V–10. Unbraced Masonry Walls During Construction

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ABSTRACT

Losses caused by wind damage to masonry walls during construction may be in excess of $500,000 per year currently in the U.S.A. A method is provided for determining the allowable heights for any thickness of brick and/or concrete masonry wall unbraced against wind during construction in any terrain in any part of the United States at any building height. Allowable spacing of wind braces for walls exceeding the allowable heights for unbraced walls is not considered.

Les dommages occasionnés par les dégâts du vent aux murs de maçonnerie lors de leur édification excèdent $500,000 par an actuellement aux U.S.A. Une méthode est fournie pour déterminer les hauteurs admissibles pour n'importe quelle épaisseur de mur de maçonnerie en briques ou en béton non soutenu contre le vent lors de sa construction sur n'importe quel terrain n'importe où dans les Etats-Unis, quelle que soit la hauteur du bâtiment. L'espacement admissible des armatures de soutien contre le vent pour les murs dépassant les hauteurs autorisées des murs sans soutien n'est pas pris en considération.

Verluste durch Windschaden während der Konstruktion an Mauerwänden belaufen sich in den Vereinigten Staaten zur Zeit auf mehr als $500,000 pro Jahr. Es wird hier eine Methode um die zulässige Höhe in bezug auf Tiefe des Mauersteins und/oder der Zementmauerwand zu bestimmen, die ungestützt gegen Wind während der Konstruktion in jeder Gegend in jedem Teil der Vereinigten Staaten für jede Gebäudehöhe verwandt werden kann, angeführt. Die empfohlenen Errichtungsabstände der Windstützen für Wände, welche die zugelassene Höhe für ungestützte Wände ;uberschreiten, wird hier nicht behandelt.

Perdite causàrono per il danno di vento ai murari durante costruzione può darsi che in eccesso di $500,000 per anno attualmente negli stati uniti. Una sistema è provvisto per determinato l'altezze consentite per qualche spessore di laterizio e/o murare di calcestruzzo allentato contro il vento durante costruzione in qualche terreno della parte degli stati uniti a qualche l'altezza da costruzzione. La spaziatura ammissibile di sostegni venti per i muri passante l'altezze consentite per i muri allentati non è meditato.

INTRODUCTION

During the 1946–57 period, the Factory Mutual System sustained 152 losses due to windstorm damage to masonry walls during construction[11]. Of the damaged walls 86% were unbraced. The height ranged from 5 ft (1.52 m) to 30 ft (9.14 m) and averaged 15.9 ft (4.85 m). The length ranged from 10 ft (3.05 m) to 500 ft (152.4 m) and averaged 136.8 ft (41.7 m). Concrete block walls were the most frequently damaged. Insurance losses in the Factory Mutual System on windstorm damage to unsupported masonry walls during construction averaged $220,000/year during the 1961 – 1970 decade[9]. Total current national annual loss due to wind damage to masonry under construction may be in excess of $500,000.

The requirements of building codes and recommended practices for wind bracing of masonry walls under construction are nonexistent or vague and inconsistent. The Brick Institute of America recommends that "Members of brick masonry in locations where they may be exposed to high winds during erection shall be adequately braced. . . ."[5]. Since all locations are exposed to high winds, this provision would require that all "engineered" brick masonry walls be braced during construction, but who determines what is adequate? The AIA General Conditions[3] leave all construction safety decisions to the contractor. Typically, a bricklaying foreman makes the decision, but he is given no engineering guidance as to when bracing is required or to its quantity or quality. If he is wrong, the determination of structural "adequacy" may be decided by a jury of housewives.

The American Concrete Institute recommends that "The height of unbraced concrete masonry walls should be limited in areas of high winds" and suggests allowable heights for unbraced walls ranging up to sixteen feet for walls of various thickness based on an assumed "peak wind velocity"[2]. Since the ACI design procedure provides for a factor of safety of one, under these conditions, presumably any factor of safety should be contained in the selection of the peak wind velocity, but no guidance is given as to how that peak should be determined. The ACI recommendations are silent on the strength and spacing of braces for walls exceeding the allowable heights for unbraced walls.

The *Uniform Building Code* of the International Conference of Building Officials requires bracing "wherever necessary." The *Basic Building Code* of the Building Officials and Code Administrators International requires that all walls be temporarily braced during erection. The *National Building Code* of the American Insurance Association requires that walls shall be "adequately braced" during erection. The *Southern Standard Building Code*[15] provides that unbraced masonry walls shall not be built higher than ten times their thickness, above which height they shall be

"adequately" braced. The *American Standard Building Code Requirements for Masonry,* American National Standards Institute (ANSI A41-1954), has the same requirement and cites as justification a 1943 article by Sweet[16], who considered only a 16.5 in. thick unbraced brick wall exposed to a wind velocity of 67 mph and provided no justification for that velocity. In any event, the ten to one ratio is consistent with the old rule of thumb, still in some antiquated building codes, that solid masonry walls should be laterally supported at intervals not exceeding twenty times their thickness. If the ten to one ratio is adequate in Miami, it should be forty to one in Los Angeles for the same resistance to overturning.

The Safety and Health Regulations for Construction [Title 29, Chapter XVII, Part 1926, Section 1926.700(a)] of the Occupational Safety and Health Administration (OSHA), U.S. Department of Labor, states that masonry work shall meet the applicable requirements of the American National Standard Safety Requirements for Concrete Construction and Masonry Work (ANSI A10.9-1970), Section 9.5 of which reads, "Masonry walls shall be temporarily shored and braced until the designed lateral strength is reached, to prevent collapse due to wind or other forces." Apparently, this requires that all masonry walls be braced until built into the supporting structure, and thereafter for a period of 28 days, which is inconsistent with the other ANSI Standard, A41-1954, cited above.

