Equation (D-19) is for cast-in headed stud anchors. It is based on the fixity of the anchor to the embedment and is appropriate for use in designing anchors welded to embedded plates.

b. Concrete Breakout Strength:

For shear force perpendicular to an edge, the capacity of the anchor group is allowed to be checked with an edge distance based on the anchor farthest from the edge as stated in ACI 318 for anchors welded to a plate.

For shear force parallel to an edge, the capacity of the anchor group is allowed to be twice the value of the shear capacity calculated perpendicular to an edge.

For anchors located at a corner, the minimum capacity calculated above, for parallel and perpendicular loads, should be taken as the design capacity, ϕV_{cbg}.

c. Concrete Pryout Strength: Concrete pryout strength should be calculated in accordance with ACI 318.

3.9.7 Interaction of Tensile and Shear Forces

Interaction between tensile and shear forces should be in accordance with ACI 318.

3.9.8 Seismic Considerations

This section is applicable for Seismic Design Category C, D, E, or F. When anchor design includes seismic forces, the anchor design strength associated with concrete failure modes should be reduced in accordance with ACI 318 requirements. The philosophy for the design of steel embedments subject to seismic loads is that the system should have adequate ductility. Anchor strength should be governed by ductile yielding of a steel element. If the anchor cannot meet these ductility requirements (which is the case for most embedded plates with welded studs because of relatively short embedment depth and close spacing), then either the attachment is designed to yield (ACI 318 Section D.3.3.5) or the calculated anchor strength is substantially reduced to minimize the possibility of a brittle failure (ACI 318 Section D.3.3.6). Alternatively, longer welded rebar may be used as opposed to welded studs. (See 3.9.4.)

3.9.9 Examples of Design of Welded Anchors for Embedded Plates

There are several examples of single and multiple studs welded to embedded plates under tension, shear, moment, and combinations of these loads in ACI 349.2R. Engineers are encouraged to use this reference when the need arises.

3.10 CONSIDERATIONS FOR VIBRATORY LOADS

3.10.1 General

Vibratory loads are only a consideration in the design of anchorage in petrochemical facilities if they are high-cycle, that is, more than 2×10^6 cycles. Neither ACI 318 nor ACI 349 addresses the design of anchors for high-cycle fatigue. Fatigue testing of adhesive anchors indicates that fatigue of the bonding materials is not critical.

Fatigue behavior is the most critical for anchor groups having anchors installed through holes in a steel plate or other fixture, since there is significant potential for unequal shear load distribution. Where fatigue due to shear is determined to be important, it is advisable to eliminate movement in the connection via welded thickened washers or supplemental grouting of the annular gap.

Fatigue due to tension loading can be reduced through tensioning of the anchor. (See 3.8.) Tensioning requires sufficient anchor length to develop strains that are large compared to the strain associated with concrete relaxation and creep. The residual tension in the anchor should exceed the peak cycling load. The resistance to fatigue is directly related to the ratio between the minimum and the maximum cyclic stress. Figure 3.24 illustrates this point.

