IN-LB	Inch-Pound Units
SI	International System of Units

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Report on Foundations for Static Equipment

Reported by ACI Committee 351

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Report on Foundations for Static Equipment

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ACI 351.2R-10
(Reapproved 2020)

Report on Foundations for Static Equipment

Reported by ACI Committee 351

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Committee 351 would like to acknowledge the contribution of Widianto, ACI 351 Associate Member.

This document addresses static equipment foundation engineering and construction. It presents various design criteria, methods and procedures of analysis, design, and construction applied to static equipment foundations by industry practitioners. This document should, hopefully, encourage discussion and comparison of ideas.

Keywords: anchorage (structural); bolts, anchor; equipment; forms; formwork (construction); foundation loading; foundations; grout; grouting; pedestals; pile loads; reinforcement; soil pressure; subsurface preparation; tolerances (mechanics).

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Background

Foundations for static equipment are used in industrial processing and manufacturing facilities throughout the world. Engineers with varying backgrounds are engaged in the analysis, design, and construction of these foundations. They often perform their work with little guidance from building codes, national standards, owner's specifications, or other published information. Because of this lack of consensus standards, most engineers rely on engineering judgment and experience. Some engineering firms and individuals, however, have developed their own standards and specifications as a result of research and development activities, field studies, or many years of engineering or construction experience. Only by sharing and discussing this information can a meaningful consensus on engineering and construction requirements for static equipment foundations be developed.

As used in this document, "state of the art" refers to state of the practice, and encompasses various engineering and construction methodologies.

1.2—Purpose

This document presents various design criteria and methods and procedures of analysis, design, and construction currently being applied to static equipment foundations by industry practitioners. The purpose of this report is to present these various methods and elicit critical discussion from the industry. This report is not intended to be a recommended practice; rather, it is a document that encourages discussion and comparison of ideas.

1.3—Scope

This report is limited in scope to the engineering and construction of static equipment foundations. Static equipment, as used herein, refers to industrial equipment that does not contain significant moving parts, or that has operational characteristics essentially static in nature. Outlined and discussed herein are the various aspects of the analysis, design, and construction of foundations for equipment, such as vertical vessels, stacks, horizontal vessels, heat exchangers, spherical vessels, machine tools, and electrical equipment such as transformers.

This report does not include foundations for:

- Equipment, such as turbine generators, pumps, blowers, compressors, and presses, which have operational characteristics that are essentially dynamic in nature. ACI 351.3R covers concrete foundations for dynamic equipment;
- Vessels and tanks whose contents structurally bear directly on soil (for example, clarifiers and large-diameter storage tanks);
- Buildings, concrete silos, chimneys, and structures that contain static equipment; or
- Equipment sensitive to external vibration. These foundations are generally isolated from the neighboring dynamic equipment foundations to minimize transmission of vibration from other equipment. These foundations rarely require their own separate foundations and are usually located and supported in buildings. ACI 351.3R provides some guidance, although its scope is for equipment that generates dynamic forces.

The geotechnical engineering aspects of the analysis and design of static equipment foundations discussed herein are limited to general considerations. This report is essentially concerned with the structural analysis, design, and construction of static equipment foundations.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A = base area of footing, ft^2 (m^2)
A_{se} = effective cross-sectional area of anchor, in.^{2} (mm^2)
B = width of footing, ft (m)
D = edge-to-edge distance of footing in direction of overturning moment, ft (m)
d_{bc} = diameter of bolt circle, ft (mm)
d_o = nominal bolt diameter, in. (mm)
e = M/P, ft (m) (Section 4.4)
e_v = M/W, ft (m) (Section 5.5.1)
F = maximum bolt force for anchors in circular pattern, kip (kN)
f'_c = specified compressive strength of concrete, psi (MPa)
f_y = specified yield strength of reinforcement, psi (MPa)
h_{ef} = effective embedment depth of anchor, in. (mm)
k_c = coefficient for basic concrete breakout strength in tension
L = length of footing, ft (m)
M = overturning moment applied to footing or pier, ft·kip (kN·m) (Section 4.4)
M_p = resisting moment provided by passive lateral soil pressure, ft·kip (kN·m)
M_v = moment about the centroidal axes of the foundation, ft-lb (N-m) (Section 5.5.1)

M_x, M_y = moments about x and y centroidal axes, respectively, ft·lb (N·m)
M_{xy} = twisting moment, ft·kip (kN·m)
N = number of bolts
N_b = basic concrete breakout strength in tension of single anchor in cracked concrete, lb (N)
n_t = reciprocal of the thread pitch, threads per in. (mm)
P = vertical load due to weight of concrete and equipment, kip (kN)
Q = maximum soil pressure at edge of footing, lb/ft^2 (Pa)
SR_1 = stability ratio for drilled piers with larger length-diameter ratios
SR_2 = stability ratio for drilled piers with smaller length-diameter ratios
S_x, S_y = section moduli of base about x and y centroidal axes, respectively, ft^3 (m^3)
W = resultant vertical load, lb (N) (Section 5.5.1)
W' = weight of equipment, lb (N) (Section 5.2.2)

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource "ACI Concrete Terminology," http://terminology.concrete.org. Definitions provided herein complement that resource.

base ring—a horizontal plate, generally fabricated in the shape of a ring, which is attached to the bottom of a vertical vessel, stack, or skirt and bears on the concrete foundation.

belled piers—a drilled pier shaft with an expanded excavation at the bottom.

bundle load—force required to pull tube bundles out of a vessel.

canister-type anchor bolt—anchorage assembly that includes a sleeve, a threaded rod, and means of removing the rod and adjusting rod location, projection, and tension.

davit—a device (small crane) used to lift, lower, support, and swing the access covers and other items away from openings of vessels and tanks.

heat exchanger—equipment used to raise or lower the temperature of a fluid by piping fluid through tubes exposed to blowing air (radiator type) or by piping fluid through tubes in a bath of fluid that is maintained at a different temperature (shell-and-tube type).

hydrotest—filling of tanks or vessels with water to check for leaks and structural integrity.

operating loads—loads applied to the equipment or structure due to nature of operation, liquid loads, internal loads, or pressure.

pad—slab-type foundation support for equipment.

saddle—a horizontal vessel support with a curved top that matches the profile of the vessel and a flat bottom supported by the foundation.

shaft—vertical portion of concrete drilled pier.

shell-and-tube heat exchanger—see **heat exchanger**.

skirt—a vertical vessel support consisting of a vertical plate rolled to match the perimeter of the vessel and located between the vessel and base ring.

sleeve—a device used around an anchor to accommodate adjustment and tensioning of the anchor after the concrete has hardened.

slide plate—a support that allows movement of the equipment relative to the foundation in one or two directions.

stack—a cylindrical vertical vent.

tube bundle—piping in a shell-and-tube heat exchanger that contains the fluid to be heated or cooled.

vessel, horizontal—a cylindrical vessel with a horizontal primary axis typically supported on one fixed support and one sliding support.

vessel, vertical—a cylindrical vessel with a vertical primary axis typically supported on a single support at the base of the vessel. Multiple supports are also used.

CHAPTER 3—FOUNDATION TYPES

3.1—General considerations

The design of foundations for equipment may depend on the following factors:

1. Equipment base configuration such as legs, saddles, solid base, grillage, or multiple support locations;
2. Anticipated loads, such as the equipment static weight, and loads developed during construction, operation, and maintenance;
3. Operational and process requirements, such as accessibility, settlement constraints, temperature effects, and drainage;
4. Construction and maintenance requirements, such as limitations or constraints imposed by construction or maintenance equipment, procedures, or techniques;
5. Site conditions, such as soil characteristics, topography, seismicity, climate, and other environmental effects;
6. Economic factors, such as capital cost, useful or anticipated life, and replacement or repair costs;
7. Regulatory or building code provisions, such as tied pile caps in seismic design categories;
8. Construction considerations; and
9. Environmental requirements, such as secondary containment or special concrete coating requirements.

3.2—Typical foundations

3.2.1 *Vertical vessel and stack foundations*—For tall vertical vessels and stacks, the plan dimensions of the foundation required to resist gravity loads and lateral wind or seismic forces is usually much larger than the support base of the vessel. Accordingly, the vessel is often connected to a concrete pedestal, smaller than the footing below, with dimensions sufficient to accommodate the anchors and base ring. Operational, maintenance, or other requirements may dictate a larger pedestal. The pedestal is then supported from below by the larger spread footing, mat, or pile cap.

For relatively short vertical vessels and guyed stacks with large bases, light vertical loads, and small overturning moments, the pedestal and footing may have the same plan dimensions. If so, the pedestal and footing are combined into a single soil-supported pedestal.

Individual pedestals are rectangular, circular, square, hexagonal, or octagonal. If the vessel has a circular base, a circular, square, or octagonal pedestal is generally provided.

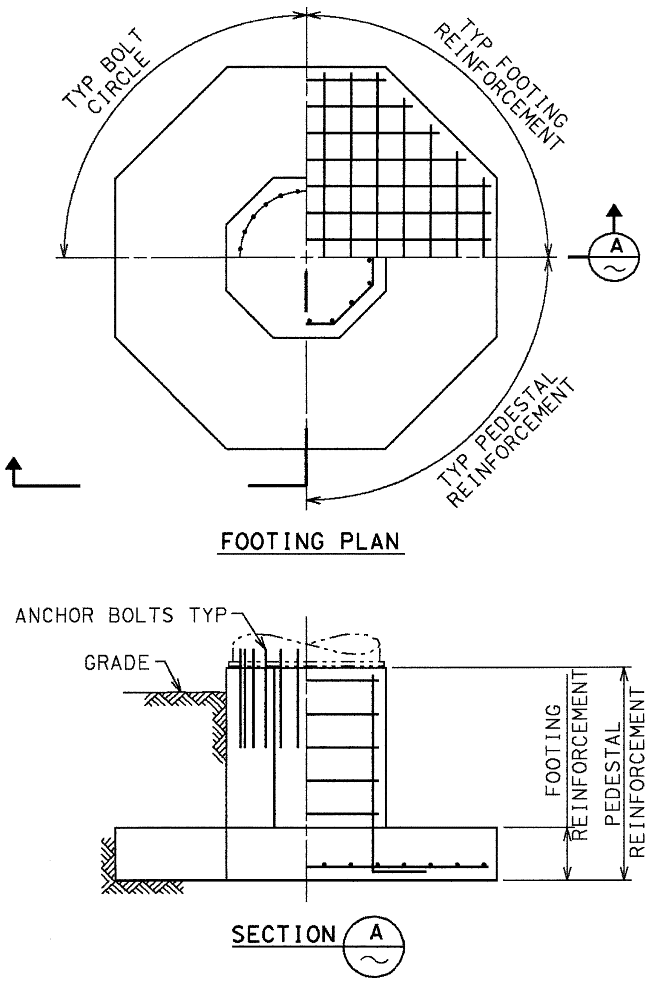


Fig. 3.1—Annular-ring type octagonal pedestal and footing for vertical vessel.

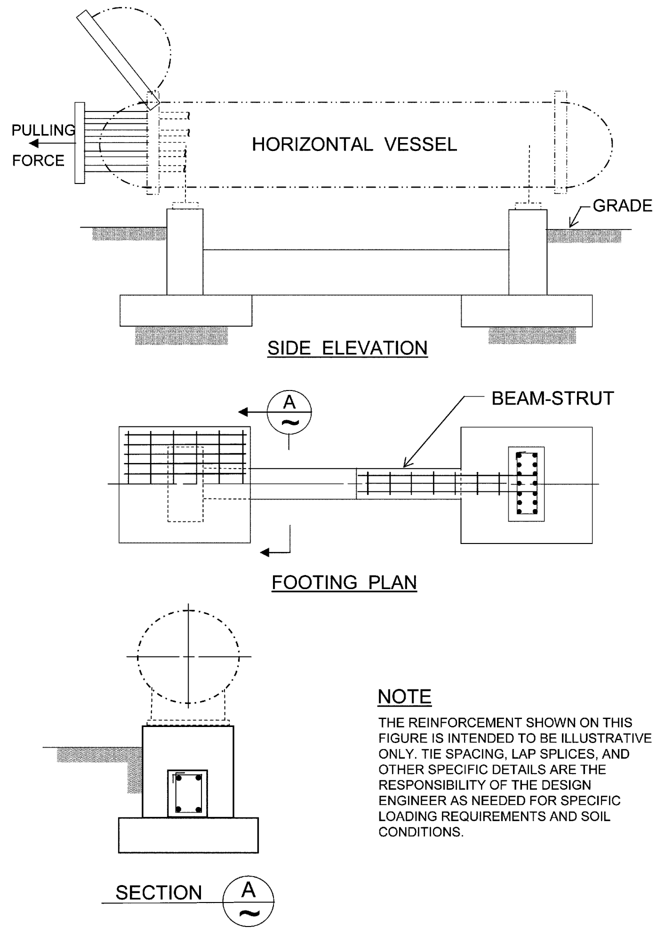


Fig. 3.2—Footing with beam-strut for horizontal vessels.

Circular pedestals may create construction difficulties in forming unless standard prefabricated forms are available. Square pedestals facilitate ease in forming, but may contain much more material than is required by analysis. Octagonal pedestals are a compromise between square and circular; hence, this type of pedestal is widely used for supporting vertical vessels and stacks with circular bases (Fig. 3.1). It is common to use square bases for 5.0 ft (1.5 m) diameter and smaller vessels and octagonal bases for large-diameter vessels.

3.2.2 *Horizontal vessel and heat exchanger foundations*—Horizontal equipment is typically supported on two separate saddles incorporated into the equipment. Each saddle is then supported by a concrete pedestal that rests on spread footings, strap footings, pile caps, or drilled piers. Elevation requirements of piping often dictate that these vessels are located several feet above grade. Consequently, pedestals are the logical means of support (Fig 3.2).

Horizontal equipment often operates at temperatures different than ambient temperature. The change in temperature causes the equipment to expand or contract, depending on whether the operating temperature is higher or lower than ambient temperature. The change in length will push the support pedestals laterally in equal and opposite directions. The loading associated with this temperature differential is covered in Chapter 4.

The configuration of pedestals varies with the type of saddles on the vessels, and with the magnitude and direction of forces to be resisted. The most common pedestal shape used for supporting the saddle is rectangular, when looking down in plan view. Pedestals or foundations may be tied together with beam-struts to reduce the effect of horizontal loading to the foundations (Fig. 3.2). T-shaped pedestals (looking in plan view) are sometimes used where additional horizontal stiffness to resist lateral loads is required (Fig. 3.3).

3.2.3 *Spherical vessel foundations*—Smaller spherical vessels are sometimes constructed with a skirt and base ring, but more often, spherical vessels have leg supports. For leg-supported spherical vessels, foundations typically consist of pedestals under each leg resting on individual spread footings, a continuous mat, or an octagonal, hexagonal, or circular annular ring. Concerns about differential settlement between legs and large lateral earthquake loads usually dictate a continuous foundation system. To economize on foundation materials, an annular ring-type foundation is often used (Fig. 3.4).

