

01 Apr 1975

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# WASHINGTON UNIVERSITY

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## SCHOOL OF ENGINEERING AND APPLIED SCIENCE DEPARTMENT OF CIVIL ENGINEERING

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### LOAD FACTORS FOR WIND AND SNOW LOADS FOR USE IN LOAD AND RESISTANCE FACTOR DESIGN CRITERIA

by

M. K. Ravindra

and

T. V. Galambos

Progress Report to the Advisory Committee  
of AISI Project 163 "Load Factor Design of Buildings"

Research Report No. 34

Structural Division

April 1975

Revised January 1976

COPIES  
62

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Research Report No. 34, Structural Division  
Civil Engineering Department  
Washington University  
St. Louis, Mo.

April 1975

Revised January 1976

This report presents results of research work  
sponsored by the American Iron and Steel Institute as  
AISI Project 163 "Load Factor Design of Steel Buildings".

ABSTRACT

This report presents the background and the derivations for the determination of the mean maximum loads and the corresponding load factors for wind and snow loading for use in Load and Resistance Factor Design Criteria for steel building structures.

## 1. INTRODUCTION

This report is concerned with load factors to be used with wind and snow loads in a design method named "Load and Resistance Factor Design" (L.R.F.D.). A previous report (1) has presented the general background of this design method as well as the basis for its development from a first-order probabilistic theory. In L.R.F.D. a structural design is deemed satisfactory if the computed internal forces, as determined by structural analysis for the assigned mean loads factored by appropriate load factors, are smaller than or equal to the factored nominal resistance of each structural element:

$$(\phi R_n)_k \geq \gamma_o \left[ \sum_{i=1}^n c_i \gamma_i Q_i \right]_j \quad (1)$$

where

- $(\phi R_n)_k$  = factored nominal resistance for limit state k
- $\phi$  = resistance factor for the appropriate limit state, accounting for the uncertainties of the resistance
- $R_n$  = nominal resistance for the appropriate limit state
- $\left[ \sum_{i=1}^n c_i \gamma_i Q_i \right]_j$  = factored internal load effect for load combination j
- $c$  = influence factor by which the factored load intensity  $\gamma Q$  is translated into a load effect (e.g., an internal force such as bending moment, shear force, axial force, torque, etc.) by structural analysis
- $\gamma$  = load factor accounting for the uncertainties of the load
- $Q$  = load intensity or load due to dead, live, wind, snow, etc., load

$\gamma_0$  = load factor accounting for the uncertainties of structural analysis and geometric and structural idealizations.

It was demonstrated in previous work (1) that the factors  $\phi$ ,  $\gamma_0$  and  $\gamma_i$  can be expressed in terms of the mean values and the coefficients of variation of the respective random variables, and that the factors are related to each other through a common term called the "safety index"  $\beta$ . This safety index is a measure of the reliability of the structural element and it is obtained through a process of calibration to an existing design criterion. In Ref. 1 such a calibration was performed for structural steel members designed according to the current (1976) AISC Specification, and a safety index of  $\beta = 3$  was chosen for the limit state of strength as being representative for structures designed according to the 1976 AISC Specification.

This present report is concerned with the right side of Eq. 1, i.e., with the load terms corresponding to wind and snow loads. The load terms to be determined are the load factors  $\gamma_i$  and the mean load intensities  $Q_i$  to be used with the L.R.F.D. criteria developed in Ref. 1. According to the load combinations used in these criteria it will be necessary to define load factors and mean load intensities for the maximum lifetime wind pressure, the maximum annual wind pressure, the maximum daily wind pressure, and the maximum annual and the maximum lifetime snow load intensity. The development will show how the basic wind velocity data given in the distribution maps in ANSI A58.1-1972 (2) are to be used in the L.R.F.D. criteria.

The appropriate load combinations involving snow and wind in the L.R.F.D. criteria can be enumerated as follows:

Maximum Lifetime Wind - Dead

Dead + Instantaneous Live + Maximum Lifetime Wind

Dead + Instantaneous Live + Maximum Lifetime Snow

Dead + Maximum Lifetime Live + Maximum Annual Snow

Dead + Ponding + Maximum Lifetime Snow

Dead + Instantaneous Live + Maximum Daily Wind + Maximum  
Lifetime Temperature

Dead + Maximum Annual Wind + Maximum Lifetime Snow.

The load factors  $\gamma$  are determined according to the formula (1)

$$\gamma = 1 + 0.55 \beta V = 1 + 1.65 V \quad (2)$$

where  $\beta = 3$  and  $V$  is the coefficient of variation of the appropriate load type.

The idea underlying this list of load combinations is that it is very unlikely that two types of load effects will simultaneously reach their maximum lifetime value. Thus each combination includes the dead load, which is always present, one of the other loads at its maximum lifetime value, and the other loads at their instantaneous, annual or daily values, as appropriate. Such a scheme of combining the loads has essentially the same effect as the use of the 0.75 multiplier used in most current specifications to modify the full nominal code loads in the case of simultaneity of occurrence. The scheme proposed herein is more reasonable in that it is possible to utilize the fact that the statistics for the different time intervals may be different.

## 2. STATISTICAL PARAMETERS OF WIND LOADS

In the following, the statistical aspects of wind loads on structures are discussed. The important random variables characterizing the wind load are the maximum wind speed in the service life of the structure, the annual

maximum wind speed and the daily maximum wind speed. The statistical parameters of the lifetime maximum wind speed are derived using the data on the annual maximum wind speed.

### 2.1 Wind Load Determination According to Current (1976) Practice

The usual practice for calculating the wind loads on buildings and other structures is to use specified minimum design pressures varied according to geographical location and height zone above the ground. The dynamic action of wind on structures is indirectly accounted for in specifying these design pressures. The 1972 version of A58.1 of the American National Standards Institute (2) and the National Building Code of Canada (3) have included procedures which explicitly recognize the dynamic effects of wind.

The procedure for calculating the wind loads according to ANSI A58.1 - 1972 is:

1. A mean recurrence interval is selected depending on the intended operational usage, anticipated life of the structure, degree of wind sensitivity and risk to human life and property in case of failure. A 50 year mean recurrence interval is recommended by ANSI for the design of permanent structures. For more important and/or more wind-sensitive structures a 100 year mean recurrence interval is recommended. For structures having no human occupants or where there is negligible risk to human life, the 25 year mean recurrence interval may be used.

2. A basic wind speed is selected from the wind maps of the United States, corresponding to the mean recurrence interval (Figs. 1, 2 and 1A in Ref. 2). This speed refers to the annual extreme fastest mile velocity. This fastest-mile wind speed is measured by recording the time required for a mile of air to pass a fixed point by means of an anemometer which makes an electrical contact with the passage of each mile of air.

The wind speeds were generally observed at airport or open country locations where the exponent in the power law relating velocity and height is  $1/7$ . Since the observations may have been taken at different elevations, they were adjusted to the standard 30 foot elevation by means of this power law prior to the preparation of the wind maps.

The extreme speeds of each year govern the annual maximum wind load on a structure so only these speeds were considered in determining the design values. The annual extreme series for each station, called the series of annual extreme miles of wind, were then fitted with a frequency distribution to determine the design values associated with various probabilities of being exceeded. The associated mean recurrence intervals are the reciprocals of these probabilities, and each gives the average time interval in years between the occurrence of all winds exceeding the design value. Fisher and Tippett Type II extreme value distributions have been employed in this analysis.

3. Wind speeds corresponding to the specified mean recurrence intervals selected from the wind maps may be converted to velocity pressures using the formula

$$q_{30} = \frac{1}{2} \rho V_{30}^2 \quad (3)$$

where  $q_{30}$  is the basic wind pressure,  $V_{30}$  is the basic wind speed and  $\rho$  is the air density. For standard air (0.07651 lb per cu. ft.) and velocity in miles per hour, velocity pressure  $q_{30}$  in pounds per square foot is given by  $q_{30} = 0.00256 V_{30}^2$ .

The effective velocity pressures of wind at various heights above the ground are computed using the formula:

$$q_z = K_z G q_{30} \quad (4)$$

where  $q_z$  is the effective velocity pressure in psf at height  $z$  in ft.,  $K_z$  is a velocity pressure coefficient which depends upon the type of exposure and height above ground and  $G$  is a gust factor which depends upon the response characteristics of the structure.

4. The effective velocity pressure varies with height and exposure. Three categories of exposure are considered: (A) centers of large cities and very rough, hilly terrain; (B) suburban areas, towns, city outskirts, wooded areas and rolling terrain; and (C) flat, open country, open flat coastal belts, and grassland. For convenience, values of  $q_z$  for ordinary buildings and structures ( $q_z = q_F$ ) and for parts and portions ( $q_z = q_P$ ) have been tabulated in ANSI-A58.1-1972 for a range of speeds as functions of height and exposure.

The effective velocity pressures given by ANSI A58.1 take into account the dynamic response to gusts of ordinary buildings and structures in a direction parallel to the wind. They do not provide for the effects of vortex shedding or instability due to galloping or flutter. ANSI recommends a detailed analysis for obtaining the effective velocity pressure where a dynamic approach to the action of wind gusts is required.

5. The resultant wind pressure  $p$  acting on an element of an enclosed structure is

$$p = C_p q_z - C_{pi} q_M \quad (5)$$

where  $q_z$  equals  $q_F$  or  $q_P$  whichever is appropriate,  $C_p$  is the external pressure coefficient,  $C_{pi}$  is the internal pressure coefficient and  $q_M$  is the corresponding effective velocity pressure (given in Tables 5 and 6 in ANSI A58.1). The pressure coefficients define the pressure acting normally at local positions on the surface of a building and hence are dependent on the external shape and orientation of the building with respect to wind.

