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THE NEW CSA S304-14 DESIGN OF MASONRY STRUCTURES: PART 1 DISCUSSION OF NON-SEISMIC CHANGES TO THE STANDARD

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ABSTRACT

The 2014 edition of the Canadian Standards Association (CSA) S304 "Design of masonry structures" contains significant changes from the preceding 2004 edition. All non-seismic related changes to the standard are presented within this paper whereas the seismic changes are presented in a companion paper. New additions to CSA S304-14 include embedded anchor bolt design and arch design provisions. Changes have been made to the minimum reinforcement requirements as well as to the maximum bar size now allowed in reinforced masonry. Significant changes have been made to design of masonry beams including an adaptation of modified compression field theory for shear strength calculations, allowances for shear stirrups in 2-course beams, clarification on masonry deep beam design and a greater emphasis on intermediate reinforcement placement closer to the tension face of beams. Lap splice provisions were largely kept the same as in 2004 with clarification given on contact versus non-contact splices as well as the inclusion of offset bar lap splices. New prism testing requirements and correction factors were introduced and minor changes were made to the f'_m strength tables.

KEYWORDS: *anchor bolts, arches, design standards, masonry beams, masonry prisms, reinforcement*

INTRODUCTION

CSA S304 was written using a consensus process that brings together volunteers representing various viewpoints and interests organized into a membership matrix of four groups (Producers, Users, General Interest, and Regulatory Authorities). Membership is limited so that the combined minimum membership of the 2 smallest groups must be larger than the maximum size of the largest group; this ensures that decisions cannot be controlled by one interest group and that consensus is

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generally required. Completion of the latest edition took 4 years with 2 face to face meetings each year; working groups continued in the intervening periods. CSA S304 [1] is currently on a 10-year development cycle and had to be completed in 2014 to be referenced in the 2015 National Building Code of Canada (NBCC) [2] which will subsequently be adopted by provinces directly or used as a model to create a provincial building code. This is scheduled to occur through the last half of 2017.

Recent research findings, feedback from designers, contractors, and regulatory authorities, and the progress in other international masonry design standards were considered in revising existing clauses and in development of new provisions. In several cases where it was deemed that insufficient information was available and could not be provided within the publication timeframe, potential changes were set aside for consideration in the next edition in 2024. Changes are discussed in the same sequence as presented in the standard not in order of importance. Changes made to the CSA S304-14 regarding seismic design were so significant that a companion paper is dedicated to just those changes [3].

1 SCOPE, 2 REFERENCE PUBLICATIONS, DEFINITIONS, STANDARD NOTATION, UNITS, AND 3 GENERAL REQUIREMENTS.

Clauses 1 to 3, covering the above topic areas, were subject mainly to editorial review but with updated references, definitions, and terminology.

Companion Standards. Designers must not only be familiar with CSA S304 but also CSA A371 "Masonry construction for buildings" [4] as construction choices may affect the design. Contractors need only follow CSA A371. Both groups should have a working knowledge of requirements for concrete and clay masonry units (CSA A165 [5] and A82 [6] respectively), mortar and grout (CSA A179) [7], and connectors (CSA A370) [8] as set out in other companion standards. New editions of all of these standards were co-ordinated to be published together in 2014 to ensure compatibility of their content.

Review of Structural Construction, Inspections, and Test Results. A note to the first sentence, points out that use of CSA S304 assumes that the above will be carried out by the designer or another suitably qualified person to determine general conformance with the design.

Partitions. In Clause 1, it is noted that a masonry partition must be designed using engineering analysis except where it can be shown that it is not "subject to unusual loads such as wind loads, significant internal air pressure differences, or large eccentric loads mounted on the wall." In such cases, the partition may be designed using the empirical design provisions of Annex F.

Solid Masonry and Composite Walls. In definitions, solid masonry is redefined as not necessarily comprising 2 or more wythes but, where multiple wythes occur, the space between wythes must be mortar filled collar joints or grout filled space. Also composite walls now need not be of wythes of dissimilar materials and connections between wythes may provide either partial or full composite action. Although these definitions are at odds with traditional usage in structural engineering, they were changed for consistency with parts of the NBCC.

4 DESIGN REQUIREMENTS

In Clause 4, load factors and load combinations were modified to correspond to values being introduced in the 2015 edition of NBCC. Resistance factors originally introduced in the 1994 edition, underwent an extensive review including new analyses including more recent data. In the end, these were deemed to be reasonable and were left unchanged.

