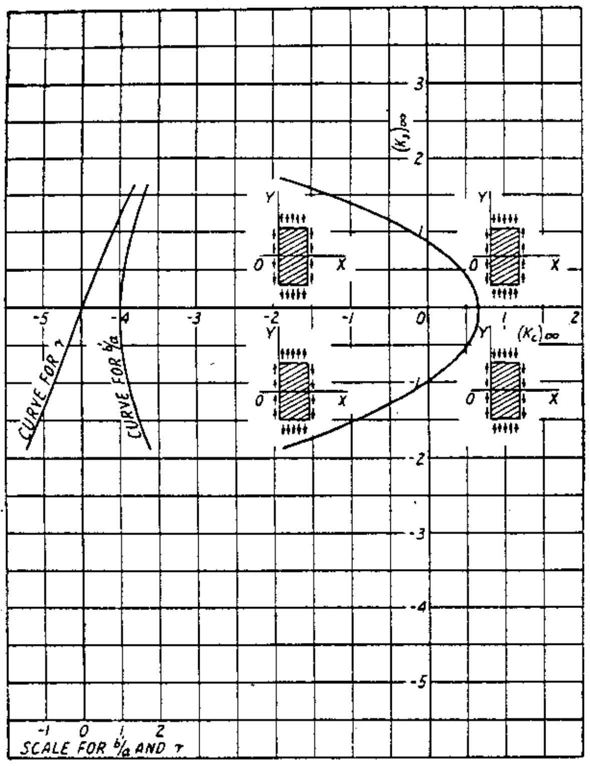
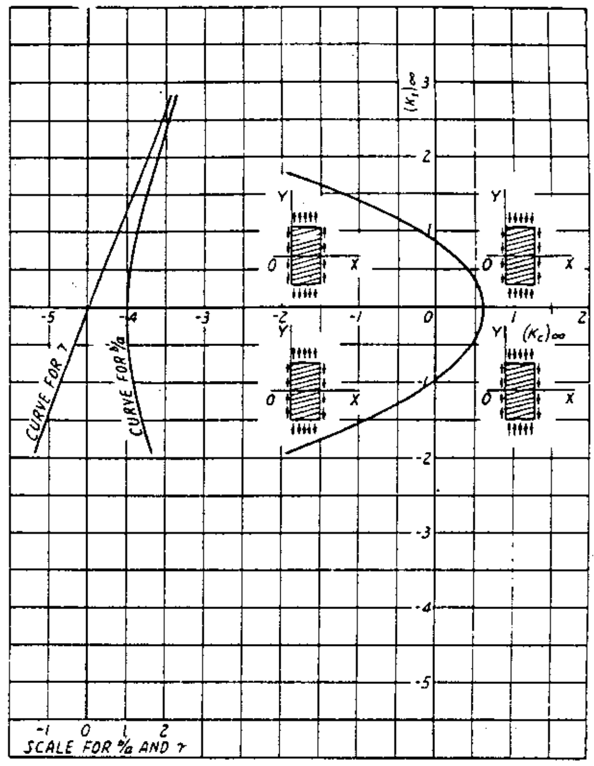
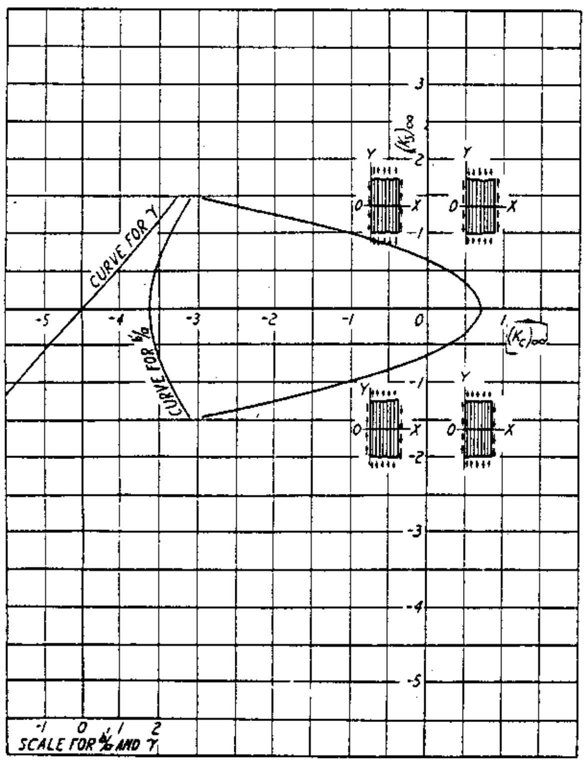


(e)
2-PLY (1:1) β = 60°

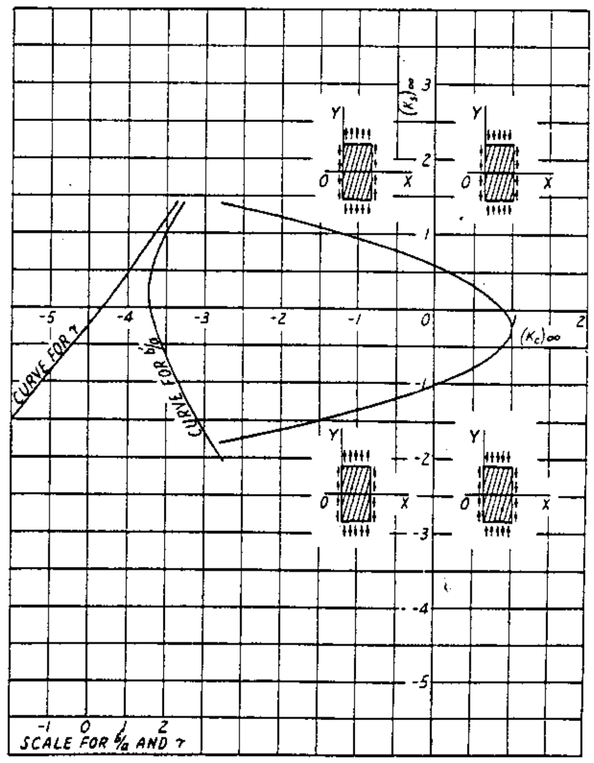


(f)
2-PLY (1:1) β = 75°

Figure 2–30 (*e*, *f*)—Continued.

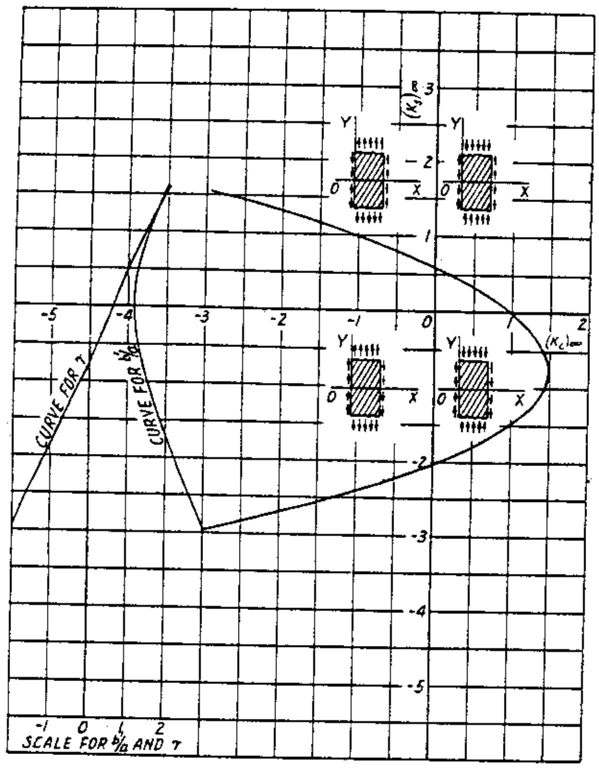


(a)
3-PLY (1:1:1) β = 0°

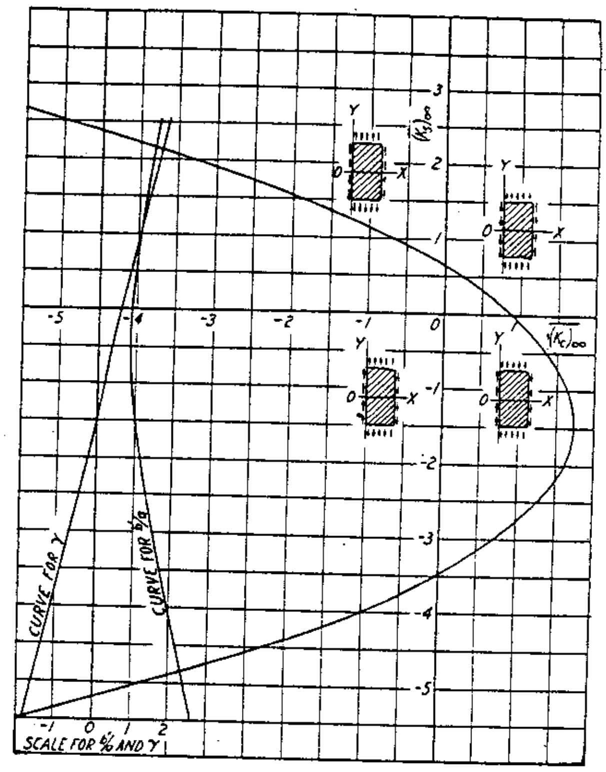


(b)
3-PLY (1:1:1) β = 15°

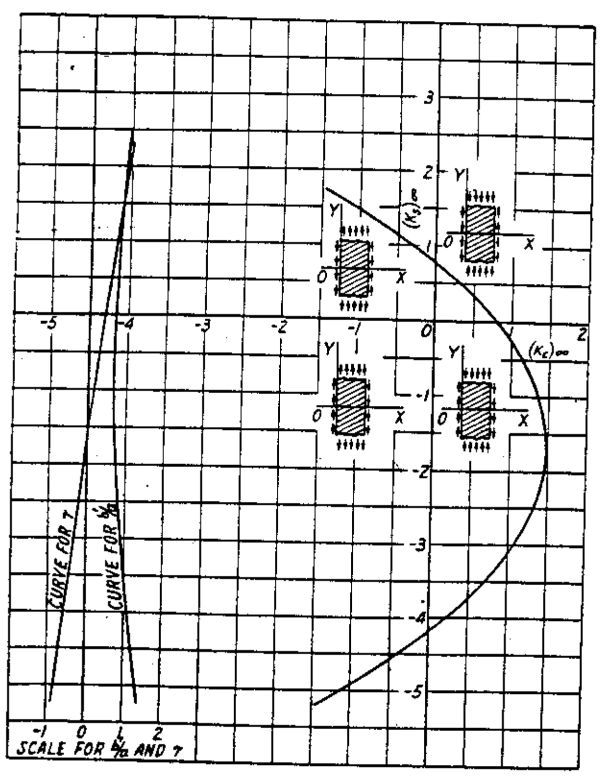
Figure 2–31 (*a*, *b*). *Curves of critical buckling constants for infinitely long rectangular plywood panels under combined loading. Edges simply supported. β = angle between face grain and direction of applied stress. Three-ply construction.*



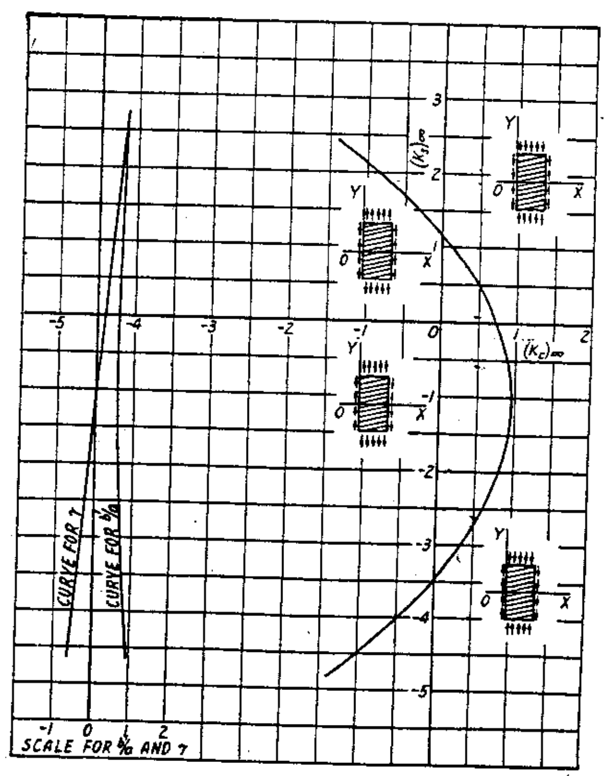
(c)
3-PLY (1:1:1) β = 30°



(d)
3-PLY (1:1:1) β = 45°

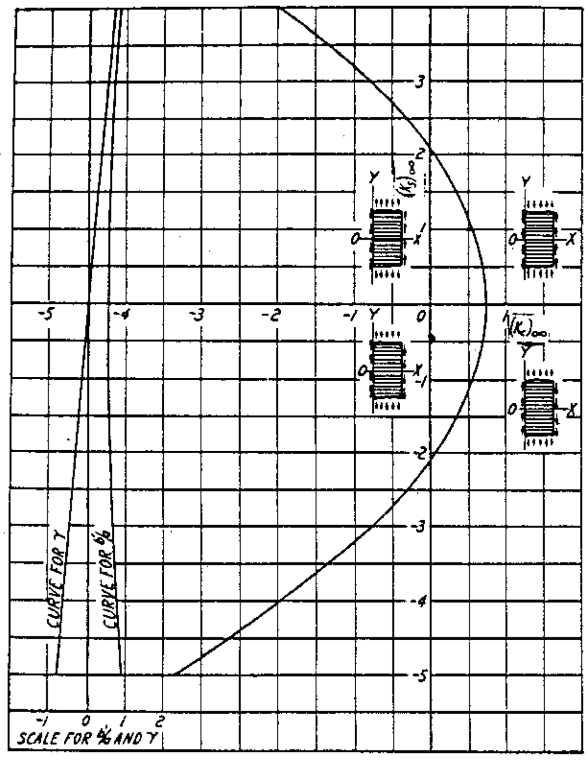


(e)
3-PLY (1:1:1) β = 60°

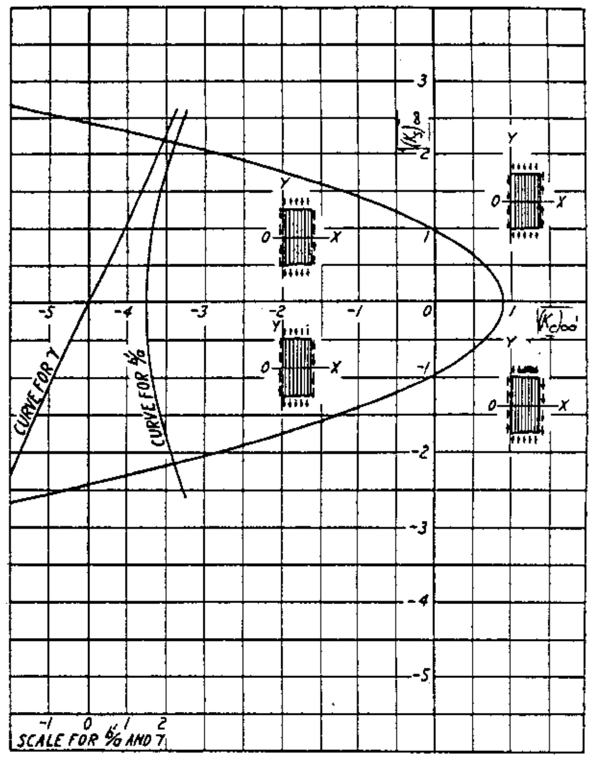


(f)
3-PLY (1:1:1) β = 75°

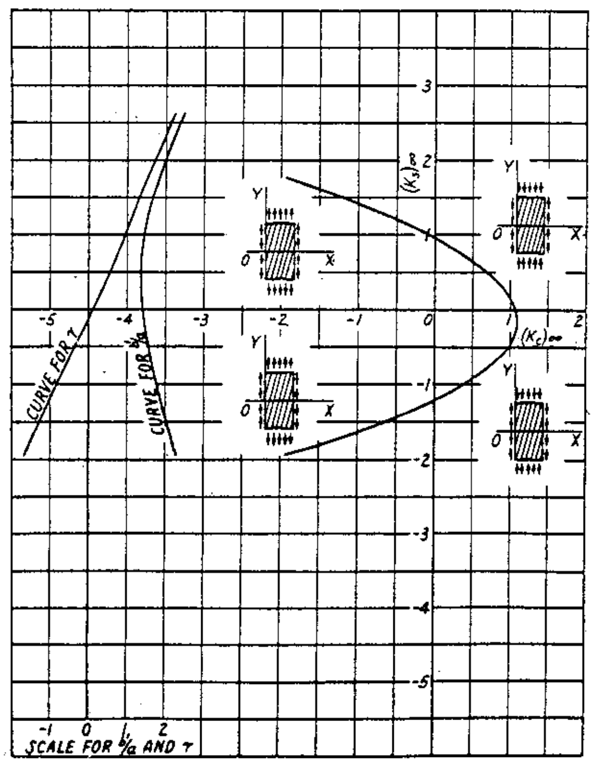
Figure 2–31 (c, d, e, f)—Continued.



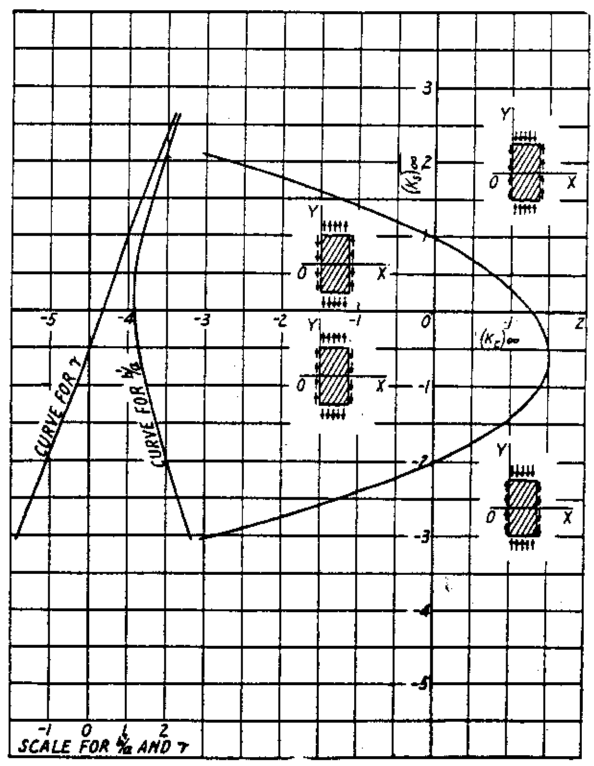
(g)
3-PLY (1:1:1) β = 90°



(h)
3-PLY (1:2:1) β = 0°



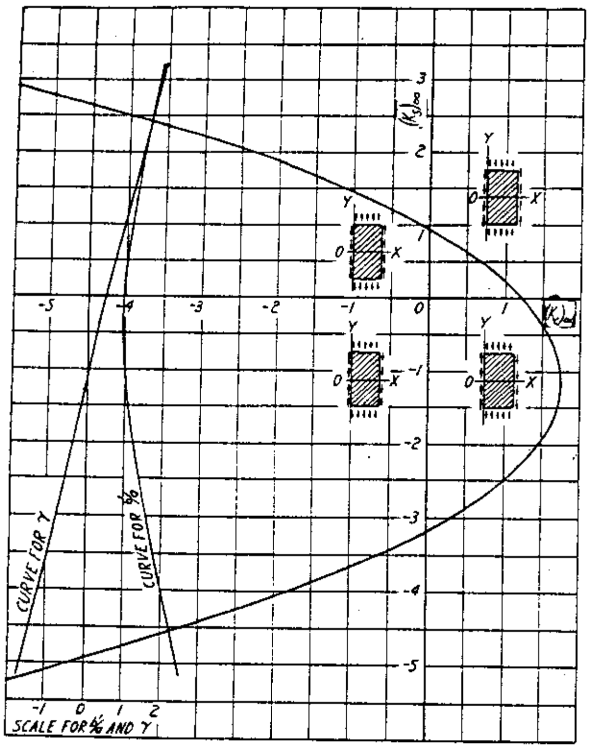
(i)
3-PLY (1:2:1) β = 15°



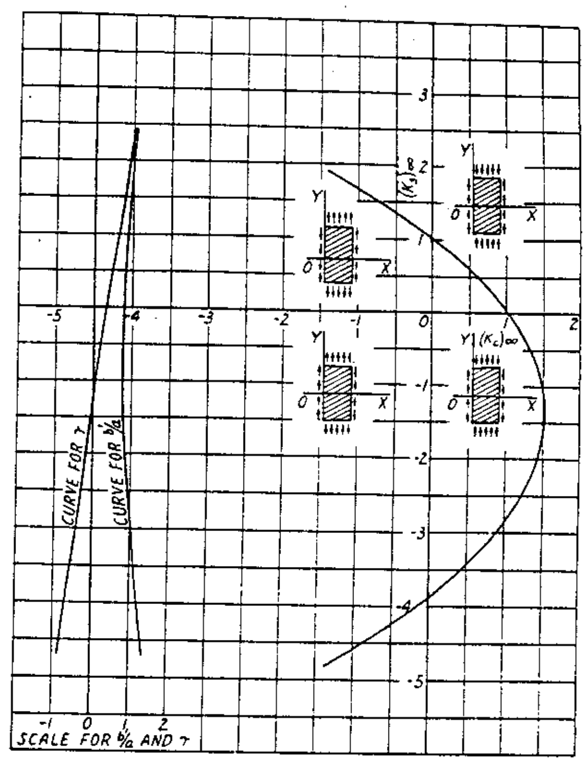
(j)
3-PLY (1:2:1) β = 30°

Z M 50604 F

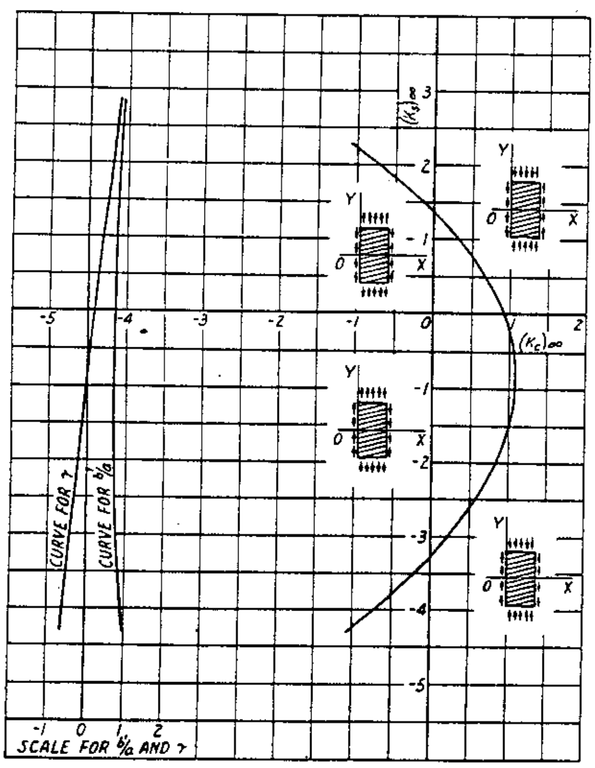
Figure 2–31 (*g, h, i, j*)—Continued.



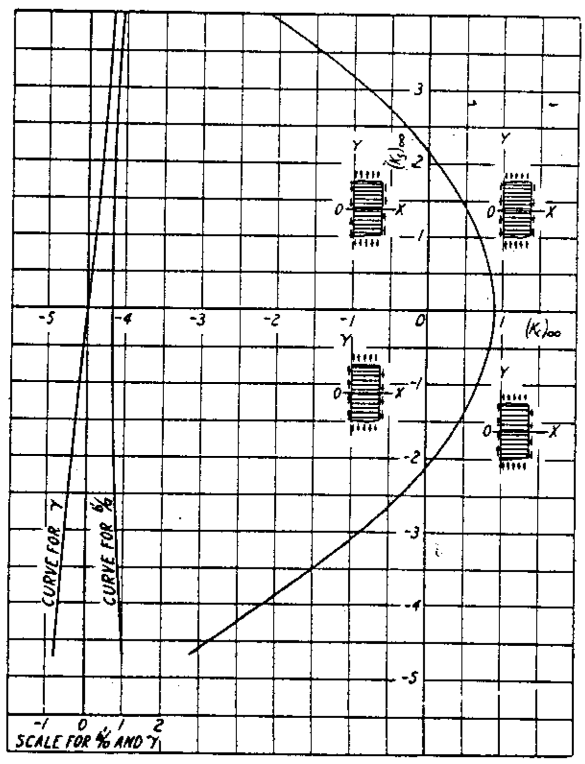
(k)
3-PLY (1:2:1) β-45°



(m)
3-PLY (1:2:1) β=60°

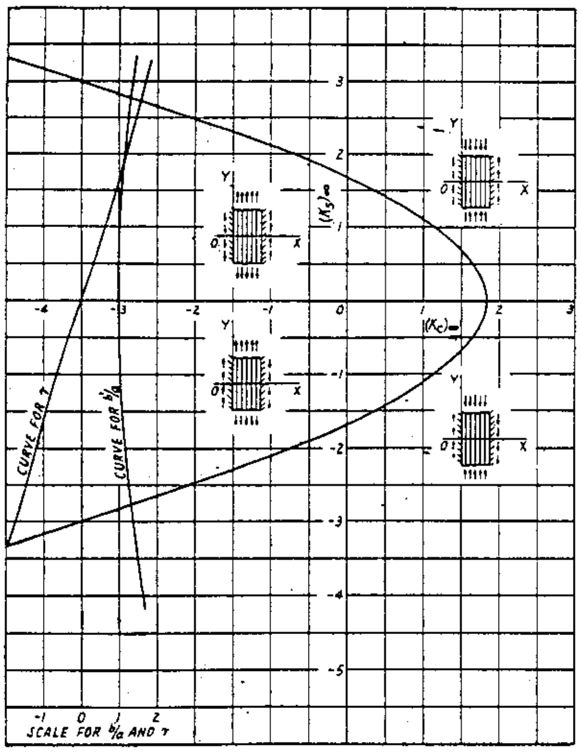


(n)
3-PLY (1:2:1) β=75°

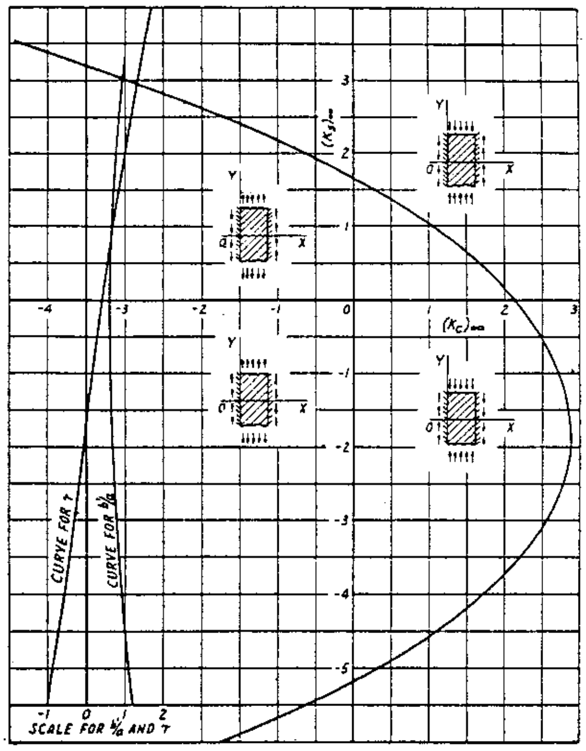


(p)
3-PLY (1:2:1) β-90°

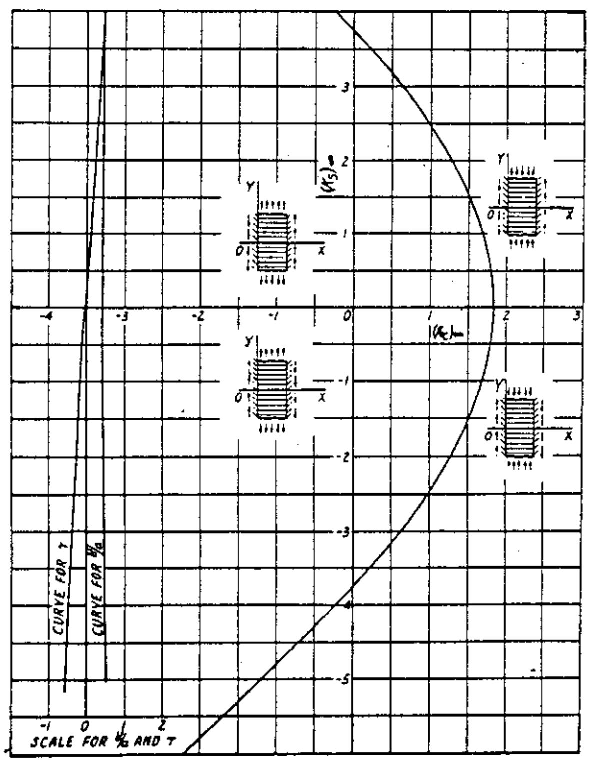
Figure 2–31 (*k*, *m*, *n*, *p*)—Continued.



