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

An ACI Standard

Reported by ACI Committee 313

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This ACI standard practice gives material, design, and construction requirements for concrete silos, stave silos, and stacking tubes for storing granular materials. It includes design and construction recommendations for cast-in-place or precast and conventionally reinforced or post-tensioned silos.

Silos and stacking tubes are special structures, posing special problems not encountered in normal building design. While this standard refers to "Building Code Requirements for Structural Concrete (ACI 318)" for many requirements, it puts forth special requirements for the unique cases of static and dynamic loading from funnel flow, mass flow, concentric flow, and asymmetric flow in silos, and the special loadings on stacking tubes. The standard includes requirements for seismic design and hopper bottom design.

Keywords: asymmetric flow; **bins**; circumferential bending; concrete; **concrete construction**; **dead loads**; dynamic loads: earthquake resistant structures; formwork (construction); funnel flow; granular materials; hoppers; jumpforms; **lateral loads**: **loads (forces)**; lowering tubes; mass flow; overpressure; quality control; reinforced concrete; reinforcing steels; **silos**; slipform construction; stacking tubes; stave silos; **stresses**; structural analysis; **structural design**; thermal stresses; walls.

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This standard was submitted to letter ballot of the committee and was approved in accordance with Institute standardization procedures.

ACI 313-97 was adopted as a standard of the American Concrete Institute on January 7, 1997, to supersede ACI 313-91, in accordance with the Institute's standardization procedures.

CHAPTER 1—GENERAL

1.1—Introduction

This document, which covers design and construction of concrete silos and stacking tubes for storing granular materials, replaced the 1968 ACI Committee 313 Report 65-37 and was adopted as an ACI Standard in March 1977 as ACI 313-77. It was subsequently revised in 1983 and 1991. The current revision reflects the most recent state-of-the-art in structural design, detail, and construction of concrete silos and stacking tubes.

Static pressures are exerted by the stored material at rest and shall be computed by methods presented. Flow pressures that differ from static pressures are exerted by the stored material during flow and also shall be computed by the methods presented.

Design of the structures shall consider both static and flow loading.

Applicable sections of ACI 318 shall apply.

1.2—Definitions

The term "silo," as used herein, applies to any upright container for storing bulk granular material.

Alternate names such as "bins" and "bunkers" are used in different localities, but for purposes of this Standard, all such structures are considered to be silos.

"Stacking tubes" or "lowering tubes" are relatively slender, free-standing, tubular concrete structures used to stack conical piles of granular materials. See Commentary Section 7.2.

"Slipformed" silos are constructed using a typically 4 ft. (1.2 m) high continuously moving form.

"Jumpformed" silos are constructed using three typically 4 ft. (1.2 m) high fixed forms. The bottom lift is jumped to the top position after the concrete hardens sufficiently.

A "hopper" is the sloping, walled portion at the bottom of a silo.

"Stave silos" are silos assembled from small precast concrete units called "staves," usually tongued and grooved, and held together by exterior adjustable steel hoops.

Other special terms are defined in the Commentary.

1.3—Scope

This Standard covers the design and construction of concrete silos and stacking tubes for storing granular materials. Silos for storing of ensilage have different requirements and are not included. However, industrial stave silos for storage of granular materials are included.

Coverage of precast concrete is limited to that for industrial stave silos.

The Standard is based on the strength design method. Provisions for the effect of hot stored material are included. Explanations of requirements of the Standard, additional design information, and typical details are found in the Commentary.

1.4—Drawings, specifications, and calculations

1.4.1 Project drawings and project specifications for silos shall be prepared under the direct supervision of and bear the seal of the engineer.

1.4.2 Project drawings and project specifications shall show all features of the work, naming the stored materials assumed in the design and stating their properties, and including the size and position of all structural components, connections and reinforcing steel, the required concrete strength, and the required strength or grade of reinforcing and structural steel.

CHAPTER 2—MATERIALS

2.1—General

All materials and tests of materials shall conform to ACI 301, except as otherwise specified.

2.2—Cements

Cement shall conform to ASTM C 150 (Types I, IA, II, IIAA, III and IIIA), ASTM C 595 (excluding Types S, SA, IS and IS-A), or ASTM C 845.

2.3—Aggregates

The nominal maximum size of aggregate for slipformed concrete shall not be larger than one-eighth of the narrowest dimension between sides of wall forms, nor larger than three-eighths of the minimum clear spacing between individual reinforcing bars or vertical bundles of bars.

2.4—Water

Water for concrete shall be potable, free from injurious amounts of substances that may be harmful to concrete or steel. Non-potable water may be used only if it produces mortar cubes, prepared according to ASTM C 109, having 7- and 28-day strengths equal to at least the strength of similar specimens made with potable water.

2.5—Admixtures

2.5.1 Air-entraining, water reducing, retarding or accelerating admixtures that may be required for specific construction conditions shall be submitted to the engineer for approval prior to their use.

2.6—Metal

2.6.1 Hoop post-tensioning rods shall be hot-dip galvanized or otherwise protected from corrosion. Connectors, nuts and lugs shall either be hot-dip galvanized or made from corrosion-resistant castings or corrosion-resistant steel. Galvanizing shall conform to ASTM A 123.

2.6.2 Malleable iron castings shall conform to ASTM A 47.

2.7—Precast concrete staves

2.7.1 Materials for staves manufactured by the dry-pack vibratory method shall conform to ASTM C 55.

2.7.2 Before a stave is used in a silo, drying shrinkage shall have caused the stave to come within 90 percent of its equilibrium weight and length as defined by ASTM C 426.

2.8—Tests of materials

2.8.1 Tests of materials used in concrete construction shall be made as required by the applicable building codes and the engineer. All material tests shall be by an agency acceptable to the engineer.

2.8.2 Tests of materials shall be made in accordance with the applicable ASTM standards. The complete record of such tests shall be available for inspection during the progress of the work, and a complete set of these documents shall be preserved by the engineer or owner for at least 2 years after completion of the construction.

2.8.3 *Silo stave tests*—The results of mechanical tests of silo staves and stave assemblies shall be used as criteria for structural design of stave silos. The application of the test results is given in Chapter 5. Example methods of performing the necessary tests are given in the Commentary.

CHAPTER 3—CONSTRUCTION REQUIREMENTS

3.1—General

Concrete quality control, methods of determining concrete strength, field tests, concrete proportions and consistency, mixing and placing, formwork, details of reinforcement and structural members shall conform to ACI 301, except as specified otherwise herein.

3.2—Concrete quality

3.2.1 The compressive strength specified for cast-in-place concrete shall be not less than 4000 psi (28 MPa) at 28 days. The compressive strength specified for concrete used in precast units shall be not less than 4000 psi (28 MPa) at 28 days. The acceptance strength shall conform to ACI 301.

3.2.2 Exterior concrete in silo or stacking tube walls that will be exposed to cycles of freezing and thawing shall be air entrained.

3.3—Sampling and testing concrete

3.3.1 For strength tests, at least one set of three specimens shall be made and tested of the concrete placed during each 8 hrs or fraction thereof.

3.3.2 Accelerated curing and testing of concrete cylinders shall conform to ASTM C 684.

3.4—Details and placement of reinforcement

3.4.1 Horizontal tensile reinforcement in silo and hopper walls shall not be bundled.

3.4.2 Horizontal reinforcement shall be accurately placed and adequately supported. It shall be physically secured to vertical reinforcement or other adequate supports to prevent displacement during movement of forms or placement of concrete.

3.4.3 Silo walls that are 9 in. (230 mm) or more in thickness shall have two layers of horizontal and vertical steel.

3.4.4 The minimum concrete cover provided for reinforcement shall conform to ACI 318 for cast-in-place concrete (non-prestressed), except as noted in Section 4.3.10.

3.5—Forms

3.5.1 The design, fabrication, erection and operation of a slipform or jumpform system for a silo or stacking tube wall shall meet the appropriate requirements of ACI 347.

3.5.2 Forms shall be tight and rigid to maintain the finished concrete wall thickness within the specified dimensional tolerances given in Section 3.9.

3.5.3 Slipform systems shall include an approved means of determining and controlling level at each jack unit.

3.6—Concrete placing and finishing

3.6.1 Construction joints in silos shall not be permitted unless shown on the project drawings or specifically approved by the engineer.

3.6.2 Concrete shall be deposited within 5 ft. (1.5 m) of its final position in a way that will prevent segregation and shall not be worked or vibrated a distance of more than 5 ft. (1.5 m) from the point of initial deposit.

3.6.3 As soon as forms have been raised (or removed), vertical wall surfaces shall be finished by filling voids with mortar made from the same materials (cement, sand and water) as used in the wall and by applying a “smooth rubbed finish” in accordance with Section 10.3.1 of ACI 301.

3.7—Concrete protection and curing

3.7.1 Cold weather concreting may begin when temperature is 24 °F (–4 °C) and rising, provided that the protection method will allow 500 psi (4 MPa) compressive strength gain before the concrete temperature drops below 32 °F (0 °C). For cold weather concreting, ACI 306R recommendations shall be used where applicable.

3.7.2 In hot weather, measures shall be taken to prevent drying of the concrete before application of a curing compound. For hot weather concreting, ACI 305R recommendations shall be used where applicable.

3.7.3 Where the wall surfaces will remain moist naturally for 5 days, no curing measures are required. Otherwise, curing measures conforming to ACI 308 shall be used.

3.7.4 Where curing measures are required, they shall be provided before the exposed exterior surfaces begin to dry, but after the patching and finishing have been completed. Wall surfaces shall be protected against damage from rain, running water or freezing.

3.7.5 Curing compounds shall not be used on the inside surfaces of silos unless required by the project drawings or project specifications, or unless specifically approved by the engineer. When curing of interior surfaces is required, non-toxic compounds and ventilation or other methods of assuring worker safety shall be used.

3.7.6 Curing compound shall be a non-staining, resin base type complying with ASTM C 309, Type 2, and shall be applied in strict accordance with the manufacturer’s instructions. Waxbase curing compounds shall not be permitted. If a curing compound is used on the interior surfaces of silos to be used for storing materials for food, the compound shall be non-toxic, non-flaking and otherwise non-deleterious.

3.8—Lining and coating

3.8.1 Linings or coatings used to protect the structure from wear and abrasion, or used to enhance flowability, shall be composed of materials that are non-contaminating to the stored material.

3.8.2 Lining materials installed in sheet form shall be fastened to the structure with top edges and side seams sealed to prevent entrance of stored material behind the lining.

3.8.3 Coatings used as barriers against moisture or as barriers against chemical attack shall conform to ACI 515.1R.

3.9—Tolerances for slipformed and jumpformed structures

3.9.1 Translation of silo centerline or rotational (spiral) of wall:

For heights 100 ft. (30 m) or less .... 3 in. (75 mm)
For heights greater than 100 ft. (30 m), 1/400 times the height, but not more than............ 4 in. (100 mm)

3.9.2 Inside diameter or distance between walls:
Per 10 ft. (3 m) of diameter or distance .... 1/2 in. (12 mm)
but not more than .... 3 in. (75 mm)

3.9.3 Cross-sectional dimensions of:
Walls ....+1 in. (25 mm)
or ....–3/8 in. (10 mm)

3.9.4 Location of openings, embedded plates or anchors:
Vertical...±3 in. (75 mm)
Horizontal ....±1 in. (25 mm)

3.9.5 Other tolerances to meet ACI 117.

CHAPTER 4—DESIGN

4.1—Notation

Consistent units must be used in all equations. Except where noted, units may be either all U.S. Customary or all metric (SI).

A = effective tension area of concrete surrounding the tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars. When the reinforcement consists of different bar sizes, the number of bars shall be computed as the total area of reinforcement divided by the area of the largest bar used. See Fig. 4-3.
D = dead load or dead load effect, or diameter
E_c = modulus of elasticity for concrete
L = live load or live load effect
M_t = thermal bending moment per unit width or height of wall (consistent units)
P_{nw} = nominal axial load strength of wall per unit perimeter
R = ratio of area to perimeter of horizontal cross-section of storage space
T = temperature or temperature effect
ΔT = temperature difference between inside face and outside face of wall
U = required strength
V = total vertical frictional force on a unit length of wall perimeter above the section in question
Y = depth from the equivalent surface of stored material to point in question. See Fig. 4-2.
d_c = thickness of concrete cover taken equal to 2.5 bar diameters, or less. See Fig. 4-3.
e = base of natural logarithms
f'_c = compressive strength of concrete
f_s = calculated stress in reinforcement at initial (filling) pressures
h = wall thickness
h_h = height of hopper from apex to top of hopper. See Fig. 4-2.
h_s = height of sloping top surface of stored material. See Fig. 4-2.
h_y = depth below top of hopper to point in question. See Fig. 4-2.
k = p/q
p = initial (filling) horizontal pressure due to stored material
p_n = pressure normal to hopper surface at a depth h_y

below top of hopper. See Fig. 4-2.

q = initial (filling) vertical pressure due to stored material

q_o = initial vertical pressure at top of hopper

q_y = vertical pressure at a distance h_y below top of hopper. See Fig. 4-2.

s = bar spacing, in. See Fig. 4-3.

v_n = initial friction force per unit area between stored material and hopper surface calculated from Eq. (4-8) or (4-9)

w = design crack width, in. or lateral wind pressure

α = angle of hopper from horizontal. See Fig. 4-2.

α_c = thermal coefficient of expansion of concrete

γ = weight per unit volume for stored material

θ = angle of hopper from vertical. See Fig. 4-2.

μ' = coefficient of friction between stored material and wall or hopper surface

ν = Poisson's ratio for concrete, assumed to be 0.2

ϕ = strength reduction factor or angle of internal friction

ϕ' = angle of friction between material and wall and hopper surface

ρ = angle of repose. See Fig. 4-2.

4.2—General

4.2.1 Silos and stacking tubes shall be designed to resist all applicable loads, including:

(a) Dead load: Weight of the structure and attached items including equipment dead load supported by the structure.

(b) Live load: Forces from stored material (including overpressures and underpressures from flow), floor and roof live loads, snow, equipment loads, positive and negative air pressure, either wind or seismic load (whichever controls), and forces from earth or from materials stored against the outside of the silo or stacking tube (see also Section 4.8).

(c) Thermal loads, including those due to temperature differences between inside and outside faces of wall.

(d) Forces due to differential settlement of foundations.

4.2.2 Structural members shall be proportioned for adequate strength and stiffness. Stresses shall be calculated and combined using methods provided in Chapter 4 for silos and Chapter 7 for stacking tubes. Design methods for reinforced or prestressed concrete members such as foundations, floors, roofs, and similar structures not covered herein shall be in accordance with ACI 318.

4.2.3 The thickness of silo or stacking tube walls shall be not less than 6 in. (150 mm) for cast-in-place concrete, nor less than 2 in. (50 mm) for precast concrete.

