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THE NEW CSA S304.1-04 "DESIGN OF MASONRY STRUCTURES"

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ABSTRACT

The new CSA S304.1-04 structural design standard for masonry contains a large number of changes from the previous CSA S304.1-94. These include: mandatory limit states design; format; materials standards; load factors; concrete block types; resistance factors for masonry strength and stiffness; seismic provisions; rigidity of supports for masonry; fibre reinforced polymer reinforcing; composite walls; toothing of walls; sliding shear; two-way action of unreinforced wall panels; infill walls acting as shear walls; concentrated bearing resistance; unit masonry veneer; deflection requirements for lateral wind loads; reinforcement requirements; direction factor for compressive strength (χ); shear strength for beams; prestressed masonry; grout strength; dimension cut stone and manufactured stone; test methods; empirical design.

KEYWORDS: Code, limit states design, resistance factor, seismic, dimension cut stone.

INTRODUCTION

The new CSA S304.1-04 [1] structural design standard for masonry contains a significant number of changes from the previous CSA S304.1-94 [2]. The following is a list of some of the more significant changes.

LIST OF CHANGES

1. <u>Limit States Design Method:</u> The limit states design method contained in the main body of CSA S304.1-04 is now mandatory for engineered masonry. CSA S304.1-04 is the only masonry design standard permitted to be used by NBCC 2005 [3], replacing both CSA S304.1-94, limit states design, and CSA S304-M84 [4], working stress design. However, S304.1-04 permits the working stress method with dimension cut stone veneer and manufactured stone veneer (found in Annex A of the standard), and still requires it with the empirical design method for unreinforced masonry (found in Annex F).

2. <u>New Format:</u> The design requirements for unreinforced masonry walls and columns are collected together in one section. The same applies to other types of masonry elements. Seismic design of shear walls with nominal ductility is relocated from an appendix to the main body of

standard. The empirical design method is now in an annex because it: uses working stress design instead of limit states design; uses gross cross-sectional area instead of effective cross-sectional area when calculating compressive stresses; uses unit strength based on gross cross-sectional area instead of net cross-sectional area when using the tables listing the allowable compressive stresses; and uses a rule of thumb method for selecting h/t ratios for walls and partitions.

3. Materials Standards: A number of CSA materials standards have been withdrawn. Some of these have been replaced with references to appropriate ASTM standards (e.g. calcium silicate brick and prestressing steel).

4. Load Factors, Load Combination Factors and Importance Factors: These factors are determined by the NBCC Standing Committee on Structural Design and are given in the NBCC 2005 [3]. They are included in all the CSA design standards so that those unfamiliar can still determine the factored loads upon which the standard is based.

Load factors: The return periods for wind and snow have been increased to 50 years, as is done in the USA [5]. As a result the loads have increased. The load factor for wind has been reduced to partly compensate for the increased wind loads. The load factor for snow load was not reduced because of a concern about the level of the reliability index for snow load found by Bartlett et al. [6]. The return period for earthquakes has been increased to 2500 years, as is done in the USA [5], but the load factor has not changed because there have been so many other fundamental changes to seismic design in the NBCC 2005.

Load combination factors: The companion-action method, first proposed by Turkstra and Madsen [7], has been adopted for load combinations in order to be consistent with the USA [5].

Importance factors: More important buildings now have their design load increased by importance factors for wind and snow, as well as earthquake.

5. New Concrete Block types and Change in Required Mortar Bedding: Formerly there were hollow, 75% solid and 100% solid blocks [8]. Now they are called hollow, semi-solid, and solid blocks respectively [9]. Formerly hollow block was face shell bedded while the two solid types were fully bedded in mortar [10]. Now both hollow and semi-solid are face shell bedded while only the solid is fully bedded [11].

6. Increased Resistance Factor for Unit Masonry: The resistance factor, φ_m, was increased from 0.55 to 0.60. This does not result in an increase in flexural tensile strength because the tabulated values were reduced to compensate. Appendix A of this paper provides the statistical derivation of the resistance factor for compressive loading of hollow concrete block masonry. It assumes that f'_m is determined from the tabulated values based on unit and mortar strength. If prism tests are used instead, it is believed that a similar resistance factor would be derived, but this has yet to be verified by the Technical Committee.

7. Effective stiffness under factored load, $(EI)_{eff}$: The resistance factor, φ_{er}, for the effective stiffness of reinforced walls and columns under factored load, has been increased from 0.65 to 0.75 . This is compatible with the change in CSA A23.3-94 [12].

