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CANADIAN LIMIT STATES DESIGN APPROACH FOR MASONRY CONNECTORS

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ABSTRACT

Traditionally, masonry design in Canada has been carried out using the Working Stress Design (WSD) method and utilizing procedures that were developed in part through theoretical and experimental studies and also through traditional practice and experience. More consistent levels of safety and economic efficiency can be achieved by incorporating the Limit States Design (LSD) approach in the design of masonry structures just as in the design of steel and concrete structures. The 1995 publication of CSA Standard S304.1-94, *Masonry Design for Buildings (LSD)*, is providing designers with a LSD procedure for masonry design. Consequently, the LSD approach also had to be extended to cover the design of masonry connectors. The publication of the 1994 CSA Standard A370, *Connectors for Masonry*, provides designers with LSD procedures for the design of connectors for masonry. This paper deals with the factored resistance of masonry connectors and the background behind selecting different values for the resistance reduction factor ϕ.

INTRODUCTION

The first edition of a CSA Standard dealing specifically with masonry connectors, CAN3-A370-M84 "Connectors for Masonry" (Ref. 1), was written in terms of connector working loads. While the Technical Committee on Masonry Connectors arrived at a set of factors of safety ranging from 2.0 (for elastic buckling in a metal connector between points of lateral support) to 3.0 (for direct tension or compression or for direct shear or bending) and to a high value of 10.0 (for power-driven fasteners in solid masonry units or poured concrete), the Standard also included in the non-mandatory Appendix C recommended working loads and strengths for standard ties (a standard tie means a connector that had been standardized to facilitate specifications) which were based on the Committee's analysis of available test evidence and the safety factors listed in the Standard. When the need arose in the early 1990's to produce an updated version of the 1984 document to incorporate LSD criteria, the Committee carried out a reassessment of available test evidence and also a review of factors of safety. The decision was made to include in the new A370-94 LSD edition (Ref. 2) compatible Working Stress Design (WSD) clauses in order to give designers a change-over period from connector working loads to ultimate loads; the 1994 edition will be the last time that WSD will be incorporated.

The reassessment of available test evidence brought with it not only the new LSD philosophy but also changes to the WSD based factors of safety which simplified its rang. Thus factors of safety were lowered firstly, from 3.0 to 2.0 for cases of direct tension or compression and of direct shear or bending, and secondly, from 5.0 to 4.0 for mortar failures (pullout, pushout, crushing). These changes essentially achieved reasonable compatibility between connector design based on LSD and WSD philosophy.

The new Standard also no longer refers to "standard" connectors but has coined the words of "conventional" and "nonconventional" connectors. A conventional connector means "a connector in general use which has been shown to meet the criteria for connectors specified in CSA Standard A370 (Ref. 2).

RATIONALE FOR DERIVATION OF RESISTANCE REDUCTION FACTOR (ϕ)

The concept of Limit States Design (LSD) is that the probability of failure of a structure can be controlled by underestimating its resistance, R, and/or overestimating the load effects, S, and ensuring that $R \geq S$. Refining this concept on a probability basis, the LSD criteria take the general form expressed in simplified terms:

$$\phi R_N \geq \alpha S_N \tag{1}$$

where R_N is the nominal resistance computed on the basis of nominal material properties and dimensions; ϕ is the resistance factor (also known as capacity reduction factor or performance factor), always less than unity, which reflects the uncertainties in determining R_N; S_N is the nominal load effect based on specified loads; and α is a load factor, usually greater than unity, which reflects potential overloads and uncertainties associated with the determination of S_N.

The resistance reduction factor can be calculated as follows:

$$\phi = \gamma_R e^{-\beta \alpha V_R} \tag{2}$$

where:

α is a "separation function" having values between 0.707 and 1.0
β is the safety index = 3.5
V_R is the coefficient of variation.

$$\gamma_R = \frac{\overline{R}}{R_N}$$

and

$$R_D = \phi R_N$$

where:

$\overline{R}$ is the average maximum strength
R_D is the design strength
R_N is the calculated strength.

While it is simple to calculate the average maximum strength $\overline{R}$ and the coefficient of the variation V_R for each set of experimental data, it is questionable how to define the calculated strength R_N. Three different approaches have been tried:

- to use a strength equation after assuming a group of coefficients of variation that contribute to the connector strength.
- to use the 5th (or 10th) percentile of the maximum strength values.
- to use the working loads listed in the A370-M84 multiplied by the applicable safety factors.

Due to a scarcity of test evidence for many types of loading for conventional ties, the third approach was adopted. By employing the most relevant published experimental data for various types of conventional connectors, different groups of reduction factors were calculated corresponding to the type of connector and action (i.e. compression, tension, pullout ....etc) as illustrated in Table 1. In calculating all the resistance reduction factors listed in Table 1, a correction factor has been applied to all coefficients of variation to incorporate the reduction in confidence of the test results based on the number of tested specimens (see Appendix 1 for details).

Two observations can be made from the results illustrated in Table 1: Firstly, the available experimental results are limited and not sufficient for all types of masonry connectors: secondly, when there are significant experimental data available as for the z-wire ties in pullout, pushout or crushing failure modes, the results for the reduction factor vary considerably (0.46 to 1.36).

Table 2 provides selected resistance reduction factors for different types of connectors and actions. These values were chosen from those listed in Table 1 based on three considerations: consistency and adequacy of experimental data, workmanship, and experience, .