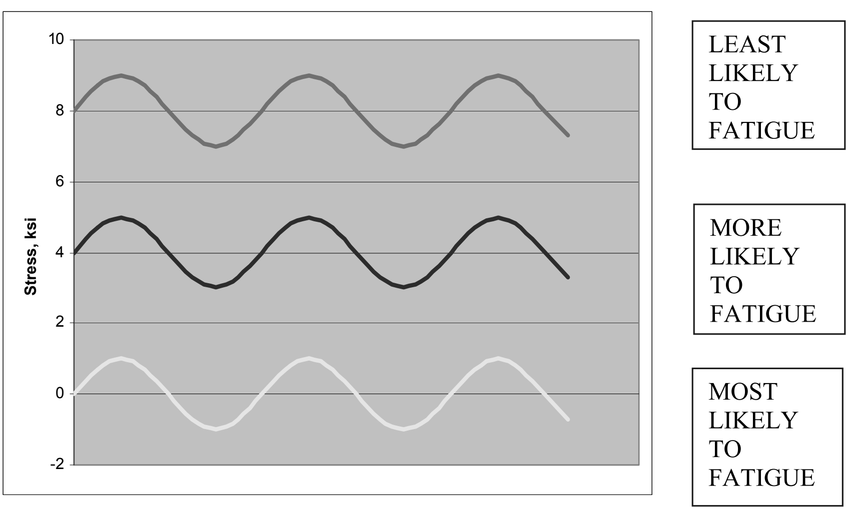


Figure 3.24: Effect of Preloading Anchors on Fatigue

The lower curve is for no static preload, the middle curve is for a static preload of 4 ksi (27.6 MPa) and the upper curve is for a static preload of 8 ksi (55.2 MPa). The cyclic load amplitude of 2 ksi (13.8 MPa) is the same in all cases. The ratio of the minimum to the maximum cyclic load for the lower curve is -1/1 = -1, the ratio for the middle curve is 3/5 = 0.6, and the ratio for the upper curve is 7/9 = 0.778.

For these load cases, the load case illustrated by the upper curve is the least likely to fatigue, the case illustrated by the middle curve is more likely to fatigue, and the case illustrated by the lower curve is the most likely to fatigue, since it has complete load reversal.

3.10.2 Rules for Avoiding Fatigue Failure

Fabrication processes (forming, cutting, welding, heat treatment, and galvanizing) and the thread production method and configuration are critical for the behavior of threaded connections subjected to high-cycle fatigue loading. This is particularly important in order to eliminate crack initiation, particularly at the first thread inside the nut, where tension fatigue failures typically occur due to the increased stress at this location. Thus, the following rules should be observed in order to avoid fatigue failure.

a. Use the proper grade of nut with the bolt and ensure full thread engagement in the nut

b. Use rolled threads to avoid stress risers in the threads and shot peening to induce residual compressive stresses in the bolt

c. Use spherical washers beneath the nut to avoid inducing bending loads in the bolt when it is tensioned due to lack of parallelism between the bottom of the nut and the bolted parts

d. Use the fewest possible elastic materials in the joint (gaskets, chocks, etc.) in order to maintain anchor preload and avoid long term relaxation

e. Avoid bending and shear loads on the anchors. Anchors loaded in pure tension are the least likely to fatigue.

f. Use the longest bolt possible to get the greatest strain (stretch) for the applied preload. Bolted joints are held together by the elastic energy stored in the bolt. The amount of energy stored goes up as the square of the stretch length, which in turn increases linearly with length. For example, a 4-in. (101.6 mm) bolt stretched to 70 percent yield will stretch twice as far as a 2-in. (50.8 mm) bolt stretched to 70 percent yield, but the longer bolt contains four times the elastic energy as the shorter one.

g. Put the maximum possible preload on the anchors.

> *Note: Many practitioners use 80-90% of the yield stress for 40 ksi steel anchors and 50-70% of the yield stress for ASTM A193 Grade B7 steel anchors because of the potential for stress corrosion cracking at higher stresses.*

When maintaining the prestress tension is important, a load monitor such as the RotaBolt® Load Monitor or equivalent can provide an easy method of checking that there has been no loss of tension that would allow a load reversal.

3.11 CONSIDERATIONS FOR SEISMIC LOADS

3.11.1 General

The flow chart shown in Figure 3.25 provides clarity to the procedure of designing anchorages for earthquake considerations. This flow chart gives a logical procedure for considering the requirements of ACI 318 Appendix D, AISC 341, and ASCE/SEI 7 regarding earthquake design.

Although ductile anchorage is recommended for all anchorages, seismic detailing is required by code only for structures assigned to Seismic Design Categories C, D, E, and F, regardless of the governing load combination.