Some guidance on wind pressures is available from the American National Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures (ANSI 58.1-1972)[1], but these requirements are barely intelligible to practicing engineers much less bricklaying foremen, who can hardly be expected to solve the series of equations described below, much less understand why all brick walls must be braced, but some concrete block walls can be built up to 16 ft. with no bracing, when experience indicates that concrete masonry walls are more likely to be blown over than brick walls.

This paper presents rational derivations for the maximum allowable heights for unbraced brick and/or concrete masonry walls in any thickness constructed at any building height above the ground in any terrain in the United States and provides solutions in tabular form.

ACCIDENTAL PROBABILITY

In 1974 the chance of a U.S. citizen being accidently injured was one in 3.5. The chance of a U.S. construction worker being injured at work that year was one in 5.46. The chance of a United States resident being killed in an accident that year was one in 2,020. Rusch and Radswitz[12] suggest that avoidable risk of personal injury is annually accepted by daring people at one chance in 1,000, while careful people want the odds increases to one in 10,000. One chance annually in 20,000 is said to be unavoidable, i.e. an act of God.

During the summer of 1977, a survey was made to determine the opinion of masonry contractors regarding their desired level of confidence that a masonry wall will not be blown down during construction. There were 24 respondent contractors, eight from Michigan, six from Texas, four from Ohio, three from Illinois, and one each from Kansas, Indiana, and New York. No contractor said the reliability should be less than 95%. Five respondents said 99.8% or more. Twenty contractors, or 83.3% of the respondents, said 98% or more. The average response was 98.45%. Accordingly, the concensus of these contractors is that the chance of a masonry wall being blown down during construction should not exceed one in 64, or that no more than about 3 walls in 200 should blow down.

WIND VELOCITY OF GIVEN RECURRENCE

The ANSI A58.1-1972 method for determining *resultant design wind pressure* requires first that a *basic wind speed* be assumed. The Code states that for structures which have no human occupants or where there is negligible risk to human life, a basic wind speed having an annual twenty-five year mean recurrent interval may be used. Since bricklayers rarely work in high wind, unbraced masonry walls under construction probably meet those criteria.

Sacks[13] utilizing statistical methods developed by Gumbel[6], provides a method for estimating the wind velocity, having any probability of recurrence during any time interval in any location for which adequate weather records available. Although this method has been questioned[14], the procedure represents the present state of the art.

$$V_{30} = ay + b \quad (1)$$

V_{30} = basic wind speed at 30 ft (9.14 m) above the ground, having given probability of not being exceeded during given time interval in given location, mph

$$a = \sigma_{vm}/\sigma_N \quad (2)$$

where: σ_{vm} = standard deviation of maximum wind velocity from mean maximum wind velocity for the same month in each of N years, mph

σ_N = a sample size correction factor, dimensionless

$$\sigma_N \simeq 0.1421\ (4.423 + \ln N) \quad (3)$$

where: N = number of maximum wind velocities for a given month from available weather records, where $8 \leq N \leq 40$, years

$$\sigma_{vm} = \{(N-1)^{-1}[\Sigma V^2 - N^{-1}(\Sigma V)^2]\}^{0.5} \quad (4)$$

where: V = maximum wind velocity at 30 ft (9.15 m) above the ground for the same month in each of N years, (8)

When maximum monthly wind velocities are reported for heights other than 30 ft. (9.15 m), they may be adjusted to 30 ft. (9.15 m) by use of Eq. 13 i.e. $V = V_{fh} K_z^{-1}$, where:

V_{fh} = fastest mile for given height in given month, mph

From eq. 1:

$$b = V_m - a\,y_n \quad (5)$$

where: V_m = mean value of V, mph

$$V_m = (\Sigma V)/N \quad (6)$$

$$y_n \simeq 0.03581\ (11.568 + \ln N) \quad (7)$$

where: y_n = a sample size correction factor, dimensionless

$$y = -\ln(-\ln P_N) \quad (8)$$

where: P_N = probability of nonrecurrence of V, dimensionless

$$P_N = m(N+1)^{-1} \quad (9)$$

where: m = rank order of maximum wind velocity V in the array of wind velocities for the same month in each of N years

Values of P_N for any month may be determined from the published data[8] by the method given in Table 1[13]. The relationship between V_{30} and y is determined by the best straight line fit.

The frequency of occurence of the extreme wind speed in a given month is the reciprocal of the mean recurrence interval must be increased[13] such that $T' = [1-R^{1/T}]^{-1}$, probability of recurrence is 1/25, and the probability of nonrecurrence is 1-(1/25) or 0.96. The recurrence interval is a mean value. To insure a high level of confidence (R) that the predicted extreme wind in any one month in any one year during the interval T years is not exceeded, the interval must be increased[13] such that $T' = [1-R^{1/T}]^{-1}$, where R is the desired level of confidence and T′ is the equivalent recurrence interval. Accordingly, the probability of nonrecurrence becomes $1-(1/T')$ or $1-[1/(1-R^{1/T})^{-1}]$ or $R^{1/T}$.

Where the exposure period is less than one month, the probability of nonreccurence (P) is $(R^{1/T})^{D/d}$ or:

$$P = R^{D/Td} \quad (10)$$

Where D is the total number of days in the month, d is the number of days of exposure, d < D, and T is the recurrence interval in years.

Equation No. 1 may now be written:

$$V_{30} = V_m - (\sigma_{vm}/\sigma_N)\,[y_n + \ln(-\ln R^{D/Td})] \quad (11)$$

For example, consider the data given in Table 1 for the fastest mile wind speed (V_{fh}) observed at various heights at Fort Worth (Amon Carter), Texas, during the month of January for the years 1957–1973. V is the product of V_{fh} and K_z determined from Eq. 13 for each value of V_{fh}. The mean maximum wind velocity (V_m), the mean of all V values, is 31.65 mph (14.1 m/sec.) The standard deviation from the mean maximum monthly wind velocity (σ_{vm}) is 5.46 mph (2.4 m/sec). The number of time intervals (N) is 17. From Eq. 3, $\sigma_N = 1.0311$. From Eq. 7, y = 0.5157. The coefficient of determination (r^2) is 0.967 and the coefficient of correlation is 0.983 for the straight line plot of V vs. y. From Eq. 2, a = 4.494. From Eq. 5, b = 29.09. From Eq. 10, where R = 0.9, d = 7, D = 31, and T = 25, P = 0.9815. From Eq. 8, y = 3.98.

