TE-C-HDA-B 22*x155			
HDA-PF 22-M12*x125/50		•				TE-C-HDA-B 22*x125			
HDA-TF 22-M12*x125/50		•				TE-C-HDA-B 22*x175			
HDA-PF 30-M16*x190/40				•	•	TE-Y-HDA-B 30*x190	TE-Y-HDA-ST 30 M16	7.0 - 9.0	150 - 350
HDA-TF 30-M16*x190/40				•	•	TE-Y-HDA-B 30*x230			
HDA-PF 30-M16*x190/60				•	•	TE-Y-HDA-B 30*x190			
HDA-TF 30-M16*x190/60				•	•	TE-Y-HDA-B 30*x250			

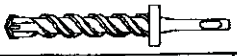
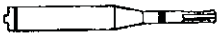
4.3.1.5 Ordering Information

HDA-T Anchor



HDA-T	HDA-T F	HDA-T R	HDA	HDA-T Anchor	Stop Drill Bit		Diamond Core Bit	Setting Tool	
Item No. Galvanized	Item No. Sherardised	Item No. 316 Stainless	Box Qty	Anchor (mm) dia x min.emb/max. thickness fastened	Item No.	Description (mm) dia. x drill depth	Diameter	Item No.	Description
331545	339361	339351	12	HDA-T M10x100/20	332090	TE-C-B20x120	20mm (25/32")	331843	TE-C-ST-M10
					237450	TE-Y-B20x120		287133	TE-Y-ST-M10
331548	339362	339352	8	HDA-T M12x125/30	332092	TE-C-B22x155	22mm (7/8")	331844	TE-C-ST-M13
					237452	TE-Y-B22x155		287134	TE-Y-ST-M12
331549	339363	339353	8	HDA-T M12x125/50	332093	TE-C-B22x175	22mm (7/8")	331844	TE-C-ST-M12
331552	339364	339354	4	HDA-T M16x190/40	332098	TE-Y-B30x230	30mm (1-1/8")	331846	TE-Y-ST-M16
331553	339365	339355	4	HDA-T M16x190/60	332099	TE-Y-B30x250	30mm (1-1/8")	331846	TE-Y-ST-M16
339267	–	–	2	HDA-T M20x250/50	339271	TE-Y-B37x300	37mm (1-3/8")	339269	TE-Y-ST-M20
339268	–	–	2	HDA-T M20x250/100	339272	TE-Y-B37x350	37mm (1-3/8")	339269	TE-Y-ST-M20