3.2.4 *Machine tool foundations*—Machine tool equipment is typically supported on mat foundations. Whether these are soil-bearing or pile-supported depends on the bearing capacity of the soil and the settlement limitations of the machinery. Where a machine tool produces impact-type loads, refer

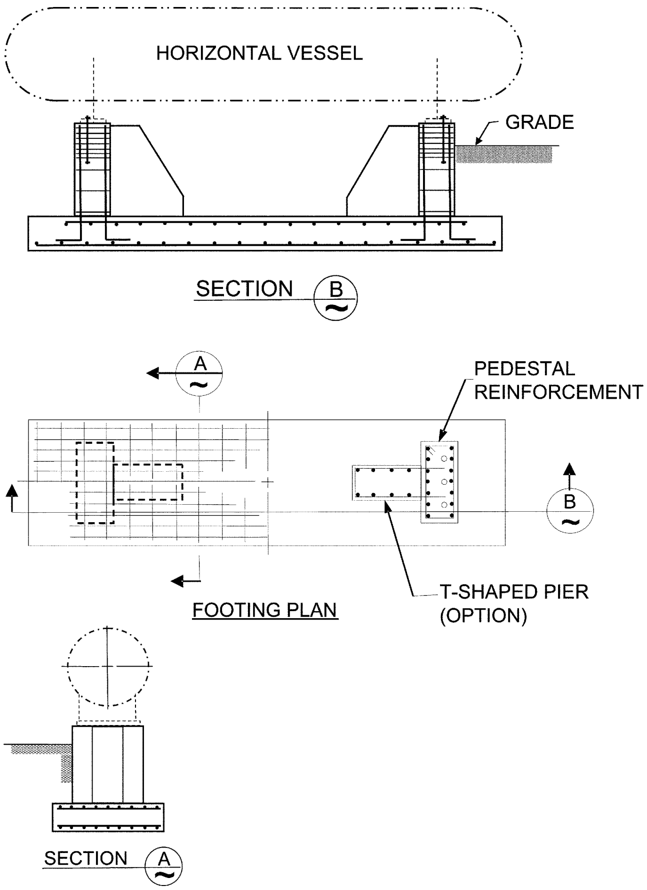


Fig. 3.3—Combined footing for horizontal vessel.

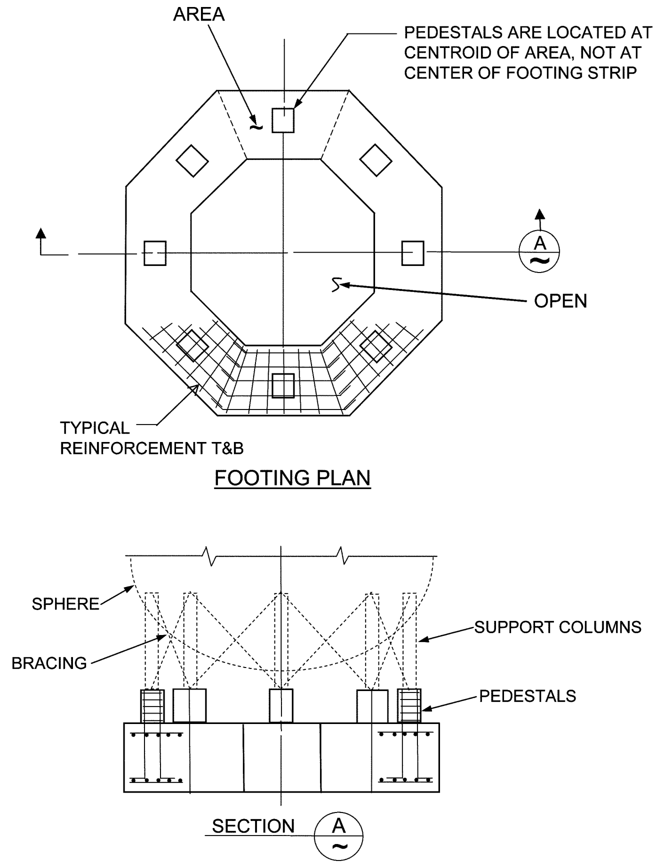


Fig. 3.4—Octagonal footing and pedestals for vertical sphere.

to ACI 351.3R for impact-type loading, design, and construction. If a machine tool foundation is sensitive to vibration, it is generally isolated from the neighboring dynamic equipment foundations to minimize transmission of vibration from other equipment.

3.2.5 *Electrical equipment and support structure foundations*—Electrical equipment typically consists of transformers, power circuit breakers, switchgear, and motor-control centers. Support structures consist of buses, line traps, switches, and lightning arrestors. Foundations for electrical equipment, such as transformers, power circuit breakers, and other massive energized equipment, are typically designed for: dead loads; wind loads; seismic loads; construction loads (that is, jacking); and operating loads. These foundations are typically slabs-on-ground or slabs-on-piles. Anchorage is provided by anchor bolts or by welding the equipment base to plates or structural shapes embedded in the top of the concrete foundation.

A typical electrical substation consists of structures supporting stiff electrical buses, switch stands, line traps, and lightning arrestors. Foundations for these types of equipment are designed to accommodate operating loads, wind loads, short circuit loads, and seismic loads. These loads are usually smaller than those of transmission line support structures. The supporting foundations commonly used are drilled piers or spread footings. Alternatively, pile-supported footings are used if soil-bearing conditions are unfavorable.

Another type of support structure consists of those for overhead electrical conductors, such as transmission towers, poles, dead-end structures, and flexible bus supports. These are designed for tensile loads from the conductors along with ice and wind loads. Drilled piers are commonly used to support such structures. Spread footings or pile-supported footings are also used when required by soil conditions.

CHAPTER 4—DESIGN CRITERIA

Criteria used for the design of static equipment foundations vary considerably among engineering practitioners. Most heavy equipment foundations are designed by or for large organizations, which may include utilities and government agencies. These organizations, with their in-house expertise, have developed their own engineering practices, including design criteria. Unfortunately, there is a limited amount of published information on the criteria used for the design of the types of static equipment foundations covered herein.

4.1—Foundation loading

Most practitioners first attempt to use the common loadings defined by local building codes or by ASCE/SEI 7. Many engineers, however, have difficulty classifying the numerous different loadings into the standard dead and live categories. There is, therefore, a need to define additional categories of loadings and load combinations with appropriate load factors.

4.1.1 *Loads*

4.1.1.1 *Dead loads*—Dead loads consist of the weight of the equipment, platforms, piping, fireproofing, cladding, duct work, and other permanent attachments. Some engineers also designate the operating contents (such as liquid or granular material) of the equipment as dead loads. Such a combination, however, is inconvenient when considering the possible combinations of loads that may act concurrently and when assigning load factors. Equipment may often be empty and still be subject to various other loads. Thus, a distinction between dead and operating loads is generally maintained.

4.1.1.2 *Live loads*—Live loads consist of the gravity load produced by personnel, movable equipment, tools, and other items that may be placed on the main piece of equipment, but are not permanently attached to it. Live loads also commonly include the lifted loads of small jib cranes, davits, or booms that are attached to the main piece of equipment or directly to the foundation.

Live loads will normally not occur during operation of the equipment. Typically, such loads will be present only during maintenance and shutdown periods.

4.1.1.3 *Operating loads*—Operating loads include the weight of the equipment contents during normal operating conditions. These are contents that are not permanently attached to the equipment. Such contents may include liquids, granular or suspended solids, catalyst material, or other temporarily supported products or materials being processed by the equipment. The operating load may include the effects of the contents movement or transfer, such as fluid surge loads in certain types of process equipment. These latter loads, however, are sometimes treated separately in project design criteria, and require different load factors (Section 4.1.3).

Operating loads also commonly include forces caused by thermal expansion (or contraction) of the equipment itself. An example would be a horizontal vessel or heat exchanger with two saddles, each supported on a separate foundation. Temperature change of the equipment can produce horizontal thrusts at the tops of the supporting piers. Allowing the equipment to slide on top of one of the pedestals can reduce the temperature-induced load. This is accomplished by adding a horizontal plate, commonly called a slide plate, between the top of one of the concrete support pedestals and the equipment saddle. The holes in the equipment saddle (used to attach to the anchor in the concrete pedestal) are slotted above the slide plate. This reduces the maximum lateral thermal load to a value commonly calculated by multiplying the vertical load at the sliding saddle times the coefficient of friction between the equipment saddle and the slide plate. This load is applied to both the fixed and sliding supports in equal and opposite directions. Alternatively, the maximum forces and moments in the pedestals may be calculated by assuming that the two pedestals deflect laterally by an amount equal to the thermal length change of the equipment. If the pedestals and footings are capable of resisting the forces and moments associated with the lateral thermal length change, the slide plate is sometimes omitted.

Operating loads also commonly include forces caused by thermal expansion (or contraction) of the piping connected to the equipment. The temperature change of connecting piping can produce up to six component reactions at the connecting flanges (three forces and three moments). For large piping, such forces may significantly affect the foundation design.

4.1.1.4 *Wind loads*—Loads due to wind on the surface areas of the equipment and the support foundation are based on the design wind speed for the particular site, and are normally calculated in accordance with the governing local code or standard.

When designing equipment foundations and support structures, most practitioners use the wind load provisions of ASCE/SEI 7. The analytical procedure prescribed by ASCE/SEI 7 provides wind pressures and forces for use in the design of the wind-force-resisting systems and anchorage of equipment components.

Most structural systems involving equipment foundations are relatively stiff (natural frequency in the lateral direction greater than 1 Hz). Consequently, the systems can be treated as rigid with respect to the wind gust effect factor, and a factor of 0.85 as defined in ASCE/SEI 7 can be used. If the equipment is supported on flexible isolators and is exposed to the wind, the rigid assumption may not be reasonable, and more elaborate treatment of the gust effects is necessary as described in ASCE/SEI 7 for flexible structural systems.

Tall, flexible process towers, stacks, and chimneys are susceptible to wind-excited oscillations. The discussion in ACI 307 and ASME STS-1 addresses this issue. Another reference (ASCE 2011) offers recommendations for the types of equipment commonly found in petrochemical plants.

Appropriate consideration of the exposure conditions and topographic, directionality, and importance factors is also required to be consistent with the facility's requirements.

4.1.1.5 *Seismic loads*—While building codes have traditionally focused on the earthquake loading and seismic design requirements for buildings, the requirements for equipment foundations can be significantly different. This is because of differences in life-safety concerns, function and dynamic responses of the foundation, equipment system interaction with piping, and flammable, explosive, or hazardous substances associated with equipment, manufacturing processes, or both.

The FEMA 450 report is a guidance document that defines earthquake loading and seismic design criteria. It contains a section that specifically addresses seismic design and earthquake loading for non-building structures. These provisions have been incorporated in ASCE/SEI 7, in a section for non-building structures, and the "International Building Code" (ICC 2018) incorporated these non-building provisions by reference to ASCE/SEI 7.

Some of the non-building items from ASCE/SEI 7 that are applicable to equipment foundation design include:

1. A list of consensus standards for use when considering seismic design of non-building structures;
2. A list of accepted standards for use when considering seismic design of non-building structures;
3. Guidance on when to consider equipment as an architectural, mechanical, or electrical component or system;

4. Seismic coefficients and height limitations for numerous types of non-building structures; and

5. Earthquake loading and seismic design provisions for numerous types of non-building structures.

The foundation designer should use ASCE/SEI 7 and its referenced standards for applicable seismic design requirements for the equipment foundation.

4.1.1.6 *Test loads*—Most process equipment, such as pressure vessels, is hydro-tested when in place on its foundation. There is also a good possibility that sometime during the life of a vessel it will be altered or repaired; a hydrotest may then be required to meet the requirements of Section VIII of the American Society of Mechanical Engineers (ASME) "Boiler Pressure Vehicle Code" (BPVC). Therefore, it is often considered necessary that all vessels, their skirts or other supports, and their foundations be designed to withstand the hydrotest loads. Testing requirements are specified by the owner, vendor, and/or governing codes for the equipment.

4.1.1.7 *Maintenance and repair loads*—For most heat exchangers, maintenance procedures require that an exchanger's tube bundles be periodically unbolted, pulled from the exchanger shell, and cleaned (Fig. 3.2). The magnitude of the required pulling force that is transmitted to the exchanger foundation can vary over a wide range, depending on several factors. These factors include: 1) the service of the exchanger, including the type of product, the temperatures, and the corrosiveness of the participating fluids; 2) the frequency of the maintenance procedure; and 3) the pulling or jacking procedure actually used. The value used will depend on the engineers' judgment and industry/client/designer practice.

Because the forces transmitted to a foundation from pulling an exchanger bundle vary, the design forces used are often based on past experience. Common criteria are to design for a longitudinal force ranging from 0.5 to 1.5 times the bundle weight. This force is assumed to act at the centerline of an exchanger and is taken in combination only with the exchanger dead (empty) load. For stacked or piggyback exchangers, the bundle pull is assumed to act on only one exchanger at a time.

4.1.1.8 *Fluid surge loads*—Many types of process vessels (such as reactors and catalyst regenerators) are subject to surge forces. Although the analogy may be less than perfect, it is often convenient to describe fluid surge as a coffee-pot effect. The essential mechanism may be similar to the boiling of a contained fluid, with the violent formation and sudden collapse of unstable gas bubbles, currents of merging fluids with fluctuating density, and sloshing of a liquid surface also contributing to the surge forces. These violent forces act erratically, being randomly distributed in both time and space within the liquid phase. Obviously, fluid surge is a dynamic load. Because of the difficulty in defining either the magnitude or the dynamic characteristics of these forces, however, they are almost always treated statically for foundation design.

Surge forces are usually represented as horizontal static forces located at the centroid of the contained liquid. The magnitude of this design force is taken as a fraction of the weight of liquid below a normal operating liquid level. The fraction of liquid weight that is used will vary from 0.1 to 0.5, depending on the type of vessel, on the violence of its contained chemical process, and on the degree of conservatism desired by the owner-operator in resisting such loads. The value used will depend on the engineers' judgment and client/industry/designer practice. For most vessels supported directly on foundations at-grade, surge forces are small, and may be neglected.

4.1.1.9 *Construction loads*—Construction procedures and the erection and setting of equipment frequently cause load conditions on a foundation that will act at no other time during the life of the equipment. For example, before a piece of equipment is grouted into position on its foundation, local bearing stresses under stacks of shims or erection wedges should be checked. Another more specific example is the case of a vertical vessel or stack that may be erected on its foundation before the installation of heavy internals or refractory lining. Once installed, these internals are categorized as part of a vessel's permanent dead load. Many practitioners, however, believe it is necessary to examine the situation that could exist for the interim weeks, or sometimes months, before installation of this considerable internal weight. The design of a tall, vertical vessel foundation may well be governed by overall stability against overturning if it is required that the temporarily light structure be capable of withstanding full design wind (Ratay 2002). Coordination is required between the designer and the constructor to ensure that foundation will withstand the construction loads.

4.1.1.10 *Buoyancy loads*—The buoyant effect of a high groundwater table (water table above the bottom of the foundation) is sometimes considered as a separate load; that is, some engineers treat it as an upward-acting force that may (or may not) act concurrently with other loads under all load conditions. Perhaps, just as frequently, the buoyant effects are treated by considering them as a different condition in which the gravity weight of submerged concrete and soil are changed to reflect their submerged or buoyant densities.

Without addressing the philosophical difference between these two perceptions, the effect is the same. The buoyant effect of a high water table may govern not only the stability (as outlined in Section 4.4), but may also contribute to the critical design forces (moments and shears) used in the design of the foundation.

When it is probable that the elevation of the water table will fluctuate, both dry (neglecting water table) and wet (including the buoyancy effects of a high water table) conditions should be considered when designing foundations.