(a)
3-PLY (1:2:1) β = 0°

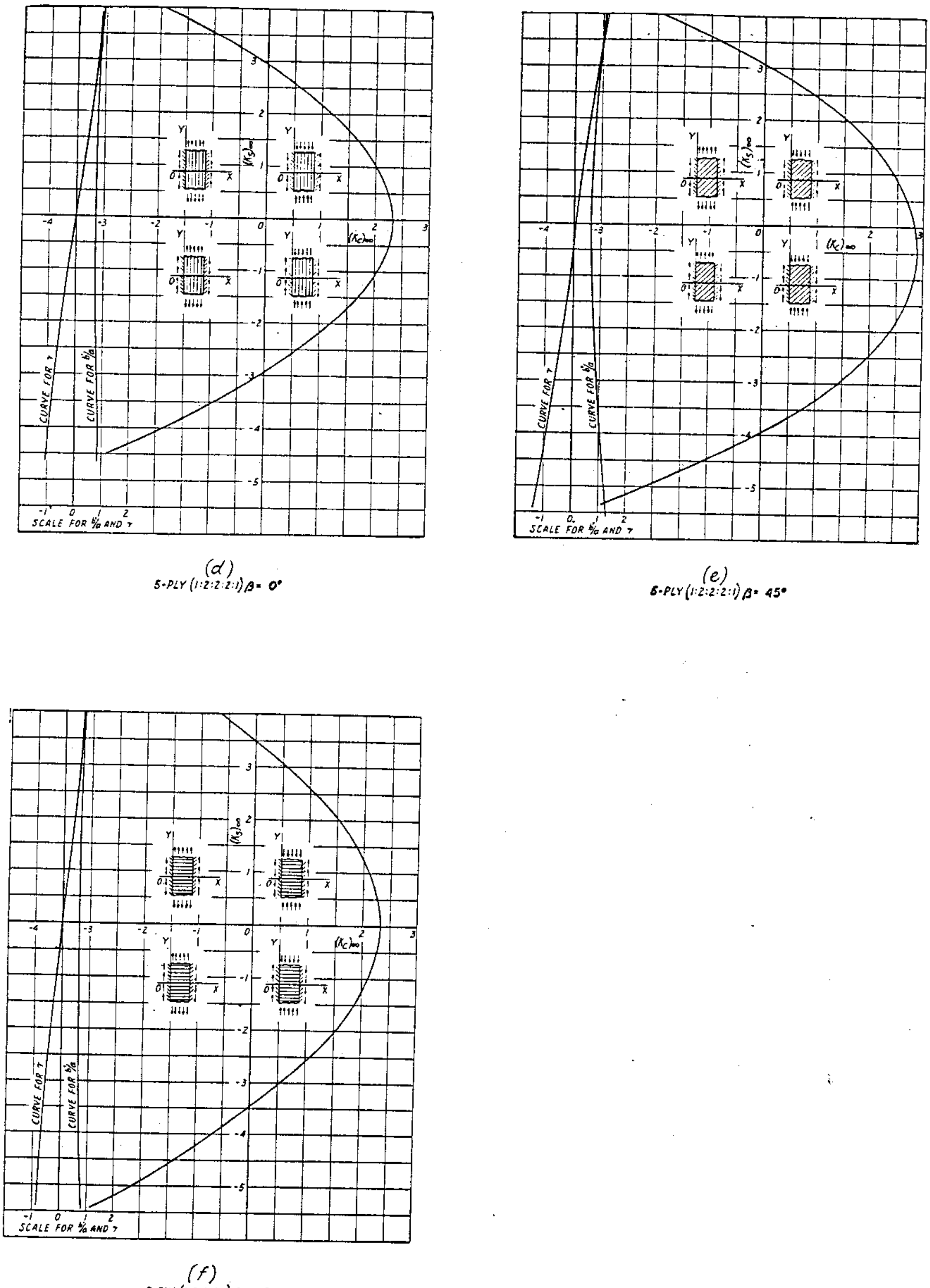


(b)
3-PLY (1:2:1) β = 45°



(c)
FIG. 37
3-PLY (1:2:1) β = 90°

Figure 2–32 (a, b, c). Curves of critical buckling constants for infinitely long rectangular plywood panels under combined loading with edges clamped. β = angle between face grain and direction of applied stress. Three-ply construction.

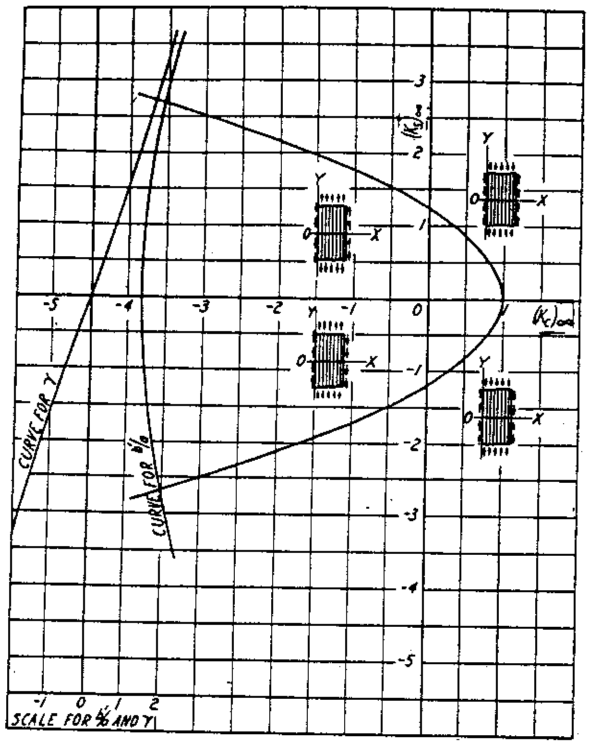


(d) 5-PLY (1:2:2:2:1) β = 0°

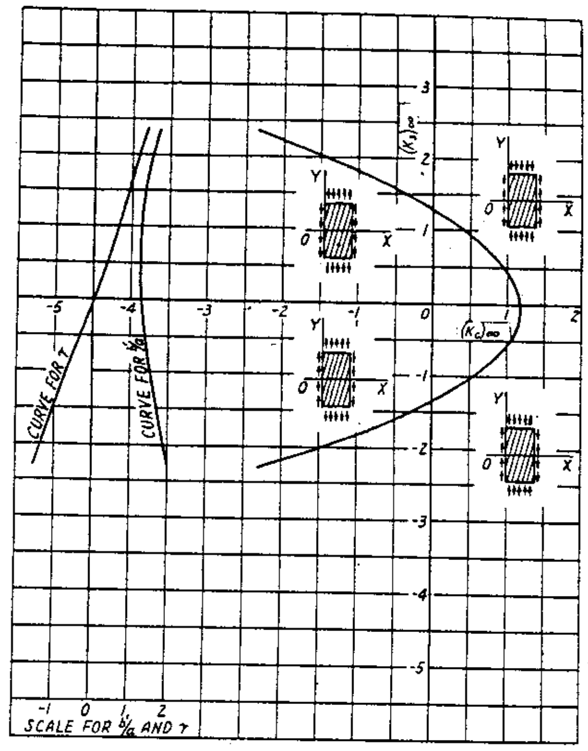
(e) 5-PLY (1:2:2:2:1) β = 45°

(f) 5-PLY (1:2:2:2:1) β = 90°

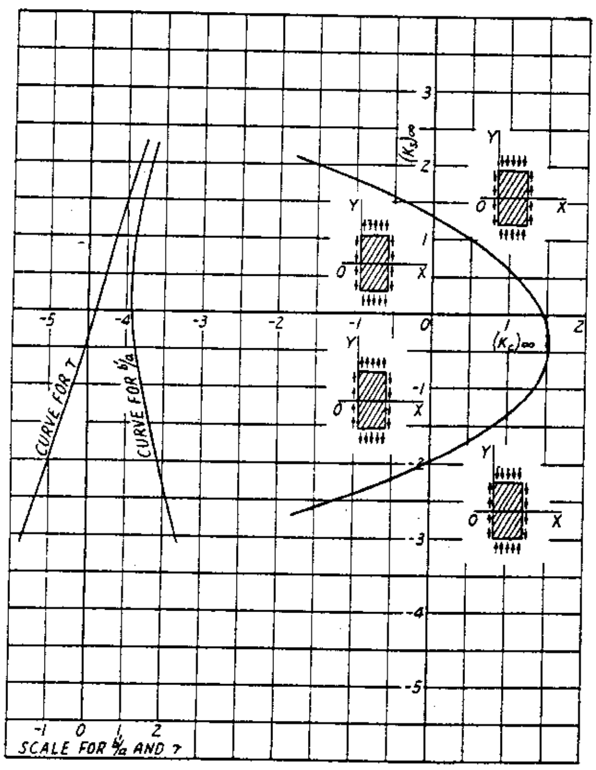
Figure 2–33 (d, e, f). Curves of critical buckling constants for infinitely long rectangular plywood panels under combined loading with edges clamped. β = angle between face grain and direction of applied stress. Five-ply construction.



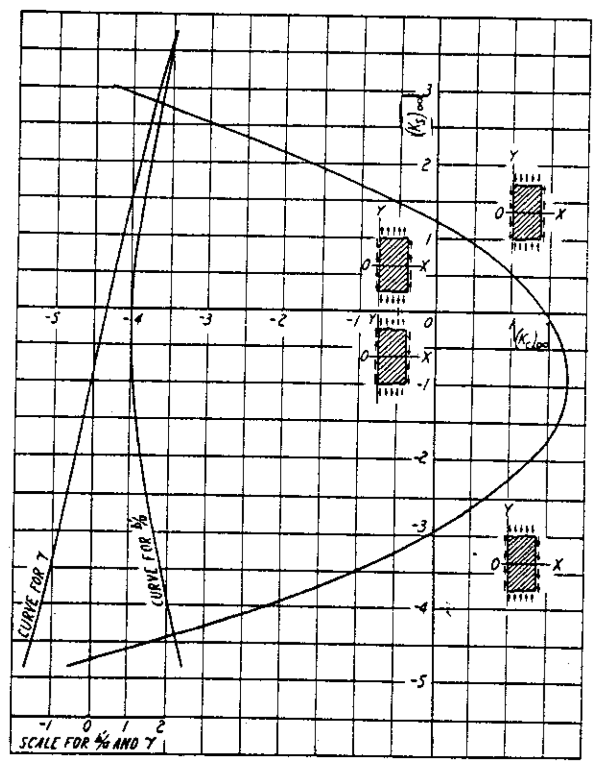
(a)
5-PLY (1:1:1:1:1) $\beta = 0°$



(b)
5-PLY (1:1:1:1:1) $\beta = 15°$

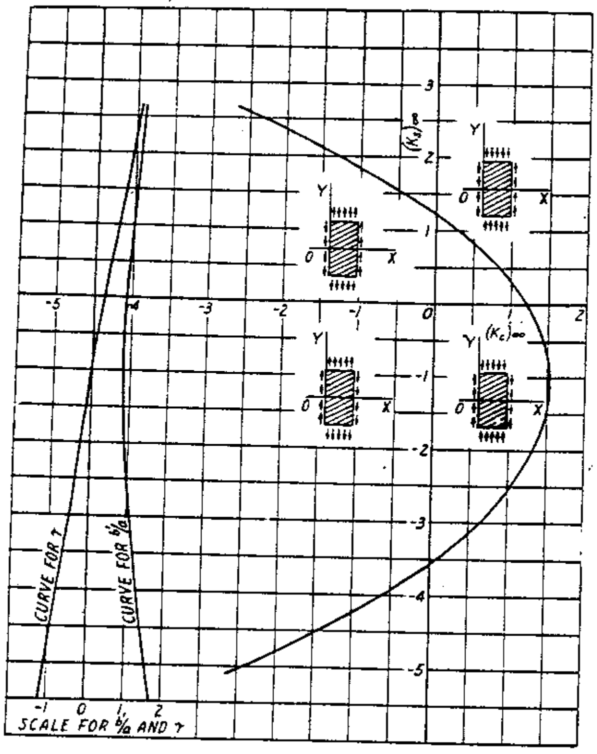


(c)
5-PLY (1:1:1:1:1) β=30°

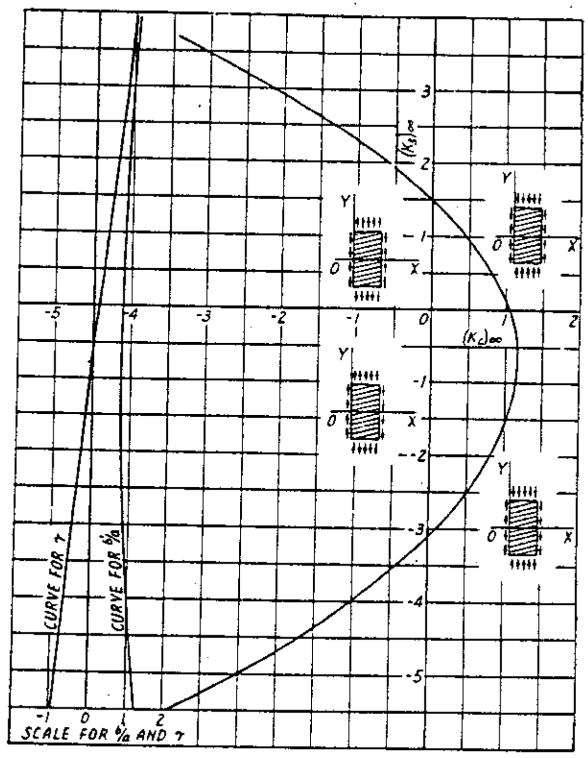


(d)
5-PLY (1:1:1:1:1) β=45°

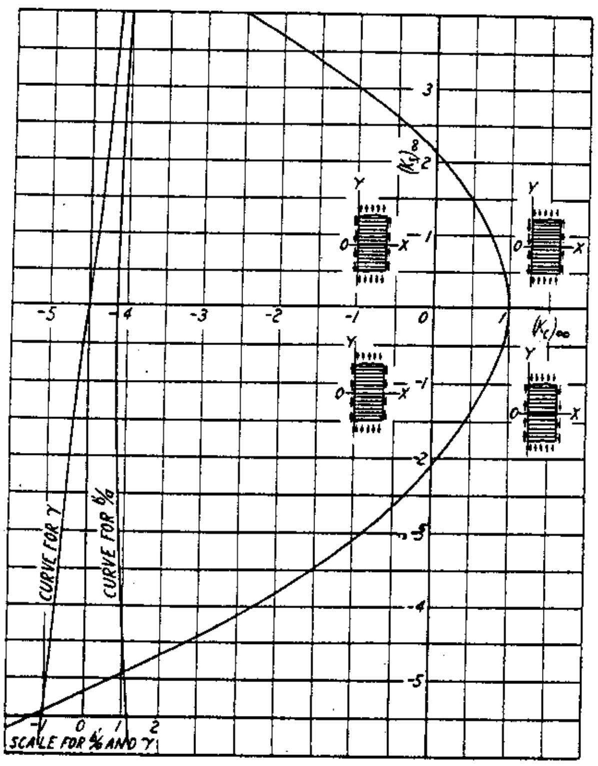
Figure 2–34 (a, b, c, d). Curves of critical buckling constants for infinitely long rectangular plywood panels under combined loading. Edges simply supported. β = angle between face grain and direction af applied stress. Five-ply construction.



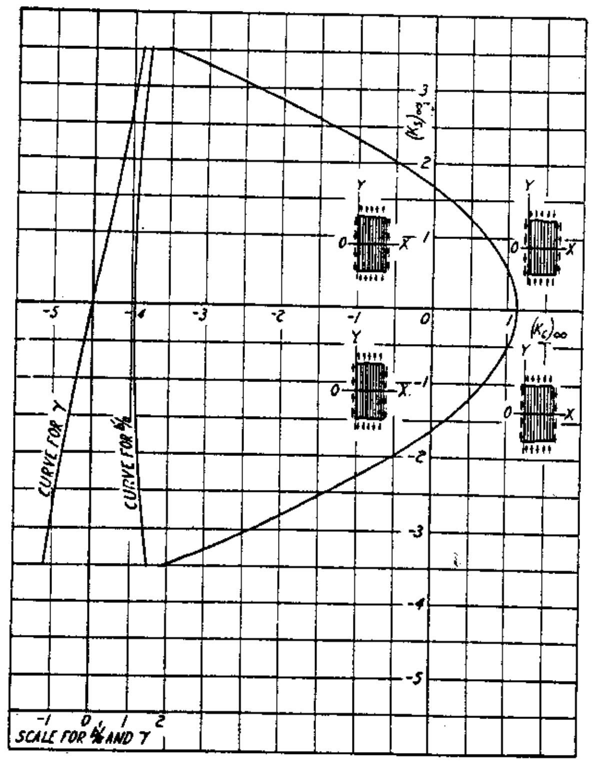
(e)
5-PLY (1:1:1:1:1) $\beta = 60°$



(f)
5-PLY (1:1:1:1:1) $\beta = 75°$



(g)
5-PLY (1:1:1:1:1) $\beta = 90°$



(h)
5-PLY (1:2:2:2:1) $\beta = 0°$

Figure 2–34 (*e*, *f*, *g*, *h*)—Continued.

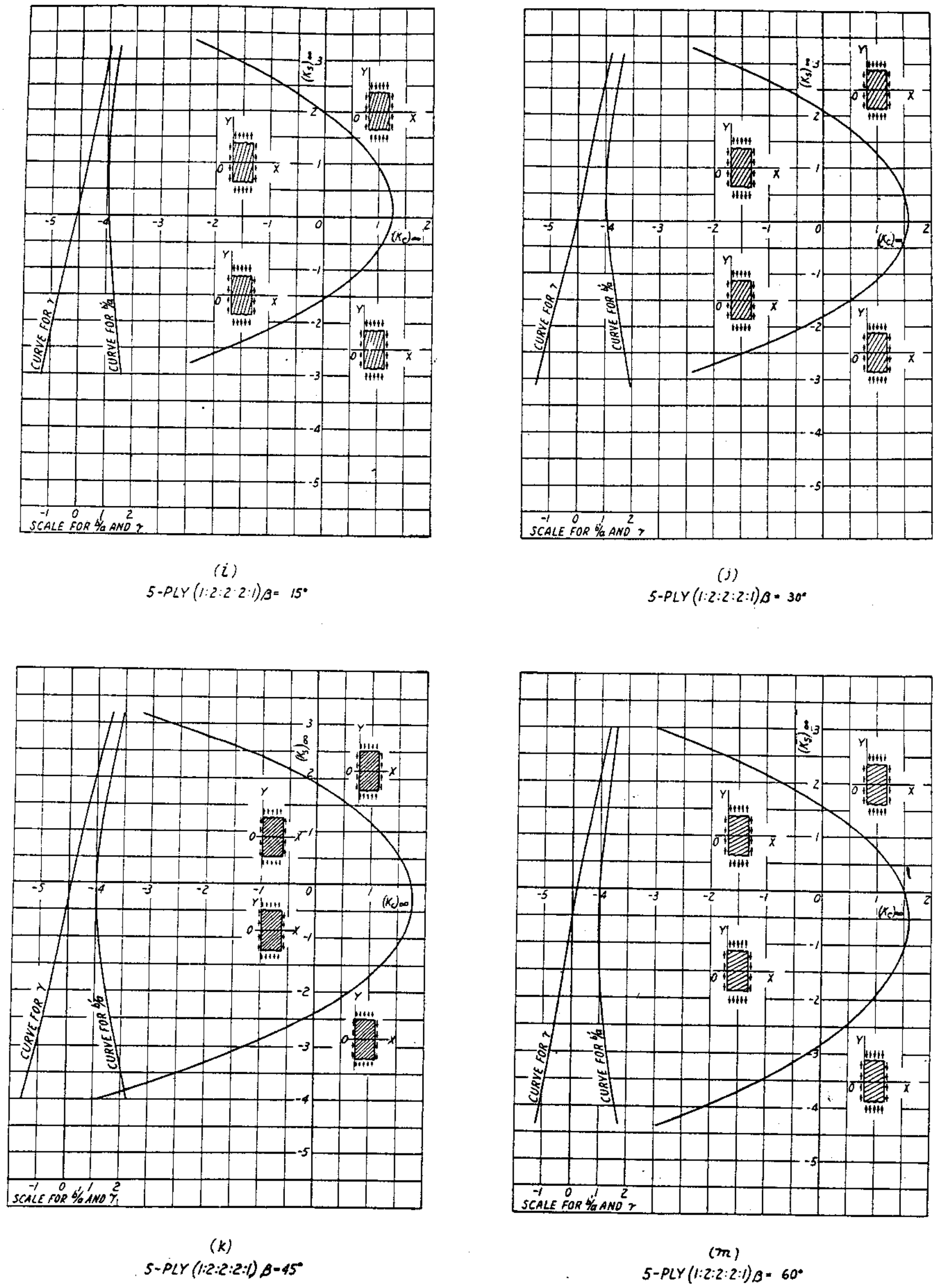
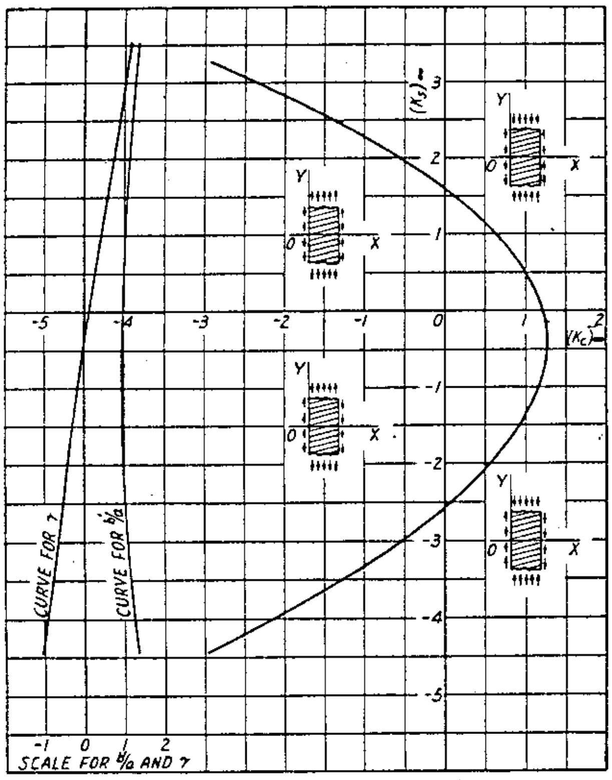
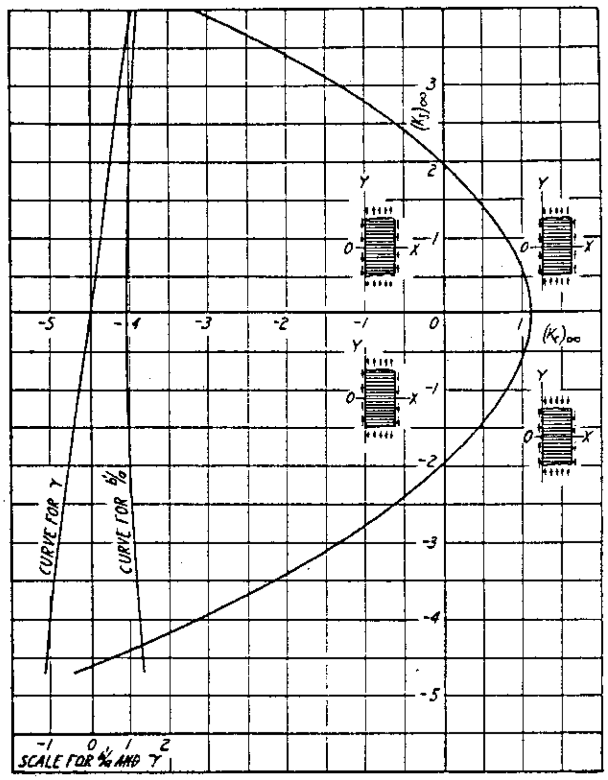


Figure 2-34 (*i, j, k, m*)—Continued.



(n)
5-PLY (1:2:2:2:1) β = 75°



(p)
5-PLY (1:2:2:2:1) β = 90°

Figure 2-34 (*n*, *p*)—Continued.

recting $K_{s\,\infty}$ values or $K_{c\,\infty}$ values in table 2–14 for panel size by means of figure 2–37. In using these figures b'/a is first obtained from table 2–14 and b/b' computed. For a more exact determination of K_s or K_c or to determine these buckling constants for a plywood construction different from those specified in AN–NN–P–511b or figures 2–30 to 2–36, calculate $\frac{E_{fw}}{E_{fw}+E_{fx}}$ in accordance with section 2.52, read $K_{s\,\infty}$ or $K_{c\,\infty}$ and b'/a from figures 2–38 to 2–41 and correct for panel size by means of figure 2–37.

2.7151. *Combined compression (or tension) and shear.* The analytical method of determining the critical buckling stresses for rectangular panels subjected to combined loadings is quite complicated, and only the graphical solutions for a few types of plywood construction are given in figures 2–30 to 2–36.

When the plywood construction being used is not the same as any of those illustrated, its buckling constants may be obtained by a straight line interpolation (or extrapolation), on the basis of $\frac{E_{fw}}{E_{fw}+E_{fx}}$, of the buckling constants for two plywood constructions whose values of the ratio $\frac{E_{fw}}{E_{fw}+E_{fx}}$ are fairly close to that of the plywood under consideration. The values of these ratios for the plywood constructions considered in figures 2–30 to 2–36 may be calculated with sufficient accuracy by assuming $E_T=0.05\ E_L$.

These figures apply to panels of infinite length and values of the buckling constants from the curves must be corrected for actual panel length. Values of the shear constant $K_{s\,\infty}$ and the compression constant $K_{c\,\infty}$ are indicated on the vertical and horizontal axes, respectively. The points at which the curve crosses these axes give the values of $K_{s\,\infty}$ or $K_{c\,\infty}$ at which buckling will just occur in a panel of infinite length in either pure shear or pure compression. The particular combination of stresses represented by each of the four quadrants is shown by the small stress sketches. Buckling will occur under these combined stresses whenever the location of a point $K_{s\,\infty}$, $K_{c\,\infty}$, lies on or outside the curve.

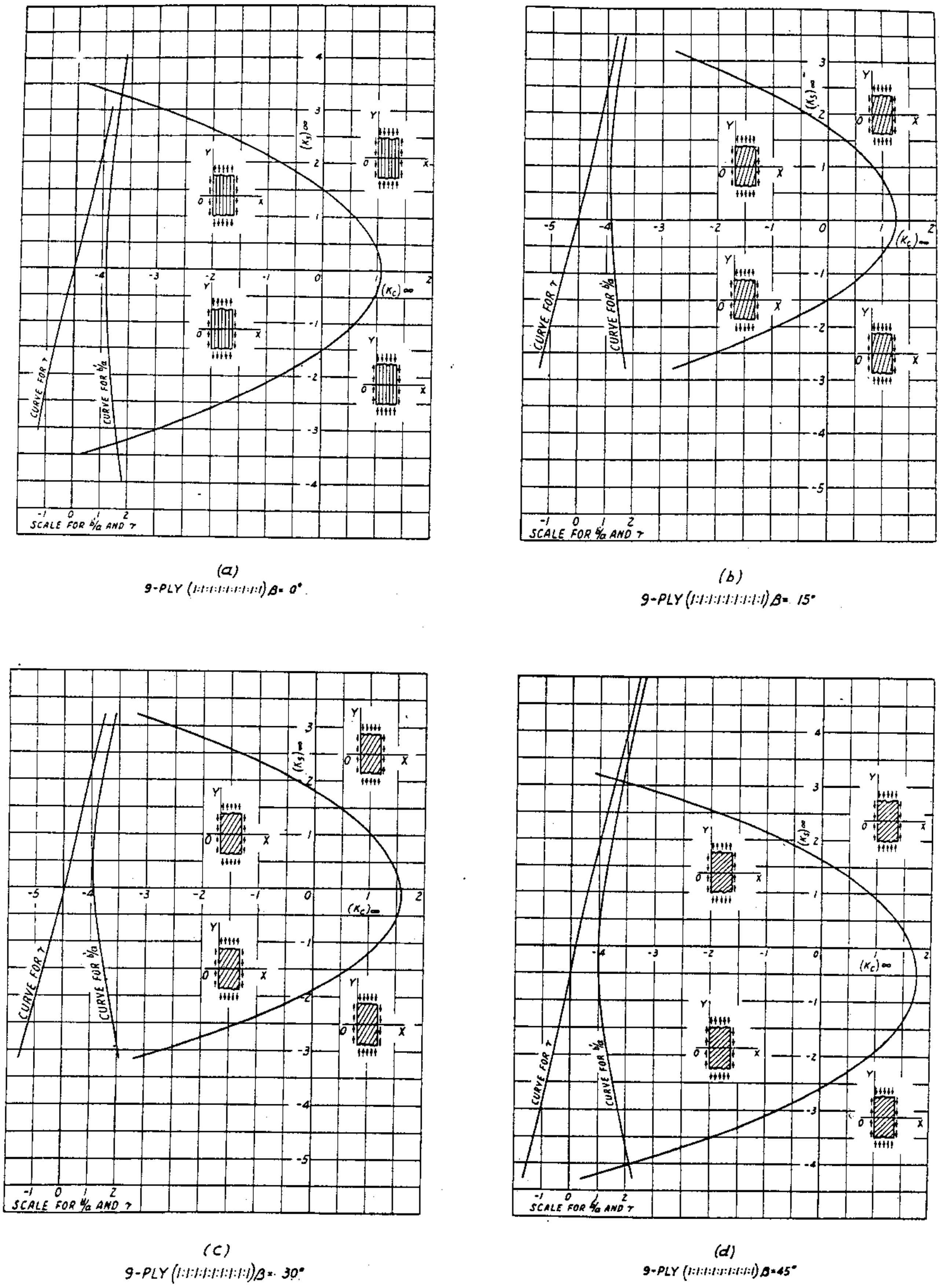
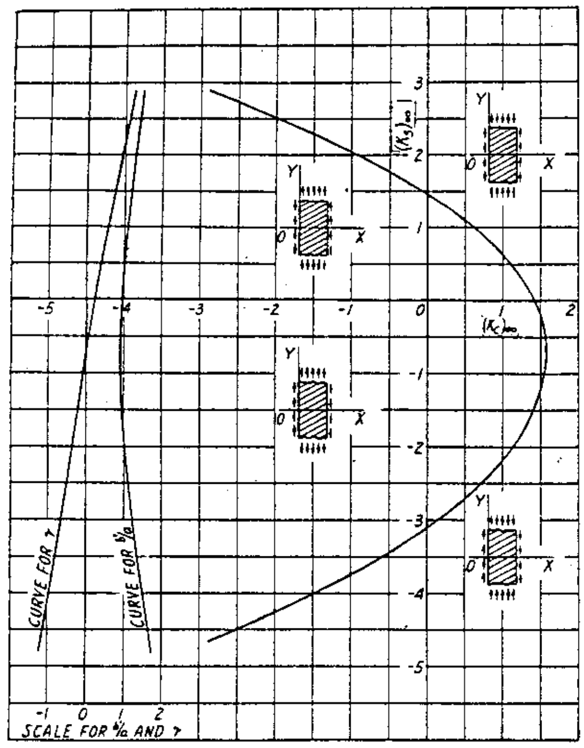
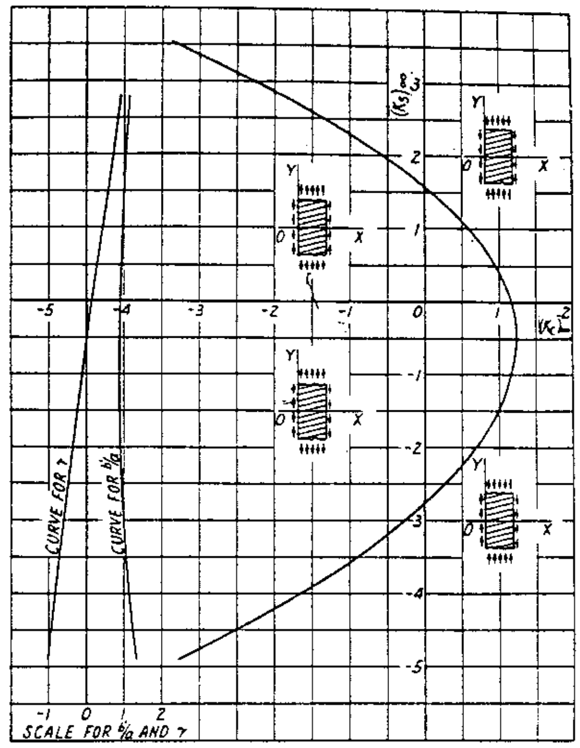


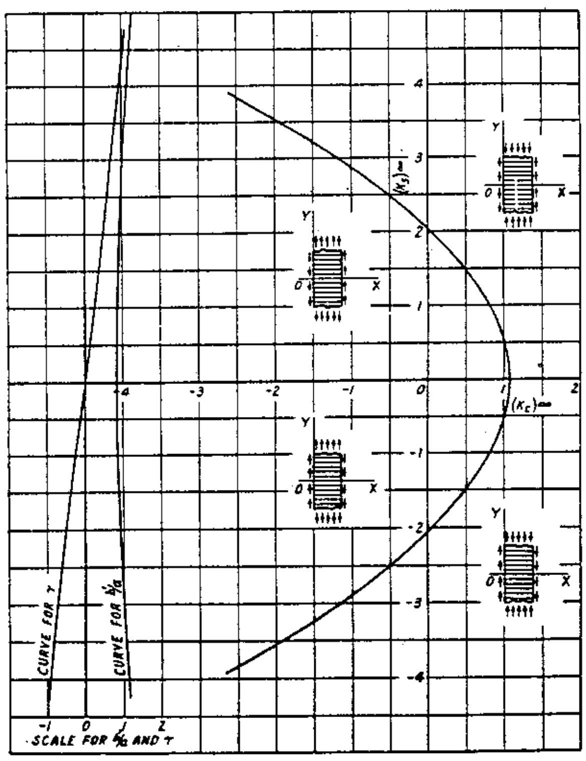
Figure 2–35 (a, b, c, d). Curves of critical buckling constants for infinitely long rectangular plywood panels under combined loading. Edges simply supported. β = angle between face grain and direction of applied stress. Nine-ply construction.



