Foundations for Static Equipment
Reported by ACI Committee 351

ACI Committee Reports, Guides, Standard Practices and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction and in preparing specifications. References to these documents shall not be made in the Project Documents. If items found in these documents are desired to be part of the Project Documents, they should be phrased in mandatory language and incorporated into the Project Documents.

The American Concrete Institute takes no position respecting the validity of any patent rights asserted in connection with any item mentioned in this report. Users of this report are expressly advised that determination of the validity of any such patent rights, and the risk of infringement of such rights, are entirely their own responsibility.

Erick N. Larson*
Chairman
Hamid Abdoveis*
William Babcock
J. Randolph Becker*
William L. Bounds*
Marvin A. Cones
Dale H. Curtis*
Shraddhakar Harsh*
C. Raymond Hays*
A. Harry Karabinis*
John C. King
Joseph P. Morawski*
Navin Pandya*
Ira W. Pearce*
Mark Porat*

ACI 351.2R-94 (Reapproved 1999)

The committee has developed a discussion document representing the state-of-the-art of static equipment foundation engineering and construction. It presents the 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 the report is to present the various methods. It is not intended to be a recommended practice, but rather a document which encourages discussion and comparison of ideas.

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

CONTENTS

Chapter 1-Introduction, p. 351.2R-2
1.1-Background
1.2-Purpose
1.3-Scope

Chapter 2-Foundation types, p. 351.2R-2
2.1-General considerations
2.2-Typical foundations

Chapter 3-Design criteria, p. 351.2R-4
3.1-Loading
3.2-Design strength/stresses
3.3-Stiffness/deflections
3.4-Stability

Chapter 4-Design methods, p. 351.2R-19
4.1-Available methods
4.2-Anchor bolts and shear devices
4.3-Bearing stress
4.4-Pedestals
4.5-Sail pressures
4.6-Pile loads
4.7-Foundation design procedures

Chapter 5-Construction considerations, p. 351.2R-24
5.1-Subsurface preparation and improvement
5.2-Foundation placement tolerances
5.3-Forms and shores
5.4-Sequence of construction and construction joints
5.5-Equipment installation and setting
5.6-Grouting

* Members of Subcommittee 351.3 which prepared this report.
The Committee also wishes to extend its appreciation and acknowledgement of two Associate Members who contributed to this report: D. Keith McLean and Alan Porush.


Copyright © 1994, American Concrete Institute.
All rights reserved including rights of reproduction and use in any form or by any means, including the making of copies by any photo process, or by any electronic or mechanical device, printed, written, or oral or recording for sound or visual reproduction or for use in any knowledge or retrieval system or device, unless permission in writing is obtained from the copyright proprietors.
CHAPTER 1 - INTRODUCTION

1.1-Background

Foundations for static equipment are used throughout the world in industrial processing and manufacturing facilities. Many engineers with varying backgrounds are engaged in the analysis, design, and construction of these foundations. Quite often they perform their work with very 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. However, some engineering firms and individuals have developed their own standards and specifications as a result of research and development activities, field studies, or many years of successful engineering or construction practice. Firms with such standards usually feel that their information is somewhat unique and, therefore, are quite reluctant to distribute it outside their organization, let alone publish it. Thus, without open distribution, review, and discussion, these standards represent only isolated practices. Only by sharing openly and discussing this information can a truly meaningful consensus on engineering and construction requirements for static equipment foundations be developed. For this reason, the committee has developed a discussion document representing the state-of-the-art of static equipment foundation engineering and construction.

As used in this document, state-of-the-art refers to state-of-the-practice and encompasses the various engineering and construction methodology in current use.

1.2-Purpose

The Committee presents, usually without preference, 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 thus elicit critical discussion from the industry. This report is not intended to be a recommended practice, but rather a document that will encourage discussion and comparison of ideas.

1.3-Scope

This report is limited in scope to the engineering and construction of static equipment foundations. The term “static equipment” as used herein refers to industrial equipment that does not contain moving parts or whose operational characteristics are 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.

Excluded from this report are foundations for machinery such as turbine generators, pumps, blowers, compressors, and presses, which have operational characteristics that are essentially dynamic in nature. Also excluded are foundations for vessels and tanks whose bases rest directly on soil, for example, clarifiers, concrete silos, and American Petroleum Institute (API) tanks. Foundations for buildings and other structures that contain static equipment are also excluded.

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

CHAPTER 2 - FOUNDATION TYPES

2.1-General considerations

The type and configuration of a foundation for equipment may be dependent on the following factors:

1. Equipment base configuration such as legs, saddles, solid base, grillage, or multiple supports locations.
2. Anticipated loads such as the equipment static weight, and loads developed during erection, operation, and maintenance.
3. Operational and process requirements such as accessibility, settlement constraints, temperature effects, and drainage.
4. Erection 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 zones.
8. Construction considerations.
9. Environmental requirements such as secondary containment or special concrete coating requirements.

2.2-Typical foundations

2.2.1 Vertical vessel and stack foundations - For tall vertical vessels and stacks, the size 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 anchored to
a pedestal with dimensions sufficient to accommodate the anchor bolts and base ring. Operational, maintenance, or other requirements may dictate a larger pedestal. The pedestal may then be supported on a 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 foundation may consist solely of a soil-supported pedestal.

Individual pedestals may be circular, square, hexagonal or octagonal. If the vessel has a circular base, a circular, square, or octagonal pedestal is generally provided. 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 in supporting vertical vessels and stacks with circular bases (see Fig. 2.2.1).

2.2.2 Horizontal vessel and heat exchanger foundations

- Horizontal equipment such as heat exchangers and reactors of various types are typically supported on pedestals that rest on spread footings, strap footings, pile caps, or drilled piers. Elevation requirements of piping often dictate that these vessels be several feet above grade. Consequently, the pedestal is the logical means of support.

The configuration of pedestals varies with the type of saddles on the vessels, and with the magnitude and direction of forces to be resisted. Slide plates are also used to reduce the magnitude of thermal horizontal forces between equipment pedestals. The most common pedestal is a prismatic wall type. However, T-shaped (buttressed) pedestals may be required if the horizontal forces are very high (see Fig. 2.2.2).

2.2.3 Spherical vessel foundations

- Large spherical vessels are sometimes constructed with a skirt and base ring, but more often have leg-supports. For leg-supported spherical vessels, foundations typically consist of pedestals under the legs 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 utilized (see Fig. 2.2.3).

2.2.4 Machine tool foundations

- Machine tool equipment is typically supported on at-grade mat foundations. These may be soil-bearing or pile-supported depending upon the bearing capacity of the soil and the settlement limitations for the machinery (see Fig. 2.2.4). Where a machine tool produces impact type loads, it is generally isolated from the neighboring mat to minimize transmission of vibration to other equipment.

2.2.5 Electrical equipment and support structure foundations

- Electrical equipment typically consists of transformers, power circuit breakers, switchgear, 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 more massive energized equipment, are typically designed for (1) dead loads, (2) seismic loads, (3) erection loads (i.e., jacking), and (4) operating loads. These foundations are typically slabs on grade, or slabs on piles. Anchorage is provided by anchor bolts or by welding the equipment base to embedded plates.

Foundations of support structures for stiff electrical buses, switch stands, line traps, and lightning arrestors 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; therefore, the supporting foundations commonly used are drilled piers. If soil bearing conditions are unfavorable, however, spread footings or pile supported footings are generally used.

Support structures for overhead electrical conductors, such as transmission towers, poles, dead-end structures, and flexible bus supports, are designed for tension loads from the conductors along with ice and wind loads.
Fig. 2.2.2-Footings with strap for horizontal vessels

Drilled piers are commonly used to support such structures. Spread footings or pile supported footings are also used when required by soil conditions.

CHAPTER 3-DESIGN CRITERIA

Criteria used for the design of static equipment foundations vary considerably among engineering practitioners. There may be several reasons for this variability. Most heavy equipment foundations are designed by or for large organizations, which may include utilities and government agencies. Many of these organizations, with their in-house expertise, have developed their own engineering practices, including design criteria. Many organizations, after investing considerable resources in development, consider such information proprietary. They find no incentive to share their experience and research with others. For these reasons, there is limited published information on the criteria used for the design of the types of static equipment foundations covered by this report.

3.1-Foundation loading

Most practitioners first attempt to use the common loadings defined by local building codes, or by ACI 318. However, many engineers have difficulty in classifying the large number of 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.

3.1.1 Loads

3.1.1.1 Dead loads - Dead loads invariably consist of the weight of the equipment, platforms, piping, fireproofing, cladding, ducting, and other permanent attachments. Some engineers also designate the operating contents (liquid, granular material, etc.) of the equipment as dead loads. However, such a combination 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.

3.1.1.2 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.
3.1.1.4 Wind loads - When designing outdoor equipment foundations to be constructed in an area under the jurisdiction of a local building code, most engineers will use the relevant provisions in that code for determining wind loads on equipment. Most codes, such as the older editions of the Uniform Building Code (UBC 79) specify wind pressures according to geographic area, height above grade, and equipment geometry. Dynamic characteristics of the structure or equipment are not recognized, nor are any types of structures or equipment specifically excluded from consideration. The procedures used are simple even though, as most engineers believe, they are somewhat crude in their representation of the actual effect of wind.

Some practitioners, particularly when designing equipment foundations outside the jurisdiction of local building codes, use the more recent and purportedly more rational wind load provisions contained in ASCE Standard 7 (formerly ANSI A58.1). However, these provisions have the reputation of being significantly more complex than those in most building codes.

The ASCE 7 wind pressure relationships can, in general, be represented by the following two equations:

\[ q_z = 0.00256K_z(IV)^2 \]  
\[ p_z = q_zGC \]

Where the various parameters are defined as follows:
- \(q_z\) = velocity pressure at height \(z\)
- \(V\) = basic wind speed (mph)
- \(I\) = importance factor
- \(K_z\) = height and exposure coefficient
- \(p_z\) = design pressure at height \(z\) (psf)
- \(G\) = gust factor
- \(C\) = pressure or drag coefficient

The reputation of complexity and unwieldiness of the ASCE 7 wind provisions is unjustified when designing rigid equipment, such as short stubby vertical vessels, horizontal tanks, heat exchangers, machine tools, and electrical equipment. For these rigid types of equipment, the ASCE 7 wind provisions require only a selection of a basic wind speed, an “importance factor,” which adjusts the basic wind speed for mean recurrence interval, and determination of a “velocity pressure.” This latter quantity is a function of both “exposure” (topography) and height above grade. Design wind pressures are then determined by multiplying the velocity pressure by a “gust factor” and a pressure (or drag) coefficient. The gust factor adjusts the mean velocity pressure to a peak value for the given exposure and height. The pressure or drag coefficients reflect the geometry and tributary exposed area of the item being investigated, and its orientation relative to the wind flow.

