Load and Resistance Factor Design of Cold-formed Steel Structural Members

Trinh-Ngoc Rang

Theodore V. Galambos

Wei-wen Yu
Missouri University of Science and Technology, wwy4@mst.edu

M. K. Ravindra

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by

Trinh-Ngoc Rang[1], Theodore V. Galambos[2]
Wei-Wen Yu[3] and M.K. Ravindra[4]

INTRODUCTION

In the design of steel buildings, the "Allowable Stress Criteria" have long been used for the design of cold-formed steel structural members in the United States,(3) Canada(7) and other countries(27). Even though the theoretical concepts of risk and reliability analyses have been available for some time(9,17,20) and the significance of such concepts in structural safety and design is well recognized, the probabilistic method has not been explicitly adopted as a basis for the American design standard for steel structures. In view of the fact that the mathematical theory of probability, which has been so successfully applied in other fields of engineering,(1) would seem to be equally applicable to cold-formed steel design by providing a more uniform degree of structural safety, the "Limit States Design", which utilizes the probabilistic concept, was introduced in the Canadian Standard on the Design of Cold-Formed Steel Structural Members in 1974(8,22) as an alternate to existing procedures for design

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[1]Civil Engineer, City of Tulsa, Oklahoma; Formerly, Research Assistant Professor, Department of Civil Engineering, University of Missouri-Rolla, Rolla, Missouri.

[2]Chairman and Professor, Department of Civil Engineering, Washington University, St. Louis, Missouri.

[3]Professor, Department of Civil Engineering, University of Missouri-Rolla, Rolla, Missouri.

[4]Supervisor, Structural Analytical Division, Sargent and Lundy Engineers, Chicago, Illinois.

calculation of building structures. Recently, the "Load and Resistance Factor Design Criteria for Steel Buildings" have also been studied by Galambos and Ravindra(18,19). However, this study deals only with the design of hot-rolled shapes and built-up members fabricated from steel plates for buildings designed in accordance with the AISC Specification(2). Since 1976, the American Iron and Steel Institute has sponsored a research project to develop the load and resistance factor design criteria for cold-formed steel structural members.

The load and resistance factor design (LRFD), as the name implies, utilizes both resistance factors and load factors. It is based on the concept of limit states which describe a limit of structural usefulness. Load and resistance factors reflect the uncertainties of analysis, design, loading, material properties and fabrication. They are derived on the basis of the first order probabilistic design principles. In this approach, only the mean values and coefficients of variation of the loads and resistances are used.

For the purpose of developing the new design criteria for cold-formed steel structural members on the basis of the theory of probability, statistical analyses have been made on material properties, geometric properties and load-carrying capacities of structural members.

This paper summarizes the results of statistical analyses of mechanical properties and material thicknesses of steel sheets generally used for cold-formed structural members. In addition, it presents the method used for calibration of the AISI effective design width formula by using the available test data on stiffened compression elements.

STATISTICAL ANALYSIS OF MECHANICAL PROPERTIES

In order to develop the load and resistance factor design criteria for cold formed steel structural members based on the probabilistic approach, it is necessary to know the actual mean values and coefficient of variation of some important mechanical properties such as yield point and ultimate tensile strength of steel sheets and strip used.

Because the design is based on the specified minimum yield point and tensile strength, the mean values of the actually measured mechanical properties established by the standard coupon tests are usually found to be higher than specified values.

The results of 4,225 test coupons for various types of steels have recently been collected and analyzed statistically. The mean value of the σ_y/F_y ratios was found to be 1.17 with a coefficient of variation of 10%. In the above expression, σ_y is the tested yield point and F_y is the specified minimum yield point. Figure 1 is a histogram for the yield point.

For the tensile strength of the same samples, the mean value of the σ_u/F_u ratios is 1.12 with a coefficient of variation of 7%, where σ_u is the tested tensile strength and F_u is the specified minimum tensile strength. Figure 2 is a histogram for the tensile strength.

Since most of the test coupons were taken from coil ends, some variations in yield point and tensile strength along the length of coil are expected. In addition, the loading rate during the tests are higher than the rate of loading in the structure. The rate of straining has a definite effect on the yield point.[18] For these reasons the following values may be used for the purpose of developing the load and resistance factor design criteria for cold-formed steel:

$$(\sigma_y)_m = 1.10\ F_y, \qquad V_{\sigma_y/F_y} = 0.10 \qquad (1)$$

$$(\sigma_u)_m = 1.10\ F_u, \qquad V_{\sigma_u/F_u} = 0.07 \qquad (2)$$

STATISTICAL ANALYSIS OF MATERIAL THICKNESS

One of the principal variables affecting the geometric properties of cold-formed steel members is material thickness. In this statistical analysis, more than 1,400 thickness measurements have been collected and analyzed. The measured thicknesses of the base metal range from 0.015 to 1 in. (0.38 to 25.4 mm). This range covers the thicknesses generally used in cold-formed steel construction. Figures 3 and 4 show the mean values, the coefficients of variation and the distribution of the ratios of the measured to specified thickness for two ranges of thicknesses. An examination of the information presented in these histograms reveals that the mean value of the measured thicknesses is about 5% greater than the ordered minimum values and that the coefficient of variation is about 5%. In view of the fact that the recent revision of the AISI Specification[4] permits the delivered minimum thickness to be 95 percent of the design value, the nominal value of the material thickness may be used as the mean value and its coefficient of variation may be taken as 5%, i.e.

$$t_m = t_s, \qquad V_{t/t_s} = 0.05 \qquad (3)$$

CALIBRATION PROCEDURE FOR LOAD AND RESISTANCE FACTOR DESIGN

Many schemes have been proposed in the past to formulate design processes for structures with a more consistent reliability. These schemes have used the concepts of the theory of probabilty[5,6,16,18,23] In the United States, the load and resistance factor design criteria for steel buildings using hot rolled shapes have been developed by Galambos and Ravindra.[18] The general format of the criteria can be expressed in the following form.

$$\phi R_n \geq \gamma_A(\gamma_D c_D D_c + \gamma_L c_L L_c) \quad (4)$$

in which the right side of the equation represents the effects of a combination of dead load, D_c, and live load, L_c, whereas, the left side relates to the nominal resistance, R_n, of a structural member; γ_A, γ_D, and γ_L are load factors associated with the structural analysis, dead load, and live load, respectively; ϕ is the resistance factor, and c_D and c_L are deterministic influence coefficients, which transform the load intensities to load effects.