Unless otherwise required, anchorages should be designed to resist seismic loads from all load combinations that include non-amplified seismic loads in accordance with the applicable building code. An example where an anchorage should be designed for member strength or amplified loads is a column base connection designed in accordance with AISC 341, *Seismic Provisions for Structural Steel Buildings*. Amplified seismic loads are loads that result from load combinations that include the overstrength factor Ω_o. An example of member strength design is designing a connection for the tensile strength of the brace for a Special Concentrically Braced Frame (SCBF) in accordance with AISC 341. When a connection with anchorage is not required to be designed for member strength or amplified seismic loads the nominal capacities of anchors for structures that have been assigned to Seismic Design Categories C, D, E, or F should be subject to the following additional requirements:

a. To reflect the uncertainty associated with anchorage resistance in a concrete structure or foundation that is undergoing inelastic deformations, anchorage design strength capacity in tension and shear associated with concrete failure modes should be taken as $0.75\phi N_n$ and $0.75\phi V_n$, where N_n and V_n are the nominal strengths associated with the controlling concrete failure modes in tension and shear, respectively, as determined in accordance with ACI 318 Appendix D. If rebar is used to develop anchor forces it should also be designed in accordance with the above guideline.

b. In order to assure a ductile anchorage, the concrete strength as determined in paragraph (a) (that is, concrete breakout, pullout, and side-face blowout) should be greater than the strength of the ductile steel embedment element.

c. Where ductility in the anchor cannot be achieved, it is acceptable to force ductile yielding in the attachment, for instance the base plate, by designing the attached component to yield at forces no greater than the design strength of the anchors as described in paragraph (a).

d. Where yielding in the attached component or in the anchor cannot be achieved, it is acceptable to design the anchorage for 2.5 times the seismic loads transmitted by the attachment. ACI 318 Section D.3.3.6 strength reduction factors should not be used in conjunction with the 2.5 amplification factor.