8. New and Revised Seismic Provisions: Earthquake design provisions in NBCC 2005 [3] are very different from the previous edition, NBCC 1995. A seismic hazard index, $I_E F_a S_a(0.2)$, is used instead of seismic zones to determine when certain requirements apply, such as when and how masonry is to be reinforced and when empirical design can not be used. The seismic hazard index considers building importance, soil conditions, and magnitude of the design earthquake.

All masonry partitions are now required to be reinforced when the seismic hazard index is 0.75 or more, irrespective of weight or laterally unsupported height. A clarification was added that veneers do not require minimum seismic reinforcement [13].

Seismic design of shear walls now recognizes five types of shear walls (where R_d is the ductility factor and R_o is an overstrength factor):

1) unreinforced ($R_d R_o$ = 1x1 = 1),
2) reinforced conventional construction ($R_d R_o$ = 1.5x1.5 = 2.25) (used when required by seismic risk factor and/or to reduce the earthquake force),
3) reinforced limited ductility ($R_d R_o$ = 1.5x1.5 = 2.25) (used when reinforced conventional construction height limits are exceeded),
4) reinforced moderately ductile ($R_d R_o$ = 2x1.5 = 3) (used when reinforced limited ductility height limits are exceeded and/or to reduce the earthquake force), and
5) reinforced moderately ductile and "squat" ($R_d R_o$ = 2x1.5 = 3) (can be used when total height-to-length ratio of shear wall is less than one).

9. Rigidity Requirements of Masonry Supports: Detailed requirements are given in a non-mandatory note. These are considered to be good practice, but are not meant to replace the judgement of an experienced designer.

10. New Provisions for Fibre-Reinforced Polymer used as Reinforcement: Reference is made to CSA S806 [14], which covers design and construction in detail. Please note that, when required, provision of adequate fire resistance to externally applied FRP is very important due to the low temperature at which the polymer loses its strength.

11. New Design Provisions for Composite Walls: Bond strength between wythes must be checked where the wall is in bending about the weak axis. The requirements are adapted from the American masonry design code [15].

12. Toothing of Masonry Shear Walls: Where temporary construction stop-offs are required for construction sequencing, toothed joints can be used in shear walls for single-storey buildings only instead of racking back, provided that the toothed joints are reinforced and grouted.

13. Revised Sliding Shear Capacity for Unreinforced Walls: The sliding shear capacity between courses of masonry now includes the bond strength of the uncracked portion of the wall, as well as friction. This is based on shear bond values in the American masonry design code [15] and work by Hamid et al. [16] and Drysdale et al. [17].

14. New Design Provisions for Two-Way Action of Unreinforced Masonry Wall Panels Subject to Lateral Loading (Flexural Walls): This allows greater strength for a wall spanning in two directions where there are no significant openings. The method of failure line analysis by Drysdale and Baker [18] was used to tabulate bending moment coefficients in both directions.

15. New Design Provisions for Masonry Infill Walls Acting as Shear Walls: It has been common practice to use masonry infill walls in low-rise framed buildings to resist lateral loads. A method is given using a diagonal compression strut model as discussed by Drysdale et al. [19].

16. New Concentrated Bearing Resistance Provisions: An increase in bearing strength for concentrated loads is now included. The designer must assume triangular distribution of stress under bearing for beams due to beam rotation at the support. If the required bearing is longer than 300 mm parallel to the beam span, the load must be centred. This is to avoid the end of the beam from lifting off its support and resulting in an even larger concentration of stress at the front end of the bearing plate. The load can be centred using bearing plates with rocker bar detail. Of course, once the load is centred, it no longer has a triangular distribution.

The designer has the option of using a simplified bearing equation or more detailed equations that allow for increased bearing strength. The formula for bearing on solid brick masonry or fully grouted masonry was proposed by Page and Hendry [20]. Complex formulas proposed by Yi and Shrive [21] are also given for bearing on walls with hollow concrete block or brick units not fully grouted. When hollow masonry is used, solid or fully grouted masonry, or a spreader beam is required directly under the bearing, sufficient to disperse the concentrated load to the hollow masonry below.

17. Revised Provisions for Unit Masonry Veneer: Flexural bond strength is required to be not less than 0.2 MPa [22] to provide at least a minimum level of structural integrity and water tightness for the veneer. Ties can now be safely staggered with stud wall structural backings as shown by Yi et al. [23] provided the top row of ties is attached to every stud. Allowable deflection of structural backing is changed to L/360. Greater stiffness of the structural backing may be necessary depending on the design of the wall's moisture management system to handle rainwater that enters the drainage cavity.

