Commentary on Standard Practice for Design and Construction of Concrete Silos and Stacking Tubes for Storing Granular Materials (ACI 313-97)

Reported by ACI Committee 313

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CONTENTS

Chapter 1—General, p. 313R-2
R1.1—Introduction
R1.2—Definitions
R1.4—Drawings, specifications and calculations

Chapter 2—Materials, p. 313R-2
R2.2—Cements
R2.3—Aggregates
R2.5—Admixtures

Chapter 3—Construction requirements, p. 313R-3
R3.1—Notation
R3.2—Concrete quality
R3.3—Sampling and testing concrete
R3.4—Details and placement of reinforcement
R3.5—Forms
R3.6—Concrete placing and finishing
R3.7—Concrete protection and curing

Chapter 4—Design, p. 313R-4
R4.1—Notation
R4.2—General

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The term “silo” used here includes both deep bins and shallow bins, the latter sometimes referred to as “bunkers.” Whenever the term “silo” is used, it should be interpreted as meaning a silo, bin or bunker of any proportion, shallow or deep.

Stave silos are used principally in agriculture for storing chopped “silage,” but are finding increasing use in other industries for storing granular materials. This Standard covers the industrial stave silo, but is not to be used as a standard for farm silos. The methods of computing pressures due to granular material are the same for industrial stave silos as for other silos (Chapter 4). However, design of stave silos relies heavily on strength and stiffness tests; consequently, this Standard includes several design requirements that are peculiar to stave silos only.

Chapter 2—MATERIALS

R2.2—Cements

Cement for exposed parts of silos or bunkers should be of one particular type and brand if it is desired to prevent variations in color of the concrete.

In general, the types of cement permitted by ACI 318 are permitted under the recommended practice, except as noted. Experience has shown that there can be some variation in the physical properties of each type of cement. Type I cement that is very finely ground (a fineness modulus greater than 2000 on the Wagner scale) can act in the same manner as Type III and cause difficulties by accelerating the initial set during a slipform operation.

Type IS and IP are not recommended for use in slipform or jumpform concrete because of long initial setting time and low strength at an early age.

R2.3—Aggregates

Aggregates for exposed parts of silos or bunkers should be the same type and source if it is desired to avoid variations in appearance of the completed work.

R2.5—Admixtures

R2.5.1 The use of admixtures in concrete silo walls is a common construction method of controlling the initial set of concrete and, therefore, the rate at which slipforms and/or jumpforms may be raised. During the actual construction operation, the amount of admixture may be adjusted in the field to suit the ambient conditions and so maintain a constant rate of rise for the forms.
Concrete which includes accelerators or retarders should be placed in uniform depths in the slipform or jumpforms to maintain a consistent time of initial set at any wall elevation.

It should be recognized that while potlifes of up to 1.5 hours are available, some superplasticizer (high range water reducer) admixtures have a relatively short useful life (30-35 minutes) after being added to a concrete mixture. This can create problems during placement of stiff mixtures of high strength concrete or mixtures using special cements such as Type K, M and S of ASTM C 845.

CHAPTER 3—CONSTRUCTION REQUIREMENTS

R3.1—Notation

The following additional term is used in the Commentary for Chapter 3, but is not used in the Standard.

\[ f_{cr} = \text{Required average compressive strength of concrete} \]

R3.2—Concrete quality

R3.2.1 The committee recommends a statistical basis to establish an average strength, \( f_{cr} \), to assure attainment of the design strength, \( f'_c \).

ACI Committee 214 has noted that, with general construction having fair control standards, the required \( f_{cr} \) should be attained in over 90 percent of field molded compression specimens provided \( f_{cr} \) is not less than 4000 psi (28 MPa). Fair control standards, indicating a 20 percent coefficient of variation, were assumed to establish the relation between the design and average strength.

It can be shown that lower coefficients of variation may reduce the average strength requirements and, consequently, larger water-cement ratios than permitted in ACI 301 should be possible. However, in the interest of durability, ratios larger than the maximums given in ACI 301 should not be used.

It is important when determining slump for slipformed concrete, that the proposed mix include the same proportions of materials that will actually be used, including admixtures such as accelerators, retarders, air-entraining agents and water-reducing plasticizers.

Historically, concrete mixtures with a slump of 4 in. (100 mm) have been used successfully for construction of slipformed concrete silo and stacking tube walls under a wide variety of field conditions.

R3.2.2 Concrete is considered exposed to freezing and thawing when, in a cold climate, the concrete is in almost continuous contact with moisture prior to freezing.

Entrained air in concrete will provide some protection against damage from freezing against the effects of de-icer chemicals.

R3.3—Sampling and testing concrete

Non-destructive testing of in-place concrete may be used to determine the approximate strength or quality, or to forecast the approximate 28-day strength. Some of these methods of testing are ultrasonic pulse, pulse echo, radioactive measurement of the absorption or scatter of x-rays or gamma radiation, and surface hardness (rebound or probe penetration).

R3.3.2 ASTM C 684 describes three different procedures for the accelerated curing of test cylinders: Warm Water Method, Boiling Water Method and Autogenous Method.

The first two methods permit testing the cylinders at 24 and 28.5 hours respectively, while the third requires hours (+15 min). ACI 214.1R Use of Accelerated Strength Testing provides guidance for interpretation of these test results.

R3.4—Details and placement of reinforcement

R3.4.2 Bars not tied can be moved during vibration or even initially mislocated in slipforming. Failures have occurred because of incorrect spacing of horizontal steel. A positive means of controlling location is essential.

Because no reinforcing bars can project beyond the face of a slipform silo wall, dowels that project into abutting walls, slabs or silo bottoms must frequently be field bent. See ACI 318-95 Commentary Section 7.3 for discussion on cold bending and bending by preheating.

If reinforcing bars are to be welded or to have items attached to them, it is essential to know the carbon content of the bars in order to select the proper procedure and materials for the weld.

R3.4.3 Designers should be cautious about selecting walls thinner than 9 in. (230 mm) since such will not generally accommodate two curtains of reinforcement. Two-face reinforcement substantially improves performance of the wall when the wall is subjected to both tension and bending forces.

R3.4.4 In general, the minimum cover for reinforcing bars placed on the inside face of silo walls should be 1 in. Additional cover should be provided where conditions of wear, chemical attack or moisture can occur.

R3.5—Forms

Slipform and/or jumpform systems should be designed, constructed and operated by or under the supervision of persons experienced in this type of construction. ACI Special Publication No. 4, Formwork for Concrete, and References 9 and 10 contain a general description of the vertical slipform process.

The rate of advancement of the slipform system shall be slow enough that concrete exposed below the bottom of the forms will be capable of supporting itself and the concrete placed above it, but rapid enough to prevent concrete from bonding to the forms.

The advancement of the jumpform system shall slow enough that hardened concrete in contact with the forms is capable of supporting the jumpform system, the construction loads and the fresh concrete placed above it.

R3.6—Concrete placing and finishing

During the construction of slipformed silo or stacking tube walls, it is possible that the concrete placing operation must be interrupted due to unforeseen or unavoidable field conditions and an unplanned construction joint will occur. In this event, the engineer should be notified and concrete placement recommended only upon the engineer’s approval.