(e)
9-PLY (1:1:1:1:1:1:1:1:1) β = 60°

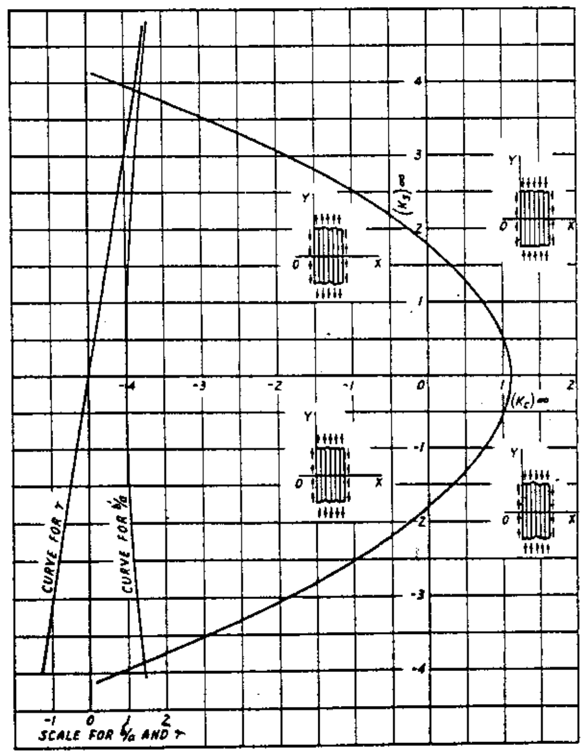


(f)
9-PLY (1:1:1:1:1:1:1:1:1) β = 75°



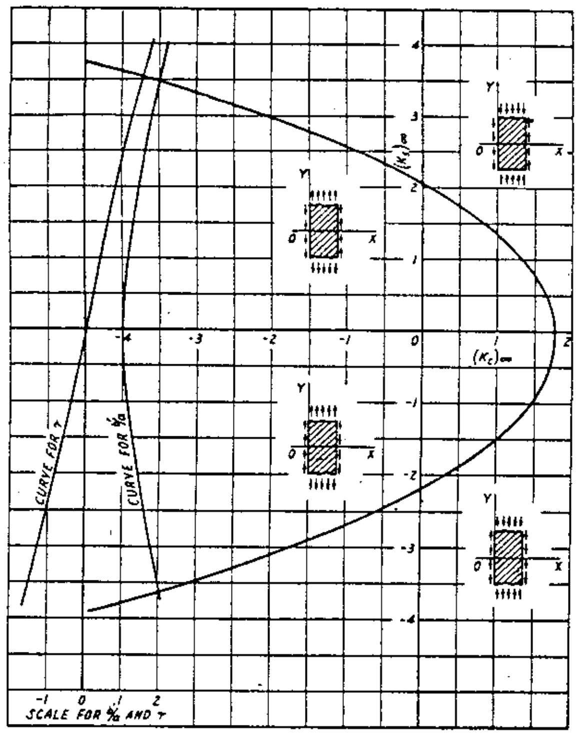
Z M 50613 F

(g)
9-PLY (1:1:1:1:1:1:1:1:1) β = 90°

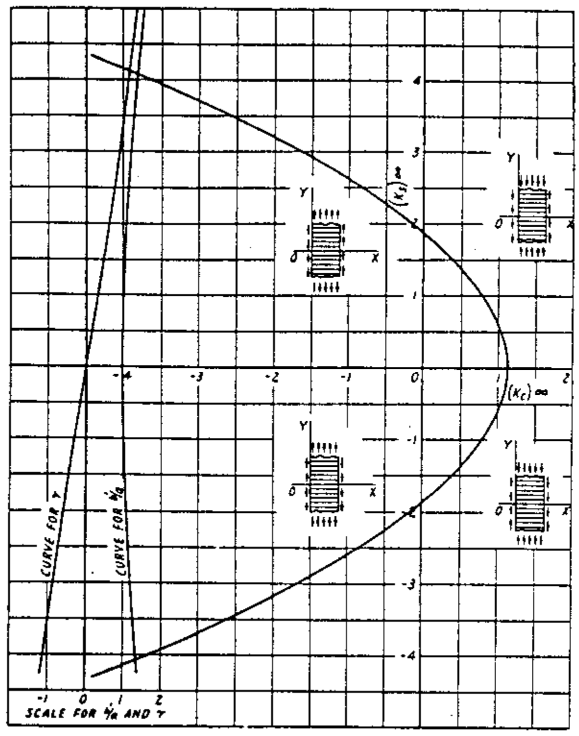


(h)
9-PLY (1:2:2:2:2:2:2:2:1) β = 0°

Figure 2–35 (*e, f, g, h*)—Continued.

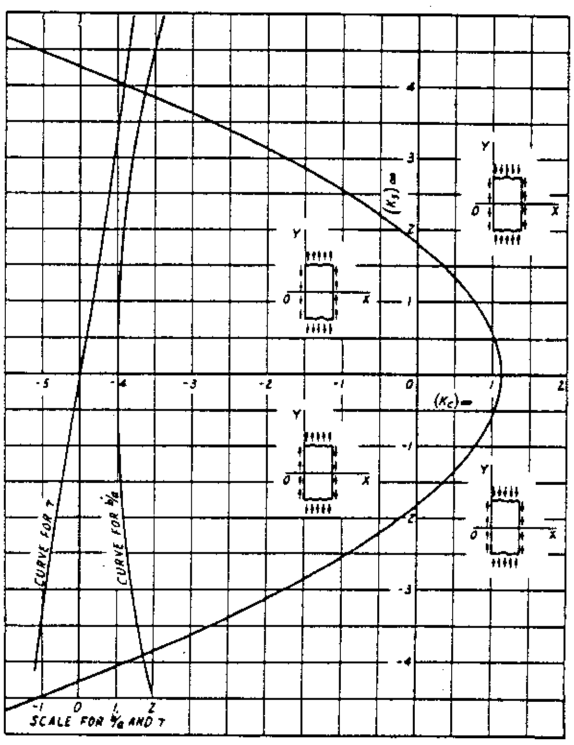


(i)
9-PLY (1:2:2:2:2:2:2:2:1) β=45°

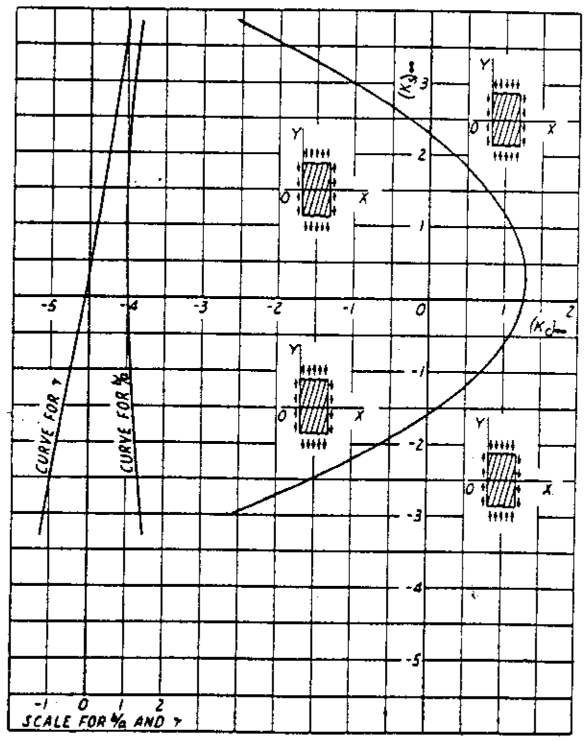


(j)
9 PLY (1:2:2:2:2:2:2:2:1) β=90°

Figure 2–35 (i, j)—Continued.



(a)
∞-PLY β=0° AND β=90°



(b)
∞-PLY β=15°

Figure 2–36 (a, b). Curves of critical bucking constants for infinitely long rectangular plywood panels under combined loading. Edges simply supported. β=angle between face grain and direction of applied stress. Infinite-ply construction.

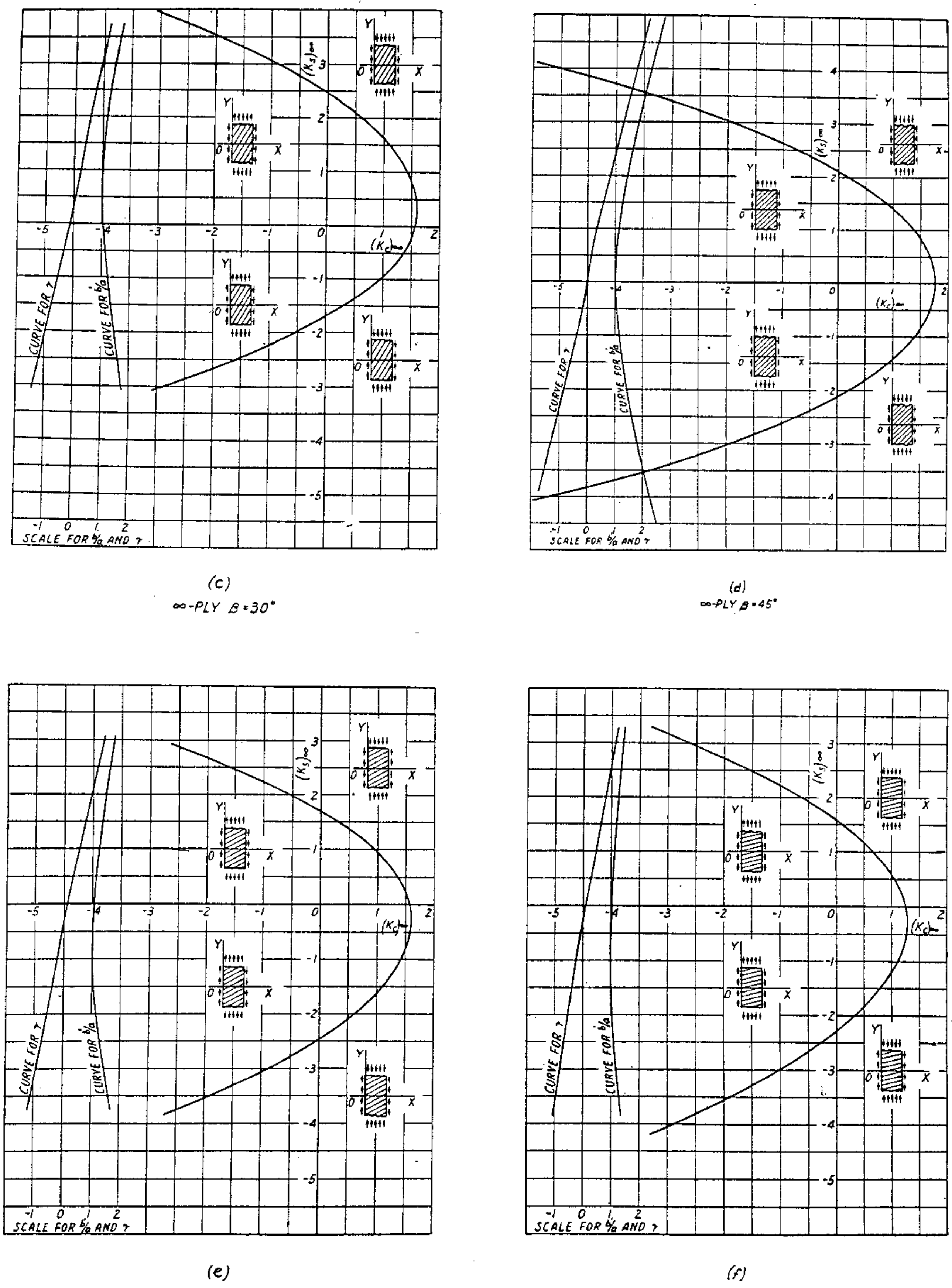
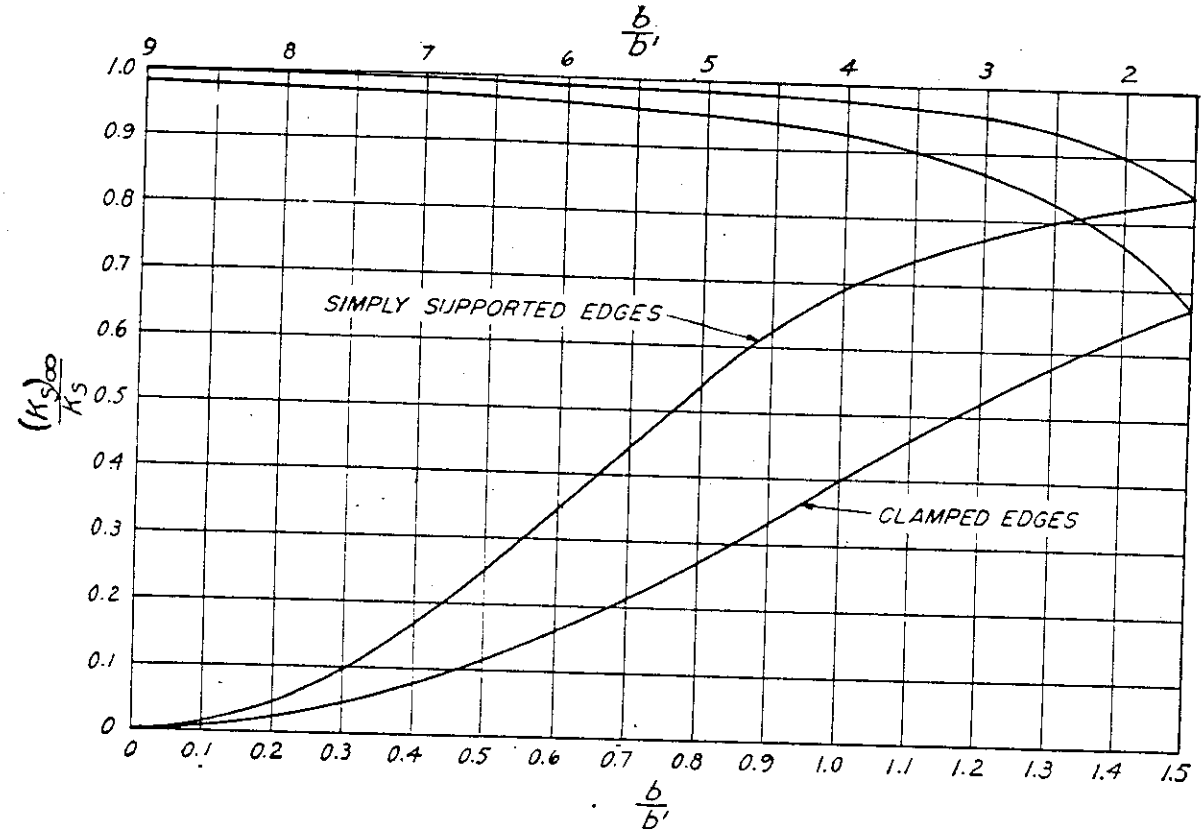


Figure 2–36 (c, d, e, f)—Continued.



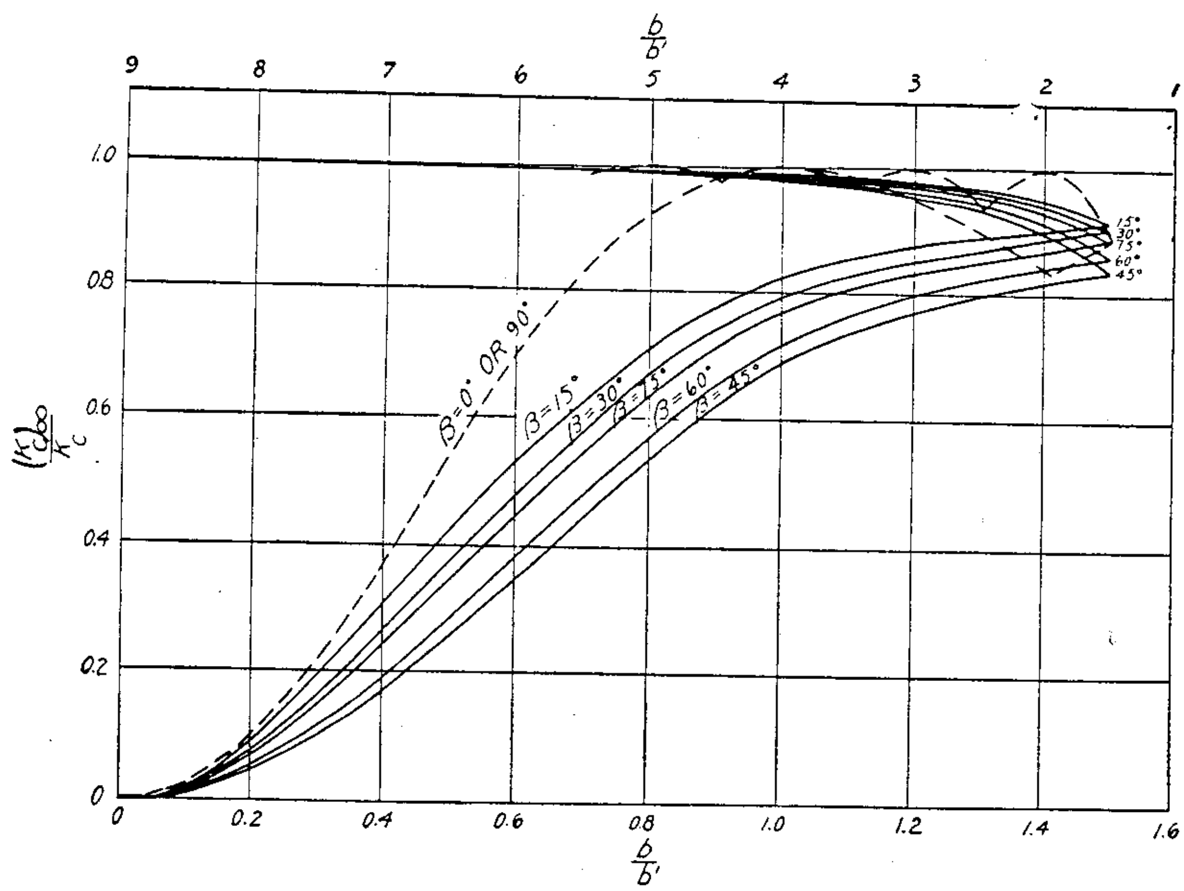
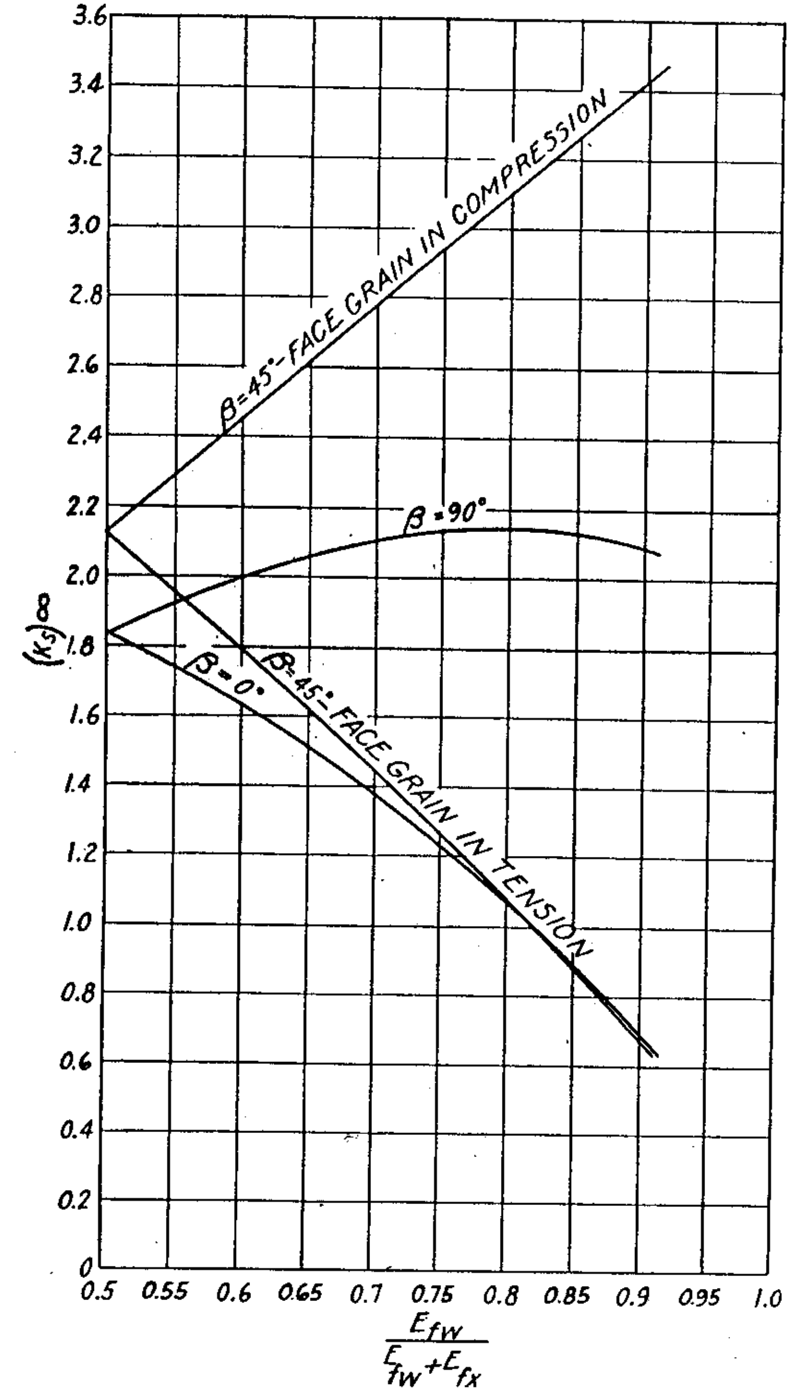


Figure 2-37. Corrections for panel size: Top, when β=0°, 45°, or 90° and panel is subjected to shear stress; bottom, when the panel is subjected to compression with the edges simply supported and β=0° or 90° is a computed curve (ref. 2-50).



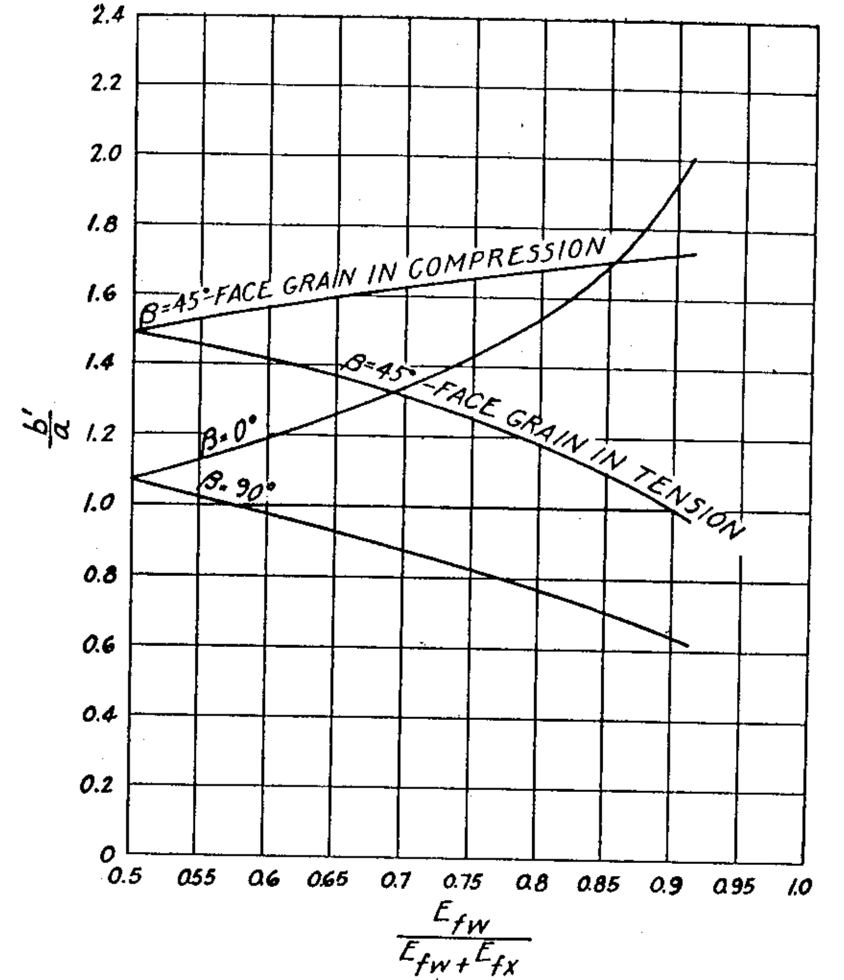


Figure 2–38 (left). Buckling of infinitely long plates of symmetrical construction under uniform shear for determination of $(K_s)_\infty$. Edges simply supported.

Figure 2–39. Buckling of infinitely long plates of symmetrical construction under uniform shear for determination of b′/a. Edges simply supported.

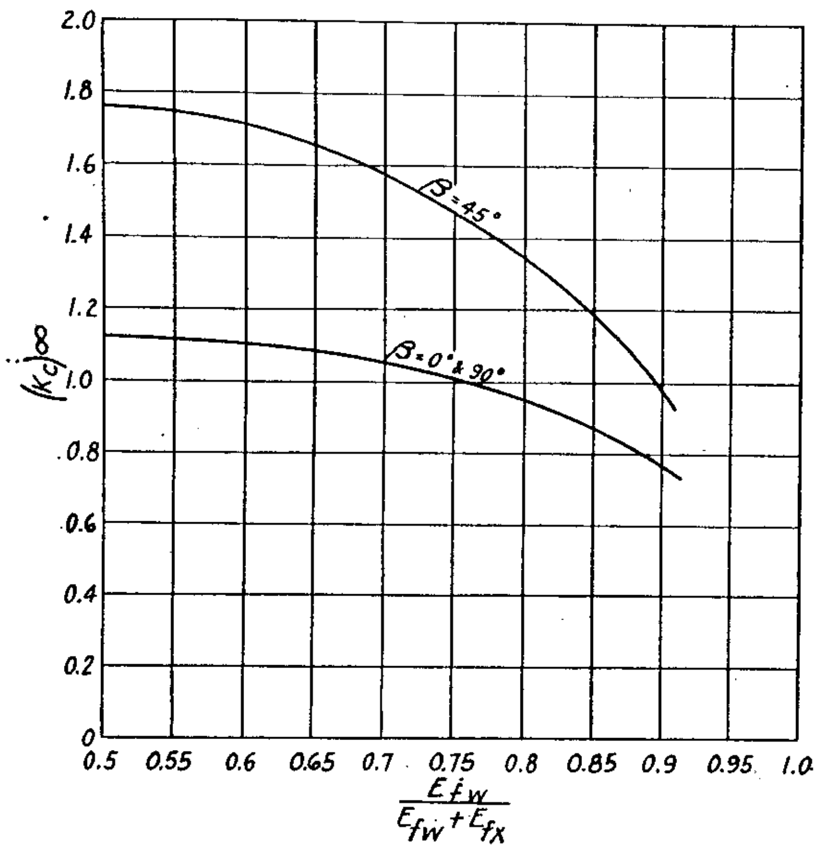


Figure 2-40. Buckling of infinitely long plates of symmetrical construction under uniform compression for determination of $(K_c)_\infty$. Edges simply supported.