From Eq. 11 the basic wind velocity at 30 ft. (9.14m) above the ground (V_{30}) is 50 mph (22.4 m/s), during any one week of any January in Fort Worth, Texas. Table 2 shows the basic wind speed (V_{30}) for Fort Worth during each month for six wall exposure periods ranging from one day to one month.

WIND LOAD

The ANSI Code[1] provides that the *basic wind pressure* be determined as:

$$q_{30} = V_{30}^2/391 \quad (12)$$

where: q_{30} = basic wind pressure at 30 ft (9.14 m) above the ground, lb/sq ft.

The basic wind pressure is adjusted for local terrain and wall heights above the ground exceeding 30 ft (9.14 m) by use of a *velocity pressure coefficient,* which can be closely approximated from Eq. 13, which provides a good fit of Fig. A2 in Ref. 1.

$$\text{Log } K_z \cong 0.4523\,[1.138 + (\text{Log } h_z - 3.3)(2.544 - \ln n)]. \quad (13)$$

where K_z = velocity pressure coefficient, dimensionless

h_z = distance from top of wall to ground level, ft. Where $h_z < 30$ set $h_z = 30$.

n = terrain parameter, dimensionless. For centers of large cities or very rough, hilly terrain n = 3. For suburban areas, towns, city outskirts, wooded areas, or rolling terrain n = 4.5. For flat, open country, open flat coastal belts, or grass lands n = 7.

The basic wind pressure is further adusted for gusts by use of a *gust factor,* which is also a function of terrain and height above the ground. For parts and portions of buildings the gust factor can be closely approximated as:

$$G_p \cong 0.65 + 4.94\,(h_z/30)^{-1/n}\,n^{-0.9473} \quad (14)$$

where: G_p = gust factor, dimensionless.

The product of Eq. 12, 13, and 14 provides values for the effective velocity pressures for parts and portions of buildings having heights of 400 ft (121.92 m) or less, which differ from those given in Table 6 of Ref. 1 by three psf (143.64 Pa) or less. The probability of a difference of one psf (47.88 Pa) or less is 83%.

The basic wind pressure is further modified by the *maximum net pressure coefficient* for oblique wind on signs at ground level, which is as close as ANSI A58.1-1972 comes to freestanding masonry walls. Oblique wind produces maximum suction on the back side of the wall adjacent to the windward end. Suction at first increases and then decreases with distance from the windward end along the length of the wall. The net pressure coefficient therefore varies from a maximum at the windward end of the wall to a minimum at the leeward end. Eq. 15 is taken from Eq. 9 in Ref. 1 with a good fit of the data in Table 16 in Ref. 1 for the net pressure coefficients for signs at ground level (r = 0.9975).

$$C_{f\,max} = 1.528\,K\,(h/\ell)^{0.198} \quad (15)$$

where: $C_{f\,max}$ = maximum net pressure coefficient for oblique wind, dimensionless (Where $h/\ell > 40$ set $h/\ell = 40$. Where $h/\ell < 3$ set $h/\ell = 3$.)

K = wall proportion factor, dimensionless. Where $h < \ell$, K = 1. Where $h > \ell$, K = 1.15.

h = wall height above its base, ft
ℓ = overall wall length, ft

The minimum net pressure coefficient ($C_{f\,min}$) is 25% of $C_{f\,max}$, and the average net pressure coefficient (C_f) is 62.5% of $C_{f\,max}$ (1).

$$\bar{p} = q_{30}\,K_z G_p C_f \quad \text{.......... (16)}$$

where: $\bar{p}$ = average resultant design wind pressure, lb/sq ft

WALL DESIGN

Considering the wall as a vertical free-standing plate, structural integrity requires that internal stress be limited to allowable values and that the plate not over turn. Consider a plate having a horizontally distributed trapezoidal load (uniformly decreasing from the windward end to leeward end). The plate is pinned along the base (cracked) except at the leeward end where it is fixed (i.e. does not rotate). If that end does not rotate, the wall does not over turn. The total resisting moment about the wall base is equal to the product of the total dead weight of the wall and the distance to the center of gravity from the heavier side of the wall. This total resisting moment is applied with a triangular distribution, varying from zero at the leeward end to a maximum at the windward end. Resistance to overturning at the windward end is greater because of the assistance offered by adjacent increments of wall length, which are less heavily loaded. The leeward end has no such restraining moment, and the resisting moment is considered equal to zero at that point, but it does not rotate because the total wall overturning moment is equal to the total restraining moment.

Under these conditions a finite element analysis reveals that the maximum bending moment is about a horizontal axis at the windward end of the wall base and is 45% greater than if the wall were cantilevered from the base with a uniform load, $\bar{p}$.

$$M = 8.715\,h^2\bar{p} \quad \text{.......... (17)}$$

where M = maximum internal wall bending moment, in. −lb/ft

From structural mechanics it can be shown that:

$$f_{ta} \geqslant \frac{M}{S_m} - \frac{wh}{A} \quad \text{.......... (18)}$$

where:

f_{ta} = allowable flexural tensile stress for weakest material in section, psi
S_m = section modulus in transverse horizontal plane, in.^{3}/ft
w = wall weight, lb/sq ft
A = horizontal, transverse net section area, sq in/ft

Values for S_m, A and w are given in Ref. 4 for masonry walls of: 1. brick or solidly grouted hollow concrete masonry units (CMU); 2. hollow CMU; 3. composite brick and hollow CMU; 4. symmetrical brick masonry bonded hollow walls; and 5. symmetrical brick masonry bonded rib walls. These values are also given in Tables 4, 5, and 6 for three wall types and limited wall thicknesses.