The resistance of a structural member can be considered as a random variable determined by the following equation:

$$R = R_n MFP \quad (5)$$

in which R_n is the nominal resistance, M is called the material factor which reflects the uncertainties in the material properties (i.e., σ_y, σ_u, etc.), F is the fabrication factor which accounts the uncertainties in the geometry of the cross section (i.e. depth, width, thickness, etc. to be used for computing section modulus, area, etc.)

and P is the professional factor which reflects the uncertainties in the design methods (i.e., assumptions, approximations of theoretical formulas, etc.).

Consequently, the mean resistance, R_m, is

$$R_m = R_n M_m F_m P_m \tag{6}$$

In the above equation, M_m, F_m and P_m are the mean values of M, F and P, respectively. The nominal resistance determined on the basis of the applicable design specification is equal to the load effects with an appropriate factor of safety, FS, i.e.

$$R_n = (FS)(c_D D_c + c_L L_c) \tag{7}$$

in which D_c and L_c are specified dead and live loads. Therefore, Eq. (6) can also be written in the following form by substituting Eq. (7) for R_n.

$$R_m = (FS) M_m F_m P_m (c_D D_c + c_L L_c) \tag{8}$$

By using the first order probabilistic theory and assuming that there is no correlation between M, F and P, one finds that the coefficient of variation of the resistance is

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} \tag{9}$$

in which V_M, V_F and V_P are coefficients of variation of the random variables M, F and P, respectively.

The mean load effect, Q_m, for a combination of dead and live loads is assumed to be of the form

$$Q_m = A_m(c_D C_m D_m + c_L B_m L_m) \tag{10}$$

in which A_m is a mean value of a random variable representing the uncertainties in structural analysis, and C_m and B_m are mean values of random variables reflecting the uncertainties in the transformation of dead and live load intensities into load effects, respectively. D_m and L_m are the mean values of random variables for dead and live load intensities.

By assuming $C_m = B_m = 1.0$ and $c_D = c_L$, the coefficient of variation of load effects, V_Q, is

$$V_Q = \sqrt{V_A^2 + \frac{D_m^2(V_C^2+V_D^2)+L_m^2(V_B^2+V_L^2)}{(D_m+L_m)^2}} \tag{11}$$

In the above equation, V_A, V_C, V_D, V_B and V_L are coefficients of variation associated with the uncertainties for the structural analysis (A), dead load (C,D) and live load (B,L) random variables, respectively. Since the application of cold-formed members are found more in residential and commercial buildings with much smaller tributary areas than hot-rolled shapes, no reduction in live load was considered in this study. It was assumed that the mean dead and live loads are equal to the specified values (i.e. $D_m = D_c$ and $L_m = L_c$). Therefore, Eq. (11) can be rewritten in the following form:

$$V_Q = \sqrt{V_A^2 + \frac{(D_c/L_c)^2(V_C^2+V_D^2)+(V_B^2+V_L^2)}{(D_c/L_c)^2+2(D_c/L_c)+1}} \tag{12}$$

In the evaluations of Q_m and V_Q, the following mean values and coefficients of variation have been used by Galambos and Ravindra in Ref. 18 for the design of steel buildings using hot-rolled shapes. The authors recommend that the same values can be applied for the design of cold-formed steel members:

$$A_m = 1.00, \; V_A = 0.05 \tag{13}$$

$$C_m = 1.00, \; V_C = 0.04 \tag{14}$$

$$B_m = 1.00, \; V_B = 0.10 \tag{15}$$

$$D_m = D_c, \quad V_D = 0.04 \tag{16}$$

$$L_m = L_c, \quad V_L = 0.13 \tag{17}$$

The basis for the development of the load and resistance factor design criteria of steel structural members is given in Fig. 5, where the two distributions for R and Q in Fig. 6 are combined as one curve ln(R/Q). As shown in Fig. 5, a limit state is reached when ln(R/Q) = 0. The area under the curve to the left of this point represents the probability of exceeding a limit state. The value β, called the safety index, which relates the distance between the mean value of $[\ln(R/Q)]_m$ and the reference line represents the probability of exceeding the given limit state. The higher the numerical value of β, the lower the probability of failure. The safety index, β, can also be expressed in the following algebraic form:

$$\beta = \frac{\ln(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \tag{18}$$

For the purpose of developing load and resistance factor design criteria, the value of the safety index has to be selected. This selection is done by calibration. The procedure of calibration is the determination of the values of safety index implied in current designs and the selection of a set of β values representative of all design situations. The resistance factor and load factors can be determined by using the following equations:

$$\phi = \frac{R_m}{R_n} \exp\,(-\alpha\beta V_R) \tag{19}$$

$$\gamma_A = \exp\,(\alpha\beta V_A) \tag{20}$$

$$\gamma_D = 1 + \alpha\beta \sqrt{V_C^2 + V_D^2} \tag{21}$$

$$\gamma_L = 1 + \alpha\beta \sqrt{V_B^2 + V_L^2} \tag{22}$$

in which α is taken as 0.55.[18]

In summary, the procedures for calibrating design provisions usually consist of the following five steps:

(1) Analyze the available information to obtain statistical values on resistance and load effects by using Eqs. (8) through (12).

