THE EMPIRICAL DESIGN DILEMMA

D.A. Laird[1]
[1] Structural Engineer, Parsons Brinckerhoff Halsall Inc., Toronto, ON, M4P 1E4, Canada, email dlaird@halsall.com

ABSTRACT
CSA S304.1-04 "Design of masonry structures" uses an engineered design method (limit states design), but also permits the use of an empirical design method. The empirical approach is only allowed for unreinforced masonry and only if the building and its location are within specified limits for building height, seismic hazard index and hourly wind pressure. Many locations in Canada are ruled out by the seismic and wind limitations. Those locations remaining are mainly in parts of the Prairies and Ontario. When unreinforced masonry walls are designed for wind and seismic loading in accordance with the engineered design method, the walls are often significantly stronger than required by the empirical design method. Should this be allowed to continue or should the two design methods be reconciled? That is the dilemma. This paper discusses the two design methods and gives example calculations to compare the two. The relevant changes to the two design methods, the design loadings, and masonry construction over the past 50 years are discussed. Some relevant differences with the current American masonry code are also mentioned. Possible reasons are examined as to why the empirical design method has had such a successful history and why the empirical design method continues to be permitted. Some recommendations are included.

KEYWORDS: empirical design, limit states design, flexural tensile strength, wind load, seismic load, reliability analysis

INTRODUCTION
This paper is a discussion of the empirical design method for masonry and a comparison with the engineered design method (limit states design), both of which are contained in CSA S304.1 [1]. The paper is limited to Canadian structural design requirements unless noted otherwise. Only unreinforced hollow concrete block walls, shear walls and partitions are considered.

THE DILEMMA
The occasional structural collapse of a building, or portion thereof, has long been considered unacceptable by society [2]. As a result, governments have codified the structural design of buildings. The limit states design method has been adopted for structural design in Part 4 of the National Building Code of Canada (NBCC) [3], but *other methods are permitted if they give at least an equivalent level of safety (See NBCC A-4.1.1.5(2))*.
When unreinforced masonry walls are designed to resist lateral loads in accordance with the engineered design method, the walls are frequently governed by the flexural tensile strength. Often the resulting walls are significantly stronger than required by the empirical design method. This means the antiquated empirical design method has a lower level of safety than the modern engineered design method in some cases. However, there is no history of failures when the walls

conform to the empirical design method. Should this difference in strength requirements be allowed to continue or should the two design methods be reconciled? This is the dilemma.

FLEXURAL TENSILE STRENGTH

The ability of a wall to resists out-of-plane lateral load is determined by many factors. The empirical design method must be adequate for all situations if it is to be safe. The weakest situation is a wall spanning vertically where there is no continuity at the base, and there is a deflection gap at the top. In this case the main factors are the flexural tensile strength normal to the bed joints and the weight and thickness of the wall. Walls that cantilever above their structural support (parapets, guards and apron walls) are assumed to be anchored to their support rather than relying on flexural tensile bond to other materials.

Values for flexural tensile strength normal to bed joint were originally based on laboratory tests on walls using Portland Cement-Lime (PCL) mortars without air entrainment. For type S mortar, the value given in the 1965 working stress version of NBCC would have been equivalent to 0.12 MPa in the current CSA S304.1, but the value jumped up to the equivalent of 0.40 MPa in 1970 (after the unpublished 1967 NCMA tests) and has essentially remained at that value ever since. The flexural tensile strength values were grandfathered into the limit states version and simply calibrated to give the same outcome. Considerable laboratory wall testing has now been done to determine the correct values for concrete block under ideal conditions [4, 5], and some air dried wall tests have also been done [6]. Factors affecting bond strength are generally well understood [7]. Some factors affecting flexural tensile strength normal to the bed joint are as follows.