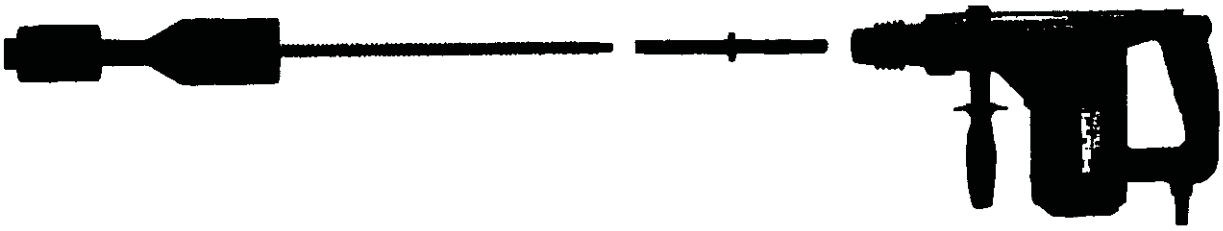
HDA-T Anchor

HDA-P	HDA-P F	HDA-P R	HDA	HDA-P Anchor	Stop Drill Bit		Diamond Core Bit	Setting Tool	
Item No. Galvanized	Item No. Sherardised	Item No. 316 Stainless	Box Qty	Anchor (mm) dia x min.emb/max. thickness fastened	Item No.	Description (mm) dia. x drill depth	Diameter	Item No.	Description
331544	339356	339346	12	HDA-P M10x100/20	332089	TE-C-B20x100	20mm (25/32")	331843	TE-C-ST-M10
					237449	TE-Y-B20x100		287133	TE-Y-ST-M10
331546	339357	339347	8	HDA-P M12x125/30	331091	TE-C-B22x125	22mm (7/8")	331844	TE-C-ST-M13
					237451	TE-Y-B22x125		287134	TE-Y-ST-M13
331547	339358	339348	8	HDA-P M12x125/50	332091	TE-C-B22x125	22mm (7/8")	331844	TE-C-ST-M12
331550	339359	339349	4	HDA-P M16x190/40	332097	TE-Y-B30x190	30mm (1-1/8")	331846	TE-Y-ST-M16
331551	339360	339350	4	HDA-P M16x190/60	332097	TE-Y-B30x190	30mm (1-1/8")	331846	TE-Y-ST-M16
339265	–	–	2	HDA-P M20x250/50	339270	TE-Y-B37x250	37mm (1-3/8")	339269	TE-Y-ST-M20
339266	–	–	2	HDA-P M20x250/100	339270	TE-Y-B37x250	37mm (1-3/8")	339269	TE-Y-ST-M20

4.3.1 HDA Removal Tool

4.3.1.6 HDA Removal

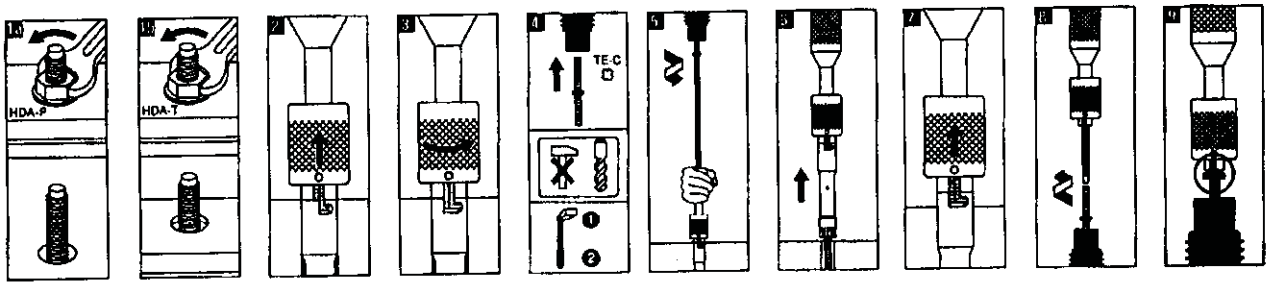


The Hilti HDA Removal Tool is designed to remove the Hilti HDA Undercut Mechanical Anchor when installed in standard applications in accordance with Hilti guidelines.

Product Features

- Complete removal of HDA design anchors for temporary applications
- The removal process strips the threads to prevent reuse of anchors for safety purposes
- Suitable for all rotary hammers with TE-C style chucks

Removal Instructions



1. Remove the nut and washer from the threaded rod, (also remove fastening part for HDA-P applications).
2. Push back the grip (against this spring pressure).
3. Allow the two drive lugs to engage the groove in the anchor sleeve using a slight twisting movement of the grip. Release the grip.
4. Insert the adapter (drive) into the drill chuck and lock (TE 25 recommended).

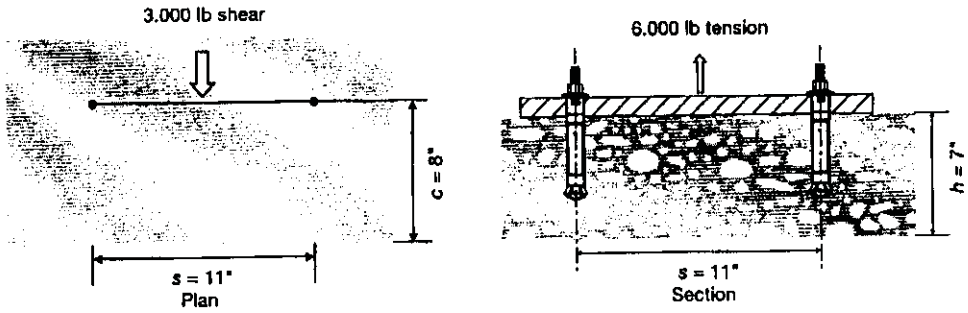
   Important:

   - **Switch off the hammering action** (the removal tool will be permanently damaged if this step is neglected.).
   - **Use low speed** (setting 1 for the TE 25).
5. Put adapter (drive) onto the threaded spindle of the removal tool and switch on the drill.
6. The anchor sleeve will be extracted.
7. Disengage the drive lugs from the groove by lifting up and twisting the grip.
8. To return the tool to its starting position, put the adapter (drive) on the other end of the threaded spindle.
9. Switch on the hammer drill until the adapter stop reaches the removal tool.

Removal Tool with Adapter

Item No.	Description	Qty/Pkg	Applicable Anchor Sizes
333433	TE-C-HDA-RT 20-M10	1	HDA M10
333434	TE-C-HDA-RT 22-M12	1	HDA M12
333435	TE-C-HDA-RT 30-M16	1	HDA M16
339273	TE-C-HDA-RT 37-M20	1	HDA M20

4.3.1.7 Sample Calculations



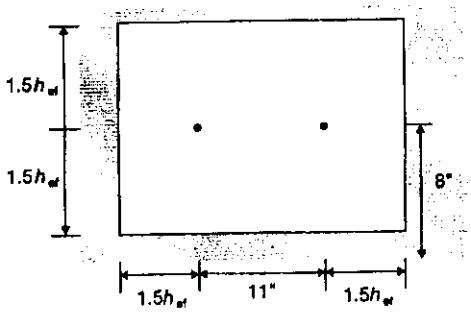
Given:

f'_c = 3,500 psi, Cracked Concrete, No Supplementary Reinforcement

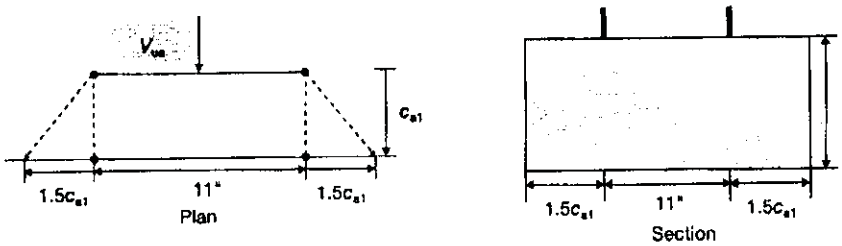
Loads:	Tension	4,000 lb DL
		2,000 lb LL
		No Eccentricity, Non-Seismic Loads
	Shear	2,000 lb DL
		1,000 lb LL
		No Eccentricity, Non-Seismic Loads

Calculation per ACI 318-05 Appendix D and ESR-1546	ACI 318-05 Code Reference	Reference
Try an HDA-T M10 Anchor h_{ef} = 3.94 in.		
Check Spacing, Edge Distance, Thickness		ESR-1546 Table 5
s_{min} = 4 in. < 11 in. OK		
c_{min} = 3.125 in. < 8 in. OK		
h_{min} = 6.75 in. < 7 in. OK		
Calculate Factored Loads		
Use ACI 318-05 Chapter 9 Load Factors	9.2.1	
N_{ua} = 1.2 (4,000 lb) + 1.6 (2,000 lb) = 8,000 lb for anchor group		
N_{ua} = 4000 lb / anchor		
V_{ua} = 1.2 (2,000 lb) + 1.6 (1,000 lb) = 4,000 lb for anchor group		
V_{ua} = 2000 lb / anchor		
Calculate Steel Strength in Tension ϕN_{sa}		
ϕ = 0.75	D.4.4(a)(i)	ESR-1546
N_{sa} = (n) (A_{se}) (f_{uta})	D.5.1.2	Table 5
= (2) (0.09 in^2) (116,000 lb/in^2)	(D-3)	
= 20,880 lb for anchor group		
N_{sa} = 10,440 lb/anchor		ESR-1546
N_{sa} = (2-anchors) (10,440 lb/anchor)		Table 5
N_{sa} = 20,880 lb for anchor group		
ϕN_{sa} = (0.75)(10,440 lb/anchor) = 7830 lb/anchor		
For the anchor group: (2)(ϕN_{sa}) = (2)(7830 lb/anchor)		
→ ϕN_{sa} for the anchor group = 15,660 lb		