(2) Assume values of mean and coefficient of variation of the variables for which no statistical information is available.

(3) Compute the safety index based on Eq. (18).

(4) Select the safety index β.

(5) Derive the resistance and load factors [Eqs. (19) through (22)] by using the selected safety index.

This procedure will be illustrated by the following section of this paper concerning the AISI effective design width formula.

CALIBRATION OF THE AISI EFFECTIVE WIDTH DESIGN FORMULA

The objective of this section is to determine the safety index from a calibration of the AISI effective width design formula for stiffened compression elements.[3] In this process, the mean values and coefficients of variation were obtained from the statistical analyses of the test data on mechanical properties, thicknesses, ultimate moments of beams, failure loads of stub columns and thin plates.

In the computation of R_m (Eq. (8)), V_R (Eq. (9)), Q_m (Eq. (10)) and V_Q (Eq. (12)), the following mean values and coefficients of variation were used.

$$M_m = 1.10, \quad V_M = 0.10$$
$$F_m = 1.00, \quad V_F = 0.06$$
$$A_m = 1.00, \quad V_A = 0.05$$
$$C_m = 1.00, \quad V_C = 0.04$$
$$B_m = 1.00, \quad V_B = 0.10$$
$$D_m = D_c, \quad V_D = 0.04$$
$$L_m = L_c, \quad V_L = 0.13$$

Among the above listed values, M_m and V_M are based on the statistical analyses of 4,225 tensile coupon tests; F_m and V_F are based on the statistical analyses of 1,436 thickness measurements with due consideration given to other dimensions including depth, width, etc.; other values are based on the previous work reported in Ref. 18. The values of P_m and V_P given in Tables 1 and 2, were determined from the tested and predicted load carrying capacities of beams, stub columns and thin steel plates by using the actual thicknesses of the cross sections. In this study, several ranges of width-thickness ratios were used for stiffened compression elements.

In the calibration, the tested ultimate moments for beams, $(M_u)_t$, were obtained from Refs. 10, 11, 12, 21, 24, 25 and 26. The tested failure compressive loads for stub columns and thin plates, $(P_u)_t$, were computed from Refs. 13 through 15. The predicted values of $(M_u)_p$ and $(P_u)_p$ were computed as follows:

$$(M_u)_p = S_{eff} F_y \tag{23}$$

$$(P_u)_p = A_{eff} F_y^{\dagger} \tag{24}$$

in which

$(M_u)_p$ = predicted ultimate moment of a beam having stiffened compression flanges.

$(P_u)_p$ = predicted failure load of a column having stiffened compression flanges.

F_y = yield point of steel

A_{eff} = effective area of the cross section

S_{eff} = section modulus about x-axis calculated on the basis of the effective design width of the compression flanges.

The effective widths, b, of the compression elements were determined in accordance with Section 2.3.1.1 of the 1968 Edition of the AISI Specification, i.e. elements are fully effective (b = w) up to $(w/t)_{lim} = 171/\sqrt{f}$. For elements with w/t larger than $(w/t)_{lim}$,

$$\frac{b}{t} = \frac{253}{\sqrt{f}}\left(1 - \frac{55.3}{(w/t)\sqrt{f}}\right) \tag{25}$$

Elements of closed square and rectangular tubes are fully effective (b = w) up to $(w/t)_{lim} = 184/\sqrt{f}$. For elements with w/t larger than $(w/t)_{lim}$,

$$\frac{b}{t} = \frac{253}{\sqrt{f}}\left(1 - \frac{50.3}{(w/t)\sqrt{f}}\right) \tag{26}$$

In the above equations, w/t = flat width ratio, b = effective width, and f = actual stress in the compression element computed on the basis

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†Overall column buckling was not considered because the slenderness ratios are low and no overall buckling was observed in the tests.

of effective design width. The tested and predicted ultimate moments are listed in Tables 1a, 1b and 1c for 43 beams. The tested and predicted failure loads are listed in Tables 2a, 2b and 2c for 44 stub columns and thin plates. Based on these tested and predicted values, the mean values of the professional factor, P_m, and their coefficients of variation, V_P, were computed and summarized in Table 3. Six different cases have been studied according to the types of structural members (i.e., beams and columns) and the ranges of w/t ratios.

Consequently, the mean resistance, R_m, and its coefficient of variation, V_R, of the structural members having stiffened compression flanges can be computed by Eqs. (8) and (9) for a given set of dead to live load ratios.

Substituting these values into Eq. (18), the safety index, β, can be computed. The ranges of β for various stiffened compression elements are listed in Table 3 for $D_c/L_c = 0.1$ to 3.0. For a representative value of $D_c/L_c = 1/3$, the values of β range from 2.66 to 3.70. From this information, a single value of β may be chosen for computation of load factors according to Eqs. (19) through (22).

SUMMARY AND CONCLUSIONS

The load and resistance factor design criteria are being developed for cold-formed steel structural members. This paper summarizes the statistical analyses of mechanical properties and material thicknesses for the steels generally used for this type of construction.

The procedures used for calibrating the AISI design provisions and the methods to be used for determining load and resistance factors are briefly discussed in this paper. As an illustration, the values of safety index for the design of members with stiffened compression elements have been developed from a calibration of the AISI effective width design formulas and the available test data. Other design provisions which have been calibrated in the same manner, will be discussed in subsequent papers.