4.2.4 ***Load factors and strength reduction factors***

4.2.4.1 Load factors for silo or stacking tube design shall conform to those specified in ACI 318. The weight of and pressures due to stored material shall be considered as live load.

4.2.4.2 For concrete cast in stationary forms, strength reduction factors, ϕ, shall be as given in ACI 318. For slipforming, unless continuous inspection is provided, strength reduction factors given in ACI 318 shall be multiplied by 0.95.

4.2.5 *Pressure zone*—The pressure zone shall be the part of the wall that is required to resist forces from stored material, hopper, or hopper forming fill.

4.3—Details and placement of reinforcement

4.3.1 Where slipforming is to be used, reinforcement arrangement and details shall be as simple as practical to facilitate placing and inspection during construction.

4.3.2 Reinforcement shall be provided to resist all bending moments, including those due to continuity at wall intersections, alone or in combination with axial and shear forces.

4.3.3 Horizontal ties shall be provided as required to resist forces that tend to separate adjoining silos of monolithically cast silo groups.

4.3.4 Unless determined otherwise by analysis, horizontal reinforcement at the bottom of the pressure zone shall be continued at the same size and spacing for a distance below the pressure zone equal to at least four times the thickness h of the wall above. In no case shall the total horizontal reinforcement area be less than 0.0025 times the gross concrete area per unit height of wall.

4.3.5 Vertical reinforcement in the silo wall shall be #4 (#10M diameter) bars or larger, and the minimum ratio of vertical reinforcement to gross concrete area shall be not less than 0.0020. Horizontal spacing of vertical bars shall not exceed 18 in. (450 mm) for exterior walls nor 24 in. (600 mm) for interior walls of monolithically cast silo groups.

Vertical steel shall be provided to resist wall bending moment at the junction of walls with silo roofs and bottoms. In slipform construction, jackrods, to the extent bond strength can be developed, may be considered as vertical reinforcement when left in place.

4.3.6 Dowels shall be provided at the bottom of columns and pilasters, and also at portions of walls serving as columns. Dowels shall also be provided (if needed to resist wind or seismic forces or forces from material stored against the bottom of the wall) at the bottom of walls.

4.3.7 Lap splices of reinforcing bars, both horizontal and vertical, shall be staggered in circular silos. Adjacent hoop reinforcing lap splices in the pressure zone shall be staggered horizontally by not less than one lap length nor 3 ft. (1 m), and shall not coincide in vertical array more frequently than every third bar. Lap splices of vertical and, whenever possible, horizontal reinforcing bars shall be staggered in non-circular silos.

4.3.8 ***Reinforcement at wall openings***

4.3.8.1 *Openings in pressure zone*

(a) Unless all areas of stress concentration are analyzed and evaluated and reinforcement provided accordingly, horizontal reinforcement interrupted by an opening shall be replaced by adding at least 1.2 times the area of the interrupted horizontal reinforcement, one-half above the opening and one-half below (see also Section 4.3.8.3).

(b) Unless determined otherwise by analysis, additional vertical reinforcement shall be added to the wall on each side of the opening. The added reinforcement shall be calculated by assuming a narrow strip of wall, $4h$ in width on each side of the opening, to act as a column, unsupported within the

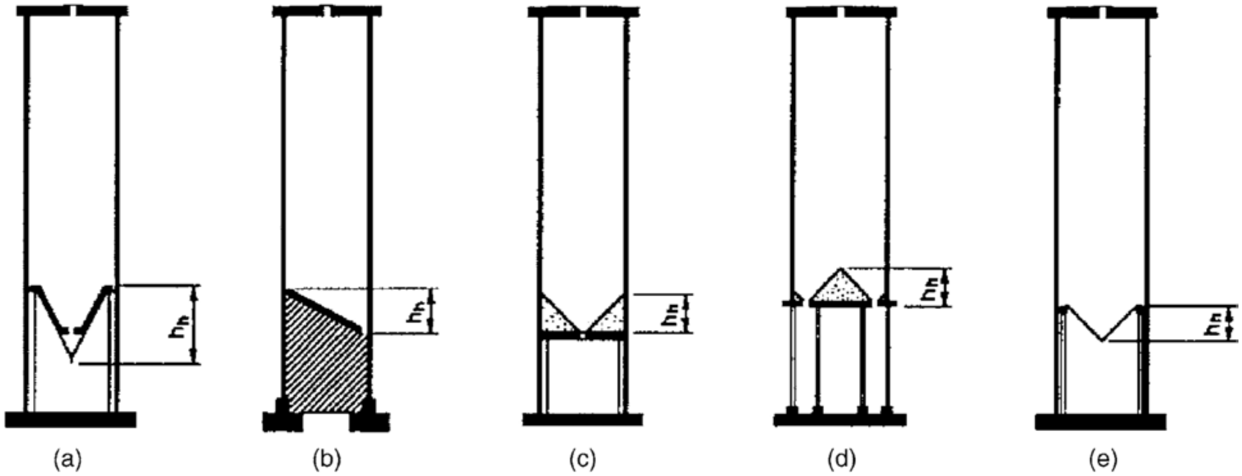

Fig. 4-1—Vertical cross-sections of silos.

opening height and carrying its own share of the vertical load plus one-half of the loads occurring over the wall opening within a height equal to the opening width. The added reinforcement area for each side shall not be less than one-half of the reinforcement area eliminated by the opening.

4.3.8.2 *Openings not in pressure zone*—Unless all areas of stress concentration are analyzed and evaluated, and reinforcement provided accordingly, the amount of added horizontal reinforcement above and below the opening shall each be not less than the normal horizontal reinforcement area for a height of wall equal to one-half the opening height.

Vertical reinforcement adjacent to openings below the pressure zone shall be determined in the manner given for openings in the pressure zone [Section 4.3.8.1(b)].

4.3.8.3 *Reinforcement development at openings*—Added reinforcement to replace load-carrying reinforcement that is interrupted by an opening shall extend in each direction beyond the opening. The extension each way shall be:

(a) Sufficient to develop specified yield strength of the reinforcement through bond;

(b) Not less than 24 in. (600 mm); and,

(c) Not less than one-half the opening dimension in a direction perpendicular to the reinforcement bars in question, unless determined otherwise by analysis.

4.3.8.4 *Narrow vertical walls between openings*—Unless determined otherwise by analysis, walls $8h$ in width or less between openings shall be designed as columns.

4.3.9 The clear vertical spacing between horizontal bars shall be not less than 2 in. (50 mm). The center-to-center spacing of such bars shall be not less than 5 bar diameters. In addition, the vertical spacing of horizontal bars in slipformed walls shall be large enough to allow time for placing and tying of bars during slipform movement.

4.3.10 The lap length of horizontal reinforcement of silo walls shall be not less than:

(a) The lap length specified by ACI 318 for Class B splices for non-circular silos with unstaggered splices.

(b) The lap length specified by ACI 318 for Class B splices plus 6 in. (150 mm) for circular silos (or any cell with circular reinforcing).

In determining the lap length, horizontal bars in jumpformed structures shall be assumed as top bars. Concrete thickness covering the reinforcement at lap splices shall be at least that specified in ACI 318 for that particular splice, but not less than 1 in. (25 mm). In addition, the horizontal distance from the center of the bars to the face of wall shall be not less than 2.5 bar diameters.

Lap splices shall not be used in zones where the concrete is in tension perpendicular to the lap, unless adequate reinforcement is provided to resist tension perpendicular to the lap.

4.3.11 In singly-reinforced walls, the reinforcement to resist thermal bending moment shall be added to the main reinforcement.

In walls with two-layer reinforcement, the reinforcement to resist thermal bending moment shall be added to the layer nearest the colder surface.

4.3.12 In singly-reinforced circular walls, the main hoop reinforcement shall be placed nearer the outer face. Walls shall not be singly-reinforced, unless such reinforcement is designed and positioned to resist all bending moments in addition to hoop tension.

4.4—Loads

4.4.1 *Stored material pressures and loads*

4.4.1.1 Stored material pressures, and loads against silo walls and hoppers, shall be determined using the provisions given in Sections 4.4.2 through 4.4.4. Pressures to be considered shall include initial (filling) pressures, air pressures and pressure increases or decreases caused by withdrawal of material from concentric or eccentric outlets. For monolithically cast silo groups, the condition of some silos full and some silos empty shall be considered.

4.4.1.2 Any method of pressure computation may be used that gives horizontal and vertical design pressures and frictional design forces comparable to those given by Sections 4.4.2 and 4.4.3.

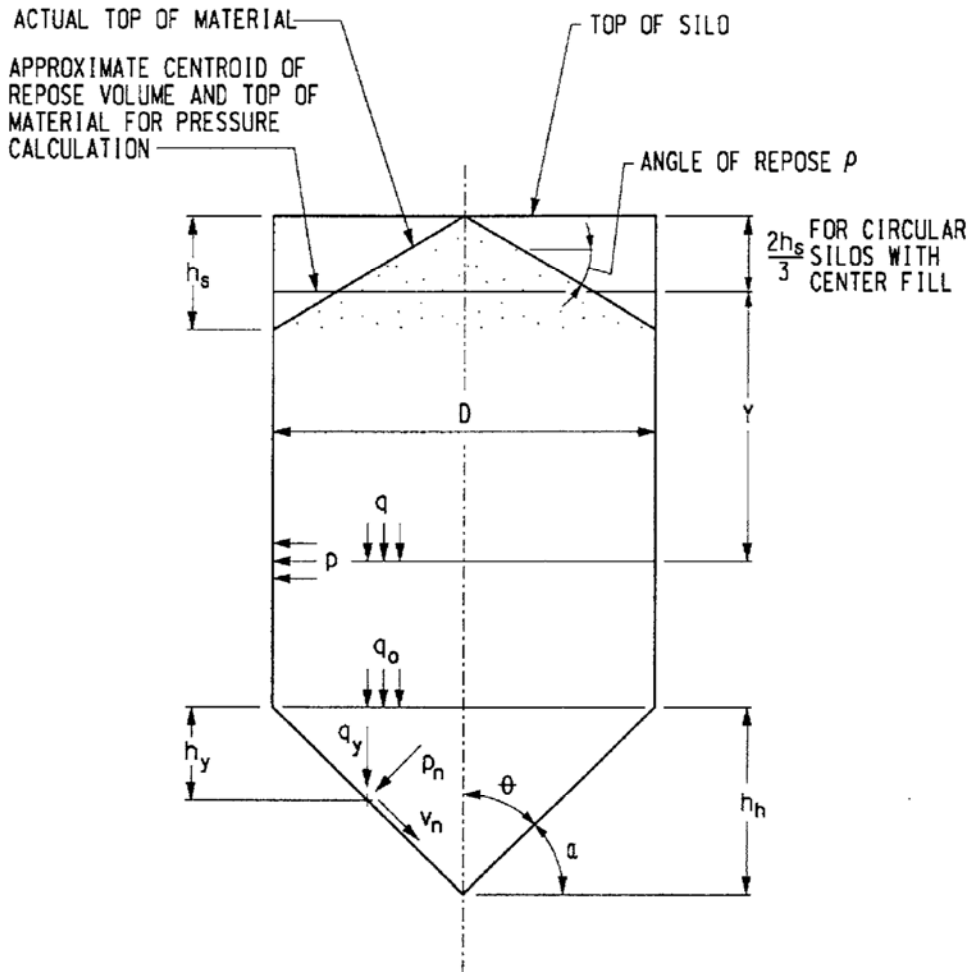

Fig. 4-2—Silo dimensions for use in calculation of pressures and loads for walls and hoppers.

4.4.1.3 Where properties of stored materials vary, pressures shall be computed using combinations of properties given in Section 4.4.2.1(e).

4.4.2 ***Pressures and loads for walls***

4.4.2.1 Pressures due to initial filling shall be computed by Janssen's method.

(a) The initial vertical pressure at depth Y below the surface of the stored material shall be computed by

$$q = \frac{\gamma R}{\mu' k}[\ - e^{-\ 'kY\ R}] \tag{4-1}$$

(b) The initial horizontal pressure at depth Y below the surface of the stored material shall be computed by

$$p = kq \tag{4-2}$$

(c) The lateral pressure ratio k shall be computed by

$$k = 1 - \sin\phi \tag{4-3}$$

where ϕ is the angle of internal friction.

(d) The vertical friction load per unit length of wall perimeter at depth Y below the surface of the material shall be computed by

$$V = (\gamma Y - q)R \tag{4-4}$$

(e) Where γ, μ' and k vary, the following combinations shall be used with maximum γ:

(1) Minimum μ' and minimum k for maximum vertical pressure q.

(2) Minimum μ' and maximum k for maximum lateral pressure p.

(3) Maximum μ' and maximum k for maximum vertical friction force V.

4.4.2.2 *Concentric flow*—The horizontal wall design pressure above the hopper for concentric flow patterns shall be obtained by multiplying the initial filling pressure computed according to Eq. (4-2) by a minimum overpressure factor of 1.5. Lower overpressure factors may be used for particular cases where it can be shown that such a lower factor is satisfactory. In no case shall the overpressure factor be less than 1.35.

4.4.2.3 *Asymmetric flow*—Pressures due to asymmetric flow from concentric or eccentric discharge openings shall be considered.

4.4.3 ***Pressures and loads for hoppers***

4.4.3.1 Initial (filling) pressures below the top of the hopper:

(a) The initial vertical pressure at depth h_y below top of hopper shall be computed by

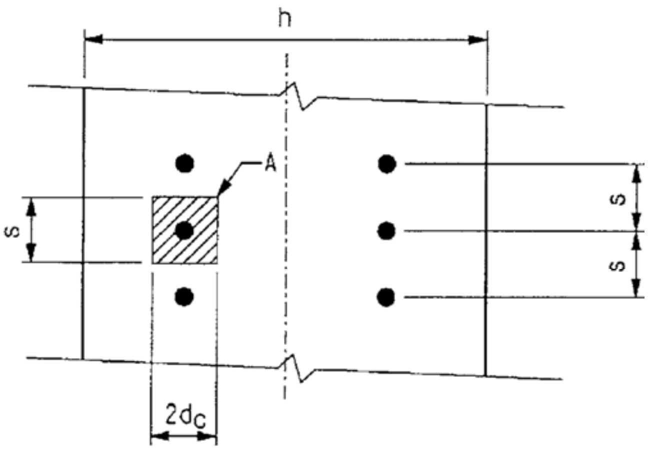

Fig. 4-3—Effective tension area "A" for crack width computation.

$$q_y = q_o + \gamma h_y \tag{4-5}$$

where q_o is the initial vertical pressure at the top of the hopper computed by Eq. (4-1).