1. Mortar: The flexural tensile strength values for mortars using Masonry Cement have long been questioned [8] because of the high air entrainment. Tests [4, 5] indicate that Masonry Cement mortar is generally weaker than the reference mortar (PCL mortar with no air entrainment), all other things being equal. TMS 402 (aka MSJC Code) [9] reflects this fact, but not CSA S304.1. Mortar Cement mortar was created [10] to have air entrainment almost as high, but with a similar flexural tensile strength as the reference mortar. Mortar Cement mortar is not widely used in Canada.
2. Blocks tensile strength: With strong mortars, standard strength blocks in wall tests can fail in flexural tension before the mortar joints can fail [4, 5].
3. Wall thickness: Tests [4, 5, and 11] show that the thinner walls have a higher flexural tensile strength. This is explained by fracture theory [12].
4. Curing: Tests [4, 6] show that high flexural tensile strengths are possible with moist curing, but allowing a wall to dry out during the curing period can reduce the 28 day flexural tensile strength substantially. Unless the walls are wetted down periodically during the curing period, walls on site will dry out to some extent depending on the weather or on enclosed winter heating conditions. There has not been any extensive testing of various types of curing on site that would allow more accurate predictions of its effect, all other things being equal. Also, there are no moist curing requirements in the construction standard CSA A371 [13] and there is no mandatory site testing in CSA S304.1 to verify that the flexural tensile strength is being achieved.
5. Workmanship: Only good workmanship is used for constructing laboratory test specimens. The effects of mortar joint variations, retempering mortar, adjusting block, dusty units, inadequate weather protection, etc. are not included. The statistical effects of workmanship on site are little known, but obviously important.

EMPIRICAL DESIGN

The empirical design method was based originally on a rule of thumb approach (proportions and minimum thicknesses) derived from satisfactory past performance, and only applied to unreinforced masonry. An empirical design approach has been around for many thousands of years. Very impressive masonry structures have been built using this method and many are still standing.

Empirical design is presently in a normative (mandatory) annex of CSA S304.1. The empirical approach is only allowed for unreinforced masonry and only if the building and its location are within specified limits for building height (11 m above first floor for loadbearing walls including shear walls; 20 m above grade for other walls exposed to wind; no limit for partitions), seismic hazard index ($I_E F_a S_a(0.2)$ less than 0.35) and mean hourly wind pressure (not more than 0.55 kN/m^2). Many locations in Canada are ruled out by the seismic and wind limitations. Those locations remaining are mainly in parts of the Prairies and Ontario

The following is a list of "recent" changes regarding the empirical design method.

•1880s: With higher strength units and mortar, first allowable stresses for gravity loads appeared.

•1941: Included in the first NBCC and was the only method recognized until 1965.

•1977: Included in the first CSA S304 design standard for masonry.

•1984: Substantially revised in CSA S304 to bring cavity walls and walls with openings more in line with engineering principles, but h/t for hollow block walls increased from 18 to 20 in order to obtain consensus. Limitations were also placed on when the empirical design can be used.

•2004: Shear wall requirements added to CSA S304.1.

•2004: Relocated from the body of the CSA S304.1 standard to an annex because the differences with the limit states design method were causing confusion among designers. Empirical design method uses working stress design for gravity loads with the stresses based on gross area of hollow units, and uses rules of thumb for lateral loads.

ENGINEERED DESIGN

Engineered design for masonry was introduced into the NBCC 1965 as a working stress design method. Limit states design was introduced into CSA S304.1 in 1994. It is considered to be the best approach for a consistent level of structural reliability (safety). It is now the only method recognized in Part 4 (structural design) of the NBCC and in provincial/territorial building codes which are all based on the NBCC. Limit states design is the only method used in the body of CSA S304.1-04. It is also the only method used in all the other CSA structural design standards (wood, concrete, structural steel, aluminum) and in the Canadian Foundation Engineering Manual (geotechnical). Guidelines for the development of limit state design standards are given in CSA S408 [14]. CSA structural design standards (which are referenced by the NBCC) require a very low probability of failure (see Appendix, item A1). This is achieved by using an ultimate limit states design approach which uses load and resistance factors.

A reliability analysis is done to determine the resistance factors by considering the bias, the coefficients of variation and the target reliability index (β_T). The bias corrects for the fact that the actual average test strength is not normally the same as the calculated nominal (unfactored) strength. The coefficient of variation is normally obtained from extensive testing to failure. The reliability index sets the probability of failure and ensures similar levels of safety between different structural materials. In addition, since sudden failures are less desirable than gradual failures, different failure modes require different reliability indices. Reliability analyses have been done for masonry in compression [15, 16], but no analysis has yet been done in Canada for flexural tensile strength.

The following is a list of some recent changes regarding the engineered design method.

•1994: limit states design method was introduced in CSA S304.1 (the .1 was to distinguish it from the working stress version during the transition period), and made the only engineered method in 2004.

•1994: A minimum average flexural bond strength requirement of 0.2 MPa was introduced into the mortar and grout standard CSA A179 [17]. It uses the bond wrench test along with the masonry units and mortar in question. This is still the only flexural bond strength requirement anywhere for PCL and Masonry Cement mortars. It is much less that the specified (unfactored) flexural tensile strength values provided in CSA S304.1, Table 5, for use by designers.