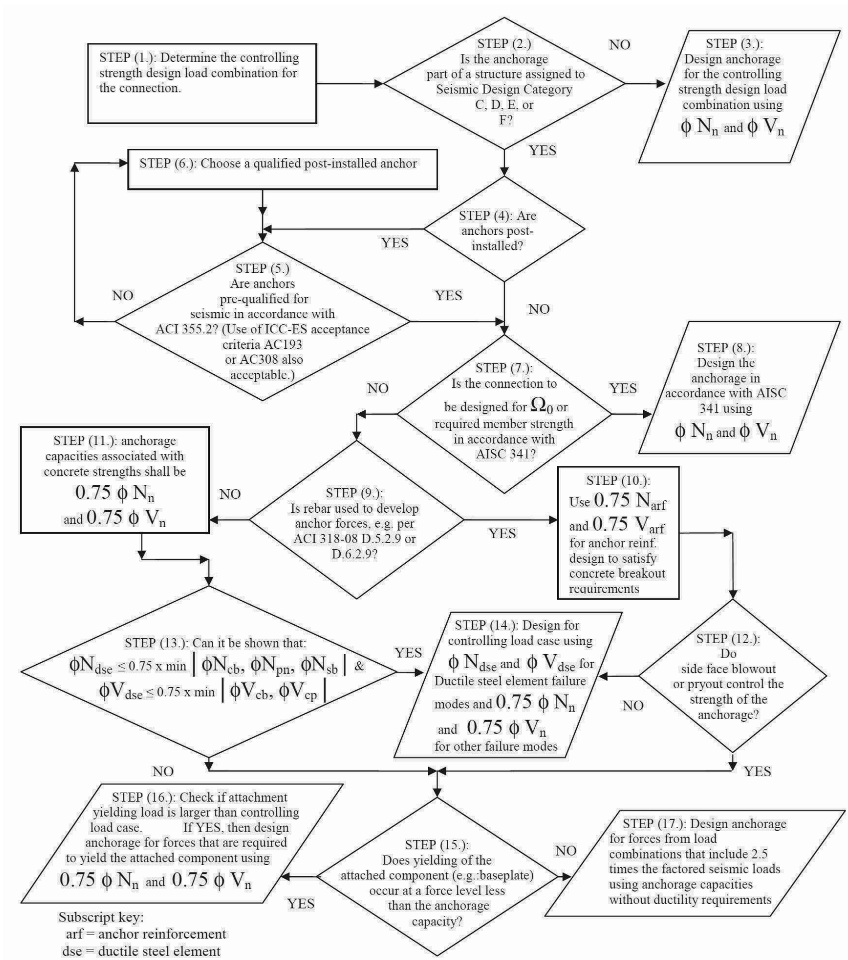


Figure 3.25: Flow Chart for Seismic Design of Anchorage

3.11.2 Connections Designed in Accordance with AISC 341

Seismic detailing of structural steel is specified in AISC 341. Steel structures assigned to Seismic Design Categories D, E, and F should be detailed in accordance with AISC 341 unless covered by exceptions provided in ASCE/SEI 7 Chapter 15. Steel structures in Seismic Design Categories B and C designed in accordance with ASCE/SEI 7 Table 12.2-1 Part H., "Steel Systems Not Specifically Detailed for Seismic Resistance, Excluding Cantilever Column Systems" are exempt from AISC 341 detailing requirements, as are all structures in Seismic Design Category A.

Column bases, including the anchorage, of structures conforming to AISC 341, are designed in accordance with Chapter 8 of that document. AISC 341 Section 8.5a requires that the axial capacity of the column base be taken as the sum of the forces and member capacities of all elements framing into the base. AISC 341 Sections 8.5b and 8.5c require column bases to be designed for the column expected shear strength and column expected flexural strength, respectively. Typically, the anchorage strength demands determined in accordance with AISC 341 (based on member strengths or overstrength factors) will govern the design, as opposed to the strength demands determined in accordance with the non-amplified seismic load combinations of ASCE/SEI 7. Anchorage for components is typically required to be designed for a higher seismic load than anchorage for items that are not components. This is due to the nature of seismic demands on components during earthquakes. The purpose of the additional requirements for component anchorage is to provide a continuous load path of sufficient strength and stiffness between the component and the supporting structure.

3.11.3 Nonstructural Components

Nonstructural components are subject to special requirements for anchorage that are not specifically addressed in this report. ASCE/SEI 7 Chapter 13 provides specific anchorage requirements for components and defines detailing and design parameters for components such as piping, conduit, cable tray, and small equipment. With the exception of storage racks, the dividing line between nonstructural components as addressed in ASCE/SEI 7 Chapter 13 and nonbuilding structures as addressed in ASCE/SEI 7 Chapter 15 is made on the basis of the weight of the component as a percentage of the overall structure weight.

3.11.4 Pedestal Anchorage

Reinforced concrete pedestals designed to receive loads from supported steel structures, tanks, and vessels are typically required to transfer large concentrated forces at the anchorage interface, typically at the top of the pedestal. The design of such anchorages is complicated by the reduced edge distances and anchor spacing as well as the need for large tension and shear capacity to accommodate the calculated lateral and overturning forces in the attachment. For typical cases, additional ties as shown in Figure 3.26 may be adequate to facilitate shear transfer. Special cases may require other solutions such as shear lugs or side plates. As previously discussed, the transfer of tension forces to the vertical pedestal reinforcing will likely be governed by the large splitting stresses generated around the anchorage, and as such, a design of the anchor embedment corresponding to development/splice length in accordance with the provisions of ACI 318 Chapter 12 should be considered. It is also recommended that additional ties be provided at and directly above the level of the head of headed anchor bolts to take up the bursting forces generated around the anchor head.

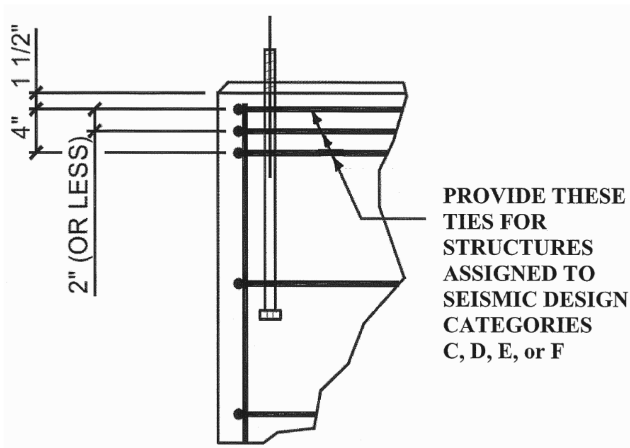


Figure 3.26: Seismic Pedestal Ties for Anchorage.