ALLOWABLE STRESSES

Matthys and Grimm[10] determined the strength in flexure of a total of 550 masonry prisms exposed to weather at ages 0.33, 1, 2, 3, 4, 5, 6, 7, 14, 21 and 28 days for clay and concrete brick layed in M, S and N mortars of non-airentrained portland cement and lime with observed (inspected) and unobserved (noninspected) workmanship. The strength for all combinations of materials and workmanship fluctuated greatly with time, generally being lowest at age 8 hr and highest on the 4th day. The 28 day strength was significantly lower than the highest strength and closer to the mean strength. For determination of allowable working stress it seems reasonable to treat strength at ages less than 28 days as a random variable with time.

The mean ratio of mean strength with unobserved workmanship to that with observed workmanship was 0.53. The mean ratio of mean strength with concrete brick to that with clay brick was 0.61. The mean ratio of mean strength with type N mortar to that with type M mortar was 0.57. Prisms with type S mortar and observed workmanship were not tested. Under most building codes the allowable working stress in flexure for masonry built with type M and S mortars is the same and is based on 28 day strengths. In the absence of sufficient data to prove some other relationship at earlier ages, the allowable working stress in flexure during construction is assumed to be the same for masonry built of type M or S mortar.

In flexure masonry fails at the joint between units and mortar (ie bond failure). In a uniformly loaded beam the bending moment exceeds 88.9% of the maximum bending moment over one third of the span. In a masonry test prism of standard modular brick one third of the span may include three mortar joints. In an 8 ft (2.43 m) high test wall of the same materials, one third of the span may include three times as many joints. For this reason the strength of walls may be expected to be somewhat lower than that of prisms, when both are loaded uniformly. The maximum bending moment on a uniformly loaded beam occurs at or near one or two mortar joints, where as for two point loading the maximum bending moment is distributed over one third of the span or three joints in a test prisms and one half the span of 18 joints in test walls. For this reason two point loading indicates lower strength than that for uniform loading.

Since prisms are typically tested in flexure under two point loading and walls under uniform loading, the apparent difference between wall and prism strength may not be great. However, the test data on this issue are meager and, therefore, inconclusive. Concrete specimens of different size and shape gain strength at the same rate. Accordingly, no size adjustment factor was applied to prism test data in determining the allowable working stress for walls under construction. Since ANSI A58.1–1972 permits no increase in allowable stress under combined dead load and wind load only, no such increase was provided for walls under construction.

Hendry[7] provides a statistical method for determining structural reliability based on coefficients of variation in strength and loading and on an over-all factor of safety.

The mean coefficient of variation in all prisms samples tested was 0.20. The coefficient of variation in wind loading varies greatly with month as indicated by the data on Table 2. For that location the mean monthly coefficient of variation in mean monthly wind velocity is 0.22 and in wind pressure 0.31. Based on these variabilities in strength and loading, to provide the mason contractors' desired mean reliability of 98.45% against wall overturning during construction would require an over all factor of safety of 2.68.

Since masonry cement mortars typically have much higher air contents than those tested in this program, the allowable working stresses in flexure during construction for masonry built with masonry cement mortars may be significantly lower than those established for masonry built with nonairentrained portland cement-lime mortars. Establishment of such working stress for masonry built with masonry cement mortars must await completion of further testing programs.

The weighted mean modulus of rupture over the 28 day period for clay brick masonry prisms built with type M mortar and observed workmanship was 169 psi. Accordingly, the allowable stress for that combination of materials and workmanship was established at 63 psi. The ratios sited above for the several types of units, mortar and workmanship were applied to 63 psi to establish the allowable stresses given in tables 4, 5 and 6. Accordingly, the allowable stress in flexural tension for masonry walls during construction (f_{ta}) may be determined as follows:

$$f_{ta} = 63\, M_t\, W\, U \qquad (19)$$

M_t = mortar type coefficient. Set M_t = 1 for ASTM C270 types M or S mortar of non-airentrained portland cement and lime.
For type N mortar set $M_t = 0.57$

U = masonry unit type coefficient, dimensionless. For walls of clay brick set U = 1. For walls of concrete masonry set U = 0.61.

W = workmanship type coefficient, dimensionless. For observed (inspected) workmanship set W = 1. For unobserved (not inspected) workmanship set W = 0.53.

Since the wall is assumed to be cracked at its base, to prevent sliding the coefficient of friction at the wall base must equal or exceed the quotient of the maximum unit wind load and the maximum dead load unit stress at the base of the wall. Where Eq. 20 is not satisfied, the allowable height of the wall is zero, unless sliding is positively resisted perhaps by a shear cleat at the wall base.

$$\phi \geq \bar{p}/w \qquad (20)$$

where ϕ = the coefficient of static friction, dimensionless (with a factor of safety of two for walls without base flashing $\phi = 0.425$ and for walls with base flashing $\phi = 0.125$).

EXAMPLE

Consider a single wythe, nominal 8 in. wall, 50 ft long, built in Fort Worth, Texas, exposed (unbraced) for seven days during January, built without flashing of hollow CMU, layed in ASTM C270, type S, nonairentrained portland cement-lime mortar with uninspected workmanship. From Eq. 18 f_{ta} = 20 psi, from Table 4 S_m = 80 in.^{3}/ft (4,532 mm^3/mm), w = 39.5 lb/sq ft (122 kg/m^2), and A = 45 sq in./ft (2.12 mm^2/mm).

Given the top of the wall is less than 30 ft (9.15 m) above the ground (h_z = 30) in a suburban area (n = 4.5). Therefore, from Eq. 13 K_z = 0.45 and from Eq. 14 G_p = 1.84.

Assume the allowable height of the wall is 6 ft (1.82 m). From Eq. 15 C_f max = 1.9. Therefore C_f = 1.19.