In general the wind pressure acting on a structure or structural element can be written as

$$W = C_p q_z = C_p K_z G (0.00256 V_{30}^2) \quad (6)$$

where  $C_p$ ,  $K_z$ ,  $G$  and  $V_{30}$  are all random variables. The mean wind pressure  $W_m$  and the coefficient of variation of the wind pressure  $V_w$  can be expressed as functions of the corresponding values of the components:

$$W_m \cong [ (C_p)_m (K_z)_m (G)_m ] [ 0.00256 (V_{30})_m^2 ] \quad (7)$$

and

$$V_w \cong \sqrt{V_{C_p}^2 + V_{K_z}^2 + V_G^2 + 4V_{V_{30}}^2} \quad (8)$$

## 2.2 Lifetime Maximum Wind Velocity

Based on wind velocity data from 141 open country stations, which were dispersed over the continental U. S., Thom (4) has suggested that the maximum annual wind velocity follows a Type II. Extreme Value distribution. He has shown that the shape parameter  $K$  of this distribution is essentially constant for all stations ( $K = 9.0$ ). Following is a derivation of the statistical parameters of the maximum lifetime wind speed using the probability model proposed by Thom.

The lifetime ( $n$ -years) maximum wind speed  $Y$  is the maximum of  $n$  annual maximum wind speeds,  $X_1, X_2, \dots, X_n$ . Then the probability

$$\begin{aligned} F_Y(y) &= P[Y \leq y] \\ &= P[\text{all } n \text{ of the } X_i \leq y] \end{aligned}$$

The annual maximum wind speeds  $X_i$  may be treated as statistically independent.

$$\begin{aligned} F_Y(y) &= P[X_1 \leq y] P[X_2 \leq y] \dots P[X_n \leq y] \\ &= F_{X_1}(y) F_{X_2}(y) \dots F_{X_n}(y) \end{aligned} \quad (9)$$

The annual maximum wind speed distribution is assumed to be constant in time (i.e.,  $X_i$  are identically distributed with the cumulative distribution function  $F_X(x)$ ). Therefore,

$$F_Y(y) = [F_X(y)]^n \quad (10)$$

Based on the Type II. Extreme Value probabilistic distribution Thom developed maps for the United States, giving the maximum annual wind velocities for mean recurrence intervals of 25, 50 and 100 years. The mean  $X_m$  and the coefficient of variation  $V_X$  of the annual maximum wind velocity are obtained using the following expressions:

$$X_m = 0.70 X_{50} \quad (11)$$

$$V_X = \sqrt{\frac{\Gamma(1 - \frac{2}{K})}{\Gamma^2(1 - \frac{1}{K})} - 1} \quad (12)$$

where  $X_{50}$  is the "50 year" annual maximum wind velocity (as given in the ANSI A58.1-1972 wind velocity map for a 50 year recurrence interval - Fig. 1 in Ref. 2). With  $K = 9$  and using a table of gamma functions,  $V_X$  is calculated by Eq. 12 to be approximately 0.16.

From Eq. 10

$$F_Y(y) = [F_X(y)]^n = \exp \left[ -n \left( \frac{u}{y} \right)^K \right] \quad (13)$$

Writing  $u' = u n^{1/K}$ ,

$$F_Y(y) = \exp \left[ - (u'/y)^K \right] \quad (14)$$

Therefore  $Y$ , the maximum annual wind speed, is also distributed according to the Type II. Extreme Value probability distribution with the parameters of mean

$$Y_m = u' \Gamma\left(1 - \frac{1}{K}\right) = X_m n^{1/K} \quad (15)$$

and coefficient of variation

$$V_Y = V_X = 0.16 \quad (16)$$

Here  $n$  is the lifetime of the structure, in years,  $Y_m$  is the mean maximum lifetime wind velocity,  $X_m$  is the mean maximum annual wind velocity (equal to 0.70 times the "n-year" wind velocity from the n-year ANSI wind distribution maps) and  $K = 9$ . For a 50 year life  $n = 50$  and  $V_m = 1.54 X_m = 1.08 X_{50}$ . The following table gives the relevant statistical values for a lifetime of 50 years.

ANSI 50 year

Basic Wind Velocity, $X_{50}$	60 mph	70 mph	80 mph	90 mph
Mean Maximum Annual				
Wind Velocity, $X_m = 0.7 X_{50}$	42 mph	49 mph	56 mph	63 mph
Mean Maximum Lifetime				
Wind Velocity, $Y_m = X_m 50^{1/9}$	65 mph	76 mph	86 mph	97 mph

Similar tables can be constructed for the 25 and 100 year life.

### 2.3 Wind Pressure

The mean wind pressure  $W_m$  and the coefficient of variation  $V_W$  are given by Eqs. 7 and 8. In the following the statistics of the component parameters  $K_z$ ,  $G$  and  $C_p$  are estimated.

The velocity pressure coefficient  $K_z$  depends on the type of exposure and on the height above ground at which the wind pressure is required. Empirical observations have led to the delineation of three exposure categories (Type A, B and C in ANSI A58.1, as described earlier in this report), and for each the variation with height is defined by an exponent in a power law relationship. The variation of  $K_z$  with height and exposure type is given in Fig. A2 of ANSI-A58.1-1972 and the resulting wind pressures have been tabulated in the same document as Tables 5 and 6 (Table 5 for  $q_F$ ,

the velocity pressure on the whole structure, and Table 6 for  $q_p$  for parts and portions of the structure) for the three exposure types. Since the velocity pressure coefficient  $K_z$  is used to cover a broad spectrum of exposure conditions, its use in calculating the wind pressure will result in some uncertainty in the prediction, characterized by the coefficient of variation  $V_{K_z}$ . As the relevant information to calculate  $V_{K_z}$  is not available, it will be assumed that  $V_{K_z} = 0.10$  for the purposes of this report.

The reliability of the gust factor  $G$  has been studied by Vickery (5) and more recently by Ellingwood and Ang (6). The gust factor, as noted earlier, depends on wind characteristics, terrain and building characteristics such as natural frequency, damping, geometry and mode shape. However, the variation in  $G$  is limited to a narrow range. Ellingwood and Ang (6) have shown that the mean gust factor is insensitive to variations in the natural frequency and the critical damping ratio. This result is significant because these quantities are often estimated from empirical expressions. The implication is that refined estimates of the dynamic characteristics of the system will not improve the mean gust factor estimation. Vickery (5) has demonstrated that the mean gust factor is insensitive to the mode shape, hence the actual mode shape is not important. Formulas and graphs for the determination of the gust factor are presented in Sec. A.6.3.4.1 of ANSI-A58.1-1972, and these have been used in the development of the pressure tables (Tables 5 and 6 in ANSI-A58.1-1972). It will be here assumed that the gust factors implied in these tables are taken to be the mean values for ordinary steel buildings. Based on the work of Vickery (5) a coefficient of variation  $V_G = 0.12$  is estimated.

The pressure coefficients,  $C_p$ , are the non-dimensional ratios of wind-induced pressures on a building to the dynamic pressure (velocity

pressure) of the wind speed that would be measured at the top of the building in the undisturbed air stream. Pressures on the surfaces of structures vary considerably with the shape, wind direction and the profile of the wind velocity. Pressure coefficients are usually determined from wind tunnel experiments on small-scale building models. It is essential in most cases that these pressures be measured in a wind tunnel in which the correct velocity profile is simulated. The pressure coefficients are all time-averaged values and usually represent spatial averages. In view of all these assumptions and simplifications, there will be uncertainty in the prediction of  $C_p$  values. Here, it is assumed that the  $C_p$  values used in the development of the pressure tables in ANSI-A58.1-1972 are mean values and that the coefficient of variation  $V_{C_p}$  is 0.10.

The mean maximum lifetime wind pressure,  $W_{L_m}$ , the coefficient of variation,  $V_{W_L}$ , and the load factor  $\gamma_{W_L}$ , for use in the L.R.F.D. criteria, are determined as follows:

$$W_{L_m} = \left( \frac{\text{Mean Maximum Lifetime Wind Velocity}}{\text{50-Year ANSI Basic Wind Velocity}} \right)^2 \text{ ANSI 50-Year Wind Pressure} \quad (17)$$

For a life  $n = 50$  years

$$(W_{L_m})_{50} = 1.17 \text{ (ANSI 50-Year Wind Pressure)} \quad (18)$$

For a life  $n = 100$  years

$$(W_{L_m})_{100} = 1.36 \text{ (ANSI 50-Year Wind Pressure)} \quad (19)$$

For a life  $n = 25$  years

$$(W_{L_m})_{25} = 1.00 \text{ (ANSI 50-Year Wind Pressure)} \quad (20)$$

The coefficient of variation  $V_{W_L}$  is determined from Eq. 8 as

$$V_{W_L} = \sqrt{V_{C_P}^2 + V_{K_z}^2 + V_G^2 + 4 V_{V_{30}}^2} = \sqrt{0.1^2 + 0.1^2 + 0.12^2 + 4 \times 0.16^2} = 0.37$$

where the individual coefficients of variation were taken from the previously presented estimates. It should be noted that  $V_{30} = V_X$  in the previous section, and that this term predominates in Eq. 8. Should the other three coefficients of variation be much larger, for example

$V_{C_P} = V_{K_z} = V_G = 0.15$ ,  $V_{W_L}$  would become equal to 0.41, resulting in a change in the load factor  $\gamma$  only in the second decimal.

