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BENEFITS OF USING THE NEW CANADIAN MASONRY LIMIT STATES DESIGN STANDARD

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ABSTRACT

With the recent approval of CSA Standard S304.1-1995 – *Masonry Design for Buildings*, masonry joins the other major structural materials in Canada which are based on the Limit States Design philosophy. The use of this new design standard will result in more consistent and more efficient use of masonry as a structural building material.

The primary purpose of this paper is to highlight the significant benefits to designers of using this new standard. The paper covers:

- the progress of masonry material standards;
- the changes in the new masonry design standard from the previous standard;
- comparison of the approaches to masonry wall design;
- design examples utilizing the current standard; and
- the benefits of using this standard.

For the designer, this paper provides background on the design of masonry wall systems using the new standard and the resulting more efficient walls that can be designed. For the owner, an awareness of the decreased costs is provided and the need for them to ensure they have retained designers who will utilize the new standard to its fullest is apparent.

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INTRODUCTION

Most Canadian structural material design standards are based on the Limit States Design (LSD) philosophy. Until this year, masonry was one of the exceptions. There are various reasons for this – masonry has a long-standing tradition as a conventional material most often satisfactorily designed using empirical methods; masonry research has lagged behind research on other building materials; masonry loadbearing capacities have been considered to be highly variable due to the significant dependence on workmanship and, as such, refinements to the design methods did not seem appropriate; and many other countries had not as yet adopted a Limit States Design approach to masonry.

The above reasons, general inertia, and the fact that adequate masonry education generally has not occurred in engineering schools have all combined to keep masonry from developing more quickly as a modern building material. However, the much improved level of knowledge due to the surge in masonry research over the past 20 years and the need to maintain or improve the competitiveness of masonry have provided the impetus for development of a more rational approach to masonry design.

The Limit States Design approach provides more consistent levels of safety and serviceability in the design of structures. Adoption of a Limit States Design approach for masonry also results in a compatible design approach for engineers familiar with designing using other materials. Particularly where structures are built with combinations of materials, use of the previous working stress approach creates additional work for the designer in the calculation of forces.

CHANGES TO THE MASONRY DESIGN STANDARD

Because of the adoption of the Limit States Design approach to masonry, the Canadian design standard has undergone significant changes. The following discussion provides a summary of the major changes to masonry design from the previous 1984 (CSA 1984a) version to the current 1995 version (CSA 1995).

Consistency with the Concrete Standard

To help designers become comfortable with designing in masonry, the format of the concrete standard was adopted for this standard. To the extent possible, the organization of topics, the language, and even the calculation processes mirror the concrete standard. In this way, designers familiar with the layout of the concrete standard and concrete design can easily find their way around this new masonry standard.

Materials

The new editions of CSA Standard A371 – *Masonry Construction for Buildings* (CSA, 1994a); A370 – *Connectors for Masonry* (CSA, 1994b); and A179 – *Mortar and Grout for Unit Masonry* (CSA, 1994c), have incorporated significant changes in 1994 to improve their technical content and make them compatible with each other and the LSD requirements. They are referenced in the design standard. Much improved and more thorough requirements for masonry reinforcing details, development and splices have also been included. There are more than twice as many clauses specifying reinforcement requirements in the new standard compared to the 1984 edition.

Load Determination
A designer performing load take-offs/calculations on a building should not have to run down the loads in two different ways because of the different materials used in the construction of a building. The new standard uses the nationally adopted load factors and a designer now need only perform one set of load calculations and refer to the appropriate material standard to determine resistances.

Calculating Resistances
In order to develop a Limit States Design approach for masonry, resistance factors were developed for masonry. The resistance factor, ϕ_m, for the ultimate limit state of masonry in compression, tension, shear and bearing is 0.55, which is close to the value for concrete. The value was based on statistical probability and, due to some uncertainty regarding on-site influences on variability due to workmanship, it was conservatively chosen at the lower bound of calculated values. Reinforcing and prestressing steel have the same resistance factors as in the concrete standard. Masonry connectors now also have resistance factors – a value of 0.9 is used for connector material failure, compatible with other steel elements, and a value of 0.6 is used for masonry-type failures, reflecting the variability of the behaviour of the connector in masonry. (These values are contained in the design standard by reference to the connector standard – CSA A370-94 (CSA, 1994b).)

