Chapter 11

MASONRY STRUCTURE DESIGN REQUIREMENTS

11.1 GENERAL

11.1.1 Scope. The design and construction of reinforced and plain masonry components and systems and the materials used therein shall comply with the requirements of this chapter. Masonry shall be designed in accordance with the requirements of ACI 530/ASCE 5/TMS 402. Masonry construction and materials shall be in accordance with the requirements of ACI 530.1/ASCE 6/TMS 602. Inspection and testing of masonry materials and construction shall be in accordance with the requirements of Chapter 2.

11.1.2 References. The following documents shall be used as specified in this chapter.

ACI 530/ASCE 5/TMS 402	*Building Code Requirements for Masonry Structures* (ACI 530-02/ASCE 5-02/TMS 402-02), American Concrete Institute/American Society of Civil Engineers/The Masonry Society, 2002.
ACI 530.1/ASCE 6/TMS 602	*Specification for Masonry Structures* (ACI 530.1-02/ASCE 6-02/TMS 602-02), American Concrete Institute/American Society of Civil Engineers/The Masonry Society, 2002.
ACI 318	*Building Code Requirements for Structural Concrete*, American Concrete Institute, 2002, excluding Appendix A.

11.2 GENERAL DESIGN REQUIREMENTS

11.2.1 Classification of shear walls. Masonry walls, unless isolated from the lateral force resisting system, shall be considered shear walls and shall be classified in accordance with this section.

11.2.1.1 Ordinary plain (unreinforced) masonry shear walls. Ordinary plain (unreinforced) masonry shear walls shall satisfy the requirements of Section 1.13.2.2.1 of ACI 530/ASCE 5/TMS 402.

11.2.1.2 Detailed plain (unreinforced) masonry shear walls. Detailed plain (unreinforced) masonry shear walls shall satisfy the requirements of Section 1.13.2.2.2 of ACI 530/ASCE 5/TMS 402.

11.2.1.3 Ordinary reinforced masonry shear walls. Ordinary reinforced masonry shear walls shall satisfy the requirements of Section 1.13.2.2.3 of ACI 530/ASCE 5/TMS 402.

11.2.1.4 Intermediate reinforced masonry shear walls. Intermediate reinforced masonry shear walls shall satisfy the requirements of Section 1.13.2.2.4 of ACI 530/ASCE 5/TMS 402.

11.2.1.5 Special reinforced masonry shear walls. Special reinforced masonry shear walls shall satisfy the requirements of Section 1.13.2.2.5 of ACI 530/ASCE 5/TMS 402.

11.2.1.6 Shear keys. Add the following new Sec. 1.13.2.2.5 (d) to the Sec. 1.13.2.2.5 of ACI 530/ASCE 5/TMS 402. The surface of concrete upon which a special reinforced masonry shear wall is constructed shall have a minimum surface roughness of 1/8 in. (3 mm). Shear keys are required when the calculated tensile strain in vertical reinforcement from in-plane loads exceeds the yield strain under load combinations that include seismic forces based on an *R* factor equal to 1.5. Shear keys that satisfy the following requirements shall be placed at the interface between the wall and the foundation:

1. The width of the keys shall be at least equal to the width of the grout space,
2. The depth of the keys shall be at least 1.5 in. (38 mm),
3. The length of the key shall be at least 6 in. (152 mm),
4. The spacing between keys shall be at least equal to the length of the key,

5. The cumulative length of all keys at each end of the shear wall shall be at least 10 percent of the length of the shear wall (20 percent total),
6. At least 6 in. (150 mm) of a shear key shall be placed within 16 in. (406 mm) of each end of the wall, and
7. Each key and the grout space above each key in the first course of masonry shall be grouted solid.

11.2.2 Modifications to ACI 530/ASCE 5/TMS 402 and ACI 530.1/ASCE 6/TMS 602.

11.2.2.1 Additional definitions. Add the following definitions to Sec. 1.6 of ACI 530/ASCE 5/TMS 402:

"**Actual dimension** – The measured dimension of a designated item (e.g., a designated masonry unit or wall).

Cleanout – An opening to the bottom of a grout space of sufficient size and spacing to allow removal of debris.

Cover – Distance between surface of reinforcing bar and face of member.

Effective period – Fundamental period of the structure based on cracked stiffness.