The load factor  $\gamma_{W_L}$  for the mean maximum lifetime wind pressure is, from Eq. 2, equal to

$$\gamma_{W_L} = 1 + 0.55 \beta V_{W_L} = 1 + 0.55 \times 3 \times 0.37 = 1.61 \cong 1.6$$

The mean maximum annual wind pressure,  $W_{A_m}$ , the coefficient of variation,  $V_{W_A}$ , and the load factor  $\gamma_{W_A}$  are determined as follows:

$$W_{A_m} = \left( \frac{\text{Mean Maximum Annual Wind Velocity}}{50\text{-Year ANSI Basic Wind Velocity}} \right)^2 \text{ ANSI 50-Year Wind Pressure} \quad (21)$$

$$W_{A_m} = 0.49 \text{ (ANSI 50-Year Wind Pressure)} \quad (22)$$

The coefficient of variation  $V_{W_A} = 0.37$  and  $\gamma_{W_A} = 1.6$ , as for the maximum lifetime wind pressure.

The mean maximum daily wind pressure,  $W_{D_m}$ , the coefficient of variation,  $V_{W_D}$ , and the load factor,  $\gamma_{W_D}$ , and the load factor,  $\gamma_{W_A}$ , are determined as follows:

$$W_{D_m} = \left( \frac{\text{Mean Maximum Daily Wind Velocity}}{50\text{-Year ANSI Basic Wind Velocity}} \right)^2 \text{ ANSI 50-Year Wind Pressure} \quad (23)$$

$$V_{W_D} = \sqrt{0.1^2 + 0.1^2 + 0.12^2 + 4 V_{V_D}^2} = \sqrt{0.0344 + 4 V_{V_D}^2} \quad (24)$$

Unfortunately no map is provided in ANSI-A58.1-1972 for the statistics of the daily maximum wind velocity. An analysis of 13 locations in the continental US is given in Table 1 for a period of one year. This table lists the location; the mean fastest mile daily wind speed in mph ( $v_{30Dm}$ ); the corresponding 50 year ANSI wind speed for the same location from the map in Fig. 1 of ANSI-58.1-1972 ( $V_{ANSI}$ ); the multiplication factor

$$\left( \frac{v_{30Dm}}{V_{ANSI}} \right)^2$$

by which ANSI 50-year wind pressure is multiplied to obtain the mean wind load intensity from Eq. 23; the coefficient of variation of the daily wind ( $V_{V_D}$ ); the coefficient of variation of the daily wind pressure ( $V_{W_D}$ ), determined by Eq. 24; and the load factor ( $\gamma_{W_D}$ ) for the daily wind effect from Eq. 2. The load factor is based on  $\alpha = 0.55$  and  $\beta = 3.0$ , where  $\beta$  is the safety index.

In view of the similarities of the results from the various cities it is recommended that the design for daily wind in LRFD be based on the following rounded off values of the mean daily maximum wind intensity

$$0.07 \times \text{ANSI 50 yr wind pressure} \quad (25)$$

and the load factor  $\gamma_{W_D} = 2.3$ .

Following is an example calculation to illustrate the determination of the factored design wind load intensities for an ordinary structure with type B exposure where the wind load is desired at an elevation of 200 ft.

From Fig. 1 of ANSI-A58.1-1972, the basic ANSI 50-Year wind velocity is estimated as 70 mph. The effective velocity pressure from Table 5 in

ANSI-A58.1-1972 is  $q_F = 18$  psf for a 70 mph wind and a height of 200 ft.

$$\gamma_{W_L} (W_{Lm})_{50} = 1.6 \times 1.17 \times 18 = 34 \text{ psf}$$

$$\gamma_{W_L} (W_{Lm})_{100} = 1.6 \times 1.36 \times 18 = 39 \text{ psf}$$

$$\gamma_{W_L} (W_{Lm})_{25} = 1.6 \times 1.00 \times 18 = 29 \text{ psf}$$

$$\gamma_{W_A} (W_{Am}) = 1.6 \times 0.49 \times 18 = 14 \text{ psf}$$

$$\gamma_{W_D} (W_{Dm}) = 2.3 \times 0.07 \times 18 = 3 \text{ psf}$$

#### 2.4 Load Effects Due to Wind in the Structure

In the previous section it was shown that the load factors to be used with mean wind pressures are equal to 1.6 for the maximum lifetime and the maximum annual wind, and 2.3 for the maximum daily wind. The development of these factors was based on the wind pressure, and the question of the effect of this wind pressure on the magnitude of the forces in the member to be designed has not been considered.

When considering this question of the actual load effects due to wind in a structural component one has to consider the following:

1) In the case of a serviceability limit state the whole building, including the structure itself and all the non-structural cladding components, is intact and there exists a considerable amount of sharing of the applied wind pressure. The wind pressure resistance is shared by (1) the structural elements which are intended to carry all the wind-induced forces, (2) the structural elements which are present but have not been figured to help in the wind load resisting tasks (e.g., a "simple" connection in a braced frame is still able to resist some wind induced moments) and (3) by the "cladding" elements such as the walls,

partitions, slabs, stair-wells, etc.

2) At the ultimate limit state it may well be that some of the cladding elements have already been lost or at least damaged, and here, it must be assumed that the designed structure itself resists the major share of the wind pressure. However, the mechanism of failure is not a purely static one, and in some way or other it must involve the dynamic properties of the building and the ductility of the structure. While failure of buildings under wind forces is but imperfectly understood, there are some parallels to the failure of steel structures under severe earthquake motions where strength, ductility and dynamic properties all play an important part.

3) Another aspect to be considered is that the nature of the wind pressures may not be as assumed in the previous analysis: e.g., it might be possible to model the lifetime wind by another distribution (Type I distribution was used by Allen in Ref. 7), the statistics of the height, roughness, gust and shape factors might be inaccurate (Ref. 7), and there might be a reduction of the wind pressure due to directionality of the wind (Refs. 7 and 8).

Unfortunately present state of research is not in any way ready yet to give definitive answers to the questions raised above. This is especially so in the case of the ultimate behavior of structures under wind, where it is difficult to visualize realistically the failure mechanism. In the absence of definite answers it is necessary to look elsewhere for a temporary solution, namely, to present satisfactory design practice.

The AISC Specification, in Sec. 1.5.6, permits an increase of 33% of the allowable stresses for any load combination involving wind. If only

wind induced forces are present, for example, in a tension brace in a diagonally braced frame, the AISC Specification requires the following net area:

$$(A_n)_{\text{AISC}} = \frac{P_w}{0.6 F_y (4/3)} \quad (26)$$

The corresponding area required by LRFD is

$$(A_n)_{\text{LRFD}} = \frac{1.1 \times 1.6 \times 1.17 P_w}{\phi F_y} = \frac{1.1 \times 1.6 \times 1.17 P_w}{0.88 F_y} \quad (27)$$

where 1.6 is the previously determined wind load factor  $\gamma_w$ , 1.1 is the analysis factor  $\gamma_o$  (Ref. 1),  $\phi$  is the resistance factor ( $\phi = 0.88$ , Ref. 9), and 1.17 is the multiplier which translates the ANSI wind pressure, entering this example through the code-specified wind force  $P_w$ , into the mean maximum 50 yr. lifetime wind pressure. If one divides Eq. 27 by Eq. 26, one finds that the brace area required by LRFD is 1.87 times the area required by AISC. Similar discrepancies are seen to exist for beams and columns, as shown by the upper curves in Figs. 1 and 3 (for the derivations see the Appendix). While the wind pressure statistics, on which the load factor  $\gamma_w = 1.6$  is based, are not by any means without inaccuracies, the wind velocity statistics are quite reliable, agreeing with similar data from Canada and Europe (Refs. 7 and 8). The wind load factor is as large as it is because the square of twice the coefficient of variation of the wind velocity must be used (Eq. 8). It is, therefore, of not too great an advantage to improve the statistical basis for  $\gamma_w$ , because there is not enough to be gained so as to make up the almost 100% difference between the AISC and LRFD designs.

Is then, the AISC wind design procedure unsafe? It appears to have served satisfactorily for steel structures for quite a number of years

already, and so it can be assumed to be an adequate basis for design. If then neither the statistical wind pressure data nor current practice are at fault, it must be assumed that the modeling of wind resistance by a static structural skeleton is inadequate. As pointed out previously, present research is unable to provide rational answers, and as a temporary expedient it is suggested that the wind pressure be modified by a multiplier  $F < 1.0$  to bring the LRFD wind designs in line with current practice. The appendix gives a calibration procedure for beams and columns, and the results are given in Figs. 1 and 3, where ratios of LRFD-to-AISC section requirements are plotted against the wind pressure-to-dead load ratios for  $F = 1.0, 0.75, 0.6$  and  $0.5$ . The reduction factor  $F = 0.6$  appears to give a satisfactory ratio between the LRFD and the AISC requirements for the types of structural elements considered. In order to be somewhat on the conservative side, however, and to account for situations not covered in the calibration, it is suggested that the mean wind pressures to be used in LRFD be multiplied by a reduction factor  $F = 0.75$  until further research permits a more rational method dealing with wind load effects in structural elements.

In view of the arguments presented above it is recommended that the mean maximum wind pressures as determined in the previous section, be multiplied by 0.75 for use in LRFD criteria. However, this modification is not to be used in the case where overturning is considered.

### 3. STATISTICAL PARAMETERS OF SNOW LOADS

The statistical aspects of snow loads on structures are discussed in this section. The parameters of snow loading are derived using climatological data. It is recognized in the study of load combinations that the important random variables characterizing the snow load are the maximum snow load in the service life of the structure and the annual maximum snow load. The statistical parameters of the lifetime maximum snow load are derived using the data on the annual maximum snow load. The load factor to be applied on the mean lifetime maximum snow load and on the mean maximum annual snow load are calculated.