(b) The initial pressure normal to the hopper surface at a depth h_y below top of hopper shall be the larger of

$$P_n = \frac{q_y \tan\theta}{\tan\theta + \tan\phi'} \tag{4-6}$$

or

$$P_n = q_y(\sin^2\theta + k\cos^2\theta) \tag{4-7}$$

(c) The initial friction force per unit area of hopper wall surface shall be computed by

$$v_n = p_n \tan\phi' \tag{4-8}$$

when Eq. (4-6) is used to determine p_n and by

$$v_n = q_y(1 - k)\sin\theta\cos\theta \tag{4-9}$$

when Eq. (4-7) is used to determine p_n.

4.4.3.2 *Funnel flow hoppers*—Design pressures at and below the top of a funnel flow hopper shall be computed using Eq. (4-5) through (4-9) with q_o multiplied by an overpressure factor of 1.35 for concrete hoppers and 1.50 for steel hoppers. The vertical design pressure at the top of the hopper need not exceed γY.

4.4.3.3 *Mass flow hoppers*—Design pressures at and below the top of mass flow hoppers shall be considered. In no case shall the design pressure be less than computed by Section 4.4.3.2.

4.4.3.4 In multiple outlet hoppers, the condition that initial pressures exist above some outlets and design pressures exist above others shall be considered.

4.4.4 *Pressures for flat bottoms*

4.4.4.1 Initial filling pressures on flat bottoms shall be computed by Eq. (4-1) with Y taken as the distance from the top of the floor to the top of the material.

4.4.4.2 Vertical design pressures on flat bottoms shall be obtained by multiplying the initial filling pressures computed according to Section 4.4.4.1 by an overpressure factor of 1.35 for concrete bottoms and 1.50 for steel bottoms. The vertical design pressure need not exceed γY.

4.4.5 Design pressures in homogenizing silos shall be taken as the larger of:

(a) Pressures computed according to Sections 4.4.2 and 4.4.3 neglecting air pressure.

(b) Pressures computed by

$$p = q = 0.6\gamma Y \tag{4-10}$$

where γ is the unaerated weight per unit volume of the material.

4.4.6 The pressures and forces calculated as prescribed in Sections 4.4.1 through 4.4.5 are due only to stored material. The effects of dead, floor and roof live loads, snow, thermal, either wind or seismic loads, internal air pressure and forces from earth or materials stored against the outside of the silo shall also be considered in combination with stored material loads.

4.4.7 *Wind forces*—Wind forces on silos shall be considered generated by positive and negative pressures acting concurrently. The pressures shall be not less than required by the local building code for the locality and height zone in question. Wind pressure distributions shall take into account adjacent silos or structures. Circumferential bending due to wind on the empty silo shall be considered.

4.4.8 *Earthquake forces*—Silos to be located in earthquake zones shall be designed and constructed to withstand lateral seismic forces calculated using the provisions of the Uniform Building Code, except that the effective weight of the stored material shall be taken as 80 percent of the actual weight. The centroid of the effective weight shall coincide with the centroid of the actual volume. The fundamental period of vibration of the silo shall be estimated by any rational method.

4.4.9 *Thermal loads*—The thermal effects of hot (or cold) stored materials and hot (or cold) air shall be considered. For circular walls or wall areas with total restraint to warping (as at corners of rectangular silos), the thermal bending moment per unit of wall height or width shall be computed by

$$M_t = E_c h^2 \alpha_c \Delta T / 12(1 - \nu) \tag{4-11}$$

E_c may be reduced to reflect the development of a cracked moment of inertia if such assumptions are compatible with the planned performance of the silo wall at service loads.

4.5—Wall design

4.5.1 *General*—Silo walls shall be designed for all tensile, compressive, shear and other loads and bending moments to which they may be subjected. Minimum wall thickness for all silos shall be as prescribed in Section 4.2.3. Required wall thickness for stave silos shall be determined by the methods of Chapter 5. Minimum wall reinforcement for cast-in-place silos shall be as prescribed in Section 4.3.

4.5.2 Walls shall be designed to have design strengths at all sections at least equal to the required strength calculated for the factored loads and forces in such combinations as are stipulated in ACI 318 and prescribed herein.

Where the effects of thermal loads T are to be included in design, the required strength U shall be at least equal to

$$U = 1.4D + 1.4T + 1.7L \tag{4-12}$$

4.5.3 Design of walls subject to axial load or to combined flexure and axial load shall be as prescribed in ACI 318.

4.5.4 ***Circular walls in pressure zone***

4.5.4.1 For concentric flow, circular silo walls shall be considered in direct hoop tension due to horizontal pressures computed according to Section 4.4.2.2.

4.5.4.2 For asymmetric flow, circular silo walls shall be considered in combined tension and bending due to non-uniform pressures. In no case shall the wall hoop reinforcement be less than required by Section 4.5.4.1.

4.5.4.3 For homogenizing silos, circular silo walls shall be considered in direct hoop tension due to horizontal pressures computed according to Section 4.4.5. In partially fluidized silos, bending moments due to non-uniform pressures shall be considered. In no case shall the wall hoop reinforcement be less than required by Section 4.5.4.1.

4.5.5 Walls in the pressure zone of square, rectangular, or polygonal silos shall be considered in combined tension, flexure and shear due to horizontal pressure from stored material.

4.5.6 Walls below the pressure zone shall be designed as bearing walls subjected to vertical load and applicable lateral loads.

4.5.7 The compressive axial load strength per unit area for walls in which buckling (including local buckling) does not control shall be computed by

$$P_{nw} = 0.55\phi f'_c \tag{4-13}$$

in which strength reduction factor ϕ is 0.70.

4.5.8 For walls in the pressure zone, wall thickness and reinforcing shall be so proportioned that, under initial (filling) pressures, the design crack width computed at 2.5 bar diameter from the center of bar (d_c = 2.5 bar diameter) shall not exceed 0.010 in. (0.25 mm). The design crack width (inch) shall be computed by

$$w = 0.0001 f_s \sqrt[3]{d_c A} \tag{4-14}$$

4.5.9 The continuity between a wall and suspended hopper shall be considered in the wall design.

4.5.10 Walls shall be reinforced to resist forces and bending moments due to continuity of walls in monolithically cast silo groups. The effects of load patterns of both full and empty cells shall be considered.

4.5.11 Walls at each side of opening shall be designed as columns, the column width being limited to no more than four times the wall thickness.

4.6—Hopper design

4.6.1 *Loads*—Silo hoppers shall be designed to withstand loading from stored materials computed according to Section 4.4.3 and other loads. Earthquake loads, if any, shall be determined using provisions of Section 4.4.8. Thermal stresses, if any, due to stored material shall also be considered.

4.6.2 ***Suspended hoppers***

4.6.2.1 Suspended conical hopper shells shall be considered subject to circumferential and meridional (parallel to hopper slope) tensile membrane forces.

4.6.2.2 Suspended pyramidal hopper walls shall be considered subject to combined tensile membrane forces, flexure and shear.

4.6.2.3 The design crack width of reinforced concrete suspended hoppers shall meet the requirements of Section 4.5.8.

4.6.2.4 Wall thickness of suspended reinforced concrete hoppers shall not be less than 5 in. (125 mm).

4.6.2.5 Hopper supports shall have adequate strength to resist the resulting hopper reactions.

4.6.3 ***Flat bottoms***

4.6.3.1 For horizontal bottom slabs, the design loads are dead load, vertical design pressure (from stored material) computed at the top of the slab according to Section 4.4.4.2, and the thermal loading (if any) from stored material. If hopper forming fill is present, the weight of the fill shall be considered as dead load.

4.7—Column design

The area of vertical reinforcement in columns supporting silos or silo bottoms shall not exceed 0.02 times gross area of column.

4.8—Foundation design

4.8.1 Except as prescribed below, silo foundations shall be designed in accordance with ACI 318.

4.8.2 It shall be permissible to neglect the effect of overpressure from stored material in the design of silo foundations.

4.8.3 Unsymmetrical loading of silo groups and the effect of lateral loads shall be considered in foundation design.

4.8.4 Differential settlement of silos within a group shall be considered in foundation, wall, and roof design.

CHAPTER 5—CONCRETE STAVE INDUSTRIAL SILOS

5.1—Notation

Consistent units must be used in all equations. Except where noted, units may be either all U.S. Customary or all metric (SI).

A_s = area of hoop reinforcement, per unit height

A_w = effective cross-sectional area (horizontal projection) of an individual stave

D = dead load or dead load effect, or diameter
E = modulus of elasticity
F_u = required hoop or horizontal tensile strength, per unit height of wall
L = live load or live load effect
M = stored material load stress
M_{pos} M_{neg} = positive (tension inside face) and negative (tension outside face) circumferential bending moments, respectively, caused by asymmetric filling or emptying under service load conditions
M_θ = circular bending strength for an assembled circular group of silo staves, per unit height; the statical moment or sum of absolute values of $M_{\theta,pos}$ and $M_{\theta,neg}$
$M_{\theta,pos}$ $M_{\theta,neg}$ = the measured or computed bending strengths in the positive moment zone and negative moment zone, respectively
P_{nw} = nominal axial load strength of wall per unit perimeter
$P_{nw,buckling}$ = strength of the stave wall as limited by buckling
$P_{nw,joint}$ = strength of the stave wall as limited by the stave joint
$P_{nw,stave}$ = strength of the stave wall as limited by the shape of the stave
W = tension force per stave from wind over-turning moment
f_y = specified yield strength of non-prestressed reinforcement
h = wall thickness
h_{st} = height of stave specimen for compression test. See Figs. 5-1 and 5-2.
w = design crack width, in., or lateral wind pressure
ϕ = strength reduction factor or angle of internal friction

5.2—Scope

This chapter applies only to precast concrete stave silos that are used for storing granular bulk material. It does not apply to farm silos for storage of "silage."

5.3—Coatings

5.3.1 Interior coatings, where specified, shall consist of a single operation, three-coat plaster (parge) application of fine sand and cement worked into the stave surface and joints to become an integral part of the wall. Final finish shall be steel troweled smooth.

5.3.2 Exterior coatings, where specified, shall consist of a thick cement slurry brushed or otherwise worked into the surface and joints of the staves to provide maximum joint rigidity and water-tightness.

5.4—Erection tolerances

5.4.1 Translation of silo centerline or rotation (spiral) of vertical stave joints:

Per 10 ft. (3 m) of height .... 1 in. (25 mm)

5.4.2 Bulging of stave wall:

For any 10 ft. (3 m) of height .... 1 in. (25 mm)

For entire height... 3 in. (75 mm)

5.4.3 Inside diameter of silo:

Per 10 ft. (3 m) of diameter.......................±1 in. (25 mm)

5.4.4 Hoops:

Number of hoop 0

Spacing of hoop ....±1 in. (25 mm)

5.5—Wall design

5.5.1 *Loads, design pressures, and forces*—Loads, design pressures and vertical forces for stave silo design shall be determined as specified in Chapter 4. Overpressure or impact (whichever controls), and the effects of eccentric discharge openings, wind, thermal stress (if any), and seismic action shall all be considered.

5.5.2 *Wall thickness*—The required stave silo wall thickness shall be determined considering circular bending, compression, tension and buckling, but shall in no case be less than given in Section 4.2.3.

5.5.3 *Circular bending*—Unless a more detailed analysis is performed, the circular bending strength M_θ for a given stave design shall satisfy the following:

a) In the case of wind acting on an unbraced wall:

$$M_\theta \geq 0.75(1.7)D^2w/8 \tag{5-1}$$

where the product 0.75(1.7) is the load factor.

b) In the case of unequal interior pressures from asymmetric filling or emptying:

$$M_\theta \geq 1.7(M_{pos} + |M_{neg}|) \tag{5-2}$$

$$M_{\theta,pos} \geq 1.0M_{pos} \tag{5-3}$$

where 1.7 and 1.0 are load factors (see Commentary), and $M_{pos} + M_{neg}$ are determined from the methods of Chapter 4 or other published methods.

The following strength relationships shall be satisfied also:

$$0.875(\phi A_s f_y - F_u)h \geq M_\theta \tag{5-4}$$

$$0.375(\phi A_s f_y - F_u)h \geq M_{\theta,\,pos} \tag{5-5}$$

Unless Eq. (5-4) and Eq. (5-5) are satisfied, a complete circular assembly of staves (Commentary Fig. 5-D) shall be tested to prove satisfactory strength.

5.5.4 *Compression and buckling*—The nominal axial load strength per unit perimeter, P_{nw}, shall be taken as the smaller of:

$$P_{nw} = 0.50\phi P_{nw,stave} \tag{5-6}$$

$$P_{nw} = 0.55\phi P_{nw,joint} \tag{5-7}$$

$$P_{nw} = 0.55\phi P_{nw,buckling} \tag{5-8}$$

In the above, ϕ is 0.7, and $P_{nw,stave}$ and $P_{nw,\,joint}$ are determined by computation and/or tests (Commentary Fig. 5-C) of stave assemblies, after taking into account the maximum eccentricities from out-of-plane deviations allowed in Section 5.4.

The wall thickness shall be such that P_{nw} is not exceeded by any of the following combinations:

$$0.75(1.4D + 1.7L + 1.7W) \tag{5-9}$$

$$(1.4D + 1.7M + 1.7L) \tag{5-10}$$

$$0.75(1.4D + 1.7M + 1.7L \times 1.1E) \tag{5-11}$$

where D is dead load, L is live load, W is wind load, M is stored material load, and E is earthquake load.

5.5.5 *Tension and shear*—The empty silo shall have a factor of safety not less than 1.33 against wind overturning. Calculations shall be based on a shape factor for rough surfaced cylinders and not more than 0.9 times the computed dead load of structure. If anchorage is necessary, the following shall be satisfied where anchors attach to the stave wall,

$$\phi A_w 5\sqrt{f_c'} \geq 1.7(2W) \tag{5-12}$$

and, unless results of tests (Commentary Fig. 5-A) indicate greater strength,

$$\phi 0.1(A_s f_y - F_u)\text{lap} \geq 1.7(2W) \tag{5-13}$$

In the above, ϕ is 0.65, the force $(A_s f_y - F_u)$ is per foot of wall height, lap is the amount of vertical stagger in feet between horizontal stave joints, and 1.7 is the load factor.

5.5.6 *Wall openings*—Wall openings in stave silos shall be framed in such a way that the vertical and horizontal bending and tensile strengths of the wall are not reduced by the opening.

5.6—Hoops for stave silos

5.6.1 *Size and spacing*—Except as noted below, the size and spacing of external hoops for stave silos shall be computed in the same manner as for horizontal reinforcing of circular, cast-in-place silos. In computing the hoop reinforcing, an average design pressure over a wall height equaling 30 times the effective thickness may be used. Hoops shall be not less than 1/2 in. (12.7 mm) in diameter. Spacing shall be not more than the stave height nor ten times the effective wall thickness.