4.1.1.11 *Miscellaneous loads*—Other types of loads are sometimes defined as separate loadings and are sometimes grouped under one of the categories described previously. Some types of loads are fairly specialized, in that they are normally applied only to certain types of structures or equipment. They include the following:

- *Thermal loads*—Thermal loads are sometimes considered as a separate load category, but are described in Section 4.1.2.3 as operating loads;

- *Impact loads*—Impact loads, such as those due to cranes, hoists, and davits, are sometimes classified separately. Just as often, they are classified as live loads or, depending on the type of equipment, as operating loads;
- *Blast loads*—Explosions and the resulting blast pressure wave represent extreme upset or accident conditions. Normally, blast pressures are only applied to the design of control buildings. Seldom is such a load considered in the design of equipment or foundations, except possibly to set locations so that there is adequate distance between critical equipment and a potential source of such an explosion;
- *Snow or ice loads*—Snow or ice loads may affect the design of access or operating platforms attached to equipment, including their support members. Seldom do they affect the design of equipment foundations, except for electric power distribution structures. Often, snow load is considered as a live load; and
- *Electrical loads*—Impact loads caused by the sudden movements within circuit breakers and load break disconnects may be greater than the dead weight of the equipment. Furthermore, the direction of the load will vary, depending upon whether the breaker is opening or closing. In alternating current devices, short-circuit loads are usually internal to the equipment and have little or no effect on the foundations. In the case of direct current transmission lines in which the earth acts as the reference, however, a short circuit between the aerial conductors and the earth may result in significant loads being applied to the supporting structures.

4.1.2 *Loading conditions*—Different steps in the construction of equipment, or different phases of its operation/maintenance cycle, represent distinct environments, or different "conditions," for such equipment. During each of these conditions, there can be one or several combinations of loads that can, with reasonable probability, act concurrently on the equipment and its foundation. The following loading conditions are often considered during the life of equipment and its foundations.

4.1.2.1 *Construction condition*—The construction condition exists while the equipment or its foundation is still being constructed, and the equipment is being set, aligned, anchored, or grouted into position. In the construction condition, the equipment may be subject to gravity, construction, and reduced environment loads such as wind and earthquake.

4.1.2.2 *Empty condition*—The empty condition will exist after erection is complete, but before charging the equipment with contents or placing it into service. Also, the empty condition will exist at any subsequent time when operating fluid or other contents are removed, the equipment is removed from service, or both. This condition does not usually include the direct effect of maintenance operations. In the empty condition, the equipment may be subject to gravity, and environmental loads such as wind and earthquake.

4.1.2.3 *Operating condition*—The operating condition exists at any time when the equipment is in service, is charged with operating fluid or contents and is about to be placed into service, or is just in the process of being turned off and removed from service. In the operating condition, the equipment may be subject to gravity, thermal, surge, and impact loads, and environmental forces such as wind and earthquake.

4.1.2.4 *Test condition*—This condition exists when equipment is being tested, either to verify its structural integrity, or to verify that it will perform adequately in service. Although the time period actually required for an equipment test is a few days, the test condition may last for several weeks. Thus, it is often assumed that during the test condition, an equipment foundation will be subjected not only to gravity loads (that is, dead load plus the weight of test fluids), but also wind or earthquake. Usually, these loads are taken at reduced intensity. Typical intensities vary from one-quarter to one-half of the wind or earthquake load. Refer to Ratay (2002) for further discussion on this subject.

4.1.2.5 *Maintenance condition*—The maintenance condition exists at any time that the equipment is being drained, cleaned, recharged, repaired, realigned, or the components are being removed or replaced. Loads may result from maintenance equipment, davits or hoists, jacking (such as when exchanger bundles are pulled), impact (such as from the recharging or replacing of catalyst or filter beds), and gravity. The gravity load is assumed to be dead load.

The duration of a maintenance condition is usually short, such as a few days. The probability of maximum wind or seismic loads occurring with maintenance loads is low. Also maintenance typically will not be scheduled or can be suspended during a major event, such as hurricane force winds. Therefore, environmental loads, such as wind and earthquake, are rarely assumed to act during the maintenance condition. Refer to Ratay (2002) for further discussion on short-term events and load probabilities.

4.1.2.6 *Upset condition*—An upset load condition exists at any time that an accident, malfunction, operator error, rupture, or breakage causes equipment or its foundation to be subjected to abnormal or extreme loads. It is often assumed that equipment subjected to severe upset loads may have to be shut down and repaired. Thus, it is not uncommon for the upset loads to be treated as ultimate (unfactored) loads rather than service level (factored) loads.

4.1.3 *Load factors*—Table 4.1 shows the general classification of loads used to determine the applicable load factors in strength design. In considering soil stresses, the normal approach is working stress design without load factors and with overall factors of safety identified as appropriate by geotechnical engineers. Load factor criteria for petrochemical facilities can be found in PIP STC01015.

4.1.4 *Load combinations*—Codes usually specify the more common loadings and combinations that are considered for building design. Industrial equipment, because of the many possible variations in operating loads, can have a far greater number of possible load combinations. Several different load combinations are often possible within a given load condition. Judgment should be used to decide which loads and corresponding load factors can reasonably be expected to act concurrently. In general, ACI 318 load combination factors are used with the load classifications in Table 4.1. With variations among different practitioners, the following

combinations are those most commonly used to design industrial equipment foundations for various load conditions:

1. Construction
   (a) Dead load + construction forces;
   (b) Dead load + construction forces + reduced wind + snow, ice, or rain; and
   (c) Dead load + construction forces + reduced seismic + snow, ice, or rain.
2. Testing
   (a) Dead load + test loads;
   (b) Dead load + test loads + live + snow, ice, or rain; and
   (c) Dead load + test loads + reduced wind + snow, ice, or rain.
3. Empty (shutdown)
   (a) Dead load + maintenance forces + live load + snow, ice, or rain.
4. Normal operation
   (a) Dead load;
   (b) Dead load + thermal load + machine forces + live loads + wind + snow, ice, or rain; and
   (c) Dead load + thermal load + machine forces + seismic + snow, ice, or rain.
5. Abnormal operation
   (a) Dead load + upset (abnormal) machine loads + live + reduced wind.

It is common to only use some fraction of full wind, such as 80% in combination with construction loads and 33% for test loads, due to the short duration of these conditions. Additional guidance is provided by Ratay (2002).

4.2—Design strength/stresses

In the design of foundations, forces and stresses in the various elements should be calculated and compared with acceptance criteria. Some types of acceptance criteria are expressed in terms of allowable stress to which a calculated service load stress is compared. Other criteria are expressed in terms of a design strength to which factored loads are compared. For many of the elements of equipment foundations, there is neither a published standard nor a clear consensus as to which type of criteria is appropriate.

Allowable soil pressures, anchor bolt stresses (tension, shear, or bond), concrete bearing stress, and the required development length of pedestal reinforcement that lap splices to anchor bolts are some of those for which variations in practice are common.

In addition to the variations between the practices used by different engineers, a second major variance is that different acceptance criteria are often used for adjacent or interacting elements. This leads to interface problems and inconsistencies in the logic of the design of the various elements. At the very least, the existence of different types of acceptance criteria for various elements presents a tedious bookkeeping problem.

The following sections describe the individual elements and the state of practice in defining acceptance criteria for use in their design.

4.2.1 *Concrete*—The foundation should be designed to withstand applied loads and act in unison with the equipment

Table 4.1—Load classifications for strength design

Design loads	Load classification
Weight of structure, equipment, internals, insulation, and platforms	Dead
Temporary loads and forces caused by construction	
Thermal loads	
Anchor and guide loads	
Operating and test loads due to fluids with well-defined pressures and maximum heights	Fluid
Platform and walkway loads	Live
Materials to be temporarily stored during maintenance	
Materials normally stored during operation such as tools and maintenance equipment	
Vibrating equipment forces	
Impact loads for hoist and equipment utilities	
Earthquake loads	Environmental
Transportation loads	
Snow, ice, or rain loads	
Wind loads	

and supporting soil or structure to meet any design criteria specified by the equipment manufacturer or equipment owner.

The service life of a concrete foundation should meet or exceed the anticipated service life of the equipment installed. Cracking should be minimized to ensure protection of reinforcing steel.

The structural design of all reinforced concrete foundations should be in accordance with ACI 318.

In foundations thicker than 4 ft (1.2 m), the engineer should follow the recommendations suggested in ACI 350 and ACI 207.2R (latest versions) to prevent cracking due to thermal or volume changes.

Chapter 4 of API RP686 includes design criteria for soil-supported reinforced concrete foundations that support general and special purpose machinery. Concrete used in the foundation should tolerate its environment during placement, curing, and service. The engineer should consider various exposures, such as freezing and thawing, salts of chlorides and sulfates, sulfate soils, acids, carbonation, repeated wetting and drying, oils, and high temperatures (ACI 201.2R).

To mitigate the risks of damaging concrete from these types of exposure, there are many technologies available, such as chemical admixtures, additives, specialty cements, and preblended products, to help improve placement, durability, and performance properties (ACI 201.2R). These technologies include items such as water-reducing admixtures, set-controlling admixtures, shrinkage-reducing admixtures, polymers, silica fumes, fly ash, slag cement, and fibers.

Many foundations, whether new or repaired, require a fast turnaround to increase production by reducing downtime without compromising durability and required strength. These systems may use a combination of preblended or field-mixed concrete and polymer concrete or grout to reduce downtime to 12 to 72 hours, depending on foundation volume and start-up strength requirements.

4.2.2 *Reinforcement*

4.2.2.1 *Vertical reinforcement*—The vertical reinforcement in foundation pedestals is, for most types of equipment,

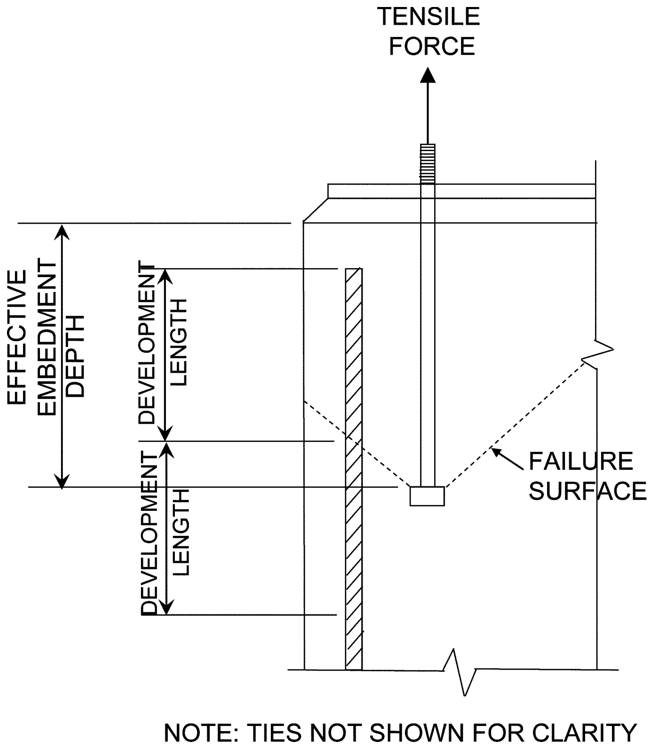


Fig. 4.1—Reinforcement development length into breakout prism.

designed as an integral part of the total concrete section, that is, by treating the pedestal and its reinforcement as a beam-column. For this approach, ACI 318 design criteria are employed, except as discussed below, with the exception of Section 16.3.4.1 (ACI 318) requirements for minimum reinforcement. It is not economical to reduce reinforcement at the foundation-column interface. The additional reinforcement for the splice requirements would be greater than the savings in reducing the reinforcement. For pedestals with a height-to-least-lateral dimension ratio of 3 or greater, the required reinforcement should not be less than minimum reinforcement applicable to columns (Sections 10.3.1.2 and 10.6.1.1 of ACI 318). Pedestals with a dimension ratio less than 3 do not meet the definition for a column, but most practitioners would still apply the minimum reinforcement provisions. For equipment such as tall, vertical vessels with large overturning forces, the vertical pedestal bars must transfer the anchor bolt tensile forces from the pedestal to the footing or pile cap (Fig. 4.1). In this situation, practice for defining the appropriate acceptance criteria for designing the vertical bars varies widely.

Some engineers design the pedestal vertical reinforcement based on using the total concrete cross-sectional area as described previously. Some use a practice where the vertical reinforcement is sized to either resist the calculated anchor bolt tensile forces, or to match the design capacity of the anchor bolts, neglecting any concrete tensile strength. In either case, the pedestal reinforcement must be developed appropriately into the breakout prism of the anchor. Chapter 17 of ACI 318 provides guidance on anchor reinforcement proportioned to transfer the full force from the anchor to the supporting member or, in this case, the foundation. Chapter 17 (ACI 318) also provides guidance on spacing and configuration of this type of reinforcement.

Other practitioners proportion the vertical reinforcement to provide a yield strength greater than or equal to the yield strength provided by the equipment anchor bolts, again without including any concrete resistance. This latter practice is used primarily in seismically active areas. The rationale for this practice is that initial yielding should take place in the more visible anchor bolt before the reinforcement to which the primary anchorage forces should be transferred (Housner 1956; Scholl et al. 1978).

4.2.2.2 *Horizontal reinforcement*—For small pedestals, or where the governing loads are primarily compression, the horizontal reinforcement in pedestals is commonly sized in accordance with ACI 318 criteria for column ties. There are a number of circumstances, however, where other types of criteria are used.

One example occurs in the case of pedestals with a large area, such as for vertical vessels and stacks. In this case, the vertical reinforcement is usually designed to resist tension. The horizontal reinforcement in the pedestal faces may be essentially nominal—perhaps just to keep the vertical bars in place during the concrete placement. Sometimes, a minimum reinforcement criterion for bars in faces of mass concrete, such as suggested in ACI 207.2R-07 (ACI Committee 207 2007), is used. Larger-size reinforcement and/or lesser spacing than defined by such minimum criterion may be provided for confinement of the anchor bolts and to preclude spalling at the pedestal face (Breen 1966; Lee and Breen 1966).

In addition to the main horizontal reinforcement provided in the face of vertical vessel pedestals, many practitioners consider it good practice to provide a group of two to four tie-bars, closely spaced at 3 to 4 in. (75 to 100 mm), near the top of the pedestal. This closely spaced top set of peripheral reinforcement assists in resisting cracking due to edge bearing on the pedestal or thermal expansion. This practice can slightly improve the resistance to cracking of the concrete near the top of the pedestal due to transfer of shear forces through the anchor bolts into the concrete (ACI 318, Chapter 17).

Pedestal size is normally dictated by equipment dimension and not by load requirements. This results in small shear forces that can be resisted by the concrete interface (friction) alone. Therefore, shear reinforcement is generally not required. In cases where shear loads exceed the concrete shear strength, closed ties or hoops shall be included in the pier design. ACI 318, Section 16.3, should be followed when equipment piers are supported by larger foundations. Nominal shear reinforcement should be used to transfer lateral forces, ensure ductility, and provide a method to secure vertical reinforcement in piers.

Horizontal reinforcement is sometimes provided in the tops of pedestals. For example, reinforcement may occasionally be required by stress calculations for relatively large, thin, or shallow pedestals (which are essentially as large as the pad), where downward load is applied at the edge or periphery of

the pedestal. In this situation, the center portion of the pedestal could potentially arch upward, resulting in tension at the top of the pedestal.

A few practitioners provide horizontal reinforcement in the top of pedestals for equipment as a matter of good practice, particularly where the equipment operates at elevated temperatures. Reinforcement congestion, however, can lead to construction problems. Engineers should review the final design to ensure that it is a constructible design.

ACI 318 criteria are used for the design of flexural reinforcement in footings or pile caps. Questions that arise about these criteria are generally concerned with minimum amounts of reinforcement, as discussed in Section 5.7.6 of this report.

4.2.3 *Anchorage*—Anchorage of equipment to its foundation is often the most critical aspect of a foundation design. This is particularly true for vertical vessel and stack foundations or for any equipment foundation where consideration of lateral loads dominates the design.

Anchorage criteria are provided in ACI 318 for buildings and most industrial applications, and in ACI 349, Appendix D, for nuclear power plants. ACI 318, Chapter 17 formulas for computing anchor design strength are based on concrete breakout as described by the concrete capacity design (CCD) method (Fuchs et al. 1995), which was an adaptation of the κ method (Eligehausen et al. 1987, 1988). The strength calculations from these methods are based on a model with a breakout prism angle of approximately 35 degrees measured from a plane perpendicular to the longitudinal axis of the anchor.