As noted previously, anchor reinforcement properly designed in accordance with ACI 318 Appendix D precludes the need to calculate concrete breakout strength. Proper detailing is critical to assure load transfer from the anchorage to the reinforcement. In Appendix D this is accomplished by requiring that the anchor reinforcement be developed on both sides of the theoretical crack plane corresponding to concrete breakout. Note that for tension-loaded anchors where splitting of the concrete will likely govern the anchor strength (that is, anchors in the top of a column or pedestal with limited edge distance), it may be advisable to treat the load transfer from anchor to reinforcement as a non-contact lap splice and to refer to the development length provisions of ACI 318 Chapter 12. It is also recommended that those provisions of ACI 318 (for example, 12.2.5) that permit the reduction of development length based on the provision of more than the required reinforcing area $(A_{s,\ required})/(A_{s,\ provided})$ should not be used when developing anchor reinforcement to resist anchorage-induced seismic loads.

3.11.5 Seismic Design of Vertical Vessel Anchors

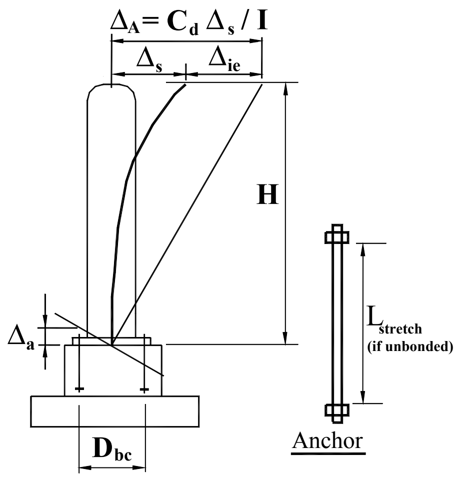
Historically, the foundation anchors for tall vertical vessels and stacks have tended to stretch beyond yield when subjected to strong ground motion, which probably prevented collapse of these vessels. Based on this experience, it is recommended that these anchors be designed with ductile embedment into the foundation. (Special care should be taken not to significantly oversize the anchors.) Oversizing could cause the anchors to not yield during a seismic event, thus increasing the load on the foundation and creating overturning moments in the foundation beyond those assumed in the design.

In specific instances where anchor elongation is required for inelastic displacement of the supported equipment or structure, a minimum stretch length of anchors should be calculated and detailed. These provisions are particularly important for facilities that rely primarily on the foundation and anchors for ductility, such as fixed base cantilever stacks and skirt supported vertical vessels. It is industry practice to use a minimum stretch length of 12 anchor diameters in these situations. Some examples of detailing provisions that provide anchor stretch are: using extended anchors with high chairs on vessel skirts, providing full length sleeves filled with elastomeric material, and using industrial tape or grease to break the concrete bond on the anchor shaft.

A procedure for determining the minimum stretch length of vertical vessel anchors is shown in Figure 3.27. In order to use this procedure the static displacement at the top of the vertical vessel due to the Equivalent Lateral Force Procedure seismic loads, Δ_s, should first be calculated. The amplified displacement at the top of the vessel, Δ_A, equals Δ_s plus Δ_{ie}. The inelastic portion of the vessel amplified displacement, Δ_{ie}, is assumed to be caused by anchor bolt stretch because inelasticity should not occur in the vessel or skirt and foundation rocking can lead to instability. The elongation length of the anchor bolts, Δ_a, required to cause the inelastic portion of vessel amplified displacement can be found from the geometry shown in Figure 3.27. The required anchor bolt stretch length, $L_{stretch}$, can be determined by assuming a reasonable amount of anchor bolt elongation strain, e_a.