Seismic Design. The section on seismic design was replaced with a reference to Clause 16, see [3] for more information on the changes made.

Veneer Supported on Wood Structures. Masonry veneer may be supported by wood on wood structures now of 6 stories or less. This reflects the NBCC-15 relaxation of height limits for wood structures up to 6 stories in height from 4 but retains the requirement that the support be designed to conform with Part 4 of the NBCC. This means that CSA S304 is used rather than the simplified rules in Part 9 of NBCC.

5 SPECIFIED STRENGTHS USED IN DESIGN

Concrete Block. For masonry compressive strengths based on unit, mortar, and grout tests, in Table 4, values for 40 MPa concrete block were removed. As part of major review of existing prism test data versus the tabulated specified strengths, the historical data for 40 MPa block was regarded as not being sufficiently comprehensive; it was thought that, where such high strengths are required in design, values for the specific manufacturers' block should be used in accordance with prism testing requirements of CSA A165.1[5]. Another small adjustment in the table was to change the f'_m value for 15 MPa hollow block and Type S mortar from 9.8 MPa to 10 MPa. In Canada, 15 MPa block is the usual default design choice which coincides with the usual minimum strength available.

Hollow Clay Brick. There continued to be insufficient data available to establish a strength table for hollow clay brick; when hollow clay brick is used, prism test data is required.

7 DESIGN OF UNREINFORCED WALLS AND COLUMNS

Compression Controlled Design. Changes clarify design requirements of sections to remain uncracked or those that satisfied the eccentricity limitations permitting cracking. Neglecting tensile stresses, the rectangular stress block may be used for the compression zones of these sections. These changes address the incorrect possible interpretation that would require linear elastic analysis to be used.

Tension Controlled Design. Linear elastic analyses are only required where tensile strength is relied upon to prevent cracking. A new note limits the calculated moment resistance for compression controlled design to the factored axial load times the eccentricity limit. Another note limits reliance on tensile strength; it requires that a member be designed as a cracked member under all factored load combinations for all directions of loading when it is found to be cracked under any unfactored combination of loading in any direction. Design for a particular load combination is not independent of the consequences of other loading combinations. When linear elastic analysis is required for sections not allowed to be cracked, a note explains that a compressive stress check as part of the elastic analysis is required to ensure that the compressive stress is sufficiently low (less than 0.6 $\phi_m f'_m$) so that linear elastic analysis is valid.

Axial Load and Biaxial Bending in Walls. A new note confirms that slenderness effects in shear wall design are considered to be satisfied when slenderness effects are accounted for separately in calculating axial load and minor axis bending capacities. Therefore, no separate check is required for shear wall action when also subjected to biaxial bending. Another change makes it clear that linear elastic analysis for minor axis bending capacity is required when a shear wall depends on masonry tensile strength for resistance to combined major and minor axis bending.

Shear in Walls and Columns. A note was changed to make it clearer that both shear capacity based on diagonal tensile stresses and shear capacity based on resistance to sliding have to be checked. Particularly at the base of a wall where bond may be compromised, sliding may control. A sentence was removed that warned against including flanges or other wall projections in determination of the wall width, b_w. This had apparently caused some confusion related to the correct inclusion of all components of the wall section in determining section properties.

Shear at Wall Intersections. For bonded intersections where units in alternating courses of one wall are imbedded at least 90 mm in the other wall, the strength equation has been changed to Equation (1).

$$V_r = \phi_m \left\lfloor 0.16\sqrt{f'_m}\, A_e \right\rfloor \tag{1}$$

Where the benefit of axial compression has been removed as not being effective in resisting development of a vertical crack along the head joint at the intersection. Minimum horizontal reinforcement across the intersection equivalent to two 3.65 mm diameter wires at 400 mm spacing is required. The effective area term may be taken as the mortar bed area for hollow and partially grouted masonry and as the gross area for fully grouted walls. For unbonded wall intersections, the required use of reinforcing bars, joint reinforcing, or other connectors to develop the required shear capacity have been reorganized for improved clarity.

Flexural Wall Panels. First introduced in the previous edition of CSA S304 (2004), it seems that designers have typically not taken potential advantage of this section which allows for two-way bending capacities significantly greater than available for one-way bending. It was suggested by the committee that industry may wish to develop design aids to promote this feature.