5.6.2 *Calculating steel area*—When calculating the required size and spacing of stave silo hoops, the hoop net area shall be used and shall be taken as the smaller of: (a) the area of the rod, or (b) the root area of the thread. Appropriate restrictions in the available strength of the hoop/lug assembly shall be considered if lugs or mechanical fasteners induce bending deformations or strains in the hoop that reduce the yield strength of the hoop.

5.6.3 *Tensioning*—Stave silo hoops shall be tensioned such that enough stress remains after all losses from shrinkage, creep, elastic shortening and temperature changes to maintain the required vertical and circular strength, and stiffness of the stave assembly.

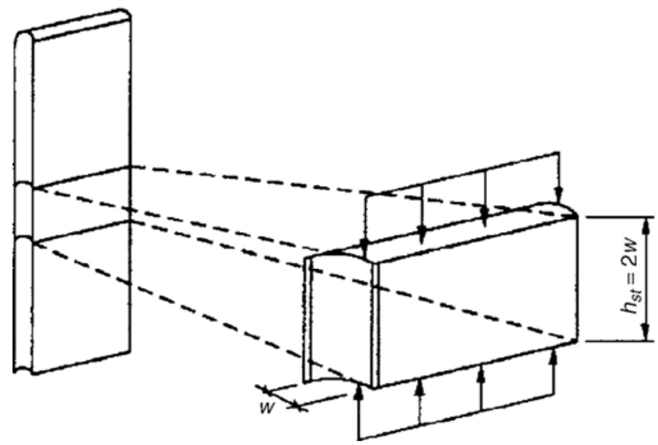

Fig. 5-1—Solid stave.

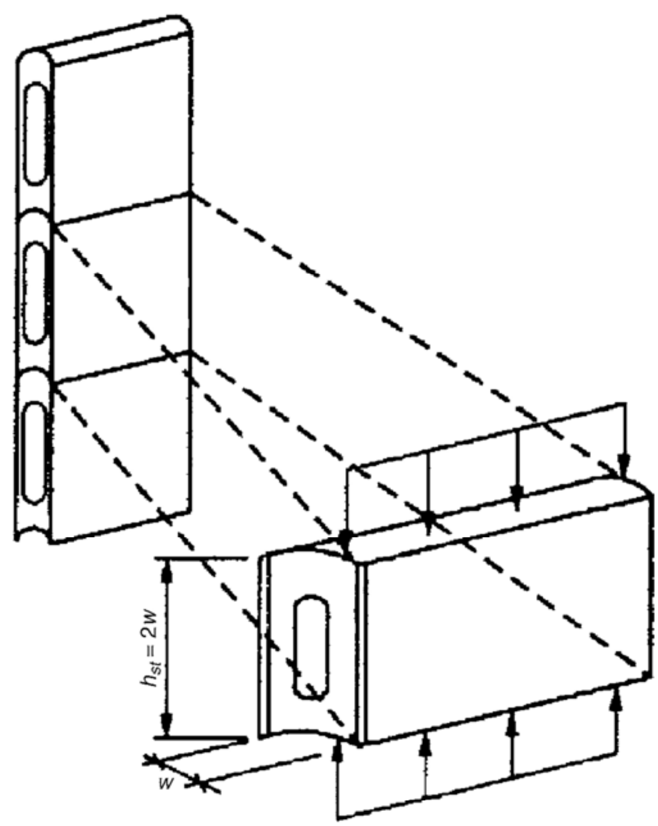

Fig. 5-2—Hollow stave.

5.7—Concrete stave testing

The following procedure shall be used for testing concrete staves to determine compressive strengths:

5.7.1 A given test section shall consist of the full width of a solid stave with the height of this section being twice the thickness of the stave (see Fig. 5-1). The stave shall be tested in a conventional compression testing machine, being loaded by the machine in the same manner as it is loaded in the silo wall.

5.7.2 When testing a cored stave, a section shall be cut with a height twice the thickness of the stave (Fig. 5-2). The maximum depth, however, shall include only one complete core, and no portion of a core shall be present on either top or bottom of the test specimen.

5.7.3 The selection and required number of test sections and the procedures for capping and testing the test sections shall conform to ASTM C 140.

5.7.4 The average minimum compressive strength on the net area shall be at least 4000 psi (28 MPa) at 28 days. The average of any five consecutive stave strength tests shall be equal to or greater than the specified ultimate strength of the concrete, and not more than 20 percent of the tests shall have a value less than the specified strength.

CHAPTER 6—POST-TENSIONED CONCRETE SILOS

6.1—Notation

Consistent units must be used in all equations. Except where noted, units may be either all U.S. Customary or all metric (SI).

D = dead load or dead load effect, or diameter

E = modulus of elasticity

f_{ci} = compressive strength of concrete at time of initial stressing

f_{pu} = specified tensile strength of post-tensioning tendons, wires or strands

f_{py} = specified yield strength of post-tensioning tendons, wires or strands

f_{se} = effective stress in post-tensioning reinforcement (after allowance for all losses)

f_y = specified yield strength of non-prestressed reinforcement

h = wall thickness, including protective cover, if any, over post-tensioning steel

h_1 = core wall thickness

6.2—Scope

6.2.1 Provisions in this chapter apply to cast-in-place concrete silo walls fully or partially post-tensioned with high-strength steel meeting the requirements of Section 3.5.5 of ACI 318. Prestressed systems, where the reinforcement is stressed before the concrete is cast, are not covered herein.

6.2.2 Provisions of other chapters of this Standard and of ACI 318 that do not conflict with provisions of this chapter shall apply.

6.3—Post-tensioning systems

6.3.1 The two most widely used post-tensioning systems for silos are tendon systems and wrapped systems.

6.3.2 Tendon systems use strands, wires or bars inside of ducts. The tendons can be either left unbonded or can be bonded after tensioning by pressure grouting the open space inside the duct. The ducts can be either embedded in the concrete wall or placed on the exterior of the wall. Exterior ducts can be left exposed if constructed of suitable materials. Ducts that cannot be left exposed are protected, usually with shotcrete.

6.3.3 Wrapped systems use high strength wires or strands that are tensioned as they are installed or wrapped around the completed core wall. The wires or strands are protected, usually with shotcrete.

6.4—Tendon systems

6.4.1 Wall thickness, h, for silos with tendons in embedded ducts shall be not less than 10 in. (250 mm), nor less than the sum of h_1 (as determined from Section 6.8.1), the duct diameter, and the concrete cover.

6.4.2 The center-to-center spacing of tendons shall not exceed three times the wall thickness h or h_1, nor, in the case of horizontal tendons, 42 in. (1.07 m).

6.4.3 The clear spacing between embedded tendon ducts shall be not less than three times the duct diameter or 6 in. (150 mm), whichever is larger. The clear spacing between non-embedded tendon ducts shall be not less than the duct diameter or 3/4 in. (20 mm), whichever is larger.

6.4.4 Horizontal embedded tendons shall be placed inside the outside face vertical wall reinforcement.

6.4.5 Stressing points may be located at vertical pilasters on the outside of the walls, at wall intersections or at blockouts. In determining the number of stressing points, consideration shall be given in design to friction loss and local concentrations of the post-tensioning force. Blockout sizes and locations shall be such that, at the time of initial stressing, the stress in the net concrete wall area remaining shall not exceed $0.55f_{ci}$ during the post-tensioning procedure. Methods of staggering pilaster and blackout stressing points are shown in Fig. 6-1.

6.4.6 Reinforcement shall be provided at vertical pilasters as required to resist forces created by the post-tensioning system during and after the stressing operation. Fig. 6-1 shows one possible arrangement.

6.4.7 Embedded tendon ducts shall have a concrete cover of not less than 1-1/2 in. (40 mm). Tendon ducts shall be supported to maintain location within vertical and horizontal tolerances.

6.4.8 Tendon anchor locations shall be staggered such that stressing points do not coincide in vertical array more often than every second tendon.

6.4.9 After stressing is completed, anchorage and end fittings shall be permanently protected against corrosion. Blockouts and pockets shall be filled with a non-shrink material that will bond to and develop the strength of the adjacent concrete.

6.5—Bonded tendons

6.5.1 Anchorages and couplers for bonded tendons shall meet the requirements of ACI 318-95. Tests of anchorages and couplers shall be performed on unbonded specimens.

6.5.2 Grout for bonded tendons shall consist of portland cement and water, or portland cement, fine aggregate and water.

6.5.3 Grout shall have at least 2500 psi (17 MPa) compressive strength at 7 days based on 2 in. (50 mm) cubes, molded, cured and tested in accordance with ASTM C 1019.

6.5.4 Proportions of grouting materials shall be based on results of fresh and hardened grout tests made prior to beginning work. Water content shall be the minimum necessary for proper placement, but in no case more than 0.45 times the content of cement by weight.

6.5.5 Grout shall be mixed and placed by equipment capable of providing a continuous flow of grout at a rate and pressure that will uniformly distribute the grout and fill the

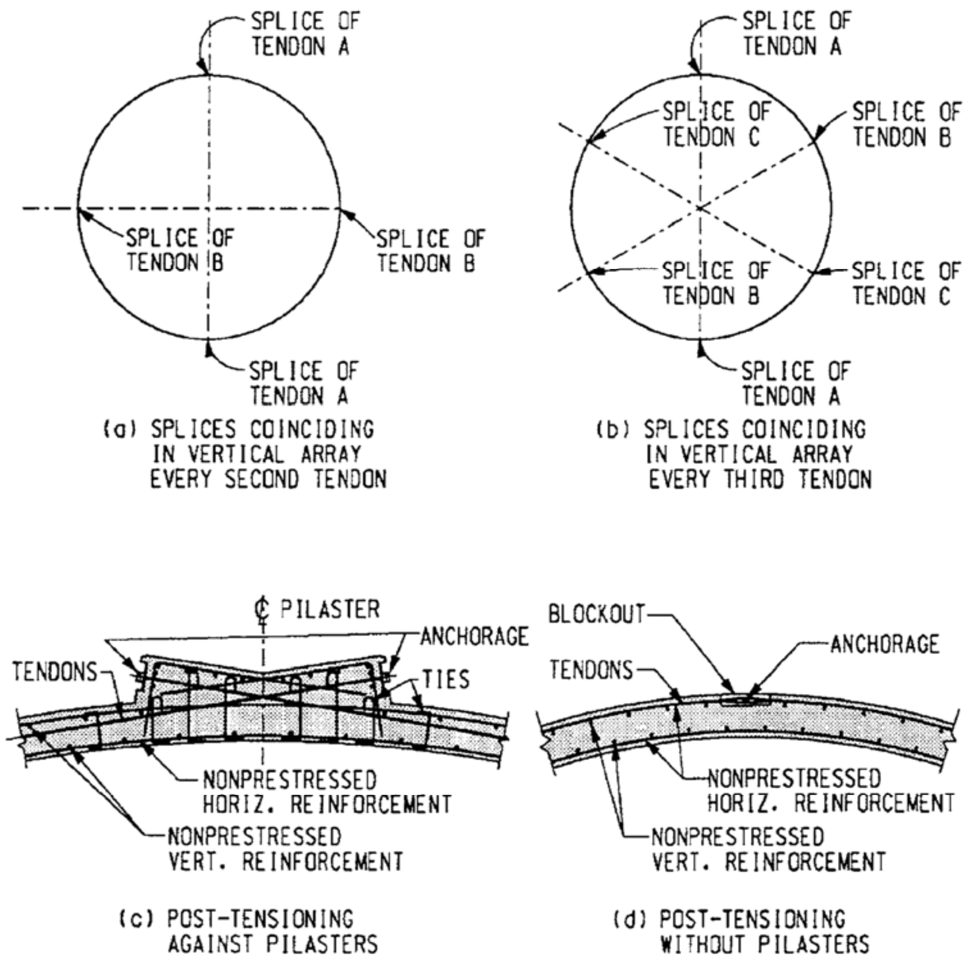

Fig. 6-1—Circumferential prestressing details.

voids in the duct. Grout shall be continuously agitated and placed as quickly as possible after mixing. Grout shall be filtered through a screen to remove lumps and coarse material that would plug the grout tubes and ducts. Grout shall be allowed to flow from the vents to ensure that free water is expelled from the duct. After full flow is obtained, vents shall be closed, pumping stopped and the system checked for leaks while pressure is maintained.

6.5.6 The temperature of the members at the time of grouting shall be above 35 °F (2 °C) and shall be maintained above this temperature until job-cured 2 in. (50 mm) cubes of grout tested as defined by ASTM C 1019 reach a minimum compressive strength of 800 psi (6 MPa). Grout temperature shall be not greater than 90 °F (32 °C) during mixing and injection. Grout shall be cooled during hot weather to avoid quick setting and blockage.

6.6—Unbonded tendons

6.6.1 Anchorages and couplers for unbonded tendons shall develop 100 percent of f_{pu} without exceeding the anticipated set. Cyclic loading and unloading of the silo that might lead to fatigue failure of anchorages or couplers shall be considered in the selection of anchorages.

6.6.2 External and internal unbonded tendons shall be coated with a protective lubricant and encased in a protective duct or wrapping to provide long-term corrosion protection. The tendon duct or wrapping shall be continuous over the entire zone to be unbonded. It shall prevent intrusion of cement paste or water (or both) and the loss of coating materials during concrete placement. The anchorage and end fittings shall be protected as specified in Section 6.4.9.

6.7—Post-tensioning ducts

6.7.1 Ducts for grouted or unbonded tendons shall be mortar-tight and non-reactive with concrete, tendons or the grout. Metal duct walls shall be no thinner than 0.012 in. (0.3 mm). Duct splices shall be staggered and ducts shall be installed free of kinks or unspecified curvature changes.

6.7.2 Ducts for grouted single wire, strand or bar tendons shall have an inside diameter at least 1/4 in. (6 mm) larger than tendon diameter.

6.7.3 Ducts for grouted multiple wire, strand or bar tendons shall have an inside cross-sectional area at least two times the net area of tendons.

6.7.4 In addition to meeting the requirements of Sections 6.7.2 and 6.7.3, duct diameter shall be compatible with tendon installation requirements, taking into consideration curvature of wall, duct length, potential blockage and silo configuration.

6.7.5 Ducts shall be kept clean and free of water. Grouting shall be performed as soon after post-tensioning as possible. When grouting is delayed, the exposed elements of the system shall be protected against intrusion of water or any foreign material that may be detrimental to the system.

6.7.6 Ducts for grouted tendons shall be capable of transferring bond between tendons and grout to the surrounding concrete.

6.8—Wrapped systems

6.8.1 Core wall thickness, h_1, for silos with wires or strands wound around the outside face of the core wall shall be not less than 6 in. (150 mm) nor less than that required to prevent the stress on the core wall from exceeding $0.55f_{ci}$ at the time of initial stressing.

6.8.2 Large voids or other defects in the core wall shall be chipped down to sound concrete and repaired before post-tensioning commences. Dust, efflorescence, oil, and other foreign material shall be removed. Concrete core walls shall have a bondable surface and may require sandblasting.