With regard to material strengths, the characteristic compressive strengths for brickwork and blockwork are the same as in the previous standard except that values for Type M mortar are no longer included. For tensile strength, ultimate limit state values have been simply calibrated to the previous values. For example, the 1.4 MPa tensile strength for tension parallel to the bed joints in clay brick masonry and Type S mortar corresponds to the previous allowable stress of 0.50 MPa when it is multiplied by the resistance factor and divided by the live load factor (i.e., 1.4(0.55)/1.5 = 0.51 MPa). Therefore, when tension in the masonry controls capacity, there is little change.

Some of the member resistances have changed significantly and, in some cases, the design loads that sections can carry have increased substantially. In compression, the increases arise from using a greater fraction of the characteristic compressive strength and from use of the rectangular stress block (i.e., a stress of $0.85 f'_m$ is typically distributed over 0.8 of the compression zone) instead of the triangular distribution from working stress calculations. A simplistic comparison is made using solid masonry with f'_m = 10 MPa and a 1 m long by 190 mm thick wall, and assuming dead load and live load are 2/3 and 1/3, respectively, of the total load. At an eccentricity of 0.2 t and an allowable compressive stress of $0.3 f'_m$, the load that could be resisted by Working Stress Design (WSD) is, $P = ((0.3 \times 10)/(2 \times 1000))\ (0.9 \times 190) \times 1000 = 257$ kN, whereas for LSD, $1.25\ (2/3\ P) + 1.5\ (P/3) = (0.55)\ 0.85\ (10)\ (1000)\ (0.6 \times 190)$ so that $P = 400$ kN. This is an increase of 55%. For a wall in flexure, say entirely due to wind load, the moment capacity is nearly proportional to the tensile stress permitted in the steel. In WSD, the 165 MPa upper limit is therefore compared to the limit state stress of 0.85 (400 MPa) divided by the 1.5 load factor. The increase in unfactored load that can be carried is 37%. Significant changes in the determination of the member resistance of masonry in shear, masonry beams, and the bearing resistance of masonry under concentrated loads, have also been made due to the considerable research performed in these areas over the past 10 years.

Design Changes
Designers will be allowed to use the old WSD standard during a five year transition period. However, they are not allowed to pick and choose between WSD and LSD for the same building. The designers must choose one or the other design method. The WSD method will be eliminated in the year 2000 when the next edition of this standard is issued. The empirical design section of this new standard is similar to the previous standard with the exception of new roof hold-down requirements and modifications to the sections on veneer and glass block wall design.

In the engineered section, the design of masonry walls and columns and masonry beams have been included in two new sections with significant changes to both. For walls and columns, the design of very slender walls (walls with $kh/t > 30$) has been included. The coefficient method for the design of masonry walls and columns subjected to flexure and axial loads in the 1984 standard has been eliminated. The load deflection method, utilizing either the load displacement method or moment magnifier method for secondary moment effects, is the only acceptable design method in the new standard.

New requirements for the design of masonry shear walls have been developed as well as new serviceability requirements for reinforced walls. For masonry beams, numerous new requirements have been provided for flexure, shear and deflection.

The new standard has also taken a big step closer to providing designers with complete requirements for the seismic design of masonry. Minimum seismic reinforcement requirements for walls have been included in the standard and a new section on seismic design has been added to the non-mandatory appendices. This appendix addresses the special seismic design requirements for nominally ductile walls (i.e., $R = 2.0$). It provides requirements covering plastic hinging, shear, ductility and reinforcement details for reinforced masonry walls.

Although not relevant to the design of masonry walls, it is worth mentioning that new sections in the design of masonry veneers and glass block walls have been added to the engineered design section as well. For veneer walls, the concept of a veneer wall with a flexible structural backing system is addressed. Some of the requirements include the specification of tie loads, requiring the backup to sustain the full wall load, and specifying a deflection limit of L/600 for the veneer to limit the crack width in the veneer for water penetration reasons. It is noted that this deflection requirement is deemed to have been met if the flexible backup deflection is limited to L/720 and tie deflection is limited to 1.0 mm under specified conditions. For the design of glass block walls, several requirements are included. With regard to serviceability, deflection limits of L/600 are provided for glass block wall supports, movement joint spacings are provided and requirements regarding volume changes in the glass block wall are specified. With regard to strength, load tests are required to determine the modulus of rupture and elastic plate theory is the basis for determining internal forces in the design.

COMPARISON OF WSD AND LSD DESIGN METHODS

Before illustrating the benefits of the new LSD method for the design of masonry walls, it is worth briefly highlighting the methods and procedures followed when using the WSD method or the LSD method.