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APPENDIX II.— NOTATION

The following symbols are used in this paper:

A = Analysis factor;

A_{eff} = Effective area of the cross section, in square inches;

B = Live load transformation factor;

b = Effective design width, in inches;

C = Dead load transformation factor;

c = Influence factor;

c = Subscript defining code-specified loads;

D = Dead load intensity, in pounds per square foot;

F = Fabrication factor;

FS = Factor of safety;

F_u = Specified ultimate tensile strength, in kips per square inch;

F_y = Specified yield point, in kips per square inch;

f = Actual stress in compression element, in kips per square inch;

L = Live load intensity, in pounds per square foot;

M = Material factor;

M_u = Ultimate moment capacity, in inch-kips;

m = Subscript defining mean value;

n = Subscript defining nominal value;

P = Professional factor;

P_u = Failure load capacity; in kips;

Q = Load effect;

R = Resistance of a structural member;

S_{eff} = Section modulus about x-axis calculated on the basis of the effective design width of the compressive flange, in inches^3;

t = Thickness of steel sheet, in inches;

V = Coefficient of variation;

w = Flat width of element exclusive of fillets, in inches;

α = Linearization constant ;

β = Safety index;

γ = Load factor;

ϕ = Resistance factor;

σ_y = Actual yield point, in kips per square inch;

σ_u = Actual ultimate tensile strength, in kips per square inch.

TABLE 1a

Comparison of Tested and Predicted Ultimate Moments of Cold-Formed Steel Beams Having Stiffened Compression Flanges with $w/t \leq (w/t)_{lim}$

Specimen	w/t	$(w/t)_{lim}$	F_y (ksi)	$(M_u)_p^*$ (in.-kips)	$(M_u)_t$ (in.-kips)	$\frac{(M_u)_t}{(M_u)_p}$	Source for test data
4-2.5-10/1	13.4	36.9	35.7	112.51	120.20	1.07	Ref. 11
6-2.5-9/1	15.8	38.4	33.1	225.85	254.70	1.13	"
8-3-9/1	16.0	38.4	33.1	333.15	375.00	1.13	"
4-2.5-12/1	17.7	37.4	35.1	91.59	108.20	1.18	"
8-3-12/1	22.0	36.7	36.2	279.76	291.70	1.04	"
6-3-12/1	22.4	37.4	35.1	182.20	199.00	1.09	"
8-4-9/1	22.5	38.4	33.1	404.34	440.00	1.09	"
4-2-16/1	27.6	40.4	30.2	39.40	43.10	1.09	"
8-4-12/1	30.5	36.7	36.2	345.48	325.00	0.94	"
E1	32.2	35.9	37.9	149.31	144.00	0.96	Ref. 10
B1	35.2	36.9	35.8	210.40	268.20	1.27	"
4-2.5-16/1	37.1	40.2	30.2	49.42	49.20	1.00	Ref. 11
Mean value of $(M_u)_t/(M_u)_p$						$P_m = 1.08$	
Coefficient of variation						$V_P = 0.08$	

Note: 1 ksi = 6.9 MN/m^2, 1 in.-kip = 113 N-m.

*The values of $(M_u)_p$ were computed on the basis of full area.

TABLE 1b

Comparison of Tested and Predicted Ultimate Moments of Cold-Formed Steel Beams Having Stiffened Compression Flanges with $(w/t)_{lim} < w/t \leq 80$

Specimen	w/t	$(w/t)_{lim}$	F_y (ksi)	$(M_u)_p$ (in.-kips)	$(M_u)_t$ (in.-kips)	$\frac{(M_u)_t}{(M_u)_p}$	Source for test data
8-3-15/1	41.6	35.9	37.9	177.61	153.90	0.87	Ref. 11
6-3-16/1	43.6	40.1	30.3	94.90	92.60	0.98	"
D1	48.3	35.9	37.9	177.71	153.00	0.86	Ref. 10
A2	48.8	36.9	35.8	238.08	266.40	1.12	"
G1	49.1	38.9	32.2	98.08	97.40	0.99	"
F-3	51.2	40.7	38.0	2.53	3.47	1.37	Ref. 21
8-4-15/1	55.5	36.1	37.9	200.11	193.20	0.97	Ref. 11
AS304F-2	71.5	35.8	38.0	3.33	3.56	1.07	Ref. 24
C1	76.5	35.9	37.9	202.58	176.40	0.87	Ref. 10
Mean value of $(M_u)_t/(M_u)_p$						$P_m = 1.01$	
Coefficient of variation						$V_P = 0.15$	

Note: 1 ksi = 6.9 MN/m^2; 1 in.-kip = 113 N-m.

*The effective design width of the compression flange used for the calibration of $(M_u)_p$ was based on Section 2.3.1.1 of the 1968 Edition of the AISI Specification.

TABLE 1c

Comparison of Tested and Predicted Ultimate Moments of Cold-Formed Steel Beams Having Stiffened Compression Flanges with w/t > 80