6.8.3 Procedures used for post-tensioning by wrapping shall be as approved by the engineer.

6.8.4 Pitch of high-tensile wire in spiral wrapping and simultaneous stressing is to be determined by requirements of the tensile forces caused by stored material lateral pressures. A clear distance of at least 1/4 in. (6 mm), but not less than one wire diameter, shall be left between successive turns of wire.

6.8.5 If multiple-layer wrapping is used, the layers shall be separated by shotcrete, conforming to ACI 506.2.

6.8.6 The outside post-tensioning wires or strand shall be coated by two or more layers of shotcrete. The total coating thickness over the wires or strand shall be not less than 1 in. (25 mm). Shotcrete coating shall conform to requirements of ACI 506.2.

6.9—Details and placement of non-prestressed reinforcement

6.9.1 Vertical non-prestressed reinforcement shall be provided to withstand bending moments resulting from post-tensioning, banding of post-tensioning reinforcement at openings, stored material loads (partially full and full), temperature and other loading conditions to which the walls are subjected. The area of vertical non-prestressed reinforcement provided shall be not less than that required by Chapter 4.

6.9.2 Horizontal non-prestressed reinforcement shall be provided to withstand bending moments from all causes and to control shrinkage and temperature-induced cracking during the period between completion of wall construction and start of post-tensioning. In any case, the total area of such reinforcement shall be not less than 0.0025 times the area of the wall. The spacing of horizontal non-prestressed reinforcement provided shall be not more than 18 in. (450 mm).

6.9.3 Number 4 (10M) stirrups at 2.5 ft. (0.75m) c/c each way shall be provided in post-tensioned walls.

6.10—Wall openings

6.10.1 For wall openings not within pressure zone, see Section 4.3.8.2.

6.10.2 For wall openings in pressure zones, post-tensioning elements that would cross an opening shall be flared to pass immediately above and below the opening. The length of flare, measured from the center of the opening, shall be not more than the silo diameter nor less than six times the opening height. Horizontal and vertical stress concentrations resulting from flaring of tendons around openings shall be considered for cases of both full and empty silos. Minimum spacing requirements shall be observed at all locations.

6.10.3 Vertical reinforcement at each side of the opening shall be not less than the minimum required by Section 4.3.8, nor less than that calculated for the vertical bending moments or forces due to flaring the post-tensioning elements.

6.11—Stressing records

6.11.1 Stressing procedures shall be documented and the records submitted to the engineer and preserved for the period specified on the project drawings and project specifications, but not less than 2 years. Records shall include type, size and source of wire, strand or bars, date of stressing, jacking pressures, sequence of stressing, elongation before and after anchor set, any deviations from expected response from jacking, and name of inspector.

6.12—Design

6.12.1 Design shall be based on the strength method and on behavior at service conditions at all load stages that may be critical during the life of the structure from the time post-tensioning stress is first applied.

6.12.2 Silo walls shall be designed to resist all applicable loads as specified in Chapter 4, plus the effect of post-tensioning forces during and after tensioning, including stress concentration and conditions of edge restraint at wall junctions with silo roof, bottom and wall intersections.

6.12.3 Stresses in concrete shall not exceed the values provided in Chapter 4 and in Section 18.4 of ACI 318, except as provided in Table 6.1.

6.12.4 Tensile stresses in strands, wires or bars of tendon systems shall not exceed the following:

(a) During jacking .... $0.85f_{pu}$ or $0.94f_{py}$ whichever is smaller, but not more than maximum value recommended by the manufacturer of tendons or anchorages.

(b) Immediately after anchoring .... $0.70f_{pu}$

Average stresses in wires or strands used in wrapped systems shall not exceed the following:

(a) Immediately after stressing .... $0.70f_{pu}$

(b) After all losses.. $0.55f_{pu}$

6.12.5 *Required area of post-tensioning reinforcement*—Prestressed reinforcement or a combination of prestressed and non-prestressed reinforcement shall be provided to resist the hoop tension due to horizontal pressures computed according to Section 4.4.2.2. In silo walls subjected to combined hoop tension and bending, resistance to bending shall be provided by non-prestressed reinforcement.

When post-tensioned reinforcement and non-prestressed reinforcement are considered to act together to provide the required resistance to axial tension or to combined axial

Table 6.1—Maximum permissible stresses in concrete (at service loads, after allowance for all losses)

	Fully post-tensioned	Partially post-tensioned
Axial compression	$0.30f_c'$	$0.225f_c'$
Combined axial and bending compression-extreme fiber	$0.45f_c'$	$0.45f_c'$
Axial tension	0	$6\sqrt{\ '}$ psi
	(0)	($0.5\sqrt{\ '}$ MPa)
Combined axial and bending tension-extreme fiber	$6\sqrt{\ '}$ psi	$12\sqrt{\ '}$ psi
	($0.5\sqrt{\ '}$ MPa)	($1.0\sqrt{\ '}$ MPa)

tension and bending in the wall, the assumed stresses in each type of reinforcement shall be determined based on stress-strain compatibility relationships.

6.12.6 The modulus of elasticity, E, of post-tensioning reinforcement shall be based on data supplied by the manufacturer or shall be determined by independent tests. Unless more accurate information is available, the following values shall be used:

Bars .... 29×10^6 psi (200×10^3 MPa)
Strands .... 28.5×10^6 psi (197×10^3 MPa)
Wires .... 29×10^6 psi (200×10^3 MPa)

6.12.7 ***Non-prestressed reinforcement***

6.12.7.1 Amount of non-prestressed reinforcement shall be determined by the strength design method as specified in ACI 318. The amount of non-prestressed reinforcement provided, however, shall be not less than required by Sections 6.9 and 6.10.

6.12.7.2 Yield strength (f_y) of non-prestressed reinforcement shall not be taken in excess of 60,000 psi (414 MPa).

6.12.7.3 The modulus of elasticity of non-prestressed reinforcement shall be taken as 29×10^6 psi (200×10^3 MPa).

6.12.8 Where a circular wall is post-tensioned within a distance of 10 wall thicknesses of a roof, silo bottom, foundation or other intersecting structural member, the minimum initial concrete circumferential compression stress, for a height of wall extending from $0.4\sqrt{Dh}$ to $1.1\sqrt{Dh}$, shall be not less than:

Edges unrestrained .... 280 psi (2.0 MPa)
Edges restrained .... 140 psi (1.0 MPa)

6.12.9 *Losses*—Stress losses that are used to establish the effective stress, f_{se}, shall be determined using the provisions of ACI 318, Section 18.6.

6.13—Vertical bending moment and shear due to post-tensioning

Non-prestressed vertical reinforcement shall be provided to resist vertical bending moments and shear forces due to post-tensioning.

6.14—Tolerances

6.14.1 Tolerances for placement of ducts at support points, relative to position shown on the project drawings, shall not exceed 1 in. (25 mm) vertically or horizontally.

6.14.2 The vertical sag or horizontal displacement between support points shall be not greater than 1/2 in. (13 mm).

CHAPTER 7—STACKING TUBES

7.1—Scope

This chapter covers the design and construction of reinforced concrete stacking tubes. Unless specifically stated otherwise, all general requirements in Chapters 1, 2, 3, and 4 (where not in conflict with this chapter), are applicable to stacking tubes.

7.2—General layout

The inside dimension shall be large enough to prevent arching across the tube. Wall discharge openings shall be large enough to prevent arching across the openings and allow free flow of material from the stacking tube. The discharge openings shall be symmetrically arranged in sets of two and 180° apart with alternate sets located at 90° to each other.

The wall between the foundation and the bottom of the first set of openings shall be sufficient to provide the necessary strength to resist the internal pressures as well as the external uneven pile loads. Discharge openings shall be located over the height of the tube in such a way as to minimize the effects of ring bending from uneven loads. If a concentric discharge is provided inside the tube through the reclaim tunnel roof at the bottom of the stacking tube, it shall be large enough to prevent arching across the opening and prevent the formation of a stable rathole in the tube.

7.3—Loads

The following loads shall be considered for the design of stacking tubes:

7.3.1 ***Vertical loads at top of tube***

a) The vertical reaction from the weight of the conveyor and headhouse structure.

b) The vertical reaction from the walkway live load, headhouse floor live load and the weight of material carried by the conveyor.

7.3.2 ***Horizontal loads at top of tube***

7.3.2.1 Acting perpendicular to the conveyor:

a) The horizontal reaction from wind on the conveyor and headhouse.

b) The horizontal reaction from seismic force on the conveyor and headhouse.

7.3.2.2 Acting parallel to the conveyor:

a) The horizontal reaction due to the belt pull, including tension from start-up.

b) The horizontal reaction due to thermal expansion or contraction of the conveyor support structure. Such force shall be taken as not less than 10 percent of the total (dead plus live) vertical reaction of the conveyor system on the top of the tube, unless provision is made to reduce the loads with rollers or rockers.

7.3.3 ***Vertical loads over the height of the tube***

a) The weight of the tube.

b) The vertical drag force from the material stored inside the tube.

c) The vertical drag force from a complete pile of material stored outside the tube.

Table 7.1—Load combinations

Types of load acting on tube	Loading cases						
	1	2	3	4	5	6	7
Vertical load (7.3.1) caused by:							
a) Conveyor and headhouse dead load	+1.4	+1.4	+1.4	+1.4	+0.9	+0.9	+0.9
b) Conveyor and headhouse live load	+1.7	+1.7	+1.7	+1.7			
Horizontal load (7.3.2.1) caused by:							
a) Wind on conveyor and headhouse			+1.7			–1.7	
b) Seismic on conveyor and headhouse				+1.87			–1.87
Longitudinal load (7.3.2.2) caused by:							
a) Belt pull of conveyor		+1.7	+1.7	+1.7	–1.7	–1.7	–1.7
b) Thermal changes of conveyor		+1.4	+1.4	+1.4	–1.4	–1.4	–1.4
Tube vertical load (7.3.3) caused by:							
a) Dead load of tube	+1.4	+1.4	+1.4	+1.4	+0.9	+0.9	+0.9
b) Material inside tube, if any	+1.7	+1.7	+1.7	+1.7	+0.9	+0.9	+0.9
c) Complete pile outside tube	+1.7						
d) Partial pile outside tube		+1.7	+1.7	+1.7	+0.9	+0.9	+0.9
Tube horizontal load (7.3.4) caused by:							
a) Wind on exposed portion of tube			+1.7			–1.7	
b) Seismic on tube mass				+1.87			+1.87
c) Unbalanced loads from partial pile		+1.7	+1.7	+1.87	–1.7	–1.7	–1.7
d) Seismic on material in tube, if any				+1.87			–1.87
e) Seismic on partial pile outside				+1.87			–1.87
Multiplier	1.00	1.00	0.75	0.75	1.00	0.75	0.75

d) The vertical drag force from a partial pile of material stored outside the tube.

7.3.4 ***Horizontal loads over the height of the tube***

a) Wind action on the exposed portion of the tube.

b) Seismic action on the mass of the tube.

c) Unbalanced forces acting on the tube as a result of a partial pile of material stored around the tube. Such forces shall be computed assuming that the conical pile is missing a radial sector and that the tube has reduced lateral support from stored material on the open side.

d) Seismic action on the material stored inside the tube.

e) Seismic action on the partial pile stored on the outside of the tube.

7.4—Load combinations

Unless it can be shown that a particular specified load combination does not apply, the required strength of the stacking tube shall be not less than that indicated for each of the loading cases of Table 7.1. The required strength is obtained by summing the loads in each column and multiplying by the multiplier at the bottom of the column.

Each of the loading cases in Table 7.1 shall be investigated to determine the critical design force at the base of the tube.

1. Maximum downward considering vertical loads only.
2. Maximum downward considering vertical and horizontal loads.
3. Maximum downward considering vertical, horizontal and wind loads.
4. Maximum downward considering vertical, horizontal and seismic loads.
5. Maximum upward considering vertical and horizontal loads.
6. Maximum upward considering vertical, horizontal and wind loads.
7. Maximum upward considering vertical, horizontal and seismic loads.

Where tubes are in close proximity to each other, consideration shall be given to possible increases in horizontal loads due to horizontal arching of the stacked material between tubes.

7.5—Tube wall design

7.5.1 The stacking tube shall be designed as a cantilevered beam fixed at the top of the foundation or reclaim tunnel roof. The concrete wall thickness and reinforcing shall be such that its strength will not be less than required by the most severe combination of loads of Table 7.1 at the base of the tube and at each level of discharge opening above.

7.5.2 The stacking tube wall shall be reinforced vertically and horizontally. For wall thicknesses of 9 in. (225 mm) or more, reinforcing shall be provided on each face. The vertical reinforcement shall be designed to resist the maximum tensile stresses resulting from the combination of vertical loads and overturning moments. In addition, the vertical reinforcing adjacent to the openings shall be designed to resist the bending and shear stresses resulting from the bending action of the wall between the openings. The ratio of vertical reinforcement to gross concrete area shall not be less than 0.0025.

7.5.3 Horizontal reinforcement shall be designed to resist hoop and circumferential bending stresses and horizontal tension caused by the redistribution of vertical loads around the openings. Horizontal reinforcement that is discontinuous at openings shall be replaced by adding not less than 60 percent of the interrupted reinforcement above the top and 60 percent below the bottom of the opening. The ratio of horizontal reinforcement to gross concrete area shall not be less than 0.0025.

7.6—Foundation or reclaim tunnel

The foundation or reclaim tunnel shall be designed to support all horizontal and vertical loads on the tube and to be stable against the overturning moments. In addition, the foundation or reclaim tunnel shall be designed to support the material above and adjacent to the tube and the tunnel.

CHAPTER 8—SPECIFIED AND 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

117	Standard Tolerances for Concrete Construction and Materials
214	Recommended Practice for Evaluation of Strength Test Results of Concrete
214.1R	Use of Accelerated Strength Testing
215R	Considerations for Design of Concrete Structures Subjected to Fatigue Loading
301	Specifications for Structural Concrete for Buildings
305R	Hot Weather Concreting
306R	Cold Weather Concreting
308	Standard Practice for Curing Concrete
318	Building Code Requirements for Structural Concrete
344R-W	Design and Construction of Circular Wire and Strand Wrapped Prestressed Concrete Structures
347R	Guide to Formwork for Concrete
506.2	Specification for Materials, Proportioning and Application of Shotcrete
515.1R	Guide to the Use of Waterproofing, Dampproofing, Protective and Decorative Barrier Systems for Concrete

ASTM International

A 47	Specification for Ferritic Malleable Iron Castings
A 123	Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products
C 55	Standard Specification for Concrete Building Brick
C 109	Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2 in. or 50 mm Cube Specimens)
C 140	Standard Methods of Sampling and Testing Concrete Masonry Units
C 150	Standard Specification for Portland Cement
C 309	Standard Specification for Liquid Membrane-Forming Compounds for Curing Concrete
C 426	Standard Test Method for Drying Shrinkage of Concrete Block
C 595	Standard Specification for Blended Hydraulic Cements
C 684	Standard Test Method of Making, Accelerated Curing and Testing of Concrete Compression Test Specimens
C 845	Standard Specification for Expansive Hydraulic Cement
C 1019	Standard Test Method of Sampling and Testing Grout

International Conference of Building Officials

1994 Edition Uniform Building Code

The above publications may be obtained from the following organizations:

American Concrete Institute
P.O. Box 9094
Farmington Hills, Mich. 48333-9094

ASTM International
100 Barr Harbor Dr.
West Conshohocken, Pa. 19428-2959

International Conference of Building Officials
5360 South Workman Mill Rd.
Whittier, Calif. 90601

APPENDIX A—NOTATION

Consistent units must be used in all equations. Except where noted, units may be either all U.S. Customary or all metric (SI).