Working Stress Design (WSD)

When using the WSD method, the designer has the choice of using the coefficient method or the load deflection method. The allowable vertical load for a masonry wall using the coefficient method is based on the formula:

$$P = C_e C_s f_m A_m \quad [1]$$

where C_e is the eccentricity coefficient, C_s is the slenderness coefficient, f_m is the allowable axial compressive stress and A_m is the mortar bedded area. Depending on whether the wall is bent in single or double curvature or is unreinforced or reinforced, there are limitations on slenderness for use of this equation.

For the WSD load deflection method, either the displacement method or the moment magnifier method can be used to determine secondary moment effects. The secondary moment using the displacement method is based on the product of the axial load and the displacement as determined using a rigidity coefficient of:

$$\frac{E_m I_{eff}}{4} \quad [2]$$

where I_{eff} is the effective moment of inertia based on formulae using cracked and uncracked moments of inertia of the sections, depending on the ratio of the end eccentricities. For the moment magnifier method, the moment magnifier expression is based on a critical load of:

$$P_{cr} = \pi^2 E_m I_{eff} / 4h^2 \quad [3]$$

where I_{eff} is as discussed above.

Limit States Design (LSD)

As mentioned in the previous section, the LSD method utilizes only the load deflection method. The coefficient method has been eliminated. Similar to WSD, the designer has the option of using direct load displacement calculations or the moment magnifier method.

For the load displacement calculations, the difference between LSD and WSD is in the calculation of the wall stiffness. Using LSD, for unreinforced masonry, the effective wall stiffness is calculated based on:

$$(EI)_{eff} = 0.4 E_m I_o \quad [4]$$

where I_o is the moment of inertia of the effective cross-sectional area. For reinforced masonry, the formula is:

$$(EI)_{eff} = E_m \left\{ 0.25 I_o - (0.25 I_o - I_{cr}) \left[\frac{(e - e_k)}{(2 e_k)} \right] \right\} \qquad [5]$$

where I_{cr} is the moment of inertia of the cracked section about the centroid of the cracked section, $e = M_p / P_f$ and $e_k = S/A_e$. Also the effective stiffness is limited to the range between I_{cr} and $0.25\ I_o$. It is worth noting that the value for E_m using the LSD method is $850 f'_m$ as compared to $1000 f'_m$ for WSD.

For the LSD moment magnifier method, the same moment magnifier expression is used as for WSD, except that the factored load and moments are used; C_m, the moment diagrams factor, is not set at 1.0, but $C_m = 0.6 + 0.4\ M_1/M_2$; and the calculation of P_{cr} is different.

$$P_{cr} = \pi^2 \phi_e (EI)_{eff} \ / \left[(1 + 0.5 \beta_d)\ (kh)^2 \right] \qquad [6]$$

where $\phi_e = 0.65$, $(EI)_{eff}$ is as discussed above, β_d is the ratio of the factored dead load moment to the total factored moment, and kh is the effective wall height but not less than $0.8\ h$.

It is difficult to compare the differences between these two design procedures based on the equations discussed above. For this reason, the next section graphically illustrates the difference between these two methods by developing typical interaction diagrams for both methods and comparing them.

MASONRY WALL DESIGN

The LSD approach (CSA, 1995) for the design of masonry walls has the direct benefits of similarity to concrete design (CSA, 1984b) and, in many areas, significant increases in design capacity compared to WSD methods (CSA, 1984a). In WSD, the designer can use either the coefficient method or the load deflection method, as mentioned previously. Although the coefficient method was originally developed for unreinforced solid masonry with eccentricities less than $t/3$ (BIA, 1969), code provisions were developed to allow this method to be used for larger eccentricities (with avoidance of cracking) and for reinforced walls (by multiplying load and moment capacities by the slenderness coefficient). Alternately, the moment magnifier method for WSD requires detailed calculation of section properties to determine the EI value to be used in calculating the moment magnification. These calculations are more complex than the corresponding calculations for reinforced concrete design. Also, the calculation of section capacities, to provide resistances greater than the combined axial load and magnified moment, are more time consuming unless design aids are available.

Properties and Calculation Methods for a Typical Wall

A 190 mm thick wall built with 20 MPa hollow concrete blocks and Type S mortar is used to compare LSD with WSD. For these materials, the compressive strength, f'_m, is

13 MPa for hollow block and 10 MPa for blockwork which is grouted solid. For partially grouted walls, interpolation is possible, where, for instance, grouting every third cell (i.e., 600 mm centre to centre spacing) would give a weighted f'_m = 10 + 2/3 (13 - 10) = 12 MPa. Although the strength of grouted masonry is lower than ungrouted masonry, the increased area of the section results in increased capacity provided that a significant part of the grouted area is in compression. However, this is not the case when the neutral axis is near to or within the face shell mortared area of the wall. In such cases, it is correct and advantageous to use the f'_m value for hollow masonry. The grout cannot cause a reduction in capacity.