•2003: Mortar Cement was introduced in CSA A3002 [18]. It requires a minimum characteristic flexural bond strength and confirmation using bond wrench test with concrete bricks. It is intended as an alternative to Masonry Cement when flexural tensile strength is an issue.

SOME SIGNIFICANT CONSTRUCTION CHANGES

The following is a list of some changes in construction that affected empirical design.

•Structural clay tile units were largely replaced by concrete block units. Brick units are now used mainly as veneer and outer wythe of cavity wall. Most structural masonry is now concrete block.

•Masonry Cement mortar has substantially replaced PCL mortar for new construction because Masonry Cement mortar is easier to use and more cost effective. Mortar Cement has replaced Masonry Cement in high seismic areas in western Canada due to similar American practice.

•Brick header units have been replaced by metal ties, so that full composite action no longer available to resist lateral loads unless the collar joint is filled.

•Solid masonry exterior walls with ties and partially filled, undrained collar joints have been replaced by insulated, drained cavity walls with ties, so that not even partial composite action is possible unless special rigid ties are used.

•A deflection gap is now often provided at the top of nonloadbearing wall panels instead of building tightly to the structure above. Preload and arching action would no longer be available to help resist lateral loads.

DESIGN COMPARISONS BETWEEN EMPIRICAL AND LIMIT STATES DESIGNS

Design comparisons have been made for gravity, wind, partition and earthquake loads. The comparisons are not comprehensive, but do show the main areas of difference. The results indicate that empirical design gives a lower level of safety than limit states design for: exterior walls resisting wind loads; shear walls resisting wind loads; and partitions resisting the suggested pressure difference (See Appendix items A2 to A4). The results also indicate that a very unsafe situation could occur with earthquake loads on empirically designed shear walls (see Appendix item A5). This would be partly because some design loads have changed in the last 50 years. For example, while wind has changed very little, earthquake has increased by a factor of five for unreinforced masonry (see Appendix items A6 and A7).

RELEVENT DIFFERENCES WITH TMS 402

1. The flexural tensile strength values for Masonry Cement mortars and air-entrained PCL mortars are lower than PCL and Mortar Cement mortars in TMS 402. The values for hollow

concrete block are 40% less for type S mortar and 50% for type N. CSA S304.1 does not recognize any difference.

2. The allowable flexural tensile strength values in TMS 402 were recently increased 33%. The increase was justified by a reliability analysis [19, 20]. See appendix Item A1.

3. In addition, the following are some of the relevant differences with the TMS 402 empirical design method:

a) The thickness t is the nominal thickness. Actual thickness is used in CSA S304.1.

b) The effective thickness for cavity walls is ($t_1 + t_2$). In reality this would only be possible with significant composite action, which is not normally the case. CSA S304.1 uses 2/3 ($t_1 + t_2$) but not less than greater of t_1 and t_2.

c) 1200 mm wide openings can be ignored for wind loads provided there is at least 2400 mm of wall between the openings. This allows 33% openings to be ignored. CSA S304.1 allows 15%.

d) The maximum h/t for hollow block walls is 18. CSA S304.1 allows 20.

e) Veneer and glass block walls are not, and partitions are proposed soon not be, part of the empirical section. The proposed partition requirements will be more stringent than CSA S304.1.

f) Basement walls retaining soil are included. They are confined to NBCC Part 9 in Canada.

g) Shear wall requirements are more stringent than CSA S304.1.

SATISFACTORY PAST PERFORMANCE

1. Structural Commentaries [21] clause L-18, discusses evaluation of an existing building structure based on satisfactory past performance. Essentially, it states that buildings built earlier than 1960 may be considered to have demonstrated satisfactory capacity to resist loads, other than earthquake, provided: careful examination shows no significant damage, distress or deterioration; the structural system is reviewed for critical details regarding load transfer; the building has demonstrated satisfactory performance for the past 30 years; and there have been no changes in the past 30 years that could significantly increase the loads or affect its durability, and none are contemplated. NBCC A-5.1.4.1.(4) also discusses the acceptance of existing cladding types in similar terms. This philosophy is useful in evaluating the empirical design method.

2. There has been no history of failures of walls due to wind loading [22] when designed and built to code. Instead, failures due to wind are mainly a result of inadequate temporary bracing during construction, long-term deterioration and extreme weather events (tornados).