When designing tall flexible towers, vertical vessels and stacks, or their foundations, the engineer is faced with a problem when using the ASCE 7 wind load provi-
sions. This problem occurs in the introductory paragraph to the ASCE 7 wind load provisions, which excludes from consideration "structures with... structural characteristics which would make them susceptible to wind-excited oscillations." Tall flexible process towers, stacks, and chimneys are indeed susceptible to wind-excited oscillations. Both the discussion in Chapter 4 of ACI 307 as well as the material presented in Chapter 5 of ASME/ANSI STS-I-1986 (steel stacks) are recommended references for these solutions.

3.1.1.5 Seismic loads - Determining lateral force requirements for equipment is a challenge for practicing engineers. The reason stems primarily from the building codes commonly used to make such determinations. Since the primary focus of building codes is upon "building type" structures, the applicability to equipment and nonbuilding type structures is less than clear, particularly when most of the codes use nomenclature applicable to structures rather than equipment.

These difficulties have been widely recognized, and steps have been taken to make the equipment requirement sections of codes more "user-friendly" for the practicing engineer. Most notably, the 1991 edition of the Uniform Building Code (UBC), widely used in the seismic zones of the western United States, adopts the refinements and improvements from recommendations of the Structural Engineers Association of California (SEAOC). SEAOC's Subcommittee on Nonbuilding Structures, a part of the Seismology Committee, continues its efforts to develop "stand-alone" requirements that expand the scope and refine the treatment for seismic loads on equipment.

These efforts and widespread refinements made by SEAOC for structures have made the Uniform Building Code the "state-of-the-art" code for lateral load requirements, even in many jurisdictions that have not specifically adopted the UBC. Other codes or standards that specify lateral force requirements on buildings or structures include ASCE 7 (formerly ANSI A58.1), The BOCA National Building Code, and the Standard Building Code (SBC). The Federal Emergency Management Agency's (FEMA) National Earthquake Hazards Reduction Program (NEHRP) Standard (1991) should also be consulted for seismic force requirements for equipment.

3.1.1.5.a UBC lateral force requirements for equipment - The UBC makes no distinction between "static" and "dynamic" equipment for seismic loads. Rather, whether the equipment is "rigid" or "nonrigid" determines the values for the variables used in the formulae for calculating lateral forces. Therefore, lateral force requirements for equipment do not depend upon equipment type, but upon rigidity. Equipment with a fundamental frequency greater than or equal to 16.7 Hertz, or a period less than or equal to 0.06 second, is considered "rigid."

The performance of many types of vendor-manufactured, floor-mounted equipment (both rigid and nonrigid) in past earthquakes has demonstrated a typically high inherent strength for resisting seismic loads. Whether for operating, manufacturing, or shipping considerations, mechanical equipment such as pumps, engine and motor generators, chillers, dryers, air handlers, and most fans fall into this category, as does most electrical equipment. Note that while these observations are specifically for the structural performance of anchored equipment, they often are true for their operational performance as well - unless electrical relays are tripped or instrumentation controls are set to automatically shut down equipment. Where operational considerations are more of a concern, as is the case for telecommunication and computer equipment, engineers often specify much more stringent criteria than would be required by any building code.

Operational criteria for equipment are beyond the scope of this document, but the practice of a west coast telecommunications company in UBC Seismic Zone 4 may be instructive. It requires shake table testing of telecommunications and computer equipment to an input acceleration of $1g$ (where $g$ = gravitational acceleration) in both the horizontal and vertical directions. Such testing is used by numerous equipment manufacturers and often governs the anchorage requirements for the equipment.

Past earthquake experience has also demonstrated that static equipment that is properly supported and adequately anchored against normal sliding and overturning moment (such as small heat exchangers, chillers, pumps, and small shop-fabricated boilers and condensers) may not require an explicit design for seismic forces. Nevertheless, seismic loads are still commonly included in engineering design criteria.

The UBC requires special seismic provisions for anchoring "life-safety" equipment supported in a structure in the form of a multiplier called the "importance factor" $I$. Facilities such as hospitals, fire stations, police stations, emergency communication facilities, and facilities housing sufficient quantities of toxic or explosive substances that could pose a danger to the general public if released are considered "Essential Facilities" or "Hazardous Facilities." These facilities require a multiplier of 1.25 with no reduction if the equipment is self-supported at or below grade. For cases not described above, $I$ is to be taken as 1.0.

3.1.1.5.b Equipment supported by structures - The UBC requires a higher degree of strength for anchoring equipment to structures than is required for the design of the structures themselves. This is because equipment supported above ground level typically: (1) has higher absolute accelerations than at ground level, (2) can be subjected to amplified responses, (3) has little redundancy or energy absorption properties, and (4) is more susceptible to attachment failures, thereby becoming a higher risk component.

Rigid equipment not directly supported at or below grade would typically be identified by the code as "nonstructural components supported by structures."
includes most pumps, motors, and skid-mounted components. For these, the minimum lateral force requirements are determined by the formula:

\[
F_p = Z I_p C_p W_p \quad \text{[UBC Formula (36-l)]} \tag{3-3}
\]

where:

- \( F_p \) = lateral seismic force
- \( z \) = seismic zone factor for effective peak ground acceleration (ranges from 0.075 to 0.40, depending upon geographic location)
- \( I_p \) = importance factor for components
- \( C_p \) = horizontal force factor for the specific component (0.75 in most cases, but 2.0 for stacks supported on or projecting as an unbraced cantilever above the roof more than one-half the equipment’s total height)
- \( W_p \) = weight of the component

If an importance factor equal to 1.0 is required, the minimum lateral force requirement for Seismic Zone 4 is \( 0.3W_p \). Only if the rigid equipment consisted of unbraced cantilevers extending above the roof more than one-half the equipment’s total height would the requirement be greater - \( 0.8W_p \) (see Table 3.1.1.5a). For nonrigid or flexibly supported equipment the minimum lateral force is determined by the same formula. The force factor \( C_p \), however, must consider both the dynamic properties of the component and the structure that supports it. In no case should this be less than \( C_p \) for rigid equipment, though it need not exceed 2.0. In lieu of a detailed analysis to determine the period for nonrigid equipment, the value for \( C_p \) for rigid equipment can be doubled, resulting in a \( C_p \) of 1.5. This simplification is generally used by practicing engineers. Thus, unless an importance factor greater than 1.0 is required, the minimum lateral force requirement for Seismic Zone 4 would be \( 0.6W_p \) for most nonrigid equipment. Only if the nonrigid equipment consists of unbraced cantilevers extending above the roof more than one-half the equipment’s total height would the requirement be greater - \( 0.8W_p \) (see Table 3.1.1.5a).

3.1.1.5c Equipment supported at or below grade -
If the rigid or nonrigid equipment is supported at or below ground level, the UBC allows two-thirds of the value of \( C_p \) to be used:

\[
F = Z I_p (0.67) C_p W_p \quad \text{[Adapted from UBC Formula (36-l)]} \tag{3-4}
\]

as long as the lateral force is not less than that obtained for nonbuilding structural systems as given in UBC Section 2338 (b). These forces are described in the next section.

3.1.1.5d Self-supporting structures other than buildings - Formula (38-l) as given in UBC-91 2338 (b), applies to all rigid nonbuilding structural systems and all rigid self-supporting structures and equipment other than buildings. This would include such equipment as rigid vessels and bins.

\[
V = 0.5ZW \quad \text{[UBC Formula (38-l)]} \tag{3-5}
\]

If the self-supporting structure is nonrigid (that is, \( f < 16.7 \) Hertz), as for tall slender vessels, most tanks on grade, and some elevated tanks and bins, the dynamic properties must be considered and the UBC prescribes using the lateral force formula for “other nonbuilding structures” with some modifications:

\[
V = ZIC W \quad R_w \quad \text{[UBC Formula (34-l)]} \tag{3-6}
\]

where:

- \( c = \frac{1.25 S}{T^{1/3}} \) Amplification coefficient (need not exceed 2.75)
- \( I = \) importance factor (either 1.0 for standard and special occupancy structures, or 1.25 for essential and hazardous facilities) [See UBC Table 23-L]
- \( R_w = \) numerical coefficient for nonbuilding type structures (either 3, 4, or 5, depending upon type) [See UBC Table 23-Q]
- \( S = \) site coefficient for soil characteristics (ranges between 1.0 and 2.0, depending on site soil conditions) [See UBC Table 23-J]
- \( T = \) fundamental period of vibration in seconds
- \( V = \) total design lateral force or shear at the base
- \( W = \) total seismic dead load (typically the operating weight of equipment)
- \( Z = \) seismic zone factor for effective peak ground acceleration (ranges from 0.075 to 0.40, depending upon geographic location) [See UBC Table 23-I]

The modifications or limitations include the following:

1) The ratio \( C/R_w \) shall not be less than 0.5.
2) The vertical distribution of the seismic forces may be determined either by static force or dynamic response methods, as long as the results are not less than those obtained with the static force method. (Note: Dynamic response methods are seldom used for equipment).
3) Where an approved national standard covers a particular type of nonbuilding structure, the standard may be used.