R3.7—Concrete protection and curing

R3.7.3 In many cases, atmospheric conditions are such that excess water from “bleeding” of concrete as placed in
the forms is sufficient to keep the surface of the newly formed walls moist for 5 days and no additional provisions for curing need be made. Where deck forms or other enclosures retain the atmosphere in a highly humid condition, no additional curing measures are needed.

Where the above conditions cannot be met, a curing compound may be used or a water spray or mist applied to keep the wall surface continuously moist, the amount of water being carefully regulated to avoid damage by erosion. At no time should the concrete be allowed to have a dry surface until it has reached an age of at least 5 days.

**R3.7.5** Curing compound is undesirable on interior surfaces which are to be in contact with the stored material. Such compound, if present, would modify the effect of the friction between the interior surface and the stored material. As the curing compound is abraded, it contaminates the stored material.

**CHAPTER 4—DESIGN**

**R4.1—Notation**

The following additional terms are used in the Commentary for Chapter 4, but are not used in the Standard.

- $A'_{c}$ = compression steel area. See Fig. 4-F.
- $B$ = constant calculated from Eq. (4D).
- $K_r$ = thermal resistance of wall. See Fig. 4-E.
- $M_u$ = required flexural strength per unit height of wall
- $T_i$ = temperature inside mass of stored material
- $T_o$ = exterior dry-bulb temperature
- $d$ = effective depth of flexural member. See Fig. 4-F.
- $d', d''$ = distances from face of wall to center of reinforcement nearest that face. See Fig. 4-F.
- $e, e', e''$ = eccentricities. See Fig. 4-F.
- $n$ = constant calculated from Eq. (4B) or Eq. (4C).
- $n'$ = constant calculated from Eq. (4E).
- $\delta$ = effective angle of internal friction
- $\theta_x, \theta_y$ = angle of conical or plane flow hopper with vertical. See Fig. 4-C.

**R4.2—General**

**R4.2.3** Walls thinner than 6 in. (150 mm) are difficult to construct. When slipforming thinner walls, concrete can be more easily “lifted,” causing horizontal and vertical planes of weakness or actual separation. Thin walls are subject to honeycomb.

**R4.2.4 Load and Strength Reduction Factors**

**R4.2.4.1** The load factors of 1.7 for live load and 1.4 for dead load are consistent with ACI 318. ACI 318 requires a higher factor for live load than for dead load since live load cannot normally be estimated or controlled as accurately as dead load. In ordinary structures, a frequent cause of over-load is increased depth or decreased spacing of stored materials. In silos, this problem cannot occur, since design is always for a full silo, and extra material can never be added. Pressures in the silo, however, are sensitive to minor changes in the stored material’s properties and overload may occur as a result of these changes. Thus, a live load factor of 1.7 is specified. Larger variations in properties are possible between dry and wet stored materials. In such cases, use the combination of properties that creates the highest pressures.

The weight per unit volume, $\gamma$, can vary significantly even for the same material. The purpose of the load factor is not to permit a silo that is designed for one material to be used for storing another (e.g. clean coal versus raw coal). If different materials are stored, consider each material, noting that one material may control for lateral pressure, while another may control for vertical pressure.

**R4.2.4.2** The lower strength reduction factor for slip-formed concrete without continuous inspection recognizes the greater difficulty of controlling reinforcement location.

**R4.3—Details and placement of reinforcement**

**R4.3.1** Fig. 4-A and 4-B illustrate typical reinforcing patterns at wall intersections, ring beams and wall openings. The illustrated details are not mandatory, but are examples to aid the designer.

**R4.3.2** The designer should be aware that bending moments may occur in silos of any shape. Bending moments will be present in walls of silo groups, especially when some cells are full and some empty.11,12 They may also occur when flow patterns change or when some cells are subjected to initial (filling) pressures while others are subjected to design (flow) pressures.13

The walls of interstices and pocket bins will have axial forces, bending moments and shear forces, and may cause axial forces, bending moments and shear forces in the silo walls to which they are attached.

Wall bending moments in a circular silo are difficult to accurately evaluate, but do exist. They result from non-uniform pressures around the circumference during discharge, especially eccentric discharge. They can also result from temperature differentials, from structural continuity and from materials stored against the outside of the silo.

**R4.3.3** Forces tending to separate silos of monolithically cast silo groups may occur when some cells are full and some empty11 (such as four empty cells with a full interstice). They may also result from non-uniform pressure around the circumference, thermal expansion, seismic loading or differential foundation settlement.

**R4.3.4** Horizontal hoop tension (or tension plus shear and bending moment) does not cease abruptly at the bottom of the pressure zone. The upper portion of the wall below has strains and displacements compatible with those of the wall above. Therefore, the pattern of main horizontal reinforcement is continued downward from the bottom of the pressure zone for a distance equal to four times the thickness $h$ of the wall above.

Since the wall below the pressure zone frequently has sizeable openings, it is often necessary to design that wall (usually as a deep beam) to span those openings. In this case, reinforcement areas must be adequate for deep beam action.

**R4.3.5** Vertical reinforcement in silo walls helps distribute lateral load irregularities vertically to successive layers of horizontal reinforcement. In addition, it resists vertical bending and tension due to the following causes:

1. Temperature changes in the walls when the wall is restrained or not free to move in the vertical direction.
2. Wall restraint at roof, floor or foundation.
3. Eccentric loads, such as those from hopper edges or ancillary structures.
4. Concentrated loads at the transition between the cylindrical and converging section of a flow channel.
5. Temperature differentials between inside and outside wall surfaces or between silos.\textsuperscript{14}

6. Splitting action from bond stresses at lapped splices of hoop bars.

To provide access for concrete buggies in slipform construction, vertical reinforcement may be spaced farther apart at specified access locations. Reinforcement should not be omitted for this purpose; only the spacing should be affected, larger than normal at the access location and smaller than normal on each side.

**R4.3.7** The possibility of bond failure, with subsequent splitting, is greater where bars are closely spaced, as at lap splices.\textsuperscript{15} Staggering of lap splices increases the average bar spacing. With adjacent splices, one splice failure can trigger another. With staggered splices, this possibility is less likely.

**R4.3.8 Reinforcement at Wall Openings**

**R4.3.8.1 Openings in pressure zone**

(a) This requirement for added horizontal reinforcement is based on the assumption that the silo strength to resist horizontal design pressures from the stored materials should not be reduced by the opening. The 20 percent increase is for stress concentrations next to the opening. Bar spacing and clearances frequently become critical where such extra reinforcement is added.\textsuperscript{16}

**R4.3.8.2 Openings not in pressure zone**

For narrow openings, this method provides a simple rule of thumb by which to provide reinforcement for a lintel-type action above and below the openings. Reinforcement for beam action below the opening is important since the wall below will usually have vertical compressive stress. For large openings, a deep beam analysis should be considered.

**R4.3.8.3 All openings, bar extension**

(a) The distance that reinforcement must be extended to replace the strength that would otherwise be lost at the opening depends not merely on bond strength, but also on the proportions of the opening. Horizontal extension must be more for deep openings than for shallow. Similarly, vertical extension should be more for wide openings than for narrow.
In each case, extension length depends on the opening dimension perpendicular to the bar direction.

**R4.3.9** For walls, the suggested spacing of horizontal bars is not less than 4 in. (100 mm) for walls with two-layer reinforcing nor less than 3 in. (75 mm) for singly reinforced walls. The use of lesser spacing makes it difficult to locate and tie bars.