From Table 1 V_m = 31.7, σ_{vm} = 5.5 and n = 17. Where R = 0.9, d = 7 and D = 31, V_{30} = 50. From Eq. 12 q_{30} = 6.39. Accordingly from Eq. 16 $\bar{p}$ = 6.3 lbs/ft^2. From Eq. 18 the stress of 19.4 psi (134 Pa) is less than the allowable of 20 psi (138 Pa) determined from Eq. 19. Where ϕ = 0.425, $\bar{p}$ = 6.3 and w = 39.5, Eq. 20 is satisfied, and sliding does not control the design. However, if the wall had base flashing, a shear cleat to prevent sliding would be necessary.

HANDBOOK ENGINEERING

When an analysis of monthly wind data for a particular city has been made and data arranged as in Table 2, the basic wind velocity for any exposure period during any month is available. Then enter Table 3 with the basic wind velocity, terrain condition and height of wall above the ground to determine the average resultant wind pressure. With that information and the wall type, i.e. hollow CMU, clay brick, or composite wall enter the appropriate Table 4, 5 or 6.

For example in Fort Worth for a wall exposed for seven days in January the basic wind velocity is found from Table 2 to be 50 mph (80.47 km/h). For a suburban area for a wall with top less than 30 ft (9.14 m) above the ground, the average resultant wind pressure at a velocity of 50 mph (80.47 km/h) is found from Table 3 to be 6 lb/sq ft (287 Pa). For a nominal 8 in. (203 mm) thick CMU wall in ASTM C 270, type S, nonairentrained portland cement-lime mortar, the allowable height of an unbraced wall is found from Table 4 to be 6.3 ft (1.89 m).

FUTURE RESEARCH NEEDS

Certain assumptions used in this paper need to be verified by further research and a professional concensus determined. An acceptable level of reliability should be rationally determined by examination of damage and injuries due to failure, e.g. the acceptable risk of failure should be higher for walls having small heights above their base than for high walls and for those built at lessor heights above the ground than for greater heights. Is a 25 year recurrent interval appropriate to wind velocities for the design of masonry walls under construction, and if so, what is the justification? In related work the author has determined basic wind velocities for various exposure periods for 13 cities in Texas, but clearly this needs to be done for all areas of the nation. The data are available, and the techniques for doing so are well documented. Use of such data could reduce the cost of most masonry construction without sacrifice of safety and would be used to rationalize

and standardize the several building code requirements for resistance of masonry walls to wind loads while under construction. The problem of determining the spacing of braces for walls exceeding the allowable height for unbraced walls will be treated in a subsequent paper.

CONCLUSION

The present vague and conflicting requirements of building codes regarding bracing for masonry walls under construction foster their disregard, which causes many avoidable accidents.

It is in the interest of construction engineers to have engineering problems solved by engineers rather than crafts foremen, arbitration panels, or juries of housewives. Determination of allowable heights for unbraced masonry walls during construction provides an opportunity to exercise that option. In his 1943 article, Sweet[16] said, "Building Codes should include provisions to guide the engineer or contractor during construction of brick walls so that wind storm damage would not result." A generation later the need is still evident.

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NOTATION

A = horizontal, transverse net section area, sq in./ft
C_{fmax} = maximum net pressure coeffieient for oblique wind, dimensionless
D = number of days in the month during which the wall is exposed (unbraced), days
G_p = gust factor, dimensionless
K= wall proportion factor, dimensionless. Where $h < \ell$, $K = 1$. Where $h > \ell$, $K = 1.5$
K_z = velocity pressure coefficient, dimensionless
M = maximum internal wall bending moment, in.-ltb/ft
N = number of maximum wind velocities for a given time interval used from available weather records, where $8 < N < 40$, years
P = frequency of nonreoccurence of V, dimensionless
R = level of confidence that V_{30} in any one month in T years will not be exceeded, dimensionless
S_m = section modules in transverse horizontal plane, in.^{3}/ft
T = recurrance interval, yrs.
V= maximum wind velocity at 30 ft above the ground for the same time interval in each of N years, $V = f(V_{fh})$, mph (see note b to Table 3)
V_{fh} = fastest mile in one minute for given height in given time interval, mph
V_m = mean value of V, mph
V = maximum wind velocity having probability P or not being exceeded by extreme wind during given time interval, mph
V_{30} = basic wind speed at 30 ft (9.15 m) above the ground, having given probability of not being exceeded during given time interval in given location, mph
d = number of days during which the wall is exposed (unbraced), days
f_{ta} = allowable flexural tensile stress for weakest material in section, psi
h = maximum allowable height above its support for unbraced wall subject to wind load, ft

h_z = distance from top of wall to ground level, ft. Where $h_z < 30$, set $h_z = 30$.
ℓ = overall wall length, ft
m = rank order of maximum wind velocity V in the array of wind velocities for the same time interval in each of N years.
n = terrain parameter, dimensionless.
$\bar{p}$ = resultant design wind pressure, lb/sq ft
q_{30} = basic wind pressure at 30 ft above the ground, lb/sq ft
t = actual wall thickness, in.
y_N = a sample size correction factor, dimensionless
σ_N = a sample size correction factor, dimensionless
σ_{v_m} = standard deviation of maximum wind velocity from mean maximum wind velocity for a given time interval during N such time intervals, mph.