3. The increase in h/t in 1984, that increased the lateral load effects on exterior walls, is not yet 30 years ago. Probably 60 years would be required so that the population of such walls would average out at 30 years. As a result, it is too early to say if the new h/t = 20 rule has had a satisfactory past performance. The same would apply to shear walls that were added in 2004.

4. Partitions have had a successful history when designed and built to code. There have been some failures due to wind loads entering the building through large openings in exterior walls, lack of adequate lateral support, or excessive load on attached shelving or brackets.

5. Design wind loads have increased a little in open terrain in the last 50 years. It is not clear that winds have really changed, although climate warming may do so in the future.

6. Satisfactory past performance is not applicable to earthquake.

DISCUSSION OF EMPIRICAL DESIGN METHOD

1. Where permitted, the empirical design method is commonly used by both architects and engineers. The masonry is easier to design and often more economical. However, concerns have been raised about wind loadings [23, 24].

2. The success of the empirical design method is based on a successful history of past performance with respect to gravity, wind and partition loads. It is obvious why gravity loads are not an issue, but wind and partition loads are a mystery. Some possible explanations would be:
a) *The specified (unfactored) flexural tensile strength may be too low:* This is possible with moist cured walls using PCL mortars or Mortar Cement mortars.
b) *The target reliability index may be too high:* See Appendix item A1.
c) *The specified (unfactored) loads may be too high:* Appendix item A8 shows that the specified wind loads may be a little too high. Earthquake design in-plane loads on shear walls have increased substantially over the last 50 years due to increased understanding of the effects on unreinforced masonry, but it is doubtful they are now too high.
d) *The specified loads may not have happened yet:* This is true for earthquakes. It is also true for some cities with respect to the last 50 years of wind gust records up to year 2000, but statistically this is normal and the 1 in 50 wind loadings can be expected to occur eventually.
3. There are other possible reasons for successful performance such as preloading, arching action, composite action, continuity and two-way action. This would explain many situations from the past. Current architectural designs tend to rule these out in many cases (deflection gaps, insulated drainage cavities, control joints, widely spaced cross walls and columns, large openings, etc.). Strengthening effects should not be relied upon to justify the empirical design method. Where these effects do exist, they can be analysed using limit states design to see if more economical walls are possible.
4. The suggested partition load represents a pressure difference that would cause a working load of 0.40 kN (90 lbs.) on a standard interior door. This could be caused, for example, by an open window in the outside wall of a reasonably air tight room during a strong wind. Walls around shaftways are also vulnerable. TMS 402 is proposing to reduce the h/t ratio for partitions.
5. There is no history of any significant seismic events in any areas where empirical design is currently permitted. As a result, there have been no known failures. However, limit state design and experience elsewhere would indicate the possibility of very poor performance for empirically designed shear walls.

CONCLUSIONS

1. The empirical design method is at least as safe as the limit states design method for gravity loads. It is generally less safe for lateral loads due to wind on exterior walls, wind on shear walls and for the suggested lateral load on partitions. It is also potentially very unsafe for seismic loads on shear walls. The current situation should not be allowed to continue.
2. There is considerable information on moist cured and air cured walls built in the laboratory. More information is required regarding the effects of curing and workmanship on site.
3. A detailed reliability analysis is required for flexural tensile strength on site in order to better estimate the true safety of the limit states design method, and thereby the true safety of the empirical design method for lateral loading.
4. The empirical shear wall requirements need to be revisited. Wall lengths are a concern.
5. Another attempt to reconcile these two methods is required. Likely it would have to be a compromise. Possibilities might include accepting a lower reliability index for wind, reducing the h/t ratios for exterior walls and partitions, lengthening shear walls to resist wind, and requiring an engineered design approach for shear walls resisting earthquake.
5. If the above items cannot be resolved successfully, consideration should be given to retiring the empirical design method.

ACKNOWLEDGEMENTS

I am indebted to the past and present members of the CSA S304 technical committee. During my 35 years on the committee, I have been exposed to many interesting opinions, industry issues, site investigations, research projects, technical papers, standards and text books.