Since internal splitting of the concrete and complete loss of bond or lap strength can be catastrophic in a silo wall, it is mandatory to select reinforcement patterns which will force strength to be controlled by tensile failure of the horizontal reinforcement rather than by splitting of the concrete.

The 5-bar diameter minimum spacing of horizontal bars assures more concrete between bars and helps prevent brittle bond failures.

**R4.3.10** Additional lap length is specified for hoop bars in walls of slipformed silos since bars may easily be misplaced longitudinally, leading to less lap at one end of the bars and more at the other. For rectangular or polygonal silos, where the shape of the bar prevents longitudinal misplacement of horizontal bars at a splice, the additional lap length may not be required.

**R4.3.11** Both horizontal and vertical thermal tensile stresses will occur on the colder side of the wall. Where these stresses add significantly to those due to stored material pressures, additional reinforcement is required. (See Section 4.4.9.)

Better crack width control on the outside face is possible when the horizontal reinforcement is near the outer face. Also, since this is frequently the colder face, reinforcement so placed is in a better position to resist thermal stress. Care should be taken to ensure adequate concrete cover over the bars on the outside surface to prevent bond splitting failures.

Crack width control and concrete cover on the inside face are also important to lessen the effects of abrasion due to flow and to reduce the possibility that any corrosive elements from the stored material might damage the reinforcement.

**R4.3.12** Singly-reinforced circular walls, with the reinforcement placed near the outside face may not effectively resist bending moments which cause tension on the inside face of the wall.

**R4.4—Loads**

**R4.4.1.1** Material pressures against silo walls and hoppers depend on the initial (filling) conditions and on the flow patterns which develop in the silo upon discharge. The procedure for pressure calculations requires definition of the following terms:

(a) *Filling*—The process of loading the material by gravity into the silo.

(b) *Discharging*—The process of emptying the material by gravity from the silo.
(c) Initial filling pressure—Pressures during filling and settling of material, but before discharge has started.

(d) Flow pressures—Pressures during flow.

(e) Aeration pressures—Air pressures caused by injection of air for mixing or homogenizing, or for initiating flow near discharge openings.

(f) Overpressure factor—A multiplier applied to the initial filling pressure to provide for pressure increases that occur during discharge.

(g) Flow channel—A channel of moving material that forms above a discharge opening.

(h) Concentric flow—A flow pattern in which the flow channel has a vertical axis of symmetry coinciding with that of the silo and discharge outlet.

(i) Asymmetric flow—A flow pattern in which the flow channel is not centrally located.

(j) Mass flow—A flow pattern in which all material is in motion whenever any of it is withdrawn.

(k) Funnel flow—A flow pattern in which the flow channel forms within the material. The material surrounding the flow channel remains at rest during discharge.

(l) Expanded flow—A flow pattern in which a mass flow hopper is used directly over the outlet to expand the flow channel diameter beyond the maximum stable rathole diameter.

(m) Rathole—A flow channel configuration which, when formed in surrounding static material, remains stable after the contents of the flow channel have been discharged.

(n) Stable arch dimension—The maximum dimension up to which a material arch can form and remain stable.

(o) Self-cleaning hopper—A hopper which is sloped steeply enough to cause material, which has remained static during funnel flow, to slide off of it when the silo is completely discharged.

(p) Expanded flow silo—A silo equipped with a self-cleaning hopper section above a mass flow hopper section.

(q) Tilted hopper—A hopper which has its axis tilted from the vertical.

(r) Pyramidal hopper—A hopper with polygonal flat sloping sides.

(s) Plane flow hopper—A hopper with two flat sloping sides and two vertical ends.

(t) Transition hopper—A hopper with flat and curved surfaces.

(u) Effective angle of internal friction (δ)—A measure of combined friction and cohesion of material; approximately equal to angle of internal friction for free flowing or coarse materials, but significantly higher for cohesive materials.

R4.4.1.2 American practice is, generally, to use Janssen’s formula [Eq. (4-1)], whereas in parts of Europe, Reimbert’s method is preferred. Rankine’s method is sometimes used for silos having small height to diameter ratios. Methods other than Janssen’s may be used to compute wall pressures. There are a large variety of hopper pressure formulas available in the literature including Jenike, McLean, and Walker. All are based on different assumptions and may yield significantly different pressure distributions.

R4.4.1.3 To compute pressures, certain properties of the stored material must be known. There are many tables in the technical literature listing such properties as silo design parameters. However, in using those parameters for structural design, the designer should be aware that they are, at best, a guide. Unquestioned use may inadvertently lead to an unsafe design. This situation exists because of a long maintained effort to associate design parameters with the generic name of the material to be stored, neglecting completely the wide range of properties that such a name may cover. The usual design pa-
rameters, density, internal friction angle and wall friction angle, all used in computing pressures, are affected by:

(a) Conditions of the material—Moisture content, particle size, gradation and angularity of particles.

(b) Operating conditions—Consolidation pressure, time in storage, temperature, rate of filling and amount of aeration.

Table 4-A gives examples of ranges of properties which have been used in silo design. Actual properties of a specific material may be quite different. It is, therefore, recommended that upper and lower bounds be determined by testing the material in question. If the actual material to be stored is unavailable, the bounds should be determined by testing or by examining representative materials from other similar installations.

**R4.4.2 Pressures and Loads for Walls**

**R4.4.2.1** Designers should consider an appropriate degree of variability in \( \gamma \), \( k \) and \( \mu' \). The design should be based on maximum \( \gamma \) with appropriate combinations of maximum and minimum values of \( k \) and \( \mu' \).

Eq. (4-1) assumes concentric filling and uniform axisymmetric pressure distribution. In the case of eccentrically filled silos in which the elevation of the material surface at the wall varies significantly around the perimeter, the pressure distribution will not be axisymmetric. Such pressure may be computed by varying \( \gamma \) according to the material surface level at the wall.

**R4.4.2.2** During initial filling and during discharge, even when both are concentric, overpressures occur because of imperfections in the cylindrical shape of the silo, non-uniformity in the distribution of particle sizes, and convergence at the top of hoppers or in flow channels.

A minimum overpressure factor of 1.5 is recommended for concentric flow silos even when they are of a mass flow configuration. The recommended factor recognizes that even though higher and lower point pressures are measured in full size silos, they are distributed vertically through the stiffness of the silo wall and can be averaged over larger areas for structural design. The 1.5 overpressure factor is in addition to the load factor of 1.7 required by Section 4.2.4 (design pressure = 1.7 x 1.5 x initial filling pressure).

**R4.4.2.3** Asymmetric flow can result from the presence of one or more eccentric outlets or even from non-uniform distribution of material over a concentric outlet.

Methods for evaluating the effects of asymmetric flow have been published. None of these methods has been endorsed by the Committee.

**R4.4.3 Pressures and Loads for Hoppers**

**R4.4.3.1** Hopper pressures are more complex to predict than wall pressures. The pressure distribution will be more sensitive to the variables discussed in Section R4.4.1.3. Naturally, there is a significant diversity within the technical literature with regard to hopper pressures. None of these methods has been endorsed by the Committee. Some pressure measurements reported in the technical literature are not significantly lower than those predicted by Eq. (4-5) in the lower part of the hopper.