Infill Shear Walls. Provisions for infill shear walls were also first introduced in the previous edition of CSA S304 (2004) and, it seems that this major section has also seen only very limited use. In part this may be because CSA S304 and industry literature in general had in previous times concentrated on the need to separate infill walls from structural frames to avoid unintentional load sharing and the effects of differential expansion or shrinkage. Furthermore, the NBCC-15 does not formally recognize infill masonry as a clearly defined seismic force resisting system (SFRS). This section requires a mindset change to take advantage of the potentially very high lateral load resistance available through use of properly designed infill shear walls created by building infill masonry walls to be in contact with the surrounding steel of reinforced concrete frame.

The provisions for calculating shear capacity have been altered for clarity of using the methods applied to masonry shear walls. For diagonal shear capacity, the strengthening effect of compression from vertical dead load cannot be included. Gaps between the frame and the infill are known to develop during lateral displacement of the frame-infill system. For shear slip capacity, 90% of the axial compression can be added to the vertical component of the strut compressive force calculated using the diagonal strut model. While the size of the equivalent diagonal strut is unchanged for strength calculations, for displacement and load distribution calculations, it has been found that strut stiffness has been overestimated. Where stiffness values are required, the dimensions used for strength calculations are reduced by 50%.

Bearing Resistance for Concentrated Load. Compressive strengths for bearing capacity were moved from notes to the main clause to help emphasize that these are mandatory requirements.

Embedded Anchor Bolts. This section is new and provides detailed guidance for calculating the resistance of embedded headed and bent bar anchor bolts under combined shear and axial forces.

Arches. New basic requirements are provided for design of non-vehicular arches. Currently, many designers are not familiar with arch design. Therefore, users may improve their understanding by referring to literature which is available through Canada Masonry Design Centre (CMDC).

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Slenderness Effects for Shear Walls. For biaxial bending, a note similar to the one for unreinforced walls clarify that slenderness calculations for axial compression and minor axis bending are considered to satisfy potential for lateral buckling under shear wall loading.

Shear. An attempt to develop modified compression field criteria for shear design of shear walls could not be completed for this edition; alternatives studied did not provide consistent results. A major factor was that behaviour of partially grouted walls differed from fully grouted walls and more study on the matter is required.

Intersections. The use connectors such as joint reinforcement, anchors, rods or bolts may be utilized to bond wall intersection together in reinforced shear walls where the development of horizontal cracks that could affect the capacity of such connectors does not occur. A note points out that using reinforcing bars in bond beams avoids the possible weakening effects of cracking along bed joints and should be used to connect intersections together in reinforced masonry shear walls where horizontal cracking may occur. Horizontal cracking is typically caused by tension in the vertical reinforcement.

In-plane Shear in Infill Shear Walls. Similar to unreinforced infill shear walls, the role of axial compressive forces is clarified for both diagonal cracking and sliding shear.

Minimum and Maximum Reinforcing for Walls and Columns. The coefficient 0.0013 was changed to 0.00125 for minimum vertical reinforcement in walls subject to axial load and bending as indicated in Equation (2) for a spacing of bars up to 4*t*, and Equation (3) for a spacing over 4*t*. Where, A_s is the minimum area of vertical reinforcement, A_g is the gross cross-sectional area of the wall tributary to the vertical bar in question and t is the thickness of the wall.

$$A_s = 0.00125 A_g \quad (2)$$

$$A_s = 0.00125(4t \times t) \quad (3)$$

This has an effect on the minimum reinforcement required for 15cm units with a spacing over 600 mm (10M now, previously 15M) and for 30cm units with a spacing of 800 mm (20M now, previously 25M). In both instances the difference between the minimum area and the bar area was nominal (101.9 mm^2 and 301.6 mm^2, respectively). Since this minimum limit was based on identifying approximately the least amount of reinforcement for which the equation for $(EI)_{eff}$ in slenderness calculations still gave reasonable results for secondary moment effects, the change has little technical significance.