Hollow masonry unit – A masonry unit whose net cross-sectional area in any plane parallel to the bearing surface is less than 75 percent of the gross cross-sectional area in the same plane.

Plastic hinge – The zone in a structural member in which the yield moment is anticipated to be exceeded under loading combinations that include earthquake. The zone in a masonry element in which earthquake energy is dissipated through the development of inelastic strains and curvatures.

Reinforced masonry – Masonry construction in which reinforcement acts in conjunction with the masonry to resist forces. Masonry in which the tensile resistance of masonry is neglected and the resistance of the reinforcing steel is considered in resisting applied loads.

Solid masonry unit – A masonry unit whose net cross-sectional area in any plane parallel to the bearing surface is 75 percent or more of the gross cross-sectional area in the same plane.

Special moment frame – A moment resisting frame of masonry beams and masonry columns within a plane with special reinforcement details and connections that provides resistance to lateral and gravity loads.

Specified – Required by construction documents.

Stirrup – Shear reinforcement in a beam or flexural member."

11.2.2.2 Additional notation. Add the following notation to Sec. 1.5 of ACI 530/ASCE 5/TMS 402:

"d_{bb} = diameter of the largest beam longitudinal reinforcing bar passing through, or anchored in, the special moment frame beam-*column* intersection.

d_{bp} = diameter of the largest *column* (pier) longitudinal reinforcing bar passing through, or anchored in, the special moment frame beam-column intersection.

h_x = height of structure above the base level to level *x*.

h_b = beam depth in the plane of the special moment frame.

h_c = cross-sectional dimension of grouted core of special moment frame member measured center to center of confining reinforcement.

L_c = length of coupling beam between coupled *shear wall*s.

M_1,M_2 = nominal moment strength at the ends of the coupling beam.

V_g = unfactored shear force due to gravity loads."

11.2.2.3. Delete Article 1.3 AE from ACI 530.1/ASCE 6/TMS 602.

11.2.2.4. Add the following exception after the second paragraph of Sec. 3.2.5.5 of ACI 530/ASCE 5/TMS 402.

"**Exception:** A nominal thickness of 4 in. (102 mm) shall be permitted where load-bearing reinforced hollow clay unit masonry walls satisfy all of the following conditions.

1. The maximum unsupported height-to-thickness or length-to-thickness ratios do not exceed 27,
2. The net area unit strength exceeds 8,000 psi (55 MPa),
3. Units are laid in running bond,
4. Bar sizes do not exceed No. 4 (13 mm),
5. There are no more than two bars or one splice in a cell, and
6. Joints are not raked."

11.2.2.5. Add the following new Sec. 1.15.3 to ACI 530/ASCE 5/TMS 402:

"**1.15.3 Separation joints.** Where concrete abuts structural masonry and the joint between the materials is not designed as a separation joint, the concrete shall be roughened so that the average height of aggregate exposure is 1/8 in. (3 mm) and shall be bonded to the masonry in accordance with these requirements as if it were masonry. Vertical joints not intended to act as separation joints shall be crossed by horizontal reinforcement as required by Sec. 1.9.4.2."

11.2.2.6. Add the following new Article 3.5 G to ACI 530.1/ASCE 6/TMS 602:

"**3.5 G.** Construction procedures or admixtures shall be used to facilitate placement and control shrinkage of grout."

11.2.2.7. Replace Sec. 3.2.3.4(b) and 3.2.3.4(c) of ACI 530.1/ASCE 6/TMS 602 with the following:

"(b) A welded splice shall be capable of developing in tension 125 percent of the specified yield strength, f_y, of the bar. Welded splices shall only be permitted for ASTM A706 steel reinforcement. Welded splices shall not be permitted in plastic hinge zones of intermediate or special reinforced walls or special moment frames of masonry.

(c) Mechanical splices shall be classified as Type 1 or Type 2 according to Sec. 21.2.6.1 of ACI 318. Type 1 mechanical splices shall not be used within a plastic hinge zone or within a beam-column joint of intermediate or special reinforced masonry shear walls or special moment frames. Type 2 mechanical splices shall be permitted in any location within a member."

11.2.2.8. Add the following new Sec. 3.2.3.4.1 and 3.2.3.4.2 to ACI 530/ASCE 5/TMS 402:

"**3.2.3.4.1** Lap splices shall not be used in plastic hinge zones. The length of the plastic hinge zone shall be taken as at least 0.15 times the distance between the point of zero moment and the point of maximum moment.