TABLE 1—Maximum Wind Velocities For Month of January in Fort Worth (Amon Carter), Texas

Year	Fastest Mile, mph V_{fh}[(a) (c)]	Fastest Mile (mph) at 30 ft above ground[(b) (c)] V	Rank Order, m	Frequency, P_N (Eq. 9)	y (Eq. 8)
1972	25	25.9	1	0.0556	−1.0614
1971	26	26.9	2	0.1111	−0.7872
1968	26	26.9	3	0.1667	−0.5832
1961	38	27.3	4	0.2222	−0.4082
1959	39	28.0	5	0.2778	−0.2476
1960	40	28.7	6	0.3333	−0.0940
1963	40	28.7	7	0.3839	0.0571
1970	28	29.0	8	0.4444	0.2096
1965	29	30.0	9	0.5	0.3665
1958	42	30.2	10	0.5556	0.5314
1962	43	30.9	11	0.6111	0.7083
1967	31	32.1	12	0.6667	0.9027
1957	46	33.0	13	0.7222	1.1226
1973	35	36.2	14	0.7778	1.3811
1964	36	37.3	15	0.8333	1.7020
1969	41	42.4	16	0.8889	2.1389
1966	44	44.5	17	0.9444	2.8619

(a) Velocities at 85 ft (25.9 m) for years preceding 1964 and at 22 ft (6.7 m) for 1964 and subsequent years.
(b) From Eq. 13 where $h_z = 85$ and $n = 7$, $K_z = 1.393$, and where $h_z = 22$ and $n = 7$, $K_z = 0.9664$. Therefore, in Fort Worth for the years preceding 1964 the velocity at 30 ft (9.14 m) is (1/1.393) V_{fh} or 0.7179 V_{fh}, and for 1964 and succeeding years, V = (1/0.9664) V_{fh} or 1.035 V_{fh}.
(c) One mile/hr = 0.44704 m/sec.

TABLE 2—Basic Wind Velocity (V_{30}) For Six Exposure Periods in Fort Worth, Texas mph[(a)]

Month D	Jan 31	Feb 28	Mar 31	Apr 30	May 31	June 30	July 31	Aug 31	Sept 30	Oct 31	Nov 30	Dec 31
V_m	31.7	31.3	33.0	34.6	30.6	30.0	29.4	26.4	30.3	27.9	29.5	31.1
σ_{v_m}	5.5	5.4	5.0	10.0	6.4	5.5	7.4	9.1	8.1	5.2	4.1	8.1
100r	97	96	99	96	98	95	98	92	97	97	98	96
1 day	40	40	40	50	40	38	40	40	43	36	36	43
3 days	46	45	46	60	47	44	48	49	51	41	40	52
7 days	50	50	50	68	52	49	54	57	58	45	43	58
14 days	54	53	53	75	56	52	59	63	63	49	46	64
21 days	56	56	55	79	59	54	62	67	66	51	48	67
1 month	58	57	57	83	61	56	65	70	69	53	49	70

(a) 1 mph = 1.609344 Km/h

TABLE 3—Average Resultant Wind Pressure on Free Standing Masonry Wall Under Construction, $\bar{p}$, lb/sq ft (Wall length assumed to exceed wall height above its base)

Terrain	Wall height above ground (h_z)	Average Resultant Wind Pressure, $\bar{p}$								
		Basic wind speed, mph (V_{30})								
		20	30	40	50	60	70	80	90	100
Central of large city or very rough, hilly country n = 3	30 or less	1	1	2	4	6	8	10	13	15
	60	1	2	3	5	7	10	13	17	21
	90	1	2	4	6	9	12	16	20	25
	120	1	3	4	7	10	14	18	23	28
	150	1	3	5	8	11	15	20	25	31
Suburbs, towns, wooded area, rolling country (n = 4.5)	30 or less	1	2	4	6	9	13	16	21	26
	60	1	3	5	8	12	16	21	26	32
	90	1	3	6	9	13	18	24	30	37
	120	2	4	7	10	15	20	26	33	41
	150	2	4	7	11	16	22	28	36	44
Flat, open grasslands or coast (n = 7)	30 or less	2	4	7	12	17	23	30	37	46
	60	2	5	8	13	19	26	34	43	53
	90	2	5	9	14	21	28	37	46	57
	120	2	5	10	15	22	30	39	49	61
	150	3	6	10	16	23	31	41	51	63

TABLE 4—Allowable Height From Wall Base of Unbraced Masonry Walls of Lightweight Hollow Concrete Masonry Units During Construction, ft[a–b]