Equipment foundations are typically reinforced, and many designers conservatively transfer the large anchor forces from the equipment directly into the foundation reinforcement, especially in cases where vibration is present. Using this method may require embedment lengths of the anchors to be longer than the length determined from equations of ACI 318, Chapter 17. This extra length is required to provide adequate development length of the foundation reinforcement into the breakout prism to transfer the anchor force (Fig. 4.1). ACI 318, Chapter 17 provides guidance on this type of design (referred to as anchor reinforcement).

Anchors can be either cast-in or post-installed. Post-installed anchors include mechanical types, such as undercut and expansion anchors, or bonded types, referred to as adhesive and grouted anchors.

An undercut anchor transfers tensile load to the concrete through bearing of an expansive device against a bell-shaped enlargement of the hole at the base of the anchor.

An adhesive anchor consists of a threaded or deformed rod installed in a hole with a diameter in accordance with adhesive anchor manufacturers' recommendations. The hole is filled with a structural adhesive. Adhesive anchors transfer tensile load to the concrete by bond between the anchor and the adhesive, and bond between the adhesive and the concrete.

A grouted anchor generally consists of a headed anchor installed per the recommendations of the grout manufacturer. The hole is filled with a non-shrink grout, usually containing portland cement, sand, and various chemicals to reduce shrinkage. Grouted anchors transfer tensile load to the concrete by bearing on the anchor head, and by bond and or friction along the grout-concrete interface.

Expansion anchors transfer tension load to the concrete by direct bearing or friction, or both, between the anchor and the hardened concrete. Expansion anchors may be torque-controlled, where the expansion is achieved by a torque acting on the screw or bolt, or displacement-controlled, where the expansion is achieved by impact forces acting on a sleeve or plug and the expansion is controlled by the length of travel of the sleeve or plug.

The selection of a post-installed anchor depends on its use and type of exposure, such as temperature, moisture, vibration, and possible chemical spills. The manufacturer should provide the required information to suit specific needs. Expansive-type, post-installed anchors are generally used for smaller loads that are not subject to vibration. This type of anchor, however, may be qualified for dynamic loads using the testing protocol specified in ACI 355.2.

A cast-in-place anchor is cast into the fresh concrete. The tensile load is transferred to the concrete through bearing on the head of the embedded anchor. ACI 318, Chapter 17 provides equations for computing the capacity of cast-in-place anchors including J-bolts and L-bolts. The capacity of J-bolt and L-bolt anchors, when compared with capacities of headed cast-in-place bolts, is considerably less due a much smaller stress allowed on the bearing component of the projected surface of the L or J portion of the bolt and neglecting any frictional or bond component.

4.2.3.1 *Design strength*—The design strengths for post-installed anchors are based on the results of tests conducted by the manufacturer of the particular anchor. In the past, safety factors of 4 to 5 relative to concrete breakout or pullout were used to determine an allowable load for post-installed anchors. ACI 318, Chapter 17 provides equations for computing the capacity of post-installed anchors which have been prequalified by testing (Section 5.2.1).

Cast-in-place anchor bolts are usually designed to develop applied tensile and shear forces. The design practices that are used to ensure adequate anchorage are described in Section 4.2.3.2. Anchor bolts for equipment are commonly designed using the corroded tensile-stress area of the threaded bolt stock or, in other words, the amount of material remaining after corrosion has occurred. For galvanized bolts, a corrosion reduction is not applied. ASME B1.1 defines the tensile-stress area for bolts, A_{se}, as follows

$$A_{se} = 0.7854\left(d_o - \frac{0.9743}{n_t}\right)^2 \qquad (4\text{-}1)$$

where d_o is the nominal bolt diameter, in. (mm); and n_t is the reciprocal of the thread pitch, threads per in. (threads per mm).

(Note: When calculating A_{se}, the assumed thread type should be indicated: unified coarse thread series (UNC) or 8-thread series (8UN). ASTM F1554-17, Section 11.1.1, states that "unless otherwise specified, uncoated threads shall be unified coarse thread series as specified in the latest

issue of ANSI/ASME B1.1." If 8UN threads are specified, the anchor rod diameter must be greater than 1 in. [25 mm].)

A corrosion allowance may be added to the calculated required bolt diameter d_o in Eq. (4-1). It will vary with both location (seacoast versus inland) and the possibility of spills of acids or other chemicals. Such values commonly range from 1/16 to 1/4 in. (2 to 6 mm).

The AISC design specification (AISC 360) permits stresses to be calculated on the nominal body or shank area of bolts and threaded parts (AISC 360, Section J.3). Designers of equipment foundations, however, may prefer greater conservatism in anchor bolt design than that used in the design of other foundation components. ACI 318, Chapter 17, equations use the effective cross-sectional area A_{se} for threaded bolts computed according to Eq. (4-1). Any conservatism expected and incorporated into the steel design of the anchor will be of no value unless the concrete tensile and concrete shear capacity of the anchor is of equal value.

Occasionally, high-strength bolt material is used in the design of anchor bolts for equipment foundations. The premium costs for the high-strength material (generally greater than 36 ksi [250 MPa]) and the additional design of the anchor attachment to the equipment to resist a greater load make their use the exception.

For example, if high-strength anchor bolt material is used, a special design of the equipment's anchor bolt lugs might have to be performed to allow higher loads on the equipment lugs. Any special design that requires a change to an equipment manufacturer's standard base detail may cost more in extras than any nominal savings afforded by a more efficient bolt pattern.

A ductile anchor is an anchor sufficiently embedded so that failure is governed by yielding and fracture, or both, in the ductile steel bolt material, and not governed by concrete failure modes. ACI 318, Chapter 2 provides a definition of ductile steel element. Using high-strength bolts in a ductile design will generally require more embedment in concrete to ensure that the anchor design is governed by the tensile and/or shear strength capacity of the ductile steel element.

4.2.3.2 *Anchorage criteria*—The behavior of anchors depends on a number of variables, including:

- Loading (axial load, moment, and shear);
- Size of the steel attachment;
- Size, number, location, and type of anchors;
- Coefficient of friction between the base plate and the concrete;
- Tension-shear interaction for a single anchor;
- Distribution of shear among the anchors and eccentricity of this force;
- Distribution of tension among the anchors and eccentricity of this force;
- Flexibility of the base plate;
- Concrete compressive strength;
- Base plate configuration (embedded, flush, or on a raised grout pad; important for anchorages subject to shear forces);
- Reinforcement in the foundation or pier;
- Embedment length;

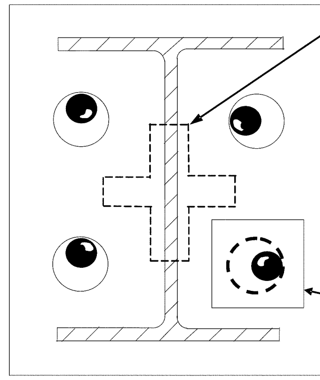


Fig. 4.2—Base plate with oversize anchor bolt holes.

- Edge distance and anchor spacing; and
- Bolt head bearing surface area.

The preferred anchor configuration is either a headed bolt or a threaded rod with a bearing plate or a nut. Standard dimensions for bolt heads are given in ASME B18.2.1. Standard dimensions for nuts are given in ASME B18.2.2. Bearing capacity evaluation at the anchor head is required by ACI 318, Chapter 17, to ensure that the concrete is not crushed under the head, which can initiate pullout failure.

There are three methods of shear transfer:

- *Anchor bolt bearing*—Where the base plates are mounted flush or above the concrete surface, the dominant mechanism of shear transfer is bearing on the anchor. Because the holes in the base plate are usually oversized according to AISC recommendations (AISC 360), there is a question of how the plate goes into bearing against the anchor and how many anchors will actually transfer the load. Some engineers assume that only half of the anchors actually transfer the shear load. Others use no more than two bolts for shear transfer. In some cases, loose plate material with holes 1/16 in. (2 mm) greater than the anchor diameter is bolted under the washer and nut and field welded to the column base to transfer the shear forces (Fig. 4.2).
- *Shear friction*—For embedded plates with tension anchors, shear transfer similar to the mechanism described in ACI 318, Section 22.9, may be used. A friction force is generated between the base plate and concrete due to external loads that provide compression and any confinement provided by the tension anchors. The tension anchor forces determined from Section 22.9 calculations must be added to the tension forces from external loads in design of the anchors to maintain the shear friction force. ACI 349 Appendix D, Section 11.0, provides additional guidance and friction factors for this approach.
- *Shear lugs and embedded plates*—If confinement of a connection is provided by anchors in tension, in combination with external loads acting on potential shear planes as described above, the combination of bearing on the side of the embedded base plate and or shear lugs and the direct shear strength of the concrete and grout may be used.

The concrete breakout strength for a headed bolt (or bolt with a nut) is calculated in ACI 318, Chapter 17, assuming a frustum of an approximate 35-degree (measured from an axis perpendicular to the axis of the anchor) breakout cone emanating from the anchor head to the free concrete surface a distance of $1.5h_{ef}$ in each direction from the centerline of the anchor. The term h_{ef} is the effective anchor embedment depth. From extensive testing of anchors in the development of Chapter 17, the basic concrete breakout strength N_b for a single anchor in tension in cracked concrete is determined from the expression

$$N_b = k_C \lambda_a \sqrt{f_c'}\, h_{ef}^{1.5} \quad (4\text{-}2)$$

where k_C is the coefficient for basic concrete breakout strength in tension; λ_a is a modification factor to reflect properties of lightweight concrete relative to normalweight concrete of the same compressive strength; h_{ef} is the effective embedment depth of anchor, in. (mm); and f_c' is the specified compressive strength of concrete, psi (MPa) (refer to ACI 318, Section 17.4.2.2).

This basic breakout strength is multiplied by modification factors to account for items such as edge distances, minimum spacing, and load eccentricity. The anchor design strength is checked using formulas for other modes of failure that may control, such as anchor pullout in tension, concrete breakout strength in shear, concrete side-face blowout strength in tension, and concrete pryout strength of the anchor in shear. In addition, the anchors must be located in concrete members with sufficient thicknesses to preclude splitting failure. Anchors that are torqued during installation or during attachment of the equipment in shallow or thin sections of concrete can cause splitting of the surrounding concrete.

Anchors that are designed to transfer large forces into the foundation system may be required to transfer these forces into the foundation reinforcement, especially for large-diameter anchors that fall outside the current range of test data used in the development of Chapter 17. This type of transfer can be accomplished by ensuring that the reinforcement is developed on both sides of the failure surface (Fig. 4.1). Hoops or ties that enclose the anchor and reinforcement are used to confine the concrete in the transfer region.

4.2.3.3 *Anchor bolt sleeves*—The practice of using anchor bolt sleeves varies. Some practitioners do not use sleeves in the foundations for static equipment, whereas others insist on using a variety of types and configurations.

Sleeves may be constructed of pipe, sheet metal, high-density polyethylene, or a hole formed using Styrofoam. Care should be exercised in cold climates to avoid water freezing inside the sleeves and spalling the surrounding concrete. After installation of the equipment, the sleeves are usually filled with grout. Some engineers, however, particularly those who use tensioned bolts, specify a bond-breaker such as grease- or mastic-type filler, for the sleeve. Having the full length of the bolt under the tensioned load decreases the amount of relaxation of the load over time. This occurs because the force is transmitted down to the head of the bolt rather than to the concrete near the surface, as is the case for a bonded bolt. The transfer of force at the lower depth reduces the relaxation because the effects of creep in the concrete over time are less. A good rule of thumb is to provide an unbonded length of at least 10 to 12 times the bolt diameter. Many applications use a much longer unbonded length.

Sleeves are often used to allow for minor bolt adjustments when equipment mounting holes are only slightly larger than the bolt diameter. Using anchor bolt sleeves for improper bolt alignment is a bad practice when the bolt has to be bent; this can bind the bolt and inhibit free elongation of the bolt. Proper anchor bolt setting, using a template, is specified by most engineers with typical tolerances of 1/8 in. (3 mm) or as required to match the difference between the base plate hole and the bolt diameter. Canister-type anchor bolt devices are available to allow for lateral adjustment and free-elongation.

4.2.4 *Soil*—The procedures for determining allowable soil pressures or pile capacities are beyond the scope of this report. These allowable pressures and capacities are usually established by a geotechnical consultant using standard procedures (not unique to equipment foundations). Besides settlement considerations, however, allowable vertical soil pressures or pile loads are also limited by dividing a nominal capacity by a safety factor that ranges from 2 to 5, depending primarily on the soil type and the type of loading (temporary or sustained) (Bowles 1995).

Criteria for the lateral resistance of soil will vary with the type of foundation as well as the type of soil. For most shallow spread footings that are excavated, formed, placed, and backfilled, passive soil pressures are neglected. Resistance to lateral loads is usually presumed to be a result of bottom friction alone. This is mainly because of uncertainty regarding the quality of the backfill material and the control of its placement. Some geotechnical engineers, however, include the lateral resistance of passive pressures to a certain degree, consistent with allowable lateral movement, if a certain depth of backfill from finish grade is ignored in the calculation.

Lateral resistance of pile foundations is often determined using the lateral resistance of the piles only. In these instances, the resistance contributed by passive soil pressure acting on the sides of the pile cap is ignored. If lateral displacements of the pile foundations become large (flexible piles), passive soil-resistance may be included in the design. Alternatively, if there is adequate space available, battered piles may be used to resist lateral loads.

Drilled caissons are often designed using horizontal soil pressures to resist horizontal shears at the top of the foundation, as well as overturning moments. The allowable lateral pressure is usually deduced from a permitted lateral displacement at the top of the foundation. The procedure may range from directly assuming a soil pressure profile to a complex caisson-soil interaction analysis.

4.3—Stiffness/deflections

Criteria for stiffness or allowable deflections for foundations supporting static equipment vary widely depending on the particular application. For many applications, there are no special requirements other than engineering judgment. For others, deflections may need to be tightly controlled.

Differential settlement or lateral movement between adjacent pieces of equipment that are connected by piping, duct work, chutes, or conveyor belts may have to be controlled to avoid overstressing the piping or misaligning the belts or chutes. Some types of vessels may be serviced by piping that is glass- or ceramic-lined. Tolerable displacements for such fragile items may be as low as a few hundredths of an inch (0.01 in. = 0.25 mm). Some equipment may require precise alignment for its proper operation. As a rough order of magnitude, however, long-term settlements of 1/2 in. (13 mm), or short-term lateral movements (such as under wind load) of 1/4 in. (6 mm) are usually suitable for most noncritical static equipment.

For some applications, flexibility rather than stiffness (or rigidity) is the desired result. Foundations that support equipment connected to high-temperature piping, or that support opposite ends of a horizontal vessel or heat exchanger subject to thermal growth, will have substantially reduced forces if they possess even modest flexibility.

4.4—Stability

In addition to soil bearing and settlement, stability should also be checked to determine a minimum foundation size. Stability checks should be made, as applicable, for sliding and overturning.

Sliding stability may be of concern for foundations on relatively weak soils supporting equipment subjected to large lateral forces. Such situations may include deadmen, retaining walls, or exchangers subject to bundle pull. Sliding stability is usually checked by verifying that lateral forces are less than allowable base friction or adhesion, plus passive pressure.

Overturning stability criteria will frequently control in the design of foundations with high allowable soil pressures, or in the design of foundations for tall equipment subjected to large wind or seismic loads. The size of foundations for tall vessels and stacks is commonly controlled by overturning. A stability ratio is used to characterize a foundation's resistance to overturning. It is defined as the resisting moment divided by the overturning moment. Moments are computed at the bottom edge of a spread footing. The resisting moment includes the permanent weight of the equipment, foundation, and soil overburden. Working levels, or service loads, are used in the computation.