A = effective tension area of concrete surrounding the tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars. When the reinforcement consists of different bar sizes, the number of bars shall be computed as the total area of reinforcement divided by the area of the largest bar used. See Fig. 4-3.
A_s = area of hoop or tension reinforcement, per unit height
A_w = effective cross-sectional area (horizontal projection) of an individual stave
D = dead load or dead load effect, or diameter
E = modulus of elasticity
E_c = modulus of elasticity for concrete
F_u = required hoop or horizontal tensile strength, per unit height of wall
L = live load or live load effect
M = stored material load stress
M_{pos}, M_{neg} = positive (tension inside face) and negative (tension outside face) circumferential bending moments, respectively, caused by asymmetric filling or emptying under service load conditions
M_θ = circular bending strength for an assembled circular group of silo staves, per unit height; the statical moment or sum of absolute values of $M_{\theta,pos}$ and $M_{\theta,neg}$
$M_{\theta,pos}$ = the measured or computed bending strengths in

$M_{\theta,neg}$ the positive moment zone and negative moment zone, respectively

M_t = thermal bending moment per unit width of height of wall (consistent units)

P_{nw} = nominal axial load strength of wall per unit perimeter

$P_{nw,buckling}$ = strength of the stave wall as limited by buckling

$P_{nw,joint}$ = strength of the stave wall as limited by the stave joint

$P_{nw,stave}$ = strength of the stave wall as limited by the shape of the stave

R = ratio of area to perimeter of horizontal cross-section of storage space

T = temperature or temperature effect

ΔT = temperature difference between inside face and outside face of wall

U = required strength

V = total vertical frictional force on a unit length of wall perimeter above the section in question

W = tension force per stave from wind over-turning moment

Y = depth from the equivalent surface of stored material to point in question. See Fig. 4-2.

d_c = thickness of concrete cover taken equal to 2.5 bar diameters, or less. See Fig. 4-3.

e = base of natural logarithms

f'_c = compressive strength of concrete

f_{ci} = compressive strength of concrete at time of initial stressing

f_{pu} = specified tensile strength of post-tensioning tendons, wires or strands

f_{py} = specified yield strength of post-tensioning tendons, wires or strands

f_s = calculated stress in reinforcement at initial (filling) pressures

f_{se} = effective stress in post-tensioning reinforcement (after allowance for all losses)

f_y = specified yield strength of non-prestressed reinforcement

h = wall thickness

h_h = height of hopper from apex to top of hopper. See Fig. 4-2.

h_s = height of sloping top surface of stored material. See Fig. 4-2.

h_{st} = height of stave specimen for compression test. See Fig. 5-1 and 5-2.

h_y = depth below top of hopper to point in question. See Fig. 4-2.

h_1 = core wall thickness

k = p/q

p = initial (filling) horizontal pressure due to stored material

p_n = pressure normal to hopper surface at a depth h_y below top of hopper. See Fig. 4-2.

q = initial (filling) vertical pressure due to stored material

q_o = initial vertical pressure at top of hopper

q_y = vertical pressure at a distance h_y below top of hopper. See Fig. 4-2.

s = bar spacing, in. See Fig. 4-3.

v_n = initial friction force per unit area between stored material and hopper surface calculated from Eq. (4-8) or (4-9)

w = design crack width, in. or lateral wind pressure

α = angle of hopper from horizontal. See Fig. 4-2.

α_c = thermal coefficient of expansion of concrete

γ = weight per unit volume for stored material

θ = angle of hopper from vertical. See Fig. 4-2.

μ' = coefficient of friction between stored material and wall or hopper surface

ν = Poisson's ratio for concrete, assumed to be 0.2

ϕ = strength reduction factor or angle of internal friction

ϕ' = angle of friction between material and wall and hopper surface

ρ = angle of repose. See Fig. 4-2.

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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This Commentary presents some of the considerations and assumptions of ACI Committee 313 in developing the provisions of the Standard Practice for Design and Construction of Concrete Silos and Stacking Tubes for Storing Granular Materials. *It also provides suggested methods for calculating crack width and through-the-wall temperature gradient due to hot stored materials.*

Comments on specific provisions of the Standard practice are made using the corresponding chapter and section numbers of the Standard practice. A list of selected references is given at the end of the Commentary. Notations, not defined herein, are defined in Appendix A of the Standard.

Keywords: asymmetric flow; **bins**; circumferential bending; concrete; **concrete construction**; **dead loads**; dynamic loads; earthquake resistant structures; formwork (construction); funnel flow; granular materials; hoppers; jumpforms; **lateral loads**; **loads (forces)**; lowering tubes; mass flow; overpressure; quality control; reinforced concrete; reinforcing steels; **silos**; slipform construction; stacking tubes; stave silo; **stresses**; structural analysis; **structural design**; thermal stresses; thickness; walls.

CONTENTS

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CHAPTER 1—GENERAL

R1.1—Introduction

Silo failures have alerted design engineers to the danger of designing silos for only static pressures due to stored material at rest. Those failures have inspired wide-spread research into the variations of pressures and flow of materials. The research thus far has established beyond doubt that pressures during withdrawal may be significantly higher[1-4] or significantly lower than those present when the material is at rest. The excess (above static pressure) is called "overpressure" and the shortfall is called "underpressure." One of the causes of overpressure is the switch from active to passive conditions which occurs during material withdrawal.[5] Underpressures may occur at a flow channel in contact with the wall and overpressures may occur away from the flow channel at the same level.[6-8] Underpressures concurrent with overpressures cause circumferential bending in the wall. Impact during filling may cause total pressure to exceed the static. While overpressures and underpressures are generally important in deeper silos, impact is usually critical only for shallow ones (bunkers) in which large volumes are dumped suddenly.

Obviously, to design with disregard for either overpressure, underpressure or impact could be dangerous.

R1.2—Definitions

The term "silo" used here includes both deep bins and shallow bins, the latter sometimes referred to as "bunkers." Wherever 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 industry 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.

R1.4—Drawings, specifications, and calculations

Silos and bunkers are unusual structures, and many engineers are unfamiliar with computation of their design loads and with other design and detail requirements. It is important that the design and the preparation of project drawings and project specifications for silos and bunkers be done under the supervision of an engineer with specialized knowledge and experience in design of such structures.

If possible, the properties of the stored materials to be used in the design should be obtained from tests of the actual materials to be stored or from records of tests of similar materials previously stored. Properties assumed in the design should be stated on the project drawings.

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 1-1/2 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 Types 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} = 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'_c should be attained in over 90 percent of field molded compression specimens provided f'_c 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 28-1/2 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 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 be 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'_s = compression steel area. See Fig. 4-F.
B = constant calculated from Eq. (4D)
K_t = 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).
β = constant calculated from Eq. (4E)
δ = effective angle of internal friction
θ_c, θ_p = 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 overload 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, γ, 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 slipformed 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 differential, 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 empty[11] (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 ancilliary structures.

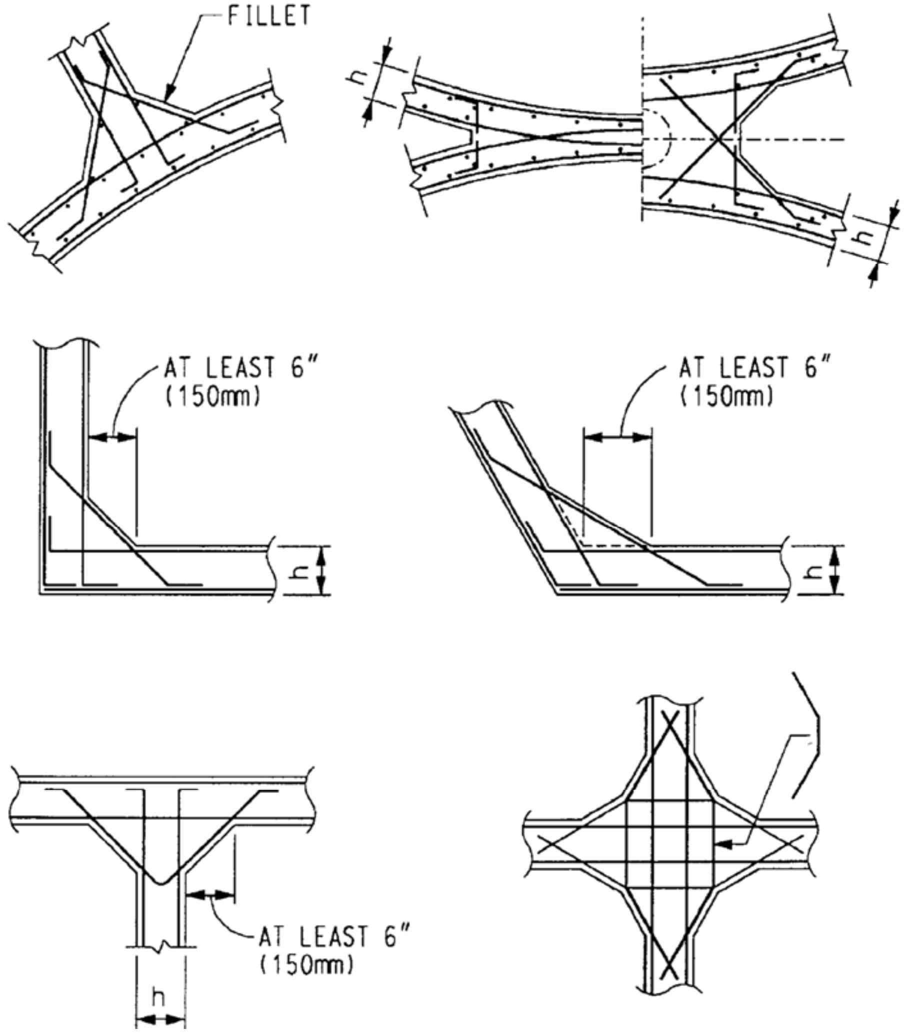


Fig. 4-A—Reinforcement pattern at intersecting walls.

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.[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.[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.[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

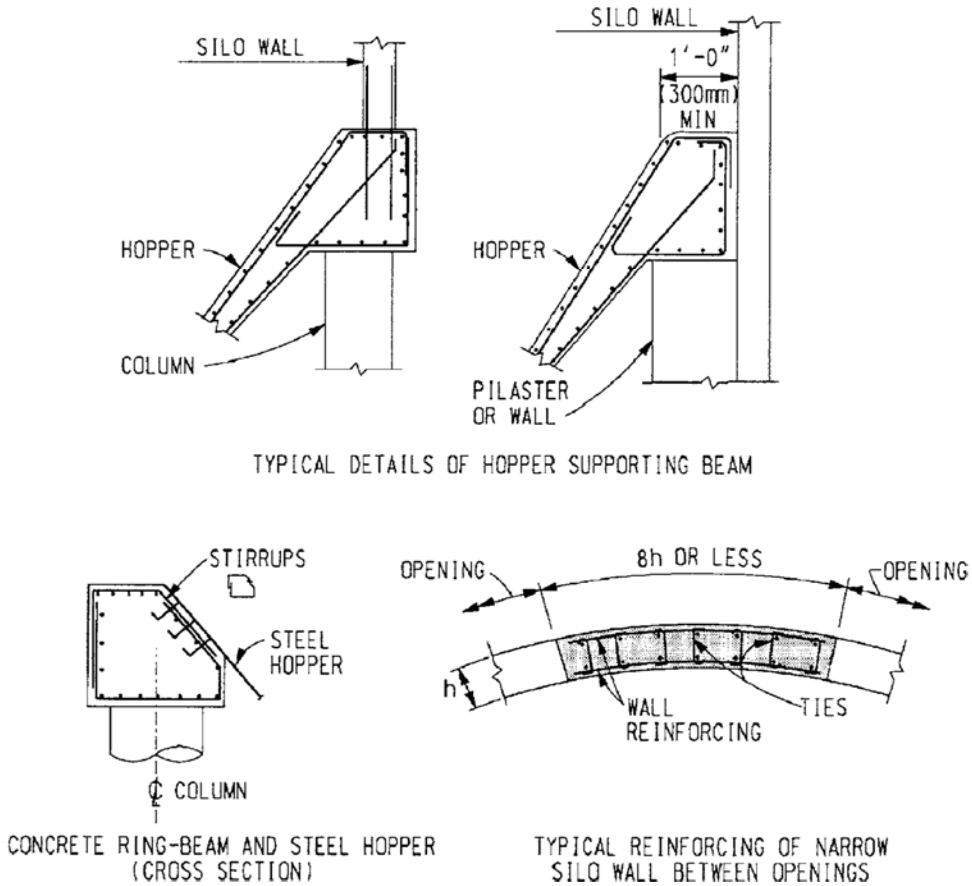


Fig. 4-B—Miscellaneous details.

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.

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

	Weight γ		Angle of internal friction ϕ	Effective angle of internal friction δ	Coefficient of friction μ'	
	lb/ft^3	kg/m^3			Against concrete	Against steel
Cement, clinker	88	1410	33	42-52	0.6	0.3
Cement, portland	84-100	1345-1600	24 to 30	40-50	0.40-0.80	0.30
Clay	106-138	1700-2200	15 to 40	50-90	0.2-0.5	0.36-0.7
Coal, bituminous	50-65	800-1040	32 to 44	33-68	0.55-0.85	0.30
Coal, anthracite	60-70	960-1120	24 to 30	40-45	0.45-0.50	0.30
Coke	32-61	515-975	35-45	50-60	0.50-0.80	0.50-0.65
Flour	38	610	40	23-30	0.30	0.30
Fly ash	50-112	865-1800	35-40	37-42	0.60-0.80	0.47-0.70
Gravel	100-125	1600-2000	25 to 35	36-40	0.40-0.45	0.29-0.42
Grains (small): wheat, corn, barley, beans (navy, kidney), oats, rice, rye	44-62	736-990	20 to 37	28-35	0.29-0.47	0.26-0.42
Gypsum, lumps	100	1600	38-40	45-62	0.5-0.8	0.38-0.48
Iron ore	165	2640	40-50	50-70	0.5-0.8	0.4-0.7
Lime, calcined, fine	70-80	1120-1280	30-35	35-45	0.5-0.7	0.4-0.6
Lime, calcined, coarse	58-75	928-1200	40	40-45	0.5-0.8	0.3-0.5
Limestone	84-127	1344-2731	39-43	45-80	0.6-0.8	0.55-0.70
Manganese ore	125	2000	40			
Sand	100-125	1600-2000	25 to 40	30-50	0.40-0.70	0.35-0.50
Soybeans, peas	50-60	800-960	23		0.25	0.20
Sugar, granular	53-63	1000	35	33-40	0.43	

*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.