Specimen	w/t	$(w/t)_{lim}$	F_y (ksi)	$(M_u)_p^*$ (in.-kips)	$(M_u)_t$ (in.-kips)	$\frac{(M_u)_t}{(M_u)_p}$	Source for test data
F1	82.7	38.9	30.7	100.96	90.54	0.91	Ref. 11
F-2	84.7	35.8	38.0	3.36	3.51	1.04	Ref. 21
16 ga #3n	85.4	41.0	31.0	21.69	28.20	1.30	Ref. 12
16 ga #1n	93.4	42.9	27.5	17.83	19.60	1.10	"
AS304F-3	103.0	35.8	38.0	3.61	3.84	1.06	Ref. 24
18 ga #3n	105.1	36.1	37.4	18.76	20.20	1.08	Ref. 12
F-6a	123.0	35.8	38.0	3.38	3.91	1.16	Ref. 21
20 ga #2n	141.6	40.2	30.1	10.18	12.00	1.18	Ref. 12
AS304F-4	150.2	35.8	30.8	3.75	4.18	1.12	Ref. 24
22 ga #Xn	150.8	43.4	25.8	8.30	8.90	1.07	Ref. 12
F-8a	153.3	35.8	38.0	3.52	4.01	1.14	Ref. 21
F-7	154.4	35.8	38.0	3.42	3.94	1.15	"
16 ga #9w	161.5	29.3	56.8	38.55	42.50	1.10	Ref. 12
22 ga #3n	167.8	44.4	24.7	6.70	7.30	1.09	"
16 ga #6w	169.3	22.1	47.2	31.25	37.40	1.20	"
22 ga #1n	170.1	43.5	25.7	6.72	8.30	1.24	"
18 ga #7w	213.6	36.8	36.0	17.77	17.90	1.01	"
18 ga #6w	219.4	44.9	24.4	12.81	14.00	1.09	"
20 ga #7w	279.6	39.9	30.6	10.36	10.50	1.01	"
20 ga #8w	298.9	44.1	25.1	8.07	10.00	1.24	"
22 ga #2w	334.2	41.7	28.0	7.38	7.50	1.02	"
22 ga #3w	338.9	42.0	27.6	7.30	7.60	1.04	"
Mean value of $(M_u)_t/(M_u)_p$						$P_m = 1.11$	
Coefficient of variation						$V_p = 0.08$	

Note: 1 ksi = 6.9 MN/m^2, 1 in.-kip = 113 N-m.

*The effective design width of the compression flange used for the calculation of $(M_u)_p$ was based on Section 2.3.1.1 of the 1968 Edition of the AISI Specification.

TABLE 2a

Comparison of Tested and Predicted Failure Loads of Steel Stub Columns and Thin Plates in Compression with $w/t \leq (w/t)_{lim}$

Specimen	w/t	$(w/t)_{lim}$	F_y (ksi)	$(P_u)_p^*$ (kips)	$(P_u)_t$ (kips)	$\frac{(P_u)_t}{(P_u)_p}$	Source for test data
RA1	20.9	38.4	38.2	1033.6	1216.0	1.18	Ref. 14
WA1	21.0	40.0	35.2	864.8	1259.0	1.46	"
WB1	30.9	42.6	31.0	678.5	863.4	1.27	"
RB1	31.4	41.8	32.3	792.3	846.9	1.07	"
16A	32.5	38.0	39.0	1678.0	1974.7	1.18	"
RC1	40.1	41.2	33.2	779.3	812.6	1.04	"
WC1	40.2	40.7	33.9	731.8	796.1	1.09	"

Mean value of $(P_u)_t/(P_u)_p$ $P_m = 1.18$

Coefficient of variation $V_P = 0.11$

Note: 1 ksi = 6.9 MN/m^2, 1 kip = 4.45 kN

*The values of $(P_u)_p$ were computed on the basis of full area.

TABLE 2b

Comparison of Tested and Predicted Failure Loads of Steel Stub Columns and Thin Plates in Compression with $(w/t)_{lim} < w/t \leq 80$

Specimen	w/t	$(w/t)_{lim}$	F_y (ksi)	$(P_u)_p^*$ (kips)	$(P_u)_t$ (kips)	$\frac{(P_u)_t}{(P_u)_p}$	Source for test data
17a	39.6	35.6	44.5	1512.1	1554.6	1.03	Ref. 14
AS	43.6	40.6	40.7	96.8	96.2	0.99	Ref. 15
ACW	43.8	40.6	29.6	53.8	84.0	1.32	"
ASW	43.8	40.6	29.6	53.8	85.8	1.35	"
AC	44.0	40.6	40.7	97.1	100.4	1.03	"
1A	44.6	35.4	45.0	1647.2	1661.3	1.01	Ref. 14
4B	44.9	35.5	44.9	1643.0	1637.2	1.00	"
WT1	48.0	36.5	42.4	1663.2	1653.7	0.99	"
RT1	48.0	35.6	44.4	1735.5	1989.1	1.15	"
WD1	49.1	42.7	31.0	524.3	584.4	1.11	"
RD1	50.0	43.6	29.7	533.5	633.9	1.19	"
BS	53.0	34.6	40.7	102.7	116.2	1.13	Ref. 15
BSW	53.8	44.4	28.4	73.1	77.7	1.06	"
BC	53.9	34.6	40.7	103.2	110.4	1.07	"
SD11	57.3	36.7	41.9	31.4	34.8	1.11	Ref. 13
5a	59.0	37.8	39.4	908.9	875.2	0.96	Ref. 14
RE1	59.4	39.4	36.3	409.1	472.9	1.16	"
WE1	59.7	41.7	32.4	367.7	386.8	1.05	"
2A	60.5	37.7	39.8	914.3	838.1	0.92	"
CCW	63.3	41.3	28.6	79.0	89.7	1.13	Ref. 15
CSW	63.3	41.3	28.6	79.0	81.8	1.04	"
WG1	64.0	37.0	41.3	330.1	325.6	0.99	Ref. 14
RG1	65.0	32.5	53.5	397.8	380.9	0.96	"
CC	66.5	34.6	40.7	108.3	130.3	1.20	Ref. 15
CS	66.7	34.6	40.7	65.0	67.4	1.04	"
Mean value of $(P_u)_t/(P_u)_p$						$P_m = 1.08$	
Coefficient of variation						$V_P = 0.10$	

Note: 1 ksi = 6.9 MN/m^2, 1 kip = 4.45 kN

*The effective design width of the compression flange used for the calibration of $(P_u)_p$ was based on Section 2.3.1.1 of the 1968 Edition of the AISI Specification.