For foundations supporting an entire piece of equipment, such as a vertical vessel on a spread footing or a heater supported on a combined mat, the stability ratio may be simplified to the following formula

$$\text{stability ratio} = \frac{PD/2}{M} = \frac{D}{2e} \tag{4-3}$$

where P is the vertical load due to weight of concrete and equipment and soil overburden, kip (kN); M is the overturning moment applied to footing or pier, ft·kip (kN·m); D is the edge to-edge distance of footing in direction of overturning moment, ft (m); and $e = M/P$, ft (m).

Alternative equations in use are based on a required footing area in compression with the soil. A stability ratio of 1.5 equates to half the footing area in compression.

For several isolated foundations supporting a single piece of equipment, such as a heater, a portion of the overturning moment can be resisted by vertical forces at each footing. The stability ratio is then determined by computing moments about one edge of each footing due to dead weight and wind or seismic loading at each footing.

Some practitioners include soil failure considerations in accordance with published design recommendations of ACI 336.2R for combined foundations and mats, and 336.3R for drilled piers. For individual foundations, the simpler methods described previously are generally used as an indication of the factor of safety against overturning. Sometimes, the simpler methods are used for combined foundations.

Although the concept of a stability ratio is straightforward, there is a wide range of minimum required values. Some of the more conservative practitioners require that the full base of a foundation remain in compression and, thus, imply a stability ratio of 3.0 to 3.75, depending on the footing geometry. Some engineers require that the stability ratio be not less than 2.0, but many permit a stability ratio of 1.5. A ratio of 1.5 is the lowest value that is commonly accepted. ASCE/SEI 7 includes an allowable-stress load combination with reduced dead load that implies a stability ratio of 1.67.

For pile foundations, the concept of a stability ratio is straightforward where the piles are not designed to resist uplift. The center of rotation is taken at top of the pile furthest from and in the direction of the applied lateral load. When the piles have a tension capacity, however, the concept becomes ambiguous and is seldom used. In this situation, the actual design loads are transmitted along load paths directly through structural elements (bolts, foundations, pile tension devices, piles) designed for this purpose and capable of carrying the loads to the supporting soil or rock. The stability of the structure is not dependent solely upon the dead weight of the structure, soil weight, and width of the footing to resist overturning.

For drilled pier foundations, the procedure for using the stability ratio is unclear, and many different practices prevail. For example, because a drilled pier can mobilize lateral passive soil pressure to resist overturning, a stability ratio might be defined by either of the following two formulas

$$SR_1 = \frac{PD/2 + M_p}{M} \tag{4-4}$$

or

$$SR_2 = \frac{PD/2}{M - M_p} \tag{4-5}$$

where M_p is the resistance to overturning provided by the lateral passive soil pressure, and the center of moments is again at the toe of the drilled pier base. The first stability ratio formula would be more meaningful for a drilled pier (particularly a straight shaft) whose diameter is relatively small compared with its depth, and that relies predominantly on the lateral soil pressure (pole action) for its resistance to overturning. The second formula, however, might be appropriate

for a large-diameter shallow drilled pier whose major resistance to overturning is the size and weight of the bell.

CHAPTER 5—DESIGN METHODS

5.1—Available methods

Foundations for static equipment are generally designed by the strength design method as defined by ACI 318. This method allows the use of unfactored (service) loads to select the base area of the foundation or the pile or pier arrangement. Foundation proportions and reinforcing requirements are then selected based on factored loads.

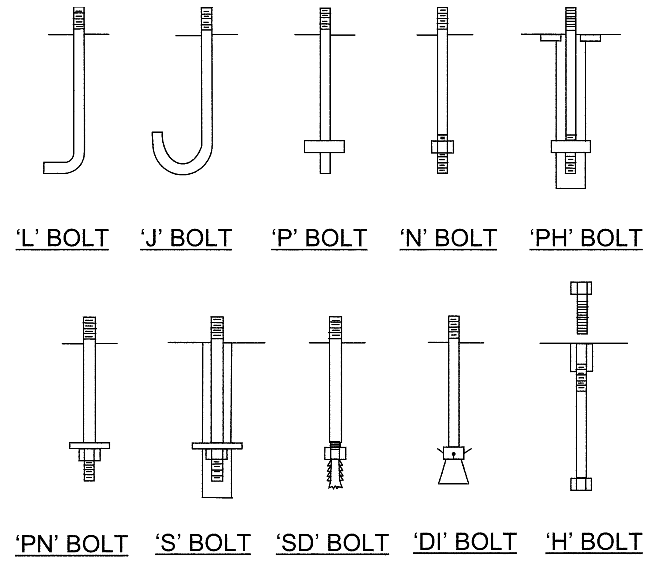
5.2—Anchor bolts and shear devices

5.2.1 *General considerations*—Forces produced by wind, seismic, thermal, and other sources must be transferred through the static equipment into the supporting foundation. Typical anchorages consist of anchor bolts that transfer tensile forces or a combination of tensile and shearing forces. When required, shear lugs may be used to transfer shear forces.

The capacity of anchor bolts may be governed by the steel strength of the anchor in tension and/or shear or by the concrete breakout strength and/or other modes of concrete failure. Splitting of thin concrete members when bolts are tensioned may also control the strength.

ACI 318, Chapter 17, is used to determine the strengths of most bolts including post-installed anchors. The current body of test data that was used to develop the concrete breakout strength capacities is based on anchors with diameters not exceeding 2 in. (51 mm) and tensile embedment not exceeding 25 in. (640 mm) in depth. Recent test programs (Lee et al. 2007) have shown that for large anchors near an edge, the concrete breakout strength in shear is considerably less than the values determined by the equations of Chapter 17. Although larger-diameter, longer anchor bolts fall outside the range of variables considered in the development of current design provisions, work is underway to include them in future editions of ACI 318 as additional test results become available. Good engineering details and judgment should be employed to ensure that all anchors are well developed in the concrete to transfer the loads into the reinforcement. When large or small anchors are near an edge, supplemental reinforcement can be placed to prevent side-face blowout or concrete breakout from shear.

In ACI 318-05 (ACI Committee 318 2005), Appendix D, construction in regions of moderate or high seismic risk required the capacity of anchors be controlled by the tensile or shear strength of the steel (ductile behavior). Non-ductile concrete failure modes were not allowed. ACI 318-08 (ACI Committee 318 2008), Appendix D, provisions were updated to allow non-ductile failure for Seismic Design Categories C, D, E, and F using an additional large reduction factor applied to the calculated design strength. The reduction factor that is applied to the anchor strengths (controlled by concrete failure) reduces the anchor strength to the point where ductile design is encouraged. One reason for this reduction is that the strength design seismic loads calculated in accordance with codes, such as the IBC (ICC 2006), permit reduced loads based on inelastic response of the structure. ACI



Note: Other anchor bolt types that are not shown may be available.

P : Plate *or* Plate washer
N : Nut
PN : Plate and Nut *or* Plate washer and Nut
S : Sleeve
SD : Self-drilling
DI : Drop-in or undercut
H : Hidden

Fig. 5.1—Anchor bolt types.

318, Appendix D, allows the use of other attachments external to the surface of the concrete connecting the anchor to the equipment to be the strength-controlling ductile element that undergoes ductile yielding rather than the anchor.

ACI 349, Appendix D, has notable differences with regard to seismic loads. ACI 349 specifies that seismic design loads be derived from an elastic analysis without any reduction factors due to energy absorption devices or inelastic structural responses. ACI 349, developed for nuclear structures, prefers that all anchors be designed with embedments that result in the design strength being controlled by the strength of the anchor steel rather than the strength of the concrete. However, nonductile anchor design is permitted in ACI 349 using an additional reduction factor. The use of J-bolts or L-bolts (Fig. 5.1) is prohibited when using the provisions of ACI 349, Appendix D, due to their inherent inability to develop the steel capacity of the anchors that is encouraged in Appendix D to meet ductility requirements deficiencies.

For anchorage design in seismic area, refer to ACI 318, Section 17.2.3, and/or ACI 349, Appendix D.

ACI 318, Chapter 17, includes provisions for determining the capacity of post-installed anchors that meet the assessment requirements of ACI 355.2 or ACI 355.4. The suitability of post-installed anchors should be demonstrated by the ACI 355.2 or ACI 355.4 prequalification tests and be assigned to a Category 1, 2, or, 3. Chapter 17 allows the use of qualified post-installed anchors in areas of seismic risk if the anchors have passed the Simulated Seismic Tests of

ACI 355.2 or ACI 355.4 and are designed to resist the required strength.

5.2.2 *Tension*—Anchor bolts are provided primarily to transfer tensile forces. They consist of several different types and generally fall into one of the categories shown in Fig. 5.1. In general, L- and J-bolts are not adequate for use in seismic regions due to their inherent inability to develop the steel capacity of the anchors that is recommended in ACI 318, Chapter 17 to meet ductility requirements.

Types P, N, H, PN, PH, and S, are cast-in bolts and generally rely on the breakout strength of the concrete. Types SD and DI are, respectively, self-drilling and drop-in post-installed bolts relying on expansive forces to transfer the tension to the concrete or to the mechanical anchorage.

The provisions of ACI 318, Chapter 17 apply to headed bolts and post-installed anchors that have been qualified for its use. Specialty anchors and adjustment sleeves are not currently covered by these provisions due to the difficulty in prescribing qualification tests and design equations for the wide variety of anchors available for use. Manufacturers' test data and good engineering judgment must be relied upon when using these devices

The various bolt types are generally carbon steel or low-alloy materials and may be provided with sleeves. A common material specification now used for anchor bolts is ASTM F1554, which replaces the older ASTM A307 material specification. High-strength anchor material may be used, except that these types of anchors will require greater embedment and development to satisfy the ductility requirements in high seismic regions.

Type S open-sleeve anchor bolts may be tensioned to ensure residual bolt stress. After the bolt is tensioned against the hardened concrete, grout may be added to provide shear resistance if this is a requirement unless other means are provided.

Tensioning equipment anchor bolts requires special attention. Generally, the allowable tensioning force of a bolt is based on a percentage of yield strength times the bolt area. Tensioning forces are measured in terms of applied torque or by directly measuring the tension. The commonly used anchor material that meets the requirements of ASTM A307 does not exhibit a well-defined minimum yield strength. These types of bolts are generally not tensioned to a specified amount. The bolt material specified in ASTM F1554 includes three grades of material with specified minimum yield strengths that may be torqued to a specified percentage of yield strength. The lowest grade, 36 (36 ksi [250 MPa] minimum yield strength), is usually not the material specified when designing tensioned anchorages because it will not be capable of high clamping forces compared with higher-strength materials such as Grades 55 and 105 (55 and 105 ksi [380 and 725 MPa], respectively).

The use of torque to induce tensile forces in the anchor may be used to resist the repeated tension in tall structures, such as those supporting wind turbines. The use of tensioning in anchors provides a clamping force to prevent movement, slip, or lift-up in a manner similar to slip-critical steel moment-resisting connections. Anchors that are tensioned experience relaxation over time due to the creep of the concrete in the area of the interface between the anchor and the concrete. Headed anchors with their shafts purposely debonded for tensioning have less relaxation due to the load transfer being located deep in the concrete at the head, rather than in the shallow region near the surface of the concrete for bonded anchors. Longer, deeper embedded anchor bolts engage more concrete in the compression zone between the bearing head area and the equipment being supported, further reducing relaxation and the potential for cracking.

The design of anchor bolts is a multi-step procedure. The tensile forces in the assumed bolt pattern are computed. Thereafter, the bolt area and embedment are determined, followed by considerations for edge distance and spacing.

The following bolt force formula (Gaylord et al.1997) has been used to compute the maximum bolt force F for anchors arranged in a circle. Such a formula (Eq. (5-1)) is easy to apply and is always conservative

$$F = \frac{4M}{Nd_{bc}} - \frac{W'}{N} \tag{5-1}$$

where W' is the weight of equipment, kip (kN); N is the number of bolts; M is the moment applied to anchorage, ft·kip (kN·m); and d_{bc} is the diameter of bolt circle, ft (m).

For high-profile static equipment (height/diameter > 7), such as tall vessels and stacks, the proper determination of anchor bolt forces is a primary concern of the foundation design. For self-supporting steel stacks an alternative method (Troitsky 1982) exists for calculating anchor bolt tension for anchors arranged in a circle and subjected to an overturning moment. With this method, the discrete point forces in the anchor bolts are replaced with an annular ring of steel located at the bolt circle centerline (Fig. 5.2). The compression force from the stack moment and weight is resisted by compression under the bearing plate. The steel portion of the ring resists the tension force (uplift). A series of trials is made to confirm the location of the neutral axis to satisfy equilibrium. This method is generally considered more accurate compared with the results computed using the force formula (Eq. (5-1)), but requires more time and effort to apply. This method assumes a rigid concrete foundation. The steel area of the bolt may be determined following the criteria given in Section 4.2.3. For the bolt type selected, the embedment is computed to satisfy anchor capacity requirements or to transfer the tensile forces to vertical reinforcement when bolts are cast in pedestals. When vertical dowels are used to transfer tensile forces to foundations, care should be exercised to assure that sufficient development length of the pedestal reinforcement beyond the concrete breakout prism, as shown in Fig. 4.1, is provided. Also, the spacing between the anchor reinforcement and the anchor should be examined to ensure good transfer. Section 25.4 of ACI 318 should be used in determining development length for vertical bars in pedestals.

5.2.3 *Shear*—Shear forces may be transferred by a variety of mechanisms: friction resulting from dead load, moment compression, and the clamping action provided as a result of

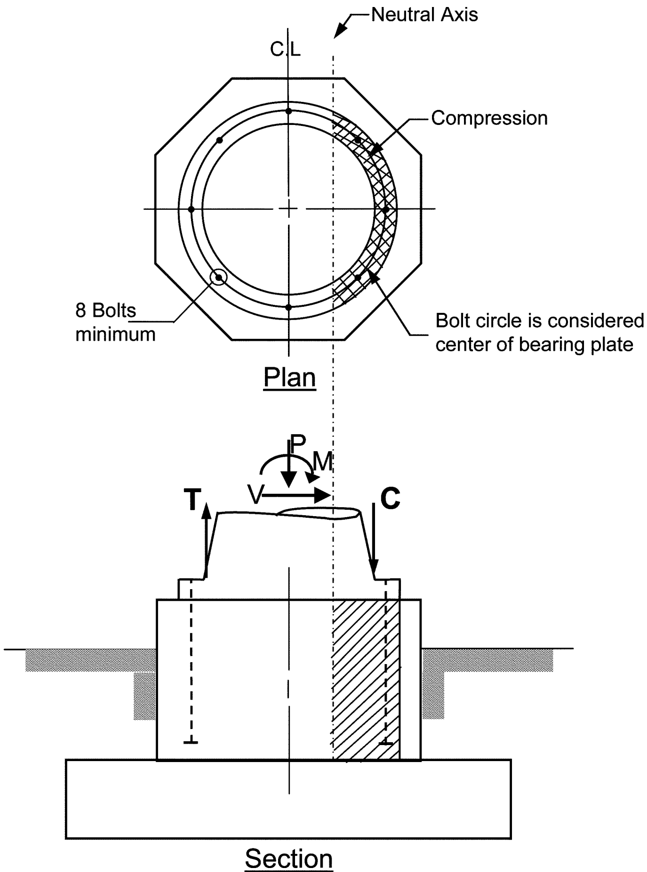


Fig. 5.2—Self-supporting stack anchor bolt design.

tightening the bolt(s); shear lugs; or direct bearing of the anchorage against the concrete. ACI 318, Chapter 17, has provisions for determining the design strength of anchors in shear for direct bearing. ACI 349, Appendix D, has design provisions based on both direct bearing on the anchors and shear friction between the connection base and the concrete. The provisions in ACI 349 may be used to determine the shear capacities of embedded plates and shear lugs.