Maximum Spacing of Horizontal Reinforcement. Where horizontal reinforcement is required to resist *flexural tensile stresses* (i.e. when a wall spans horizontally between vertical elements), the requirement of providing reinforcement at the top of the wall was added for the case where the wall is connected to roof or floor assemblies. This is to ensure that there is effective anchorage at this level. Where horizontal reinforcement is required to resist *in-plane shear forces*, it is now clearly stated that it must satisfy the spacing requirements for flexural reinforcement and, where joint reinforcement is used be spaced at not more than 600 mm for construction in 50% running bond and 400 mm spacing for other patterns. For bond beams to be used as part of the shear reinforcement, the spacing is now limited to the lesser of 2400 mm or 0.5 l_w.

Seismic Design. The provisions of clause 10.16 for seismic design have been removed. Seismic design requirements have undergone extensive changes. They are included in a new clause 16.

11 DESIGN OF REINFORCED BEAMS

Applicability. For use of solid brick units, it is made clear that grouting of cavities (minimum width of 50 mm) or pockets is required and that maximum allowed brick height is 400 mm. For tying the wythes of multi-wythe beams together, the requirements of CSA A371 [4] are referenced.

Intermediate Reinforcement. Changes help ensure that intermediate reinforcement is located within the beam where it will be more effective in controlling cracking and in adding to flexural capacity. In beams more than 600 mm in height, intermediate reinforcement is now distributed over only the two-thirds of the height of the beam nearest the main tension reinforcement. In addition, rather than a simple maximum vertical spacing of 400 mm, the first layer of intermediate reinforcement is now placed not more than 300 mm above the main tension reinforcement. It is also made clearer that the intermediate reinforcement "shall be taken into account in determining maximum reinforcement" limits and that it may be included in calculating the flexural resistance of beams.

Shear Design. New shear design provisions for beams based on a modified compression field theory have been included in this edition. A *Simplified Method* can be used for beams not subject to significant axial tension as well as conforming to several other prescriptive requirements. Otherwise, a more accurate method known as the *General Method* is to be used. While these methods appear to be much more complex and perhaps time consuming than the empirical or semi-empirical methods previously applied, they very closely parallel shear design provisions for reinforced concrete beams [9] and adapted for masonry [10]. As such, it was felt that designers trained in concrete design would not have difficulty making the transition. The design program, MASS™, also facilitates this transition.

12 REINFORCEMENT: DETAILS, DEVELOPMENT, AND SPLICES

Maximum Size of Reinforcement. Discussion of practical considerations for construction and the limited available anchorage and splice length data led to the decision to reduce the maximum deformed reinforcement size from 30M to 25M. In the few past cases where 30M bars had been used, the opinion of the committee was that it did not represent good design practice.

Non-contact Lap Splices. Recent research [11] showed that normal splice lengths may not be appropriate for non-contact splicing of tensile reinforcement placed in adjacent cells. Although this form of splicing is still allowed, a change is added that the required splice length must be established by analysis and test data. Non-contact splices when the bars remain in the same cell are unaffected by this change.

Offset Bars. A new section regarding use of offset bars limits the slope of the inclined portion to 1:6 with respect to the vertical axis of the masonry member and requires that details of offsets be specified in contract documents. In addition, ties, spirals, or parts of floor construction must be designed to resist a horizontal thrust equal to 1.5 times the horizontal component of the factored resistance in the inclined portion of the offset bar. Where ties or spirals are used, they must be placed within 150 mm from the point of the bend.

13 DESIGN OF PRESTRESSED MASONRY

Stress in Tendon at Ultimate Strength of Member. For ultimate strength calculations, it is made clearer that the stress in the prestressing tendon may be determined either from strain compatibility analysis using the actual stress-strain curve for the reinforcement used or by equations provided for cases of bonded or unbonded steel tendons.

Shear Resistance of Walls and Columns. As part of the attempt to rationalize the treatment of shear design, this clause directs designers back to the shear design provisions for reinforced walls and columns but with the following addition. In this case only, the effects of axial compression load, P_d, is to be taken as 90% of the dead load plus the effective prestressing force after losses plus any axial load arising from bending in coupling beams.

15 FIELD CONTROL TESTS DURING CONSTRUCTION

Test Frequency of Masonry Units. In an addition to the first sentence of this clause, it is stated that field control testing may be waved or test frequency reduced where evidence of compliance and assured uniformity is provided by the supplier to the satisfaction of the designer. While this may seem to place added responsibility on the designer, it was argued that many producers have adopted the practice of routinely carrying out such tests for the purpose of quality control; availability of such information could be considered to be satisfactory.