Although they would seldom apply to equipment, certain other restrictions as described in UBC 2338(b) for Seismic Zones 3 and 4 apply for Occupancy Categories III and IV (Occupancy Categories in UBC Table No. 23-K). The structure must be less than 50 feet in height, and
<table>
<thead>
<tr>
<th>Equipment or non-building structures</th>
<th>UBC formula</th>
<th>Typical examples</th>
<th>Minimum values (importance factor = 1.0)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Supported by structures and $W_p &lt; 0.25W$:</td>
<td>$F_p = ZI_p C_p W_p$ ( (36-1) )</td>
<td>Rumps, motors, skid mounted equipment, small heat exchangers</td>
<td>Zone 1: 0.06$W_p$  Zone 2A: 0.11$W_p$  Zone 2B: 0.15$W_p$  Zone 3: 0.23$W_p$  Zone 4: 0.3$W_p$</td>
<td>Minimum values increase 1.33 times for unbraced cantilevers, stacks, or trussed towers where $C_p = 2.0$</td>
</tr>
<tr>
<td>Rigid (T $\leq$ 0.06 sec) where $C_p = 0.75$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nonrigid (T &gt; 0.06 sec) where $C_p = 2 \times 0.75$</td>
<td>$F_p = ZI_p C_p W_p$ ( (36-1) )</td>
<td>Leg-mounted vessels &amp; equipment, stacks, or slender process columns</td>
<td>Zone 1: 0.11$W_p$  Zone 2A: 0.23$W_p$  Zone 2B: 0.3$W_p$  Zone 3: 0.45$W_p$  Zone 4: 0.6$W_p$</td>
<td></td>
</tr>
<tr>
<td>Supported at or below grade:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rigid (T $\leq$ 0.06 sec) where $C_p = 0.75$</td>
<td>$F_p = ZI_p C_p W_p$ ( (36-1) )</td>
<td>Pumps, motors, skid mounted equipment, heat exchangers</td>
<td>Zone 1: 0.04$W_p$  Zone 2A: 0.08$W_p$  Zone 2B: 0.1$W_p$  Zone 3: 0.15$W_p$  Zone 4: 0.2$W_p$</td>
<td>Lateral force cannot be less than that from Formula (38-l) in Section 2338 (b)</td>
</tr>
<tr>
<td>Nonrigid (T &gt; 0.06 sec) where $C_p = 2 \times 0.75$</td>
<td>$F_p = ZI_p C_p W_p$ ( (36-1) )</td>
<td>Leg-mounted vessels &amp; equipment, stacks, or slender process columns</td>
<td>Zone 1: 0.08$W_p$  Zone 2A: 0.15$W_p$  Zone 2B: 0.2$W_p$  Zone 3: 0.3$W_p$  Zone 4: 0.4$W_p$</td>
<td>Lateral force cannot be less than that from Formula (38-l) in Section 2338 (b)</td>
</tr>
<tr>
<td>Self-supporting structures other than buildings:</td>
<td>$V = 0.5 \frac{ZI W}{R_p}$ ( (38-1) )</td>
<td>Rigid vessels and bins</td>
<td>Zone 1: 0.04$W$  Zone 2A: 0.08$W$  Zone 2B: 0.1$W$  Zone 3: 0.15$W$  Zone 4: 0.2$W$</td>
<td>Based on forces distributed by UBC Formula (34-6)</td>
</tr>
<tr>
<td>Rigid (T $\leq$ 0.06 sec)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nonrigid (T &gt; 0.06 sec) (or where $W_p &gt; 0.25W$)</td>
<td>$V = ZIC W \frac{R_p}{R_w}$ ( (34-1) )</td>
<td>Tall slender vessels. tanks on grade, and some elevated tanks and bins</td>
<td>Zone 1: 0.07$W$  Zone 2A: 0.14$W$  Zone 2B: 0.18$W$  Zone 3: 0.28$W$  Zone 4: 0.37$W$</td>
<td>See Note 2. If Seismic Zones 3 and 4 the code prohibits or restricts numerous concrete structural systems, or imposes height limitations on others (see UBC Table 2.3-0)</td>
</tr>
</tbody>
</table>

1) See UBC Section 2334 (j) for vertical force requirements in Seismic Zones 3 and 4, and 2335 and 2336 for all zones.
2) Formula (34-l) may govern over (36-l) where $W_p > 0.25W$ because of vertical distribution of forces.
a $R_\text{c}$ = 4.0 must be used for design. Additionally, the UBC prohibits or restricts numerous concrete structural systems in the higher seismic zones [UBC 2334 (c)].

Using Formula (3-6) and an importance factor of 1.0, the minimum design lateral force or shear at the base for nonrigid nonbuilding structures would be $0.37W$ (see Table 3.1.1.5a).

### 3.1.1.5e Vertical seismic loads

No vertical earthquake component is required by the UBC for equipment supported by structures [UBC 2334 (j)]. For equipment with horizontal cantilever components in Seismic Zones 3 and 4, however, the UBC specifies a net upward force of $0.2W$ for that component.

If the dynamic lateral force procedure is used, the vertical component is two-thirds of the horizontal acceleration. However, since the dynamic force procedure has little or no application to most equipment, many engineers designing structures in Seismic Zones 3 and 4 conservatively use a vertical component of three-quarters or two-thirds of the horizontal component of the static lateral force procedure, combining it simultaneously with the horizontal component.

The UBC also cautions about uplift effects caused by seismic loads. Only 85 percent of the dead load should be considered in resisting such uplift. [UBC 2337 (a)].

### 3.1.1.6 Test loads

Most process equipment, such as pressure vessels, must be hydrotested when in place on its foundation. Even when such a test is not initially required, there is a good possibility that sometime during the life of a vessel it will be altered or repaired, and a hydrotest may then be required to meet the requirements of Section VIII of the ASME Boiler and Pressure Vessel Code. Therefore, most engineers consider it necessary that all vessels, their skirts or other supports, and their foundations be designed to withstand test loads. For the foundation, this consists of the weight of water required to fill the vessel.

### 3.1.1.7 Maintenance and repair loads

For most heat exchangers, maintenance procedures require that periodically an exchanger’s tube bundles be unbolted, pulled from the exchanger shell, and cleaned. The magnitude of the required pulling force, and the fraction 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.

Since the forces transmitted to a foundation from pulling an exchanger bundle are so uncertain and variable, the design forces used are often based on past experience and rule-of-thumb. Common criteria are to design for a longitudinal force that is a fraction of the tube bundle weight, 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.

### 3.1.1.8 Fluid surge loads

Many types of process vessels (reactors, catalyst regenerators, etc.) 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. However, because of the difficulty in defining either the magnitude or the dynamic characteristics of these forces, 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 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. For most vessels supported directly on foundations at grade, surge forces are small and are usually neglected.

### 3.1.1.9 Erection loads

Frequently, construction procedures and the erection and setting of equipment 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 prior to the installation of heavy internals or refractory lining. Once installed, these internals are categorized as part of a vessel’s permanent dead load. However, many practitioners feel it necessary to examine the situation that could exist for the interim weeks or even months prior to installation of this considerable internal weight. Design of a tall vertical vessel foundation may well be governed by overall stability against overturning, if it is required that the temporary light structure be capable of withstanding full design wind.

### 3.1.1.10 Buoyancy loads

The buoyant effect of a high ground water table (water table above bottom of 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 (see Section 3.1.2).
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 3.5), 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, most engineers will consider both “dry” (neglecting water table), and “wet” (including the buoyancy effects of a high water table) conditions when designing foundations.

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

1) Thermal loads-Thermal loads are sometimes considered as a separate load category, but were described earlier in the section on operating loads.

2) Impact loads-Impact loads, such as those due to cranes, hoists, and davits, are sometimes classified separately. Just as often they are classified (as described above) under live loads or, depending on the type of equipment, as operating loads.

3) Blast loads-Explosion and the resulting blast 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.

4) 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.

5) 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 will have little or no effect on the foundations. However, in the case of direct current transmission lines, in which the earth acts as the reference, a short circuit between the aerial conductors and the earth may result in very significant loads being applied to the supporting structures.

3.1.2 Loading conditions - Different steps in the construction of equipment, or different phases of its operation/maintenance cycle, can be thought of as representing distinct environments, or different “conditions” for such equipment. During each of these conditions, there can be one or perhaps 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.

3.1.2.1 Erection condition - The erection condition exists while the equipment or its foundation are still being constructed, and the equipment is being set, aligned, anchored or grouted into position.

3.1.2.2 Empty condition - The empty condition will exist after erection is complete, but prior to 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, or the equipment is removed from service or both. This condition usually does not include the direct effect of maintenance operations.

3.1.2.3 Operating condition - The operating condition exists at any time when the equipment is in service, or 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.

3.1.2.4 Test condition - The test 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.

3.1.2.5 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), as well as from gravity. The gravity load is usually assumed to be the dead (empty) load.

The duration of a maintenance condition is usually quite short, such as a few days. Therefore, environmental loads, such as wind and earthquake, are rarely assumed to act during the maintenance condition.

3.1.2.6 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. Often it is assumed that equipment subjected to severe upset loads may have to be shut down and repaired. Thus, it is not uncommon for ultimate strength to be used as the acceptance criteria for upset loads.

3.1.3 Load combinations - Codes usually specify which of the more common loadings should be assumed to act concurrently for building design. Industrial
TABLE 3.1.3a - REPRESENTATIVE LOAD CONDITIONS AND COMBINATIONS

<table>
<thead>
<tr>
<th>Load case</th>
<th>Condition</th>
<th>Load combinations</th>
<th>Range of load factors*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Erection</td>
<td>Dead load + erection</td>
<td>1.1-1.5</td>
</tr>
<tr>
<td>2</td>
<td>Erection</td>
<td>Dead load + erection + ( \frac{1}{2} ) wind</td>
<td>1.2-1.3</td>
</tr>
<tr>
<td>3</td>
<td>Empty</td>
<td>Dead load + wind</td>
<td>1.3-1.5</td>
</tr>
<tr>
<td>4</td>
<td>Empty</td>
<td>Dead load + seismic</td>
<td>1.4-1.6</td>
</tr>
<tr>
<td>5</td>
<td>Operating</td>
<td>Dead load + operating load + live load + thermal expansion + surge + piping forces</td>
<td>1.6-1.7</td>
</tr>
<tr>
<td>6</td>
<td>Operating</td>
<td>Dead load + operating load + live load + thermal expansion + surge + piping forces + wind</td>
<td>1.3-1.5</td>
</tr>
<tr>
<td>7</td>
<td>Operating</td>
<td>Dead load + operating load + live load + thermal expansion + surge ( \pm ) piping forces + seismic</td>
<td>1.4-1.6</td>
</tr>
<tr>
<td>8</td>
<td>Test</td>
<td>Dead load + hydrotest</td>
<td>1.1-1.5</td>
</tr>
<tr>
<td>9</td>
<td>Test</td>
<td>Dead load + hydrotest + ( \frac{1}{2} ) wind</td>
<td>1.2-1.3</td>
</tr>
<tr>
<td>10</td>
<td>Maintenance</td>
<td>Dead load + bundle pull (heat exchanger)</td>
<td>1.4-1.6</td>
</tr>
<tr>
<td>11</td>
<td>Maintenance</td>
<td>Dead load + maintenance/service</td>
<td>1.4-1.6</td>
</tr>
<tr>
<td>12</td>
<td>Upset</td>
<td>Gravity + malfunction loads</td>
<td>1.0</td>
</tr>
</tbody>
</table>

* Load factors may vary. See Sections 3.1.3 and 3.1.4.

equipment, primarily because of the many possible variations in operating loads, can have a far greater number of possible load combinations. Often several different load combinations are possible within a given load condition. Judgment, not codes, must be used to decide which loads and corresponding load factors can reasonably be expected to act concurrently. Table 3.1.3a gives a list of twelve representative load combinations. With some variations among different practitioners, these combinations are the ones most commonly used to design industrial equipment and machinery foundations.

3.1.4 Load factors - Soil pressures and resistance to overturning are calculated by most practitioners using a series of load combinations similar to those listed in Table 3.1.3a with the individual combined loads at the “working” or in “service” level (unfactored loads).

When it comes to analysis of a foundation, however, it is not always clear which load factors apply to the many loads and load combinations, particularly those that include “nonstandard” loads peculiar to industrial equipment. Most engineers, since they do not have a recognized or legal criteria to cite, feel obliged to conform to the building code. They group the many loads unique to equipment under the common building code categories of “dead” and “live,” and directly apply the code’s prescribed load factors.

Other engineers contend that there are significant differences between loads applicable to equipment foundations, and those applicable to the design of commercial or residential buildings. They conclude that these differences warrant departures from a literal application of common building code load factors. Differences include the relative magnitudes of the different loads, and differences in their durations. These considerations, taken together, lead many engineers to select load factors that, although they may look similar to those in ACI 318, do contain important departures.