Designs that rely on the anchor bolts to transfer shear through bearing between the sides of anchors and the inside of the baseplate holes should be approached with caution. The likelihood that all bolts in a large group or pattern will participate equally in the transfer of the shear load in bearing is unrealistic. Given the normal practice of using oversized holes in the base plate and the small misalignments that occur between bolts, only a fraction of the bolts will bear simultaneously against the base plate and hence be capable of transferring shear load (Fig. 4.2).

5.2.4 *Tension/shear interaction*—When both tension and shear forces are present in an anchorage, the interaction of the two should be considered. Section D.7 of ACI 349 and ACI 318, Section 17.6, recommend a trilinear expression for shear and tension interaction on the anchor. The new recommendations have chosen limits that eliminate the requirements for computation of interaction effects where small values of the secondary force (tension or shear) are present. When shear friction is used, the required steel strength of the anchor is a sum of the tensile strength required for direct tension from external loads and the tensile strength required for shear friction.

5.3—Bearing stress

Portions of the foundation in contact with the equipment base plates or mounting rings must be designed to comply with the nominal bearing strength given in ACI 318, Section 14.5.6, and modified by the strength reduction factor given in ACI 318, Section 21.2.

5.4—Pedestals

In the design of equipment foundations, the piece of equipment is typically located one or more feet (1 ft = 0.3 m) above grade for various functional and operational reasons. The foundation pad may be founded several feet below grade. One or more pedestals may be necessary to support the equipment and transfer the design loads to the foundation.

Pedestals should be designed for the critical combination of vertical load, horizontal load, and overturning moment. Octagonal pedestals are sometimes designed as circular columns of equivalent area.

Vertical reinforcement is provided to resist the tensile stresses in the pedestal. The controlling loading condition for reinforcement is often produced by the maximum moment with minimum vertical loading. The reinforcement is designed by one of three methods:

1. Designing the pedestal as a column with the vertical reinforcement in tension and concrete in compression;

2. Applying the combined stress formula for vertical loads and overturning moments to the reinforcement area alone. This method is commonly used by practitioners for vertical vessel foundations, when the vertical pedestal reinforcement is installed in a circular pattern, adjacent to the vessel anchors; and

3. Designing the pedestal as a flexural member, neglecting axial compression. This method is applicable only when the cross-sectional area of the pedestal is sufficiently large to justify neglecting axial compression, based on loading and slenderness considerations. A typical maximum axial compressive stress of $0.10f_c'$, based on factored loads, is common. This method is commonly used by practitioners for horizontal vessel foundations.

5.5—Soil pressure

5.5.1 *Spread footings*—Based on service loads, spread footings may be divided into two general categories: those subject to bearing pressure over the full area of the footing, and those subject to bearing pressure over part of the footing.

For full-contact pressure, the gross properties of the base area may be used to determine the soil pressure distribution. The applicable form of the combined stress formula for this condition is

$$Q = (W/A) \pm (M_x/S_x) \pm (M_y/S_y) \tag{5-2}$$

where Q is the soil pressure at the corners or far edge of the footing, lb/ft^2 (Pa); W is the resultant vertical load, lb (N); A is the base area of footing, ft^2 (m^2); M_x, M_y are moments

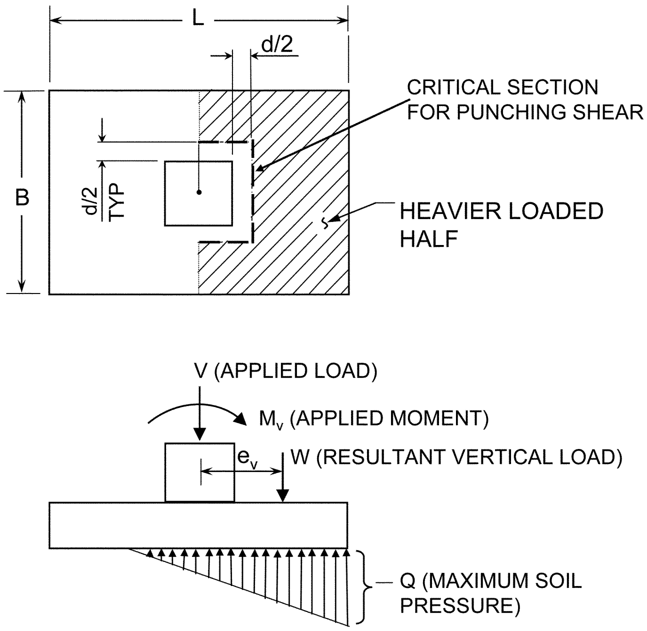


Fig. 5.3—Punching shear and maximum soil pressure due to eccentric loading.

about the x and y centroidal axes, respectively, ft·lb (N·m); and S_x, S_y are section moduli of the base about x and y centroidal axes, respectively, ft^3 (m^3).

The use of this formula assumes a rigid footing with linear distribution of strain/stress in the supporting subgrade.

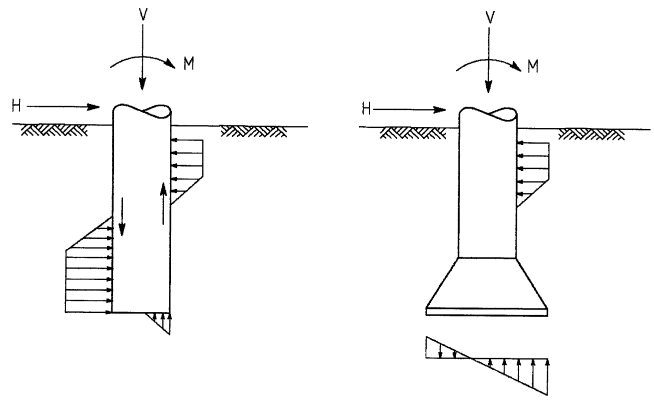
For the partial contact case, the combined stress formula is not applicable, as it would require development of tensile resistance between the soil and the footing, which is never considered. If the overturning stability criteria are met, then the mathematical assumptions of the following formula (Peck et al. 1973) for partial contact between soil and foundation can be used to determine maximum triangular distribution, soil pressure along one principal axis (refer to Fig. 5.3)

$$Q = \frac{2W}{3B(L/2 - e_v)} \tag{5-3}$$

where M_v is the moment about the centroidal axes of the foundation, ft-lb (N·m); W is the resultant vertical load, lb (N); $e_v = M_v/W$, ft (m) = eccentricity of the vertical resultant load from the footing centroidal axis; B is the width of footing, ft (m); and L is the length of footing, ft (m).

5.5.2 *Drilled piers*—Drilled piers consist of straight shafts with or without belled ends. Drilled piers are generally used in cohesive soils where the sides of the hole can be maintained. In sand, a casing is provided that can be withdrawn as concrete is placed. Care should be taken during removal to ensure that the concrete will not be disturbed, pulled apart, or pinched off by earth movements. Bells can only be drilled in cohesive soils with sufficient strength to prevent their collapse into the base during drilling. Design recommendations for drilled piers are provided in ACI 336.3R.

5.5.2.1 *Base pressure and pier capacity*—Vertical loads for a long pier are resisted by skin friction on the surface of the shaft or are resisted by bearing pressures against the base (point-bearing). For a short pier, the vertical load is carried largely by the base. If lateral loads and moments are small, the pier's capacity is approximately equal to the base area times the bearing capacity of the soil at the base. If lateral loads and moments are significant, the pier is assumed to resist the applied loads as depicted in Fig. 5.4.



(a) straight shaft *(b) belled shaft*

Fig. 5.4—Pressure distribution on drilled piers.

For a straight-shaft pier, the vertical pressure on the base is assumed to be distributed over the far-side half of the base away from the lateral load direction. For a belled pier, the base pressure is computed using the combined stress formula, assuming that the soil over the base half opposite to the direction of the lateral load would interact with the bell in a manner that provides uplift resistance. Bell and shaft diameters are selected to keep vertical soil pressures within allowable values.

Uplift resistance of drilled piers is usually a function of the skin friction and pier dead load.

5.5.2.2 *Lateral pressures*—Lateral soil pressures on a long pier, due to lateral loads or moments at the top of the pier, are determined by the consideration of load displacement characteristics of the soil and the elastic properties of the pier. The pier is treated as a flexible beam on nonlinear soil springs. Allowable lateral pressures are based on limiting the lateral displacement of the pier at the ground surface, with typical values being 1/4 to 1/2 in. (6 to 13 mm).

In the case of short piers (those with a shaft length-diameter ratio less than 10), the pier is considered rigid, with the lateral pressures varying in the manner necessary to satisfy statics (Fig. 5.4(a)). Where weathered soils exist at grade, the top 2 or 3 ft (0.6 or 0.9 m) of soil is often ignored in determining the resistance to lateral loads.

5.5.2.3 *Lateral deflection*—Lateral deflections may be determined by treating the pier as a flexible beam on nonlinear soil springs. To do so, horizontal subgrade reaction is selected based on soil consistency and published P-Y data or actual field test data.

5.5.2.4 *Settlement*—Drilled piers are generally founded on rock, on high-bearing-capacity granular soils, or on stiff, incompressible clays. For these types of soils, and where due regard for settlement has been accounted for in the allowable

bearing capacities, no appreciable vertical settlements are likely. Consequently, settlement is usually not computed for these foundations. When piers are located in or underlain by weaker clays, however, a settlement analysis is required.

5.5.3 *Raft or mat foundations*—Bearing pressures under raft or mat foundations are dependent on several factors. These factors include the type and compressibility of the soil, and the relative rigidity of the mat compared with the soil.

Procedures for design of such foundations are presented in ACI 336.2R. A simplifying assumption, which is conservative from the point of view of flexural design, considers the mat to be rigid and assumes a linear distribution of the soil subgrade reaction.

Where equipment layout or mat geometry is complex, some engineers use a finite element analysis in which the soil is represented as a series of elastic springs. Refer to Section 5.7 for additional information.

5.6—Pile loads

When upper soil strata are too weak to support spread footings, then mat foundations or piles of the endbearing or friction type are used to support the loads. Generally, the piles are assumed to be stiff vertically, and the pile cap is assumed to be flexurally rigid.

When piles are subject to uplift, and tension connectors (ties) are provided to anchor the foundation, the pile loads are assumed to vary linearly, with the resultant pile force coinciding with the location of the applied force at its eccentricity from the neutral axis. If there is no tension uplift required from the piles, and tension connectors are not used, the combined stress formula (Eq. (5-2)) may be applied to determine vertical pile loads.

Lateral loads can be assumed to be equally distributed in the pile group with resultant pile shears generally considered to be independent of the vertical forces. Other methods, such as Saul's Procedure (Saul 1968), could be used for analysis of groups of piles. For static equipment with large surface area subject to wind or with mass distribution resulting in high seismic forces, lateral loads may control the number of piles required. Passive earth resistance on the pile cap is sometimes relied upon to reduce the shear in the piles. Lateral deflections of the pile cap should be anticipated to engage passive resistance from the soil.

There are several sophisticated procedures for determining loads in pile groups (Bowles 1995). Typically, these are used where a combination of vertical piles and batter piles, or all batter piles, are selected. Usually, the simplistic procedure of setting the batter based on the minimum vertical load-maximum shear loading condition is used.

Allowable loads on piles are determined in accordance with the principles of soil mechanics. For large jobs where a pile-testing program is warranted, selection of the most efficient pile type and the maximum permissible capacity may be made. Allowable vertical capacity that is then determined may be subject to reduction for group action. Allowable horizontal capacity is based on limiting lateral deflection under shear load, and it is not generally considered to be diminished by group action.

5.7—Foundation design procedures

Typical foundation design procedures consist of selection of foundation type and foundation shape, calculation of design loads, analysis of the foundation, determination of foundation plan dimensions based on soil pressures or pile loads, design of steel reinforcement for positive moments and negative moments at critical sections, evaluation of concrete shear capacity for maximum shear forces at critical sections, checking the development of reinforcement, design of reinforcement (dowels) for load transfer at base of equipment pedestals and at the pile-to-pile cap interface, and providing seismic ductile detailing where required by the applicable building code.

5.7.1 *Factored loads*—Foundation base area or the number of piles or piers is typically determined from service (unfactored) loads. The use of the strength design method for the structural design of reinforced concrete foundation elements requires the application of factored loads. The load factors and load combinations should follow the applicable building code and Chapter 4 of this report for general guidance.

5.7.2 *Analysis of foundations*—Design moments and shears for isolated footings supporting single-column or combined footings supporting multiple columns can be determined on the basis of simple static analysis of a rigid base considering the applied loads and soil-resisting pressures. Although soil pressure distribution may be highly nonuniform, depending on footing rigidity, soil type, state, and time response to stress, common practice is to use a linear pressure distribution. Moments (positive and negative) and shears at critical sections should be computed per ACI 318 and the following subsections. Determination of design forces in large combined footings supporting multiple lines of columns or pedestals or large mat foundations supported on piles/piers may be beyond the capability of hand calculations. The use of elastic finite element computer analysis can be effective for more complex foundations. Distribution of soil pressure under the foundation must be consistent with properties of soil and the structure and with established principles of soil mechanics. Structural stiffness or flexibility of mats and combined footings can be accurately represented by plate finite elements. Soil stiffness and pile stiffness can be represented by linear springs or compression-only springs based on the proper soil subgrade modulus and pile test results. Detailed recommendations for analytical modeling can be found in ACI 336.2R.

5.7.3 *Positive moments and one-way shears*—Foundations for static equipment generally consist of isolated footings, mats, or pile caps below grade, with one or more pedestals projecting above grade. For square or rectangular foundations, critical sections for moment and shear (Fig. 5.5) are as described in Chapter 13 of ACI 318. An exception to the ACI procedure occurs with deep or thick pile caps with high-capacity piling (CRSI 2002).

Reinforcement design for octagonal foundations can be cumbersome given the mat shape and the reinforcement configuration (Fig. 3.1). Therefore, octagonal geometries are often converted to equivalent circular shapes, as shown in Fig. 5.6. An equivalent circular shape makes it easier to

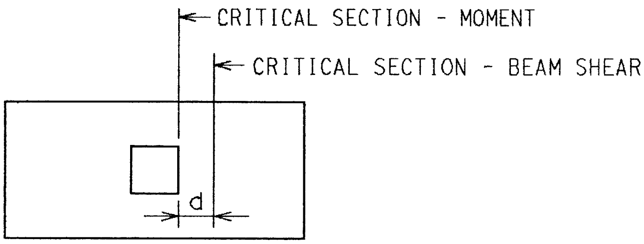


Fig. 5.5—Critical sections.

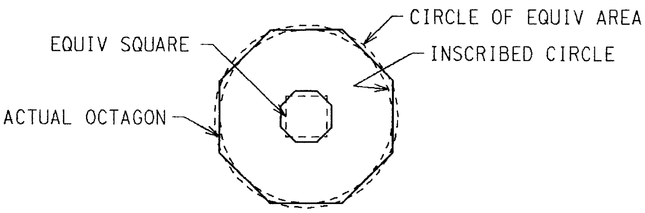
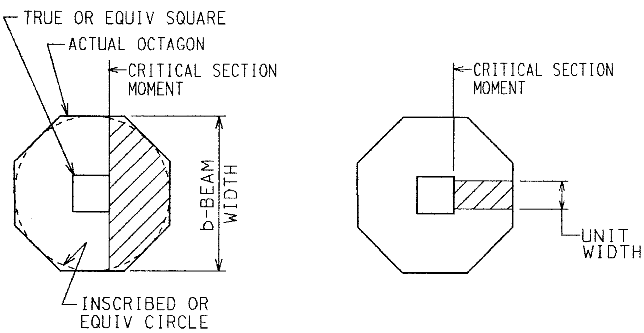
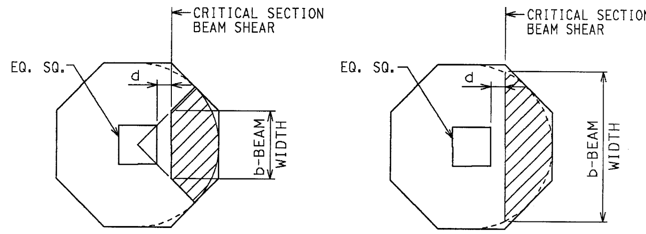


Fig. 5.6—Octagonal base options.