Mortar Tests. The first sentence in this section still stipulates that, where not otherwise specified, the proportion specification is to be used as the basis for acceptance of mortar. However, some confusion has persisted amongst contractors and designers. This confusion seems to stem from the occasional use of measured properties to confirm adherence to correct proportions or the sometimes use of proportion measurements to confirm adherence to property requirements. To try to clarify the intent, a note to this sentence states that "The proportion and property specifications are independent compliance paths for mortar contained in CSA A179 and no interplay is either intended or recognized in this standard or in CSA A179." Designers and contractors should review the practice of reproducing parts of previous specifications to ensure that these requirements are properly reflected in the content of current specifications.

Property Specification Grout Tests. As was the case for masonry units, field control testing of grout may be waived or test frequency reduced for property specification grout manufactured off-site in a batching plant where evidence of compliance and assured uniformity is provided by the supplier to the satisfaction of the designer. The quality assurance program of the manufacturer may be found to satisfy this requirement.

16 SPECIAL PROVISIONS FOR SEISMIC DESIGN

This is a length new clause that provides requirements for seismic design using design criteria that have undergone substantial changes. Detailed provisions for increased ductility are provided. Readers are referred to [3] for an in-depth overview of these new and expanded design provisions.

ANNEX A: DIMENSIONED CUT STONE AND MANUFACTURED STONE VENEER

Scope. Annex A underwent a major review and extensive changes now make it clear that the requirements pertain only to *engineered* stone systems where the stone units, or the masonry assembly constructed of the stone units, are attached to a structural backing using masonry connectors; it does not apply to thin veneer adhered by mortar or adhesives. Also, it does not apply to rough stone veneer or rough stone masonry.

Stone Units. For natural stone units, ASTM references are added for Serpentine and for Travertine Dimensioned Stone. As is frequently the case when new and different requirements are first introduced into design standards, notes to requirements may be extensive to serve as an educational tool; typically there is no readily available source of background information. This is the case here where durability considerations are discussed in some detail based on many years of experience of the working group assigned this task.

For *other natural stone units*, not covered by an ASTM standard or not meeting the applicable standard, a process for establishing compliance through the intent of the NBCC [3] and using reference ASTM test methods is presented. Similarly, for manufactured stone units, the applicable ASTM and CSA standards are referenced and a process similar to other natural stone units is specified for compliance of *other manufactured stone units* not covered by a standard. Although not stipulated, it would seem logical that suppliers would have to supply the appropriate compliance information. Nonetheless, it also seems likely that specifiers would have some duty of diligence in accepting that information.

Metals. The title *Dissimilar Materials* has been changed to *Material Compatibility* to help emphasise the intent in avoiding galvanic corrosion. Similarly "Testing" has been changed to "Assembly Testing" to emphasise that it is the assembly not just the components that must be tested.

Stone Veneer. For minimum thickness, the note to this clause has been changed to say that the minimum thickness is the actual specified thickness and that these minimums have been set to allow for tolerances; nothing less than the specified minimum is accepted. An ASTM reference was added to provide recommendations for minimum thickness by stone type. A note to Sentence A4.2 was added on working stress design noting that safety factors should be chosen not only through consideration of inherent variability of the stone and assembly properties and the type of failure but also type and duration of loading, stone fabrication tolerances, workmanship, and the consequences of failure. Reference to ASTM 1242 provides a generally accepted table of safety factors by stone type. The notes for support of stone are slightly expanded for clarification.

Structural Backing. The deflection limit for structural backing remains as span of the backing divide by 360. However, a note has been added to indicate that "tighter deflection limits can be specified based on the type of structural backing, moisture management (interior/exterior application), and stone matrix (layout) including position of engagement/anchoring, and means of anchoring/engagement".

ANNEX D: METHOD OF TEST FOR COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF MASONRY PRISMS

Apparatus. For bearing blocks, the requirement that the spherical seat in the test machine not rotate after initial alignment until ultimate load is reached has been removed. Most test machines did not conform to this requirement; it is very important that prisms be carefully aligned and centered under the spherical seat.

Testing. For testing of concrete block and concrete brick, CSA A165 is specified instead of the previous ASTM standard. Instead of stating that prisms shall normally be laid is stack pattern, it is now stated that prisms should be laid in the same pattern as specified in the structure unless it is known that the effect is minimal. For grouted construction, alignment of cells associated with stack pattern is pointed out as a case where stack pattern may be inappropriate.