The factored loads are applied as follows: (1) Factor the loads at the top of the pedestal, (2) factor the service moments and shears in the footing, and (3) factor the differences between multiple analyses. These different approaches are further explained in Sections 4.1 and 4.7. If different load factors are to be used on the individual contributing loads in a combination, and if compression over the full width of the footing is not required, then these different approaches will give different results. This results from the fact that when the resultant load is outside the kern, the maximum soil pressure is not a linear function of the loads. Therefore, to avoid this possible confusion, some engineers apply a single composite load factor to all the loads in the entire load combination, rather than a different factor to each individual load.

Table 3.1.3a provides the range of load factors that is commonly applied to the listed load combinations. These may be single factors used for the entire combination or, where different factors are used for the various contributing loads, they may be the average ratio of total factored load to total service load.
3.2-Design strength/stresses

In the design of foundations, forces and stresses in the various elements must 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 to be compared. Other criteria are expressed in terms of a design strength to which calculated loads are to be 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, 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 strength design procedure for proportioning concrete elements is referred to as Strength Design Method (SDM). The working stress procedure is now called the “Alternate” Design Method (ADM) in the current ACI 318, and appears in Appendix A therein.

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

3.2.1 Concrete

3.2.1.1 Bending - The flexural (bending) capacity of concrete elements in a foundation for static equipment is usually determined using design criteria contained in ACI 318. These criteria from ACI 318 appear in the following table:

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>SDM</td>
<td>$M_u$</td>
<td>0.85</td>
<td>0.9</td>
</tr>
<tr>
<td>ADM</td>
<td>$M_w$</td>
<td>0.45</td>
<td>-</td>
</tr>
</tbody>
</table>

where:

- $f_b$ = extreme fiber stress in compression due to bending
- $M_u$ = factored moment
- $M_w$ = moment

The factor $\phi$ is the strength reduction factor, to take into account the probability that an element may be unable to perform at nominal strength due to inaccuracies and adverse variations in material strength and workmanship during construction.

3.2.1.2 Flexural shear - Concrete shearing stresses are of two general types. Where the foundation member is long relative to its width, or the pedestal dimensions are a significant fraction of the pad dimensions (say more than one-third), or both, then the most critical diagonal tension stresses occur at approximately a distance $d$ from the support pedestal. The quantity $d$ is the effective depth of the concrete foundation pad, measured from the extreme compression fiber to the centroid of the tension steel area. In this case, the stress state is termed “wide beam shear,” or simply “beam shear.” As previously indicated, the present trend is toward the use of strength design and the use of factored loads (moments) in proportioning concrete elements. The normally used stress criteria prescribed by ACI 318 are as follows:

$$V = \frac{V_u (bd)}{2f_c'^2} \phi$$

where

- $V_u$ = total working shear on the section through the foundation pad
- $V_u = \phi x V_c$ is the factored total shear
- $V_c$ = nominal limiting or allowable shear strength
- $b$ = section width located at a distance $d$ from the supporting face.

Although most engineers use the ACI 318 criteria described in the previous paragraphs without modification, some practitioners choose to use Ferguson and Rajagopalan. These authors point out that the code criteria for ultimate beam shear stress are significantly nonconservative for low percentages of reinforcement, with reductions in shear capacity approaching 50 percent for foundations with minimum steel. The authors recommend a reduced value for beam shear resistance for flexural sections where the tensile reinforcement ratio is less than 0.012. The following equation for determining the design nominal shear stress $v_c$ is suggested.

$$v_c = (0.8 + 100 \rho) \sqrt{f_c'} < 2 \sqrt{f_c'} \quad (3-7)$$

where:

- $\rho$ = ratio of nonprestressed tension reinforcement
- $v_c = \frac{V_u}{bd}$

Most foundations have reinforcement ratios less than 1 percent. Many equipment foundations have reinforcement ratios less than 0.5 percent. Thus, some engineers use values for design beam shear stress reduced from ACI 318 criteria in accordance with the recommendations of Ferguson and Rajagopalan.

3.2.1.3 Punching shear (two-way shear) - When a foundation pad or pile cap is square, or nearly so, or the pedestal dimensions are small relative to the main foun-
dation member (pad or pile cap), or both, then a shearing stress state different from the one described in Section 3.2.1.2 usually becomes critical. This alternative shearing failure mode occurs when a small pedestal tends to punch through its supporting foundation pad. The diagonal tension stress for this shearing stress state is aptly termed “punching shear.” The critical section, \( b_o \), for this potential failure mode is taken at a distance \( d/2 \) from the supporting face. For heavily loaded piles in a cluster, consideration for possible misalignment during pile driving should be included in the calculation.

The normally used stress criteria from ACI 318 are as follows:

\[
\begin{align*}
V & = V_u (2 + 4/\beta_c) < 4.0 \quad 0.85 \\
& \text{or} \quad V_u \frac{a_s d}{b_o} + 2 < 4.0 \\
A & \text{ M} \quad V_w \quad (1 + 2/\beta_c) < 2.0
\end{align*}
\]

where:

- \( V_u \) = total factored shear force at the critical section
- \( V_w \) = total working shear force at the critical section
- \( V_c \) = allowable or limiting shear strength at the critical section
- \( \beta_c \) = ratio of longer to shorter pedestal dimension, \( \beta_c = 1.0 \) for round or octagonal pedestals
- \( a_s \) = 40, but reduced to 30 if the pedestal is off-centered

Although ACI 318 allows some refinements of these relationships when shear reinforcement is added, such reinforcement is rarely used in equipment foundations.

The discussions of shear in concrete foundations in this and the previous section are directed toward individual footings. ACI 318 is unclear as to the appropriate shear stress criteria for mat foundations. However, most practitioners use the punching shear provisions when checking shear in such foundations.

### 3.2.1.4 Tension

ACI 318.1 permits plain concrete (unreinforced) spread footings. ACI 318.1 for plain concrete limits the use of plain concrete to footings that are continuously supported by soil or where arch action assures compression under all conditions of loading. However, unreinforced concrete spread footings are seldom used for equipment foundations, except for very small, minor equipment such as for residential air conditioner support pads. In the rare cases where unreinforced foundations are used, the maximum concrete tensile stresses permitted by ACI 318.1 are as follows:

<table>
<thead>
<tr>
<th>SDM</th>
<th>ADM</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_u )</td>
<td>( M_w )</td>
</tr>
</tbody>
</table>

\[
\begin{align*}
& SDM \quad M_u \quad f_i \sqrt{f_c} \quad \phi \\
& ADM \quad M_w \quad 1.6 \quad -
\end{align*}
\]

where \( f_i \) = extreme fiber stress in tension.

Foundations are often subjected to overturning moments large enough to produce uplift over a portion of their base. Since soil cannot resist uplift by tension, this results in a zone of zero pressure, with the resulting triangular pressure prism shown in Fig. 4.7.3. In the absence of upward soil pressure, a negative bending moment can be produced in the cantilevered portion of the footing which must be resisted by tensile forces in the top of the pad. This negative moment is limited to the full gravity weight of the uplifted part of the footing, plus any overburden or surcharge components, regardless of the magnitude of the applied overturning moment.

The tensile capacity of concrete should not be utilized in a seismic zone, or when a footing is supported by piles (UBC). However, there are differences of opinion and practice concerning treatment of overturning forces causing a negative moment in a spread footing in a non-seismic zone.

When the magnitude of this reversed or negative moment is small, some engineers use the allowable concrete tensile stresses given by ACI 318.1 for unreinforced footings to check the adequacy of the footing. Others consider the fact that a reinforced section subjected to positive moment develops cracks through as much as 80 percent of its thickness. Relying on such a cracked section for reversed bending (negative moment) is considered unsafe by many practitioners. Some engineers use top reinforcement if there is any calculated tension in the top “fibers” of the footing. Others, although aware of the uncertainty in the section’s capacity, are reluctant to provide a top mat of reinforcing steel to resist what is often a very nominal stress level. They may arbitrarily use the tensile capacity of the uncracked concrete section, but use only a fraction of the tensile stresses permitted by ACI 318.1 for unreinforced footings. The values used range from 20 to 50 percent of the nominal code values. Although there is reason to question the validity of this latter practice, there are no reported failures of footings designed with such an approach.

The above discussion of concrete tensile strength is often rendered academic by the use of minimum slab reinforcement in the top of a footing, provided ostensibly as temperature and shrinkage steel. There is no code requirement that, in the absence of calculated stresses, such reinforcement be inserted in the top of a foundation. However, some practitioners consider it good practice to always have a top mat of steel.

### 3.2.1.5 Bearing

The allowable bearing stresses on concrete contained in the current ACI 318 reflect recent studies showing that a triaxial state of stress is produced in the concrete in the zone beneath the base.
(bearing) plate. This effect is considerably more pronounced if the equipment or column base plate is centrally located so that the loaded zone is surrounded on all sides by concrete.

The allowable and design bearing stresses permitted by ACI 318 are as given in the following table:

<table>
<thead>
<tr>
<th>Function</th>
<th>Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>SDM</td>
<td>( \phi f )</td>
</tr>
<tr>
<td>ADM</td>
<td>0.35 ( f'c' )</td>
</tr>
</tbody>
</table>

where \( f \) = bearing stress.

When \( A_1 > A_n \), the design bearing strength may be multiplied by \( \sqrt{A_2/A_1} \leq 2.0 \).

\[
A_1 = \text{area in bearing on concrete}
\]

\[
A_2 = \text{area of the largest frustum of a right pyramid or cone contained wholly in the foundation when the upper base is area } A_1 \text{ and the side slopes are 1 vertical to 2 horizontal.}
\]

When designing base plates and annular base rings for concrete bearing, many engineers use the strength design concepts as defined in ACI 318. However, particularly for equipment foundations such as vertical vessels and stacks, many engineers choose working stress criterion instead. There are two reasons for this departure from the normally accepted ACI approach. First, anchor bolt design is commonly based on a working stress criterion. The determination of required bearing area is an interrelated function of the anchor bolt area provided. Therefore, a desire for consistency leads many engineers to use an allowable working stress for bearing.

The second reason is that design of equipment base plates and base rings is performed by equipment designers. Equipment designers are usually mechanical engineers with little or no experience in concrete design or in the strength design concepts of ACI 318. The need to simplify communication of design criteria, points toward the selection of working stress criteria for concrete bearing.

When working stress criteria are selected for the design of equipment base plates, the allowable stresses specified in the AISC-ASD specification, Chapter J9, are usually used. This is because equipment manufacturer’s engineers are usually familiar with this specification.

One question that arises in the design of vertical vessels and stacks that are supported on annular base rings is that the bearing area is not centrally located in the pedestal. Rather, the most heavily loaded area is immediately adjacent to the edge of the concrete pedestal. This fact leads many engineers to neglect the area ratio increases in the allowable stress.