Mortar type ASTM C270	Wall Properties				Allowable Height, ft																	
	Nominal Thickness, in. (t)	Weight, lbs/sq ft[a] (w)	Section Modulus in^3/ft[c] (S_m)	Area, in^2/ft	Average Resultant Wind Pressure, lbs/sq ft (from Table 3)																	
					1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Inspected M or S mortar (f_{ta} = 38 psi)	4	30	21	17.8	11.8	7.9	6.2	5.3	4.7	4.3	3.9	3.6	3.4	3.2	3.1	2.9	2.8	2.7	2.6	2.5	2.4	2.4
	6	33.5	50	40.5	17.3	11.7	9.4	8.0	7.1	6.4	5.9	5.5	5.2	4.9	4.7	4.5	4.3	4.1	4.0	3.8	3.7	3.6
	8	39.5	80	45.0	23.1	15.4	12.2	10.4	9.2	8.3	7.7	7.1	6.7	6.3	6.0	5.7	5.5	5.3	5.1	4.9	4.8	4.6
	10	45	125	61.0	29.2	19.4	15.4	13.1	11.6	10.5	9.6	8.9	8.4	7.9	7.5	7.2	6.9	6.6	6.4	6.2	6.0	5.8
	12	52	190	70.0	38.0	24.8	19.5	16.6	14.6	13.2	12.1	11.2	10.5	9.9	9.4	9.0	8.6	8.3	8.0	7.7	7.5	7.2
Not Inspected M Mortar (f_{ta} = 20 psi)	4	30	21	17.8	9.3	6.0	4.7	4.0	3.5	3.2	2.9	2.7	2.6	2.4	2.3	2.2	2.1	2.0	1.9	1.9	1.8	1.8
	6	33.5	50	40.5	13.3	8.9	7.0	6.0	5.3	4.8	4.4	4.1	3.8	3.6	3.5	3.3	3.2	3.0	2.9	2.8	2.7	2.7
	8	39.5	80	45.0	18.2	11.8	9.3	7.9	6.9	6.2	5.7	5.3	5.0	4.7	4.5	4.3	4.1	3.9	3.8	3.6	3.5	3.4
	10	45	125	61.0	23.0	14.9	11.7	9.9	8.7	7.9	7.2	6.7	6.3	5.9	5.6	5.3	5.1	4.9	4.7	4.6	4.4	4.3
	12	52	190	70.0	30.5	19.4	15.1	12.7	11.1	10.0	9.1	8.5	7.9	7.5	7.1	6.7	6.4	6.2	6.0	5.8	5.6	5.4
Inspected N Mortar (f_{ta} = 22 psi)	4	30	21	17.8	9.6	6.3	4.9	4.2	3.7	3.3	3.1	2.8	2.7	2.5	2.4	2.3	2.2	2.1	2.0	2.0	1.9	1.8
	6	33.5	50	40.5	13.9	9.2	7.3	6.2	5.5	5.0	4.6	4.3	4.0	3.8	3.6	3.4	3.3	3.2	3.1	3.0	2.9	2.8
	8	39.5	80	45.0	18.8	12.3	9.7	8.2	7.2	6.5	6.0	5.6	5.2	4.9	4.7	4.5	4.3	4.1	3.9	3.8	3.7	3.6
	10	45	125	61.0	23.8	15.5	12.2	10.3	9.1	8.2	7.5	7.0	6.5	6.2	5.9	5.6	5.4	5.1	5.0	4.8	4.6	4.5
	12	52	190	70.0	31.4	20.1	15.6	13.2	11.5	10.4	9.5	8.8	8.3	7.8	7.4	7.0	6.7	6.5	6.2	6.0	5.8	5.6
Not Inspected N Mortar (f_{ta} = 12 psi)	4	30	21	17.8	7.8	5.0	3.9	3.2	2.8	2.6	2.3	2.2	2.0	1.9	1.8	1.7	1.7	1.6	1.5	1.5	1.4	1.4
	6	35.5	50	40.5	11.0	7.2	5.6	4.8	4.2	3.8	3.5	3.2	3.0	2.9	2.7	2.6	2.5	2.4	2.3	2.2	2.2	2.1
	8	39.5	80	45.0	15.3	9.7	7.5	6.4	5.6	5.0	4.7	4.2	4.0	3.7	3.6	3.4	3.2	3.1	3.0	2.9	2.8	2.7
	10	45	125	61.0	19.4	12.3	9.5	8.0	7.0	6.3	5.8	5.3	5.0	4.7	4.5	4.3	4.1	3.9	3.8	3.6	3.5	3.4
	12	52	190	70.0	26.2	16.2	12.4	10.4	9.0	8.1	7.4	6.8	6.4	6.0	5.7	5.4	5.2	4.9	4.8	4.6	4.4	4.3

NOTES: a. For heights to the left of the dotted line no shear cleat is needed at the wall base. For hieghts between dotted and solid lines no shear cleat is needed, if no base flashing is used, but a shear cleat is needed if base flashing is used. Heights to the right of the solid line require a shear cleat at wall base.

b. 1 in. = 25.4 cm, 1 lb/ft² = 4.88 kg/², 1 in³/ft = 53.76 mm³/mm, 1 in²/ft = 1.11 mm²/mm, 1 psi = 0.0703 kg/cm², and 1 ft = 0.3048m

c. Weight of concrete in two core, 8 in. x 8 in. x 16 in. (203 cm x 203 cm x 406) unit is 87 lb/cu ft (16.02 kg/m³)

d. Face shell bedded only

TABLE 5—Allowable Height From Wall Base of Unbraced Clay Brick Masonry Walls During Construction, ft[ab]

Mortar type ASTM C270 Portland Cement-lime	Work-manship (allowable flexural stress, psi)	Wall Properties: Nominal Thickness, in. (t)	Weight lbs/sq ft[3] (w)	Section Modulus in³/ft	Area in²/ft (A)	Allowable Height, ft — Average Resultant Wind Pressure, lb/sq ft (From Table 3): 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
M or S	inspected (f_{ta} = 63)	4	40	26	43.5	15.1	10.4	8.4	7.2	6.4	5.8	5.4	5.0	4.7	4.5	4.3	4.1	3.9	3.8	3.6	3.5	3.4	3.3
		6	60	63	67.5	24.8	16.8	13.4	11.5	10.2	9.3	8.5	8.0	7.5	7.1	6.7	6.4	6.2	5.9	5.7	5.5	5.4	5.2
		8	80	116	91.5	35.4	23.6	18.8	16.0	14.2	12.8	11.8	11.0	10.3	9.8	9.3	8.9	8.5	8.2	7.9	7.6	7.4	7.2
		12	120	270	139.5	59.5	38.6	30.3	25.7	22.6	20.4	18.7	17.4	16.3	15.4	14.6	13.9	13.3	12.8	12.3	11.9	11.5	11.2
	not inspected (f_{ta} = 33)	4	40	26	43.5	11.4	7.7	6.2	5.3	4.7	4.3	4.0	3.7	3.5	3.3	3.1	3.0	2.9	2.8	2.7	2.6	2.5	2.4
		6	60	63	67.5	19.0	12.6	10.1	8.6	7.6	6.9	6.3	5.9	5.5	5.2	5.0	4.7	4.5	4.4	4.2	4.1	3.9	3.8
		8	80	116	91.5	27.6	18.0	14.2	12.0	10.6	9.6	8.8	8.2	7.7	7.2	6.9	6.6	6.3	6.0	5.8	5.6	5.4	5.3
		12	120	270	139.5	48.0	30.2	23.4	19.7	17.2	15.5	14.1	13.1	12.2	11.5	10.9	10.4	10.0	9.6	9.2	8.9	8.6	8.3
N	inspected (f_{ta} = 36)	4	40	26	43.5	11.8	8.0	6.5	5.5	4.9	4.5	4.1	3.8	3.6	3.4	3.3	3.1	3.0	2.9	2.8	2.7	2.6	2.5
		6	60	63	67.5	19.7	13.1	10.4	8.9	7.9	7.1	6.6	6.1	5.7	5.4	5.2	4.9	4.7	4.5	4.4	4.2	4.1	4.0
		8	80	116	91.5	28.5	18.7	14.7	12.5	11.0	10.0	9.1	8.5	8.0	7.5	7.2	6.8	6.5	6.3	6.1	5.8	5.7	5.5
		12	120	270	139.5	49.3	31.2	24.2	20.4	17.8	16.0	14.7	13.6	12.7	12.0	11.4	10.8	10.3	9.9	9.6	9.2	8.9	8.6
	not inspected (f_{ta} = 19)	4	40	26	43.5	9.0	6.1	4.8	4.1	3.7	3.3	3.0	2.8	2.7	2.5	2.4	2.3	2.2	2.1	2.0	2.0	1.9	1.9
		6	60	63	67.5	15.4	10.0	7.9	6.7	5.9	5.3	4.9	4.6	4.3	4.0	3.8	3.7	3.5	3.4	3.2	3.1	3.0	2.9
		8	80	116	91.5	22.8	14.5	11.3	9.5	8.4	7.5	6.9	6.4	6.0	5.6	5.4	5.1	4.9	4.7	4.5	4.4	4.2	4.1
		12	120	270	139.5	41.0	25.1	19.1	15.9	13.8	12.4	11.3	10.4	9.7	9.1	8.6	8.2	7.8	7.5	7.2	7.0	6.7	6.5