3.2.3.4.2 Bars spliced by non-contact lap splices shall not be spaced transversely farther apart than the lesser of one-fifth the required length or 8 in. (203 mm)."

11.2.2.9. Add the following new Sec. 3.2.2(h) to ACI 530/ASCE 5/TMS 402:

"(h) For out-of-plane bending, the width of the equivalent stress block shall not be taken greater than 6 times the nominal thickness of the masonry wall or the spacing between reinforcement, whichever is less."

11.2.2.10. Add the following new Sec. 3.2.7 to ACI 530/ASCE 5/TMS 402:

"3.2.7 Flanged shear walls

3.2.7.1 Effective width. Where wall intersections are constructed in accordance with Sec. 1.9.4, the effective flange width for design shall be determined in accordance with this section.

3.2.7.2 Compression. The width of flange considered effective in compression on each side of the web shall be taken equal to 6 times the thickness of the flange or the actual width of the flange on that side, whichever is less.

3.2.7.3 Tension. The width of flange considered effective in tension on each side of the web shall be taken equal to 3/4 of the wall height or the actual width of the flange on that side, whichever is less."

11.2.2.11. Add the following new Sec. 3.2.4.2.6 to ACI 530/ASCE 5/TMS 402:

"**3.2.4.2.6 Coupling beams.** Structural members that provide coupling between shear walls shall be designed to reach their moment or shear nominal strength before either shear wall reaches its moment or shear nominal strength. Analysis of coupled shear walls shall comply with accepted principles of mechanics.

The design shear strength, ϕV_n, of the coupling beams shall satisfy the following criterion:

$$\phi V_n \geq \frac{1.25\left(M_1 + M_2\right)}{L_c} + 1.4V_g$$

where:

M_1 and M_2	=	nominal moment strength at the ends of the beam;
L_c	=	length of the beam between the shear walls; and
V_g	=	unfactored shear force due to gravity loads.

The calculation of the nominal flexural moment shall include the reinforcement in reinforced concrete roof and floor systems. The width of the reinforced concrete used for calculations of reinforcement shall be six times the floor or roof slab thickness."

11.2.2.12. Add the following new Sec. 3.2.5 to ACI 530/ASCE 5/TMS 402:

"**3.2.5 Deep flexural member detailing.** Flexural members with overall-depth-to-clear-span ratio greater than 2/5 for continuous spans or 4/5 for simple spans shall be detailed in accordance with this section.

3.2.5.1. Minimum flexural tension reinforcement shall conform to Sec. 3.2.4.3.2.

3.2.5.2. Uniformly distributed horizontal and vertical reinforcement shall be provided throughout the length and depth of deep flexural members such that the reinforcement ratios in both directions are at least 0.001. Distributed flexural reinforcement is to be included in the determination of the actual reinforcement ratios."

11.2.2.13. Add the following new Sec. 1.13.7.4 to ACI 530/ASCE 5/TMS 402:

"**1.13.7.4** For structures in Seismic Design Category E or F, corrugated sheet metal anchors shall not be used."

11.2.2.14. Revise Sec. 1.13.3.2 of ACI 530/ASCE 5/TMS 402/ to read as follows.

" The calculated story drift of masonry structures due to the combination of design seismic forces and gravity loads shall not exceed the allowable story drift Δ_a for masonry walls shown in Table 4.5-1 of the 2003 *NEHRP Recommended Provisions.*

11.2.2.15. Add the following section to ACI 530/ASCE 5/TMS 402:

"1.13.4.3 Anchoring to masonry. Anchorage assemblies connecting masonry elements that are part of the seismic force resisting system to diaphragms and chords shall be designed so that the strength of the anchor is governed by steel tensile or shear yielding. Alternatively, the anchorage assembly may be designed to be governed by masonry breakout or anchor pullout provided that the anchorage assembly is designed to resist not less than 2.5 times the factored forces transmitted by the assembly. "

11.2.2.16. Revise the following Sec. 3.1.4.4 of ACI 530/ASCE 5/TMS 402:

"3.1.4.4 *Anchor bolts* For cases where the nominal strength of an anchor bolt is controlled by masonry breakout or masonry pryout, φ shall be taken as 0.50. For cases where the nominal strength of an anchor bolt is controlled by anchor bolt steel, φ shall be taken as 0.90. For cases where the nominal strength of an anchor bolt is controlled by anchor pullout, φ shall be taken as 0.65."