18. Revised Provisions for Reinforced Masonry Slender Walls: Slender walls are now defined as walls with slenderness kh/t greater than 30. The maximum area of reinforcing increased from 80% to 100% of balanced steel ratio based on reconsideration of the work by Amrhein et al. [24], which showed very large deflections at the ultimate limit state. Slender walls must now meet the same requirements as other walls, as well as the special requirements for slender walls.

19. Revised Serviceability Requirements for Reinforced Walls and Columns: Wind load deflection changed to L/360 for masonry veneer to be consistent with stud wall structural backings (see Item 17). The Branson equation for I_{eff} has been replaced with simplified equations adopted from the American masonry design code [15]. Crack control requirements have been deleted since it was considered to be unnecessary to control cracking of reinforced walls and columns.

20. Revised Reinforcement Provisions for Reinforced Masonry: Minimum vertical reinforcement in walls subject to axial load was clarified. Minimum seismic reinforcement was clarified. Masonry columns are now required to have the minimum 4 vertical bars placed one in each corner. Changes to development lengths and lap splices were based mainly on provisions in CSA A23.3-94 [12] and modified to suit masonry.

21. Direction Factor in Compression (χ) Increased: This factor applies when the compression load in the structural element is not in same direction as the load in a prism test. It usually applies to beams and diagonal compression struts (see Item 15). The direction factor has been increased from 0.5 to 0.7 when grout is continuous parallel to the bed joints in the beam.

22. Revised Shear Capacity for Beams: Shear capacity of masonry components is reduced and depth effect equation modified. Provisions for ungrouted solid brick masonry beams are added. These changes were based on a re-evaluation of test results by Fereig and others [25 to 28].

23. New Provisions for Prestressed Masonry Beams, Walls and Columns: These are based mainly on prestressed concrete provisions in CSA A23.3-94 [24] modified to suit masonry. British [29] and Australian [30] standards were also consulted.

24. Grout Strength: Clarification was provided. "The designer, when testing the compressive strength of grout prepared in accordance with the proportion specifications [22] and cast in non-absorbent cylinder moulds, should expect average 28 d minimum strengths around 10 to 12 MPa. Such grout strength levels are accepted by this Standard and lead to satisfactory performance." The in-situ strength of the same grout would be significantly higher than that of the test cylinders. This is due to the concrete blocks, which absorb some of the water from the grout, thereby improving its strength.

25. New Provisions for Dimension Cut Stone Veneer and Manufactured Stone Veneer: There was a need for an engineered analysis instead of relying on the rules-of-thumb in CSA A371-94 [10]. This type of masonry veneer is used extensively, including on some of Canada's tallest buildings. Dimension cut stone is made from natural stone and is also known as "dimension stone" and "sawn stone". Manufactured stone includes masonry products manufactured from concrete, clay (shale) and calcium silicate except those products classed as unit masonry. Two types of veneer are recognized: mortar bedded or individually supported. Minimum thickness for mortar bedded veneer is 75 mm (90 mm with raked joints). For mortar bedded veneer, flexural bond strength is to be not less than 0.2 MPa where the bond is relied upon. Absolute minimum thickness for individually supported exterior veneer is 28 mm. Testing and calculations must be done to determine the required thickness and connection strength. Calculations can be based on working stress design with appropriate safety factors. The resistance factors given for unit masonry are not applicable if limit states design is used.

26. Revised Test Method During Construction: Each field test of masonry units now consists of 5 units instead of 3. The average compressive strength must not be less than the specified strength, and individual units must have not less than 85% of the specified strength.

27. Revised Equation to Determine the Specified Strength of Masonry Units and Prisms: The "specified" compressive strength of masonry units and prisms for design purposes is now calculated as the average measured strength minus 1.64 (formerly 1.5) standard deviations (coefficient of variation times the average value) of the measured strengths. If the number of test specimens is less than 10, the coefficient of variation shall be the higher of the calculated value or 10%. The overall objective is to obtain a compressive strength that will exceed the specified strength 95% of the time.

27. New Test for Flexural Tensile Bond Strength: Suitable bond wrench test methods are given for determining flexural bond between mortar and masonry units at the design and materials selection stage.

28. CSA A369.1-M90 "Method of Test for Compressive Strength of Masonry Prisms": This Standard was to be withdrawn by CSA. It was decided to incorporate a revised version as an annex to CSA S304.1 rather than relying totally on ASTM C1314.

29. New Empirical Design Requirements: These include shear walls to resist lateral loads, anchorage of floors and roofs into shear walls, and column box-outs. The shear wall requirements are taken partly from the American masonry design code [15]. The empirical method cannot be used if the seismic hazard index, $I_E F_a S_a(0.2)$ (see Item 8), is equal to or greater than 0.35.