4.3.1 HDA Undercut Anchor

Calculate Concrete Breakout Strength in Tension ϕN_{cbg} **Category 1, Condition B** $\phi = 0.65$	**D.4.4(c)(ii)**	**ESR-1546 Table 5**
$N_{cbg} = \left[\left(\frac{A_{Nc}}{A_{Nco}}\right)(\Psi_{ec,N})(\Psi_{ed,N})(\Psi_{c,N})(\Psi_{cp,N})(N_b)\right]$	**D.5.2.1 (D-5)**	
$h_{ef} = 3.94$ in. → $1.5h_{ef} = 5.91$ in. → $3h_{ef} = 11.82$ in.		**ESR-1546 Table 5**
		
$A_{Nco} = 9(h_{ef})^2 = 139.7$ in^2	**D.5.2.1(b) (D-6)**	
$A_{Nc} = (5.9 + 11 + 5.9)(5.9 + 5.9) = 269.7$ in^2 → $\boxed{\frac{A_{Nc}}{A_{Nco}} = 1.93}$		
no eccentricity $\boxed{\Psi_{ec,N} = 1.0}$	**D.5.2.4 (D-9)**	
$c_{a,min} = 8$ in. $1.5h_{ef} = 5.91$ in.	**D.5.2.5 (D-10)**	
$c_{a,min} > 1.5h_{ef}$ → $\boxed{\Psi_{ed,N} = 1.0}$		
use $\boxed{\Psi_{c,N} = 1.0}$	**D.5.2.6**	
use k_{cr} for cracked concrete to calculate N_b		
splitting does not govern	**D.5.2.7**	**ESR-1546 Sect. 4.1.2**
$\boxed{\Psi_{cp,N} = 1.0}$		
$N_b = k_{cr}\sqrt{f'_c}\,(h_{ef})^{1.5}$		
$k_{cr} = 24$, $h_{ef} = 3.94$ in.		**ESR-1546 Table 5**
$\phi N_{cbg} = (0.65)[(1.93)(1.0)(1.0)(1.0)(1.0)(11{,}104\text{ lb})] = (0.65)[(21{,}431\text{ lb})]$		
$\boxed{\phi N_{cbg} = 13{,}930\text{ lb}}$		
Calculate Pullout Strength ϕN_{sa} **Category 1, Condition B** $\phi = 0.65$ **Anchor Category 1, Condition B**	**D.4.4(c)(ii)**	**ESR-1546 Table 5**
$N_{pn} = \Psi_{c,P} N_p$	**D.5.3.1 (D-14)**	
N_p for post-installed anchors based on testing		**D.5.3.2**