11.2.2.17. Revise the following Sec. 3.1.6.3 of ACI 530/ASCE 5/TMS 402:

"3.1.6.3 *Nominal shear strength of headed and bent-bar anchor bolts* — The nominal shear strength, B_{vn}, shall be computed by Eq. (3-8) (strength governed by masonry breakout) and Eq. (3-9) (strength governed by steel), and shall not exceed 2.0 times that computed by Eq. (3-4) (strength governed by masonry pryout). In computing the capacity, the smallest of the design strengths shall be used."

11.2.2.18. Revise the following commentary to Sec. 3.1.6.3 of ACI 530/ASCE 5/TMS 402:

"3.1.6.3 *Nominal shear strength of headed and bent-bar anchor bolts* — The shear strength of a headed or bent-bar anchor bolt is governed by yield and fracture of the anchor steel, ~~or~~ by masonry shear breakout, or by masonry shear pryout. Steel strength is calculated conventionally using the effective tensile stress area (that is, threads are conservatively assumed to lie in the critical shear plane). Under static shear loading, bent-bar anchor bolts (J- or L-bolts) do not exhibit straightening and pullout. Under reversed cyclic shear however, available research[3.1] suggests that straightening and pullout may occur."

11.3 SPECIAL MOMENT FRAMES OF MASONRY

Special moment frames of masonry shall be designed and detailed in accordance with the requirements of Sec. 3.2 of ACI 530/ASCE 5/TMS 402 and this section.

Special moment frames shall be fully grouted and constructed using open-end hollow -unit concrete masonry or hollow-unit clay masonry.

Column nominal moment strength shall not be less than 1.6 times the column moment corresponding to the development of beam plastic hinges, except at the foundation level. The column axial load corresponding to the development of beam plastic hinges and including factored dead and live loads shall not exceed 0.15 A_n f'_m The plastic hinge zone shall be assumed equal to the depth of the member.

11.3.1 Calculation of required strength. The calculation of required strength of the members shall be in accordance with principles of engineering mechanics and shall consider the effects of the relative stiffness degradation of the beams and columns.

11.3.2 Flexural yielding. Flexural yielding shall be limited to the beams at the face of the columns and to the bottom of the columns at the base of the structure.

11.3.3 Materials. Neither Type N mortar nor masonry cement shall be used.

11.3.4 Reinforcement

11.3.4.1. The nominal moment strength at any section along a member shall not be less than 1/2 of the higher moment strength provided at the two ends of the member.

11.3.4.2. Lap splices are permitted only within the center half of the member length. Lap splices are not permitted in transverse reinforcement in beams, in plastic hinge zones in the column or in the beam-column joint.

11.3.4.3. Welded splices and mechanical connections may be used for splicing the reinforcement at any section, provided that not more than alternate longitudinal bars are spliced at a section and the distance between splices on alternate bars is at least 24 in. (610 mm) along the longitudinal axis and shall comply with the requirements of Section 11.3.7.4.

11.3.4.4. Reinforcement shall have a specified yield strength of 60,000 psi (414 MPa). The actual yield strength shall not exceed 1.3 times the specified yield strength.

11.3.5 Beams

11.3.5.1 Compression limit. The factored axial compression force on the beam shall not exceed 0.10 times the net cross-sectional area of the beam, A_n, times the specified compressive strength, f'_m.

11.3.5.2 Shear. The value of V_m shall be zero within any plastic hinge zone and in any columns subjected to net factored tension loads. The depth of the plastic hinge zone shall be assumed equal to the member depth..

11.3.5.3 Reinforcement ratio. The reinforcement ratio for beams that connect vertical elements of the seismic-force-resisting system shall not exceed the lesser of $0.15\, f'_m / f_y$ or that determined in accordance with Sec. 3.2.3.5.1 of ACI 530/ASCE 5/TMS 402. All reinforcement in the beam and adjacent to the beam in a reinforced concrete roof or floor system shall be used to calculate the reinforcement ratio.

11.3.5.4 Proportions. The clear span for the beam shall not be less than 4 times its depth.

The nominal depth of the beam shall not be less than 4 units or 32 in. (813 mm), whichever is greater. The nominal depth to nominal width ratio shall not exceed 4.