REFERENCES

1. CSA S304.1-04 (2004) "Design of masonry structures", Canadian Standards Association, Mississauga, ON, Canada.
2. The Code of Hammurabi (ca. 1770 BCE), Babylon, Iraq, posted on the internet. [Code is carved in stone. Laws 229 to 233 were concerning penalties if a house was not constructed properly and collapsed. An empirical design method would have been used back then, and would already have had a history of over 5,000 years of successful stone and brick construction.]
3. National Building Code of Canada 2010 (NBCC), National Research Council of Canada, Ottawa, ON, Canada.
4. Research Evaluation of Flexural Tensile Strength of Concrete Masonry (1994), Research and Development Laboratory Project No. 93-172, National Concrete Masonry Association, Herndon, VA, USA, posted on the internet.
5. Melander, J.M, and Thomas, R.D. (1998) "Flexural Tensile Strength of Concrete Masonry Constructed with Type S Masonry Cement Mortar" 8th Canadian Masonry Symposium, Jasper, AB, Canada.
6. Matthys, J.H. (1990) "Concrete Masonry Prism and Wall Flexural Bond Strength Using Conventional Mortars" ASTM STP 1063 Masonry: Components to Assemblages, American Society for Testing and Materials, West Conshohocken, PA, USA.
7. Factors Affecting Bond Strength of Masonry (1994), PCA Masonry Information, Portland Cement Association, 542 Old Orchard Rd., Skokie, IL, USA.
8. Drysdale, R.G., Hamid, A.A., Toneff, J.D. (1983) "Comments on Block Masonry Design Code Provisions for Allowable Stresses" 3rd Canadian Masonry Symposium, Edmonton, AB.
9. TMS 402-11/ACI 530-11/ASCE 5-11 "Building Code Requirements for Masonry Structures" (2011), Masonry Standards Joint Committee, The Masonry Society, Longmont, CO, USA.
10. Melander, J.M., and Ghosh, S.K. (1996) "Development of Specifications for Mortar Cement" ASTM STP 1246, Masonry: Esthetics, Engineering and Economy, American Society for Testing and Materials, West Conshohocken, PA, USA.
11. Hamid, A.A., Drysdale, R.G. (1982) "Effect of Strain Gradient in Tensile Strength of Concrete Blocks" ASTM STP 778 Masonry: Materials, Properties and Performance, American Society for Testing and Materials, West Conshohocken. PA, USA.
12. Lourenco, P.B., Barros, J.A.O. (2000) "Size Effect Due to Out-of-Plane Bending" 12th International Brick and Block Masonry Conference, Madrid, Spain, posted on the internet.
13. CSA A371-04 (2004) "Masonry construction for buildings" Canadian Standards Association, Mississauga, ON, Canada
14. CSA S408-11 (2011) "Guidelines for development of limit states design standards" Canadian Standards Association, Mississauga, ON, Canada.
15. Laird, D.A., Drysdale, R.G., Stubbs, D.W., Sturgeon, G.R. (2005) "The New CSA S304.1-04 Design of Masonry Structures" 10th Canadian Masonry Symposium, Banff, AB, Canada. [Uses $\beta_T = 3.5$ and recommends $\phi = 0.60$ for masonry in compression.]

16. Kazemi, S., Mahoutian, H., Moosavi, H., Korany, Y. (2011) "Reliability Analysis of Masonry Members under Compression" ASCE/SEI Structures Congress 2011, Las Vegas, NV, USA. [Uses β_T = 3.5 and recommends ϕ = 0.65 for reinforced masonry in compression, and uses β_T = 3.8 and recommends ϕ = 0.50 for unreinforced masonry in compression.]
17. CSA A179-04 (2004) "Mortar and grout for masonry" Canadian Standards Association, Mississauga, ON, Canada.
18. CSA A3002-08 (2008) "Masonry and mortar cement" Canadian Standards Association, Mississauga, ON, Canada.
19. Kim, Y.S., Bennett, R.M. (2002) "Flexural Tension in Unreinforced Masonry: Evaluation of Current Specifications" TMS Journal, Vol.20, No.1, The Masonry Society, Longmont, CO, USA.
20. Bennett, R.M., Huston, E.T., McLean, D.I., Throop, D.B. (2011) "Allowable Stress Recalibration in the 2011 TMS 402 Code" 11th North American Conf., Minneapolis, MN, USA.
21. User's Guide – NBC 2010 Structural Commentaries (2010), National Research Council of Canada, Ottawa, ON, Canada.
22. Drysdale, R.G. (2012) "The Evolution of the Design and Construction of Masonry Buildings in Canada" 15th International Brick and Block Masonry Conference, Florianopolis, Brazil, posted on the internet.
23. Al-Menyawi, Y.M (2001) "Concrete Block Masonry to Resist Severe Winds" PhD. Thesis, Texas Tech University, Lubbock, TX, USA, posted on the internet.
24. Harris, B.H. (2010) "Investigation of the Lower Bound Flexural Strengths of Conventional Concrete and Clay Masonry" Master's Thesis, University of Texas at Arlington, Arlington, TX, USA, posted on internet.
25. Masonry Analysis Structural Systems (MASS) computer program, Canada Masonry Design Centre, Mississauga, ON, Canada.
26. National Climate Data and Information Archive, Environment Canada, Ottawa, ON, Canada, "Climate Normals" posted on the internet.