(a) (b)

Fig. 5.7—Octagonal bases: moment selection options.



(a) (b)

Fig. 5.8—Beam shear options.

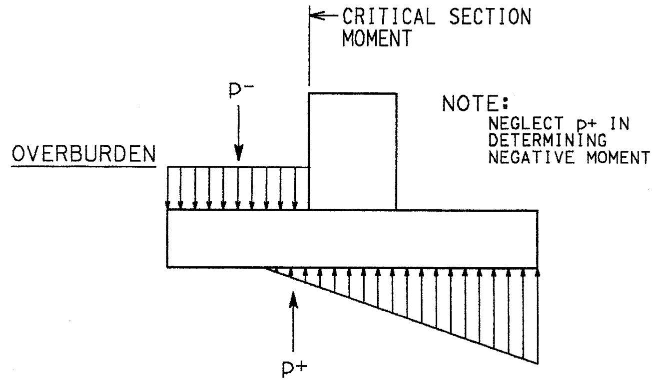


Fig. 5.9—Negative moment.

handle controlling load combinations that are not oriented on major octagonal axes. The design moment for an octagonal foundation is generally determined in one of two ways (Fig. 5.7). In Fig. 5.7(a), the moment at the face of the equivalent square pedestal is based on the area of the footing lying outside the critical section and extending the full width of the footing. In Fig. 5.7(b), known as the “one foot strip” method, a strip of unit width is subjected to the maximum soil pressure distribution; this method provides the most conservative results. Reinforcing steel for the entire footing is based on the requirements of this strip.

Beam (one-way) shear for an octagonal foundation is generally determined as shown in Fig. 5.8. In Fig. 5.8(a), the shear is computed based on the area of the base octagon bounded by the critical section at a distance *d* from the equivalent square pedestal and 90-degree radial lines drawn from the center of the base extending through the corners of the equivalent square. Alternatively, in Fig. 5.8(b), the shear is computed based on the area of the base octagon (or circle) lying outside the critical section and encompassing the full width of the footing at the critical section.

For a rigid mat foundation supporting multiple pedestals, the magnitude of positive and negative moments may be determined by elastic one-way or two-way slab theory. If the mat is designed as a flexible system, a computer analysis treating the mat as a beam or plate on an elastic foundation is used (ACI 336.2R).

5.7.4 *Negative moments*—For a spread footing with the resultant outside the kern, the footing is only partially subjected to positive soil pressure. The design negative moment is determined by summing the negative moment components produced by footing weight, overburden, surcharge loading, and any positive component from base pressure. As illustrated in Fig. 5.9, the positive moment component is often conservatively neglected.

In the case of a pile foundation where pile tension capacities are developed, the negative moments should be considered.

Location and width of critical sections for negative moments are identical to those for positive moment. Likewise, the computational procedure for octagonal or circular foundations is the same as that for positive moment.

5.7.5 *Punching shear (two-way)*—The critical sections for punching shear near columns or pedestals is as described in Section 22.6.4.1 of ACI 318. For cases where moment and shear are present, the procedure involves computing the shear on the more heavily-loaded half of the critical section, as shown in Fig. 5.3.

For piers subjected to large moments in addition to vertical forces, the punching surface departs from a simple truncated cone or pyramid. ACI 445R should be consulted in such cases.

Punching shear should also be evaluated for individual piles at critical sections in the pile cap in accordance with Section 22.6.4.1 of ACI 318. If shear parameters between

adjacent piles overlap, modified critical parameters for punching shear should follow Section R13.2.7.2 of ACI 318.

5.7.6 *Flexural reinforcement*—Minimum reinforcement requirements of ACI 318 have been variously interpreted where foundation design is concerned. Minimum reinforcement for flexural members as specified in ACI 318 is the greater of $2\sqrt{f_c'}/f_y$ or $200/f_y$ unless one-third more reinforcement than required by analysis is provided. This minimum reinforcement provision for flexural members may result in excess reinforcement for members that may be oversized for architectural or other reasons. Section 13.3.4.4 of ACI 318 allows a minimum reinforcement for mat foundations. This type of reinforcement may be evenly distributed across the uniform thickness. The portion of this shrinkage reinforcement that resists bending is generally sized as one-third more than required by analysis.

Code-specified criteria should be followed to provide adequate anchorage on each side of the critical section. Particular attention should be given in the case of octagonal footings designed using the procedure of Fig. 5.7(a). For octagonal foundations, the flexural reinforcement is placed in mats as shown in Fig. 3.1. Where hexagonal foundations are used, similar configurations are typically provided.

5.7.7 *Pedestal reinforcement*—Large equipment pedestals generally encompass a greater area than that required by the loads involved; therefore, only a small amount of reinforcement is required. Some engineers use the 0.5% minimum from Section 10.3.1.2 of ACI 318. The 0.5% minimum steel is also recommended for design of large concrete pedestals by ASCE *Design of Large Steam-Turbine Generator Foundations* (ASCE 1987). The 0.5% minimum steel is consistent with Section 16.3.4.1 of ACI 318, which states the minimum amount of reinforcement required at the pedestal to foundation interface is 0.5% of gross area of the pedestal. The purpose of having this minimum steel is to provide a degree of structural integrity during the concrete construction stage and during the life of the structure.

5.7.8 *Foundation mats designed by the finite element analysis (FEA)*—Most commercial software allows designers to design the mat slab based on either maximum moments or average moments in the design strips. The locations of the design strips are usually governed by the locations of the supports and applied loads and can also be specified by users. The program calculates the design moments per unit width using the internal forces of elements. If the design for maximum moment option is chosen, the program will report the maximum values of positive and negative moments within the strip at various sections. Conversely, the program will calculate the average positive or negative moments per unit width at various sections for the design strips. Most software has capabilities to calculate the required reinforcing steel areas. ACI 318 does not have any explicit provisions regarding how design strips should be selected for foundation mats. Experience and prudence are needed when design is based on strip average moments.

Another important consideration is twisting moments M_{xy}, which are typically ignored in conventional hand-calculation design. Of course, M_{xy} are inconsequential in the foundation design when the M_{xy} values are small. This may not always be the case, however. Most commercial software today provides an option to use the Wood-Armer equations (Wood 1968) for combining M_{xy} with M_x and M_y in design of steel reinforcement. Design based on Wood-Armer equations is more appropriate when M_{xy} is large, and cracking can occur diagonal to the design strip direction. Engineers are encouraged to use this design option when foundation configuration and loading result in significant M_{xy}.

5.7.9 *Special provisions for seismic design*—Equipment foundations (including pedestals, columns, footings, foundation mats, pile caps, piles, piers, and caissons) located in regions of high seismic risk and subjected to seismic forces should comply with the special design and detailing requirements of Section 18.13 of ACI 318. In addition, the foundations should be designed and detailed to conform to the additional requirements of the applicable building code that are not covered by ACI 318. The report (ASCE 2011b) provides design examples related to seismic design of equipment foundation.

CHAPTER 6—CONSTRUCTION CONSIDERATIONS

6.1—Subsurface preparation and improvement

6.1.1 *General considerations*—Equipment foundations can be supported on soil, rock, piles, or drilled piers, depending on the geotechnical conditions of the site. The engineer and geotechnical consultant determine the extent of soil investigation and subsurface preparation, which may vary from minimal to extensive. The construction contractor executes the subsurface preparation, and the geotechnical engineer verifies it. Adjustments are made as required as work progresses.

The site is prepared in a manner consistent with the design, and with particular attention to the engineering properties of soils. Compaction or consolidation of soft soils is commonly used to increase bearing capacity and reduce the potential for foundation settlement. In many cases, unsuitable soils are removed and replaced by sound material that is compacted to meet the design requirements. Where unsuitable foundation soils are encountered and in-place improvement or replacement of the soils is not practical, piles or drilled piers may be used to extend the foundations to suitable bearing soil or rock.

6.1.2 *Specific subsurface preparation and improvements*—The contractor should prepare the site consistent with the assumptions made and parameters used in the foundation analysis.

Specific subsurface preparation and related treatment may be required for one or more of the following reasons:

- If the exploratory borings, field tests, and observations as well as subsequent laboratory tests dictate the necessity of a subsurface treatment;
- If the exploratory borings reveal nonuniform and heterogeneous conditions and irregularities requiring local remedies; and
- If close inspection of the foundation excavation indicates conditions other than those extrapolated from the borings, thereby requiring special preparation and treatment, generally of a localized nature. Nonuniform

conditions could result in differential settlement or tilting of the foundation.

Common site-specific subsurface preparations and treatment for the aforementioned conditions are:

1. *Unstable slopes of excavation*—Unstable slopes may be stabilized by flattening the slope, benching, dewatering, shoring, freezing, injecting with chemical grouts, or supporting with dense slurries;

2. *Stratification*—Excavations with slopes parallel to the direction of stratification are avoided by flattening the slope or by providing adequate shoring;

3. *Wet excavation*—During construction, groundwater is normally lowered below the bottom of the excavation; deep well pumps or well points are commonly used. Another method is to create an impervious barrier around the excavation with cofferdams or caissons, chemical grout injection, sheet piles, or slurry trenches. A sump pit is typically provided to collect groundwater intrusion. The selection of an appropriate method depends on the characteristics of the subsurface soils encountered, costs, and the preferences of the constructor;

4. *Small surface pockets of loose sand*—Loose sand pockets are normally compacted to the degree of specified compaction. Alternatively, if the predominant soil is hard, the loose sand may be removed and replaced with flowable fill;

5. *Large deposits of loose sands*—The loose sands may be stabilized by vibrofloatation or dynamic consolidation, whichever offers an economic advantage. An analysis of long-term settlements may be necessary;

6. *Presence of organic material or unconsolidated soft clays*—All organic materials and soft clays are normally removed and replaced with suitable, well-compacted fill that provides the characteristics desired for the proper performance of the foundation. Alternatively, piling or drilled piers may be used to carry foundation loads to sound bearing strata;

7. *Fissured rock*—The extent of fissures is evaluated to determine if remedial treatment is needed. From the evaluation, the geotechnical service provider can recommend suitable remedies for some types of fissures, such as pressure grouting or rock anchors. In the case of seismic faults, thorough geotechnical and geological evaluation is required to ascertain the potential hazard. Where significant hazards exist, relocation of the entire facility to avoid the hazard is a suitable remedy;

8. *Irregularly weathered rock*—The weathered seams are cleaned and replaced with lean concrete. Alternatively, the foundation depth may be revised to reach sound rock;

9. *Solution cavities in limestone deposits*—The voids, if small, are pumped full of grout or, in the case of large holes, lean concrete under a pressure head;

10. *Unconsolidated clay*—Clays may be preloaded and related settlements monitored. (Early identification is important to gain lead time and avoid slippage in the construction schedule.) Alternatively, piling or drilled piers may be used to carry foundation loads to firm bearing strata; and

11. *Cold climates*—The construction crew should not place foundations on frozen subgrade material. The material should be thawed and recompacted if necessary.

6.2—Foundation placement tolerances

Foundation placement tolerances depend largely on the type of equipment being supported and are specified by the engineer on the drawings or in the specifications. The contractor should use templates during concrete placement to support anchor bolts and other embedments that must be precisely positioned (ACI 117).

6.3—Forms and shores

6.3.1 *General requirements for forms*—Forms and shoring for construction of concrete foundations should follow the recommendations of ACI 347R. As applicable, provisions of ACI 301 should be specified.

6.3.2 *Shoring*—Shoring should support the concrete loads, impact loads, and temporary construction loads. Transverse and longitudinal bracing may be required to sustain lateral forces.

The formwork engineer should consider wind loads in the shoring design. It is not usually necessary to consider seismic loads due to the limited time shoring is in place. A licensed professional engineer should prepare the design of formwork and submit it to the design engineer for review.

6.3.3 *Shoring systems and formwork for large elevated foundations*—Temporary formwork systems are generally used for large elevated foundations. Less frequently, permanent systems may be used for special applications. The contractor usually selects a temporary support system. The selection is influenced by the construction sequence of the building (if the equipment is enclosed), the equipment installation procedure, and access requirements at the time of placement of the foundation. Some of the permanent systems may affect the design and cost of the foundation. Therefore, the design engineer may wish to consult with building contractors before deciding on a permanent formwork system.

Some of the systems used are:

- Standard construction shoring consisting of temporary shore legs supported by the foundation mat and supporting the soffit forms of a foundation deck;
- Shoring consisting of structural steel beams supported on brackets attached to the foundation columns. The forms rest on top of the beams. Jacking devices are used to lower the beams and forms for removal after the concrete reaches sufficient strength;
- Embedded structural steel shapes (rolled wide flange beams, girders, angles, or channels) supported on the foundation columns and carrying the permanent deck forms. The forms (steel decking) usually rest on the bottom flanges of the steel shapes. Because the steel members are embedded in the foundation deck, the design engineer should be careful to avoid interferences with the reinforcing bars and with other embedments (anchor bolts, plates, pipe sleeves, and conduits);
- Embedded structural steel trusses supported on the foundation columns and carrying the permanent deck form on the bottom truss cords. The trusses, if specially designed, can also be considered as reinforcement to carry the operating loads acting on the foundation deck. Checking for interferences between the trusses and the

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reinforcing bars and other embedments is important to avoid serious construction problems; and

- Precast concrete deck forms supported by the foundation columns. These can be flat-bottom U and double-U shapes.

All of these systems, except for the standard construction shoring system, allow immediate access under the foundation deck because shore legs are absent. The standard shoring system, however, has the least impact on foundation design. The remaining four systems should be coordinated, in varying degrees, with the foundation design.

In all five cases, the design engineer should review the constructor's proposed construction procedure.

6.4—Sequence of construction and construction joints

Many equipment foundations are too massive for the concrete to be placed in one continuous operation. Construction joints subdivide large foundations into smaller placement units. Subdivision of large structures by construction joints also helps reduce internal heat of hydration in concrete and shrinkage cracks in the foundation. To gain the maximum benefit, the contractor should place alternate foundation segments and allow them to cure and shrink before the intermediate segments are placed.

The structural integrity of the foundation requires that joints be carefully constructed using accepted practices for construction joints in major concrete structures, such as those specified in ACI 301. Project specifications normally require that the constructor obtain the approval of the engineer for construction joint locations and details.

The location of construction joints should follow normal reinforced concrete building practice. Joints in columns should be located at or near the floor line and at the underside for supported beams. If beams are haunched, the joints should be located at the underside of the deepest haunch. Joints in beams and mats should be located at sections of low stress.

Transverse construction joints should be at right angles to longitudinal reinforcement. Horizontal joints in beams and slabs placed in more than one lift should be supplied with sufficient transverse reinforcement to develop the required horizontal shear capacity by shear friction. Preparation of construction joints should be in accordance with ACI 304R.