### 3.2.2 Reinforcement

#### 3.2.2.1 Vertical reinforcement

- The vertical reinforcement in foundation pedestals is, for most types of equipment, 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 usually employed. For pedestals with a height-to-lateral dimension ratio of 3 or greater, the required reinforcement should be not less than minimum reinforcement applicable to columns. However, for equipment such as tall vertical vessels, the purpose of the vertical pedestal bars is to lap the anchor bolt anchorage zone (see Fig. 4.2.1c), and to transfer the anchor bolt tensile forces from a pedestal into the footing or pile cap. In this situation, practice for defining the appropriate acceptance criteria for designing the vertical bars varies widely. Some engineers design the pedestal bars using the total concrete section as described above. Some use a practice similar to that used in designing anchor bolts. They proportion the vertical bars either to resist the calculated anchor bolt tensile forces, or to match the design capacity of the anchor bolts, ignoring the concrete.

Still other practitioners replace the yield strength of the equipment anchor bolts with an equivalent or greater yield strength in the lapping vertical reinforcement, again ignoring the concrete section. This latter practice is used primarily in seismically active areas, the rationale being that initial yielding should take place in the more visible anchor bolt before the reinforcement to which the primary anchorage forces must be transferred.

#### 3.2.2.2 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. However, there are a number of circumstances 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 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.

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 near the top of the pedestal, closely spaced at 3 to 4 in. (see Fig. 2.2.4). This closely

* A one-third increase is permitted for wind and seismic loads.
spaced top set of peripheral reinforcement is to assist in resisting cracking due to edge bearing on the pedestal or to thermal expansion, as well as to provide confinement for resistance to shear. This practice reduces cracking of the concrete near the top of the pedestal due to transfer of shear forces through the anchor bolts into the concrete.

Sometimes, horizontal reinforcement is 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 the load is applied at the edge or periphery of the pedestal. In this situation, the pedestal could tend to dish upwards, and there would be a calculated 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 assure that it is a buildable design.

Design of horizontal reinforcement in footings (or pile caps) uses ACI 318 criteria for flexural reinforcement. The only questions that arise concern minimum amounts of reinforcement, as outlined in Section 4.7.5.

3.2.3 Anchorage - Anchorage of a piece 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 other equipment foundation where consideration of lateral loads dominates the design. ACI 355.1R summarizes the most widely used types of anchors and provides an overview of anchor performance and failure modes.

Anchors can be either cast-in-place or retrofit. Retrofit anchors are installed after the concrete has hardened, and can be either undercut, adhesive, grouted, or expansion.

- An undercut anchor transfers tensile load to the concrete by 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 rod installed in a hole with a diameter of about 1/4 to 1/6 in. larger than the diameter of the rod. The hole is filled with a structural adhesive such as epoxy, vinyl ester, or polyester. Adhesive anchors transfer tensile load to the concrete by bond of the epoxy to the concrete along the embedded length of the anchor.
- A grouted anchor consists of a headed anchor installed in a hole with a diameter about 1½ in. larger than the diameter of the anchor. 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 along the grout/concrete interface.
- Expansive anchors transfer tension load to the concrete by friction between the anchor and the concrete. The friction force results from a compressive reaction generated in opposition to the movement of an expansion mechanism at the embedded end of the anchor.

Normally, adhesive anchors have higher allowable load values than mechanical anchors. The selection of a retrofit anchor would depend 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.

A cast-in-place anchor is cast into the fresh concrete. The tensile load is transferred to the concrete either through bearing on the head of the embedded anchor, or through bond strength between the anchor and the concrete. The results of the latest research recommend using headed anchors rather than the “J” or “L” bolts, which depend upon bond.

3.2.3.1 Allowable stresses - Allowable stresses for retrofit anchors are based on the results of tests conducted by the manufacturer of the particular anchor. Although some manufactured expansion anchors are capable of developing the capacity of their bolt stock, most are designed using allowable loads much lower than would be determined by the strength of the bolt metal. Commonly, safety factors of four to five relative to pull-out are used to determine an allowable load for retrofit type anchor bolts.

Cast-in-place anchor bolts are usually designed to develop applied tensile forces, up to and including the capacity of the bolt, with appropriate safety factors. The amount of embedment is dependent on concrete strength, edge distance, and bolt spacing. The design practices that are used to insure adequate anchorage are described in Section 3.2.3.2. Most commonly, cast-in-place anchor bolts are sized using the allowable stresses specified by the AISC-ASD specification. In the AISC-ASD specification, both the allowable stress and, in the past, the effective area vary with the specific material. For example, anchor bolts fabricated from ASTM A 307 material commonly have been designed using the AISC specified allowable stress of 20 ksi together with the corroded “tensile-stress” area of the threaded bolt stock. The corroded tensile-stress area \( A_t \) is usually defined as follows:

\[
A_t = 0.7854 \left( D - \frac{0.9743}{n} \right)^2
\]  

where:

- \( D \) = nominal bolt diameter in in.
- \( n \) = number of threads per in. (the reciprocal of the thread pitch)

A corrosion allowance may be required and it should
be added to the required bolt area. It will vary with both location (seacoast versus inland, etc.) and the possibility of spills of acids or other chemicals. Such values commonly range from \( \frac{1}{4} \) to \( \frac{3}{4} \) in.

The AISC design specification permits stresses to be calculated on the nominal body or shank area of bolts and threaded parts (AISC-ASD Specification, Section 1.5.2). However, designers of equipment foundations prefer greater conservatism in anchor bolt design than that used in the design of other foundation components. For example, when designing anchor bolts, many engineers do not take advantage of the one-third increase in allowable stress that is normally permitted under temporary loads such as wind and earthquake. Similarly, many engineers, perhaps acknowledging the possibility of dynamic loadings, use the bolt tensile area rather than the larger shank area, when calculating the effective bolt tensile capacity.\(^1\)\(^2\)\(^4\)

ACI 349 uses strength design where the service loads for the anchors are factored. A strength reduction factor for the steel and concrete is consistent with AISC-LRFD, and ACI 318.

Occasionally, higher strength bolt materials are used in the design of anchor bolts for equipment foundations. However, the high material cost of high-strength bolts and the greater complications of attaching them to the equipment being anchored makes their use the exception. For example, if high-strength anchor bolt material is used, a special design of the equipment’s bolt anchor lugs might have to be performed. Any special design which 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 will occur by yielding and fracture of the steel when loaded in direct tension. Higher strength bolts would require more embedment in concrete to reach their capacity. Because there is insufficient test data on the ductility of high strength anchors, ACI 349 recommends a yield strength \( f_y \) greater than 120,000 psi not be used.

Also, particularly for tall vertical vessels and stacks, the lesser ductility of high-strength anchor bolts often rules against their use in seismic areas. This is because the common assumption that ductile behavior is desirable in large earthquakes, leads to the conclusion that the primary source of energy absorption will be in yielding of the vessel’s anchor bolts.\(^8\)\(^1\)\(^1\)

### 3.2.3.2 Anchorage criteria

In the past, there have been wide variations in the criteria used for the design of the embedded portion of cast-in-place anchorages which attach equipment to their foundations. Prior to 1975, many practitioners used the relatively low allowable loads on anchor bolts (both tension and shear) contained in the Uniform Building Code. The allowables contained in the UBC cover only headed bolts \( 1\frac{1}{4} \) in. in diameter and smaller. These allowables were originally based on minimal test data on bolts \( \frac{3}{8} \) in. in diameter and smaller, and are appropriate for nominal embedment lengths in unreinforced concrete sections.

In the absence of more definitive criteria, some engineers have extrapolated UBC values. They have calculated bolt capacities using a variety of approaches. These have included using an allowable bond stress on the bolt shank (ACI 318), or a code allowable bearing stress on the anchorage head (usually a plate or washer or both). Configurations used have included either hooks (“L” or “J” type hooks), or a plate or washer at the anchor head.

The lack of accepted definitive criteria for the design of cast-in-place anchorages has been largely a result of the absence of reliable test data. However, since 1964, there has been a major increase in the amount of basic research in the area of anchorage to concrete.\(^1\)\(^2\)\(^4\)\(^6\)\(^7\)\(^9\)\(^10\)\(^13\)

Based on this relatively new data, several guides or suggested practices have been published (ACI 349, PCI Design Handbook, and References 4 and 10). The Center for Transportation Research, The University of Texas at Austin, has published the following research reports: Research Report 1126-1, Research Report 1126-2, Research Report 1126-3 and Research Report 1126-4F.

In spite of the new data and newly suggested criteria, industry practice has changed slowly. First, many questions remain unanswered. Thus, a diversity of practices and opinions exist. Second, perhaps because full consensus has not yet been achieved on the appropriate criteria, model codes such as the UBC still have not updated their provisions. The lack of full consensus can also be explained by reviewing the series of tests referenced above.

The behavior of anchors depends on a number of variables, including the following:

- Loading (axial load, moment, 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
- Distribution of tension among the anchors
- Flexibility of the base plate
- Concrete strength
- Base plate configuration (embedded, flush, or on raised grout pad, important for anchorages subject to shear forces)
- Reinforcement in the foundation or pier
- Embedment length
- Edge distance and anchor spacing

Smooth bolts with hooks (“J” or “L” type bolts) have been fairly well discredited by the recent research. As a consequence, their use has declined substantially in recent years. The preferred configuration is now either a headed bolt or a threaded rod with a bearing plate or a nut, or both.

A report from the University of Texas (Research Report 1126-4F) states that headed anchors should have
dimensions equivalent to a standard bolt head or standard nut. Standard dimensions for bolt heads are given in ANSI B18.2.1. Standard dimensions for nuts are given in ANSI B18.2.2. Bearing at the anchor head does not require evaluation.

ACI 349 uses the following two methods of shear transfer:

- Bearing—In connections where the baseplates are mounted flush or above the concrete surface, the dominant mechanism of shear transfer is bearing on the anchor. Since the holes in the baseplate are usually oversized according to AISC recommendations, there is a question of how the plate goes into bearing against the anchor and how many anchors will actually transfer the load (see Fig. 3.2.3.2). Some engineers assume only half of the anchors actually transfer the shear load. Others only use no more than two bolts for shear transfer.

- Shear friction—Shear transfer is similar to the mechanism described in 11.7 of ACI 318. A friction force is generated by a clamping force that acts as a fractured shear plane in the concrete.

Though ACI 349 provides comprehensive procedures for anchor bolt design, there remains a considerable difference of opinion and practice in the provision for full ductile embedment for anchor bolts. ACI 318 contains one paragraph concerning ductility (15.8.3.3) that most engineers consider too vague. ACI 349 specifies that bolt embedment be provided for the bolt’s tensile and shear capacities, regardless of actual loads. This would assure a ductile connection. Some practitioners consider ACI 349 to be too conservative and provide anchorage based on actual strength. Others, using UBC criteria, design anchorages based on factored loads. If the bolt embedment is designed for the applied loads, the anchor should not be considered a ductile connection.