NOTE: a. Heights to left of dotted line do not require a shear cleat at wall base. Heights to the right of dotted line require shear cleat at wall base, if base flashing is used, but no cleat is required, if no base flashing is used.

b. 1 in. = 25.4 cm, 1 lb/ft² = 4.88 kg/m², 1 in³/ft = 53.76 mm³/mm, 1 in²/ft = 1.11 mm²/mm, 1 psi = 0.0703 kg/cm², and 1 ft = 0.0348 m.

TABLE 6—Allowable Height From Wall Base of Unbraced Masonry Walls of Composite Clay Brick and Hollow Concrete Masonry Units During Construction[a,b,d]

Mortar Type ASTM C270	Wall Properties: Nominal Thickness, in. (t)	Wall Properties: Weight lbs/sq ft (w)	Wall Properties: Section Modulus (in.^{3} ft)c (S_m)	Wall Properties: Area in.^{2}/ft (A)	Allowable Height, ft — Average Resultant Wind Pressure, lbs/sq ft (From Table 3): 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Inspected	8	73	105	65.8	29.1	18.8	14.8	12.5	11.0	9.9	9.1	8.4	7.9	7.5	7.1	6.8	6.5	6.2	6.0	5.8	5.6	5.4
M or S Mortar	10	76	159	88.5	35.3	22.9	18.0	15.3	13.4	12.1	11.1	10.3	9.7	9.1	8.7	8.3	7.9	7.6	7.3	7.1	6.9	6.7
(f_{ta} = 38 psi)	12	82	235	93.0	46.0	29.3	22.9	19.3	16.9	15.2	13.9	12.9	12.1	11.4	10.8	10.3	9.8	9.4	9.1	8.8	8.5	8.2
Not inspected	8	73	105	65.8	23.6	14.8	11.5	9.6	8.4	7.5	6.9	6.4	6.0	5.6	5.3	5.1	4.8	4.7	4.5	4.3	4.2	4.0
M or S Mortar	10	76	159	88.5	28.5	18.0	13.9	11.7	10.3	9.2	8.4	7.8	7.3	6.9	6.5	6.2	5.9	5.7	5.5	5.3	5.1	5.0
(f_{ta} = 20 psi)	12	82	235	93.0	38.0	23.4	17.9	15.0	13.0	11.7	10.6	9.8	9.2	8.6	8.2	7.8	7.4	7.1	6.8	6.6	6.4	6.2
Not Inspected	8	73	105	65.8	24.3	15.3	11.9	10.0	8.7	7.9	7.2	6.7	6.2	5.9	5.6	5.3	5.1	4.9	4.7	4.5	4.4	4.2
N Mortar	10	76	159	88.5	29.3	18.6	14.5	12.2	10.7	9.6	8.8	8.1	7.6	7.2	6.8	6.5	6.2	5.9	5.7	5.5	5.3	5.2
(f_{ta} = 22 psi)	12	82	235	93.0	39.0	24.2	18.6	15.5	13.5	12.1	11.1	10.2	9.5	9.0	8.5	8.1	7.7	7.4	7.1	6.9	6.6	6.4
Not Inspected	8	73	105	65.8	20.4	12.5	9.5	7.9	6.9	6.1	5.6	5.2	4.8	4.5	4.3	4.1	3.9	3.7	3.6	3.5	3.3	3.2
N Mortar	10	76	159	88.5	24.6	15.1	11.5	9.6	8.4	7.5	6.8	6.3	5.9	5.5	5.2	5.0	4.8	4.6	4.4	4.2	4.1	3.9
(f_{ta} = 12 psi)	12	82	235	93.0	33.4	20.0	15.1	12.4	10.8	9.6	8.7	8.0	7.5	7.0	6.6	6.3	6.0	5.7	5.5	5.3	5.1	5.0

a. One nominal 4 in. brick wythe and remaining wall thickness hollow CMU.
b. Heights to left of dotted line do not require a shear cleat at wall base. Heights to right of dotted line require a shear cleat at wall base, if base flashing is used but no shear cleat is required, if no base flashing is used.
c. CMU face shell bedded only
d. 1 in. = 25.4 cm, 1 lb/ft^2 = 4.88 kg/m^2, 1 in^3/ft = 53.76 mm^3/mm, 1 in^2/ft = 1.11 mm^2/mm, 1 psi = 0.0703 kg/cm^2, and 1 ft = 0.3048 m.