ESR-1546 (Table 5)	$N_{pn,rc} = N_{pn,cr}\sqrt{\frac{f'_c}{2500}}$ for cracked concrete $N_{pn,cr}$ = 8992 lb for one anchor $N_{pn,rc} = \left[(8992\text{ lb})\sqrt{\frac{3500}{2500}}\right]$ $N_{pn,rc}$ = 10,639 lb / anchor $\boxed{\phi N_{pn,rc} = (0.65)(10{,}639) = 6915\text{ lb /anchor}}$		ESR-1546 Sect. 4.1.3 ESR-1546 Table 5
	Concrete side-face blowout strength of a headed anchor in tension The design requirements for side-face blowout...."are applicable to headed anchors that usually are cast-in anchors. Splitting during installation rather than side-face blowout generally governs post-installed anchors.....". Side-face blowout for post-installed anchors could be calculated if the bearing area A_{brg} is known. A_{brg} is given for HDA anchors in Table 1 of this section. Side-face blowout will not control for this example.	D.5.4 RD.5.4	
Summary for Tension			
	Design steel strength = ϕN_{sa} = 7830 lb/anchor Factored service load = ϕN_{ua} = 4000 lb/anchor $\phi N_{sa} > N_{ua} \rightarrow$ OK Design concrete breakout strength = ϕN_{cbg} = 13,930 lb for anchor group Factored service load = N_{ua} = 8000 lb for anchor group $\phi N_{cbg} > N_{ua} \rightarrow$ OK Design pullout strength = ϕN_{pn} = 6915 lb/anchor Factored service load = N_{ua} = 4000 lb/anchor $\phi N_{pn} > N_{ua} \rightarrow$ OK		
Calculate Steel Strength in Shear ϕV_{sa}			
	ϕ = 0.65	D.4.4(a)(ii)	ESR-1546 Table 5
	V_{sa} = 13,938 lb/anchor Assume base plate thickness = 0.50 in.		ESR-1546 Table 6
	ϕV_s = (0.65)(13,938 lb/anchor) = 9059 lb/anchor		
Calculate Concrete Breakout Strength in Shear ϕV_{cbg}			
Condition B	ϕ = 0.70	D.4.4(c)(i)	ESR-1546 Table 5
	$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vco}}\right)(\Psi_{ec,V})(\Psi_{ed,V})(\Psi_{c,V})(V_b)\Big]$	D.6.2.1(b) (D-22)	



4.3.1 HDA Undercut Anchor

$A_{Vco} = (4.5)(c_{a1})^2 = (4.5)(8\text{ in.})^2 = 288\text{ in}^2$	D.6.2.1 (D-23)	
$A_{Vc} = (1.5c_{a1} + 11\text{ in.} + 1.5c_{a1})(7\text{ in.}) = (12\text{ in.} + 11\text{ in.} + 12\text{ in})(7\text{ in.}) = 245\text{ in}^2$		
$\boxed{\frac{A_{Vc}}{A_{Vco}} = 0.85}$		
$\boxed{\Psi_{ec,V} = 1.0}$ no eccentricity	D.6.2.5 (D-2)	
$c_{a2} > 1.5_{ca1}$ no edge condition perpendicular to shear load	D.6.2.6 (D-27)	
$\boxed{\Psi_{ed,V} = 1.0}$		
cracked concrete , no supplementary reinforcement	D.6.2.7	
$\boxed{\Psi_{c,V} = 1.0}$		
$V_b = 7\left(\frac{\ell_e}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f'_c}\ (c_{a1})^{1.5}$	D.6.2.2 (D-24)	
$\ell_e = h_{ef}$ for HDA-T M10 $\rightarrow \ell_e = 3.94$ in		
$d_o = 0.75$ in. for HDA-T M10 anchor Note: ESR-1546 only lists do for HDA-T anchors. If HDA-P anchors are being used, refer to the Hilti Product Tech Guide to obtain values of d_o. $d_o = 0.394$ in. for HDA-P M10 anchor For this example, an HDA-T anchor is being considered so use $d_o = 0.75$ in.		ESR-1546 Table 5
$f'_c = 3{,}500$ psi $\qquad c_{a1} = 8$ in.		
$V_b = 7\left(\frac{3.94\text{ in.}}{0.75\text{ in.}}\right)^{0.2}\sqrt{0.75\text{ in.}}\sqrt{3{,}500\text{ in.}}\ (8\text{ in.})^{1.5}$		
$\rightarrow \boxed{V_b = 11{,}308\text{ lb / anchor}}$		
$\phi V_{cbg} = (0.70)[(0.85)(1.0)(1.0)(1.0)(11{,}308\text{ lb}) = 11{,}308\text{ lb}$ $= (0.70)[(9612)]$		
$\boxed{\phi V_{cbg} = 6728\text{ lb}}$		
Calculate Pryout Strength ϕV_{cpg}		
Condition B $\quad \phi = 0.70$	D.4.4(c)(i)	ESR-1546 Table 5
$V_{cpg} = (k_{cp})(N_{cbg})$	D.6.3.1 (b) (D-30)	
$h_{ef} = 3.94\text{ in.} > 2.5\text{ in.} \rightarrow k_{cp} = 2.0$		ESR-1546 Table 5
$N_{cbg} = 21{,}431$ lb		
$\phi V_{cpg} = (0.70)(2)(21{,}431\text{ lb}) = 30{,}003$ for anchor group		