TABLE 2c

Comparison of Tested and Predicted Failure Loads of Steel Stub Columns and Thin Plates in Compression with $w/t > 80$

Specimen	w/t	$(w/t)_{lim}$	F_y (ksi)	$(P_u)_p$* (kips)	$(P_u)_t$ (kips)	$\frac{(P_u)_t}{(P_u)_p}$	Source for test data
WF3	80.2	34.5	47.5	276.0	275.6	1.00	Ref. 14
DS	80.3	34.5	40.7	112.1	120.1	1.07	Ref. 15
RF3	80.4	32.1	54.8	205.0	318.5	1.04	Ref. 14
DCW	80.5	32.1	33.0	95.7	101.8	1.06	Ref. 15
DC	80.6	32.1	40.7	112.2	130.8	1.17	"
WF4	81.1	34.4	47.6	276.4	289.0	1.05	Ref. 14
DSW	81.3	34.4	33.0	96.0	91.7	0.95	Ref. 15
3A	82.1	43.0	30.5	370.1	442.7	1.20	Ref. 14
SD21	83.2	36.7	41.9	32.0	34.9	1.09	Ref. 13
6A	85.2	40.9	33.7	394.0	418.9	1.06	Ref. 14
SD31	117.7	36.7	41.9	32.4	36.9	1.14	Ref. 13
SD41	152.2	36.7	41.9	32.7	36.6	1.12	"
Mean value of $(P_u)_t/(P_u)_p$						$P_m = 1.08$	
Coefficient of variation						$V_p = 0.06$	

Note: 1 ksi = 6.9 MN/m^2, 1 kip = 4.45 kN

*The effective design width of the compression flange used for calculation of $(P_u)_p$ was based on Section 2.3.1.1 of the 1968 Edition of the AISI Specification.

TABLE 3

Values of Safety Index for Stiffened Compression Elements

Case	P_m^*	V_P	Range of β for $D_c/L_c = 0.1$ to 3.0	β at 1/3	No. of tests
1. Beam flanges, $w/t < (w/t)_{lim}$	1.08	0.08	3.24 - 4.34	3.52	12
2. Beam flanges, $(w/t)_{lim} < w/t \le 80$	1.01	0.15	2.50 - 3.01	2.66	9
3. Beam flanges, $w/t > 80$	1.11	0.08	3.36 - 4.41	3.65	22
4. Stub columns and thin plates, $w/t < (w/t)_{lim}$	1.18	0.11	3.44 - 4.32	3.70	7
5. Stub columns and thin plates, $(w/t)_{lim} < w/t < 80$	1.08	0.10	3.14 - 4.03	3.39	25
6. Stub columns and thin plates, $w/t > 80$	1.08	0.06	3.34 - 4.49	3.65	12

*P_m is the mean value of the ratios of the tested strength/predicted strength.