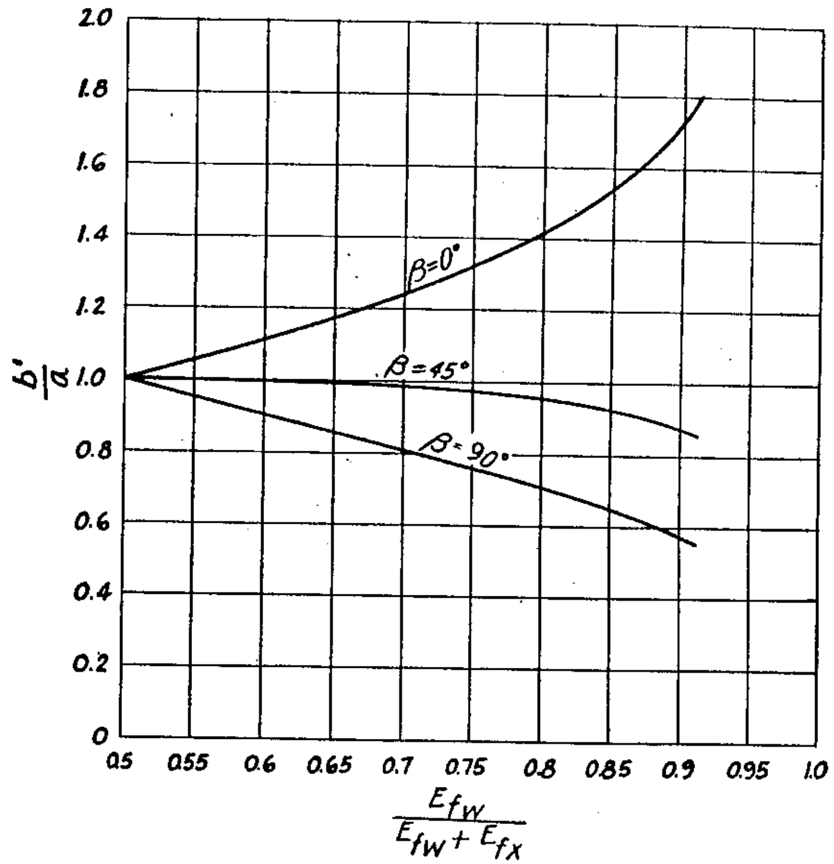


Figure 2-41. Buckling of infinitely long plates of symmetrical construction under uniform compression for determination of b'/a. Edges simply supported.

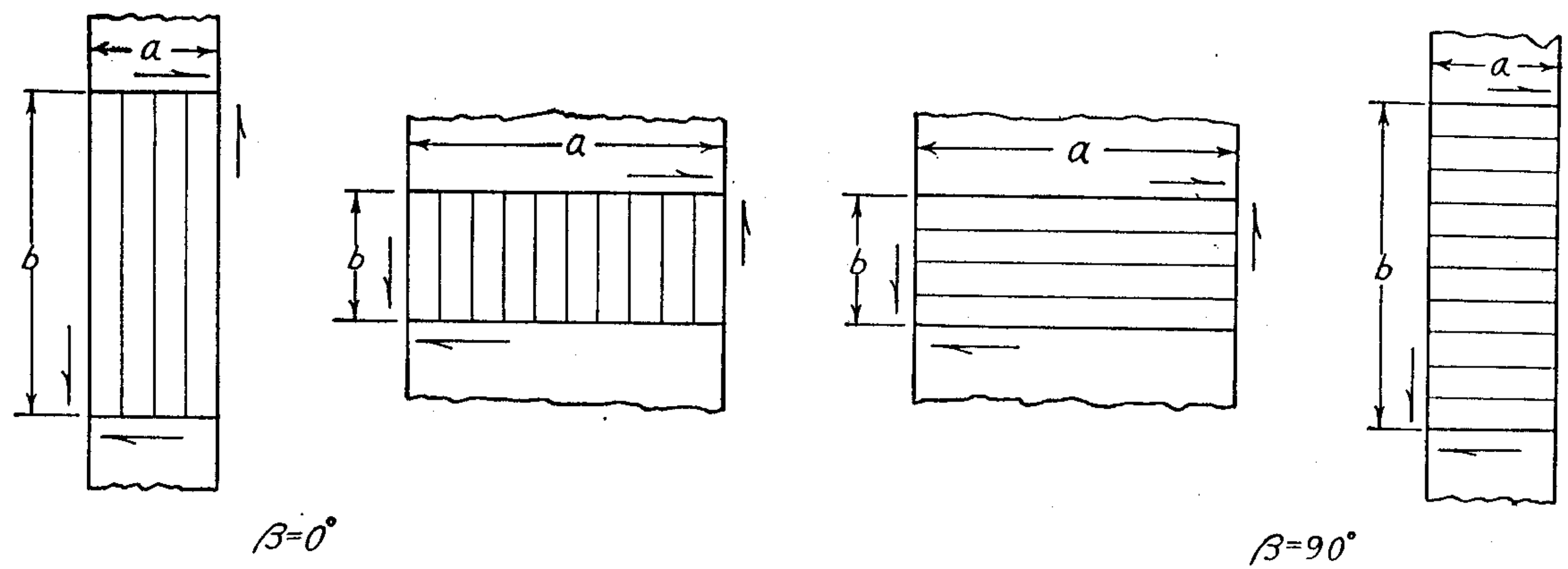


Figure 2–42. In panels loaded in shear, a may be a dimension of either edge. For β=0°, face grain is perpendicular to a; for β=90°, face grain is parallel to a.

Table 2–14. Buckling constants for plywood [1]

Face grain angle	Shear								Compression				
	0°		90°		45°				0°	0° and 90°	90°	45°	
					Face grain in tension		Face grain in compression						
Nominal thickness	$(K_s)_\infty$	b'/a	$(K_s)_\infty$	b'/a	$(K_s)_\infty$	b'/a	$(K_s)_\infty$	b'/a	b'/a	$(K_c)_\infty$	b'/a	$(K_c)_\infty$	b'/a
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
THREE-PLY													
Inch													
0. 035	0. 60	2. 13	2. 05	0. 60	0. 57	0. 95	3. 50	1. 74	1. 88	0. 71	0. 53	0. 86	0. 84
. 070	. 80	1. 79	2. 11	. 68	. 78	1. 06	3. 34	1. 72	1. 62	. 82	. 62	1. 08	. 91
. 100	. 75	1. 85	2. 10	. 66	. 73	1. 03	3. 38	1. 72	1. 67	. 80	. 60	1. 03	. 90
. 125	. 88	1. 70	2. 13	. 71	. 87	1. 10	3. 27	1. 71	1. 55	. 87	. 65	1. 17	. 93
. 155	. 94	1. 65	2. 14	. 72	. 93	1. 12	3. 22	1. 70	1. 50	. 89	. 67	1. 23	. 94
. 185	. 95	1. 64	2. 14	. 73	. 94	1. 13	3. 22	1. 70	1. 50	. 90	. 68	1. 24	. 94
FIVE-PLY													
0. 160	1. 25	1. 42	2. 13	0. 83	1. 29	1. 26	2. 91	1. 66	1. 31	1. 02	0. 77	1. 49	0. 97
. 190	1. 35	1. 36	2. 12	. 87	1. 41	1. 30	2. 81	1. 64	1. 26	1. 04	. 80	1. 56	. 98
. 225	1. 37	1. 35	2. 11	. 88	1. 43	1. 31	2. 79	1. 63	1. 25	1. 05	. 81	1. 57	. 98
. 250	1. 30	1. 38	2. 12	. 85	1. 35	1. 28	2. 86	1. 64	1. 28	1. 04	. 79	1. 53	. 98
. 315	1. 29	1. 39	2. 12	. 85	1. 34	1. 28	2. 87	1. 65	1. 29	1. 03	. 78	1. 52	. 98
. 375	1. 48	1. 28	2. 08	. 92	1. 57	1. 36	2. 66	1. 60	1. 19	1. 08	. 84	1. 64	. 99
SEVEN-PLY (all plies of equal thickness)													
Any	1. 40	1. 32	2. 10	0. 89	1. 46	1. 32	2. 75	1. 62	1. 23	1. 06	0. 82	1. 59	0. 99
NINE-PLY (all plies of equal thickness)													
Any	1. 52	1. 26	2. 06	0. 94	1. 63	1. 37	2. 60	1. 59	1. 17	1. 09	0. 86	1. 66	0. 99
ELEVEN-PLY (all plies of equal thickness)													
Any	1. 59	1. 22	2. 03	0. 96	1. 72	1. 40	2. 52	1. 58	1. 14	1. 10	0. 88	1. 70	0. 99

[1] The buckling constants listed in this table correspond only to the plywood thicknesses and constructions listed in table 2-13 that correspond to Army-Navy specification AN-NN-P-511b (Plywood and Veneer; Aircraft Flat Panel). The values in this table were computed as follows: For each construction given in table 2-13 a value of $\frac{E_{fw}}{E_{fw}+E_{fx}}$ was computed from columns 5 and 6 of table 2-13. These values for each thickness were averaged and the average values were used in entering figures 2-38, 2-39, 2-40, and 2-41 from which the values of this table were obtained. For a more exact determination of these buckling constants or to determine the buckling constants of a plywood construction different from those specified in AN-NN-P-511b, see section 2.7.

The curve marked b'/a is the ratio of half the wave length (b') of a buckle in an infinitely long panel to its width (a). This ratio is to be used in conjunction with figures 2–37 to 2–41 in obtaining the correction factors for panels of finite length to be applied to $K_{s\infty}$.

The curves in figures 2–30 to 2–36 marked γ give the slope of the panel wrinkles with respect to the O-X axis indicated on the stress sketches.

The procedure in the use of these figures is as follows:

(1) From the analysis the shear stress (f_s) and the compression (f_c) or tension stress ($-f_c$) acting on a particular plywood panel will have been calculated.

(2) Determine the ratio f_s/f_c and, on the figure giving the same plywood construction and angle β, draw a line through the origin having a slope (positive or negative) equal to this ratio. When the plywood construction is not the same as that given in the figures, this procedure for determining the buckling constant will have to be run through on the two most similar constructions and an interpolation of the results made on the basis of

$$\frac{E_{fw}}{E_{fw}+E_{fx}}.$$

(3) The point at which the constructed line crosses the curve gives the critical buckling constants $K_{s\infty}$ and $K_{c\infty}$ at which an infinitely long panel will just buckle when subjected to the same ratio of shear to compression that exists on the panel in question.

(4) Read the value of b'/a for the point on the b'/a curve which is obtained by projecting horizontally from $K_{s\infty}$ determined in step (3).

(5) From the panel dimensions compute b' and b/b'.

(6) Figures 2–37 to 2–41 will give the ratio of $K_s/K_{s\infty}$ from which the value of K_s can be computed (K_s is always taken as positive).

(7) The critical buckling shear stress (F_{scr}) may then be determined by equation (2:80). This represents the maximum allowable shear stress which the panel in question can sustain without buckling when subjected simultaneously to a compressive stress equal to that given in step (1).

2.72. Strength After Buckling.

2.721. *General.* Plywood panels may sustain greater loads than those sufficient to cause buckling. When buckling takes place the stresses within the panel are redistributed, the maximum stresses occurring at the edges. The panel will continue to accept load until these stresses reach the ultimate value. The load at failure is obtained from empirical curves in which the ratio of the average stress at failure to the ultimate strength of the plywood is plotted against the ratio of the width of the panel to the width of a hypothetical panel that will fail at its buckling load.

2.722. *Compression (β=any angle).* The abscissa of figure 2–43 is obtained from the equation

$$\frac{a}{a_o}=\sqrt{\frac{F_{cu\theta}}{F_{ccr}}} \qquad (2:81)$$

in which $F_{cu\theta}$ is obtained from equation (2:51) or (2:52) and F_{ccr} from equation (2:77) or (2:79). The ordinates give the ratio of the average stress at which failure will occur to the ultimate compressive strength ($F_{cu\theta}$) of the plywood.

2.723. *Shear ($\beta=0°$, 45°, or 90°).* The abscissa of figure 2–44 is obtained from the equation

$$\frac{a}{a_o}=\sqrt{\frac{F_s}{F_{scr}}} \qquad (2:82)$$

in which F_s is obtained from equation (2:50) (2:57), or (2:58) and F_{scr} from equation (2:77) or (2:80). The ordinates gi e the ratio of the average stress at which ailure will take place to the ultimate shear stress (F_s) of the plywood.

2.73. Allowable Shear in Plywood Webs.

2.730. *General.* Beams are required to have a high strength-weight ratio and, therefore, they are generally designed so that they will fail in shear at about the load which will cause bending failures. A higher strength-weight ratio is usually obtained if the beams fail in bending before shear failure can occur.

Plywood when used as webs of beams is subjected to different stress conditions from those when it is used in simple shear frames. It is essential, therefore, that tests to determine the strengths of shear webs be made upon specimen beams designed with flanges only sufficiently strong to hold the load at which shear failure is expected. Plywood webs tested in heavy shear frames with hinged corners will give shear strengths that are too high for direct application to beam design.

In any case where buckling is obtained, the stiffeners must have adequate strength to resist the additional loads due to such buckling, and the webs must be fastened to the flanges in such a manner as to overcome the tendency of the buckles in the web to project themselves into this fastening and cause premature failure (ref. 2–23 and 2–42).

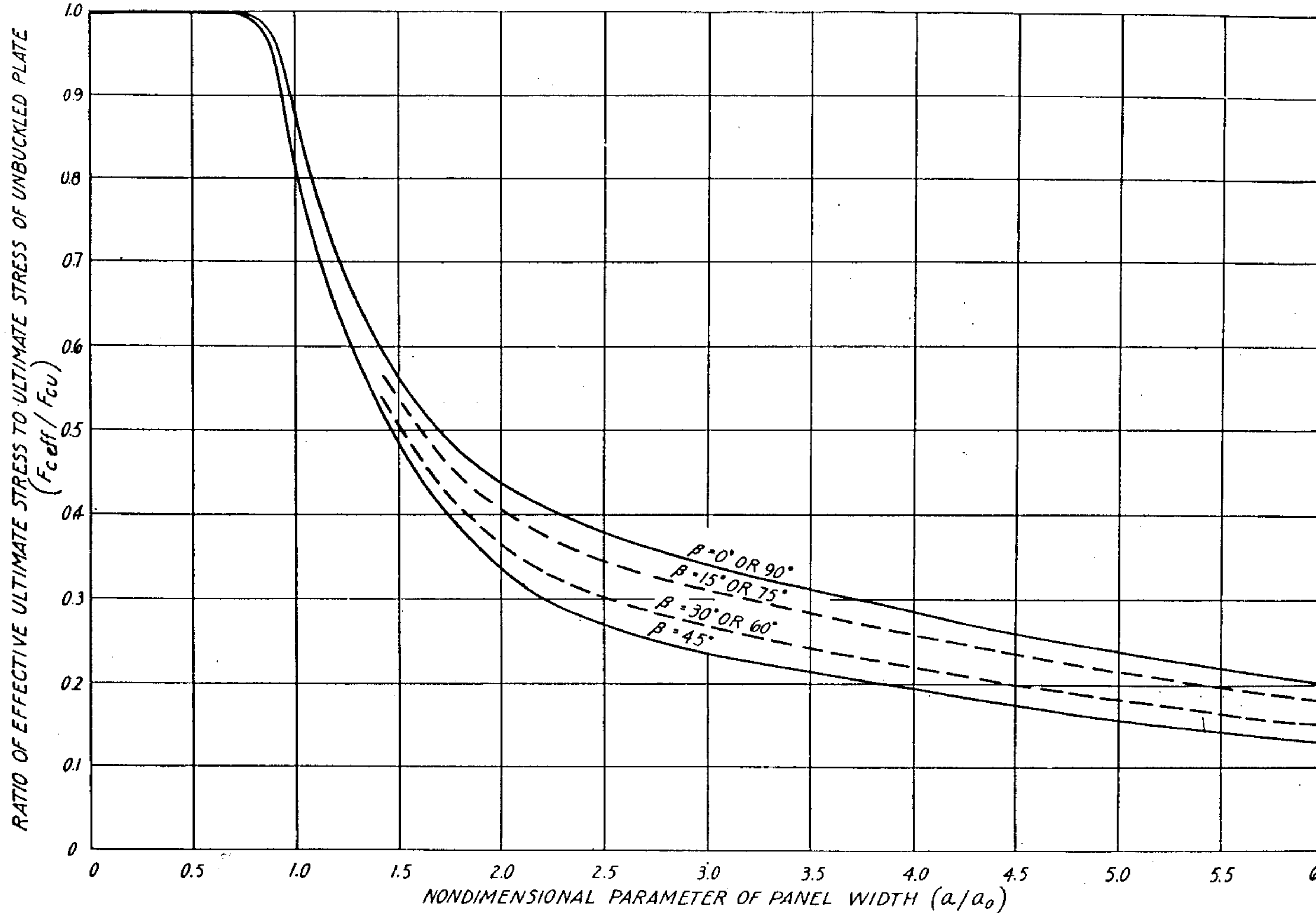


Figure 2–43. Effective width of flat plywood plates in compression at 0°, 15°, 30°, 45°, 60°, 75°, or 90° to direction of face grain. (*Curves based on buckling tests of plates and compression tests of coupons 1 inch wide by 4 inches long for 0° or 90° and 6 inches wide by 2 inches or less long for 15°, 30°, 45°, 60°, or 75°.*)

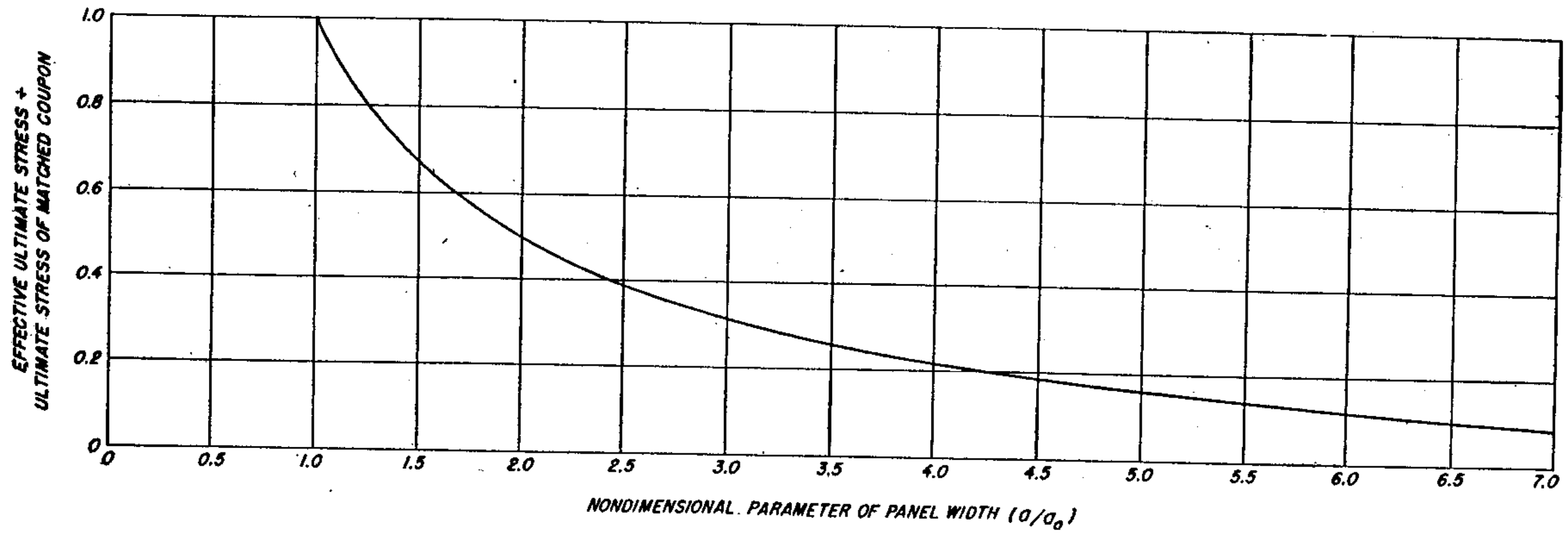


Figure 2–44. Effective ultimate stress divided by ultimate stress of matched coupon.

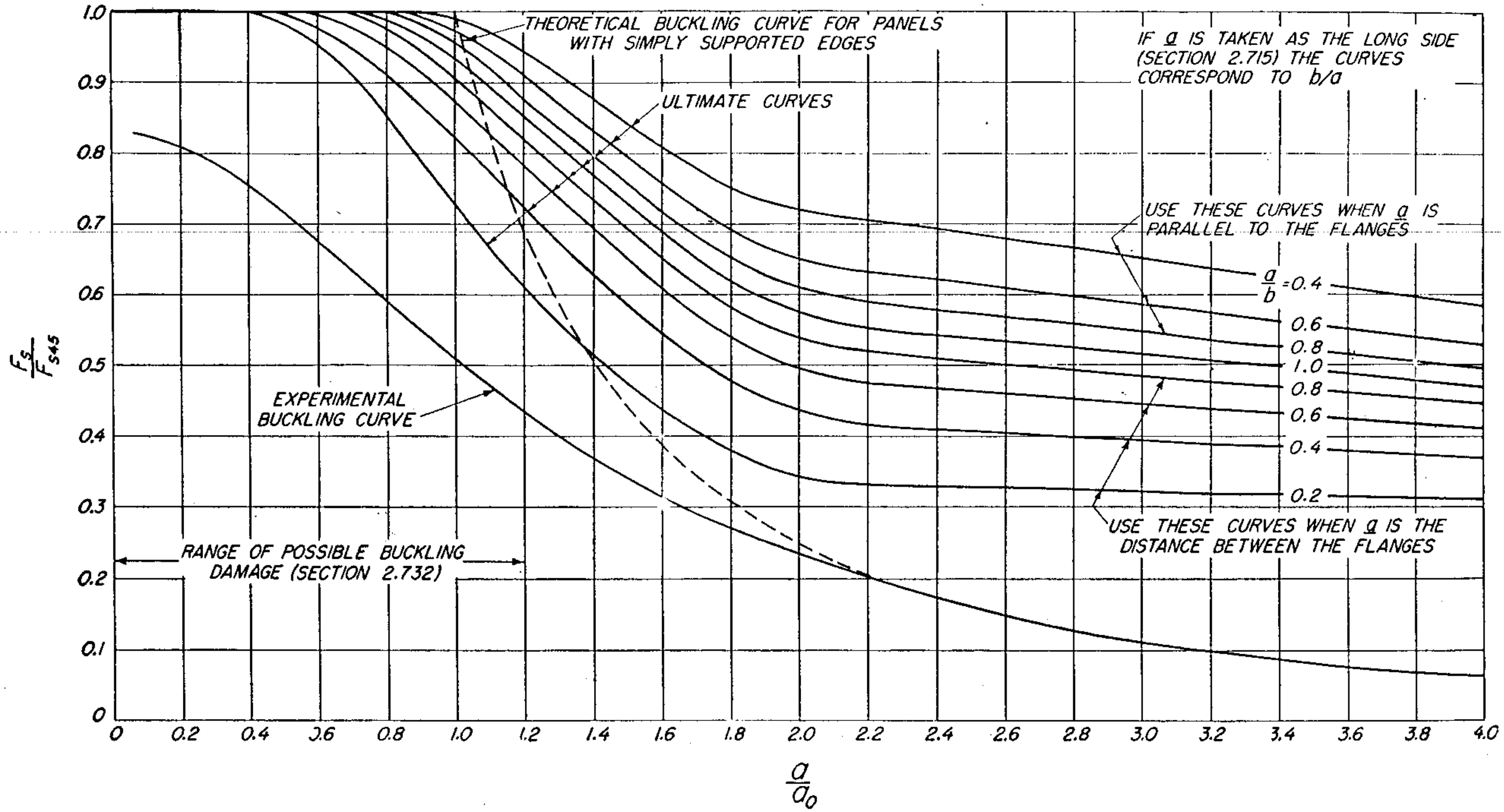


Figure 2–45. Allowable ultimate stress and probable buckling stress for plywood webs in shear. These curves were obtained by using constants for panels with simply supported edges; they do not apply when clamped-edge constants are used.

2.731. *Allowable shear stresses.* The allowable shear stresses of plywood webs having the face grain direction at 0°, 45°, or 90° to the main beam axis may be obtained from figure 2–45.

Values of the parameter $\frac{a}{a_o}$ are obtained from equation (2:82) as explained in section 2.723. Each curve of the family shown in the figure is similar to the curve of figure 2–44. The allowable shear stress (F_s) for the web can be obtained in terms of $F_s/F_{s\theta}$ from figure 2–45. (For a/a_o values greater than 4.0, the a/b curves may be extrapolated as straight lines to meet at a point corresponding to $a/a_o=10$ and $F_s/F_{s\theta}=0.2$.)

The direct use of figure 2–45 for any type of beam having 45° shear webs has been verified by numerous tests of I- and box-beams. A few exploratory tests of beams having 0° and 90° plywood shear webs has indicated that the allowable ultimate shear stresses obtained for these constructions by using figure 2–45 are conservative.

Plywood shear webs of 45° are more efficient than 0° or 90° webs.

The designer is cautioned that box beams may fail at a load lower than that indicated by the strength of the webs as shown in figure 2–45, because of inadequate glue areas of webs at stiffeners or flanges. Such premature failures result from a separation of the web from the flanges or stiffeners.

Figure 2–45 contains a parameter a/b in the form of a family of curves. The $a/b=1$ curve represents a spacing between stiffeners just equal to the clear depth between flanges. The curves below $a/b=1$ should be used for the design of shear webs of beams whose stiffener spacing exceeds the clear distance between flanges. The upper set of curves should be used for the design of beams whose stiffener spacing is less than the clear distance between flanges.

2.732. *Buckling of plywood shear webs.* In connection with shear web tests on various types of beams, it was observed that for plywood webs in the a/a_o range of less than 1.2, buckling was of the inelastic type that often caused visible damage soon after buckling and sometimes just as the buckles appeared for those webs designed to fail in the neighborhood of $F_{s\theta}$. No accurate criteria can be presented at this time, but the designer is cautioned to avoid the use of webs that may be damaged by buckling before the limit or yield stress is reached. The buckling curve established by these tests is shown in figure 2–45.

2.74. Lightening Holes. When the computed shear stress for a full depth web of practical design is relatively low, as in some rib designs, the efficiency, or strength-weight ratio, may be increased by the careful use of lightening holes and reinforcements. General theoretical or empirical methods for determining the strength of plywood webs with lightening holes are not available, and tests should, therefore, be made for specific cases (ref. 2–64).

2.75. Torsional Strength and Rigidity of Box Spars. The maximum shear stresses in plywood webs for most types of box spars subjected to torsion may be calculated from the following formula:

$$f_s=\frac{T}{b't(C'-2b')} \qquad (2:83)$$

where

t=thickness of one web

b'=mean width of spar (total width minus thickness of one web)

C'=average of the outside and inside periphery of the cross section.

The allowable ultimate stress in torsion of plywood webs is determined as in section 2.723.

The torsional rigidity of box beams up to the proportional limit, or to the buckling stress (whichever is the lesser) is given by the formula:

$$\theta=\frac{TC'L}{4Gtb'(C'-2b')^2} \qquad (2:84)$$

2.76. Plywood Panels Under Normal Loads.

2.760. *General.* When rectangular plywood panels, which have the face grain direction parallel or perpendicular to the edges, are subjected to normal loads, the deflections and in some cases the stresses developed, are given by the following approximate formulas. If the maximum panel deflection exceeds about one-half its thickness, the formulas for large deflections will give results which are somewhat more accurate than those given by the formulas for small deflections (ref. 2–51).

2.761. *Small deflections.*

(*a*) Uniform load—all edges simply supported.

$$w_o=0.155\,K_1\,\frac{pa^4}{E_1t^3} \qquad (2:85)$$

where

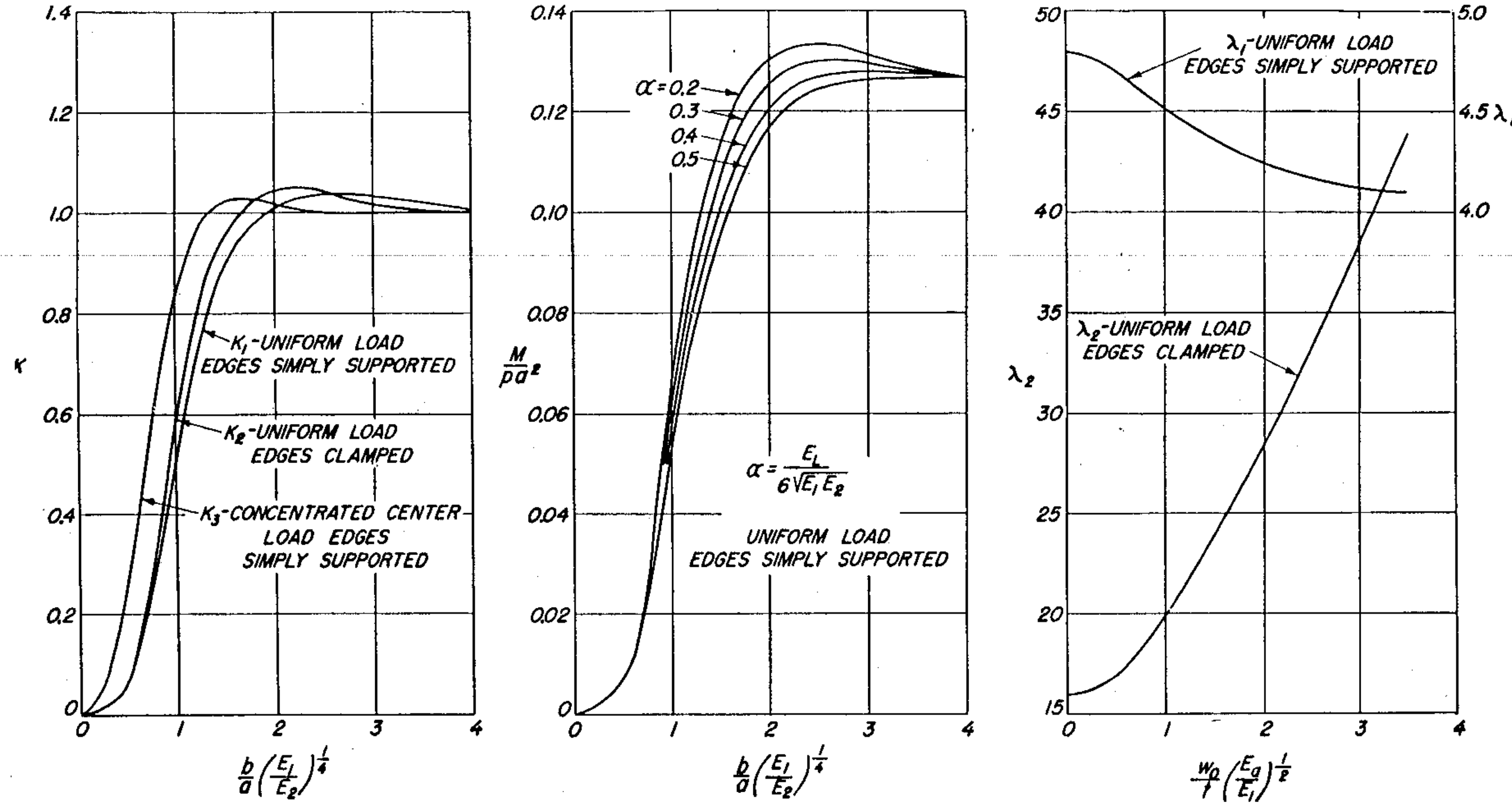
w_o=deflection at center of panel

p=load per unit area

a=width of plate (short side)

K_1=constant from figure 2–46 (*a*)

The maximum bending moment at the center of the panel on a section perpendicular to side *a*



(a)
CONSTANTS FOR USE IN DEFLECTION EQUATIONS (SMALL DEFLECTIONS)

(b)
BENDING MOMENTS AT CENTER OF PANEL (SMALL DEFLECTIONS)

(c)
CONSTANTS FOR USE IN BENDING MOMENT EQUATIONS (LARGE DEFLECTIONS)

Figure 2–46. Curves of bending moments and deflection constants for flat rectangular plywood panels subjected to normal loads.

may be obtained from figure 2–46 (b). The maximum bending moment on a section perpendicular to side b is given by the same curve, provided a and b, and E_1 and E_2 are interchanged in the abscissa, and a is replaced by b in the ordinate. The corresponding stresses can be calculated from the formulas given in section 2.614.