### Table 4-A—Example physical properties of granular materials*

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight ( \gamma )</th>
<th>Angle of internal friction ( \phi )</th>
<th>Effective angle of internal friction ( \delta )</th>
<th>Coefficient of friction ( \mu' )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>lb/ft(^3)</td>
<td>kg/m(^3)</td>
<td>Against concrete</td>
<td>Against steel</td>
</tr>
<tr>
<td>Cement, clinker</td>
<td>88</td>
<td>1410</td>
<td>33</td>
<td>42-52</td>
</tr>
<tr>
<td>Cement, portland</td>
<td>84-100</td>
<td>1345-1600</td>
<td>24 to 30</td>
<td>40-50</td>
</tr>
<tr>
<td>Clay</td>
<td>106-138</td>
<td>1700-2200</td>
<td>15 to 40</td>
<td>50-90</td>
</tr>
<tr>
<td>Coal, bituminous</td>
<td>50-65</td>
<td>800-1040</td>
<td>32 to 44</td>
<td>33-68</td>
</tr>
<tr>
<td>Coal, anthracite</td>
<td>60-70</td>
<td>960-1120</td>
<td>24 to 30</td>
<td>40-45</td>
</tr>
<tr>
<td>Coke</td>
<td>32-61</td>
<td>515-975</td>
<td>35-45</td>
<td>50-60</td>
</tr>
<tr>
<td>Flour</td>
<td>38</td>
<td>610</td>
<td>40</td>
<td>23-30</td>
</tr>
<tr>
<td>Fly ash</td>
<td>50-112</td>
<td>865-1800</td>
<td>35-40</td>
<td>37-42</td>
</tr>
<tr>
<td>Gravel</td>
<td>100-125</td>
<td>1600-2000</td>
<td>25 to 35</td>
<td>36-40</td>
</tr>
<tr>
<td>Grains (small): wheat, corn, barley, beans (navy, kidney), oats, rice, rye</td>
<td>44-62</td>
<td>736-990</td>
<td>20 to 37</td>
<td>28-35</td>
</tr>
<tr>
<td>Gypsum, lumps</td>
<td>100</td>
<td>1600</td>
<td>38-40</td>
<td>45-62</td>
</tr>
<tr>
<td>Iron ore</td>
<td>165</td>
<td>2640</td>
<td>40-50</td>
<td>50-70</td>
</tr>
<tr>
<td>Lime, calcined, fine</td>
<td>70-80</td>
<td>1120-1280</td>
<td>30-35</td>
<td>35-45</td>
</tr>
<tr>
<td>Lime, calcined, coarse</td>
<td>58-75</td>
<td>928-1200</td>
<td>40</td>
<td>40-45</td>
</tr>
<tr>
<td>Limestone</td>
<td>84-127</td>
<td>1344-2731</td>
<td>39-43</td>
<td>45-80</td>
</tr>
<tr>
<td>Manganese ore</td>
<td>125</td>
<td>2000</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>100-125</td>
<td>1600-2000</td>
<td>25 to 40</td>
<td>30-50</td>
</tr>
<tr>
<td>Soybeans, peas</td>
<td>50-60</td>
<td>800-960</td>
<td>23</td>
<td></td>
</tr>
<tr>
<td>Sugar, granular</td>
<td>53-63</td>
<td>1000</td>
<td>35</td>
<td>33-40</td>
</tr>
</tbody>
</table>

*The properties listed here are illustrative of values which might be determined from physical testing. Ranges of values show the variability of some materials. Design parameters should preferably be determined by test and the values shown used with caution. See Commentary on Section 4.4.1.
Eqs. (4-6) and (4-8) generally control for steep smooth hoppers where the friction along the material-hopper interface is fully developed. Eq. (4-7) and (4-9) generally control for shallow hoppers where the friction along the material-hopper interface is not fully developed. The value of \( k \) to be used in Eq. (4-7) is to be conservatively computed by Eq. (4-3). However, because of the uncertainty inherent in hopper pressure estimates, the designer should check Eq. (4-6) and (4-7), and use the equation which yields the larger \( p_n \).

While designers may be able to justify lower pressures, a hopper failure can result in significant damage or total collapse of a silo; therefore, the use of the slightly conservative procedure of Eqs. (4-5) through (4-9) is recommended. Pressures on gates and feeders at hopper outlets are usually lower than the pressures computed using Eq. (4-5).

**R4.4.3.2** Funnel flow occurs only when the outlet is large enough for the material to flow without forming a stable arch or rathole, and the hopper walls are not sufficiently smooth or sufficiently steep to develop a mass flow pattern. To obtain self-cleaning, the hopper slope must be sufficiently steep to cause the material to slide off of it when the silo is discharged completely. Jenike\(^{38}\) suggests that \( \alpha > \phi' + 25^\circ \). Some designers select \( \alpha \) such that \( \tan \alpha > 1.5 \tan \phi' \) for hoppers having flat surfaces and \( 1.5 \sqrt{2} \tan \phi' \) for conical hoppers or the valley of pyramidal hoppers. The slope of a funnel flow hopper should be selected to avoid the possibility of mass flow (see Section R4.4.3.3).

The recommended overpressure factors for hoppers and flat bottoms are essentially the same as in the earlier version of the Standard and are intended to cover dynamic loads which normally occur during funnel flow.

Collapse of large stable arches and ratholes can subject the silo to severe shock loads which can cause structural damage. Such loading requires additional analysis which is not covered herein. Selection of silo and hopper configurations which minimize the potential for forming stable arches and ratholes is highly recommended. A common approach is to select an expanded flow pattern.

**R4.4.3.3** Mass flow occurs only when the outlet is large enough for the material to flow without arching, the flow control device permits flow through the entire outlet,
and the hopper walls are smooth enough and steep enough to allow material to slide.

Jenike\textsuperscript{38,39} has provided design information in graph form for selecting the slopes of two common shapes of hoppers (conical and plane flow). Approximate slopes necessary for mass flow to occur may be estimated using Fig. 4-C. The occurrence of mass flow or funnel flow is seen to depend on the values of hopper slope angles $\theta_c$ and $\theta_p$ and the hopper wall friction angle $\phi'$. The region labeled “uncertain” on the graphs of Fig. 4-C indicates conditions for which flow may shift abruptly between funnel flow and mass flow, with large masses of material being in non-steady flow and the consequent development of shock loads.\textsuperscript{40} Such flow conditions will also lead to non-symmetric flow patterns and, hence, to non-symmetric loads on the silo. Designers should avoid selecting hopper slopes in this region.

Other hopper configurations include pyramidal and transition hoppers. For mass flow to develop in a pyramidal hopper, the slope of the hopper valleys should be steeper than $\theta_c$. For transition hoppers, the side slope should be steeper than $\theta_p$, and the slope of the curved end walls should be steeper than $\theta_c$. For tilted hoppers with one vertical side, mass flow will develop when the included angle is $1.25 \theta_c$ or $1.25 \theta_p$.

Fig. 4-D is a flow chart showing a recommended procedure for selecting a silo hopper configuration. Detailed procedures for computing hopper slopes and outlet sizes are given by Jenike.\textsuperscript{38}

Mass flow results in high pressures at the top of hopper (at and directly below the transition). Two methods for computing mass flow pressures are given by Jenike\textsuperscript{13,39} and Walker.\textsuperscript{20} The two methods result in slightly different pressure distributions with Jenike yielding peak pressures at the transition higher than Walker. Comprehensive reviews of hopper pressures are given in References 18, 41 and 42.