Transfer of loads across a construction joint should be provided by specific means. Tensile loads, for example, should be transferred by extending reinforcing bars across the joint. Transfer of compressive loads can be accomplished by assuring that the concrete on both sides of the joint is clean and free of laitance and located where they cause the least weakness in the structure as described in ACI 318, Chapter 6.4. Additional measures are needed to transfer shear loads. Shear keys should be cast in the face of the joint. Alternatively, the face of the joint can be roughened sufficiently for shear loads to be transferred by shear friction. With the latter method, sufficient reinforcing bars should extend across the joint to hold the surfaces of the joint in close contact.

6.5—Equipment installation and setting

6.5.1 *Shims, wedges, and bolts*—Shims, wedges, and bolts typically are the interface system between the foundation and the equipment base. The chosen interface system can be influenced by the manufacturer's recommendations and requirements, the foundation construction procedures, the setting and adjustment of the equipment, and the final tolerances.

Shims, which are usually carbon steel or brass stock in various thicknesses, have both economical and high-load-bearing qualities. Shims should be fabricated with rounded corners to avoid crack initiation in the grout.

Wedges are usually the double-wedge type and are offered by several mounting-equipment manufacturers. The double-wedge mount often has one or more threaded studs for precise vertical adjustment and for locking the sliding wedge into the required position. A lock nut may also be used for locking the main horizontal stud into the final position.

Other types of wedges often used by millwrights include various-shaped temporary steel wedges. Temporary wedges or tools used to adjust the location of the equipment to within tolerance are placed before grouting, and are removed after the hardening of the grout material. Permanent wedge assemblies allow for future adjustments on ungrouted equipment bases.

The manufacturer's drawings should give the required bolt diameters. For construction, the design drawings or specifications should provide bolt diameters, types, overall lengths, threaded lengths, projections, materials, method of tightening, and required tension or torque.

Required bolt tightening can be accomplished with a tensioning jacking procedure, the turn-of-the-nut method, or sequential calibrated wrench. Tensioning jacking is used on the deeper anchorages with unbonded shanks. When the shank length is embedded in concrete, which is generally not recommended, the turn-of-the-nut method or sequential calibrated wrench tightening is specified. Section 5.2.2 presents comments on monitoring the bolt tension. Impact wrenches are not used for tightening of a bolt when the anchorage is embedded the full length in concrete because of the extremely high torque and tensile forces delivered by such tools.

6.5.2 *Embedments*—Embedments in the concrete include the anchor bolt assemblies previously described, shear lugs, and shear transferring devices. Because shear is a component of the total load transferred to the concrete foundation, steel lugs can be integral parts of the equipment base. Such lugs are grouted into shear key grooves previously cast into the concrete base.

6.6—Grouting

6.6.1 *Types of grout*—There are two basic types of grout: cementitious (cement-based) grouts and polymer (including epoxy-based) grouts. Cementitious grouts are lower in cost, but polymer grouts have higher resistance to chemicals, shock, and vibratory loads.

6.6.2 *Applications*—ACI 351.1R contains details on the application of grouts. In specifying grout systems, the engineer should consider the different characteristics of each type of grout along with field limitations, and match these with

specific requirements of the job. In evaluating which cementitious grout should be used, the engineer should consider the workability of the grout and its physical properties: volume change; compressive strength; working time; consistency; and setting time. In evaluating polymer grouts, the engineer should consider the same aforementioned factors in addition to creep and the effects of temperature-induced volume changes. The engineer should review the design of the equipment base, accessibility of the grouting location, clearances provided for the grout, and the design of the anchor bolts. Most of the grouts on the market are premixed, prepackaged materials, and contain manufacturer's instructions on surface preparation, formwork, mixing, placing, and curing procedures. Refer to ACI 351.4 and ACI 351.5 for cementitious and epoxy grouting installation specifications.

6.7—Concrete materials

Large equipment foundations require special attention to the design and control of the concrete mixture. Refer to ACI 207.1R, 207.4R, 211.1, and 301.

Many foundation members are massive enough for the heat of hydration of the cement to generate a large thermal differential between the inside and the outside, which may cause unacceptable surface cracking unless steps are taken to reduce the rate of release of this heat. Creep, differential thermal expansion, and shrinkage can cause distortion of the foundation and consequent unacceptable changes in equipment alignment. It is important to proportion the concrete mixture to minimize creep and shrinkage and to reduce the thermal expansion of the hardened concrete. Temperatures may be monitored using thermocouples or resistance thermometers. If excessive temperatures are detected, surface-cooling systems can be used to provide limited benefits in controlling temperatures (ACI 207.4R).

To minimize the rate of release of the heat of hydration and to control shrinkage and creep, the following steps are normally followed:

- The lowest content of cementitious material consistent with attaining the required strength and durability is used;
- Part of the cement is replaced with fly ash or other pozzolan;
- The placing temperature of fresh concrete is lowered by chilling the aggregate, using chipped ice for mixing water, or both;
- The largest practical size of aggregate is used to allow further reduction in the amount of cement;
- Type II cement (that satisfies the moderate heat option of ASTM C150/C150M) or Type IV cement is used;
- A water-reducing agent is used to allow further reduction of the cement factor;
- Low slump and normal vibration are used;
- Concrete placement by pumps, which may use concrete mixtures with high amounts of cement and small aggregate sizes, is avoided; and
- Sizes of placements for large foundations are reduced.

High-range water-reducing admixtures may be an appropriate choice because they are consistent with their general applicability to mass concrete applications and heavily reinforced installations for which workability is an issue.

The coefficient of thermal expansion of the hardened concrete can be somewhat controlled by the aggregate size choice because overall expansion is a function of the coefficient of thermal expansion of the aggregate. When excessive thermal expansion may be a problem, the coefficient of expansion of available aggregates is measured to determine their suitability for the application. (In many regions of the world, there may be limited choices in the types and sources of aggregates.)

Expansion of concrete from alkali-aggregate reaction can be minimized by using a low-alkali cement, by replacing a portion of the cement with a fly ash or non-fly ash pozzolan meeting the requirements of ASTM C618, and by selecting low-reactivity aggregates. The potential reactivity of aggregates can be evaluated with the procedures and tests described in ASTM C295, C227, C289, and C586. ASTM C33/C33M and ACI 225R cover the evaluation methods of the potential reactivity of aggregates.

The cement content should be low enough to help meet heat of hydration requirements, but high enough to meet strength, creep, and shrinkage requirements. It may not be possible to completely solve the heat problem by reducing the heat of hydration. Cooling, small placements, or pozzolans may also be needed.

6.8—Quality control

Because the foundation for the equipment acts as an integral part of the equipment-foundation-soil system, an appropriate quality-control program should be implemented to ensure that the design requirements are met during construction. ACI 311.4R contains guidance on items to include in the quality program. ACI 311.5 contains guidance on concrete plant inspection and testing of ready mixed concrete. ACI 311.1R (SP-2) contains general guidance on inspection of concrete. The quality-control program should include requirements for control of material quality, the engineer's approval of critical construction procedures, and on-site verification of compliance with design drawings and project specifications by a qualified inspector, preferably certified by ACI as a Concrete Construction Inspector.

Design drawings and project specifications provide foundation requirements to the constructor and inspector. The inspector is typically required to report to the design engineer or owner any changes or modifications to the specified design warranted by the conditions in the field. The design engineer or owner should approve any acceptable changes in the specified design and document them in accordance with pre-established procedures.

The quality-control program and inspections should be thoroughly documented and be available for the owner's review. The quality-control program should be consistent with those commonly implemented for construction projects of similar importance. ACI 301 can be cited as part of that quality-control program. Agencies providing testing and inspections should be accredited, satisfying the requirements of ASTM E329.

CHAPTER 7—REFERENCES

7.1—Referenced standards and reports

The standards and reports listed below were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest version.

American Concrete Institute

117	Specifications for Tolerances for Concrete Construction and Materials and Commentary
201.2R	Guide to Durable Concrete
207.1R	Guide to Mass Concrete
207.2R	Report on Thermal and Volume Change Effects on Cracking of Mass Concrete
207.4R	Cooling and Insulating Systems for Mass Concrete
211.1	Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete
225R	Guide to the Selection and Use of Hydraulic Cements
301	Specifications for Structural Concrete
304R	Guide for Measuring, Mixing, Transporting, and Placing Concrete
307	Code Requirements for Reinforced Concrete Chimneys and Commentary
311.1R	Manual of Concrete Inspection (SP-2)
311.4R	Guide for Concrete Inspection
311.5	Guide for Concrete Plant Inspection and Testing of Ready-Mixed Concrete
318	Building Code Requirements for Structural Concrete and Commentary
336.2R	Suggested Analysis and Design Procedures for Combined Footings and Mats
336.3R	Design and Construction of Drilled Piers
347	Guide to Formwork for Concrete
349	Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary
351.1R	Grouting Between Foundations and Bases for Support of Equipment and Machinery
351.3R	Foundations for Dynamic Equipment
351.4	Specification for Installation of Cementitious Grouting between Foundations and Equipment Bases
351.5	Specification for Installation of Epoxy Grout between Foundations and Equipment Bases
355.2	Qualification of Post-Installed Mechanical Anchors in Concrete and Commentary
445R	Recent Approaches to Shear Design of Structural Concrete

American Institute of Steel Construction

AISC 360	Specification for Structural Steel Buildings

American Petroleum Institute

API RP686	Recommended Practices for Machinery Installation and Installation Design

American Society of Civil Engineers

ASCE/SEI 7	Minimum Design Loads for Buildings and Other Structures

American Society of Mechanical Engineers

B1.1	Unified Inch Screw Threads (UN and UNR Thread Form)
B18.2.1	Square and Hex Bolts and Screws, Inch Series
B18.2.2	Square and Hex Nuts
BPVC	Boiler and Pressure Vessel Code (BPVC)
STS-1	Steel Stacks

ASTM International

A307	Standard Specification for Carbon Steel Bolts and Studs, 60 000 PSI Tensile Strength
C33/C33M	Standard Specification for Concrete Aggregates
C150/C150M	Standard Specification for Portland Cement
C227	Standard Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)
C289	Standard Tests Method for Potential Alkali-Silica Reactivity of Aggregates (Chemical Method)
C295	Standard Guide for Petrographic Examination of Aggregates for Concrete
C586	Standard Test Method for Potential Alkali Reactivity of Carbonate Rocks as Concrete Aggregates (Rock-Cylinder Method)
C618	Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
E329	Standard Specification for Agencies Engaged in Construction Inspection and/or Testing
F1554	Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength

Federal Emergency Management Agency

FEMA 450	National Earthquake Hazards Reduction Program (NEHRP) Recommended Provisions and Commentary for Seismic Regulations for New Buildings and Other Structures

Process Industry Practices

STC01015	Structural Design Criteria
STE05121	Application of ASCE Anchorage Design for Petrochemical Facilities

The above publications may be obtained from the following organizations:

American Concrete Institute
38800 Country Club Drive
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www.concrete.org

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American Institute of Steel Construction
One East Wacker Drive
Chicago, IL 60601
www.aisc.org

American Petroleum Institute
1220 L Street NW
Washington, DC 20005
www.api.org

American Society of Civil Engineers
1801 Alexander Bell Drive
Reston, VA 20191
www.asce.org

American Society of Mechanical Engineers
Three Park Avenue
New York, NY 10016
www.asme.org

ASTM International
100 Barr Harbor Dr.
West Conshohocken, PA 19428-2959
www.astm.org

Federal Emergency Management Agency Earthquake Programs
500 C Street, S.W.
Washington, DC 20472

Process Industry Practices
3925 West Braker Lane (R4500)
Austin, TX 78759
www.pip.org

7.2—Cited references

ACI Committee 207, 1995, "Effect of Restraint, Volume Change, and Reinforcement on Cracked Concrete," American Concrete Institute, Farmington Hills, MI, 26 pp.

ACI Committee 318, 2014, "Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary," American Concrete Institute, Farmington Hills, MI, 519 pp.

American Society of Civil Engineers (ASCE), 1987, *Design of Large Steam Turbine-Generator Foundations*, ASCE, Reston, VA, 83 pp.

American Society of Civil Engineers (ASCE), 2011a, *Wind Loads for Petrochemical Facilities*, Task Committee on Wind Induced Forces of the Petrochemical Committee of the Energy Division of ASCE, Reston, VA, 168 pp.

American Society of Civil Engineers (ASCE), 2011b, *Guidelines for Seismic Evaluation and Design of Petrochemical Facilities Second Edition*, Task Committee on Seismic Evaluation and Design of Petrochemical Facilities of the Petrochemical Committee of the Energy Division of ASCE, Reston, VA, 363 pp.

American Society of Civil Engineers (ASCE), 2013, *Anchorage Design for Petrochemical Facilities*, Task Committee on Anchorage of the Petrochemical Committee of the Energy Division of ASCE, Reston, VA, 164 pp.

Bowles, J. E., 1995, *Foundation Analysis and Design*, fifth edition, McGraw-Hill Book Co., New York, Chapters 4, 8, and 16.

Breen, J., 1966, "Development Length of Anchor Bolts," *Highway Research Record* No. 147.

Concrete Reinforcing Steel Institute (CRSI), 2002, *CRSI Design Handbook*, ninth edition, CRSI, Schaumburg, IL, 648 pp.

Eligehausen, R.; Fuchs, W.; and Mayer, B., 1987, "Load Bearing Behavior of Anchor Fastenings in Tension," *Betonwerk + Fertigteiltechnik*, Dec., pp. 826-832, and Jan. 1988, pp. 29-35.

Eligehausen, R.; Fuchs, W.; and Mayer, B., 1988, "Load Bearing Behavior of Anchor Fastenings Under Shear, Combined Tension and Shear or Flexural Loadings," *Betonwerk + Fertigteiltechnik*, Dec., pp. 48-56.

Fuchs, W.; Eligehausen R.; and Breen, J., 1995, "Concrete Capacity Design (CCD) Approach for Fastening to Concrete," *ACI Structural Journal*, V. 92, No. 1, Jan.-Feb., pp. 73-74.

Gaylord, E. H.; Gaylord, C. N.; and Stallmeyer, J. E., 1997, *Structural Engineering Handbook*, fourth edition, McGraw-Hill, 1264 pp.

Housner, G. W., 1956, "Limit Design of Structures to Resist Earthquakes," *Proceedings*, World Conference on Earthquake Engineering, University of California, Berkeley, CA, 14 pp.

International Code Council (ICC), 2006, "International Building Code," Falls Church, VA, 680 pp.

Lee, D., and Breen, J., 1966, "Factors Affecting Anchor Bolt Development," *Report 8E-1F*, Center for Highway Research, University of Texas at Austin, Aug.

Lee, N.; Park, K.; and Suh, Y., 2007, "Shear Headed Anchors with Large Diameter and Deep Embedment in Concrete," *Paper* #H07/2, Transactions, SMiRT 19, Toronto, ON, Canada, Aug.

Peck, R; Hanson, W.; and Thornburn, T., 1973, *Foundation Engineering*, second edition, 387 pp.

Ratay, R. T., 2002, *Design Loads on Structures during Construction Standard—Background, Purpose and Scope*, SEI/ASCE 37, ASCE, Reston, VA, 6 pp.

Saul, W., 1968, "Static and Dynamic Analysis of Pile Foundation," *Journal of the Structural Division*, ASCE, V. 44, No. ST5, May.

Scholl, R.; Czamecki, R.; Kirchner, C.; Shah, H.; and Gerie, J., 1978, "Seismic Analysis of Oil Refinery Structures, Part II—Evaluation of Seismic Design Criteria," *Technical Report* No. 32, John A. Blume Earthquake Engineering Center, Stanford University, Stanford, CA, Sept.

Troitsky, M. S., 1982, *Tubular Steel Structures*, J. F. Lincoln Arc Welding Foundation, Section 5.5.

Wood, R. H., 1968, "The Reinforcement of Slabs in Accordance with a Predetermined Field of Moments," *Concrete*, Feb., pp. 69-76.

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