#### 3.1 Snow Load Determination According to Current (1976) Practice

The current version of the American National Standard (1) - A58.1-1972 (2) on minimum design loads in buildings and other structures has given the following procedure to calculate the snow loads acting on the structures:

1. A mean recurrence interval is selected depending on the intended operational usage and risk to human life and property in case of failure. A 50-year mean recurrence interval is recommended for use for all permanent structures except those that present an unusually high degree of hazard to life and property in the event of failure. In the latter case, a 100-year mean recurrence interval is recommended. For structures having no human occupants or where there is negligible risk to human life, a 25-year mean recurrence interval may be used.

2. A basic snow load is selected using Figs. 3, 4 or A7 in Ref. 2 corresponding to the mean recurrence interval. These figures show the isolines of ground snow load for portions of the United States.

3. The minimum snow load for the design of ordinary and multiple series of roofs is determined by multiplying the basic snow load by an appropriate snow load coefficient. The basic snow load coefficient,  $C_s$ , is taken as 0.8 and is varied to reflect differences in types and slopes of roofs and location (e.g. shielding and valleys).

### 3.2 Lifetime Maximum Snow Load

The snow load  $q$  acting on a structure is a random variable; it is a function of the ground snow load, wind speed and direction, geometry of the structure and the temperature gradient between the inside of the structure and the outside. Isyumov (10) has investigated the influences of these variables on the snow load acting on a structure. However, current design practice is to model the roof snow load  $S$  as a snow load coefficient  $C_s$  times the ground snow load  $q$ . i.e.,

$$S = C_s q \quad (28)$$

where both  $C_s$  and  $q$  are random variables. The statistics of the ground snow load  $q$  are obtained from meteorological data. Information on  $C_s$  is obtained by observations relating the roof snow load to the ground snow load.

From Eq. 28, the mean,  $S_m$ , and the coefficient of variation,  $V_s$ , of roof snow load are calculated as:

$$S_m = (C_s)_m q_m \quad (29)$$

$$\text{and } V_s \approx \sqrt{V_{C_s}^2 + V_q^2} \quad (30)$$

The statistical parameters of the lifetime maximum snow load are derived using the data on the annual maximum snow load.

Thom (11) has presented meteorological data on the annual maximum ground snow loads in different parts of the United States. Figs. 5 and 6, reproduced from Thom's paper, give the contours of mean and standard deviation of the logarithms of water equivalent of ground snow. He has observed that the maximum annual snow load follows a lognormal probability density function:

$$f_X(x) = \frac{1}{x \sigma_{\ln X} \sqrt{2\pi}} \cdot \left\{ \exp \left\{ -\frac{1}{2} \left[ \frac{\ln \left( \frac{x}{\tilde{X}_m} \right)^2}{\sigma_{\ln X}^2} \right] \right\} \right\} \quad (31)$$

where  $X$  is the maximum annual snow load,  $\tilde{X}_m$  is the median of the random variable  $X$ , expressed in terms of  $(\ln X)_m$  as

$$\tilde{X}_m = \exp [ (\ln X)_m ] \quad (32)$$

The term  $(\ln X)_m$  is the mean of the natural logarithm of the maximum annual snow load from Thom's map (Fig. 5) and  $\sigma_{\ln X}$  is the standard deviation of  $\ln X$  from the map in Fig. 6. The probability density function  $f_Y(y)$  of the lifetime maximum snow load  $Y$  can be obtained from Eq. 31 and Eq. 10. As the integrations involving  $f_X(x)$  from Eq. 31 cannot be performed in closed form, the mean and the coefficient of variation of the maximum lifetime snow loads were calculated by Monte Carlo simulation using Eqs. 31 and 10. Table 2 shows these values for ten stations selected to reflect the geographical variations of snow load in the United States. This table lists for each station the values  $(\ln X)_m$  and  $\sigma_{\ln X}$  (i.e., the mean of the logarithm of the maximum annual snow load and the standard deviation of  $\ln X$ , respectively, as obtained from Figs. 5 and 6), and the mean maximum lifetime snow load  $Y_m$  and the coefficient of variation of the maximum lifetime snow

load  $V_Y$ , as determined by the Monte Carlo simulation. The latter coefficient of variation is the value  $V_q$ , to be used in Eq. 30.

Table 2 also gives the statistics of the maximum annual snow load, i.e.,  $X_m$ , the mean and  $V_X$  the coefficient of variation. For the assumed lognormal distribution  $X_m$  and  $V_X$  can be computed from the data given in Figs. 5 and 6 by the relationships

$$X_m = \exp \left[ (\ln X)_m + \frac{1}{2} (\sigma_{\ln X})^2 \right] \quad (33)$$

and

$$V_X \cong \sqrt{\exp (\sigma_{\ln X})^2 - 1} \quad (34)$$

An approximate value of the mean maximum lifetime snow load intensity can be obtained from the expression

$$Y_m = X_m (1 + K V_X) \quad (35)$$

where  $X_m$  and  $V_X$  are determined from Eqs. 33 and 34, respectively, and  $K = 3.70$ . The actual value of  $K$ , as determined by using  $Y_m$  from the Monte Carlo simulation into Eq. 35, is tabulated also in Table 2. This  $K$  varies from 3.1 to 4.1, and  $K = 3.7$  is the average value.

For a given location the designer would look up  $(\ln X)_m$  and  $\sigma_{\ln X}$  from Fig. 5 and 6, compute  $X_m$ ,  $V_X$  and  $Y_m$  from Eqs. 33, 34 and 35, and then determine the mean maximum annual snow load intensity by

$$q_{Am} = \left( \frac{62.4}{12} \right) X_m \quad \text{in psf} \quad (36)$$

and the mean lifetime maximum snow load intensity by

$$q_{Lm} = \left( \frac{62.4}{12} \right) Y_m \quad \text{in psf} \quad (37)$$

for use in Eq. 29 as appropriate. The ratio 62.4/12 performs the transformation from inches of  $H_2O$  to the usual psf units. In order to permit a more rapid calculation of the mean maximum snow load intensities, values of  $q_{Am}$  and  $q_{Lm}$  are tabulated in Table 3 for at least one location for each of the states in the continental US.

It should be pointed out that these values of  $q$  apply only insofar as the charts given by Thom (Figs. 5 and 6) are valid. Local conditions in valleys of mountainous regions will require special treatment.

### 3.3 Evaluation of the Snow Load Factor $\gamma_s$

The load factor  $\gamma_s$  to be applied to the mean snow load  $S_m$  (Eq. 29) is determined by Eq. 2 with the coefficient of variation  $V_s$  from Eq. 30. The roof snow load is calculated by multiplying the mean ground snow load  $q_m$  by a coefficient  $C_s$ . This coefficient depends on the wind speed and direction, the geometry of the structure and the temperature gradient between the inside of the structure and the outside. Although some of these factors have been studied expressing all these influences by one factor is at best uncertain. ANSI-A58.1-1972 specifies a basic snow load coefficient  $C_s = 0.8$ , which is then modified for different types of roofs, slopes and locations. Here it will be assumed that  $C_s$  determined according to ANSI-A58.1-1972 is a mean value having an assumed coefficient of variation of  $V_{C_s} = 0.15$ . This means that the basic snow load coefficient  $C_s = 0.8$  lies between 0.56 and 1.04 with a probability of approximately 95 percent.

The load factor  $\gamma_s$  to be applied to the mean maximum snow loads is

$$\gamma_s = 1 + 0.55 \beta V_s \quad (38)$$

where

$$V_S = \sqrt{V_{C_s}^2 + V_q^2} = \sqrt{0.15^2 + V_q^2} \quad (39)$$

Using  $V_q = V_\gamma$  from Table 2 for the maximum lifetime snow load one obtains  $\gamma$  varying from 1.43 to 1.97, from which the average of 1.7 is recommended for use. Thus

$$\gamma_{SL} = 1.7 \quad (40)$$

Similarly, using  $V_q = V_X$  from Table 2 for the maximum annual snow load, a variation of  $\gamma$  from 1.7 to 2.6, from which the average of 2.3 is recommended for use, giving

$$\gamma_{SA} = 2.3 \quad (41)$$

#### 4. SUMMARY

This report has developed methods for determining mean wind and snow load intensities and the corresponding load factors for use with the Load and Resistance Factor Design Criteria presented in Ref. 1.

The wind load determination for ordinary steel structures involves the use of the recommended wind velocity pressure intensities given in Tables 5 and 6 of "Building Code Requirements for Minimum Design Loads in Buildings and Other Structures" (ANSI-A58.1-1972). The mean maximum wind loads are determined by obtaining the value of the velocity pressure  $q_{ANSI}$  for the whole structure,  $q_F$ , from Table 5, or for part of the structure,  $q_p$ , from Table 6 of ANSI-A58.1-1972, as appropriate, for the type exposure (A, B or C), the height above ground for which the wind load is required, and for the 50 year wind velocity obtained from Fig. 1 of ANSI-A58.1-1972. The mean maximum lifetime wind pressure for use in

the L.R.F.D. criteria is then determined as follows:

$$\begin{aligned} 50 \text{ yr. life:} & \quad W_{Lm} = 1.17 q_{ANSI} \\ 100 \text{ yr. life:} & \quad W_{Lm} = 1.36 q_{ANSI} \\ 25 \text{ yr. life:} & \quad W_{Lm} = 1.00 q_{ANSI} \end{aligned}$$

The load factor corresponding to each of these mean wind load intensities is  $\gamma_{W_L} = 1.6$ .

The mean maximum annual wind pressure is determined by

$$W_{Am} = 0.49 q_{ANSI} \quad \text{and} \quad \gamma_{WA} = 1.6$$

The mean maximum daily wind pressure is calculated by the formula

$$W_{Dm} = 0.07 q_{ANSI} \quad \text{and} \quad \gamma_{WD} = 2.3$$

In case of structures for which the ANSI velocity pressure tables do not apply, the procedure outlined in this report may be used to determine mean loads from the velocity pressures calculated by the detailed methods provided in the Appendix of ANSI-A58.1-1972.