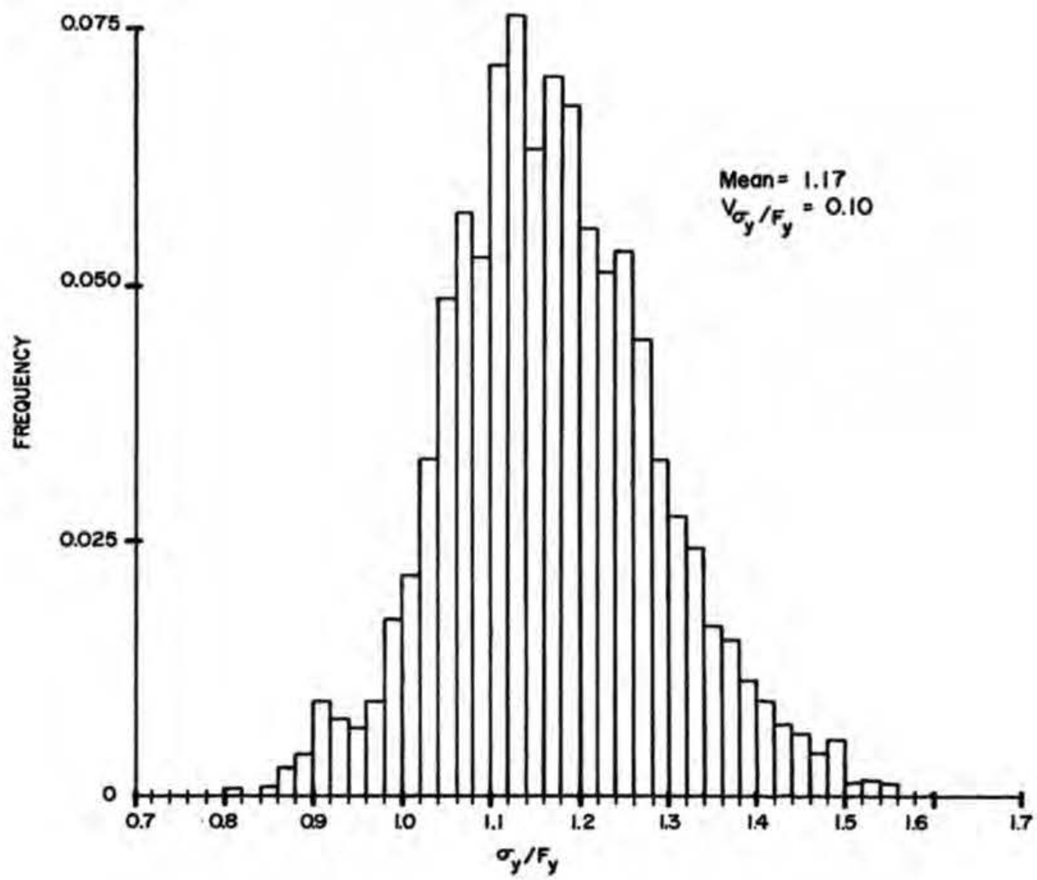

Fig. 1. Histogram for the Ratios of σ_y/F_y
F_y = 33-55 ksi, 4,225 tests

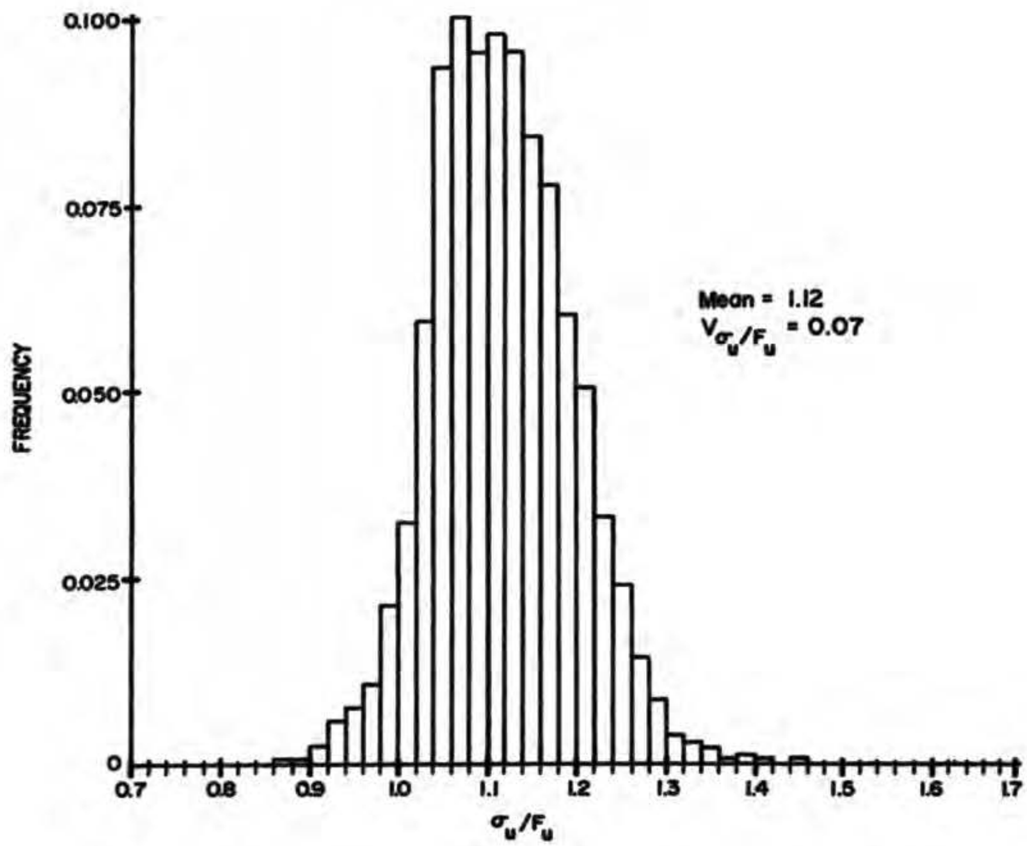

Fig. 2. Histogram for the Ratios of σ_u/F_u
F_u = 45-75 ksi, 4,225 tests

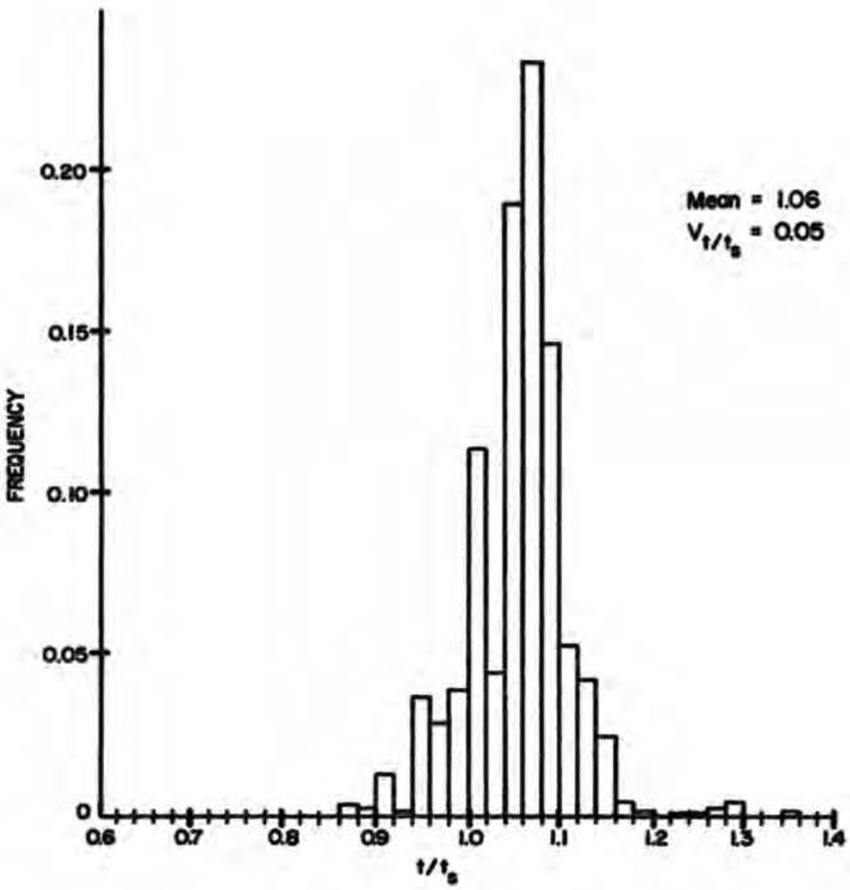

Fig. 3. Histogram for the Ratios of t/t_s