A method that has been used to determine design pressures in mass flow hoppers based on Walker’s\textsuperscript{20} follows.

(a) The vertical pressure at depth $h_y$ below top of hopper is computed by:

$$q_v = \frac{\gamma}{n-1} \left( \frac{h_y - h_i}{h_s} \right) \left[ 1 - \left( \frac{h_y - h_i}{h_s} \right)^{n-1} \right] + q_o \left( \frac{h_0 - h_i}{h_s} \right)^n \quad (4A)$$

where $q_o$ is computed by Eq. (4-1) and,
for circular cones \( n = \frac{2B}{\tan \theta} \) (but not less than 1.0) \((4B)\)

for Plane Flow Hoppers \( n = \frac{B}{\tan \theta} \) (but not less than 1.0) \((4C)\)

where

\[
B = \frac{\sin \delta \sin 2(\theta + \beta)}{1 - \sin \delta \cos 2(\theta + \beta)}
\]

and

\[
\beta = 1/2 \left[ \beta' + \arcsin \left( \frac{\sin \phi'}{\sin \delta} \right) \right]
\]

(b) Except for the vertical end walls of plane flow hoppers, the pressure normal to the hopper surface at a depth \( h_y \) below top of hopper is computed by:

\[
P_n = \frac{1 + \sin \delta \cos 2(\theta + \beta)}{1 - \sin \delta \cos 2(\theta + \beta)} q_y
\]

The pressure normal to the vertical end wall of plane flow hoppers should be not less than computed by Section 4.4.3.2.

(c) The unit friction load between the stored material and hopper surface is computed by Eq. (4-8) with \( p_n \) computed by Eq. (4F).

Pressures in mass flow tilted hoppers, where the angle between the hopper axis and the vertical does not exceed \( \theta_c \) or \( \theta_p \), may be computed using this method with \( \theta \) taken as the angle between the hopper axis and the hopper surface.

R4.4.3.4 In multiple-outlet hoppers, flow may occur over some outlets while initial filling pressures exist over others. The differential lateral pressures on hopper segments between outlets can be substantial.

R4.4.4 Pressures for flat bottoms—Eq. (4-1) assumes a uniform vertical pressure distribution across the diameter of the silo. Vertical pressures may be lower at the wall and higher at the center of the silo particularly if the silo height to diameter ratio is low. Such pressure variations should be considered in the design of flat bottom floors.

R4.4.5 Pressures in homogenizing silos—Homogenizing silos are those in which air pressure is used to mix dust-like materials. The material being mixed may behave as a fluid; thus, the possibility of hydraulic pressures should be considered. The factor 0.6 reflects the fact that the suspended particles are not in contact, and the average density is less than for the material at rest. Partially aerated silos may experience aeration pressure directly additive to non-aerated intergranular pressures.\(^{43}\)

R4.4.8 Earthquake forces—In computing lateral seismic force due to the mass of the stored granular material, the silo is assumed to be full, but the lateral force is less than it would be for a solid mass. The reduction of lateral force is allowed because of energy loss through intergranular movement and particle-to-particle friction in the stored material.\(^{44-46}\)

R4.4.9 Thermal effects—Computation of bending moments due to thermal effects requires determining the temperature differential through the wall. To determine this differential, the designer should consider the rates at which heat flows from the hot material to the inside surface of the wall, through the wall thickness and from the wall to the atmosphere. There are two distinct and different conditions to be analyzed.

(a) The worst thermal condition is usually found in the wall above the hot material surface where the air is maintained at a high temperature, while fresh hot material is fed into the silo. In that portion of the wall, high thermal loads will co-exist with wall dead load and no material loads.

(b) A less severe condition exists below the hot material surface, where temperatures fall as heat flows through the wall to the outside and a temperature gradient develops through some thickness of the granular material.\(^{47}\) In that portion of the wall, material loads will co-exist with reduced thermal loads.

The temperature differential may be estimated by:\(^{14}\)

\[
\Delta T = (T_i - T_o - 80^\circ F) K_f
\]

where \( K_f \) for cement is given by Fig. 4-E.

Other methods for computing bending moments due to thermal effects are available.\(^{1,48,49,50}\)

The designer should also recognize that structural steel items like roof beams inside a concrete silo may expand more rapidly than the concrete and cause an overstress at contact areas if space for expansion is not provided.
R4.5—Wall design

R4.5.2 Storage of hot materials may cause appreciable thermal stresses in the walls of silos. Thermal stresses may or may not occur concurrently with the maximum hoop forces.

The reinforcement added for thermal bending moments should be placed near the cooler (usually outside) face of the wall. In singly-reinforced walls, it should be added to the main hoop reinforcement, which should be near the outside face. In walls with two-layer reinforcing, the entire amount should be added to the outer layer. (For simplicity, an equal amount is often added to the inner layer to avoid having bar sizes or spacings differ from one layer to the other).

Horizontal and vertical thermal moments will be present in the wall above the hot material surface and must be considered in the design. Where the vertical dead load compressive stress is low, added vertical temperature reinforcement may be required.

R4.5.3 Strength design of walls subject to combined axial tension and flexure shall be based on the stress and strain compatibility assumptions of ACI 318 and on the equilibrium between the forces acting on the cross-section at nominal strength. For small eccentricity, Fig. 4-F (\(e = M_u/F_u < h/2-d''\)) the required tensile reinforcement area per unit height:

\[
A_s = \frac{F_u e''}{f_y (d - d'')} \quad (4H)
\]
on the side nearest to force \(F_u\), and

\[
A'_s = \frac{F_u e''}{f_y (d - d'')} \quad (4I)
\]
on the opposite side. Both reinforcement areas \(A_s\) and \(A'_s\), are in tension. For large eccentricity (\(e = M_u/F_u > h/2-d''\)), refer to textbooks on strength design of reinforced concrete sections.

R4.5.4 Circular Walls in Pressure Zone

R4.5.4.1 Even though circular walls of concentric flow silos are analyzed as subject to direct hoop tension only, bending moments may occur due to temperature differential, wind or seismic loads. The hoop tensions and bending moments should be combined according to Section 4.5.2 and the wall thickness and hoop reinforcement determined according to Section 4.5.3.

R4.5.4.3 Aeration systems which fluidize only portions of the silo can cause significant circumferential and vertical bending moments in walls.

R4.5.5 Suggested procedures for the analysis and design of non-circular silo walls are given in Reference 11.

R4.5.7 Eq. (4-13) is obtained from an equivalent ACI 318 equation for walls. Proportions of cast-in-place circular silo walls are such that buckling due to vertical compressive stress ordinarily does not control, and the axial load compressive strength given by Eq. (4-13) need not be reduced for slenderness effects.

However, for silos of unusual proportions, and for some silo walls next to openings, the design vertical compressive strength may be less than given by Eq. (4-13). Suggested formulas for such conditions are given in References 11 and 51.