APPENDIX

Note: All calculations assume 15 MPa normal density hollow concrete block and type S mortar.

A1. Probability of failure: The ultimate limit states design target reliability index β_T for CSA S304.1 was set at 3.5 for masonry in compression [15]. For a normal distribution, 3.5 standard deviations represent a 1 in 5,000 failure rate over 50 years for walls designed for the strength required and subjected to the specified loads. Since most walls are stronger than necessary for practical reasons, the actual failure rate for all walls in compression would be lower.

No reliability analysis has been done for the flexural tensile strength values in CSA S304.1, but one [19] has been done for TMS 402. A total of over 200 laboratory wall tests that used hollow block were found in the literature. Four different mortars were considered separately, but wall thickness was not. A log-normal distribution was assumed. Reliability indices of 3.56 to 3.85 were obtained for the allowable flexural tensile strength values before they were increased by 33%. The effects of site curing and workmanship were not included in the analysis. It should also be noted that TMS 402 uses lower values for Masonry Cement and air-entrained PCL mortars.

A2. Gravity Load - comparing empirical and limit states design: Consider a 190 mm thick loadbearing interior wall h = 3800 mm and no openings. Axial load eccentricity e = t/6 = 32 mm.

•Empirical Design: 15 MPa hollow blocks based on net area is equivalent to 7.5 MPa based on gross area. Allowable load capacity is $P = f_{allow} A_g = 0.7x190 = 133$ kN/m. For D/(D+L)=0.75, equivalent factored load $P_f = 175$ kN/m.
•Limit States Design: Using Masonry Analysis Structural Systems (MASS) program [25], for $\beta_d = 0.7$, $P_f = 210$ kN/m > 175 (okay).
Conclusion: Walls designed with the empirical method meet the limit states design requirements for gravity loads.

A3. Wind Loads - comparing empirical and limit states design:
Exterior Walls: Consider a low-rise Category 2 building with a nonloadbearing exterior wall, h=3800 mm, and no openings. Building height is 11 m and importance factor is 1.0.
•Empirical Design:
t = h/20 = 3800/20 = 190 mm wall thickness
•Limit States: Maximum allowable hourly wind pressure $q = 0.55$ kN/m^2 for empirical design.
In rough terrain, $p = qC_e(C_gC_p) + qC_eC_{gi}C_{pi} = 0.55x0.7x1.6 + 0.55x0.7x2.0x0.45 = 0.96$ kN/m^2.
Check the 190 mm wall thickness –
$M_r = \phi\ (f_t + P_f/A_e)S_e = 0.60(0.40+0.9x1.9x(2.19x10^3)/72{,}400)x(4.59x10^6x10^{-6}) = 1.24$ kN-m/m.
Assuming base of wall rocks on face shell.
In rough terrain, $M_f = w_f\, h^2/8 - P_f\, e/2 = (1.4x0.96)x3.8^2/8 - (0.9x3.8x2.19)x(0.095-0.005)/2$
= 2.43 – 0.34 = 2.09 kN- m/m >> 1.24 kN-m/m (no good).
In open terrain, $C_e = 1.02$ and $M_f = (1.02/0.7)$ x 2.43 – 0.34 = 3.54 - 0.34 = 3.20 kN-m/m >> 1.24 kN-m/m (no good!). A high-rise or higher importance building would be even worse.
Shear Walls: Consider a 3 storey office building in open terrain with shear walls conforming to empirical requirements. Dead load per level is 2.2 kN/m^2. Floor live load plus partitions is 2.4+1.0 = 3.4 kN/m^2 and roof snow load is 1.1 kN/m^2. Height of shear walls above first floor is 11 m. There is no limit on size, so assume the building is 15 m x 40 m in plan with 8 m bays.
•Empirical Design: Two parallel walls placed at opposite ends of the building in both directions. Total length of walls required in each direction is 0.4x40 = 16 m. Assume two 4 m long walls each end.
Check gravity load - P = (4x11x2.19) + (2x30x5.6+30x3.3) = 531 kN < 0.7x190x4 = 532 kN.
•Limit States Design: Wind load $p = qC_e\,(C_gC_p) = 0.55x1.02x\,(1.43) = 0.80$ kN/m^2.
$V_{fb} = (1.4x0.80)$ x (11-1.8) x40 = 412 kN base shear.
Factored shear per wall $V_f = 103$ kN. Factored moment $M_f = 103x11/2 = 567$ kN-m.
Dead load is (4x11x2.19) + (3x30x2.2) = 294 kN. Factored is 0.9x294 = 256 kN
Check the 4 m long walls: $f_t = 567/170 - 265/256 = 2.28$ MPa >> 0.6x0.4=0.24 (wall is cracked).
e = 567/ 256 = 2.14 m > t/3 = 1.33 m (no good). It is worse with higher importance factor.
Conclusions: Importance factor should be used to limit use of empirical design (maximum $q = 0.55/I_w$); and exterior walls and shear walls designed with the empirical method do not meet the limit states design requirements for wind.