(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[17] [Eq. (4-1)], whereas in parts of Europe, Reimbert's method[4] 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,[13,18] McLean[19] and Walker.[20] 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 parameters, 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 γ, k, and μ'. The design should be based on maximum γ with appropriate combinations of maximum and minimum values of k and μ'.

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 $\boldsymbol{Y}$ 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, nonuniformity 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 × 1.5 × 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.[21-33] 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.[20,21,34,35] Eqs. (4-5) through (4-9), which are based on Walker,[20] provide a generally acceptable method to estimate initial pressures in hoppers. Eq. (4-5) reflects Walker's assumption of an incompressible material and, therefore, yields conservative pressures near the outlets of steep hoppers. However, some pressure measurements reported in the technical literature[36,37] are not significantly lower than those predicted by Eq. (4-5) in the lower part of the hopper.

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 $\boldsymbol{k}$ 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 $\boldsymbol{p_n}$.

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 $\alpha \geq \phi' + 25°$. Some designers select α 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[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 θ_c and θ_p and the hopper wall friction angle ϕ'. 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.[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 θ_c. For transition hoppers, the side slope should be steeper than θ_p, and the slope of the curved end walls should be steeper than θ_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.[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[13,39] and Walker.[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[20] follows.

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

$$q_y = \frac{\gamma}{n -}\left(h_h - h_y\left[\; - \left(\frac{h_h - h_y}{h_h}\right)^{n -}\;\right] + q_o\left[\frac{h_h - h_y}{h_h}\right]^n \qquad \text{(4A)}$$

where $\boldsymbol{q_o}$ is computed by Eq. (4-1) and,

for circular cones, $n = 2B/\tan\theta$ (but not less than 1.0) (4B)

for Plane Flow Hoppers, $n = B/\tan\theta$ (but not less than 1.0) (4C)

where

$$B = \frac{\sin\delta\sin2(\theta+\beta)}{1 - \sin\delta\cos2(\theta+\beta)} \qquad \text{(4D)}$$

and

$$\beta = 1/2\left[\phi' + arc\sin\frac{\sin\phi'}{\sin\delta}\right] \qquad \text{(4E)}$$

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

$$p_n = \frac{1 + \sin\delta\cos(2\beta)}{1 - \sin\delta\cos2(\theta+\beta)}q_y \qquad \text{(4F)}$$

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 $\boldsymbol{p_n}$ computed by Eq. (4F).

Pressures in mass flow tilted hoppers, where the angle between the hopper axis and the vertical does not exceed θ_c or θ_p, may be computed using this method with θ 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.

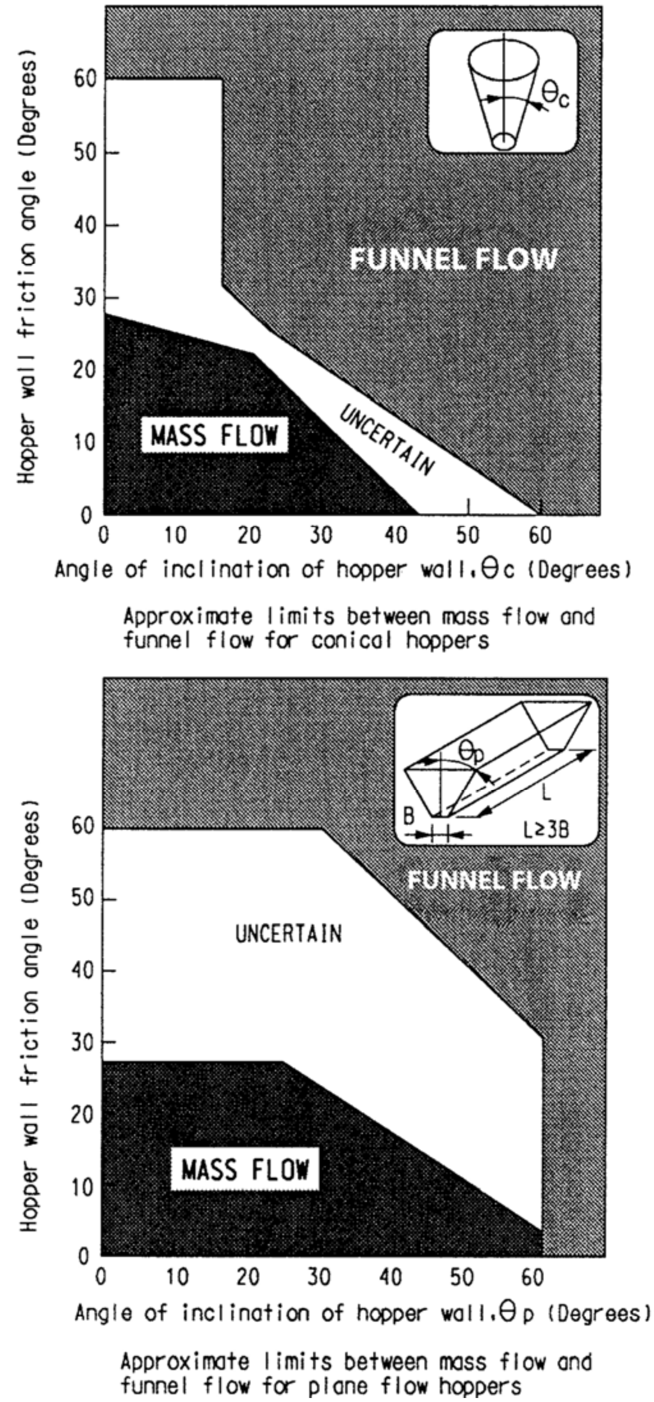


Fig. 4-C—Mass flow versus funnel flow bounds.

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

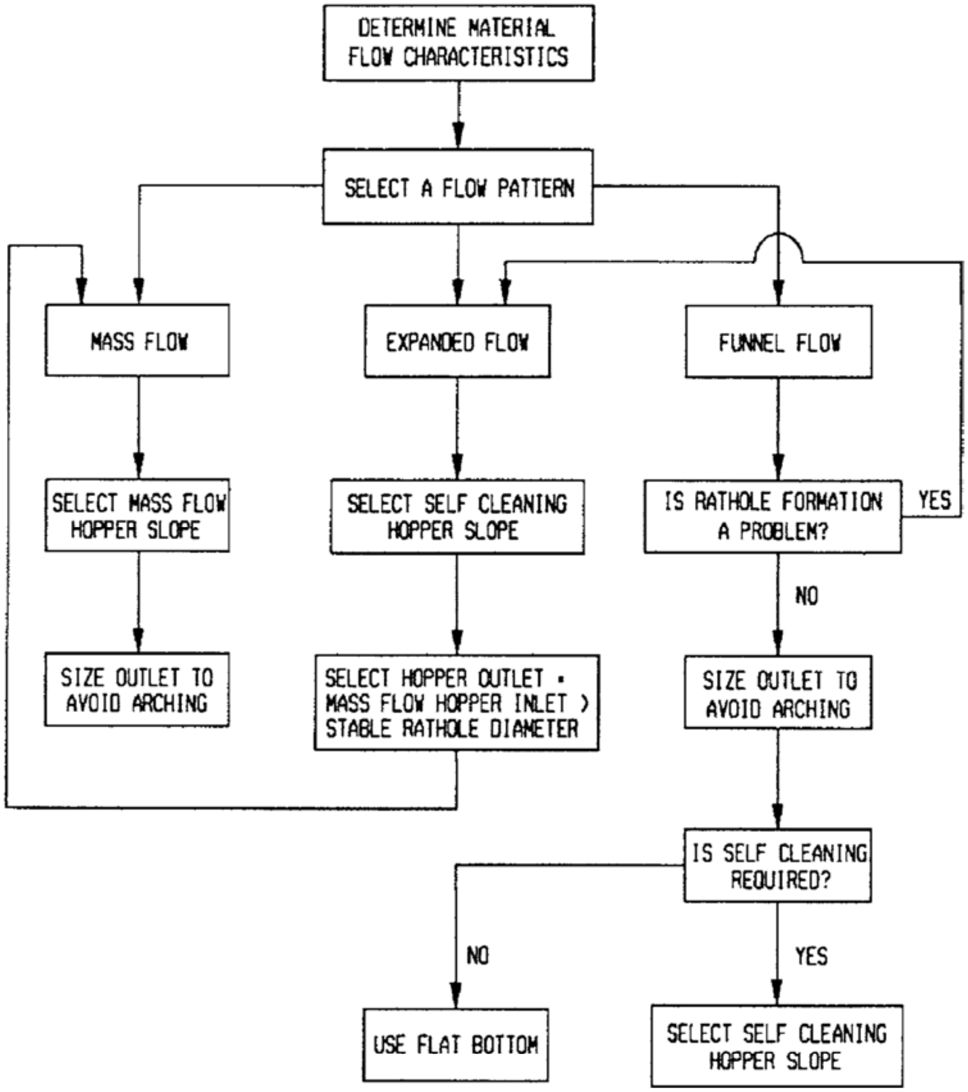


Fig. 4-D—Flow chart for selecting hopper configuration.

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]

$$\Delta T = (T_i - T_o - 80\ °\mathrm{F})K_t \qquad (4G)$$

where K_t 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.

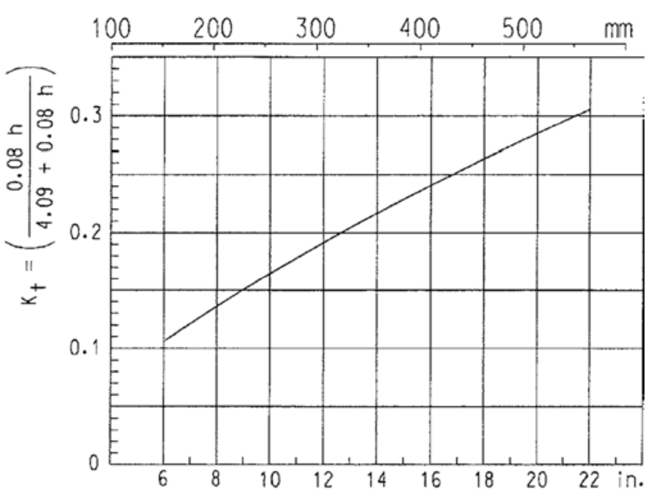
R4.5—Wall design

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:



THE ABOVE CURVE IS BASED ON THE FOLLOWING ASSUMPTIONS:[5]

1. Resistance of 8 in. (203 mm) cement (considered to act as insulating material) = 3.92
2. Resistance of 1 in. (25.4 mm) thick concrete = 0.08
3. Resistance of outer surface film = 0.17

Fig. 4-E—Determination of K_t *for use in computing* ΔT *for a wall of a cement storage silo.*

$$A_s = \frac{F_u e'}{\phi f_y (d - d')} \tag{4H}$$

on the side nearest to force F_u, and

$$A'_s = \frac{F_u e''}{\phi f_y (d - d')} \tag{4I}$$

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

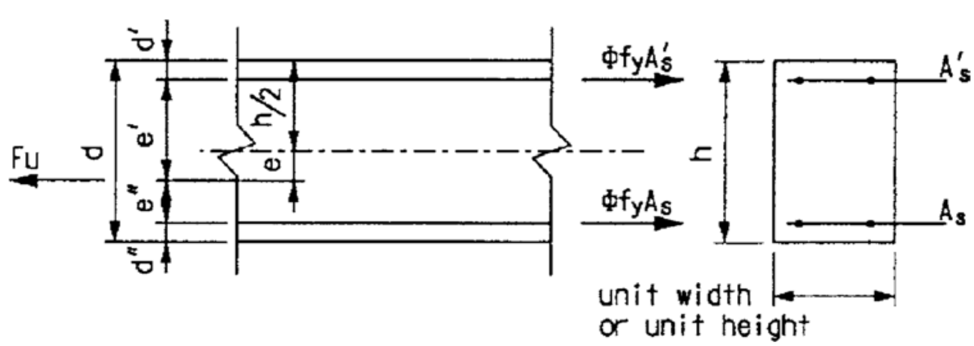


Fig. 4-F—Axial tension and flexure with small eccentricity.

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_s is the stress in the reinforcement under initial filling pressures computed by Equations (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 = 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 straightedge 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

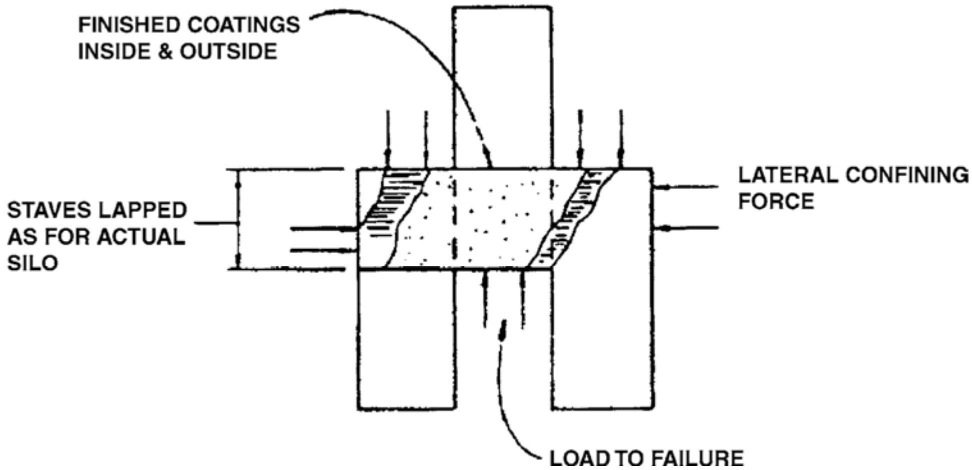


Fig. 5-A—Stave assembly joint shear (tension) test.

$0.875(\phi A_s f_y - F_u)h$ 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 effected by assumptions about where and to what extent the stave wall cracks under 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 $P_{nw,stave}$ in Eq. (5-6) is the strength obtained from tests illustrated by Fig. 5-1 or Fig. 5-2. The $P_{nw,joint}$ in Eq. (5-7) is the strength from tests illustrated by Fig. 5-B and is typically lower. The $P_{nw,buckling}$ 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 $P_{nw,stave}$ 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.