R4.5.8 The primary concern of crack control is to minimize crack width. However, in terms of protecting the reinforcement from corrosion, surface crack width appears to be relatively less important than believed previously. Therefore, it is usually preferable to provide a greater thickness of concrete cover even though this will lead to wider surface cracks. Construction practices directed towards minimizing drying shrinkage will have significant impact on crack control. Additional information on this subject can be found in ACI 318 and in Reference 52.

Similarly, to protect against splitting of the concrete around the reinforcement, it is preferable to limit the minimum center-to-center spacing and the minimum concrete cover of the reinforcement to those prescribed by Sections 4.3.9 and 4.3.10 even though this may also lead to wider surface cracks.

The design crack width limit of 0.010 inch (0.25 mm) under initial filling conditions results in reasonable reinforcement details which reflect experience with existing silos. The actual crack width will, in all probability, be different than the computed design crack width and will vary depending on the amount of cover provided. Eq. (4-14), given in Reference 52, does not reflect the effects of excessive drying shrinkage which can result in a significant increase in crack width. In Eq. (4-14), \(f_y\) is the stress in the reinforcement under initial filling pressures computed by Eqs. (4-1) through (4-3) (at service load level, load factor = 1.0, overpressure factor = 1.0).

R4.6—Hopper design

R4.6.1 Hoppers should be designed to withstand flow pressures prescribed by Sections 4.4.3.2 and 4.4.3.3, in addition to other loads.

R4.6.2 Formulas for computing stresses in hoppers can be found in References 11, 18, 41 and 42. The design of structural steel hoppers should be as prescribed in References 18 and 53.

R4.7—Column design

Under sustained compressive load, creep in a reinforced concrete column causes the concrete stress to reduce, putting additional load on the steel reinforcement. With subsequent unloading, the concrete may be placed in tension and develop horizontal cracks. This condition is more pronounced in columns with large ratios of reinforcement-to-concrete area.

The problem of such cracking is seldom experienced in normal building structures since dead load exceeds vertical live load and extreme unloading cannot occur. However, in storage silos, live load (stored materials) usually accounts for the major portion of the load, and it can be quickly removed. Thus, the horizontal cracking of heavily reinforced silo support columns can be severe.

Such cracking will be serious if it is accompanied by vertical cracking as could occur with high bond stresses during unloading. This latter condition can be dangerous. To prevent this dangerous condition:

1. If lateral forces are not a problem, keep the vertical reinforcement ratio low to prevent horizontal cracking upon unloading; or
(2) If lateral forces must be resisted, use larger columns with a low reinforcement ratio.

**R4.8—Foundation design**

R4.8.3 Unsymmetrical loading should be considered for its effect on stability (against overturning), soil pressures and structural design of the foundation.

**CHAPTER 5—STAVE SILOS**

**R5.1—Notation**

The following additional term is used in the Commentary for Chapter 5, but is not used in the Standard.

\[ EI = \text{flexural stiffness of wall} \]

**R5.4—Erection tolerances**

R5.4.1 Spiral means the distortion that results if the staves are tilted slightly so that, even though their outer faces are vertical, their edges are inclined. The combined effect of such misplacement is to cause vertical joint lines to be long-pitch spirals rather than straight lines. The resulting assembly appears twisted.

R5.4.2 A “bulge” is the vertical out-of-plane deviation of a stave wall as measured from a prescribed length straight-edge or string.

**R5.5—Wall design**

R5.5.1 Loads, design pressures and forces—The pressure formulas in Chapter 4 are not applicable to silos storing silage. Guidance for farm silo design can be found in Reference 54.

R5.5.2 Wall thickness—Because of wide variation among silo staves produced by various manufacturers, it is desirable to supplement analytical data by tests.

Physical tests useful in determining design criteria include compressive and flexural tests of individual staves, and tests of stave assemblies to determine joint shear strength (tension), vertical compressive strength, and both vertical and horizontal bending strength. Tests of stave assemblies are considered important since the silo strength depends not so much on the strength of any one component as on the way these components and their connections act in the finished silo.

Recommended methods of concrete stave testing are given in Section 5.7 of this Commentary.

R5.5.3 Circular bending—Stave silos have less circular rigidity and less circular bending strength than monolithic silos. The thin walls and the vertical joints between staves contribute to the lack of rigidity. If the joints are not manufactured to the exact bevel to suit the silo diameter or are not shaped so they can be pointed with grout after erection, they are free to rotate and allow the silo to assume an oval shape.

The decreased circular strength results from the placement of steel hoops on the exterior surface. When the curvature of the wall increases, the hoops are effective in creating circular strength, but when the curvature decreases, the hoops are ineffective except to create compression in the concrete stave which must first be overcome before the wall can crack.

While a stave wall has the undesirable tendency to go out-of-round if it is not stiff enough, it also has the desirable ability to redistribute circumferential bending moments from weaker positive moment (tension inside face) zones to stronger negative moment zones (tension outside face).

The circular strength and stiffness of a stave silo can be increased by additional hoops, thicker staves or better vertical joint details. The strength of any particular stave design is difficult to determine without testing full-scale stave assemblies. However, it can be estimated that the total statistical moment strength is typically not more than 0.875 \((\phi A_s f_y - F_u)h\) and that the positive moment strength is typically not more than 0.375 \((\phi A_s f_y - F_u)h\).

Equation (5-2) requires the total statical moment strength to be 1.7 times the total moment acting on the wall. Equation (5-3) requires the positive moment strength to be 1.0 times the positive moment acting on the wall. The assumption is that moments in the positive moment zones will redistribute to the negative moment zones and the factor of safety against total failure will be maintained even though there may be some cracking on the inside face in the positive moment zones.

The designer should recognize when determining circumferential bending from unequal pressures, the magnitudes and distribution of moments can be affected by assumptions about where and to what extent the stave wall cracks under

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**Fig. 5-A—Stave assembly joint shear (tension) test**
the tension and bending loads. The circumferential membrane tension force from filling pressures, $F_u$, can significantly reduce the circumferential bending capacity available to resist asymmetric flow pressures.

The designer should also recognize that significant circular deformation can occur and that unexpected distress may result where circular walls are restrained from free movement by attached structures.

R5.5.4 Compression and buckling—Deformation from asymmetric flow, particularly over a side withdrawal, may significantly reduce the wall curvature and increase the possibility of the wall buckling under vertical loads.

The $P_{nw,\text{stave}}$ in Eq. (5-6) is the strength obtained from tests illustrated by Fig. 5-1 or Fig. 5-2. The $P_{nw,\text{joint}}$ in Eq. (5-7) is the strength from tests illustrated by Fig. 5-B and is typically lower. The $P_{nw,\text{buckling}}$ in Equation (5-8) is obtained by test, or by a combination of test results and published methods of computing critical buckling strength, and must take into account the sometimes large out-of-plane deviations found in stave silo walls.

R5.5.5 Tension and shear—Silo stave walls are subjected to vertical tension most often when the silo has insufficient self weight to resist overturning from wind forces. In such cases, anchor straps secured to the foundation are extended up the silo wall an appropriate distance and secured to the hoops. Where the straps are discontinued, the wall must resist the remaining tension.

Tension failure of the wall can occur if the stave breaks in tension or if the stave slips out of the lapped position depicted in Fig. 5-A. Compliance with Eq. (5-12) will prevent a tension failure of the concrete in the stave. Compliance with Eq. (5-13) will prevent slipping of the stave from the lapped position. The force $W$ in Eq. (5-12) and Eq. (5-13) is doubled because only half of the staves are continuous at any horizontal joint.