For WSD, the allowable compressive stress is 0.3 f'_m and the allowable tensile stress is 0.16 and 0.25 MPa for hollow and grouted blockwork, respectively. For LSD, the rectangular stress block of 0.85 f'_m is applied to 0.8 of the depth of the compression zone and the tensile strengths are 2.8 times the corresponding allowable stresses. All LSD strengths are multiplied by ϕ_m = 0.55. For reinforcement with a 400 MPa yield strength, the allowable stress is 165 MPa whereas LSD uses 0.85 (400) = 340 MPa.

In the calculations for the figures discussed below, the grout is considered as only necessary to bond the reinforcement into the wall. This allows a more meaningful comparison between unreinforced and reinforced sections because unreinforced walls are seldom grouted. This does mean that the benefits of partial grouting at low eccentricities are neglected for reinforced walls. However, since reinforcement is normally required for large eccentricities where the neutral axis is generally near the face shell zone, this is usually not an excessively conservative approach.

For face shell mortaring of the block, the effective area is approximated as corresponding to an equivalent 37.5 mm face shell thicknesses rather than the 32 mm minimum thickness of the face shell. This accounts for overlap of thickened areas of the block laid in running bond. Reinforcement consisting of 500 mm^2 per metre of wall is located in the middle of the wall.

For the WSD calculations, unsymmetric single curvature with a ratio of end eccentricities of $e_1/e_2 = 0$ is used. For LSD, the effective heights are taken as equal to the clear height (i.e., $kh = h$). Also, the factored dead load moment is conservatively taken as half of the total factored moment.

Comparison of Capacities

Section capacities and capacities for slenderness of h/t = 20 and 30 are shown in Figs. 1, 2 and 3 using the WSD coefficient method, the WSD moment magnifier method and the LSD moment magnifier method, respectively. Both the reinforced and the unreinforced cases are shown and these share the capacities for unreinforced masonry for eccentricities up to the point where the reinforcement begins to take tension. Compression in the reinforcement is neglected because provision of tie support for the reinforcement is not practical.

Comparison of Figs. 1 and 2 show that at low eccentricities, the coefficient method gives higher capacities. The reason is that there is no minimum eccentricity and the full capacity for concentric axial load is used for eccentricities up to 0.05 t. Alternately, the moment magnifier method prescribes a minimum primary moment corresponding to an eccentricity of 0.1 t or 25 mm, whichever is larger. For eccentricities above about