(*b*) Uniform load—all edges clamped.

$$w_o = 0.031\, K_2 \frac{pa^4}{E_1 t^3} \qquad (2:86)$$

where

K_2=constant from figure 2–46 (a)

(*c*) Concentrated load at center—all edges simply supported.

$$w_o = 0.252 K_3 \left(\frac{E_1}{E_2}\right)^{1/4} \frac{Pa^2}{E_1 t^3} \qquad (2:87)$$

where

K_3=constant from figure 2–46 (*a*).

2.762. *Large deflections.*

(*a*) Uniform load—all edges simply supported.

The relation between the load and deflection is given by the formula:

$$p = K_4 E_L w_o \frac{t^3}{a^4} + K_5 E_L w_o^3 \frac{t}{a^4} \qquad (2:88)$$

where

K_4 and K_5 are constants whose approximate values are given in table 2–15.

E_L is taken for the species of the face ply.

The maximum bending moment at the center of the panel can be calculated from the following approximate formula provided the length of the panel exceeds its width by a moderate amount.

$$M_{\max.} = \lambda_1 E_1 w_o \frac{t^3}{6a^2} \text{ (long narrow panels only)} \qquad (2:89)$$

where

λ_1=constant from figure 2–46 (*c*).

Although the edge support conditions are taken as simply supported, it is assumed that the panel length and width remain unchanged after the panel has been deflected. Therefore, in addition to the bending stress, there will be a direct tensile or membrane stress set up in the plane of the plywood, and the total stress in any ply will be the algebraic sum of the bending stress and direct stress in that ply. The maximum total stress will occur in the extreme fiber of the outermost ply having its grain direction perpendicular to the plane of the section upon which the moment was taken; the bending stress being calculated from formula (2:67) and the direct stress from section 2.601 after first determining the average direct stress across the section from the formula:

$$f_{t\,(\text{av.})} = 2.55\, E_a \left(\frac{w_o}{a}\right)^2 \text{ (long narrow panels only)} \qquad (2:90)$$

(*b*) Uniform load—all edges clamped.

The load-deflection relation, formula (2:88), will also apply to this case provided K_6 and K_7 from table 2–15 are substituted for K_4 and K_5, respectively. The maximum total stress may also be determined as outlined in (*a*) above, provided λ_2 from figure 2–46 (*c*) is substituted for λ_1 in formula (2:89).

Table 2–15. Values of constants in the approximate deflection formulas for plywood panels under normal loads [1]

Panel construction [2]	Uniform load all edges simply supported			Uniform load all edges clamped		
	(*b/a*)	K_4	K_5	(*b/a*)	K_6	K_7
3 ply, $\theta=0°$	1.0	(see $\theta=90°$)	(see $\theta=90°$)	1.0	(see $\theta=90°$)	(see $\theta=90°$)
	1.5	1.7	5.9	2.0	3.6	6.0
	2.0	.9	4.7	>3.0	2.5	7.0
	>3.0	.5	4.7			
$\theta=90°$	>1.0	6.3	13.3	1.0	33.3	27.9
				>2.0	32.0	19.2
5 ply, $\theta=0°$	1.0	(see $\theta=90°$)	(see $\theta=90°$)	1.0	(see $\theta=90°$)	(see $\theta=90°$)
	1.5	2.4	6.5	2.0	8.3	8.2
	>2.0	1.5	6.0	>3.0	7.9	9.4
$\theta=90°$	1.0	6.2	12.3	1.0	28.7	17.7
	>1.5	5.0	10.0	>2.0	26.5	15.5

[1] The values given in this table are for spruce plywood, all plies of equal thickness, but they may also be considered applicable to plywood of other species and of the same constructions. For plywood made of more than five plies or of unequal ply thickness, the above table may be used as a rough guide in arbitrarily selecting values of these constants.

[2] θ is the angle between the face grain direction and side b of the panel.

2.77. Stiffened Flat Plywood Panels.

2.771. *The stiffness of a stiffener affixed to a plywood panel (ref. 2.71).* When a stiffener is affixed to a panel the neutral surface of the panel moves toward the stiffener as illustrated in figure 2–47. The amount of this movement is given by the equation:

$$Z_n = \frac{1}{2} \frac{t+d}{\dfrac{2at\sqrt[4]{E_a^3 E_b}}{c\pi h E_s \alpha d} + 1 + \dfrac{E_a t}{E_b d}} \qquad (2:91)$$

in which

$$\alpha=\sqrt{\psi^2+\sqrt{\psi^2-1}}$$

$$\psi=\frac{1}{2}\sqrt{E_w E_z}\left(\frac{1}{G_{wz}}-\frac{2\mu_{wz}}{E_w}\right)$$

$c=1$ if the edge of length b is simply supported

$c=2$ if the edge of length b is clamped

E_s=modulus of elasticity of the stiffener in the direction of its length

The stiffness in the neighborhood of the stiffener added to the plate by the presence of the stiffener is approximately:

$$(EI)_s=\frac{hdE_s}{12}[d^2+3(t+d-2Z_n)^2]+thE_aZ_n^2 \quad (2:92)$$

2.772. *A single stiffener bisecting a panel.* If the stiffener is sufficiently stiff it will substantially divide the panel into two identical panels that can be designed according to the methods of sections 2.71 and 2.72. The minimum stiffness of the stiffener that will accomplish this purpose is defined in the following sections.

2.7721. *Stiffened panel subjected to edgewise compression, stiffener perpendicular to the direction of the stress and parallel to side a (β=0° or 90°)* (ref. 2–30).

$$(EI)_{scr}=\frac{t\,a^4}{11b}(F_{crm}-F_{crp}) \quad (2:93)$$

in which $(EI)_{scr}$ is the critical value of $(EI)_s$ as determined by equation (2:92); F_{crp} and F_{crm} are the critical buckling stresses of the panel, considered to be unstiffened and adequately stiffened

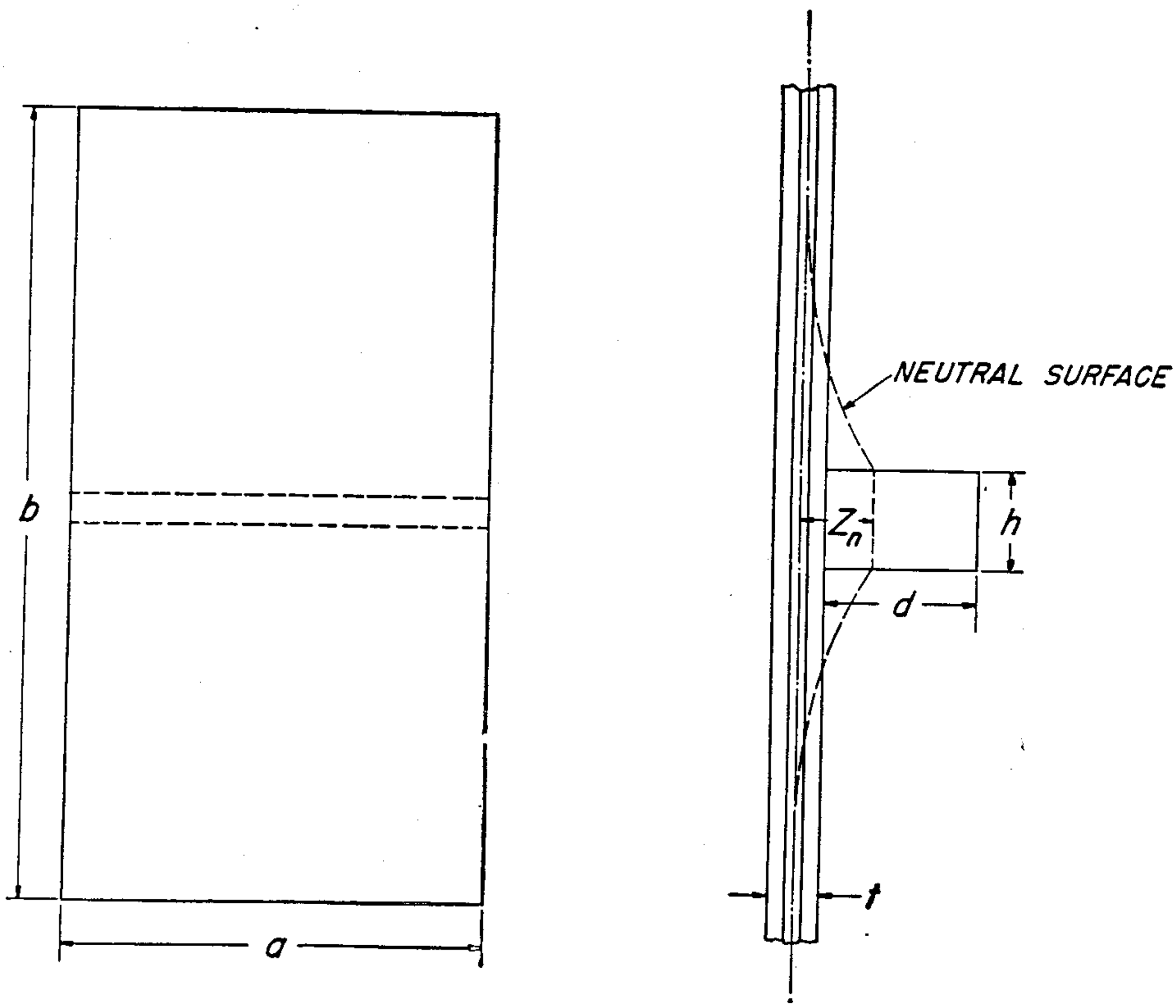


Figure 2–47. Nomenclature for equations (2:91) and (2:92).

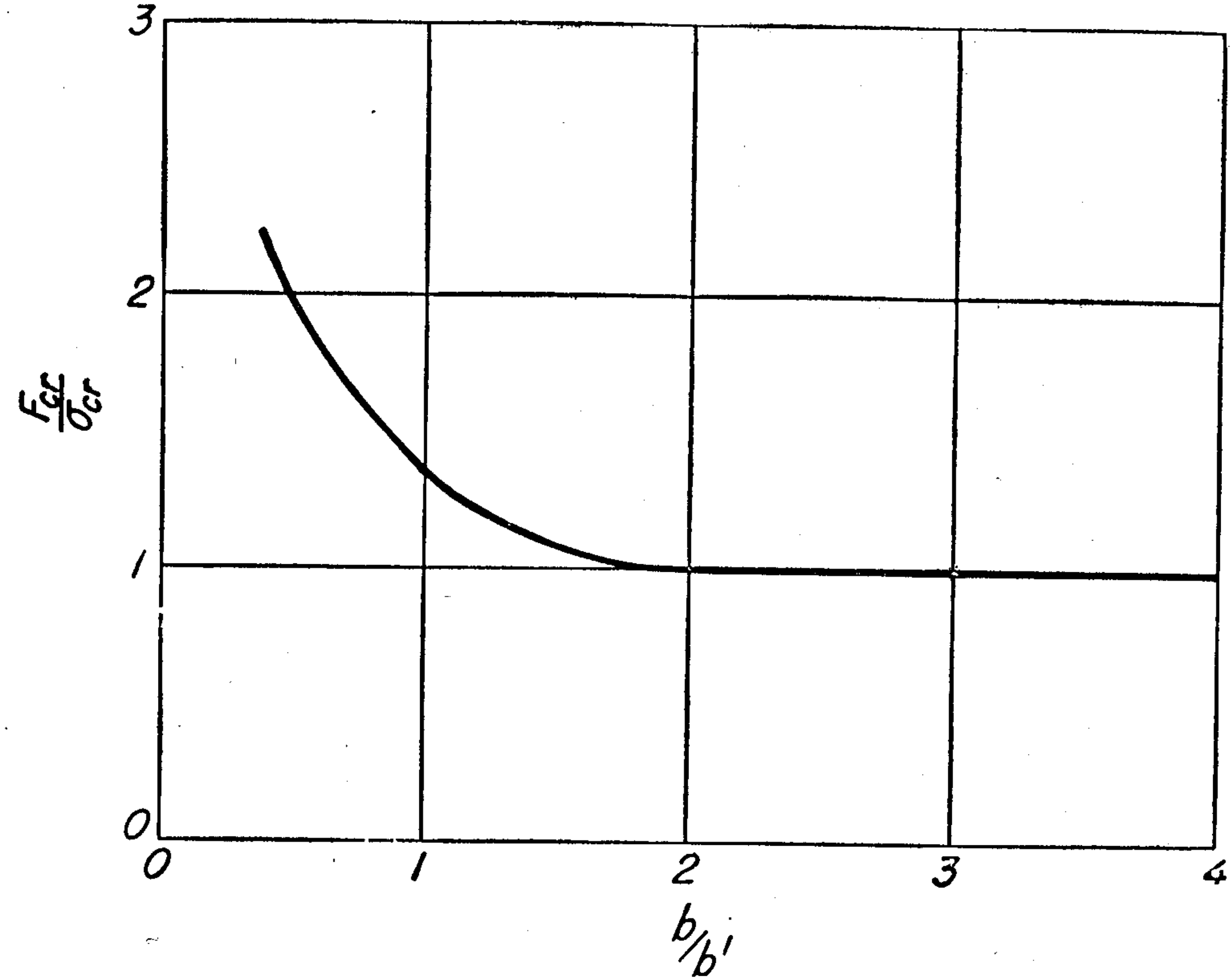


Figure 2–48. Curve for the determination of values of σ_{crp} and σ_{crm} from values of F_{crp} and F_{crm}.

respectively, obtained by means of the method of section 2.712, Case I.

2.7722. *Stiffened panel subjected to edgewise compression, stiffener perpendicular to the direction of the stress* ($\beta=45°$). The stiffness in the neighborhood of the stiffener added to the plate by the stiffener is assumed to be the stiffness of the stiffener alone

$$(EI)_s=\frac{1}{12}\,E_s h\,d^3 \qquad (2:94)$$

and the critical stiffness of the stiffener is approximately

$$(EI)_{scr}=\frac{t\,a^4}{40\,b}\,(\sigma_{crm}-\sigma_{crp}) \qquad (2:95)$$

in which

a=the dimension of the panel perpendicular to the direction of the stress

b=the dimension of the panel parallel to the direction of the stress

and in which the values of σ_{crp} and σ_{crm} are obtained from the critical buckling stresses of the panel, considered to be unstiffened and adequately stiffened respectively, by means of figure 2–48 and the method of section 2.715 for panels with edges simply supported and having their face grain at 45° to their edges.

2.7723. *Stiffened panel subjected to edgewise compression, stiffener parallel to the direction of the stress and to side b* ($\beta=0°$ or $90°$).

$$(EI)_{scr}=\frac{ta\,b^2}{10(n+1)^2}\left[F_{crm}\left(1+\frac{2dh\,E_s}{at\,E_b}\right)-F_{crp}\right] \qquad (2:96)$$

in which $(EI_{scr}$ is the critical value of $(EI)_s$ as determined by equation (2:92); F_{crp} and F_{crm} are the critical buckling stresses of the panel, considered to be unstiffened and adequately stiffened respectively, obtained by the means of the method of section 2.712, Case I, and n is the number of half-waves that occur in the unstiffened panel. It may be noted that the dimensions of the stiffener, d and h, appear in equation (2:96) as well as in equations (2:91) and (2:92) and, therefore, it is necessary to estimate the values of these dimensions and verify the estimate by use of equation (2:96).

The compressive stress in the stiffener associated with the critical stress of the panel is:

$$f_{scr}=\frac{E_s}{E_b}\,F_{crm} \qquad (2:97)$$

The load carried by the panel and the stiffener at the critical stress of the panel is:

$$P_{cr}=F_{crm}\left[at+\frac{E_s}{E_b}\,dh\right] \qquad (2:98)$$

The ultimate load of the stiffened panel cannot be greater than the sum of the ultimate loads of the two half panels according to section 2.722 plus the ultimate load of the stiffener considered as a short column. The reduction of this sum in terms of the a/a_o of one of the half panels is given by figure 2–49 (ref. 2–30).

2.7724. *Stiffened panel subjected to edgewise compression, stiffener parallel to the direction of the stress* ($\beta=45°$). The stiffness in the neighborhood of the stiffener added to the panel by the stiffener is assumed to be the stiffness of the stiffener alone and is given by formula (2:94). The critical stiffness of the stiffener is:

$$(EI)_{scr}=\frac{1}{66}\,\mathrm{tab}^2(F_{crm}-F_{crp}) \qquad (2:99)$$

in which $(EI)_{scr}$ is the critical value of $(EI)_s$ as determined by equation (2:94); F_{crm} and F_{crp} are the critical buckling stresses of the panel, considered to be adequately stiffened and unstiffened, respectively, obtained by means of the method of section 2.715; and a and b are the dimensions of the panel perpendicular and parallel to the direction of the stress, respectively (ref. 2–70).

2.7725. *Stiffened panel subjected to edgewise shear. Stiffener parallel to edges (or ends) of panel and* $\beta=0°$ *or* $90°$.

$$(EI)_{scr}=8\,\frac{E_1ht^3L^4}{K_1a^4} \qquad (2:100)$$

in which

$(EI)_{scr}$=the critical value of $(EI)_s$ as determined by equation (2:92)

L =length of stiffener

a =width of panel (independent of the direction of the stiffener)

K_1 is determined by means of figure 2–46

The critical stress of the stiffened panel is computed by means of section 2.713 equation (2:77) applied to one half of the panel as divided by the stiffener, provided that $(EI)_s$ is equal to or greater than $(EI)_{scr}$.

2.773. *A plywood panel stiffened with a multiplicity of closely spaced stiffeners parallel to one of its edges* ($\beta=0°$ or $90°$). If the spacing of the stiffeners is not too great the formulas for plywood

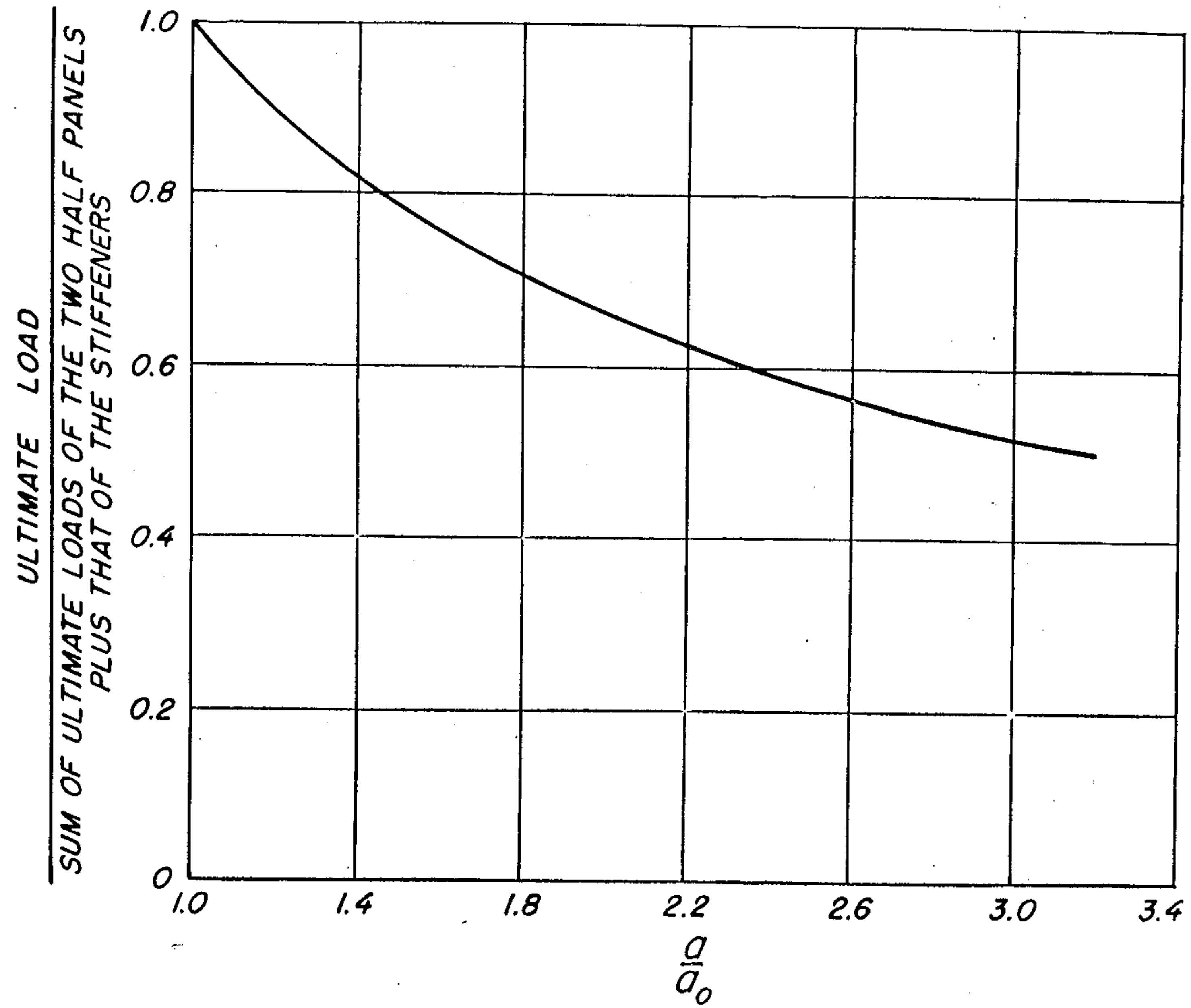


Figure 2-49. Curve for computation of ultimate load on a panel having a vertical stiffener.

of section 2.71 can be applied to such panels. It is convenient to employ formulas for the load per inch of length of the edge of the stiffened panel rather than formulas for stress, thus

$$P_{ccr}=F_{ccr}\,t_e$$

where t_e is the effective thickness of the stiffened panel, which it will not be necessary to compute. The following nomenclature is convenient.

$$\left.\begin{aligned} D_w&=\frac{E_{fw}t^3}{12\lambda_f}\text{ or }\frac{E_{fx}t^3}{12\lambda_f}\\ D_x&=\frac{E_{fx}t^3}{12\lambda_f}\text{ or }\frac{E_{fw}t^3}{12\lambda_f}\end{aligned}\right\}\ \text{for stiffeners parallel or perpendicular to the direction of the face grain of the panel, respectively.}$$

$$D_{wx}=\frac{G_{fwx}t^3}{12}$$

D_{wc}, D_{xc}, and D_{wxc} are computed similarly to the above except that the stiffener is considered as an extra ply of the plywood. The location of the neutral axis is taken into account as described in section 2.52.

D_{we}, D_{xe}, and D_{wxe} are effective values that apply to the stiffened panel.

The subscripts w and x applied to D denote directions parallel and perpendicular, respectively, to the direction of the stiffeners, and subscripts 1 and 2 denote directions parallel and perpendicular, respectively, to the direction of the stress for panels subjected to compression and to the direction of side b for panels subjected to shear.

Equations (2:72) and (2:77) become

$$P_{ccr}=12H_c\sqrt{D_{we}D_{xe}}\,\frac{1}{a^2}\qquad(2{:}101)$$

$$P_{scr}=12H_s\,[D_{1e}{}^3D_{2e}]^{1/4}\,\frac{1}{a^2}\qquad(2{:}102)$$

in which

$$k=\frac{D_{we}\,\mu_{fxwe}+2D_{wxe}}{\sqrt{D_w\,D_x}}$$

$$r=\frac{b}{a}\left(\frac{D_{1e}}{D_{2e}}\right)^{1/4}$$

2.7731. *Determination of D_{we}.* The stiffeners being closely spaced, the usual engineering formula that takes into account the location of the neutral axis, can be employed.

$$D_{we}=D_w+\frac{nhd}{12g}\,E_s\left[d^2+\frac{3(t+d)^2}{\dfrac{nhd\,E_s}{g\,E_g\,t}+1}\right]\qquad(2{:}103)$$

in which

$g=$ the width of the panel across the stiffeners and is equal to a or b as required

n$=$ the number of stiffeners

$E_s=$modulus of elasticity of the stiffeners in the direction of their length.

2.7732. *Determination of D_{xe}.* When a panel is bent across the stiffeners, the variation of the stiffness at the stiffeners and between the stiffeners, and the presence of a sharp kink at the edges of the stiffeners due to stress concentrations, are taken into account.

$$\frac{D_x}{D_{xe}}=\frac{6}{19}\left[1-\left(1-\frac{h}{s}\right)^2\right]\left[1-\frac{D_x{}^2}{D_{xc}{}^2}\right]-\frac{h}{s}\left[1-\frac{D_x}{D_{xc}}+1\right]\qquad(2{:}104)$$

in which

$s=$distance center to center of two adjacent stiffeners.

2.7733. *Determination of D_{wxe}.*

$$D_{wxe}=\frac{1}{g}\,\{[g-nh]\,D_{wx}+n\,[h-\epsilon(t+d)]\,D_{wxc}\}\qquad(2{:}105)$$

in which g is the width of the panel across the stiffeners, n is the number of stiffeners, and ϵ is determined from figures 2–50 and 2–51.

2.7734. *Determination of μ_{fxwe}.*

$$\mu_{fxwe}=\frac{\sqrt{\mu_{fwx}\,\mu_{fxw}\,D_{we}D_{xe}}}{D_{we}}\qquad(2{:}106)$$

in which the values of μ_{fwx} and μ_{fxw} are taken from equations (2:32) and (2:33) of section 2.52, assuming that the stiffener is an added ply of the plywood.

2.774. *Stiffened plywood panels subjected to bending in the direction of the stiffeners.* The maximum bending stress in stiffened plywood panels can be calculated from the following formula, when the face grain direction is 0° or 90° to the direction of the span:

$$f_b=\frac{ME_Lc'}{D_{we}}\qquad(2{:}107)$$

where:

$c'=$distance from the neutral axis of the composite section to the extreme longitudinal fiber

E_L is taken for the species of the outermost longitudinal fiber.