The mean maximum wind pressures obtained above, are to be multiplied by 0.75 to account for the translation of wind pressure on the structure to wind load effects on the structural component. This factor is not to be applied when overturning is considered.

The snow load determination involves the use of data from Figs. 5 and 6 of this report for ordinary structures not located in special snow regions.

The mean maximum snow load intensity is determined by the formula

$$S_m = C_s q_m$$

where  $C_s$  is the roof snow load coefficient and  $q_m$  is either the mean maximum annual ( $q_{Am}$ ) or lifetime ( $q_{Lm}$ ) ground snow load intensity, as appropriate. According to Sec. 7 of ANSI-A58.1-1972  $C_s = 0.8$  shall be used unless a modification to account for other than ordinary roof conditions is required (see Sec. 7.2.1 of ANSI-A58.1-1972 for the details of this modification). The mean maximum annual ground snow intensity is determined by the formula

$$q_{Am} = \frac{62.4}{12} \left\{ \exp \left[ (\ln X)_m + \frac{1}{2} (\sigma_{\ln X})^2 \right] \right\}$$

and the mean maximum lifetime ground snow intensity is equal to

$$q_{Lm} = q_{Am} \left( 1 + 3.70 \sqrt{\exp (\sigma_{\ln X})^2 - 1} \right)$$

where  $(\ln X)_m$  is the mean of the logarithm of the water equivalent of the ground snow (obtained for any location in the US from Fig. 5) and  $\sigma_{\ln X}$  is the standard deviation (from Fig. 6). The corresponding load factors are  $\gamma_{S_A} = 2.3$  for the annual snow load and  $\gamma_{S_L} = 1.7$  for the lifetime snow load.

Values of  $q_{Lm}$  and  $q_{Am}$  are given in Table 3 for various locations in the US for a close enough spacing so that the ground snow load intensity can be directly obtained. It should be pointed out again that the procedure does not account for special snow regions.

##### 5. ACKNOWLEDGEMENTS

The work in this report was sponsored by the American Iron and Steel Institute as AISI Project 163 "Load Factor Design of Buildings". The advice and the discussion of the Project Advisory Committee is gratefully acknowledged. Members of this group are Messrs. Viest (Chairman), Hansell (engineering supervisor), Beedle, Cornell, Gaylord

Gilligan, Hooper, Milek, Pinkham and Winter. The manuscript was typed by Mrs. Bletch whose patience in retyping various drafts of this report is appreciated.

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## 7. NOMENCLATURE

$C_p$	- External wind pressure coefficient
$C_{pi}$	- Internal wind pressure coefficient
$C_s$	- Roof snow load coefficient
$c$	- Influence factor translating load intensity into load effect
$f$	- Probability density function
$F$	- Cumulative distribution function
$G$	- Gust factor
$(\ln X)_m$	- Mean of logarithm of ground snow, from Fig. 1
$K$	- Coefficient in statistical calculations
$K_z$	- Velocity pressure coefficient for wind load
$n$	- Lifetime, in years
$P$	- Probability
$p$	- Effective wind pressure
$Q$	- Load effect
$q$	- Load intensity
$q_{Am}$	- Mean maximum annual ground snow load intensity
$q_{ANSI}$	- ANSI 50-year wind load intensity
$q_F$	- Wind load intensity on whole structure
$q_{Lm}$	- Mean maximum lifetime ground snow load intensity
$q_p$	- Wind load intensity on part of structure
$q_{30}$	- Wind load intensity at 30 ft.
$R_n$	- Nominal resistance
$S$	- Snow load intensity
$S_m$	- Mean snow load intensity

- V - Coefficient of variation, subscripts denoting the appropriate variable
- $W_{Am}$  - Mean maximum annual wind load intensity
- $W_{Dm}$  - Mean maximum daily wind load intensity
- $W_{Lm}$  - Mean maximum lifetime wind load intensity
- X - Maximum annual wind velocity or ground snow as a random variable
- Y - Maximum lifetime wind velocity or ground snow as a random variable
- $\beta$  - Safety index
- $\gamma$  - Load factor, subscripts denoting the appropriate load type
- $\sigma_{\ln X}$  - Standard deviation of annual maximum snow intensity from Fig. 2

TABLE 1: Maximum Daily Wind Statistics for 1974

Location	$v_{30Dm}$ mph	$v_{ANSI}$ mph	$\left(\frac{v_{30Dm}}{v_{ANSI}}\right)^2$	$V_{v_D}$	$V_{w_D}$	$Y_{w_D}$
Boston	21	90	0.05	0.32	0.67	2.1
Denver	19	80	0.06	0.58	0.78	2.3
Minneapolis	18	75	0.06	0.33	0.69	2.1
Chicago	18	80	0.05	0.30	0.63	2.0
St. Louis	18	70	0.07	0.37	0.76	2.3
Kansas City	18	70	0.06	0.39	0.80	2.3
Salt Lake City	18	80	0.05	0.39	0.80	2.3
Washington, D.C.	17	75	0.05	0.36	0.74	2.2
Dallas	17	70	0.06	0.35	0.72	2.2
Atlanta	17	80	0.04	0.38	0.78	2.3
Pittsburgh	16	70	0.05	0.33	0.69	2.1
Seattle	16	80	0.04	0.37	0.76	2.3
New York City	14	80	0.03	0.32	0.67	2.1

TABLE 2. STATISTICAL PARAMETERS OF ANNUAL MAXIMUM (X) AND LIFETIME MAXIMUM (y) WATER EQUIVALENT  
(INCHES OF WATER) GROUND SNOW

Station	Billings Montana	Duluth Minn.	DesMoines Iowa	Chicago Ill.	Kansas City, Mo.	St.Louis Mo.	Indianapolis Indiana	Detroit Mich.	Albany N.Y.	Caribou Maine
$(\ln X)_m^*$	0.00	1.25	0.00	-0.10	-0.30	-0.60	-0.50	-0.25	-0.55	1.50
$\sigma_{\ln X}^*$	0.75	0.40	0.80	0.70	0.80	0.90	0.75	0.55	0.55	0.60
$Y_m(\text{in. H}_2\text{O})^{**}$	5.78	8.73	6.53	4.63	4.84	4.59	3.50	2.78	6.18	18.00
$V_Y = V_q^{**}$	0.45	0.21	0.49	0.41	0.49	0.57	0.45	0.31	0.31	0.34
K	3.92	3.11	3.93	3.75	3.94	4.10	3.88	3.48	3.49	3.56
$X_m(\text{in. H}_2\text{O})^\#$	1.32	3.78	1.38	1.16	1.02	0.82	0.80	0.91	2.02	5.37
$V_X^\#$	0.87	0.42	0.95	0.80	0.95	1.12	0.87	0.59	0.59	0.66
$(Y_m)_{\text{approx}}^{\#\#}$ in. H <sub>2</sub> O	5.57	9.65	6.23	4.59	4.61	4.22	3.38	2.90	6.43	18.48
psf	29	50	32	24	24	22	18	15	34	96

\* obtained from Figs. 1 and 2

\*\* for 50 yr. life by Monte Carlo simulation

# from Eqs. 30 and 31

## from Eq. 32 with K = 3.7

TABLE 3. MEAN SNOW LOAD INTENSITIES FOR VARIOUS U.S. CITIES  
FOR USE IN L.R.F.D. CRITERIA

City	State	$\sigma_{\ln X}$	$(\ln X)_m$	$q_{Am}$ (psf)	$q_{Lm}$ (psf)
Birmingham	, Alabama	0.99	-1.4	2	12
Tucson	, Arizona	.90	-1.5	2	9
Phoenix	, Arizona	.85	-1.0	3	13
Flagstaff	, Arizona	.40	- .5	3	9
Little Rock	, Arkansas	.84	-1.4	2	9
San Francisco	, California	.80	-1.0	3	12
Los Angeles	, California	.86	-1.0	3	14
Denver	, Colorado	.60	- .5	4	13
Grand Junction	, Colorado	.85	- .7	4	18
Hartford	, Connecticut	.70	.0	7	26
Dover	, Delaware	.90	- .5	5	24
Atlanta	, Georgia	0.98	-1.2	3	14
Boise	, Idaho	.85	-1.0	3	13
Pocatello	, Idaho	.35	- .5	3	8
Chicago	, Illinois	.70	- .1	6	24
Springfield	, Illinois	.82	- .4	5	23
South Bend	, Indiana	.70	.0	7	26
Indianapolis	, Indiana	.75	- .5	4	18
Dubuque	, Iowa	.82	.1	8	37
Des Moines	, Iowa	.80	- .0	7	32
Kansas City	, Kansas	.80	- .3	5	24
Wichita	, Kansas	.60	- .5	3	13
Louisville	, Kentucky	.60	- .8	4	10

City	State	$\sigma_{\ln X}$	$(\ln X)_m$	$q_{Am}$ (psf)	$q_{Lm}$ (psf)
Paducah	, Kentucky	.92	-1.0	3	15
New Orleans	, Louisiana	1.0	-2.0	1	7
Shreveport	, Louisiana	.86	-1.7	1	5
Augusta	, Maine	.60	1.3	23	78
Baltimore	, Maryland	.90	- .5	5	24
Boston	, Massachusetts	.60	.0	6	21
Marquette	, Michigan	.40	1.5	25	64
Detroit	, Michigan	.55	- .25	5	15
Minneapolis	, Minnesota	.75	.5	11	48
Duluth	, Minnesota	.40	1.2	20	50
Jackson	, Mississippi	0.95	-1.5	1	10
St. Louis	, Missouri	.90	- .6	4	22
Great Falls	, Montana	.35	.1	6	14
Billings	, Montana	.75	.0	4	29
North Platte	, Nebraska	.40	- .3	6	11
Lincoln	, Nebraska	.82	.2	9	41
Winnemucca	, Nevada	.50	-1.0	3	6
Las Vegas	, Nevada	.50	-0.6	2	10
Concord	, New Hampshire	.55	0.8	13	43
Trenton	, New Jersey	.80	.0	7	32
Raton	, New Mexico	.8	-1.0	3	12
Albuquerque	, New Mexico	.8	-1.3	1	9
Las Cruces	, New Mexico	.9	-1.5	2	9
Albany	, New York	.55	- .55	11	34
New York	, New York	.80	.0	7	32
Raleigh	, North Carolina	.9	-1.0	3	14