Nominal width of the beams shall equal or exceed all of the following criteria:

1. 8 in. (203 mm),
2. width required by Sec. 3.2.4.2.5 of ACI 530/ASCE 5/TMS 402 , and
3. 1/26 of the clear span between column faces.

11.3.5.5 Longitudinal reinforcement.

11.3.5.5.1. Longitudinal reinforcement shall not be spaced more than 8 in. (203 mm) on center.

11.3.5.5.2. Longitudinal reinforcement shall be uniformly distributed along the depth of the beam.

11.3.5 5.3. The minimum reinforcement ratio shall be $130/f_y$, where f_y is in psi (the metric equivalent is $0.90/f_y$, where f_y is in MPa).

11.3.5. 5.4. At any section of a beam, each masonry unit through the beam depth shall contain longitudinal reinforcement.

11.3.5.6 Transverse reinforcement

11.3.5.6.1. Transverse reinforcement shall be one-piece and shall be hooked around top and bottom longitudinal bars and shall be terminated with a standard 180-degree hook.

11.3.5.6.2. Within an end region extending one beam depth from Special Moment Frame column faces and in any region at which beam plastic hinges may form during seismic or wind loading, the maximum spacing of transverse reinforcement shall not exceed one-fourth the nominal depth of the beam.

11.3.5.6.3. The maximum spacing of transverse reinforcement shall not exceed the lesser of 1/2 the nominal depth of the beam or the spacing required for shear strength.

11.3.5.6.4. The minimum transverse reinforcement ratio shall be 0.0015.

11.3.5.6.5. The first transverse bar shall not be more than 4 in. (102 mm) from the face of the column.

11.3.6 Columns

11.3.6.1 Compression limit. Factored axial compression force on the Special Moment Frame column corresponding to the development of beam plastic hinges shall not exceed 0.15 times the net cross-sectional area of the column, A_n, times the specified compressive strength. The compressive stress shall also be limited by the maximum reinforcement ratio.

11.3.6.2 Proportions. The nominal dimension of the column parallel to the plane of the Special Moment Frame shall not be less than two full units or 32 in. (810 mm), whichever is greater and shall not exceed 96 in.

The nominal dimension of the column perpendicular to the plane of the Special Moment Frame shall not be less than 8 in. (203 mm) or 1/14 of the clear height between beam faces, whichever is greater.

The clear-height-to-depth ratio of column members shall not exceed 5.

11.3.6.3 Longitudinal reinforcement

11.3.6.3.1. A minimum of 4 longitudinal bars shall be provided at all sections of every Special Moment Frame column member.

11.3.6.3.2. The flexural reinforcement shall be uniformly distributed across the member depth.

11.3.6.3.3. The nominal moment strength at any section along a member shall be not less than 1.6 times the cracking moment strength and the minimum reinforcement ratio shall be $130/f_y$, where f_y is in psi (the metric equivalent is $0.90/f_y$, where f_y is in MPa).

11.3.6.3.4. Vertical reinforcement in wall-frame columns shall be limited to a maximum reinforcement ratio equal to the lesser of $0.15\, f'_m/f_y$ or that determined in accordance with Sec. 3.2.3.5 of Section 1.13.2.2.5 of ACI 530/ASCE 5/TMS 402. The minimum vertical reinforcement in wall-frame columns shall be 0.002 times the gross cross section.

11.3.6.4 Lateral reinforcement.

11.3.6.4.1. Transverse reinforcement shall be hooked around the extreme longitudinal bars and shall be terminated with a standard 180-degree hook.

11.3.6.4.2. The spacing of transverse reinforcement shall not exceed 1/4 the nominal dimension of the column parallel to the plane of the Special Moment Frame.

11.3.6.4.3. The minimum transverse reinforcement ratio shall be 0.0015.

11.3.6.4.4. Lateral reinforcement shall be provided to confine the grouted core when compressive strains caused by the factored axial and flexural loads at the design story drift,), exceed 0.0015. The unconfined portion of the cross section with a strain exceeding 0.0015 shall be neglected when computing the nominal strength of the section. The total cross sectional area of rectangular tie reinforcement for the confined core shall be not less than $0.9sh_c \dfrac{f'_m}{f_{yh}}$. Alternatively, equivalent confinement which can develop an ultimate compressive strain of 0.006 may substituted for rectangular tie reinforcement.