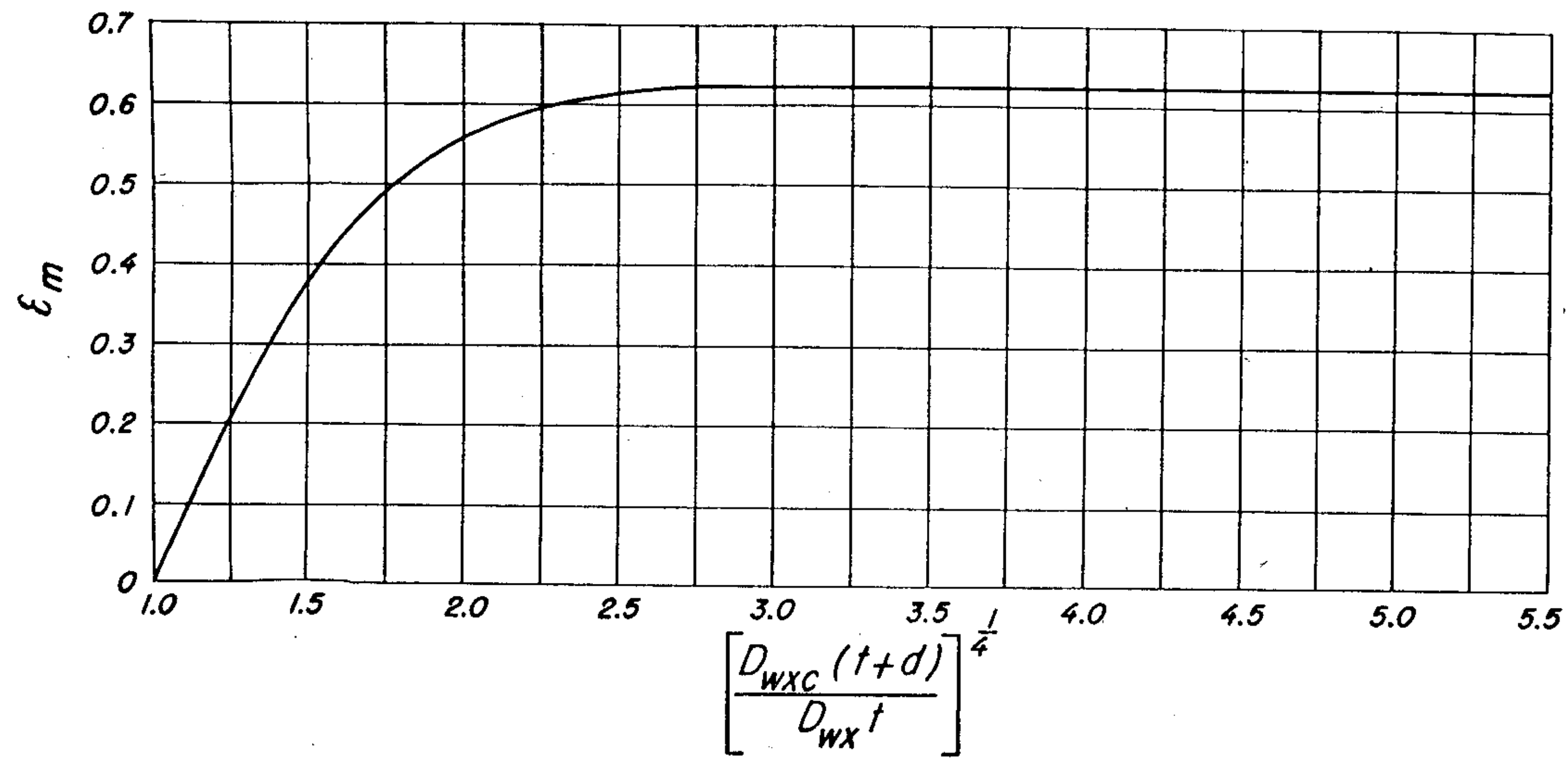


Figure 2–50. Curve for the determination of the values of ϵ_{m^0}

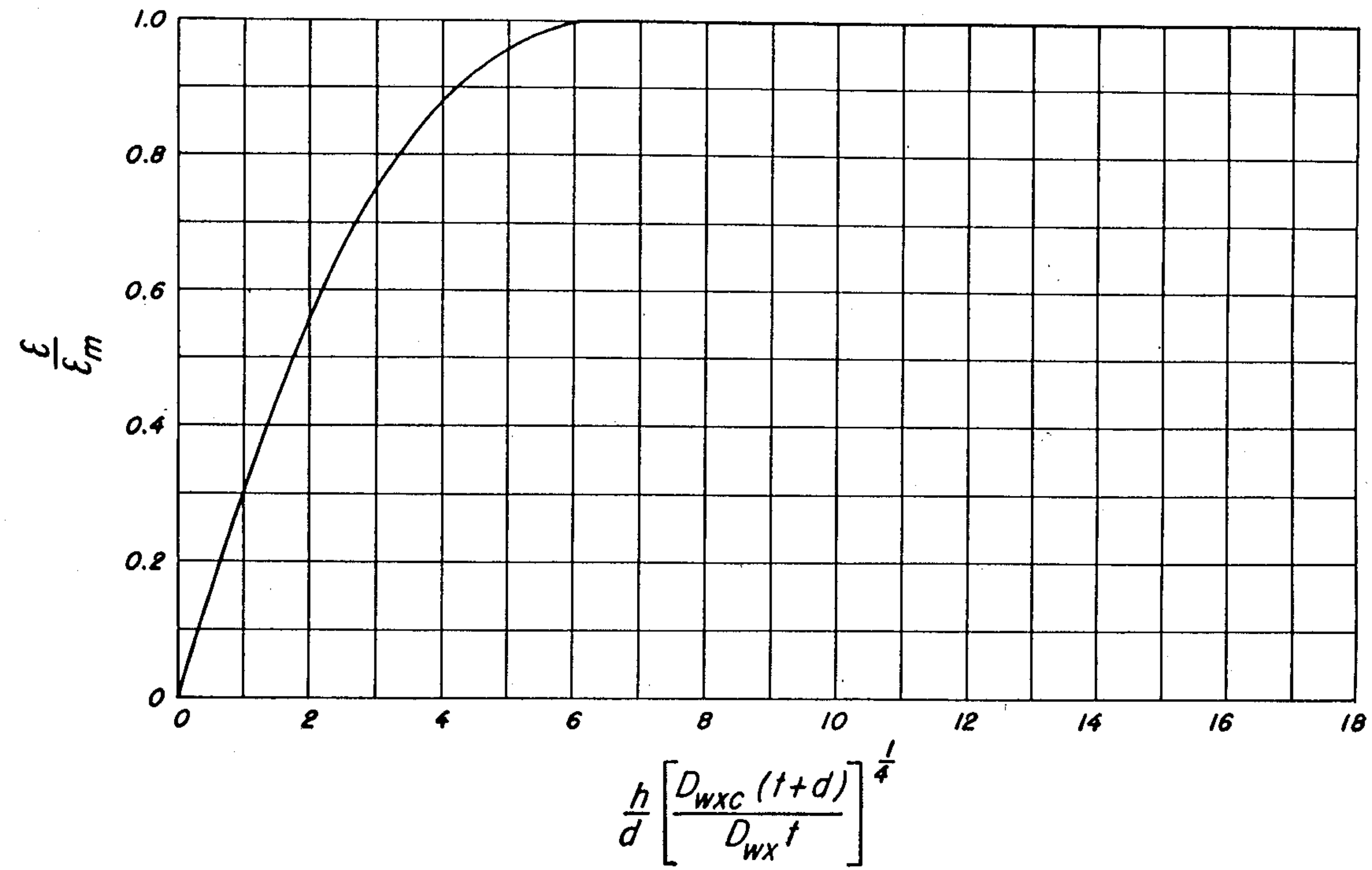


Figure 2-51. Curve for the determination of the values of $\epsilon/\epsilon_m{}^{o}$

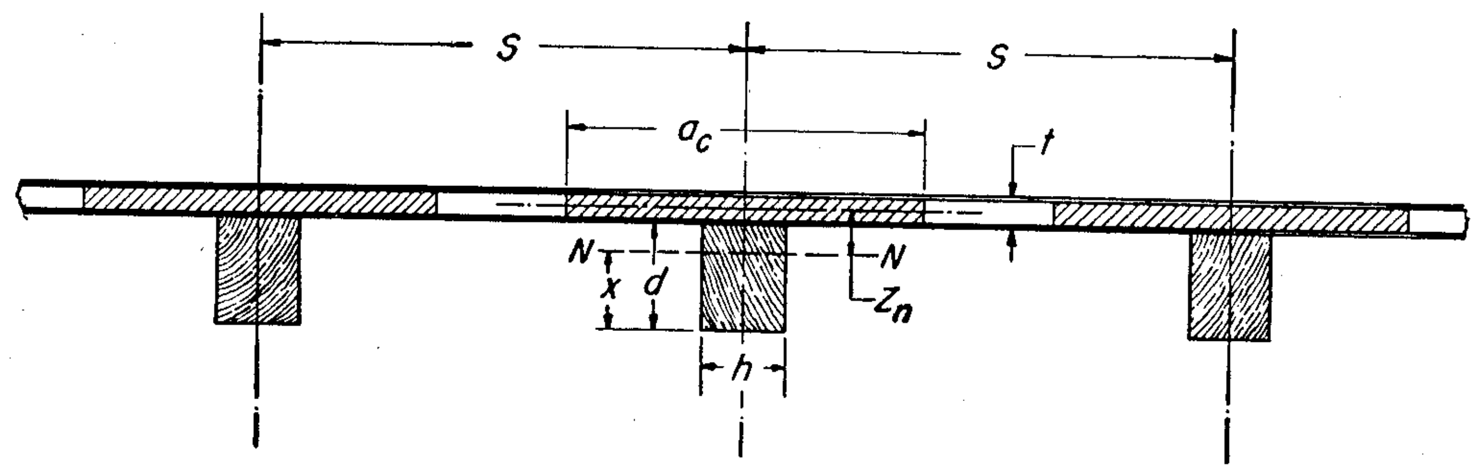


Figure 2–52. Cross section of panel stiffened with multiple stiffeners.

This maximum bending stress should not exceed the modulus of rupture of the material in which the maximum stress exists. If the stiffener is of an I or box section, the modulus of rupture must be corrected by a form factor as follows: When the load is applied so that the outer flange of the stiffener will fail in compression, the proper form factor to use is that for a beam having the same flange dimensions as the outer flange of the stiffener, and the same web thickness as the stiffener, but of a depth equal to $2x$. If the load is applied so that the panel will fail in compression, the proper form factor to use is that of a beam having flange dimensions equal to that of the effective sheet plus the flange of the stiffener adjacent to the panel, and a web thickness equal to that of the stiffener but a depth of $2(d+t-x)$. In either case no form factor need be used if the neutral axis lies within the compression flange, where x is the distance from the neutral axis to the stiffener face away from the panel as shown in figure 2–52.

The effective width of the panel for stresses below the proportional limit is:

$$a_e=\frac{dhE_s(t+d-2Z_n)}{2tE_bZ_n} \tag{2:108}$$

in which Z_n is obtained from equation (2:91) and b is in the direction of the stiffeners.

If the spacing of the stiffener (s in fig. 2–52) is less than a_e, the value of D_{we} is obtained from equation (2:103) and

$$Z_n=\frac{1}{2}\,\frac{t+d}{\frac{ts}{dh}\frac{E_b}{E_s}+1} \tag{2:109}$$

If this is not the case, then

$$D_{we}=D_w+\frac{1}{a_e}\frac{hd^3}{12}E_s+tE_bZ_n^2+\frac{dh}{4a_e}E_s(t+d-2Z_n)^2 \tag{2:110}$$

in which Z_n is obtained from equation (2:91), a_e from equation (2:108), and b is in the direction of the stiffeners.

For stiffened panels having the face grain direction 45° to the length of the stiffeners, the plywood is neglected in the computations and the stiffeners designed to carry the total load.

2.775. *Modes of failure in stiffened panels.* Modes of failure other than failure of the panel or the stiffeners are not considered here.

A possible mode of failure, which has been investigated for only one particular type of construction, is the premature separation of the plywood panel from its stiffeners occurring when the forces required to restrain the edges of the buckled panels become too great for the strength of the plywood or its attachment to the stiffeners.

Since no criteria suitable for general application are available for predicting the critical modes of failure, it is recommended that typical panels of each particular type of construction be tested.

2.8. Curved Plywood Panels

2.81. Strength in Compression or Shear; or Combined Compression (or Tension) and Shear. When failure by buckling does not occur, the ultimate strength of curved plywood panels subjected to compression or shear, or combined compression (or tension) and shear may be obtained by the method given in section 2.613. This method is applicable when the face grain direction is at any angle.

2.82. Circular Thin-Walled Plywood Cylinders.

2.821. *Axial compression.*

2.8211. *Compression with face grain parallel or perpendicular to the axis of the cylinder.* The theoretical buckling stress for a long cylinder (to be modified for design as described later in the section) is given by the formula:

$$F_{ccr}=K_\sigma\,[E_{fw}+E_{fx}]\,\frac{t}{r} \tag{2:111}$$

where

E_L is for the species of the face plies

t= thickness of plywood

r= radius of cylinder

K_σ is a buckling constant that is a function of $\frac{E_1}{E_1+E_2}$ and is determined from figure 2–53. In using figure 2–53, E_1 is the flexural stiffness of the plywood in the direction parallel to the longitudinal axis of the cylinder. E_1 is equal to E_{fw} when the face grain is longitudinal and is equal to E_{fx} when the face grain is circumferential. E_2 is the flexural stiffness of the plywood in the circumferential direction. E_1+E_2 is equal to $E_{fw}+E_{fx}$.

Because of the steepness of the curve for K_σ at the extreme right and left portions, it appears advisable to avoid, when possible, the use of types of plywood for which the ratio $\frac{E_1}{E_1+E_2}$ is small or nearly equal to unity.

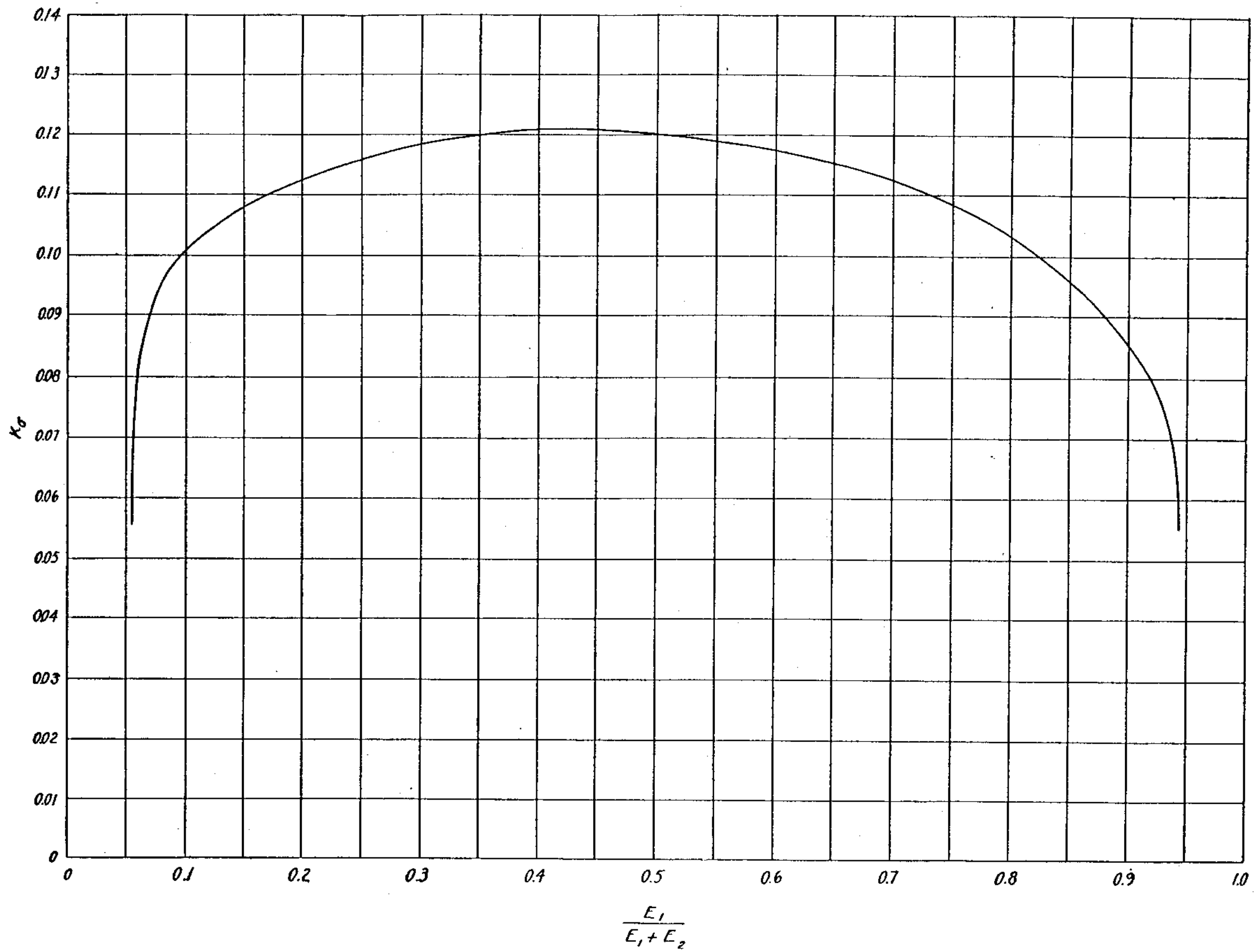


Figure 2–53. Theoretical curve for long, thin plywood cylinders in axial compression.

For use in design, the theoretical buckling stress must be modified as the proportional-limit stress is approached. This is accomplished by the use of figure 2–54. The proportional-limit stress used with this chart is the compressive proportional limit for the plywood in the direction of the cylinder axis and is determined from table 2–13 or from section 2.600. $F_{cp}=F_{cpw}$ when the face grain is longitudinal. $F_{cp}=F_{cpx}$ when the face grain is circumferential. The chart is entered along the abscissa with the ratio F_{ccr} *(theoretical)*$/F_{cp}$. The design buckling stress, F_{ccr}, is then obtained by multiplying the ordinate by F_{cp}.

Limited amounts of double curvature have negligible effect on buckling loads.

2.8212. *Compression with 45° face grain.* When the face grain is at an angle of 45° to the cylinder axis, the theoretical buckling stress may be taken as the average of the theoretical buckling stresses obtained by assuming the face grain direction to be: (1) parallel to the cylinder axis, (2) circumferential. In using figure 2–54, however, to obtain the design buckling stress, the proportional-limit value F_{cp} should be that for the plywood at 45° to the face grain. F_{cp45}, may be taken as 0.55 F_{cu45}, where F_{cu45} is determined by section 2.610.

2.8213. *Compression—effect of length.* If the cylinders are not long, an adjusted value by K_σ, designated by $K_{\sigma a}$, should be used in formula (2:111). Values of $K_{\sigma a}$ can be determined from figure 2–55 in which L is the length of the cylinder, and the subscripts 1 and 2 apply to the axial and circumferential directions respectively.

2.822. *Bending.* For bending, the design buckling stress determined as for compression may be increased 10 percent.

2.823. *Torsion.* The buckling stresses of thin plywood cylinders can be computed by the formula

$$F_{scr}=K_\tau(E_{fw}+E_{fx})\frac{t}{r} \qquad (2:112)$$

in which the value of K_τ depends upon values of $W, U, \frac{E_1}{E_1+E_2}$, and θ

$$W=\frac{1}{2}\,\frac{G_{fwx}+G_{wx}}{E_{fw}+E_{fx}}$$

$$U=\frac{L^2}{rt}$$

θ=angle between face grain and generator of cylinder (fig. 2–56)

Values of K_τ for different values of W, U, and θ are given in figures 2–57, 2–58, and 2–59. The nomenclature is illustrated in figure 2–56. (Ref. 2–93)

2.824. *Combined torsion and bendin.g* Cases of combined loading can be checked by the following interaction formula:

$$\left(\frac{f_{st}}{F_{stcr}}\right)^{4/3}+\left(\frac{f_b}{F_{bcr}}\right)^{4/3}=1.0 \qquad (2:113)$$

where:

f_{st}=applied torsional shear stress

f_b=applied bending stress

F_{stcr}=pure torsion design buckling stress

F_{bcr}=pure bending design buckling stress

2.83. Curved Panels.

2.831. *Axial compression.* The buckling stress is that of a complete cylinder, of which the curved panel can be considered to be a part, of a length equal to the axial dimension of the panel. It can be obtained by use of formula (2:111) corrected for length by the method of section 2.8213.

If the curved panel is very accurately made, higher values may be obtained by test but cannot be counted upon in design.

2.832. *Shear.* An approximation of the buckling stress is obtained by adding the buckling stress of the panel considered to be flat to that of the cylinder of which the panel can be considered to be a part. Thus the buckling stress is given approximately by (ref. 2–40)

$$F_{scr}=H_s\,\frac{(E_1{}^3E_2)^{1/4}}{3\lambda_f}\left(\frac{t}{a}\right)^2+K_\tau(E_{fw}+E_{fx})\,\frac{t}{r} \qquad (2:114)$$

or

$$F_{scr}=[E_{fw}+E_{fx}]\left[K_s\,\frac{t^2}{a^2}+K_\tau\,\frac{t}{r}\right] \qquad (2:115)$$

in which formula (2:114) comes from (2:77) and (2:112); formula (2:115) comes from (2:80) and (2:112).

2.84. Longitudinally Stiffened Cylinders. A multiplicity of evenly spaced idèntical stiffeners attached to the inner surface of the cylinders.

2.841. *Stresses when buckling does not occur.* The strengths can be computed by use of sections 2.610 to 2.614.

2.8411. *Axial compression (or tension).* The compressive stress in the plywood is:

$$f_c=\frac{PE_a}{nhdE_s+2\pi rtE_a} \qquad (2:116)$$

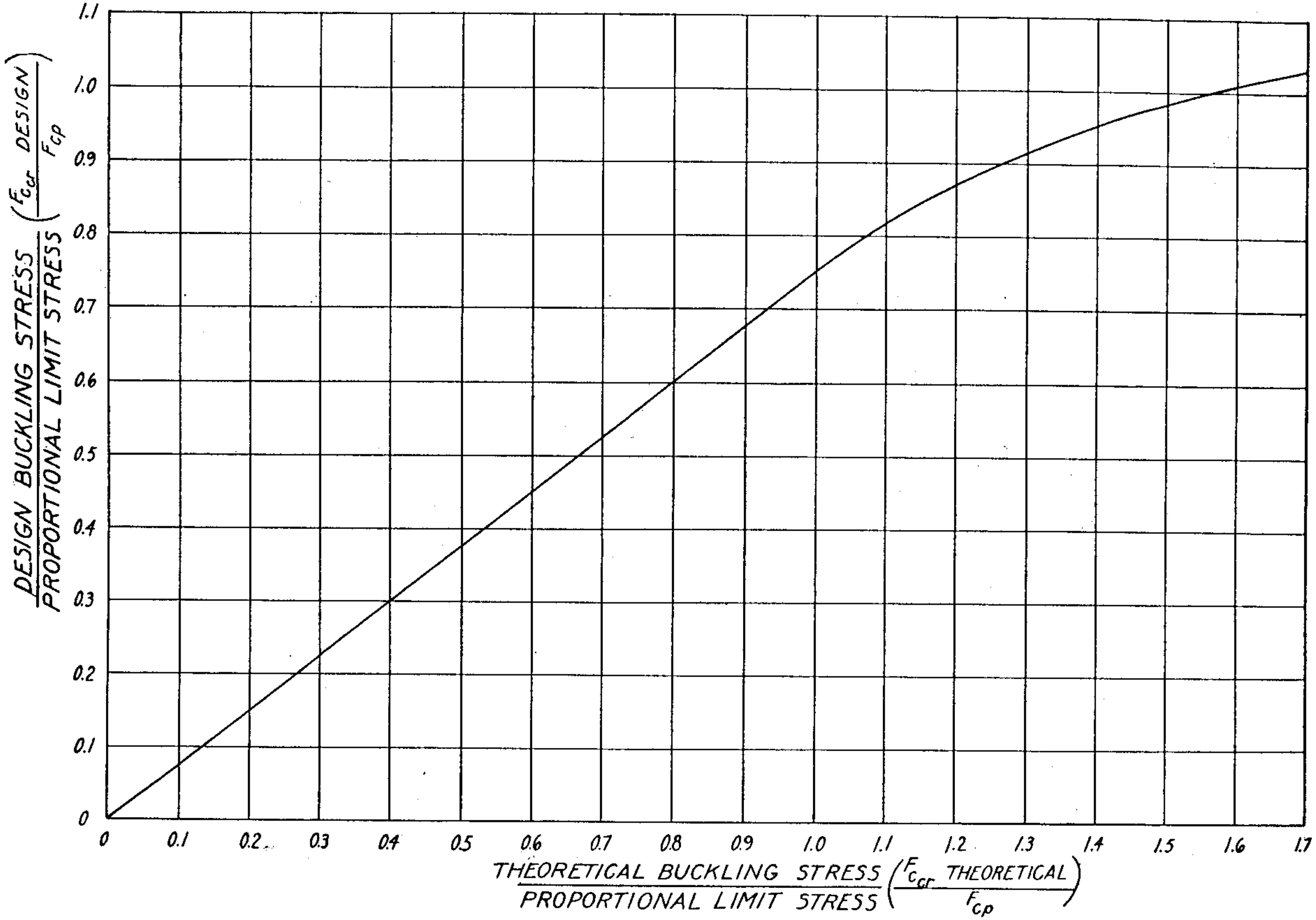


Figure 2–54. Design curve for long, thin-walled plywood cylinders.

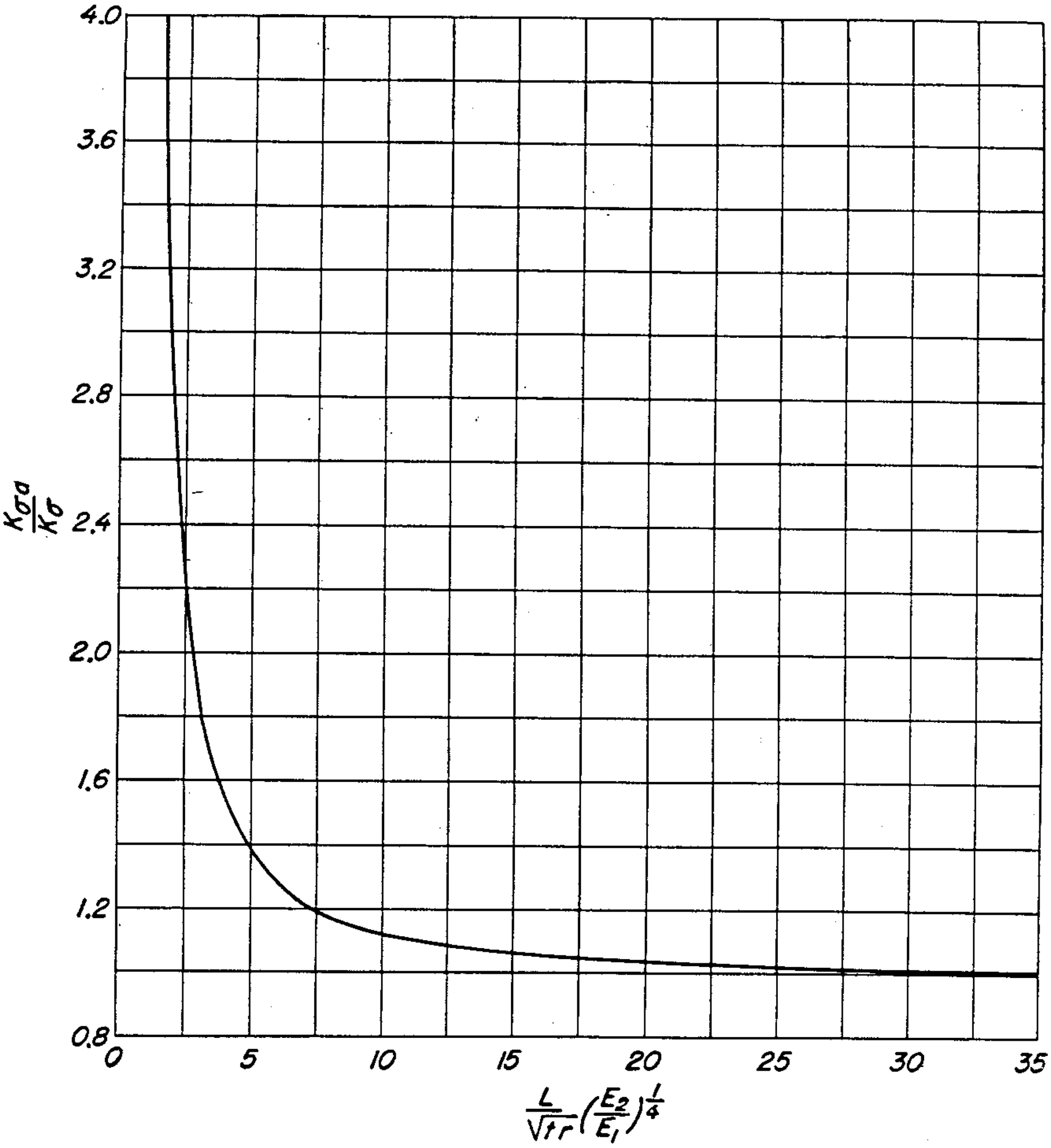


Figure 2–55. Curve showing length effect of cylinders subjected to axial compression.

and that in the stiffeners is

$$f_c=\frac{PE_s}{nhdE_s+2\pi rtE_a} \qquad (2{:}117)$$

in which P is the total load on the stiffened cylinder, E_a and E_s apply to the plywood and the stiffeners, respectively, in the direction of the axis of the cylinder, r is the mean radius of the plywood cylinder, n is the number of the stiffeners, h and d are the cross-sectional dimensions of an individual stiffener, and t is the thickness of the plywood.

2.8412. *Shear stress due to torsion.* The shear stress in the plywood is:

$$f_s=\frac{2G_{ab}Tr_3}{nhG_s\frac{r_2^4-r^4}{r_2+r_1}+\pi G_{ab}(r_3^4-r_2^4)} \qquad (2{:}118)$$

and that in the stiffener is:

$$f_s = \frac{2G_sTr_2}{nhG_s\frac{r_2^4-r_1^4}{r_2+r_1}+\pi G_{ab}(r_3^4-r_2^4)} \qquad (2:119)$$

in which r_1 is the radius of a cylinder tangent to the inner surfaces of the stiffeners, r_2 and r_3 are the inner and outer radii of the cylinder, respectively, G_{ab} and G_s are the moduli of rigidity of the plywood and the stiffeners, respectively, with reference to longitudinal and circumferential axes, and T is the applied torque.

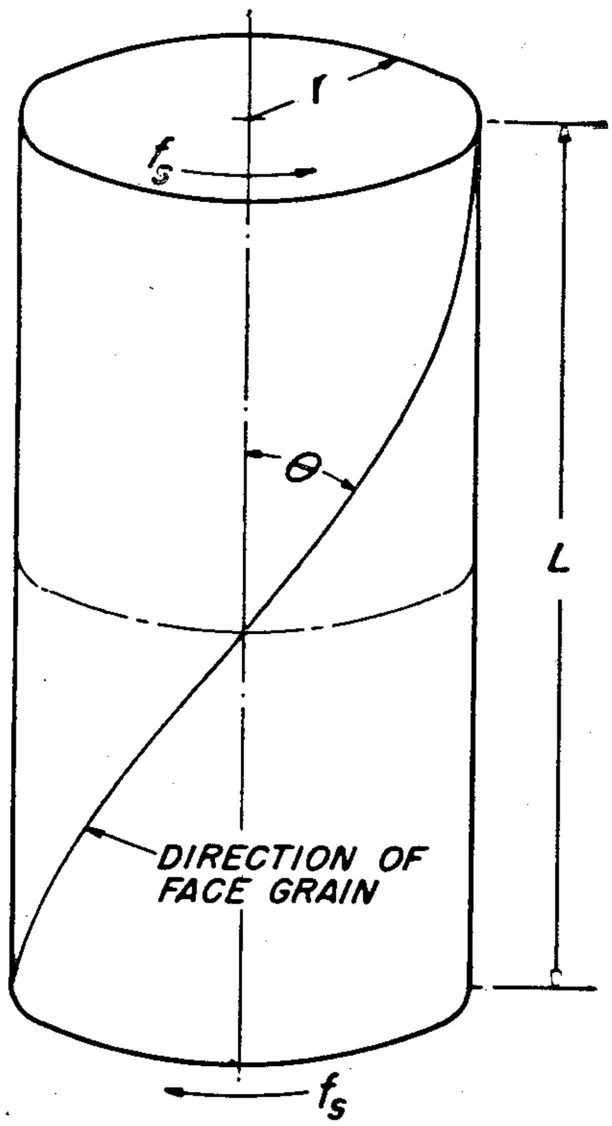


Figure 2-56. Illustration of the meaning of the symbols used for cylinders subjected to torsion.

2.842. *Buckling of stiffened cylinders.*

2.8421. *Axial compression.* In general the plywood will buckle between the stiffeners when the stress in it equals the buckling stress for the cylinder, the effect of the stiffeners being ignored, and, therefore, formula (2:111) employed. The load at which such buckling occurs can be found by setting this stress (F_{ccr} from formula 2:111) equal to f_c in formula (2:116) and solving for the load P.

Tests indicate that unless the stiffeners are quite stiff they will buckle with the cylinder and fail at the load computed in the above manner. If the stiffeners are so stiff that they do not buckle with the cylinder, the maximum load will be greater than that computed. However, no methods are available for the determination of the size stiffeners required to obtain this effect nor to compute the maximum loads that are obtained. (Ref. 2–95)

2.8422. *Torsion.* The shear buckling stress of the curved plywood shell between the stiffeners is about 85 percent of that obtained by formula (2:112) for the cylinder, neglecting the effect of the stiffeners. The torque at which buckling occurs can be found by setting this stress $0.85\times F_{scr}$ from (2:112) equal to f_s in formula (2:118) and solving for the torque T.

Tests indicate that the maximum torque coincides with the torque at which buckles form.