R5.6—Hoops for stave silos

R5.6.1 Tensioning—Hoops generally consist of three or more rods, connected together by connecting lugs of malleable iron or pressed steel. Experience shows that even though tightening is done only at the lug, within a short time the hoop stress will be uniform along the entire hoop length.

R5.7—Concrete stave testing

Tests of individual staves:

(a) Compressive strength tests to determine $P_{nw,\text{stave}}$ are defined by Sections 5.7.1 and 5.7.2 for solid and cored staves, respectively. Compressive test samples should be cut from five or more randomly selected staves. The specimens shown in Fig. 5-1 and Fig. 5-2 are full stave width with height equal to twice the stave thickness. The compressive
load is vertical, with the specimen positioned as for use in the silo wall.

(b) Flexural strength (measures concrete quality and can be used in lieu of the compressive strength test). Bending specimens are cut from five or more randomly selected staves. The specimen length is sufficient to permit testing on a 24 in. (0.61 m) simple span with concentrated midspan load. End reactions and midspan load are distributed across the full width of the specimen and are applied through padded bearing plates 2 in. (50 mm) wide. The span direction is selected to be parallel to the vertical direction of the stave as used in the silo. Test speed is not over 0.05 in. (1.3 mm) per min. The bending strength is computed as the bending modulus of rupture.

Tests of stave assemblies:

(a) Joint shear strength (tension), i.e., resistance to sliding, may be determined by testing a group of three staves as shown in Fig. 5-A. Lateral confining forces are proportioned to simulate the forces applied by hoop prestress in the unloaded actual silo. The test measures the vertical pull necessary to cause the center stave to slide with respect to the two adjacent staves. The word “tension” is used in describing this test since such joint shear and sliding result from loadings which place the silo wall in vertical tension (such as wind load on the empty silo).

(b) Stave joint compressive strength, $P_{nw, joint}$. Fig. 5-B shows a typical specimen for this test, which is intended to measure the compressive force that can be transferred from stave to stave across a horizontal joint. Joints and surfaces should be grouted and/or coated in the manner that will be used in the actual silo.

(c) Vertical stiffness. Fig. 5-C shows a typical specimen and test set-up for determining vertical stiffness. An assembly four staves high by four wide is coated in the manner that will be used in the actual silo. Confining forces are applied to the assembly in a manner to simulate the prestress force (after losses) of the hoop rods. Lateral load is applied and deflections are measured. From the loads and deflections, the value of effective $EI$, and then effective wall thickness, can be computed for use in obtaining $P_{nw, buckling}$.

(d) Horizontal strength and stiffness. A typical specimen and set-up for testing horizontal strength and stiffness are shown by Fig. 5-D. The assembled staves are coated in the manner that will be used in the actual silo. Deflection and load values are observed. The effective $EI$ and wall thickness are then computed from the test results for use in determining the circumferential critical buckling strength.

When the test is used to determine circular bending strength for purposes of checking resistance to bending from asymmetric pressures, the hoops should be loosened an appropriate amount to simulate the loss of compression across the vertical joints that would occur from the internal pressure of the stored material.
CHAPTER 6—POST-TENSIONED SILOS

**R6.1—Notation**

The following terms are used in the Commentary for Chapter 6, but are not used in the Standard:

- $F$ = radial force on the wall that results from the stressing (jacking) of the tendon
- $M_{max}$ = maximum vertical bending moment per unit width of wall calculated from Eq. (6F)
- $V_{max}$ = maximum shear force per unit width of wall calculated from Eq. (6G)
- $M_r$ = vertical bending moment caused by force $F$ on the wall
- $V_{hy}$ = shear caused by a force $F$ on the wall. See Fig. 6-A.
- $y$ = distance above and below tendon location
- $\psi_f$ = factor obtained from Eq. (6D) or Table 6-A
- $\theta_f$ = factor obtained from Eq. (6E) or Table 6-A
- $\beta_f$ = factor relating to Poisson’s ratio, silo diameter and wall thickness

**R6.2—Scope**

Provisions of this Standard sometimes exceed those of ACI 318 because the severity of silo loadings and field operating conditions differ substantially from those of buildings.

**R6.4—Tendon systems**

**R6.4.1** A minimum 10 in. (250 mm) wall thickness is recommended to provide adequate room for placing and controlling location of tendons and non-post-tensioned reinforcement.

**R6.4.4** Tendon ducts are placed on the inside face of the outer layer vertical steel to help ensure proper duct position, curvature and cover.

**R6.4.5** Jacking locations should be spaced uniformly around the circumference of the silo to avoid unnecessary concentrations of stresses. Wall pilasters should be located and proportioned to avoid reverse curvature of the tendons. If this is not done, radial forces due to reverse curvature should be considered in designing the pilaster and its web reinforcement.

**R6.4.6** Horizontal tie reinforcement should be provided in pilasters to prevent radial forces from continuing tendons and forces from anchored tendons from splitting the wall. Ties to resist splitting forces should be provided at pilasters common to two silos, as at wall intersections.

**R6.4.9** Dry-packed mortar consisting of one part shrinkage-compensating portland cement and two parts sand is recommended for filling blockouts and pockets.

**R6.5—Bonded tendons**

**R6.5.2** Effect of grout admixtures in concrete at a later age should be considered.

**R6.6—Unbonded tendons**

**R6.6.1** A discussion of the factors to be considered in cases of cyclic loading which might lead to premature fatigue failures can be found in ACI 215R.

**R6.7—Post-tensioning ducts**

Duct sizes required by Sections 6.7.2 and 6.7.3 are minimums. Larger sizes may be advisable. For example, in slip-formed work, control of duct location is more difficult and the potential for duct damage greater than for fixed-form construction. In such case, a larger-than-minimum duct might be preferable. Currently, the smallest nominal diameter rigid metal duct available is $1 \frac{3}{8}$ in. (48 mm). The field forming of such rigid metal ducts to a small radius is difficult and can result in kinks or reductions of duct cross-section. If ducts are placed during slipforming of a silo wall, they should be checked for blockage or section reduction as soon as they are exposed below the forms so that repairs can be made while the concrete is still in a workable state. Larger ducts, while more difficult to bend, may result in fewer section reduction problems.

**R6.8—Wrapped systems**

**R6.8.3** Guidance on techniques and procedures for wrapped systems is available in ACI 344R-W.

**R6.12—Design**

**R6.12.3** It is recommended in fully post-tensioned systems that a residual compressive stress of about 40 psi (0.30 MPa) be maintained under service load conditions (including thermal loads) if it is desired to minimize the likelihood of open cracks.

When partial post-tensioning and the higher permissible concrete stresses of Table 6.1 are used in a design, the wall can be expected to crack more than if it were fully post-tensioned. Therefore, a careful evaluation should be made of the expected cracking and the effects such cracking might have on protection of the post-tensioning tendons from weather or abrasion.

Even so, the preferred solution might be to provide partial post-tensioning to avoid having a fully post-tensioned wall become overstressed in compression because of circumferential bending moments.

In either system, care should be taken to properly evaluate bending as well as axial stresses in the silo wall under all service load conditions.