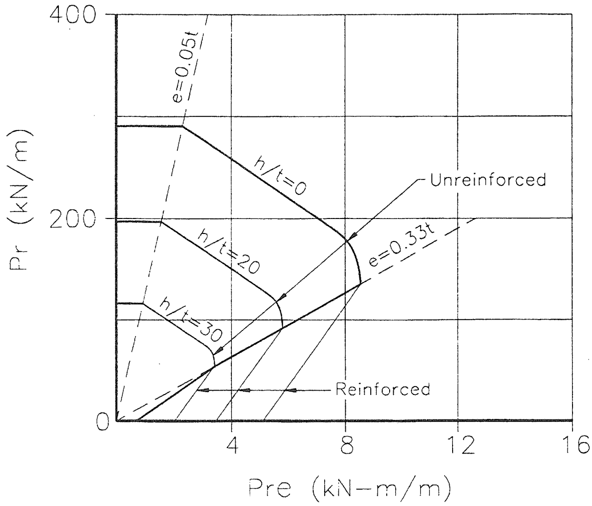


Fig. 1. Interaction Diagram* – WSD Coefficient Method

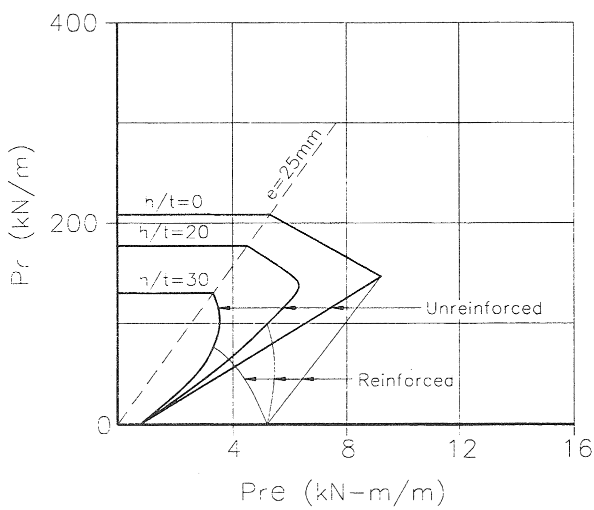
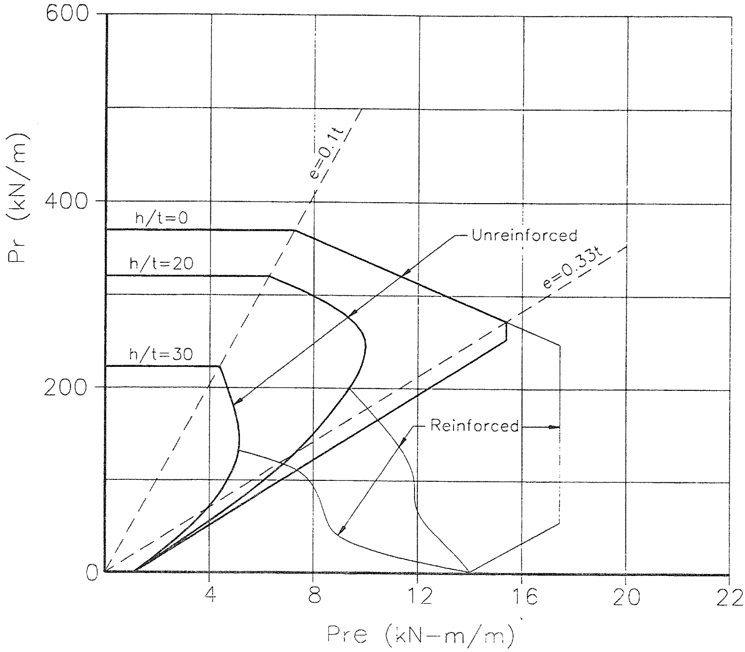


Fig. 2. Interaction Diagram* – WSD Moment Magnifier Method

*Key:	190 Hollow Concrete Block	$A_s = 500\ \text{mm}^2/\text{m}$ (where used)
	20 MPa Block Strength	$e_1/e_2 = 0$
	Type S mortar	$f_y = 400$ MPa



Key: 190 Hollow Concrete Block
20 MPa Block Strength
Type S mortar
$A_s = 500\ mm^2/m$ (where used)
$e_1/e_2 = 0$
$f_y = 400$ MPa
$\beta_d = 0.5$

Fig. 3. Interaction Diagram – LSD Moment Magnifier Method

25 mm, the section capacities (h/t = 0) are similar but, for slender walls, the capacities calculated using the moment magnifier method are significantly higher at the low eccentricities and again near pure bending for the reinforced wall.

To compare the LSD design capacities in Fig. 3 with the WSD values in Figs. 1 and 2, the values in Fig. 3 can be converted to unfactored loads by dividing by a factor between 1.25 and 1.5, depending on the ratio of dead to live load. A factor of 1.35 corresponds to a live load equal to 2/3 of the dead load. Of the WSD methods, only the design loads using the coefficient method for very small eccentricities exceed the unfactored LSD values. These low eccentricities are, in fact, not permitted in the moment magnifier method where it seems proper that imperfections and accidental misplacement of loads limit the axial load carrying capacities of walls.

A large part of the increased load carrying capacity using LSD is due to the differences between calculations using allowable stresses and linear elastic analyses versus strength design. For instance, at $e = t/3$, the unfactored capacities are 138, 146 and 193 kN/m, respectively, for the coefficient, WSD moment magnifier and LSD methods. At pure moment, the unfactored LSD section capacity is almost double the WSD values.

For the slender reinforced walls, these differences tend to be amplified. For instance, at $h/t = 30$ and $e = 25$ mm, the unfactored design loads are 89, 133 and 145 kN/m for the coefficient, WSD moment magnifier and LSD methods, respectively. At $e = t/3$, the corresponding unfactored capacities are 54, 57 and 86 kN/m whereas at $e = 100$ mm, the unfactored capacities are 28, 44 and 62 kN/m.

For slender unreinforced walls at eccentricities near $t/3$, the LSD method does not provide additional load-carrying capacity. This is appropriate because the capacities are extremely sensitive to small changes in eccentricity. The coefficient method has been criticized for not providing sufficient safety in this region (Drysdale *et al.*, 1994).