2.8423. *Bending.* For bending, the design buckling stress determined as for compression may be increased 10 percent. Tests indicate that for very stiff stiffeners this percentage may be increased. (Ref. 2–96)

2.85. STIFFENED CURVED PANELS. A single stiffener bisecting the panel.

2.851. *Axial compression.*

2.8511. *Stiffener axial.* The critical stress in the plywood is computed according to the method of section 2.831, and the critical load of the stiffened panel can be obtained by substituting this value for f_c in equation (2:116), placing n equal to unity, and solving for P.

2.8512. *Stiffener circumferential.* The critical stress is computed according to the method of section 2.831, using the distance between the stiffener and the end of the panel as the length of the cylinder, provided that

$$(EI)_s > 0.4F_{ccr}r^2h^2 \qquad (2:120)$$

in which $(EI)_s$ is given by formula (2:84), and F_{ccr} is the critical stress of the entire panel, neglecting the stiffener, computed by the method of section 2.831.

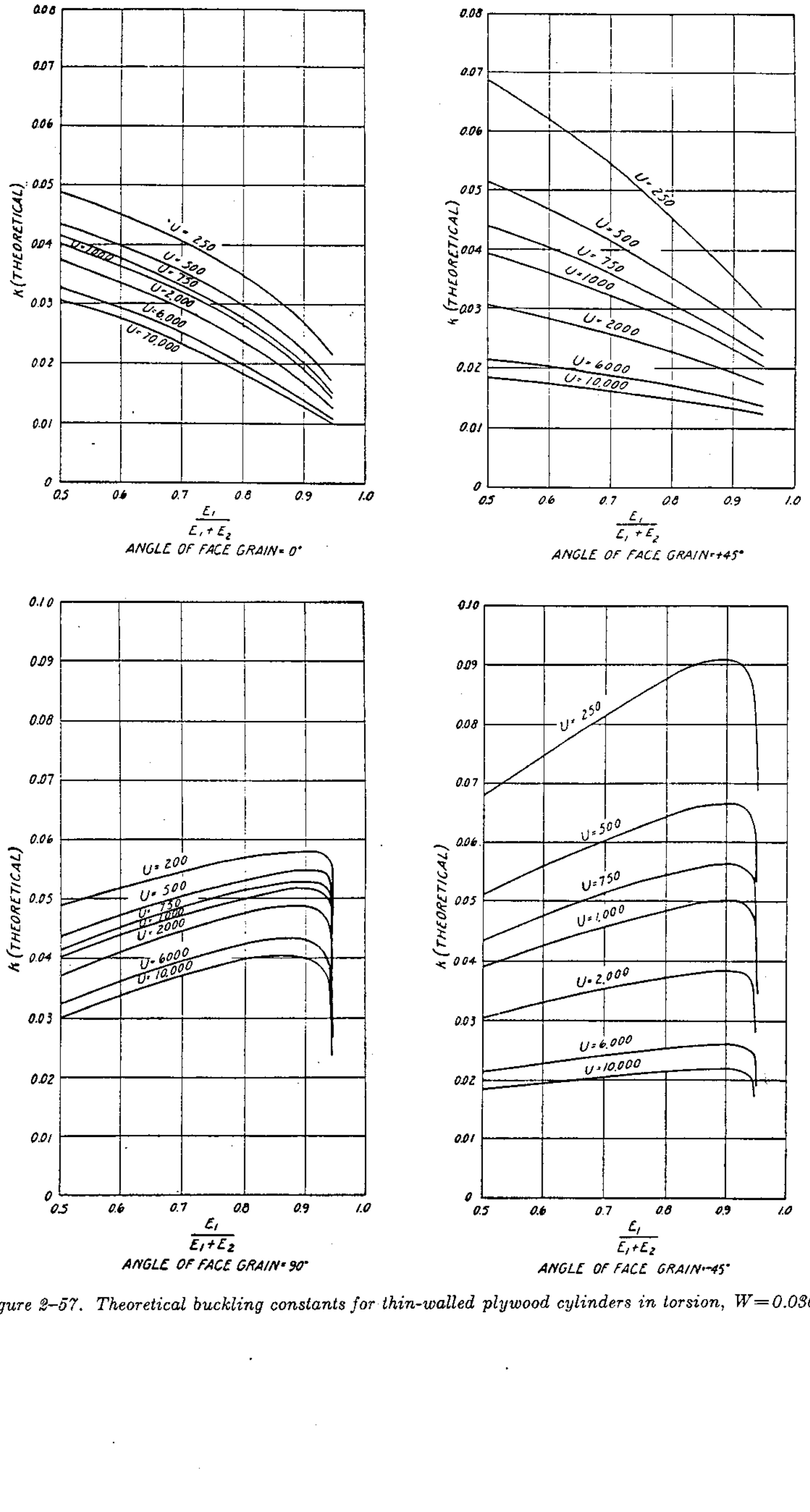


Figure 2–57. Theoretical buckling constants for thin-walled plywood cylinders in torsion, W = 0.036.

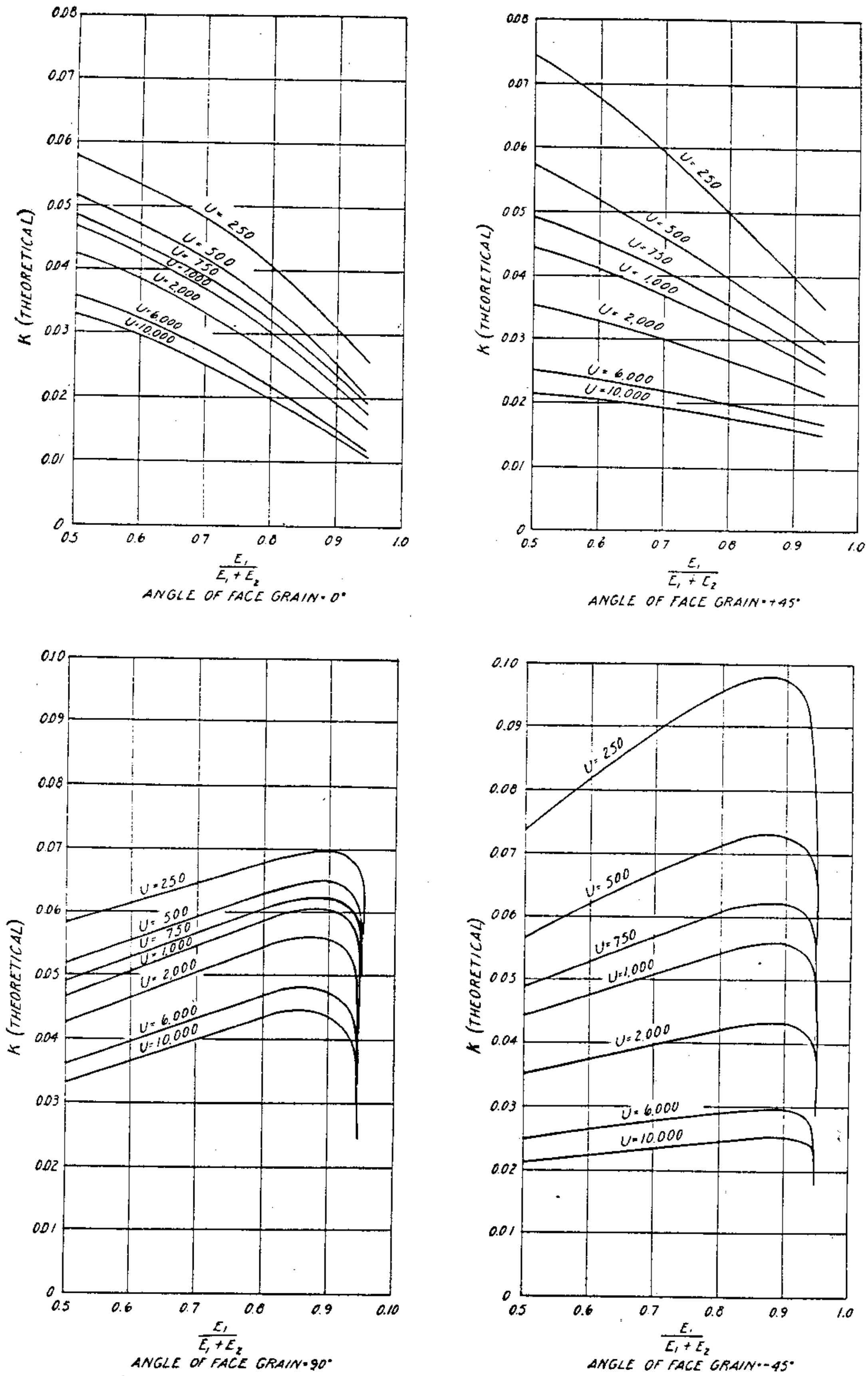


Figure 2–58. Theoretical buckling constants for thin-walled plywood cylinders in torsion, W=0.056.

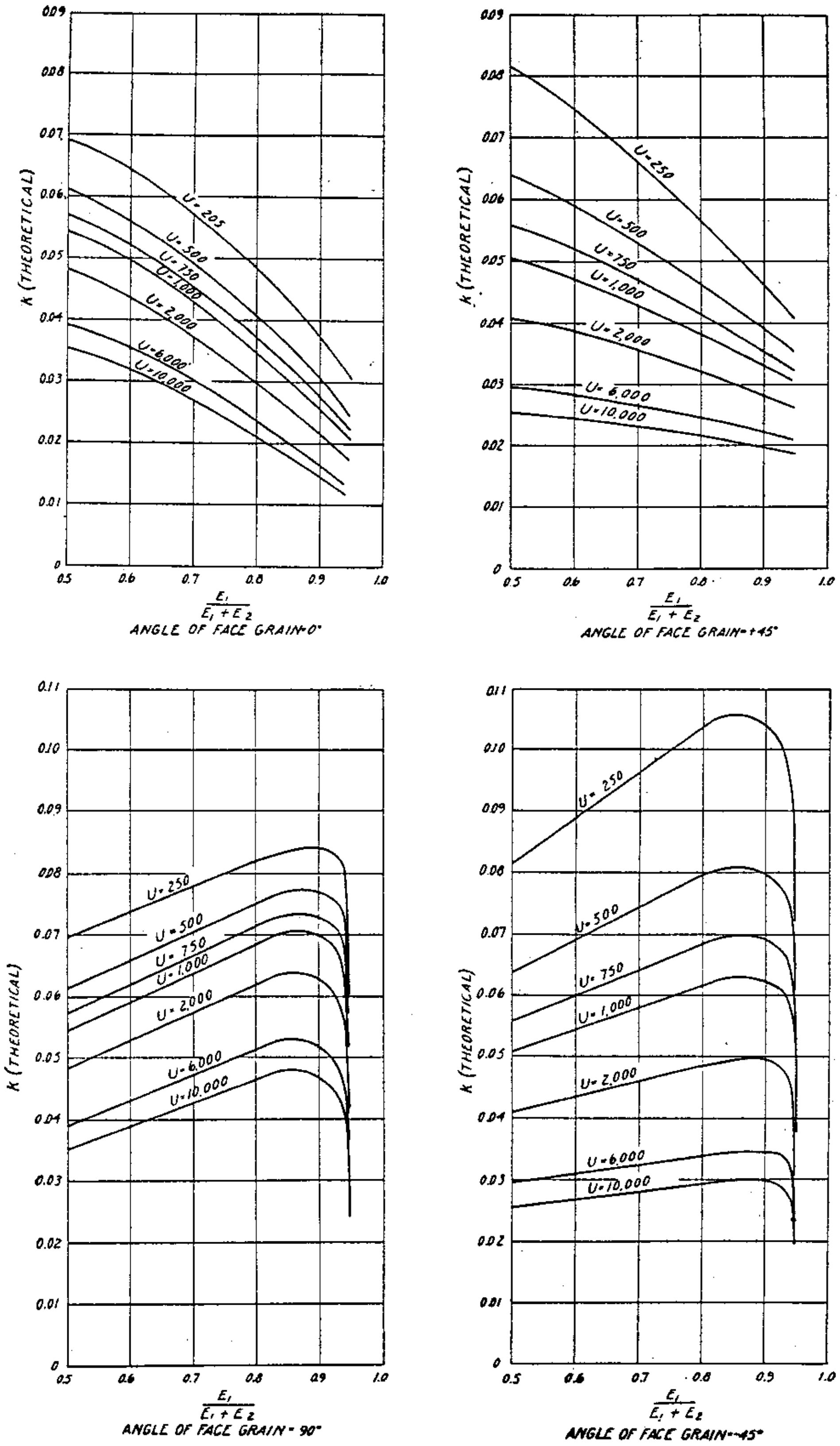


Figure 2–59. Theoretical buckling constants for thin-walled plywood cylinders in torsion, W = 0.090.

2.852. *Shear.*

2.8521. *Stiffener axial.* The stiffened curved panel can be considered to be a part of a stiffened cylinder. Thus the critical stress of the plywood panel without the stiffener is computed according to section 2.832. This stress is substituted for the left hand member of equation (2:118) using:

$$n=\frac{2\pi r}{a} \tag{2:121}$$

and the equation solved for the torque T. The stress applied to the edges of the stiffened panel which will cause it to buckle is then given by:

$$f_{scr}=\frac{T}{2\pi r^2 t} \tag{2:122}$$

This method leads to values which are slightly conservative.

2.9. Joints

2.90. BOLTED JOINTS.

2.900. *Bearing parallel or perpendicular to grain.* The strength of wood in bearing parallel to the grain against solid steel aircraft bolts disposed along the member in single or double lines with the load divided equally between the two ends of the bolt (concentric loading) can be determined by use of figure 2–60. The stress at ultimate and at the proportional limit is expressed in terms of the maximum crushing strength for L/D ratios up to 16. The stress does not vary significantly below an L/D of 8 for softwoods and 5 for hardwoods but drops rapidly as the L/D ratio is increased above these values.

The ratio of ultimate bearing stress to the bearing stress at the proportional limit is 1.4 or less (fig. 2–61) at low L/D ratio for both softwoods and hardwoods. Thus, if a ratio of ultimate to limit bearing load higher than 1.4 is desired, it follows that the limit load in the low L/D range must be based on stresses below the proportional limit. For example, if a ratio of 1.5 is desired for softwoods (shown by broken lines in figs. 2–60 and 2–61) the limit load will be less than the proportional limit load up to an L/D ratio of 8.5 beyond which the proportional limit stress is used to determine the limit load.

The bearing strength of wood perpendicular to grain under aircraft bolts can be found by use of figure 2–62 (ref. 2–77). It may be noted that while bearing stress is only moderately reduced as the L/D ratio becomes greater than 9, there is a marked variation with bolt diameter, particularly in the smaller sizes. The bearing stress at proportional limit when bearing perpendicular to grain, in general may be found with sufficient accuracy by dividing the ultimate bearing strength by 1.33 for all L/D ratios.

2.901. *Bearing at an angle to the grain* (ref. 2–61). When the load on a bolt is applied at an angle between 0° and 90° to the grain, the allowable load (proportional limit or ultimate) may be computed from the expression

$$N=\frac{PQ}{P\sin^2\theta+Q\cos^2\theta} \tag{2:123}$$

where

N=the allowable bolt load at angle θ

P=the allowable bolt load parallel to the grain

Q=the allowable bolt load perpendicular to the grain

θ=the angle between the applied load and the direction of the grain

Equation (2:123) is solved graphically by the Scholten Nomograph, figure 2–63.

2.902. *Eccentric loading.* When load is applied at only one end of a bolt (eccentric loading), the allowable ultimate load may be taken as one-half the ultimate two-end load computed as above. At proportional limit, however, the allowable eccentric load may be taken as only one-fourth of the two-end proportional limit load for two-end loading parallel to grain. This ratio may be increased to one-half if deformations approximately equal to those occurring at proportional limit under two-end loading are not objectionable even though they are well beyond those corresponding to the one-end proportional limit load.

Proportional limit values for one-end loading perpendicular to grain may be taken as one-half of the proportional limit values for two-end loading.

2.903. *Combined concentric and eccentric loadings; bolt groups.* When the design loads on a group of bolts are either all concentric or all eccentric and are all in the same direction, the allowable loads for the individual bolts may be added directly to determine the total allowable load for the group. When the design loads are in different directions (as when the load causes a moment about the centroid of the bolt group) or when they are partly concentric and partly eccentric, each bolt must be treated separately. The design loads and moments

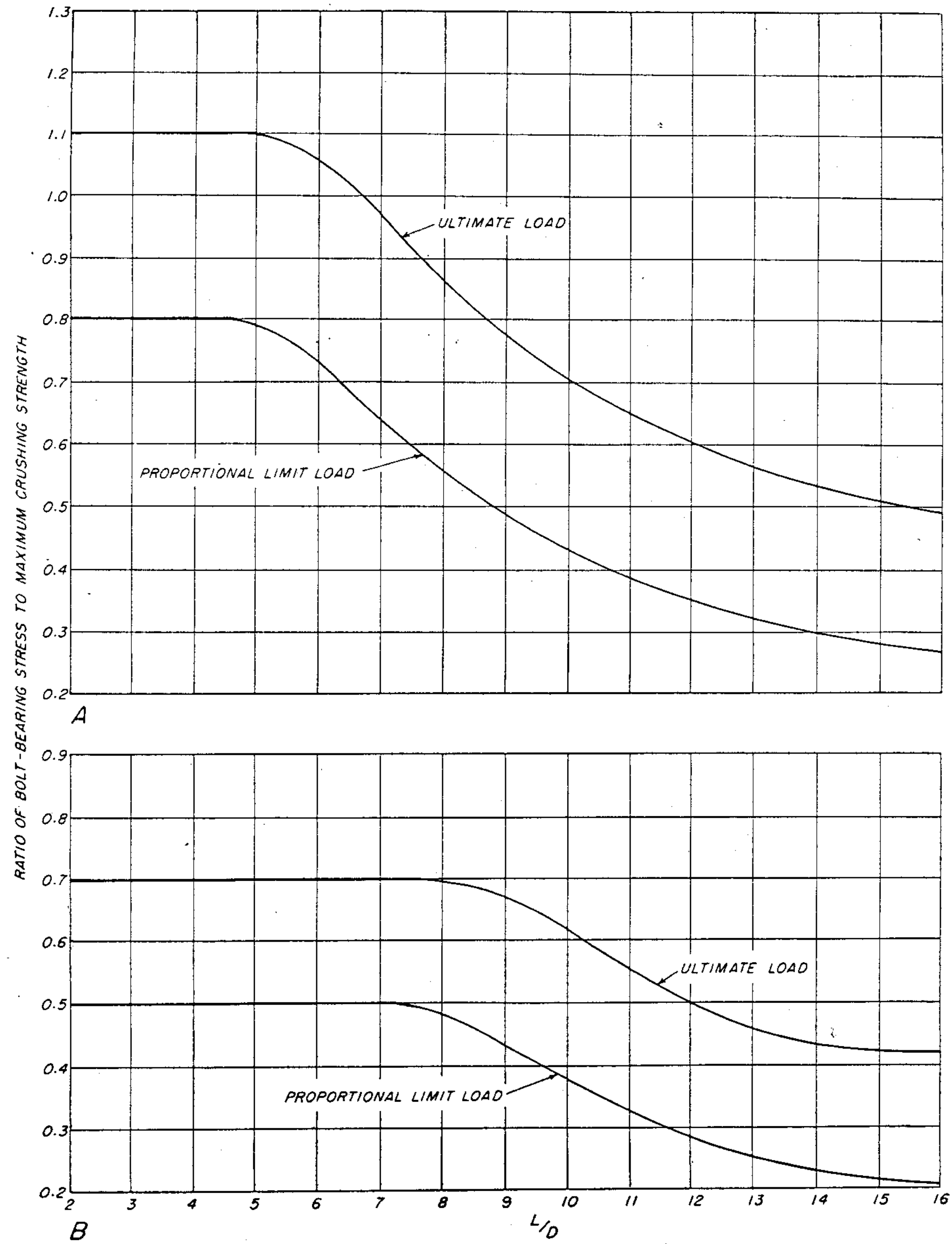
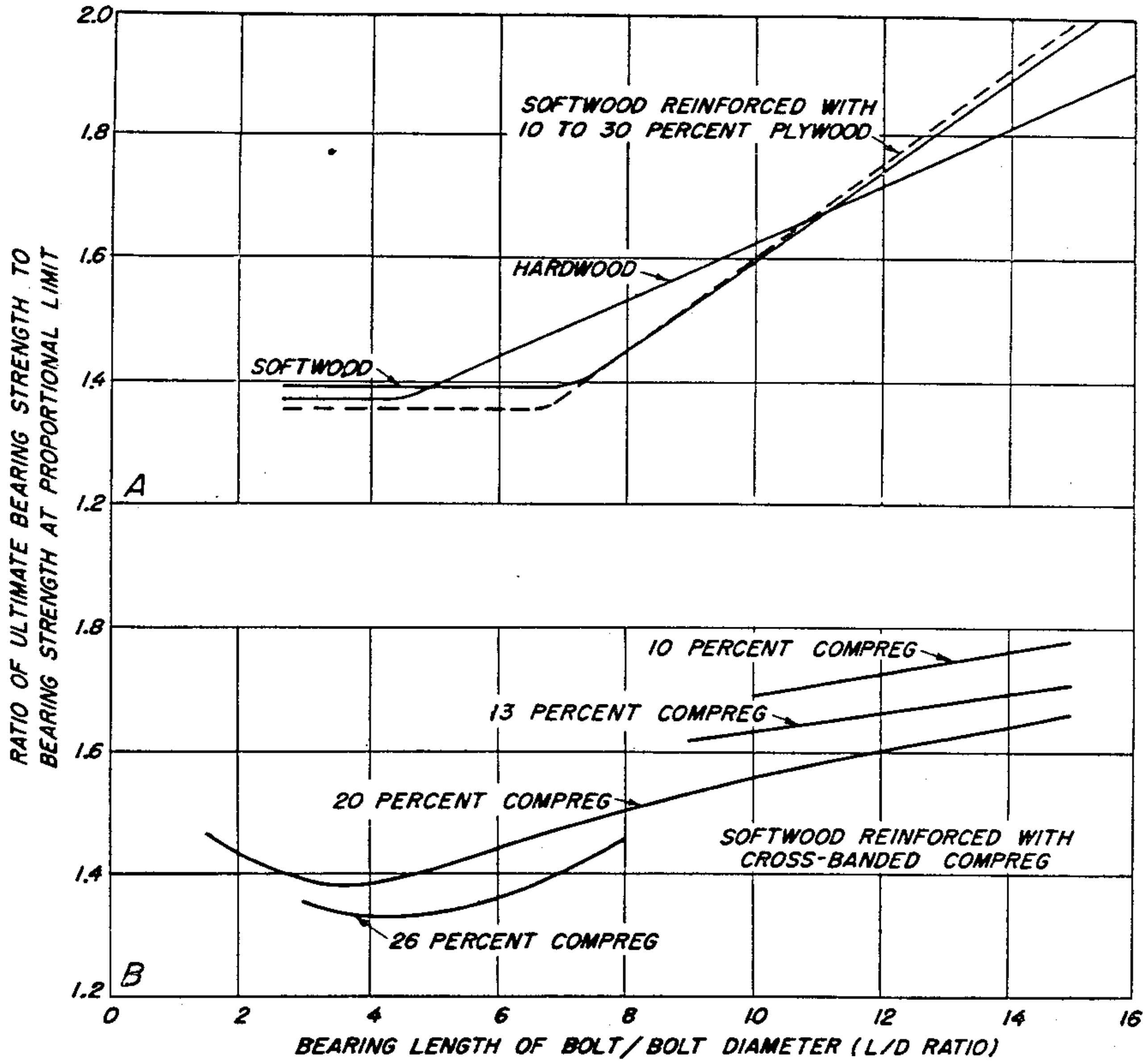


Figure 2–60. Relation between bearing strength and maximum crushing strength for wood under aircraft bolts bearing parallel to grain. A, hardwoods; B, softwoods.



Z M 75272 F

Figure 2–61. Relation between ultimate bearing strength and bearing strength at proportional limit for various types of members.

must be distributed to each bolt in proportion to its resistance and the geometry of the bolt group. This often requires a trial and error calculation.

2.904. *Bolt spacings.* The following bolt spacing criteria are based on spruce. For other species the parallel-to-grain spacings and end margins should be multiplied by the expression:

$$K=\frac{F_{cp}}{3.57F_{su}} \qquad (2:124)$$

where

F_{cp}=allowable stress at proportional limit in compression parallel to the grain

F_{su}=allowable shearing stress parallel to the grain of the material

Spacings perpendicular to grain and edge margins as given below are applicable to all species.

2.9040. *Spacing of bolts loaded parallel to the grain.*

(1) *Spacing parallel to the grain.* The minimum distance from the center of any bolt to the edge of the next bolt in a spruce member having cross-banded reinforcing plates, subjected to either tension or compression, is given in figure 2–64. The minimum distance from the edge of a bolt to the end of such a member subject to tension is also given. For spruce members without reinforcement these values must be increased by 50 percent.

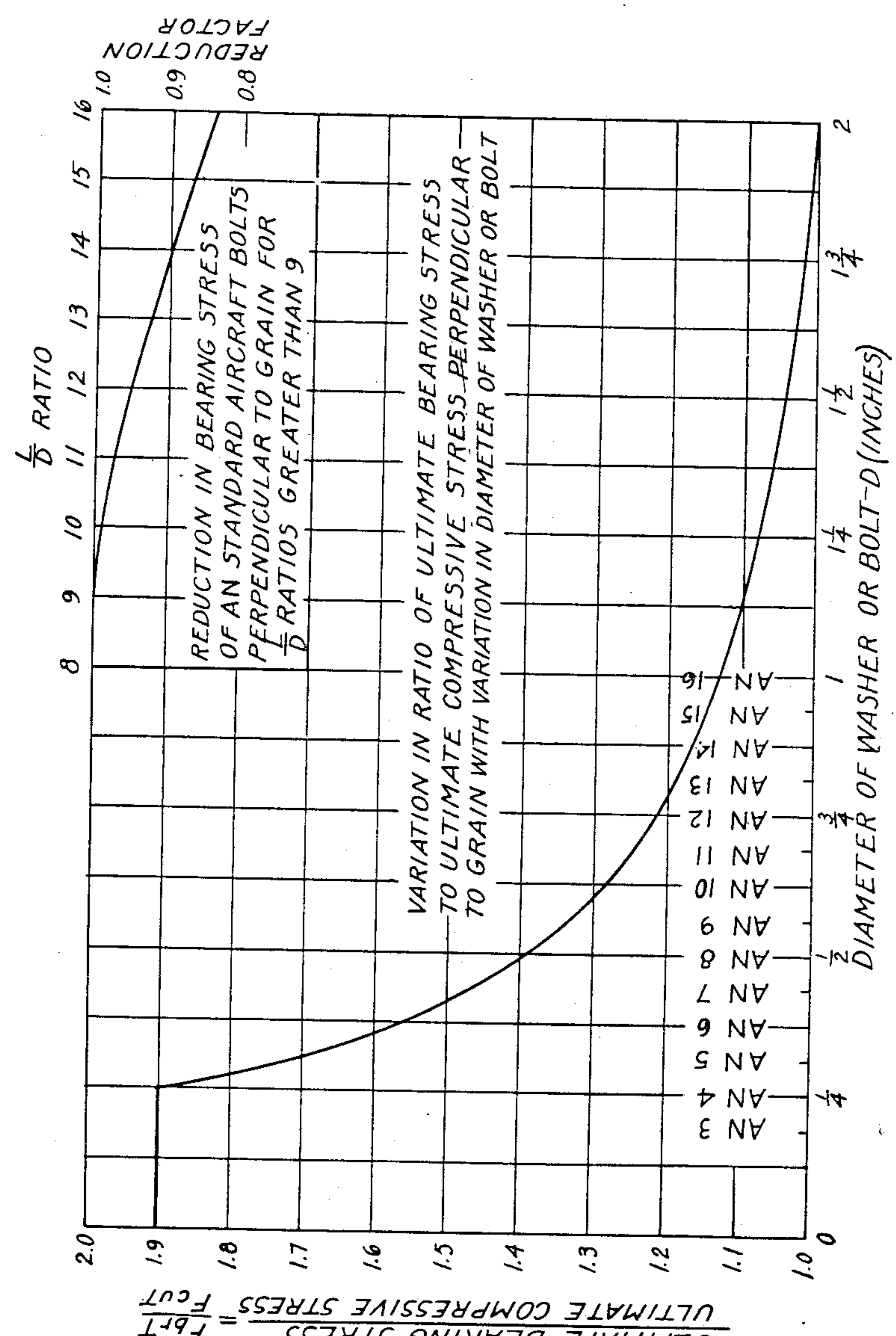
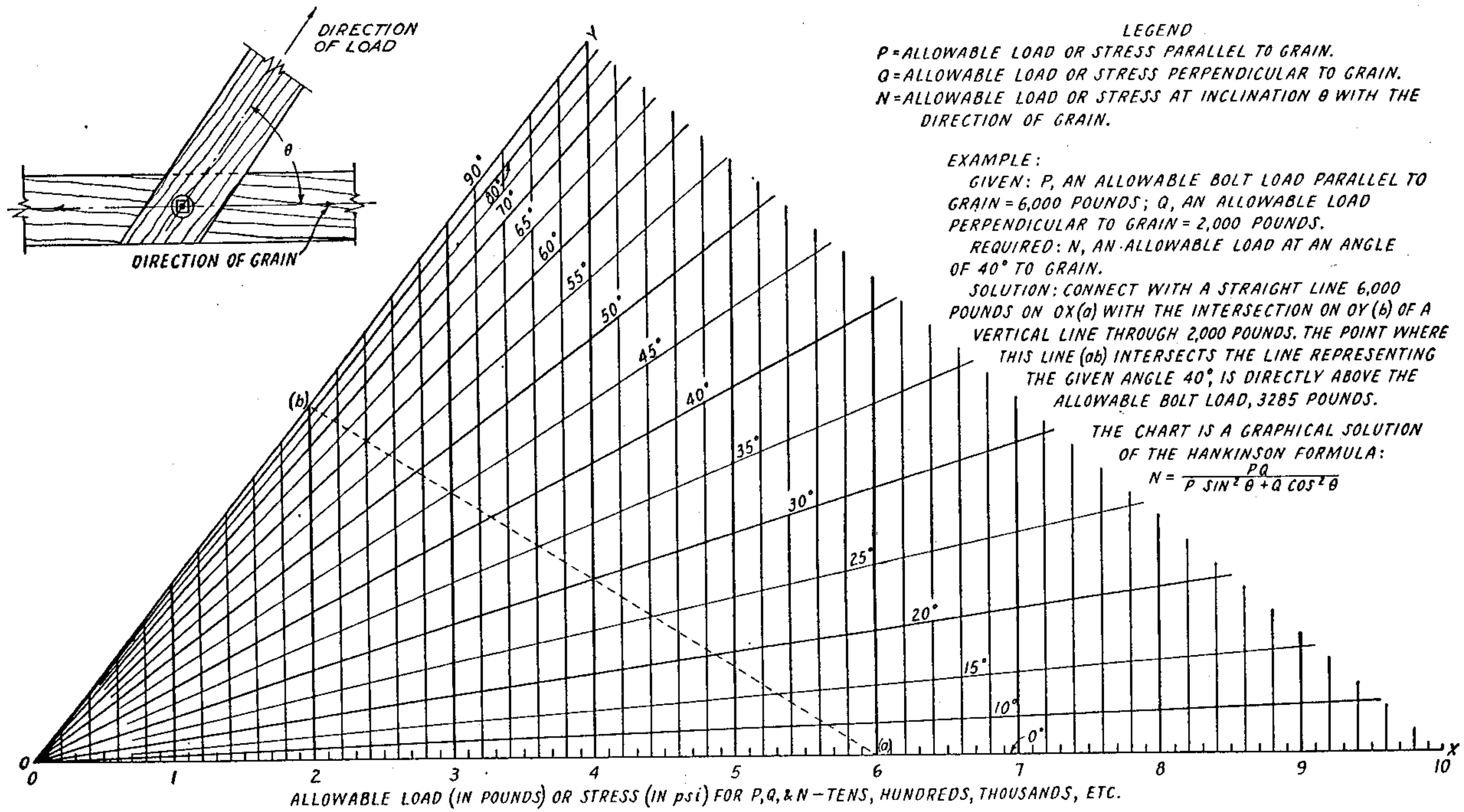


Figure 2-62. Bolt bearing stresses perpendicular to grain.