$t < 0.05$ in., 955 samples

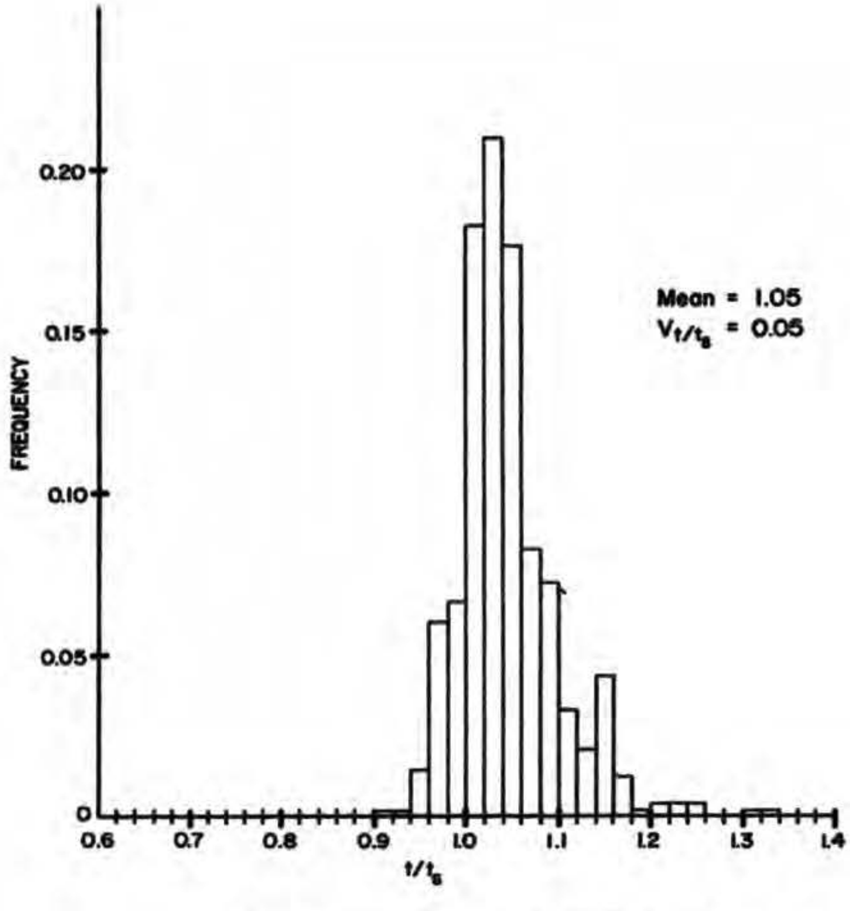

Fig. 4. Histogram for the Ratios of t/t_s
0.05 in. $< t_s \leq 1$ in., 481 samples

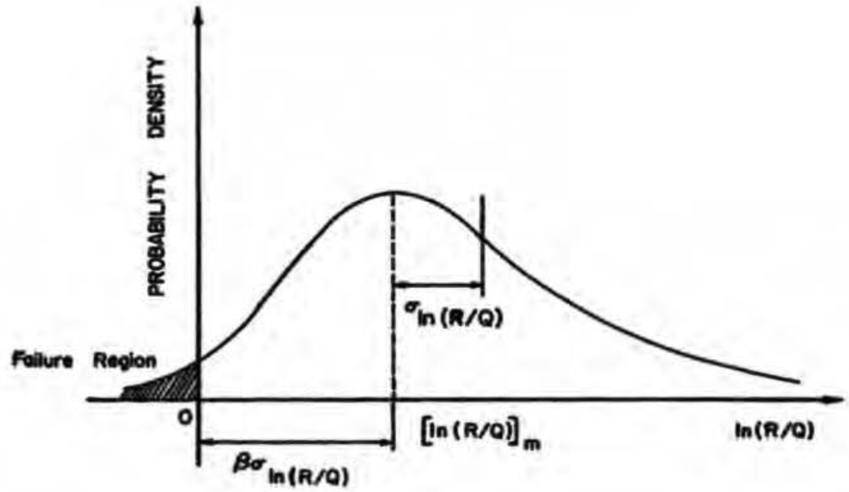

Fig. 5. Definition of Safety Index

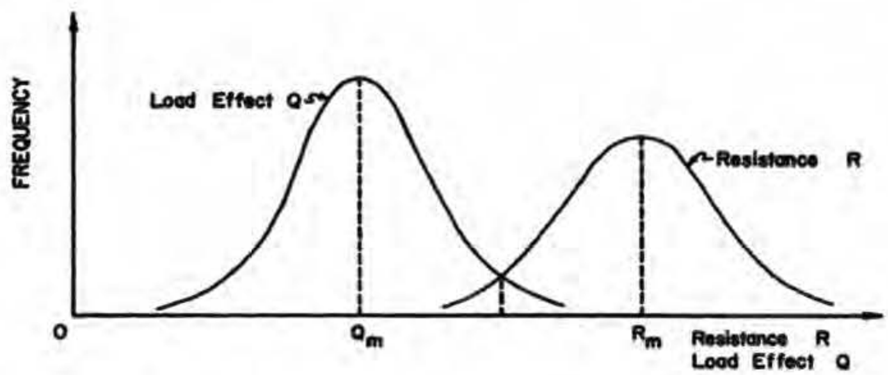

Fig. 6. Frequency Distribution of Resistance R and Load Effect Q