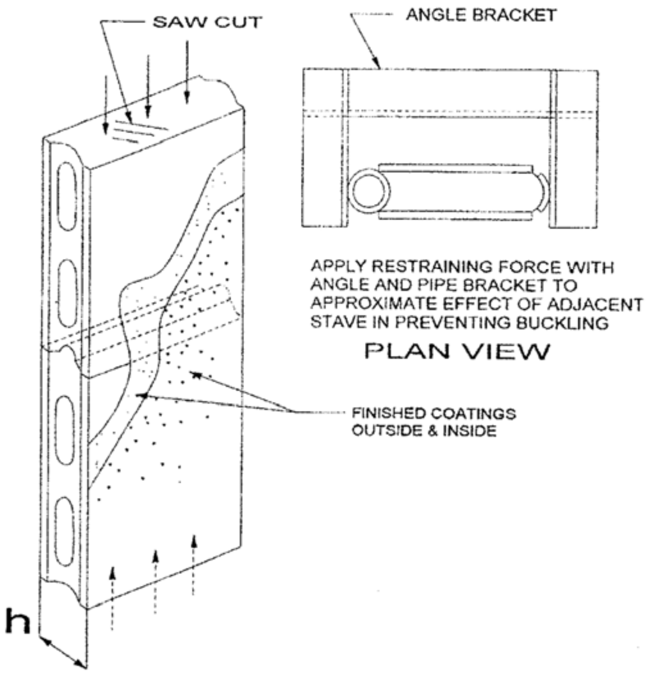


Fig. 5-B—Stave assembly compression test.

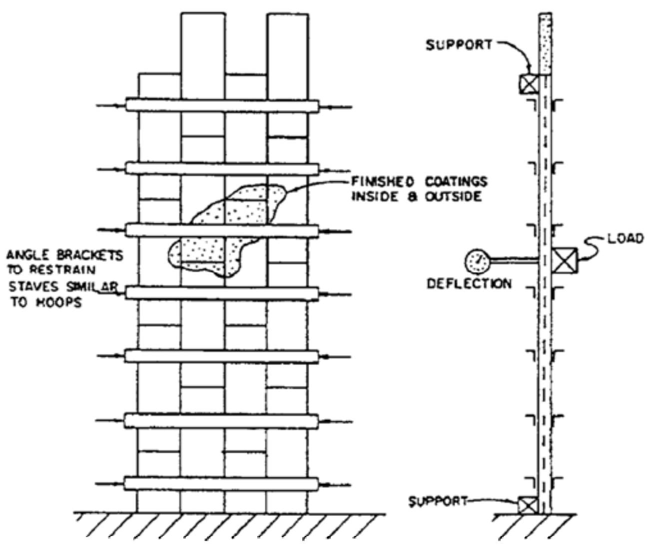


Fig. 5-C—Stave assembly test for vertical stiffness.

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

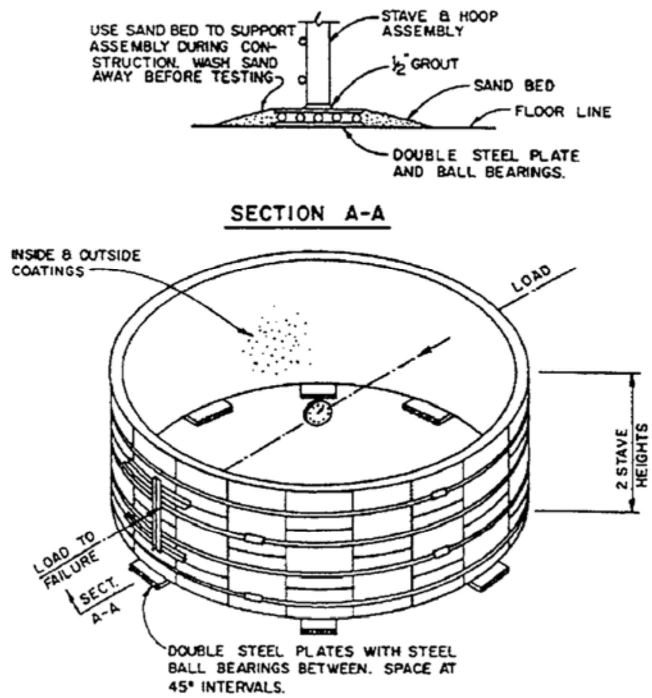


Fig. 5-D—Stave assembly test for horizontal stiffness.

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

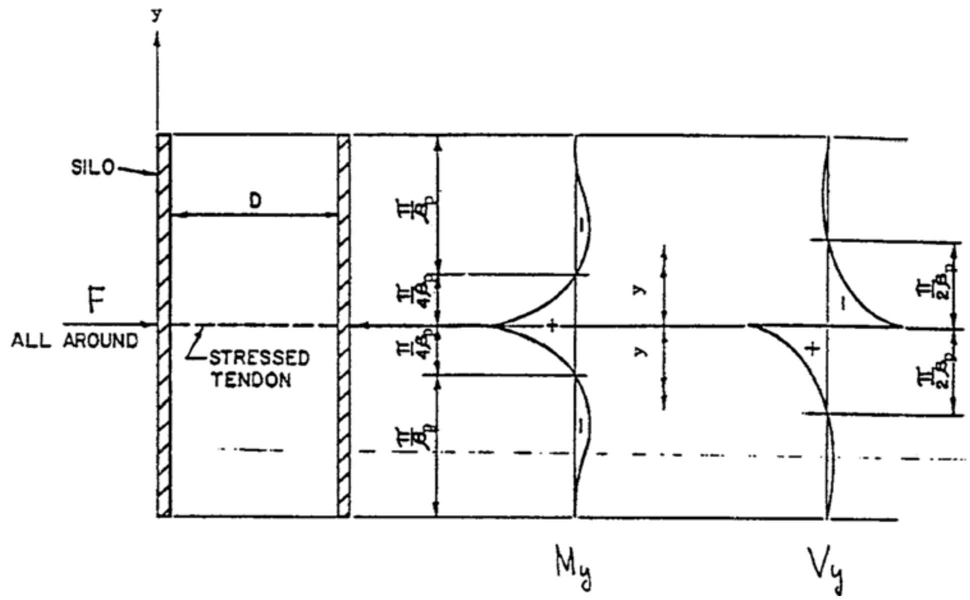


Fig. 6-A—Bending moment and shear diagrams due to uniform loading along a circular section.

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_y = 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

ψ_f = factor obtained from Eq. (6D) or Table 6-A

θ_f = factor obtained from Eq. (6E) or Table 6-A

β_p = factor relating to Poisson's ratio, silo diameter and wall thickness

R6.2—Scope

R6.2.2 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-prestressed 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 slipformed 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 1-7/8 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.

Table 6-A—Values of factors ψ_f and θ_f for use in Eq. (6A) and (6B)

$\beta_p y$	ψ_f	θ_f	$\beta_p y$	ψ_f	θ_f	$\beta_p y$	ψ_f	θ_f
0	1.0000	1.0000	2.4	–0.1282	–0.0669	4.8	0.0089	0.0007
0.1	0.8100	0.9003	2.5	–0.1149	–0.0658	4.9	0.0087	0.0014
0.2	0.6398	0.8024	2.6	–0.1019	–0.0636	5.0	0.0084	0.0019
0.3	0.4888	0.7077	2.7	–0.0895	–0.0608	5.1	0.0080	0.0023
0.4	0.3564	0.6174	2.8	–0.0777	–0.0573	5.2	0.0075	0.0026
0.5	0.2415	0.5323	2.9	–0.0666	–0.0534	5.3	0.0069	0.0028
0.6	0.1431	0.4530	3.0	–0.0563	–0.0493	5.4	0.0064	0.0029
0.7	0.0599	0.3798	3.1	–0.0469	–0.0450	5.5	0.0058	0.0029
0.8	–0.0093	0.3131	3.2	–0.0383	–0.0407	5.6	0.0052	0.0029
0.9	–0.0657	0.2527	3.3	–0.0306	–0.0364	5.7	0.0046	0.0028
1.0	–0.1108	0.1988	3.4	–0.0237	–0.0323	5.8	0.0041	0.0027
1.1	–0.1457	0.1510	3.5	–0.0177	–0.0283	5.9	0.0036	0.0026
1.2	–0.1716	0.1091	3.6	–0.0124	–0.0245	6.0	0.0031	0.0024
1.3	–0.1897	0.0729	3.7	–0.0079	–0.0210	6.1	0.0026	0.0022
1.4	–0.2011	0.0419	3.8	–0.0040	–0.0177	6.2	0.0022	0.0020
1.5	–0.2068	0.0158	3.9	–0.0008	–0.0147	6.3	0.0018	0.0018
1.6	–0.2077	–0.0059	4.0	0.0019	–0.0120	6.4	0.0015	0.0017
1.7	–0.2047	–0.0235	4.1	0.0040	–0.0095	6.5	0.0012	0.0015
1.8	–0.1985	–0.0376	4.2	0.0057	–0.0074	6.6	0.0009	0.0013
1.9	–0.1899	–0.0484	4.3	0.0070	–0.0054	6.7	0.0006	0.0011
2.0	–0.1794	–0.0563	4.4	0.0079	–0.0038	6.8	0.0004	0.0010
2.1	–0.1675	–0.0618	4.5	0.0085	–0.0023	6.9	0.0002	0.0008
2.2	–0.1548	–0.0652	4.6	0.0089	–0.0011	7.0	0.0001	0.0007

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.[29]

Even so, the preferred solution might be to provide partial post-tensioning[55] 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.[61] 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[62] 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 $2\pi/\beta_p$, the vertical bending moment $\boldsymbol{M_y}$ and the shearing force $\boldsymbol{V_{hy}}$ on a horizontal section at distance $\boldsymbol{y}$ above or below the tendon may be determined by Eq. (6A) and (6B), respectively, per unit width of wall.

$$M_y = (F\psi_f)/4\beta_p \tag{6A}$$

and

$$V_{hy} = F\theta_f/2 \tag{6B}$$

in which $\boldsymbol{F}$ is the radial force and ψ_f and θ_f are factors obtained from Eq. (6D) and Eq. (6E) or Table 6-A as a function of $\beta_p y$

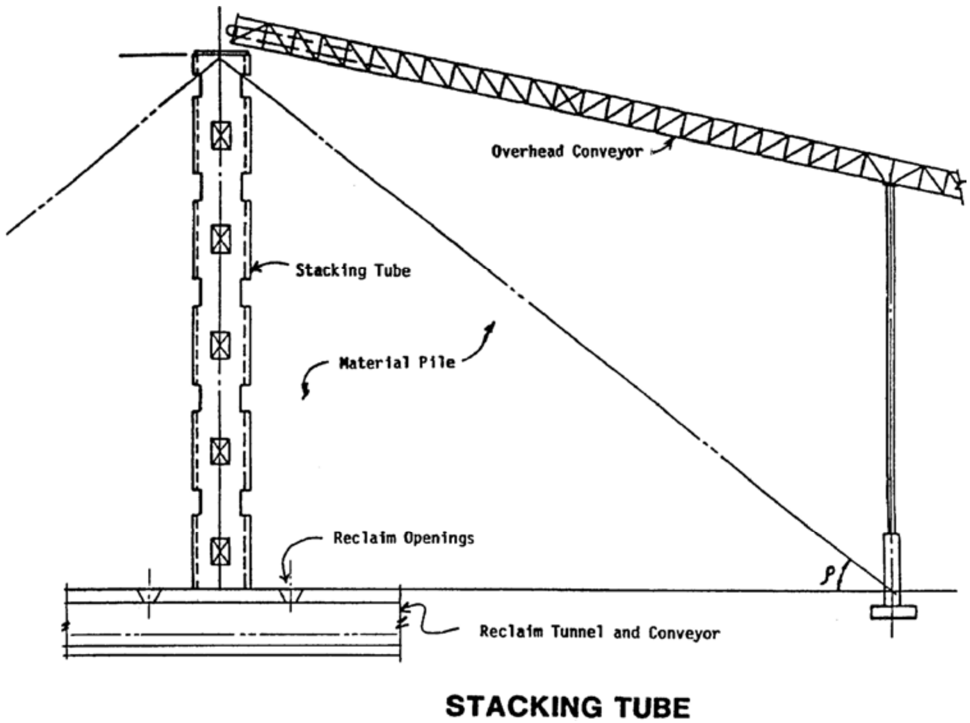


Fig. 7-A.

$$\beta_p = [12(1-0.2)/(D^2h^2)]^{0.25} \tag{6C}$$

$$\psi_f = e^{-\beta y}(\cos\beta_p y - \sin\beta_p y) \tag{6D}$$

$$\theta_f = e^{-\beta y}(\cos\beta_p y) \tag{6E}$$

b) When the spacing between tendons exceeds $2\pi/\beta_p$, 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

$$M_{max} = F/(4\beta_p) \tag{6F}$$

$$V_{max} = F/2 \tag{6G}$$

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

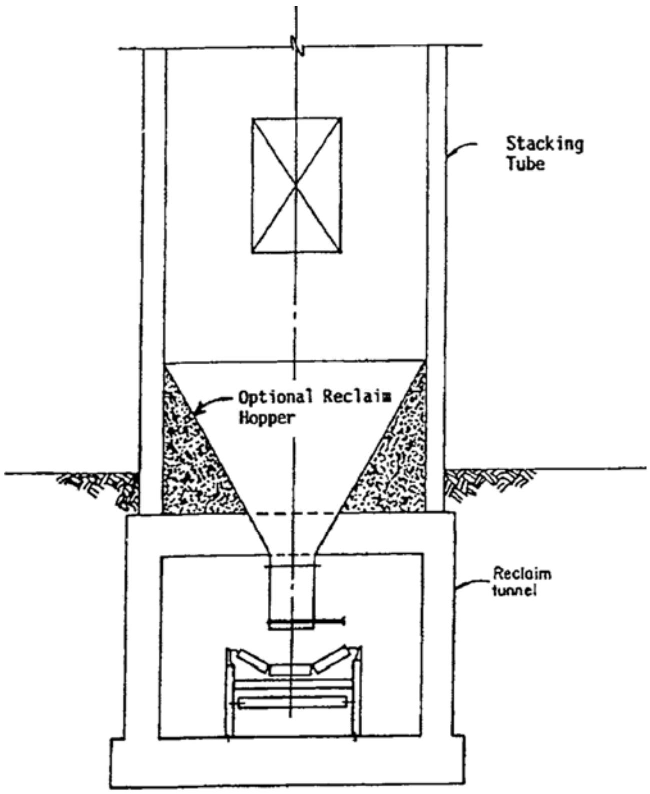


Fig. 7-B—Reclaim tunnel under stacking tube.

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.

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[69-72] 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[71] 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.

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