City	State	$\sigma_{\ln X}$	$(\ln X)_m$	$q_{Am}$ (psf)	$q_{Lm}$ (psf)
Wilmington	, North Carolina	0.95	-1.2	2	13
Bismarck	, North Dakota	.4	.2	9	24
Fargo	, North Dakota	.7	.1	7	29
Cleveland	, Ohio	.45	.0	6	16
Columbus	, Ohio	.5	-.5	4	11
Cincinnati	, Ohio	.5	-.7	3	9
Oklahoma City	, Oklahoma	.6	-0.9	3	9
Tulsa	, Oklahoma	.7	-1.0	2	10
Blue Mountains	, Oregon	.8	.0	7	32
Eugena	, Oregon	.7	-1.0	2	10
Portland	, Oregon	.7	-.3	5	19
Pittsburgh	, Pennsylvania	.5	-.2	5	14
Harrisburg	, Pennsylvania	.7	-.1	6	24
Philadelphia	, Pennsylvania	.8	-.3	5	24
Providence	, Rhode Island	.7	.0	7	26
Columbia	, South Carolina	0.95	-1.2	2	13
Rapid City	, South Dakota	.5	.0	6	18
Sioux Falls	, South Dakota	.8	.2	9	39
Memphis	, Tennessee	.92	-1.4	2	10
Knoxville	, Tennessee	.88	-0.8	3	17
Amarillo	, Texas	.7	-0.9	3	11
Forth Worth	, Texas	.8	-1.5	2	7
Austin	, Texas	.9	-2.0	1	5
Salt Lake City	, Utah	.25	-.3	4	8
Lake Powell Area	, Utah	.4	-0.8	3	6
Montpelier	, Vermont	.6	1.0	17	58

City	State	$\sigma_{\ln X}$	$(\ln X)_m$	$q_{Am}$ (psf)	$q_{Lm}$ (psf)
Richmond	, Virginia	.85	- .6	4	20
Seattle	, Washington	.6	- .8	3	10
Spokane	, Washington	.6	.0	6	21
Charleston	, West Virginia	.7	- .8	3	12
Green Bay	, Wisconsin	.6	.4	9	32
Madison	, Wisconsin	.8	.0	7	32
Worland	, Wyoming	.6	.0	6	21
Cheyenne	, Wyoming	.35	- .3	4	10

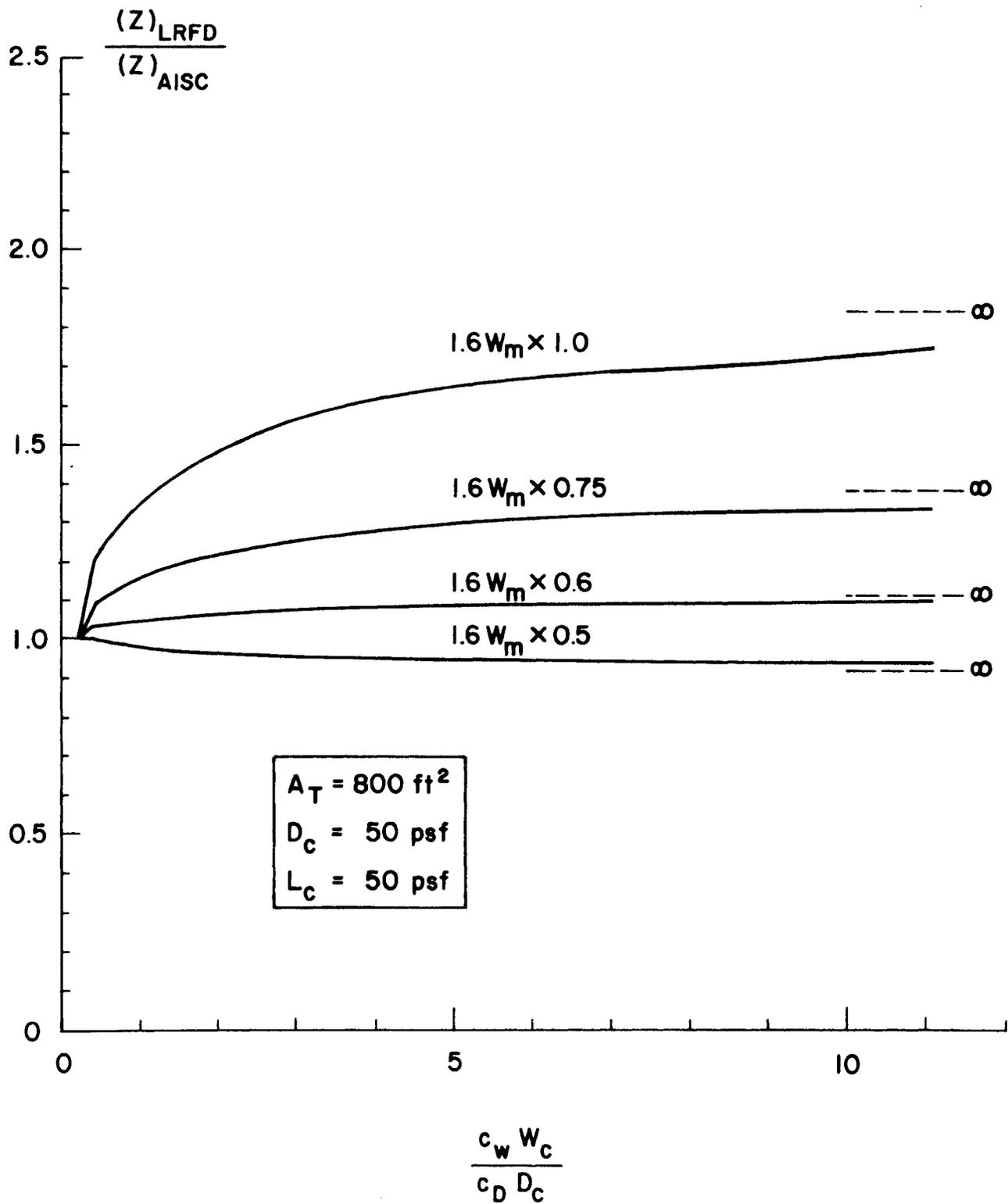


Fig. 1 Ratios of Beam Section Moduli for Beams in an Office Building.

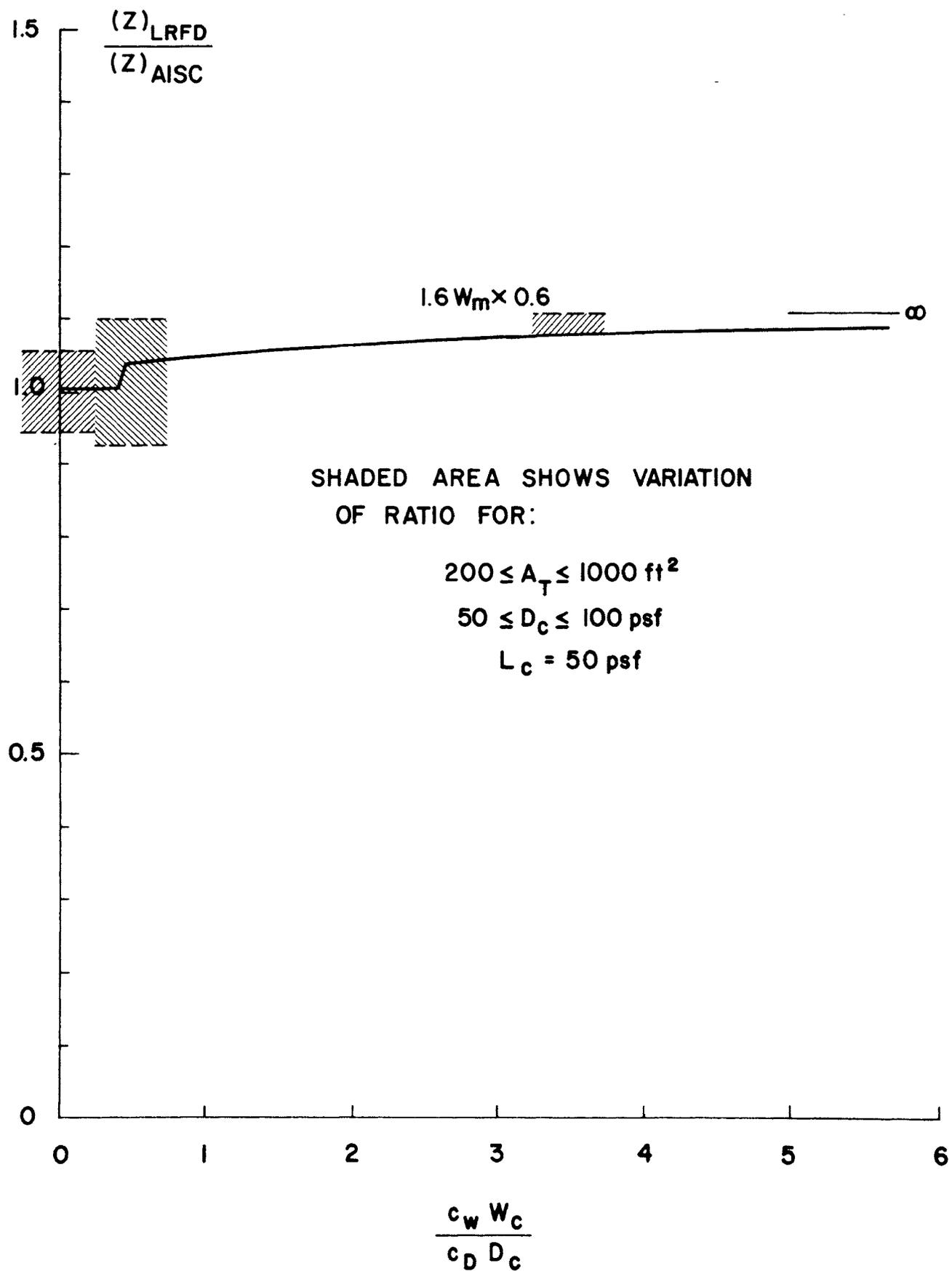


Fig. 2 Variation of Beam Section Modulus Ratios.