11.3.7 Beam-column intersections

11.3.7.1 Proportions. Beam-column intersection dimensions in masonry special moment frames shall be proportioned such that the special moment frame column depth in the plane of the frame satisfies Eq. 11.3-1:

$$h_c > \frac{4800 d_{bb}}{\sqrt{f'_g}} \qquad (11.3\text{-}1)$$

where:

h_p = column depth in the plane of the special moment frame, in.;

d_{bb} = diameter of the largest beam longitudinal reinforcing bar passing through, or anchored in, the special moment frame beam-column intersection, in.; and

f'_g = specified compressive strength of grout, psi (shall not exceed 5,000 psi (34.5MPa) for use in Eq. 11.3-1).

The metric equivalent of Eq. 11.3-1 is

$$h_c > \frac{400 d_{bb}}{\sqrt{f'_g}}$$

where h_p and d_{bb} are in mm and f'_g is in MPa.

Beam depth in the plane of the frame shall satisfy Eq. 11.3-2:

$$h_b > \frac{1800 d_{bp}}{\sqrt{f'_g}} \qquad (11.3\text{-}2)$$

where:

h_b = beam depth in the plane of the special moment frame, in.;

d_{bp} = diameter of the largest column longitudinal reinforcing bar passing through, or anchored in, the special moment frame beam-column intersection, in.; and

f'_g = specified compressive strength of grout, psi (shall not exceed 5,000 psi (34.5 MPa) for use in Eq. 11.3-2).

The metric equivalent of Eq. 11.3-2 is

$$h_b > \frac{150 d_{bp}}{\sqrt{f'_g}}$$

where h_b and d_{bp} are in mm and f'_g is in MPa.

11.3.7.2 Shear strength. The design shear strength, NV_n, of the beams and columns shall not be less than the shear corresponding to the development of 1.4 times the nominal flexural strength of the member, except that the nominal shear strength need not exceed 2.5 times V_u. The nominal shear strength of beam-column intersections shall exceed the shear calculated assuming that the stress in all flexural tension reinforcement of the special moment frame beams at the column face is 1.4 f_y .

Vertical shear forces may be considered to be carried by a combination of masonry shear-resisting mechanisms and truss mechanisms involving intermediate column reinforcing bars.

The nominal horizontal shear stress at the beam-column intersection shall not exceed the lesser of 350 psi (2.5 MPa) or $7\sqrt{f'_m}$ (the metric equivalent is $0.58\sqrt{f'_m}$ MPa)

11.3.7.3 Horizontal reinforcement. Beam longitudinal reinforcement terminating in a special moment frame column shall be extended to the far face of the column and shall be anchored by a standard hook bent back into the special moment frame column.

Special horizontal shear reinforcement crossing a potential diagonal beam-column shear crack shall be provided such that:

$$A_s \geq \frac{0.5V_{jh}}{f_y} \tag{11.3-3}$$

where:

A_s = cross-sectional area of reinforcement;

V_{jh} = total horizontal joint shear; and

f_y = specified yield strength of the reinforcement .

Special horizontal shear reinforcement shall be anchored by a standard hook around the extreme special moment frame column reinforcing bars.

11.4 GLASS-UNIT MASONRY AND MASONRY VENEER

11.4.1 Design lateral forces and displacements. Glass-unit masonry and masonry veneer shall be designed and detailed to satisfy the force and displacement requirements of Sec. 6.3.

11.4.2 Glass-unit masonry design.

11.4.2.1. The requirements of Chapter 7 of ACI 530/ASCE 5/TMS 402. shall apply to the design of glass unit masonry. The out-of-plane seismic strength shall be considered to be the same as the strength to resist wind pressure as specified in Sec. 7.3 of ACI 530/ASCE 5/TMS 402.

11.4.3 Masonry veneer design.

11.4.3.1. The requirements of Chapter 6 of ACI 530 shall apply to the design of masonry veneer.

11.4.3.2. For structures in Seismic Design Category E, corrugated sheet metal anchors shall not be used.

11.5 PRESTRESSED MASONRY

11.5.1. Prestressed masonry shall be designed in accordance with of ACI 530/ASCE 5/TMS 402 Chapter 4.

This page is intentionally left blank