A4. Suggested Partition Loads - comparing empirical and limit states design: Consider a partition (interior nonloadbearing wall) made of 90 mm block and containing 25% door openings (75% wall remaining). Structural Commentaries (clause I-61) suggest a minimum 0.25 kPa unfactored pressure difference.
•Empirical Design:
h = 36t = 36x90 = 3240 mm or 3.24 m.

•Limit States Design:
Check the 3.24 m height between lateral supports -
$M_r = 0.75\phi_m (f_t + P_f /A_e)S_e = 0.75x0.60[(0.40+ 0.9x1.62x(1.35x10^3)/52000)]/1.25= 0.16$ kN-m/m.
$M_f = w_f h^2/8 - P_f e/2 = (1.4x0.25)x3.24^2/8 – (0.9x3.24x1.35)(0.045-0.005)/2 = 0.46 – 0.08$
= 0.38 kN-m/m >> 0.16 kN-m/m (no good)
Conclusion: Partitions designed with the empirical method do not meet the current limit states design requirements for the suggested partition load.

A5. Earthquake Loads - comparing empirical and limit states design:
Exterior Walls: Consider the exterior wall in item A3 above with brick veneer located at the top storey of a building. The maximum permitted seismic hazard index ($I_EF_aS_a(0.2)$) is just under 0.35. Ductility and overstrength do not appear to be considered in the formula for S_p. The out-of-plane seismic force is:
$V_p = 0.3(F_a S_a(0.2) I_E) S_p W_p = 0.3x0.34x1.2xW_p = 0.12W_p$ (12% of dead weight).
= 0.12x (2.19+1.77+0.25) = 0.51 kN/m^2.
$M_f= w_f h^2/8 - P_f e/2 = (1x0.51)x3.8^2/8 - (0.9x3.8x2.19)x (0.095-0.005)/2 = 0.92 – 0.34$
= 0.58 kN- m/m << 1.24 kN-m/m (okay).
Shear Walls: Consider the building in item A3 above. Assume I_E and F_a = 1. The maximum permitted seismic hazard index ($I_EF_aS_a(0.2)$) is just under 0.35. Therefore, $S_a(0.2)$ = 0.34. The in-plane seismic force is:
$V_{fb} = S(T_a) x M_v x I_E x W/(R_oR_d)$ where $T_a = 0.05(h_n)^{0.75} = 0.05(11)^{0.75} = 0.30$ s.
V_{fb} = 0.28 x 1 x 1 x W/ (1x1) = 0.28W (28% of dead weight).
W ≈ 3 x 40x15x (2.0+0.4) + (110x (11-1.8) x1.8) = 4320+1822 = 6142 kN.
V_{fb} = 0.28x6142 = 1720 kN base shear. M_f ≈ 1720x7.3=12,556 kN-m factored moment.
By comparison with item A3, the shear walls are greatly overloaded (no good!!).
Partitions: Consider the partition in item A4 above located at top storey of a building. The out-of-plane seismic force is:
V_p = 0.12x (1.35+0.25) = 0.19 kN/m^2.
$M_f= w_f h^2/8 - P_f e/2 = (1x0.19)x3.24^2/8 - (0.9x3.24x1.6)x (0.045-0.005)/2$
= 0.25 – 0.09 = 0.16 kN- m/m = 0.16 kN-m/m (okay).
Conclusion: Walls and partitions designed with the empirical method meet the current limit states design requirements for earthquake. Shear walls are greatly overloaded.