The required embedment for a headed bolt (or bolt with a nut) is calculated assuming a frustum of a 45-deg pullout cone emanating from the anchor head to the free concrete surface. A uniform nominal (tensile) stress of \(4\sqrt{f'_c}\) (with \(f'_c\) in psi) acting on the projected area of this cone on the concrete surface is recommended as a design capacity under factored load. Interference or overlap with concrete edges or cones from adjacent bolts is deducted from this effective stress area. A criterion is provided for computing the edge distances required for resisting both tensile and/or shear forces.

One other aspect of anchorage that merits mention is that of sleeving of anchor bolts. Here again, practice varies and some practitioners do not use sleeves in the foundations for static equipment. Others insist on their necessity, but use a variety of types and configurations.

The primary purpose of sleeving an anchor bolt is to ease the alignment of the bolt with the home in the baseplate of the equipment. Sleeves may be constructed of pipe, sheet metal, high-density polyethylene or a hole formed using Styrofoam. After installation of the equipment, the sleeves are usually filled with grout. However, some engineers, particularly those who design a post-tensioned type bolt, will specify a grease or mastic type filler for the sleeve.

When a bolt is fully embedded in concrete, a bond breaker to increase ductility of the bolt can be achieved by wrapping the upper portion of the bolt with duct tape or insulation.

Some engineers specify double nuts for anchors under tension to prevent backoff.

3.2.4 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). However, it is worth noting that besides settlement considerations, 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).

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. However, some geotechnical engineers will include the lateral resistance of passive pressures to a certain degree, consistent with allowable lateral movement, if a certain depth of backfill finish grade is ignored in its 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. However, if the 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.

3.3-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, ducting, chutes, conveyor belts, etc., may have to be controlled to avoid over stressing 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. Some equipment may require precise alignment for its proper operation. However, as a rough order of magnitude, long-term settlements of 1/2 in., or short-term lateral movements (such as under wind load) of 1/4 in. 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 a modest flexibility.

3.4-Stability

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

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 high wind or seismic loads. The size of foundations for tall vessels and stacks are 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 level, 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:

\[
SR_1 = \frac{PD/2 + Mp}{M} \quad (3-10)
\]

where:

- \( P \) = vertical load due to weight of concrete and equipment
- \( M \) = overturning moment applied to footing
- \( D \) = edge to edge distance of footing in direction of overturning moment
- \( e \) = \( M/P \)

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 isolated foundations supporting a piece of equipment such as a heater, on separate spread footings, a portion of the overturning moment can be resisted by vertical forces at each footing. This is a situation of combined uplift and overturning stability. The stability ratio is then determined by computing separately the footing edge moments due to dead weight and wind or seismic.

Generally, a foundation will most probably fail in other modes before overturning. Some practitioners include soil failure considerations in accordance with published design recommendations of ACI 336.2R and ACI 336.3R. Most practitioners, however, use the simpler methods described above as a general indication of the factor of safety against overturning.

Although the concept of a stability ratio is quite 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 and is the minimum specified by UBC, SBC, and BOCA for wind loads. None of these building codes specifies a minimum stability ratio for seismic loads.

For pile foundations, the concept of a stability ratio is straightforward where the piles are not designed to resist uplift. The center of moments is taken at the most leeward pile. However, when the piles have a tension capacity, the concept becomes ambiguous and is seldom used.

For drilled pier foundations, the procedure for using the stability ratio is unclear and many different practices prevail. For example, since 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:
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 of the above definitions of stability ratio would be more meaningful for a drilled pier (particularly a straight shaft) whose diameter is relatively small compared to its depth, and that relies predominantly on the lateral soil pressure (pole action) for its resistance to overturning. However, the second definition might be appropriate for a large diameter shallow drilled pier whose major resistance to overturning is the size of the bell.

**CHAPTER 4-DESIGN METHODS**

4.1-Available methods

Foundations for static equipment are generally designed by either the Strength Design Method or the Alternate Design Method (formerly termed the “Working Stress” method as defined by ACI 318). While practitioners in general have adopted the Strength Design Method, there are still many engineers who use the Alternate Design Method. Such usage persists, largely due to the familiarity gained through many years of use.

A third design method that has yet to achieve formal recognition or endorsement by an American code-writing body is Limit Design. Limit Design in reinforced concrete parallels Plastic Design in structural steel. Its most significant feature is its reduction of complex analysis problems to relatively simple yield line problems. This method is sometimes utilized in the design of complex foundations which present complicated elastic analysis problems. The Limit Design Method is not covered in this chapter.

4.2-Anchor bolts and shear devices

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 to transfer tensile forces or a combination of tensile and shear forces. When required, shear lugs may be used to transfer shear forces.

4.2.1 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. 4.2.1a. Types “L” and “J”, which are cast-in-place bolts, rely on bond to develop the capacities of the bolts. Types “P”, “N”, “H”, “PN”, “PH”, and “S”, which are also cast-in-place bolts, rely on the pullout strength of the concrete. Types “SD” and “DI” are, respectively, self-drilling and drop-in bolts relying on expansive forces to transfer the tension to the concrete or to the mechanical anchorage.

The various bolt types are generally carbon steel or low alloy materials and may be provided with sleeves.

For high-profile static equipment (height/diameter > 7), typically tall vessels and stacks, the proper determination of anchor bolt forces is a primary requirement of the foundation design. For such equipment, a more exact method is often used. For less critical installations the anchor bolts are assumed to comprise an annular ring of steel in a hollow concrete column section (see Fig. 4.2.1b). The neutral axis of the section shifts to where there is a condition of equilibrium between the steel and concrete. This procedure often results in a considerably reduced bolt requirement as compared to that computed using the force formula, but it requires more time and effort to apply.
Having determined the force(s) in the anchor bolts, one must determine the required area, select the type of bolt, and compute the required embedment. The area may be determined following the criteria given in Section 3.2.3. For the bolt type selected, the embedment is computed to satisfy pullout requirements or to transfer the tensile forces to vertical reinforcement from bolts cast in pedestals. When vertical dowels are used to transfer tensile forces to foundations, care should be exercised to assure that sufficient development length as shown in Fig. 4.2.1c is provided. Chapter 12 of ACI 318 should be used in determining development length for vertical bars in pedestals.

Embedment lengths for “L” and “J” type anchor bolts have been traditionally determined using bond stress provisions for plain bars taken from Chapter 18 of ACI 318-63. Edge distance and bolt spacing are most important in the design of anchor bolts that rely primarily upon the pullout strength of concrete (types “P”, “N”, “H”, “PN”, “PH”, and “S”, Fig. 4.2.1a).

4.2.3 Shear - Shear forces may be transferred by a variety of mechanisms: friction resulting from the clamping action provided as a result of tightening the bolt(s), shear lugs, or direct bearing of the anchorage against the concrete.

Table 26-E of the Uniform Building Code (UBC-91) can be used to design anchor bolts for shear. The tabulated values apply to headed anchor bolts (type “N” and “H”) cast in plain concrete.

Designs that rely on the anchor bolts to transfer shear through bearing on the sides of a baseplate or through shear plates welded to the bottom of the baseplate 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 baseplate and the small misalignments that occur between bolts, only a fraction of the bolts will bear simultaneously against the baseplate and hence be capable of transferring shear load (see Fig. 3.2.3.2).

4.2.3 Tension/shear interaction - When bolt tension and shear forces are present in an anchorage, the interaction of the two should be considered. Frequently, interaction relationships are used such as those specified by the AISG-ASD Section J3.6 (AISC-1989) for structural bolts. Recently, Cannon et al. have recommended that the areas of steel required for shear and tension be additive. Current thinking of ACI 349 recommends two methods for shear and tension on the anchor. For anchors that transfer shear by bearing, a linear tension-shear interaction is conservative. An elliptical tension-shear interaction is acceptable, but is more difficult to apply. Where shear friction is used, the required strength of the anchor is a sum of the tensile strength required for direct tension and the tensile strength required for shear friction.

4.3-Bearing stress

Portions of the foundation in contact with the equip-
ment base plates or mounting rings must be designed to comply with permissible bearing stresses given in 3.2.1.5.

4.4-Pedestals

In the design of equipment foundations, the piece of equipment may be located one or more feet 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.

By definition, a pedestal is a short column. They should be designed for the critical combination of vertical load and moment. Octagonal pedestals are usually 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 four methods: (1) supplying vertical reinforcement with a design capacity equal to or greater than that provided by the anchor bolts, (2) designing the pedestal as a column with the vertical reinforcement in tension and concrete in compression, (3) applying the combined stress formula to the reinforcement area alone, or (4) designing the pedestal as a flexural member, neglecting axial compression.

4.5- Soil pressure

4.5.1 Spread footings - Spread footings may be divided into two general categories: those subject to full bearing pressure where the resultant vertical force is within the kern of the base; and those subject to partial pressure where the resultant force lies outside the kern. 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:

\[ Q = (W/A) \pm (M_f/S_f) \pm (M_y/S_y) \]  \hspace{1cm} (4-2)

where:
- \( Q \) = soil pressure at the corners of the footing
- \( W \) = resultant vertical load
- \( A \) = base area of footing
- \( M_x, M_y \) = moments about the x and y centroidal axes
- \( S_x, S_y \) = section moduli of the base about x and y centroidal axes

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. If the overturning stability criteria are met, then the mathematical assumptions of the following formula are met.

For partial contact:

\[ Q = \frac{2W}{3B(L^2 - e)} \]  \hspace{1cm} (4-3)

where:
- \( e = M/W \)
- \( B \) = width of footing
- \( L \) = length

4.5.2 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 insure that the concrete will not be disturbed, pulled apart, or pinched off by earth movements (ACI 336.3R, Section 4.3.3). 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.

4.5.2.1 Base pressure and pier capacity - Vertical soil pressures for a long pier are resisted as skin friction on the surface of the shaft. Depending on whether or not the base of the pier rests on rock, the contribution of bearing pressures against the base (point-bearing) to the overall capacity of such piers may or may not be significant. Uplift resistance of long piers is usually a function of the skin friction and pier dead load.

For a short pier, the vertical force 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. 4.5.2. For a straight shaft pier, the vertical pressure on the base is assumed to be distributed over the leeward half of the base. For a belled pier, the base pressure is

![Fig. 4.5.2-Pressure distribution on drilled piers](#)
computed using the combined stress formula, assuming that the soil over the windward half of the bell would cause the bell to interact with the soil in the manner of a pile foundation with uplift capacity. Bell and shaft diameters are selected to keep vertical soil pressures within allowable values.

Uplift capacity of belled piers is usually taken as the weight of soil above the bell. One practice is to assume a cone with an angle of 45 to 65 deg with the horizontal. An alternative, more conservative approach considers only the cylinder of soil above the bell.