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APPENDIX A

Rationale for Masonry Resistance Factor - by R. G. Drysdale, Sept. 11, 1992

The material resistance factor for compressive strength of masonry is independent of other member capacity factors such as slenderness. An analysis has been done to assess the impact of various factors on compressive strength.

The basic formulation is

$$\varphi = \gamma_R \, e^{-\beta\,\alpha\,V_R}$$

where $\gamma_R = R_{mean}/R_{calc}$
V_R = coefficient of variation of resistance R
α = separation function taken as 0.75
β = safety index chosen as 3.5 to be consistent with other materials
R_{mean} = mean resistance including any bias
R_{calc} = calculated resistance

On a simple force basis, the effect of compressive strength can be formulated as

$$C = A_n \, f'_m \, K_c$$

where A_n = area
f'_m = design compressive strength
$K_c = K_{workmanship} \, K_{member} \, K_{unit}$

Then $\gamma_R = \underset{A_n}{1.0} \times \underset{f'_m}{1.25} \times \underset{K_{workmanship}}{0.80} \times \underset{K_{member}}{0.85} \times \underset{K_{unit}}{1.25} = 1.063$

where the average area is taken equal to the specified area, the mean compressive strength is taken as 1.25 times the specified value, workmanship on average is taken as resulting in 0.80 capacity, member strength is taken as 0.85 of prism strength, and on average the unit used has a strength of 1.25 times the specified value.

The variability is determined by

$$V_R = [\,(V_{area})^2 + (V_{f'm})^2 + (V_{workmanship})^2 + (V_{member})^2]^{\,0.5}$$
$$= [\,(0.01)^2 + (0.08)^2 + (0.20)^2 + (0.05)^2]^{\,0.5} = 0.22$$

(note: $V_{f'm}$ includes unit strength variation as well as prism test variation)

where the variabilities associated with each factor affecting compression force are best estimates based on data and judgement. The result is

$$\varphi = 1.063 \, e^{-3.5(0.75)(0.22)} = 0.60$$

Obviously the result is directly proportional to γ_R and is a function of the exponent term. Increasing or decreasing this by 15% gives a range of φ from 0.55 to 0.65.

The $\varphi = 0.60$ value was arrived at using what are thought to be reasonable and slightly conservative estimates of the factors involved. For a specified unit strength, the mean prism strength of 1.25 times the design strength is justified by SCPI (note: now Brick Industry Association) and Chahine data (note: Chahine, G.N., Behaviour Characteristic of Face Shell Mortared Block Masonry Under axial Compression. M. Eng. Thesis, McMaster University, Hamilton, Ontario. 1989.). The 1.25 ratio of unit strength provided versus specified is much lower than the 1.60 value found from 29 block companies for 15 MPa block, but is used because of lack of data at higher strengths. It is the right sort of ratio for manufacturers to avoid understrength. The use of a 20% average strength decrease due to workmanship is very much a judgement call but along with the $V_{workmanship}$ value of 20% is justified based on documented effects of weak mortar, thick mortar joints, misalignment of units and partial filling of joints. Compressive strengths based on 2 unit high prisms tend to overestimate member strength. The 0.85 factor applied is a conservative allowance for this factor.

Turkstra found φ values of 0.60 and 0.20 respectively for inspected and uninspected masonry using β values between 4 and 5. Slenderness effects were included in this analysis (note: subsequent to Turkstra's analysis, it was decided that slenderness effects would be considered separately and have a resistance factor applied directly to $(EI)_{eff}$ (see Item 7)). His proposed φ = 0.40 in the original draft of S304.1 (note: reference is to early drafts of S304.1-94) seems unduly conservative.

Comparisons with other codes (note: ca. 1992) show:

<u>UBC (USA)</u>

Slender walls	$\varphi = 0.80$
Shear walls	$\varphi = 0.65$ for axial load
	$\varphi = 0.85$ for flexure
	$\varphi = 0.60 – 0.80$ for shear

<u>AS 3700 (Australia)</u>

Unreinforced	$C_m = 0.45$ for compression
	= 0.60 for shear
	= 0.60 for flexure
Reinforced	$C_r = 0.75$ for transient out-of-plane
	= 0.70 for others

<u>BS 5628 Part 2 (UK)</u>

(note: reinforced masonry)

1 / 2.3 = 0.44 for compression and bending

1 / 2.0 = 0.50 for shear

1 / 1.5 = 0.67 for bond

SIA U177/2(Switzerland)

$1 / \gamma_R = 1 / 2.0 = 0.50$ for compression

NZ 4230 (New Zealand)

Reinforced $\varphi = 0.85$ for flexure

$= 0.65$ for axial load