WALL DESIGN EXAMPLES

The interaction diagrams shown in Figs. 1, 2 and 3 provide a clear indication of the benefits of LSD. Two design examples representing extremes of design situations are provided to further illustrate this situation.

Example 1: Warehouse Wall
An 8 m high concrete block wall is to be designed to resist axial compression due to dead load of 24 kN/m and wind pressure of 1 kN/m^2.

LSD. For LSD, the factored axial load is 1.25 x 24 = 30 kN/m and the factored wind moment is 1.5 (1.0) (8^2)/8 = 12 kN-m/m. For a foundation which provides some restraint against rotation at the base of the wall, $k = 0.9$ is used giving $kh = 0.9 \times 8 = 7.2$ m. For the limit of $h/t = 30$, a 240 mm block is satisfactory. Using a 15 MPa block and Type S mortar, the compressive strength is 9.8 MPa which does not require extra testing of block where satisfactory quality control is in place (CSA, 1995).

Using an effective face shell thickness of 40 mm and 25 M bars (A_s = 500 mm^2) at 1 m spacing, the magnified moment is 16.8 kN-m/m whereas the moment capacity of the section at 30 kN/m load is 17.6 kN-m/m. Therefore the design is satisfactory with a margin of about 5%.

WSD. Unless a larger block is used, the 8 m high wall cannot be designed using WSD because of the limitation on the h/t ratio. Therefore, the warehouse wall would have to be reduced to a height of 7.2 m. The unfactored axial load is then 24 kN/m and the unfactored wind moment is 6.48 kN-m/m. Even with the 10% reduction in height, the previous design will not work. The most effective means of increasing the wall capacity would be to increase the block strength. Significantly increasing the amount of reinforcement would be another means, however, it is not an efficient approach. Thus, increasing the block strength to 20 MPa, resulting in a compressive strength of 13 MPa for hollow masonry, results in a magnified moment of 7.4 kN-m/m which is about 8% less than the moment resistance at the specified axial load of 80% of the dead load.

The reduced height of the wall (or, alternatively, use of a larger block), the higher block strength required, and the requirement for testing of blocks before and during construction combine to make the WSD approach more costly than the wall designed by LSD. It is also worth mentioning that this masonry wall could not be designed using the simplistic conventional method without the addition of masonry pilasters due to the limitation of $h/t = 20$.

Example 2: Multi-storey Commercial/Apartment Building
The wall height at the ground floor of a six-storey loadbearing masonry building is 3.2 m. The unfactored dead and live loads are 136 kN/m and 56 kN/m, respectively. Bending is only due to accidental eccentricity.

LSD. Try using a 15 MPa hollow 190 mm block wall with Type S mortar. The factored axial load is 1.25 (136) + 1.5 (56) = 254 kN/m. The minimum eccentricity is 0.1 t = 19 mm and e_1/e_2 is taken as 1.0. With a moment magnification of 1.42, the magnified factored moment is 6.85 kN-m/m. At the 254 kN/m axial load, the moment capacity of 7.64 kN-m/m is 11.5% more than required and the design is satisfactory.

WSD. Using the unfactored loads for the above design, the moment magnifier is 1.58 resulting in an unfactored design moment of 192 (0.025) 1.58 = 7.58 kN-m/m for the 25 mm minimum eccentricity. The working stress allowable load is actually about 30% less than this required load carrying capacity. Grouting the wall solid, increasing the block strength to 20 MPa, or using a larger block size, would be required for WSD.

All of these scenarios would add to the cost of the wall system, again, in addition to the requirement for testing, to make the WSD wall more costly than the wall designed by LSD.

BENEFITS OF USING THE LSD STANDARD

As has been discussed and as illustrated by the examples, there are significant benefits from using the new LSD standard for masonry walls. The benefits are best summarized as follows:

For Building Owners
- significant cost savings
- more consistent levels of safety in the walls being built

For Designers
- no need to perform separate load run-down calculations for different materials
- ease of use due to similarity with concrete design and ability to use certain concrete design aids
- other new requirements now provide for the design of masonry veneers with flexible back-up, glass block walls, very slender walls, and seismic design.

Designers familiar with design of reinforced concrete should be able to use the new LSD standard for masonry (CSA, 1995) with relative ease, especially when design aids become available. Overall, it should help them feel comfortable with use of masonry construction. The more uniform levels of safety and the often significant cost savings help the competitiveness and desirability of masonry structures.

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