**R6.12.8** The height limits given in Section 6.12.8 for the transition zone have been obtained by shell analysis. Specified minimum levels of initial compressive stress are lower than recommended by ACI 344R since some cracking can be tolerated, whereas cracking in liquid storage tanks cannot be tolerated.

**R6.12.9** Formulas for estimating losses due to anchorage set and tendon elongation within the jack and for calculation of the length influenced by anchor set may be found in References 56 and 57. Methods of estimating prestress losses due to elastic shortening and time-dependent losses may be found in References 56, 57, 58, 59, and 60.

**R6.13—Vertical bending moment and shear due to post-tensioning**

Vertical bending moment will be caused whenever a tendon is tensioned, due to inward movement of the wall at the tendon location, while the wall at some distance above and below that tendon is relatively unaffected. During prestressing, vertical bending moment is also caused by the restraint to inward movement of the wall offered by the foundation, non-sliding roofs, silo bottom slabs, etc. These bending moments should be considered in design. References 61, 62, 63, 64, 65, and 66 suggest methods for computing these bending moments.
For the effect of a single tendon, a method based on analysis of the wall as a beam on an elastic foundation could be used as in Reference 67.

Another method for calculating these bending moments is Timoshenko’s method introduced below. It assumes that a cylindrical shell is subjected to a uniformly distributed inward load along a circular section.

a) When the spacing between tendons is less than \( \frac{2\pi}{\beta_p} \), the vertical bending moment \( M_y \) and the shearing force \( V_{hy} \) on a horizontal section at distance \( y \) above or below the tendon may be determined by Eq. (6A) and (6B), respectively, per unit width of wall.

\[
M_y = \frac{(F\psi_f)}{4\beta_p} \quad (6A)
\]

and

\[
V_{hy} = \frac{F\Theta_f}{2} \quad (6B)
\]

in which \( F \) is the radial force and \( \psi_f \) and \( \Theta_f \) are factors obtained from Eq. (6D) and Eq. (6E) or Table 6-A as a function of \( \beta_p y \)

\[
\beta_p = \left[ 12(1 - 0.2)/(D^2 h') \right]^{0.25} \quad (6C)
\]

\[
\psi_f = e^{-\beta_p y} (\cos \beta_p y - \sin \beta_p y) \quad (6D)
\]

\[
\Theta_f = e^{-\beta_p y} (\cos \beta_p y) \quad (6E)
\]

b) When the spacing between tendons exceeds \( \frac{2\pi}{\beta_p} \), then adjacent tendons do not contribute significantly to the magnitude of bending moment and shear at the tendon under consideration. In that case, the maximum vertical bending moment and maximum shear per unit width of wall are

\[
M_{max} = \frac{F}{(4\beta_p)} \quad (6F)
\]

\[
V_{max} = \frac{F}{2} \quad (6G)
\]
Values of bending moments due to prestress of wires may be obtained from References 61 and 68.

R6.14—Tolerances

Control of vertical location of tendons in slipforming is fairly easy while control of horizontal location is more difficult. Unfortunately, control of the horizontal location is more important; hence the horizontal tolerance should be observed closely, both at support points and between support points.

CHAPTER 7—STACKING TUBES

R7.2—General layout

Stacking tubes (sometimes known as lowering tubes) are free-standing tubular structures used to stack conical piles of granular bulk materials up to 150 ft. (45 m) high (Fig. 7-A). They are used mechanically as lowering tubes to control loss of significant dust to the atmosphere and they are used structurally to support the stacking conveyor. Concrete stacking tubes normally vary in diameter from 6 to 16 ft. (2 to 5 m) and in wall thickness from 6 to 16 in. (150 to 400 mm).

The bulk material is discharged into the top of the tube and as the material builds up in the bottom, it spills out through the wall openings to form the pile. The openings are generally equipped with hinged dust flaps.

Stacking tubes are frequently built directly over conveyor-equipped tunnels which reclaim material by gravity from the pile above. Typically, tunnel reclaim openings are furnished on either side of the tube. Sometimes openings are furnished directly under the tube (Fig. 7-B). Even though the latter location is less effective in reclaiming from the pile, it does provide a method of keeping non-free flowing material from plugging the tube.

Operators of stacking tube systems (especially for coal) frequently work on top of the piles with bulldozers to push the material away from the tube during stockpiling and back toward the pile during reclaiming. The bulldozers create fines and compact the material into a denser state. This action, added to the natural densification of fines in the center of the pile from segregation during stockpiling, frequently causes flow problems in the vicinity of the tube. Such problems include:

1. Formation of stable ratholes into which dozers and workers can inadvertently fall. (A stable rathole forms when the stockpiled material has sufficient cohesion and internal strength to arch horizontally around a flow channel and remain stable even after the flowing material is gone. Stable ratholes vary in size from 5 to 20 ft. (1.5 to 6 m) in diameter.

2. Creation of high vertical walls of dense material which can collapse on front end loaders trying to reclaim the material.

3. Formation of stable arches which can prevent material from flowing into or out of the stacking tube openings.

4. Plugged material inside the tube, the unexpected falling of which can create hazards for workers and the structure, if cleaning operations are attempted from the bottom.

5. Structural damage to the tube walls and dust flaps by dozer blades as operators try to reclaim dense material close to the tube.

6. Failure of dust-flap hinges as the open flap gets pinched on both surfaces by material pressure and gets torn off by downward movement of the material during reclaiming.

Table 6-A—Values of factors $\Psi_f$ and $\Theta_f$ for use in Eq. (6A) and (6B)

<table>
<thead>
<tr>
<th>$\beta_p$</th>
<th>$\Psi_f$</th>
<th>$\Theta_f$</th>
<th>$\beta_p$</th>
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An attempt should be made to select tube diameters, outlet opening sizes, wall thicknesses, reclaim opening configurations, dust-flap designs, and operating and maintenance procedures which will minimize the above potential problems.

R7.3—Loads

The design of the stacking tube should take into account the most severe probable loading condition the tube might experience from operation of the stockpiling and reclaiming system. Reclaim hoppers large enough to prevent stable ratholes should be used if possible; if they are not, the tube design should take into account the uneven lateral loading that might result from a pile that is complete, except for a stable rathole on one side of the tube. The design should also consider all likely configurations of excavated material removed by bulldozers or front-end loaders operating on one side of the tube, but not the other.

When considering the forces imparted on the tube from conveyor expansion and contraction or belt tension, the stiffness of the tube relative to the conveyor structure should be taken into account.

R7.6—Foundation or reclaim tunnel

The vertical loads that the bulk material pile imparts on the stacking tube and reclaim tunnel should be carefully considered if the pile is supported on compressible soils while the tube and/or tunnel is supported on rigid foundations. In such cases, the stacking tube and other associated rigid structures can be subjected to extremely large negative skin friction loads from the pile as the pile base settles either elastically or inelastically.

Careful consideration should also be given to differential settlement.

REFERENCES

54. International Silo Association, Recommended Practice for Design and Construction of: Top Unloading Monolithic Farm Silos; Bottom Unloading Monolithic Farm Silos; Top Unloading Concrete Stave Farm Silos; and Bottom Unloading Concrete Stave Farm Silos, Lenexa, Kansas, December, 1981, 4 Volumes.

This report was submitted to letter ballot of the committee and was approved according to institute balloting procedures.