For simplicity and till further experimental results are made available, values for the reduction factor ϕ were grouped into two divisions: $\phi = 0.9$ for material failure of the metal components of the connectors and $\phi = 0.6$ for embedment failure or failure of the fasteners, or elastic buckling failure, of the connector. These values are published under Clause 8.4.2.1.2 of the CSA Standard A370-94 "Connectors for Masonry" (Ref. 2).

While the aforementioned methodology in defining the calculated value of strength, R_N, is sufficient for the calculation of the reduction factors for the listed conventional connectors, a new approach can be followed as more test results become available for the conventional and non-conventional connectors. The minimum value between the 5th percentile of the maximum strength and the average strength at the end of the elastic zone should be appropriate as the calculated strength R_N. By following the procedure outlined in the paper and employing the same safety index, safety factors for both the conventional and non-conventional connectors can be obtained. Applying the reliability based criteria assures a structural design with more consistent safety indices.

CONCLUSIONS

The Limit State Design approach for masonry connectors was implemented in the 1994 CSA Standard A370 "Connectors for Masonry". Values for the resistance reduction factor ϕ were introduced for the first time. Designers are now provided with more rational criteria to ensure safety indices and factors of safety compatible with those used to design the structure. The reliability based criteria for the masonry connectors will provide an expandable base for future development as more experimental data become available from testing both conventional and non-conventional connectors.

ACKNOWLEDGEMENT

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REFERENCES

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2. Canadian Standards Association, Connectors for Masonry. CAN3-A370-M94, Rexdale, Ontario.

APPENDIX 1: DETERMINATION OF COEFFICIENT OF VARIATION OF AVERAGE MAXIMUM STRENGTH USING TEST DATA

A minimum of five identical specimens must be tested in order to use the safety factor index procedure. It is clear that with only five specimens one does not have as valid an estimate of the coefficient of variation, V_R, as would exist if a larger number of specimens were tested. Therefore, it is reasonable to multiply the value of V calculated using n test specimens by a factor C in order to incorporate this reduction in confidence. Then

$$V_R = C\,V \tag{3}$$

where:
$C = 1.0$ if $n \geq 10$
$= [(1-b)/5]\,n + [2b-1]$ for $n < 10$
The recommended value of $b = 1.5$, and therefore
$C = 2.0 - 0.10\,n$

APPENDIX 2: INTERPRETATION OF SAFETY FACTOR PROCEDURE

Consider first the working design equation

$$F_w = R_w \tag{4}$$

where:

F_w = Working stress loads
R_w = Working stress design allowable

The strength design loading, F_s, can be expressed using a wind load factor of 1.5 as

$$F_s = 1.5 F_w \tag{5}$$

and the design strength, R_D, must be such that

$$F_s = R_D \tag{6}$$

and since

$$R_D = \phi R_N \tag{7}$$

Formulas 5 and 6 could be arranged as follows:

$$1.5 R_w = R_D \tag{8}$$

$$1.5R_w = \phi R_N \tag{9}$$

But from the basic definition of the safety factor (SF) for working stress design

$$R_w = \bar{R}/SF \tag{10}$$

so

$$1.5(\bar{R}/SF) = \phi R_N \tag{11}$$

or

$$SF = 1.5(\bar{R}/(\phi R_N)) \tag{12}$$

or

$$SF = 1.5e^{\beta \alpha V_R} \tag{13}$$

Table 1 Summary of the Calculated ϕ Factors

Type of Tie	Action	ϕ Factors	
Z-tie	Pullout. pushout, crushing	1.36, 1.06, 0.46, 1.23, 0.97, 0.58, 1.01, 0.48, 0.53	●
	Shear strength	1.0, 1.37	○
	Buckling	0.8	○
Truss	Pullout. pushout, crushing	0.5, 2.2, 1.52, 1.2	●
	Shear strength	0.75	○
	Buckling	0.88, 1.4	○
Rectangular	Pullout. pushout, crushing	1.42, 1.23, 1.87	●
	Shear strength	1.89	○
	Buckling	0.75	○
Corrug. Strip	Pullout. pushout, crushing	0.42 block masonry	○
		0.66 wood stud	○
		0.61 steel stud	○
	Shear strength	N.A.	
	Buckling	0.132 (non-conventional tie)	○

● Reasonable amount of experimental data available
○ Not sufficient data available
N.A. Not available

Table 2 Factor of Safety and ϕ Values for Connectors

Mode of Loading	Factor of Safety A370-M84	 A370-M94	ϕ A370-M94
Direct tension or compression in the metal connector	3.0	2.0	0.9
Elastic buckling in the metal connector between points of lateral support	2.0	2.0	0.6
Direct shear or bending in the connector or a combination of the two actions	3.0	2.0	0.9
Mortar failures (pullout, pushout, crushing, etc.)	5.0	4.0	0.6
Nail pullout from wood (short duration load)	4.0	4.0	0.6
Dovetail slot failures in concrete	4.0	4.0	0.6
Self-tapping screw pullout from steel stud	4.0	4.0	0.6
Drilled-in fasteners in solid masonry units or poured concrete	4.0	4.0	*
Power-driven fasteners in solid masonry units or poured concrete	10.0	10.0	*

* Drilled-in and power-driven fasteners are not specifically dealt with in the LSD-based A370.