A6. Wind Loads on Cladding - comparing 1965 and 2010: Consider a Category 2 low-rise building in Toronto with importance factor I_w = 1.0 and building height = 11 m.
•In 1965 the design wind load for the cladding in all terrains for 1:10 return period was:
$p = q C_h (C_{pe}+C_{pi}) = 19x1x (0.9+0.2) = 20.9$ psf or 1.0 kN/m^2.
The equivalent factored wind load would be 1.5x1.0 = 1.50 kN/m^2.
•In 2010, the specified (unfactored) wind load for the cladding in rough terrain (built-up areas for at least a km in all directions) for 1:50 return period is:
$p = I_wqCe (C_gC_p) + I_wqC_eC_{gi}C_{pi} = 1x0.44x0.7x1.6 + 1x0.44x0.7x2.0x0.45 = 0.77$ kN/m^2.
In rough terrain the factored wind load would be 1.4x0.77 = 1.08 kN/m^2.
In open terrain, the factored wind load would be increased to (1.02/0.7)x1.08 = 1.57 kN/m^2.
Conclusion: Since 1965, the design wind loading in Toronto for cladding has been reduced by about 25% for low-rise buildings in rough terrain, but has been increased by about 5% for open terrain conditions.

A7. Earthquake Loading on Building - comparing 1965 and 2010: The first earthquake requirements in the NBCC were in 1941, but it was not until 1967 that the first city in Canada actually required that buildings be designed for earthquake.

Consider a 3 storey (9m high) apartment building in Toronto with unreinforced masonry on very dense soil and with importance factor of 1.0.

•In 1965 the design 1:500 earthquake base shear load was:

$V = RCIFSW = 1x1.25x1x1x0.021xW = 0.026W$, where $S = 0.25/(9+N) = 0.25/(9+3) = 0.021$

The equivalent factored base shear $V_f = 1.5x0.026W = 0.039W$ kN. Note that W did not include any snow allowance in 1965.

•In 2010 the factored 1:2500 earthquake base shear was:

$V_f = S(T_a) \times M_v \times I_E \times W/(R_oR_d)$ where $T_a = 0.05(h_n)^{0.75} = 0.05(9)^{0.75} = 0.26$ s.

$V_f = 0.21 \times 1 \times 1 \times W/ (1x1) = 0.21W$ kN.

Conclusion: Since 1965, the design earthquake base shear has increased by a factor of more than five for a low-rise unreinforced masonry building on very dense soil in Toronto. Unreinforced masonry is now assumed to have no overstrength or ductility.

A8. Wind Loads - maximum measured wind gust load in some Canadian cities: The NBCC designs buildings using the wind gust pressures, but the climatic data in the NBCC gives only the mean hourly wind pressures. These hourly pressures are required to be multiplied by the gust factor to get the design wind gust loading. The gust factor for cladding is 2.5. The measured maximum gust speeds for some Canadian cities [26] for the 50 years or so up to year 2000 were examined. Most major cities where empirical design could be permitted are shown below.

City	Specified Hourly Wind Pressure kPa	Design Gust Pressure kPa	Measured Max. Gust Speed (year) km/h	Measured Max. Gust Pressure kPa	Ratio of Measured Gust Press. / Design Gust Press.
Saskatoon	0.43	1.08	151 (1967)	1.14	1.06
Regina	0.49	1.23	153 (1962)	1.17	0.95
Edmonton	0.45	1.13	146 (1965)	1.06	0.94
Windsor	0.47	1.18	148 (1959)	1.09	0.92
London	0.47	1.18	148 (1992)	1.09	0.92
Toronto	0.44	1.10	135 (1956)	0.91	0.83
Sudbury	0.46	1.15	137 (1964)	0.94	0.82
Thunder Bay	0.39	0.98	122 (1960)	0.76	0.78
Hamilton	0.46	1.15	133 (1978)	0.88	0.74
Winnipeg	0.45	1.13	129 (1965)	0.83	0.73
Calgary	0.48	1.20	127 (1976)	0.80	0.67

Conclusion: Statistically, about 64% of locations would be expected to exceed the 1 in 50 year design gust pressure in any 50 year period. On that basis, the specified hourly wind pressure is a little too high at the present time. The load factor for wind was reduced to partially compensate.