4.5.2.2 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 beam on nonlinear soil springs. Allowable lateral pressures are based on limiting the lateral displacement of the pier at the ground surface, typical values being \( \frac{1}{4} \) to \( \frac{1}{2} \) in.

In the case of short piers (those with a shaft length to diameter ratio less than 10), the pier is considered rigid, with the lateral pressures varying in the manner necessary to satisfy statics (see Fig. 4.5.2a). Where weathered soils exist at grade, the top two or three feet of soil are often ignored in determining the resistance to lateral loads.

4.5.2.3 Lateral deflection - Lateral deflections may be determined by treating the pier as a beam on nonlinear soil springs. In order to do so, a coefficient of horizontal subgrade reaction is selected based on soil consistency and published data, or on an actual field test.

4.5.2.4 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 will occur. Consequently, settlement is usually not computed for these soils. However, when piers are located in or underlain by weaker clays, a settlement analysis is required; in this case, standard consolidation theory may be applied.

4.5.3 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 as compared to 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.

4.6-Pile loads

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

When connectors are not used, the combined stress formula may be applied to determine vertical pile loads. If the piles are subject to uplift, 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.

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,\(^{12}\) 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 on to reduce the shear in the piles.

There are several sophisticated procedures for determining loads in pile groups. 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 thus 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.

4.7-Foundation design procedures

4.7.1 Factored loads - Foundation base area or the number of piles or piers is determined from service (unfactored) loads. Use of the Strength Design Method for the structural design of reinforced concrete foundation elements requires the application of factored loads. Owing to the difficulty of tracking the contribution of each type of loading (dead, live, wind, etc.), application of a single load factor is often used in the design of equipment foundations designed using the Strength Design Method. Typically, a composite load factor equal to 1.6 is used.

4.7.2 Positive moments and flexural 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 are as described in Chapter 15 of ACI 318 (see Fig. 4.7.2a). An exception to the ACI procedure occurs with deep/thick pile caps with high capacity piling (refer to CRSI Handbook).

Reinforcing design for octagonal foundations can be cumbersome given the mat shape and the reinforcement...
CRITICAL SECTION - MOMENT

Fig. 4.7.2a--Critical sections

CRITICAL SECTION BEAM SHEAR

Fig. 4.7.2b--Octagonal base options

CRITICAL SECTION - MOMENT

Fig. 4.7.2c--Octagonal bases - moment section options

CRITICAL SECTION BEAM SHEAR

Fig. 4.7.2d--Beam shear options

CRITICAL SECTION

Fig. 4.7.3-Negative moment

configuration (Fig. 2.2.1). Therefore, octagonal geometries are often converted to equivalent circular shapes as shown in Fig. 4.7.2b. An equivalent circular shape makes it easier to handle controlling load combinations which are not oriented on major octagonal axes. The design moment for an octagonal foundation generally determined in one of two ways (see Fig. 4.7.2c). In method “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 method “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. Of the two methods described above, the full width section requires the least reinforcement.

Beam shear for an octagonal foundation is generally determined as shown in Fig. 4.7.2d. In method “a,” the shear is computed on the area of the base octagon, bonded by the critical section at a distance \( d \) from the equivalent square pedestal, and 90 deg radial lines drawn from the center of the base extending through the corners of the equivalent square. Alternatively, in method “b” the shear is computed 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 (refer to ACI 336.2R).

4.7.3 Negative moments - For a spread footing with the resultant outside the kern, the footing is only partially subjected to positive soil pressure on the windward side. 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. 4.7.3, 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 not be ignored.

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.

4.7.4 Punching shear (two-way) - The critical section for punching shear is as described in Section 11.12 of ACI 318. An alternative procedure involves computing the shear on the heavier loaded half of the critical section as shown in Fig. 4.7.4.

For piers subjected to large moments in addition to vertical forces, the punching surface departs from a simple truncated cone or pyramid. The report ACI 426R should be consulted in such cases.

4.7.5 Flexural reinforcement - Minimum reinforcement requirements of ACI 318 have been variously interpreted where foundation design is concerned. Some engineers specify a minimum reinforcement of 200/f_y, unless one-third more reinforcement than required by analysis is provided. Others specify a minimum temperature or shrinkage reinforcement. Section 10.5.3 of ACI 318R recommends a minimum shrinkage and temperature reinforcement for mats and other slabs that provide vertical support. Assuming the category of “mats and other slabs” to include foundations, the provision of temperature and shrinkage reinforcement would appear to meet the code’s intent. On the other hand, the 200/f_y provision applies specifically to beams that have been oversized for architectural or other reasons. Consequently, most engineers do not consider the 200/f_y provision applicable to foundation design.

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. 4.7.2c, method “a.” For octagonal foundations, the flexural reinforcement is placed in mats as shown in Fig. 2.2.1. Where hexagonal foundations are used, similar configurations are typically provided.

4.7.6 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 1/2 percent minimum from Sections 10.8.4 and 1.9.1 of ACI 318. Others question this practice on the basis that pedestal areas associated with static equipment are generally much larger than those associated with building columns to which the ACI 318 provisions are primarily addressed.

CHAPTER 5--CONSTRUCTION CONSIDERATIONS

Foundations for static equipment are similar in configuration and construction to foundations for structures. In addition, foundations must meet any specific requirements of the equipment manufacturer for maintaining precise grade and alignment, as well as for transferring the loads from the equipment to the supporting structures or soil. For more massive equipment foundations, this may require rigid foundations supported by firm soils or rock.

Foundation mats may be supported directly by soil or rock, or piles or drilled piers may be used to extend the foundation to firm soil or rock. The selection of the most appropriate type of foundation depends upon the geotechnical conditions of the site. The extent of the subsurface investigation and resulting subsurface preparation, if any, is determined by the engineer and geotechnical consultant.

5.1- Subsurface preparation and improvement

5.1.1 General - 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 situ 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.

5.12 Specific subsurface preparation and improvements

- Specific subsurface preparation and related treatment may be required if the geotechnical investigation or excavation during construction indicates that the existing soil characteristics will not achieve the required foundation performance. Conditions requiring special preparation and treatment are:
  - Nonuniform conditions that could result in differ-
entailment or tilting of the foundation
- Soil conditions found to be different than those assumed for the design
- Unstable slopes
- Loose sands
- Soft compressible soils such as unconsolidated clays and highly organic soils (e.g., peat)
- Slip planes or faults
- High water table or other saturated conditions

The most common site specific subsurface preparations and treatment for the above conditions are:

a) Unstable slopes of excavation- Unstable slopes may be stabilized by flattening the slope, benching, dewatering, shoring, freezing, injection with chemical grouts, or supporting with dense slurries.

b) Stratification- Excavations with slopes parallel to the direction of stratification are avoided by flattening the slope or by providing adequate shoring.

c) Wet excavation- During construction, ground water is normally lowered below the bottom level of the excavation. One method commonly used to achieve this is by using deep well pumps or well points. 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 ground water intrusion.

The selection of an appropriate method depends on the characteristics of the subsurface soils encountered, costs, and the preferences of the constructor.

d) 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 lean concrete.

e) Large deposits of loose sands-- The loose sands may be stabilized by vibroflotation or dynamic consolidation, whichever offers an economic advantage.

f) 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 firm bearing strata.

g) Fissured rock- The extent of fissures is evaluated to determine if remedial treatment is needed. Pressure grouting is a suitable remedy for some types of fissures. In the case of seismic faults, thorough geotechnical and geological evaluation is required to ascertain the potential hazard. Where significant hazards are found to exist, relocation of the entire facility to avoid the hazard is a suitable remedy.

h) Irregularly weathered rock- The weathered seams are cleaned and replaced with lean concrete. Alternatively, the foundation may be lowered to sound rock.

i) Solution cavities in limestone deposits- The voids are pumped full of grout, if small, or lean concrete under a pressure head in the case of large holes.

j) 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.

k) Cold climates- Foundations are not placed on fine grained soils subject to the phenomenon of frost heave. Proper drainage should be provided by placing a free draining sand or gravel layer under the foundation to mitigate the possibility of frost heave where such hazard exists. As an alternate, the bottom of the footings is placed below the frost line.

5.2-- Foundation placement tolerances (ACI 117)

Foundation placement tolerances depend largely on the type of equipment being supported. They are specified by the engineer on the drawings or in the specifications. It is good practice to use templates during concrete placement to support anchor bolts and other embedments that must be precisely positioned.

5.3-- Forms and shores

5.3.1 Forms and shoring for construction of concrete foundations should follow the recommendations of ACI 347R.

5.3.2 Shoring must support the concrete loads, impact loads, and temporary construction loads. Transverse longitudinal bracing may be required to sustain lateral forces.

Wind loads should be taken into account. It is not usually necessary to consider seismic loads due to the limited time shoring will be in place. The design of the formwork should be prepared by a registered professional engineer and submitted to the design engineer for review.

5.3.3 For large equipment foundations, temporary formwork systems are generally used. Less frequently, permanent systems may be used for special applications. The selection of a temporary support system is normally made by the constructor. It is influenced by the erection 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 prior to deciding on a permanent formwork system.

Some of the temporary 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.

Permanent support systems include:

- Structural steel beams or trusses supported by foundation columns and carrying the permanent deck forms. The beams or trusses are part of the deck design and will also carry operating loads. The deck forms (steel decking) usually are supported on the bottom flanges of the beams or trusses. Since the steel members are embedded in the foundation deck, the design engineer has to be careful to avoid interferences with the reinforcing bars and with other embedments (anchor bolts, plates, pipesleeves, and conduits).
- Precast concrete deck forms supported by the foundation columns.
- Plate steel forms used in the steel industry.

The engineer should review the constructor’s proposed construction procedure to assure that the design is not compromised.

5.4-Sequence of construction and construction joints

Many equipment foundations are too large for the concrete to be placed in one continuous operation. Construction joints are used to subdivide large foundations into smaller units that can be placed in one continuous operation.

Subdivision of large structures by construction joints also affords a means for reducing stresses due to concrete shrinkage. To gain maximum benefit, alternate segments should be placed and allowed to cure and shrink as long as the construction schedule permits before the intervening segments are placed.

The structural integrity of the foundation requires that joints be constructed with care in accordance with accepted practices for construction joints in major concrete structures. Project specifications normally require that the constructor obtain the approval of the engineer for construction joint locations and details.

5.5-Equipment installation and setting

5.5.1 Shims, wedges, and bolts - The design engineer’s choice of the interface system is influenced by the manufacturer’s recommendations and requirements, the foundation construction procedures, the setting and adjustment of the equipment, and the final tolerances required.

Shims, which are usually carbon steel or brass stock in various thicknesses, have both economical and high load bearing qualities.

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 (1) precise vertical adjustment, and (2) 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 utilized by millwrights include various shaped temporary steel wedges. Temporary wedges are usually tolerance adjustment tools placed prior to grouting, and they are removed after the setting-up of the grout material. Permanent wedge assemblies allow future adjustments on ungруtted equipment bases.