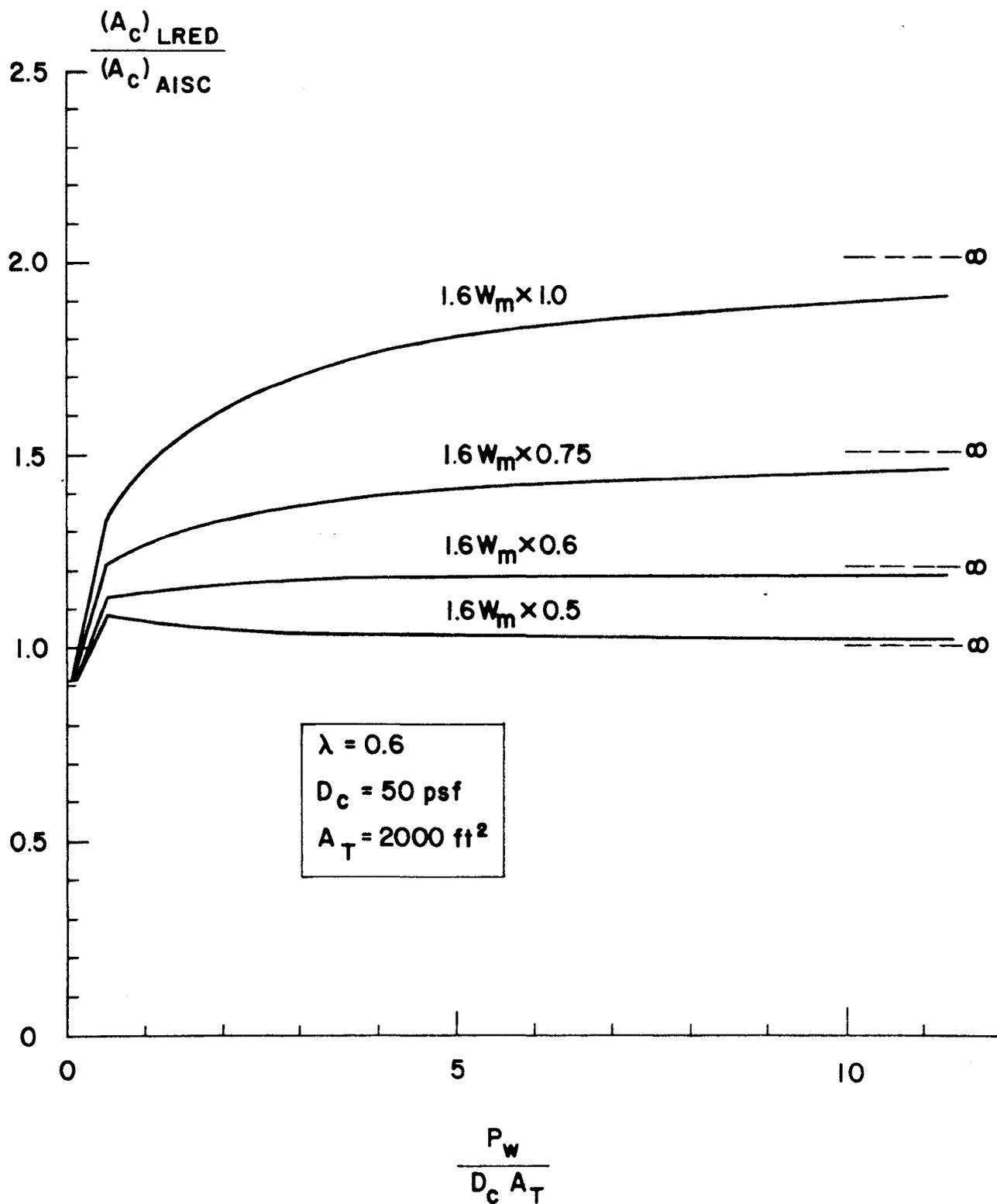


Fig. 3 Ratios of Column Areas for Simple Column in a Braced Frame Office Building ( $L_c = 50 \text{ psf}$ ).

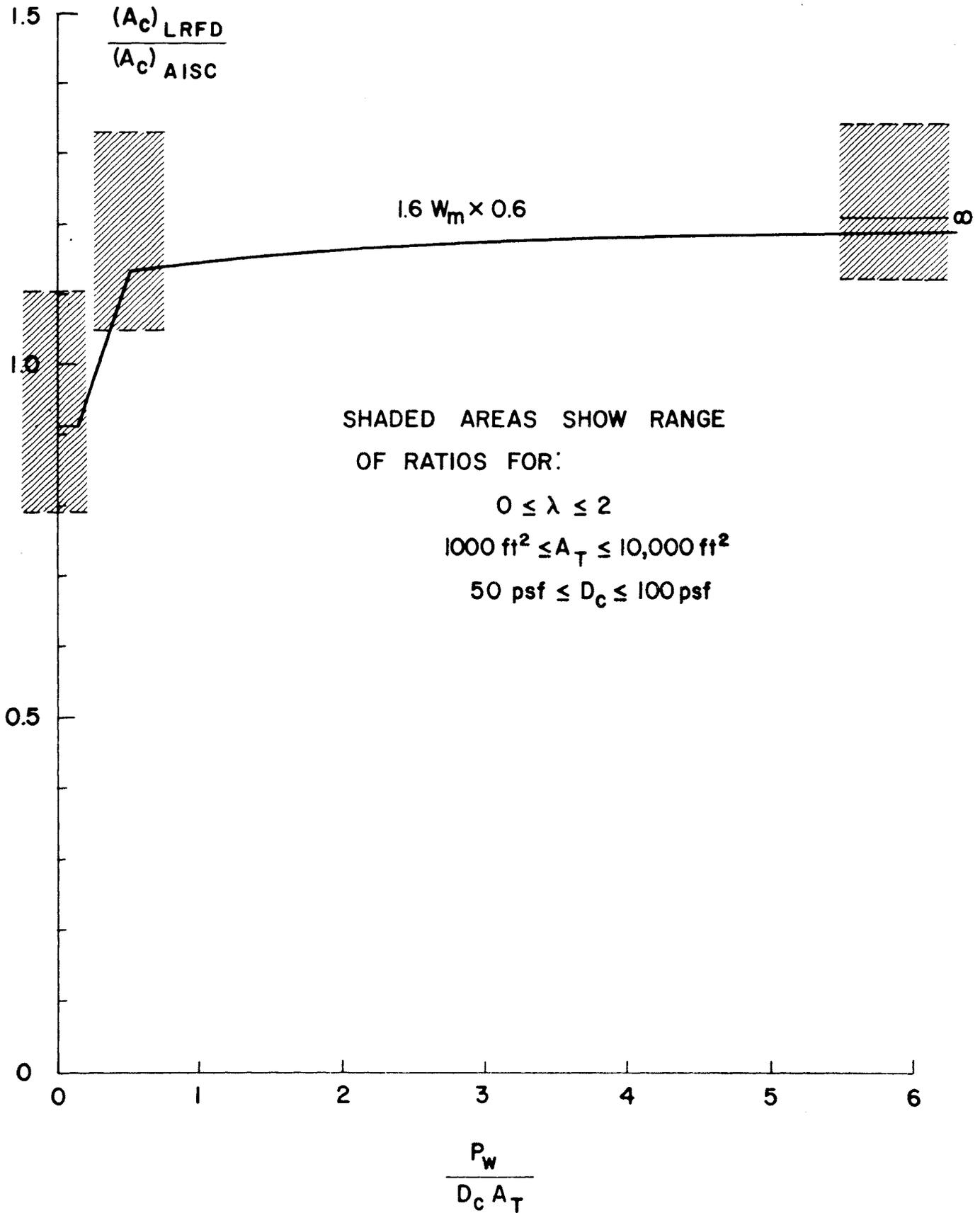


Fig. 4 Variation of Column Area Ratios ( $L_c = 50 \text{ psf}$ )



Fig. 5 - Mean of the logarithms of the water equivalent of ground snow. (From Ref. 11).