When the anchors extend only into the pedestal, the pedestal dowels should be designed to transfer the overturning moment into the footing (minus the resisting moment developed by the pedestal self weight). The dowels should be able to develop an overturning moment equivalent to the overturning moment based on anchor strength. If the anchor bolts extend into the footing, which is often the case for very tall vessels, pedestal dowels do not transfer overturning moment to the footing, and in this case it is only necessary to provide a nominal number of dowels to minimize concrete cracking.

The anchors should be designed to resist the entire seismic shear load at the base if the overturning moment from the seismic forces, acting alone, cannot develop the required frictional resistance between the vessel base and the top of pedestal. In most cases, this frictional resistance is adequate to resist seismic shear forces; therefore, there is no shear force transferred through the anchors.



$$\Delta_s = \text{ deflection from elastic analysis}$$
$$\Delta_{ie} = \Delta_s\,[(C_d / I) - 1]$$
$$\Delta_a = (\Delta_{ie})\,(D_{bc}) / H \;= (e_a)\, L_{stretch}$$
$$L_{stretch} \geq \Delta_a / e_a,\ \text{ Where } e_a \text{ is approx. 5\%}$$

Figure 3.27: Determining the Minimum Stretch Length of Vertical Vessel Anchors

The following equations may be used to calculate the frictional resistance (Figure 3.28).

$$P^E_u = M^E_u / LA + 0.9\,(1/2)\,D - (1/2)\,E_v$$

$$V_f = \mu P^E_u$$

Where:

M^E_u	=	factored overturning moment at the vessel base due to seismic effect acting alone
P^E_u	=	factored compression force at top of pedestal due to seismic effect acting alone (including the vertical component of seismic load acting upward)
D	=	vertical dead load
E_v	=	vertical component of seismic load
LA	=	lever arm between centroid of tension loads on anchors and the centroid of compression load on the pedestal. A conservative approximation of this distance is to use 2/3 of the bolt circle diameter as the lever arm.
μ	=	coefficient of friction. For the normal case of grout at the surface of the pedestal, $\mu = 0.55$.

V_f = frictional resistance force

In order to avoid shear loading on the anchor bolts:

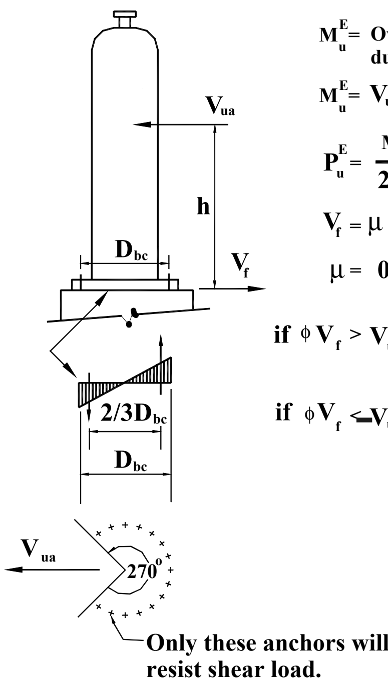
$$V_u \leq \phi V_f$$

Where:

V_u = factored shear load at base of vessel, calculated using load factors in load combinations for uplift cases (see loading combinations and load factors – Strength Design)

ϕ = strength reduction factor = 0.75

In order to minimize the need for excessive bolt edge distance or shear reinforcement when the anchors are designed for seismic shear, the bolts on a 90-degree arc in the direction of the horizontal force are ignored, and the horizontal seismic force is then carried only by the bolts on the remaining 270-degree arc (that is, three-fourths the total number of bolts). (See Figure 3.28.) If this force transfer methodology is followed, special detailing will be required to transfer the lateral load from the vessel to the anchors and foundation.



M_u^E = Overturning Moment due to earthquake loads

$M_u^E = V_{ua} \times h$

$$P_u^E = \frac{M_u^E}{2/3D_{bc}} + \frac{0.9\,D_{bc}}{2} - E_v$$

$V_f = \mu \times P_u^E$

$\mu = 0.55$

if $\phi V_f > V_{ua}$ then anchors do not carry shear load

if $\phi V_f \leq V_{ua}$ then anchors carry all shear load

Figure 3.28: Shear Transfer Methodology for Vertical Vessel Anchors