For curing and testing, in addition to the previously existing requirement for masonry expected to be in permanently damp conditions during use to be tested in a damp condition, it is pointed out that "dampness generally results in prisms having lower strengths than similar ones that have been allowed to dry".

Correction factors in Table D.1 to account for the effects of different height-to-thickness ratios of test prisms has been changed so that the factors are the same for solid and hollow clay brick (CSA A82), calcium silicate units (ASTM C73), concrete block and concrete brick (CSA A165).

Test Procedures. At the beginning of this section, it is pointed out that while either hard capping without fibreboard or capping with fibreboard may be used, "fibreboard capping generally results in lower prism strength". Fibreboard capping is convenient for prism preparation but use of this method should take into account the intended use of the information. The requirements for hard capping are provided.

In calculating compressive strength of face shell mortared prism of hollow units, the commentary clearly specifies that the cross-sectional area used in the strength calculation shall be the same as the effective mortared area to be used in the design. This can vary from manufacturer to manufacturer based on minimum face shell thickness and the shapes of the cells as well as the bond pattern.

ANNEX F: EMPIRICAL DESIGN FOR UNREINFORCED MASONRY

General. The empirical design of unreinforced masonry has been a longstanding simplified method of design intended to facilitate economic design of small, relatively simple buildings. Its development goes back to a period when masonry design was not taught to engineering graduates; it had to be learned independently and simplicity of design helped in this regard. Limits on building height, hourly wind pressure, and seismic hazard index exist to try to avoid major under design relative to engineered design while at the same time not being excessively conservative. In the course of the normal lengthy review of the empirical requirements, a sensible balance between simplicity and consistency with engineered design is always the goal.

In the revisions to Annex F, it is now more clearly pointed out that, except as allowed for some partitions and nonloadbearing walls and unless shown by rational analysis of compatibility of displacements, empirical design cannot be used for parts of a building designed using the engineered masonry design method. That is, design methods are not to be mixed. For empirical design, use of a simplified version of the traditional working stress design method are explained in a note to the design section. In this regard, the simplified use of gross area for calculation of stresses for both hollow and solid units is clarified so that area of cavity space is not included for cavity walls supporting load on both wythes.

Lateral Support and Anchorage. The requirements for anchorage of walls have been made explicit and design criteria relative to wall thickness and slenderness have been reworded. The section on design and construction of parapets has been expanded to be more consistent with the requirements for engineered masonry.

Veneer. A change indicates that masonry or concrete structural backing for veneer may be designed using engineered masonry design rather than just empirical rules as the previous edition seemed to suggest. For other backing types, engineered design for the veneer must be employed. For support of the veneer, because of some potential for misinterpretation, it is made clearer that the maximum height for support above the foundation is 11 m but that when the total height does exceed 11m, it must be supported at every floor level starting at the second floor and that the maximum spacing of supports above the second floor may not exceed 3.6 m. Some designers had incorrectly concluded that on multi-storey construction, the first 11 m could be supported on the foundation and additional support only start at floors above 11 m.

Glass Block Walls. New provisions draw a distinction between interior and exterior glass block walls as well as the thickness of the glass block. Glass block with a thickness greater than or equal to 98 mm may have a maximum panel length between movement joints of 7.6 m and a maximum panel height between movement joints of 6.1 m. Units less than 98 mm are limited to a maximum panel length and height between moment joints of 3 m. Exterior glass block walls are limited to a maximum individual panel area of 8.5 m^2, however, interior glass block walls are limited to a maximum panel area of 8.5 m^2 for units less than 98 mm and up to 13 m^2 for units equal or greater than 98 mm in thickness. Finally, the reference to Table F.2 has been removed in lieu of these explicit provisions.

Arches. In conjunction with the inclusion of requirements for design of arches in Clause 7, new empirical rules have been include here for minor arches not exceeding 2.4 m span used in nonloadbearing walls. Minimum abutment lengths to resist arch thrust are specified based on arch span, arch depth, and height of abutments. More significant arches must be designed using engineered masonry methods.

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meetings. The views expressed represent those of the authors and, while they have attempted to be as factual as possible, the standards committee was not consulted on this content.

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