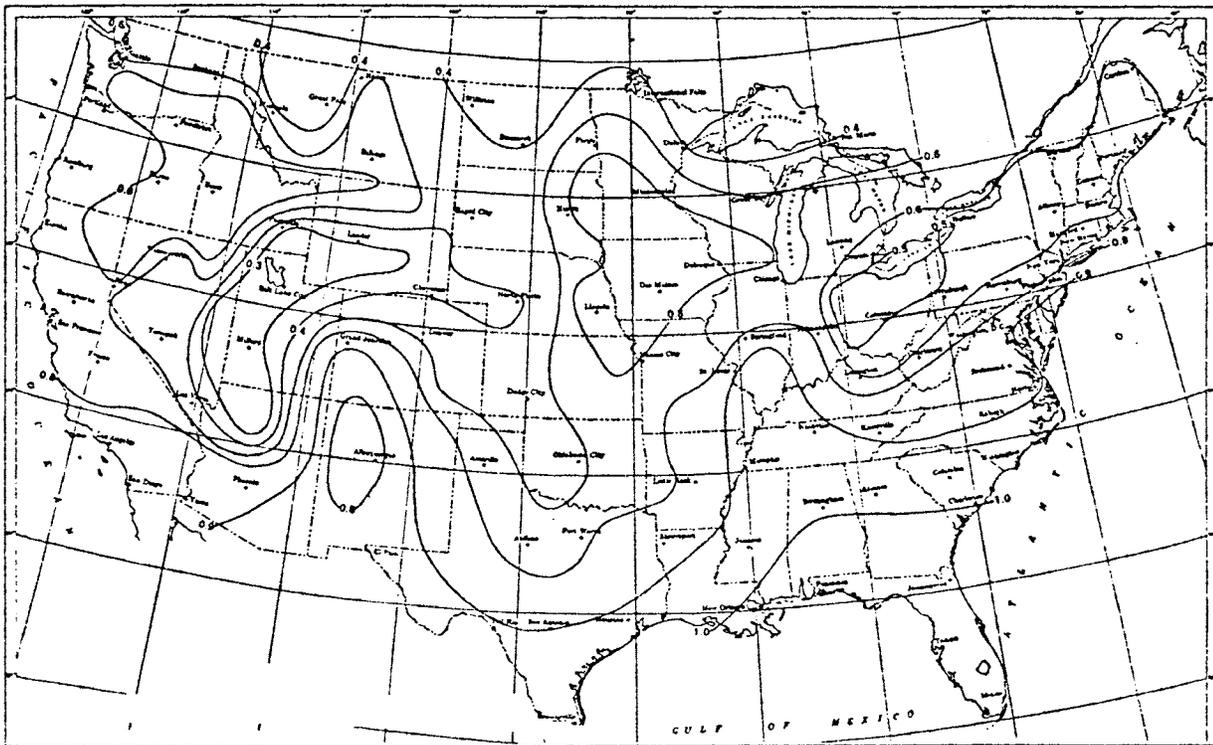


Fig. 6 - Standard deviation of the logarithms of the water equivalent of ground snow. (From Ref. 11).

APPENDIX: AISC-VERSUS-LRFD BEAM AND COLUMN DESIGNS UNDER WIND LOADING

The design requirement in Part 2 of the AISC Specification for load combinations involving wind loads is that for beams

$$F_y Z_A = 1.3 [c_D D_c + c_L L_{rc} + c_W W_c] \quad (A-1)$$

where  $F_y$  = specified yield stress

$Z_A$  = plastic section modulus required according to Part 2 of the AISC Specification

$c_D$  = influence factor translating the dead load into a bending moment

$c_L$  = influence factor translating the live load intensity into a bending moment

$c_W$  = influence factor translating the wind load intensity into a bending moment

$D_c$  = code-specified dead load intensity

$L_{rc}$  = code-specified live load intensity, reduced in accordance with ANSI - A58.1 (1972)

$W_c$  = code specified wind load intensity according to ANSI - A58.1 (1972).

An additional requirement is that

$$F_y Z_A \geq 1.7 [c_D D_c + c_L L_{rc}] \quad (A-2)$$

It can usually be assumed that both the dead and the live loads act in the same direction as uniformly distributed loads, and therefore,

$c_D = c_L$ , and thus the plastic section modulus required by the AISC Specification under combined wind and gravity loading is equal to

$$Z_A = \left[ \frac{1.3 c_D D_c}{F_y} \right] \left[ 1 + \frac{L_{rc}}{D_c} + \frac{c_W W_c}{c_D D_c} \right] \quad (A-3)$$

The design requirement for the plastic section modulus in LRFD for the strength limit state is equal to the following equation

$$\phi F_y Z_L = 1.1 [1.1 c_D D_m + 2.0 c_L L_{Im} + 1.6 F c_W W_m] \quad (A-4)$$

where  $\phi$  = resistance factor,  $\phi = 0.86$  (Ref. 1)

$Z_L$  = plastic section modulus required by LRFD

$D_m$  = mean dead load intensity

$L_{Im}$  = the mean instantaneous live load intensity

( $L_{Im} = 12$  psf, Ref. 1)

$W_m$  = the mean maximum lifetime wind pressure

$F$  = a factor, yet to be determined, by which the wind pressure is reduced to achieve calibration with the AISC Specification requirement

In addition, it is required that

$$\phi F_y Z_L \geq 1.1 [1.1 c_D D_m + 1.4 c_L L_m] \quad (A-5)$$

where  $L_m$  is the mean maximum lifetime live load intensity. The value of  $L_m$  is determined by the formula (Ref. 1)

$$L_m = 14.9 + \frac{763}{\sqrt{A_I}} \quad (A-6)$$

in units of psf;  $A_I$  is the influence area which is equal to twice the tributary area  $A_T$  for beams and  $4 A_T$  for columns.

In Ref. 1 it is shown that  $D_m = D_c$ , and for a 50 yr. life the relationship between the ANSI - A58.1 (1972) code-specified wind pressure and the mean maximum lifetime wind pressure is  $W_m = 1.17 W_c$ . Assuming, again, that  $c_D = c_L$ , the plastic section modulus required by LRFD is

$$Z_L = \frac{1.1 c_D D_c}{\phi F_y} \left\{ 1.1 + \frac{2 \times 12}{D_c} + 1.6 F \times 1.17 \left( \frac{c_W W_c}{c_D D_c} \right) \right\} \quad (A-7)$$

The ratio  $Z_L/Z_A$  is plotted versus the wind load-to-dead load moment ratio  $c_W W_c / c_D D_c$  in Fig. 1 for a tributary area of  $A_T = 800 \text{ ft}^2$  ( $L_{rc} = 27 \text{ psf}$  for an office live load intensity of  $L_c = 50 \text{ psf}$ , and  $L_m = 34 \text{ psf}$ ) and for a dead load intensity of  $50 \text{ psf}$ . Curves show the variation of the ratios of the section moduli for  $F = 1.0, 0.75, 0.6$  and  $0.5$ . It is evident that if the applied wind pressure is not reduced (i.e.,  $F = 1.0$ ), LRFD requires considerably larger sections than the AISC Specification. The left corner of each curve, where the ratio is approximately unity, corresponds to the case where gravity loading only governs.

From Fig. 1 it appears that  $F = 0.6$  is the best value for the factor by which the wind pressure is reduced to achieve calibration for the specific instances for which the curves apply. The curve for  $F = 0.6$  is reproduced in Fig. 2, where, in addition, shaded areas define the variation of the ratio of the section moduli for the domain of the parameters indicated. The spread becomes smaller as the wind load participation increases, and it is largest in the range where the wind load is small. The LRFD-to-AISC ratio does not, however, go below 90%.

A similar comparison is shown in Figs. 3 and 4 for simple columns in braced frames. In these figures the ratio of the required column areas is plotted against the ratio of the code-specified wind load  $P_w$  to the code-specified dead load  $D_c A_T$ .

The AISC column area requirement is

$$(A_c)_A F_a = 0.75 [A_T D_c + A_T L_{rc} + P_w] \quad (A-8)$$

In addition

$$(A_c)_A F_a \geq A_T D_c + A_T L_{rc} \quad (A-9)$$

The LRFD requirements are (Ref. 1)

$$(A_c)_L \phi F_{cr} = 1.1 [1.1 A_T D_m + 2.0 A_T L_{Im} + 1.6 F P_{wn}] \quad (A-10)$$

and

$$(A_c)_L \phi F_{cr} \geq 1.1 [1.1 A_T D_m + 1.4 A_T L_m] \quad (A-11)$$

The column parameters are defined in Sec. 1.6 of the AISC Specification and in Ref. 1 as follows:

$$F_a = \frac{F_y (1 - 0.25 \lambda^2)}{\frac{5}{3} + \frac{3\lambda}{8\sqrt{2}} - \frac{\lambda}{16\sqrt{2}}} \quad \text{for } \lambda \geq \sqrt{2} \quad (A-12)$$

$$F_a = \frac{12 F_y}{23 \lambda} \quad \text{for } \lambda \geq \sqrt{2} \quad (A-13)$$

$$\phi = 0.86 \quad \text{for } \lambda \leq 0.16 \quad (A-14)$$

$$\phi = 0.90 - 0.25 \lambda \quad \text{for } 0.16 \leq \lambda \leq 1.0 \quad (A-15)$$

$$\phi = 0.65 \quad \text{for } \lambda \geq 1.0 \quad (A-16)$$

$$F_{cr} = F_y (1 - 0.25 \lambda^2) \quad \text{for } \lambda \leq \sqrt{2} \quad (A-17)$$

$$F_{cr} = \frac{F_y}{\lambda} \quad \text{for } \lambda \geq \sqrt{2} \quad (A-18)$$

The curves in Figs. 3 and 4 were determined by setting  $c_D = c_L$  and  $P_{wm} = 1.17 P_w$ , where  $P_{wm}$  is the axial force due to the mean maximum 50 yr. lifetime wind pressure, and  $P_w$  is the corresponding force due to the ANSI-specified wind pressure.

An examination of Figs. 3 and 4 indicates that  $F = 0.6$  is again a reasonable value for achieving a reasonable correlation with the AISC design. The spread is much larger than for beams (see Fig. 4), mainly because of the larger variation of the ratio  $F_a / \phi F_{cr}$ .