Required bolt diameters are usually given on the manufacturer’s drawings. Bolt lengths, threaded lengths, bolt projections, material, stress levels, and the method of tightening should be clearly shown on the design drawings. When the manufacturer requires a specific preload for a bolt, the following equation can be used to select the bolt torque:

\[ T = W_p \mu d_n \]

where:

- \( T \) = tightening torque, in-lb
- \( W_p \) = initial load (at preload) lb
- \( \mu \) = friction factor
- \( d_n \) = nominal bolt diameter, in.

The following values of \( \mu \) are typical:

- Steel fasteners (as manufactured) 0.20
- Hot dipped galvanized steel 0.14
- Lightly oiled steel 0.15
- Plated (cadmium, chromium, etc.) 0.15
- Graphite with mineral oil 0.10

Special coatings may require manufacturer’s data.

When preload values are not given, a suggested minimum preload value of 15 percent of the yield strength of the anchor is often used.

Bolt tightening is specified as being accomplished with either a post-tensioning jacking procedure, turn-of-the-nut method, or with a calibrated wrench. Post-tensioning jacking is usually used on the deeper anchorages with the non-bonded shanks. When the shank length is embedded in concrete, the turn-of-the-nut method or sequential calibrated wrench tightening is specified. Impact wrenches are not allowed for tightening of a bolt component when part of the anchorage is embedded in concrete because of the extremely high torques and tensile forces delivered by such tools.

5.5.3 Embedments - Embedments in the concrete include the anchor bolt assemblies previously described, shear lugs, and shear transferring devices.

Since shear is one of the combined loads transferred to the concrete foundation, steel lugs may be integral parts of the base of the equipment. Such lugs are grouted into shear key grooves previously cast into the concrete base.

5.6-Grouting

5.6.1 Types of grout - There are two basic types of
grout: cement-based grouts and epoxy-based grouts. Cement-based grouts are more commonly used because of their availability, ease of use, strong physical properties and lower cost. Epoxy grouts are generally used because of their high resistance to chemicals, to shock, and to vibratory loads. There are four types of cement-based grouts: (1) gas generating; (2) air release; (3) oxidizing aggregate, and (4) expansive cement. In evaluating which cement grout should be used, one should take into account the placeability of the grout as well as its physical properties. The physical properties that are evaluated are: volume change, compressive strength, working time, consistency, and setting time. In evaluating the properties of an epoxy grout, one should look at placeability as well as the physical properties of volume change, compressive strength, creep, working time, consistency, and setting time. The effects of temperature induced volume changes on the epoxy concrete interface should be considered. In addition, any specific requirements of the application should be addressed.

5.6.2 Applications - In specifying grout systems, the designer should consider the different characteristics of each type of grout along with field limitations, and match these with the specific requirements of the job. In particular, the designer should review the design of the equipment base, the accessibility of the grouting location, the clearances provided for the grout, and the design of the anchor bolts. Most of the grouts on the market are pre-mixed, prepackaged materials, and contain manufacturer’s instructions on surface preparation, formwork, mixing, placing, and curing procedures.

A detailed discussion on the application of grouts can be found in ACI 351.1R.

5.7-Materials (ACI 211.1)

Large equipment foundations require special attention to the design and control of the concrete mix (see ACI 207.1R and ACI 207.4R).

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 and this may cause unacceptable surface cracking unless steps are taken to reduce the rate of release of this heat. Also, creep, differential thermal expansion, and shrinkage may cause distortion of the foundation and consequent unacceptable changes in equipment alignment. Design of the concrete mix to minimize creep and shrinkage, and to reduce the thermal expansion of the hardened concrete is therefore important. Finally, expansive reaction of the concrete aggregate with alkali in the cement can be avoided by proper choice of cement and aggregate.

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 used.
- Part of the cement is replaced with fly ash or non-fly ash pozzolan.
- The placing temperature of fresh concrete is lowered by chilling the aggregate and/or using chipped ice for mixing water.
- The largest practical size aggregate is used to allow further reduction in the amount of cement.
- Moderate heat cement (Type II) is used.
- A water reducing agent is used to allow further reduction of the cement factor.
- Low slump and effective vibration are used.
- Concrete placement by pumps, which requires concrete mixes having high amounts of cement and small aggregate sizes, is avoided.
- Sixes of placements for large foundations are reduced.

The coefficient of thermal expansion of the hardened concrete can be controlled by the choice of aggregates because it depends primarily on 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 country there may be very 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 C 618, and by selecting low reactivity aggregates. The potential reactivity of aggregates can be evaluated with the procedures and tests described in ASTM C 295, ASTM C 227, ASTM C 289, and ASTM C 586. The evaluation methods of the potential reactivity of aggregates are covered by ASTM C 33 and ACI 225R.

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

5.8-Quality control

Foundations for equipment should be parts of an integrated system and are designed as such. Thus, the design requirements should be implemented during construction by the imposition of an appropriate quality control program. 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 qualified field engineers responsible to the engineer and/or owner.

Requirements for foundations should be provided to the constructor and field engineer through the design drawings and project specifications. The field engineer should maintain close liaison with the design engineer on
any revisions to the design requirements when field conditions differ from those assumed.

The quality control program and inspection and verification activities should be thoroughly documented. The program should be consistent with those commonly implemented for construction projects of similar importance.

CHAPTER 6- REFERENCES

6.1-Recommended references

The documents of the various standards producing organizations referred to in this document are listed below with their serial designation.

American Concrete Institute (ACI)

116R Cement and Concrete Terminology
117 Standard Tolerances for Concrete Construction and Materials
207.1R Mass Concrete for Dams and Other Massive Structures
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
307 Standard Practice for the Design and Construction of Cast-in-Place Reinforced Concrete Chimneys
318/318R Building Code Requirements for Reinforced Concrete and Commentary
318.1/318.1R Building Code Requirements for Structural Plain Concrete and Commentary
336.2R Suggested Design Procedures for Combined Footings and Mats
336.3R Suggested Design and Construction Procedures for Pier Foundations
347R Recommended Practice for Concrete Formwork
349/349R Code Requirements for Nuclear Safety Related Concrete Structures and Commentary
351.1R Grouting for Support of Equipment and Machinery
355.1R State-of-the-Art Report on Anchorage to Concrete
426R Shear Strength of Reinforced Concrete Members

American Institute of Steel Construction (AISC)

AISC-ASD Manual of Steel Construction - Allowable Stress Design
AISC-LRFD Manual of Steel Construction - Load and Resistance

American National Standards Institute (ANSI)

ANSI AS8.1 Minimum Design Loads for Buildings and Other Structures (revised and redesignated as ASCE 7)
ANSI B18.2.1 Square and Hex Bolts and Screws (Inch Series)
ANSI B18.2.2 Square and Hex Nuts (Inch Series)
ANSI STS-1 Steel Stacks

American Society of Civil Engineers

ASCE 7 Minimum Design Loads for Buildings and Other Structures (formerly ANSI A58.1)

American Society of Mechanical Engineers (ASME)

ASME STS-1 Steel Stacks
ASME Boiler and Pressure Vessel Code

ASTM

ASTM A 307 Standard Specification for Carbon Steel Bolts and Studs
ASTM C 33 Standard Specification for Concrete Aggregates
ASTM C 289 Standard Tests Method for Potential Reactivity of Aggregates (Chemical Method)
ASTM C 295 Standard Guide for Petrographic Examination of Aggregates for Concrete
ASTM C 586 Standard Test Method for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock Cylinder Method)
ASTM C 618 Standard Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete

Building Officials and Code Administrators International, Inc. (BOCA)

The BOCA National Building Code

Center for Transportation Research (University of Texas)

Research Report 1126-1 Load-Deflection Behavior


Research Report 1126-3 Behavior and Design of Ductile Multiple-Anchor Steel to Concrete Connections; Cook, Klingner, Mar. 1989.


Concrete Steel Reinforcing Institute (CRSI)

CRSI Handbook

International Conference of Building Officials

Uniform Building Code (UBC)

FEMA, National Earthquake Hazards Reduction Program (NEHRP)

Standard (1991)

Precast-Prestressed Concrete Institute (PCI)

PCI Design Handbook

Southern Building Code Congress International, Inc.

Standard Building Code (SBC)

The above publications may be obtained from the following organizations:

American Concrete Institute
PO Box 9094
Farmington Hills, MI 48333

American Institute of Steel Construction
400 North Michigan Avenue
Chicago, IL 60611

American National Standards Institute
11 West 42nd Street
New York, NY 10036

American Society of Civil Engineers
345 East 47th Street
New York, NY 10017-2398

American Society of Mechanical Engineers
345 East 47th Street
New York, NY 10017

ASTM
1916 Race Street
Philadelphia, PA 19103-1187

Building Officials and Code Administrators International, Inc. (BOCA)
4051 West Flossmoor Road
Country Club Hills, IL 60478-5795

Center for Transportation Research (University of Texas)
3208 Red River, Suite 200
Austin, TX 78705-2650

Concrete Steel Reinforcing Institute
933 North Plum Grove Road
Schaumburg, IL 60173-4758

Federal Emergency Management Agency (FEMA)
Earthquake Programs
500 “C” Street, S.W.
Washington, DC 20472

International Conference of Building Officials (ICBO)
5360 South Workman Mill Road
Whittier, CA 90601

Precast/Prestressed Concrete Institute
175 West Jackson Blvd.
Chicago, IL 60604

Southern Building Congress International, Inc. (SBCCI)
900 Montclair Road
Birmingham, AL 35213

6.2- Cited references


5. Ferguson, P.M. and Rajagopalan, K.S., “Exploratory Shear Tests Emphasizing Percentage of Longitudinal


GLOSSARY

Terms used in this report generally follow ACI Glossary of Terms, ACI 116R. The following terms and their definitions, however, are unique to this report and are in addition to those given in ACI 116R.

Base Ring- A device used to provide a common surface between the foundation and the equipment for aligning, leveling, and distributing vertical loads as for vessels or process columns.

Belling- Excavating process used to provide additional bearing surface at tip of concrete drilled piers.

Bundle load-- load required to break the bond between a tube bundle and exchanger shell.

Davit- A device used to support/swing the covers off openings of vessels, tanks, etc.

Hydrotest- Filling of equipment (tanks, vessels, etc.) 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, etc.

Pad- Slab-type foundation support for equipment.

Shaft- Vertical portion of concrete drilled pier.

Sleeve- A device used around anchor bolts to allow movement of the bolt after casting of concrete.

Stack- Cylindrical shaped vertical vent.

METRIC (SI) CONVERSION FACTORS

1 in. = 25.4 millimeters
1 in.² = 645.2 mm²
1 pound = 4.448 Newton
1 psi = 0.006895 MPa
1 kip = 4.448 kN
1 ksi = 6.895 MPa

This report was submitted to letter ballot of the committee and approved in accordance with ACI balloting procedures.