Summary for Shear

Design steel strength = ϕV_{sa} = 9059 lb/anchor
Factored service load = V_{ua} = 2000 lb/anchor
$\phi V_{sa} > V_{ua} \rightarrow$ OK

Design concrete breakout strength = ϕV_{cbg} = 6728 lb for anchor group
Factored service load = V_{ua} = 4000 lb for anchor group
$\phi V_{cbg} > \rightarrow$ OK

Design concrete pryout strength = ϕV_{cp} = 15,001 lb/anchor
Factored service load = V_{ua} = 2000 lb/anchor
$\phi V_{cp} > V_{ua} \rightarrow$ OK

Check Interaction Equation

Determine controlling design loads:

Tension: ϕN_{sa} = (2-anchors) (7830 lb/anchor) = 15,660 lb for anchor group

ϕN_{cbg} = 13,930 lb for anchor group

CONTROLS ϕN_{pn} = (2-anchors) (6915 lb/anchor) = **13,830 lb** for anchor group

Shear: ϕV_{sa} = (2-anchors) (9059 lb/anchor) = 18,118 lb for anchor group

CONTROLS ϕV_{cbg} = **6728 lb** for anchor group

ϕV_{cbg} = 30,003 lb for anchor group

$(0.2)(\phi V_{cpg})$ = 1346 lb $\quad V_{ua}$ = 4000 lb $\quad V_{ua} > (0.2)(\phi V_{cbg})$ **D.7.1**

$(0.2)(\phi N_{pn})$ = 2766 lb $\quad N_{ua}$ = 8000 lb $\quad N_{ua} > (0.2)(\phi N_{pn})$ **D.7.2**

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2$$ **D.7.3 (D-31)**

$$\frac{8{,}000 \text{ lb}}{13{,}830 \text{ lb}} + \frac{4{,}000 \text{ lb}}{6{,}728 \text{ lb}} = 0.579 + 0.595 = 1.17$$

$1.17 < 1.2 \rightarrow$ OK

USE HDA-T M10 Anchor

Analysis Plan for
ANALYSIS OF TRA-670 UTILITY BATTERY ROOM WALL ANCHORAGE

Analysis Plan

Scope: The purpose of this calculation is the evaluate the proposed bracing necessary to attach the tops of the reinforced concrete block walls enclosing the TRA-670-E58/E59 battery room to the ceiling above the walls. This is based upon recommendations in a recent seismic evaluation of these walls performed by others (Ref. 1). The block walls are not themselves a seismic category 1 SSC, but are considered a potential seismic interaction hazard to the safety related, seismic category 1 battery banks. The objective of the bracing modification is therefore to provide adequate anchorage to preclude adverse interaction with the batter banks during a safe shutdown earthquake (SSE). Scope includes the effect of the bracing on the wall, and the structural capacity of the bracing itself.

Deliverables: The deliverable shall be a checked and approved ECAR.

Assumptions: Assumptions are made as to material of construction, since the design is not fully specified during analysis. These do not require verification, as they are conservative relative to the possible choices.0

Quality Level: 1 per QLD RTC-000633, "Seismic Interaction Structural Anchorage

Natural Phenomena Hazards criteria: PC-4 per PLN-588 Rev 3.

Load scenarios and acceptance criteria – bracing only supports out of plane loads, and seismic are the only such loads. Use DOE-STD-1020-02 load combination, where Dns=0 (for the bracing). DOE-STD-1020-02 acceptance criteria apply.

Calculation and analysis software to be used: Mathcad will be used

Verification/validation requirements for calculation and analysis software: None – calculations will be checked by hand.

Budget: not defined

Schedule: not defined

Approvals

SR Brown for Rowsell by telecon — 10/23/08
Requester – Dave Rowsell — Date

SR Jensen — 10/23/08
Performer – Stuart Jensen — Date

[signature] — 10/23/08
Performer's Manager – C. C. O'Brien — Date