SCHOLTEN NOMOGRAPH FOR DETERMINING BEARING STRENGTH OF WOOD AT VARIOUS ANGLES TO THE GRAIN
(REPRODUCED BY PERMISSION OF THE FOREST PRODUCTS LABORATORY)

Figure 2-63. Scholten nomograph for determining bearing strength of wood at various angles to the grain.

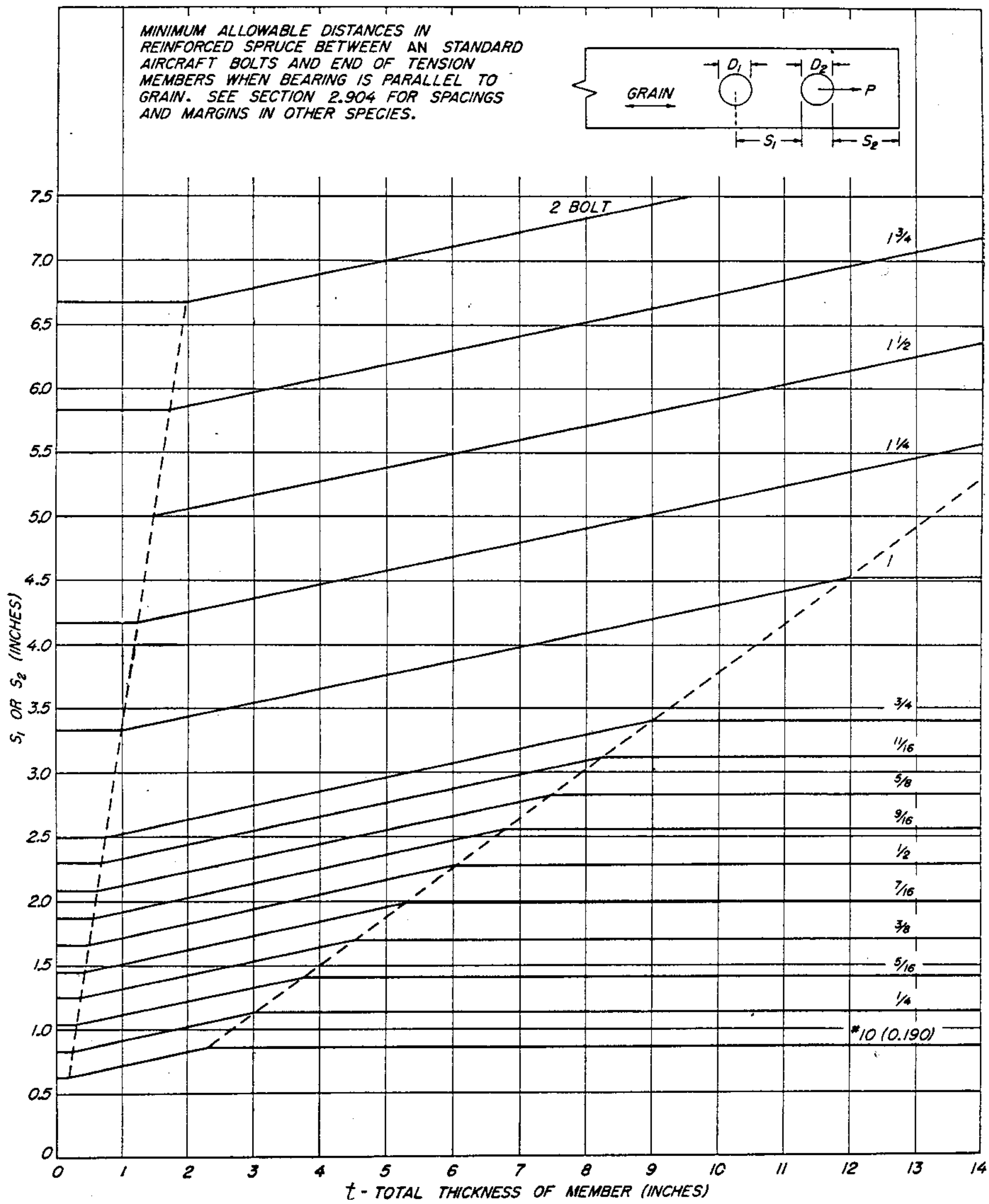


Figure 2–64. Allowable distances between bolts and allowable end margin for bolts in spruce members having cross-banded reinforcing plates when bearing is parallel to grain of spruce.

The minimum distance from the edge of a bolt to the end of a member subject to compression should be 3½ bolt diameters.

(2) *Spacing perpendicular to the grain.* The minimum distance between the edges of adjacent bolts or between the edge of the member and the edge of the nearest bolt should be one bolt diameter for all species. It is recommended that the stress in the area remaining to resist tension at the critical section through a bolt hole not exceed two-thirds the modulus of rupture in static bending when cross-banded reinforcing plates are used; otherwise one-half the modulus of rupture shall not be exceeded.

(3) *When a bolt load is less than the allowable load parallel to the grain,* the spacing may be reduced in the following way: The bolt spacing given in figure 2–64 can be multiplied by the ratio of actual load to allowable load except that the spacing should be not less than three bolt diameters. The bolt spacing perpendicular to the grain cannot be reduced below one bolt diameter.

2.9041. *Spacing of bolts loaded perpendicular to the grain.*

(1) *Spacing perpendicular to the grain.* The minimum distance from the edge of a bolt to the edge of the member toward which the bolt pressure is acting should be 3½ bolt diameters. The margin on the opposite edge and the distance between the edges of adjacent bolts should be not less than one bolt diameter.

(2) *Spacing parallel to the grain.* The minimum distance between edges of adjacent bolts should be three bolt diameters and the distance between the end of the member and the edge of the nearest bolt should be not less than four bolt diameters.

(3) *When a bolt load is less than the allowable load perpendicular to the grain,* all bolt spacings may be multiplied by the ratio of actual load to allowable load except that the spacing should be not less than one bolt diameter. The distance between the end of the member and the edge of the nearest bolt, measured parallel to the grain, should be not less than three bolt diameters, however.

2.9042. *Spacing of bolts loaded at an angle to the grain.* When bolts are loaded at some angle to the grain, the load can be resolved into components parallel and perpendicular to the grain and the spacings thereafter determined in accordance with sections 2.9040 and 2.9041.

2.9043. *General notes on bolt spacing.* When bushings are used in combination with bolts, the spacing should be based upon the outside diameter of the bushing. When adjacent bolts or bushings are of different diameters, the spacing should be based upon the larger.

When staggered rows of bolts are employed in design, the distance between the center lines of adjacent bolt rows should be not less than the sum of the diameters of the largest bolt in each row.

2.905. *Bearing in wood-base materials.*

2.9050. *Bearing in plywood* (ref. 2–47). For plywood constructed of a single species in accordance with Specification AN–P–69a (*Plywood and Veneer; Aircraft Flat Panel*) or any other approximately balance construction (nearly equal thickness of material in both directions), the proportional limit bearing strength under solid steel aircraft bolts loaded at any angle to the face grain can be determined from figure 2–65. The proportional limit stress expressed in terms of the ultimate compressive stress is related to diameter of bolt for various thicknesses of plywood. Ultimate loads can be assumed to be at least 50 percent above these values.

For appreciably unbalanced constructions or for balanced constructions in which the use of two species results in an appreciable difference between F_{cuw} and F_{cux}, the proportional limit bearing stresses under aircraft bolts may be found by multiplying the appropriate ratio from figure 2–65 by F_{cuw} for bolts loaded at 0° to the face grain and by F_{cux} for bolts loaded at 90° to the face grain. For loadings at other angles, the proportional limit stresses may be found by straight-line interpolation between values found by the procedures given above for loadings at 0° and 90°.

The minimum distance from the edge of a bolt to the edge of a member in a single-bolt connection loaded parallel to the face grain is one diameter for either tensile or compressive loading. When the face grain is at 45° or 90° to the direction of loading, the edge distance must not be less than one and one-half diameters. Where several bolts disposed along the center line are employed in a connection, the edge distance should be determined by multiplying the single-bolt edge distance given above by

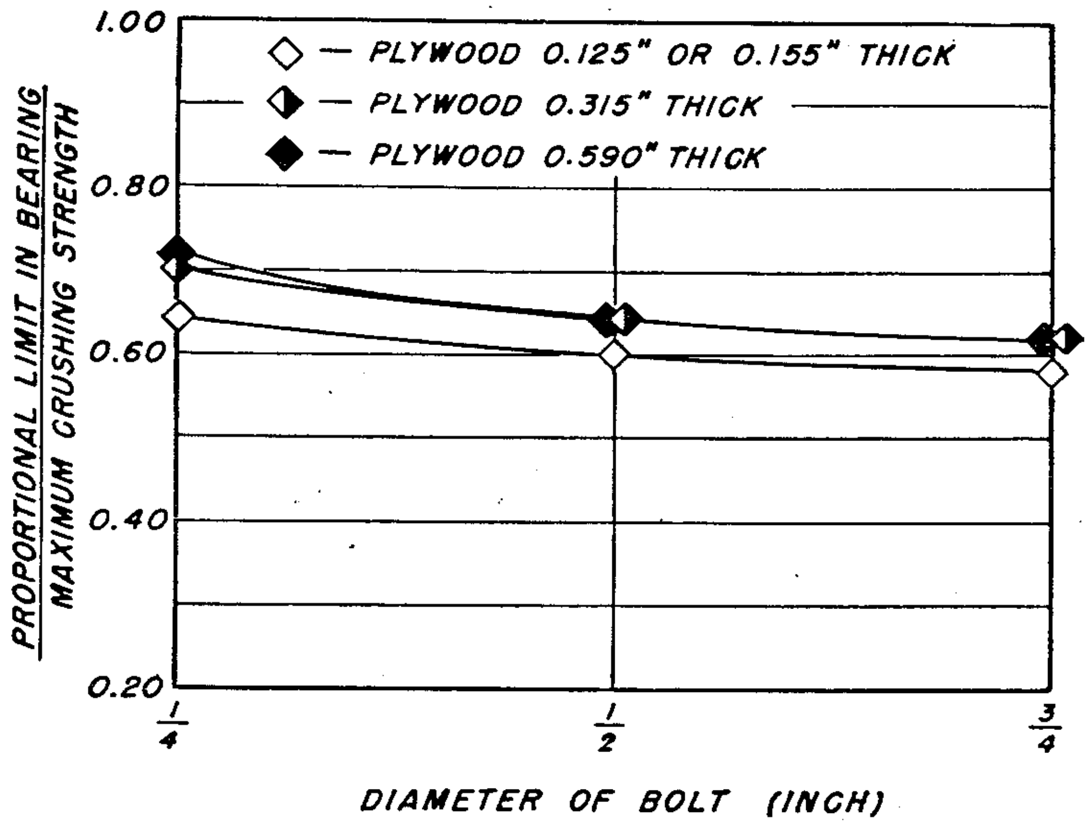


Figure 2–65. Relation of proportional limit bearing stress to maximum crushing strength of plywood for three bolt diameters and four thicknesses of plywood.

the number of bolts. Where bolts are disposed in two lines, each line should coincide with the center line of the half-width of the member in which the line of bolts is placed, and the edge distance for each line should be equal to the number of bolts in that line multiplied by the edge distance for a single bolt.

The minimum distance from the edge of the bolt to the end of the member is two diameters under tensile loading for any grain orientation. For compressive loading a minimum of one diameter should be used.

The most common use in which plywood will have to sustain boltbearing loads will be as reinforcing plates on solid wood members (sec. 2.906).

2.9051. *Bearing in compreg* (ref. 2–31). For cross-banded compreg of approximately balanced construction that conforms to AAF Specification 15065–B (*Panels: Compressed Wood, Impregnated*), the bearing strength under solid-steel aircraft bolts loaded at any angle to the grain can be determined from figure 2–66. The stress at proportional limit and at ultimate, expressed in terms of the ultimate compressive stress, is related to bolt diameter for several thicknesses of compreg. Ultimate loads are at least 50 percent above the proportional limit value.

No variation in bearing strength with direction of loading has been noted for unbalanced constructions tested. It is suggested, however, that when the unbalance exceeds a 60–40 relationship, the bearing stresses may be found by multiplying the appropriate ratio from figure 2–66 by F_{cuw} for bolts loaded at 0° to the face grain and by F_{cux} for bolts loaded at 90° to the face grain. For loadings at other angles, the bearing stresses may be found by straight-line interpolation between values found by the procedures outlined above for loadings at 0° and 90°.

For a single-bolt joint under compressive loading, the minimum distance from the edge of the bolt to the edge of the member is one and one-half diameters for any grain orientation. The distance from the edge of the bolt to the end of the member should be at least one bolt diameter.

For a single-bolt joint loaded in tension, the minimum end or edge distances are the same and vary with the face grain orientation as follows: parallel and perpendicular to face grain, 4½ diameters; 45° to face grain, 2½ diameters.

For connections employing more than one bolt, edge distances should be determined as indicated for plywood in section 2.9050.

At a ratio of bearing length to bolt diameter of

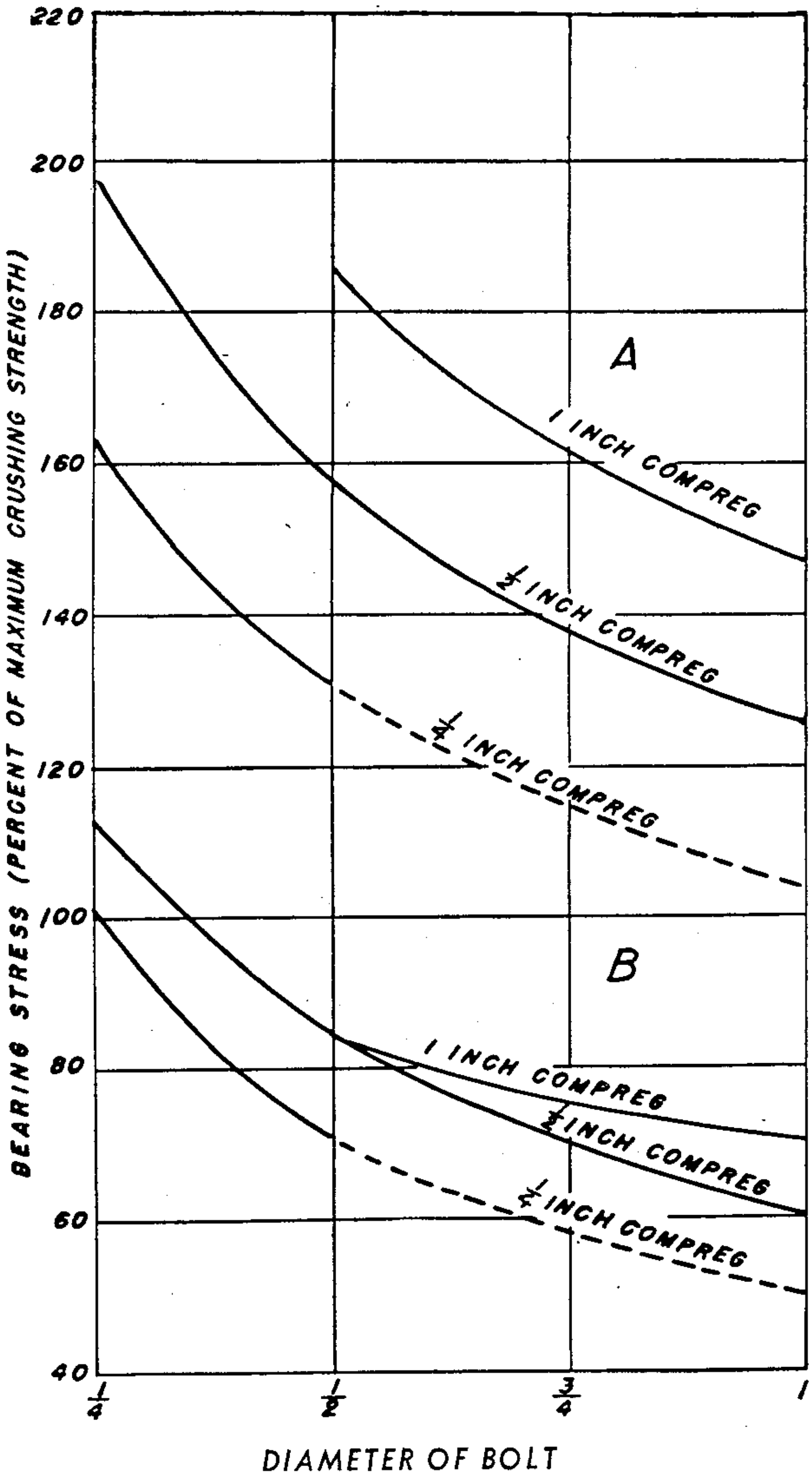


Figure 2–66. Ultimate (A) and proportional limit (B) bearing stresses of commercial cross-banded compreg expressed as percent of ultimate compressive stress for three diameters of bolts (¼, ½, and 1 inch) and for three thicknesses of compreg (¼, ½, and 1 inch).

4, the bearing strength of compreg exceeds the double shear strength of the bolt.

The most common use in which compreg will have to sustain bearing loads will be as reinforcing plates on solid wood members.

2.906. *Bearing in reinforced members* (ref. 2–68).

2.9060. *Wood members with plywood reinforcing plates.* The allowable limit bearing load parallel to the grain of members symmetrically reinforced with plywood (AN–P–69a), the thickness of which (two plates) is 10 to 30 percent of the total thickness of the member and reinforcement, under solid steel aircraft bolts may be determined as follows:

(1) Select tentative thickness of reinforcement and diameter of bolt.
(2) Compute the L/D ratio based on the total length of the bolt in bearing.
(3) From figure 2–60 read the ordinate on the proportional limit curve for the wood member corresponding to the L/D ratio found in (2).
(4) From figure 2–65 read the ordinate on the proportional limit curve for the thickness of reinforcement (one plate) and at the bolt diameter chosen.
(5) Multiply the factors determined in steps (3) and (4) by the appropriate maximum crushing strengths to obtain the allowable proportional limit bearing stresses of the materials involved.
(6) Multiply the stresses so obtained by the corresponding bearing areas to obtain the bearing load for each material. The summation of these bearing loads closely approximates the proportional limit bearing strength of the reinforced member, being only slightly conservative.

If the ratio of the ultimate stress to the proportional limit stress indicated by the curve of figure 2–61 for the construction chosen is less than the ratio of ultimate load to limit load (usually 1.5) specified by the design requirements, it is obvious that the limit bearing load chosen for use in design must be less than the load corresponding to proportional limit stress computed by the steps outlined above, and will be equal to the computed load multiplied by the ratio of the ordinate from the curve of figure 2–61 to the desired ratio. If on the other hand, the ratio from figure 2–61 is greater than that specified, the design ultimate bearing load must be less than the actual ultimate load if the limit bearing load is to be no greater than the bearing load at proportional limit.

The preceding method applies to plywood reinforcing plates regardless of the angle between the load and the face grain direction.

The allowable concentric bearing load perpendicular to the grain can be obtained in a similar manner except that in step (3) figure 2–62 shall be used.

When the load on a bolt is applied at an angle between 0° and 90° to the grain, the allowable load on the wood member may be computed by substituting in equation (2:123) the parallel and perpendicular bearing loads determined by the methods outlined in the preceding paragraphs. For loads on the reinforcing plates refer to section 2.9050.

2.9061. *Wood members with cross-banded compreg reinforcing plates.* The allowable bearing stress parallel to the grain of wood members symmetrically reinforced with cross-banded compreg, the thickness of which (two plates) is 10 to 30 percent of the total thickness of the member and reinforcement, under solid steel aircraft bolts may be determined as follows:

(1) Select tentative thickness of reinforcement and diameter of bolt.
(2) Compute the L/D ratio based on the total length of the bolt in bearing.
(3) From figure 2–60 read the ordinate on the ultimate stress curve corresponding to the L/D ratio found in (2).
(4) From figure 2–66 read the ordinate on the ultimate stress curve for the thickness of reinforcement (one plate) and at the bolt diameter chosen in (1).
(5) Multiply the factors determined in steps (3) and (4) by the appropriate maximum crushing strengths to obtain the allowable bearing stresses of the materials involved.
(6) Multiply the stresses so obtained by the corresponding bearing areas to obtain the maximum bearing load for each material. The summation of these bearing loads is the maximum bearing strength.

The ratio of the ultimate stress to the proportional limit may be obtained from the curves in figure 2–61.

When compreg reinforcing plates are applied to members under tensile loading, the grain of the compreg must be at 45° to the direction of loading.

The allowable concentric bearing load perpendicular to the grain can be obtained in a similar manner except that in step (3) figure 2–62 shall be used.

When the load on a bolt is applied at an angle between 0° and 90° to the grain, the allowable load on the wood member may be computed by substituting in equation (2:123) the parallel and perpendicular bearing loads determined by the methods outlined in the preceding paragraphs. For loads on the reinforcing plates, refer to section 2.9051.

2.907. *Bushings.* Bushings of light alloys or fiber materials may be used to increase the bearing strength of bolts. Since the possible combinations of materials for bolts and bushings are numerous, a specific set of allowable loads for all possible combinations cannot be given.

The allowable bearing loads for aluminum bushings used in combination with steel bolts, and for other combinations of materials, should be determined by a special test.

2.908. *Hollow bolts.* The use of hollow bolts with comparatively thin walls for bearing in wood is not recommended, as tests at the Forest Products Laboratory show that such bolts are little if any more efficient on a weight basis than solid bolts. When used, the allowable stress parallel to the grain may be obtained from N. A. C. A. Technical Note 296 (ref. 2–77). In general, tests should be made to determine the allowable loads at other angles to the grain.

2.909. *General features of bolted joints.*

2.9090. *Drilling of holes* (ref. 2–27). In order to use the bolt-bearing stresses shown in the preceding sections, holes must have accurate alinement and spacing and the surfaces must be smooth and true. This requires control of rate of feed and rotational speed as well as selection of the proper type of drill. Most successful results have been obtained with a twist drill carefully centered in the chuck, rotated at the highest speed compatible with a reasonable drill life, and fed at a rate that will produce cutting, not tearing. In general, the smoothest hole produces the most desirable bolt-bearing characteristics.

2.9091. *Repeated loading of bolted joints.* The proportional limit load may be repeatedly applied without producing an appreciable increase in the deformation or "slip" of the joint. In general, loads as high as 75 percent of the ultimate may be safely repeated without excessive deformation. Since this is close to the proportional limit for low L/D ratios, it is seen that the amount the load may be increased above the proportional limit increases with the L/D ratio. Since in a few cases in reinforced members the maximum safe value is below 75 percent of ultimate, it is probably best to consider the proportional limit to be the optimum limit load.

2.91. GLUED JOINTS.

2.910. *Allowable stress for glued joints* (ref. 2–48).

(1) An allowable glue joint stress equal to one-third F_{su} (column 14 of table 2–6) for softwoods or one-half F_{su} for hardwoods for the weaker species in the joint should be used for all plywood-to-plywood or plywood-to-solid-wood joints regardless of face grain direction and for joints between solid wood members in which the relative grain direction is essentially perpendicular. The reduction for joints in which the face grain direction of the plywood is parallel to the grain of the solid wood is necessary primarily because of the unequal stress distribution common to most plywood glue joints.

(2) The allowable shear stress on the glue area for all joints between pieces of solid wood having parallel-grain gluing, is equal to the allowable shear stress parallel to the grain for the weaker species in the joint. This value is found in column 14 of table 2–6 and should be used only when uniform stress distribution in the glue joint is assured.

(3) The allowable shear stress on the glue area for joints between pieces whose grain directions make an angle of other than 0° or 90° may be found by use of formula (2:123) (sec. 2.901), using allowable values for 0° and 90° joints computed as in (1) and (2) above. Figure 2–64 may be used for a graphical solution of formula (2:123). When the angle between the grain directions of the adjacent pieces does not exceed 15°, the shearing strength allowed for parallel-grain gluing as described in (2) above may be assumed to apply without correction.

2.911. *Laminated and spliced spars and spar flanges.* Requirements for laminated and spliced spars and spar flanges are presented in ANC-19, *Wood Aircraft Inspection and Fabrication* (ref. 2-24). Provisions for limiting the location of scarf joints and for the required slope of grain are included.

2.912. *Glue stress between web and flange.* The stress on the glue area between web and flange may be determined by dividing the maximum shear per inch in plywood by the area of contact per inch. For example, the shear stress on the area of contact is

$$f_g=\frac{f_s t}{d}=\frac{q}{d} \qquad (2{:}125)$$

where

f_g=shear stress on the area of contact
f_s=the maximum shear stress in the plywood
t=thickness of one web
d=depth of the flange
q=shear per inch in the plywood

The allowable stress is determined according to section 2.910. If, for example, the flange were of spruce and the web of mahogany-yellow-poplar, the allowable stress would be one-third the value for spruce, or 330 pounds per square inch.

2.92. Properties of Modified Wood. It is at times desirable to impart modified properties to wood for reinforcement at joints, bearing plates, and for other specific uses. Such modifications can be obtained by treating with synthetic resins, by compressing, or by a combination of treating and compressing.

Investigations at the Forest Products Laboratory have produced several types of modified-wood combinations, such as "impreg," "compreg," "semicompreg," and "staypak," which are described in ANC Bulletin 19 (ref. 2-24). When the resin is set within the structure by the application of heat prior to the application of assembly pressures, thus greatly limiting the compression of the wood, the material is called "impreg." When the treated wood is subjected to pressures in the range of 1,000 to 3,000 pounds per square inch prior to the setting of the resin, resulting in a product with a specific gravity of 1.2 to 1.4, the material is called "compreg." Resin-treated wood with specific gravity values between that of impreg and compreg is known as "semicompreg." Ordinary laminated wood or solid wood with no resin within the intimate structure when compressed under conditions that cause some flow of lignin is known as "staypak." It differs from material made according to conventional pressing methods in that the tendency to recover its original dimensions when exposed to swelling conditions has been practically eliminated.

Some properties of parallel-laminated and cross-laminated modified wood made by the Forest Products Laboratory from 17 plies of 1/16-inch rotary-cut yellow birch, sweetgum, and Sitka spruce veneer are presented in tables 2-16 through 2-21 (ref. 2-18), in which average values resulting from the specified number of tests, together with maximum and minimum values are given. Values for normal laminated wood (controls), impreg, semicompreg, compreg, and staypak are presented. Conclusions drawn from these comparative tests must be regarded only as indicative, because the number of tests is limited.

2.920. *Detailed test data for tables 2-16 to 2-21, inclusive.* Specimens for test were obtained from three sets of 24- by 24-inch panels of each of the three species. Each set consisted of two seventeen-ply panels of each of the five materials, one panel parallel-laminated and one cross-laminated. Panels of a set were formed by assembling corresponding plies of the panels from successive sheets of veneer as it came from the lathe. So far as possible, the veneer for each set was taken from a different log or bolt.

Except as otherwise noted, tests were made on specimens with the original or formed surfaces of the material undisturbed. In general, an equal number of specimens was tested from each of the two principal grain directions, lengthwise and crosswise (0° and 90°), that is, parallel and perpendicular, respectively, to the grain of parallel-laminated panels, and to the face grain of the cross-laminated panels.

Table 2–16. Some properties of parallel-laminated modified wood made by the Forest Products Laboratory from 17 plies of 1/16-inch rotary-cut yellow birch veneer

Thickness at test, specific gravity, type of test and property	Normal laminated wood [1] Unimpregnated, uncompressed				Impreg [2] Impregnated, uncompressed				Semicompreg [2] Impregnated, moderately compressed				Compreg [2] Impregnated, highly compressed				Staypak [1] Unimpregnated, highly compressed			
	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
Thickness (t) of laminates......inch		0.944	0.921	0.972		0.990	0.905	1.052		0.884	0.881	0.889		0.665	0.620	0.709		0.483	0.458	0.504
Specific gravity (based on weight and volume at test)		0.73	0.69	0.79		0.94	0.84	1.03		1.00	0.92	1.07		1.34	1.31	1.36		1.40	1.36	1.43
Test and property:																				
1. Tension—parallel to grain (lengthwise)	6				6				6				6				5			
Proportional limit stress psi		14,780	13,160	16,310		13,780	11,160	15,620		13,360	10,880	15,200		22,150	21,150	24,340		22,370	18,920	24,780
Ultimate strength...psi		22,180	20,140	25,600		16,990	12,110	20,140		18,090	13,760	20,880		30,290	28,670	32,150		45,070	44,080	49,900
Modulus of elasticity 1,000 psi		2,300	2,006	2,712		2,495	2,076	2,814		2,742	2,423	3,024		3,701	3,468	4,041		4,610	4,380	4,860
Elongation [3]....Percent		1.02	0.94	1.06		0.70	0.65	0.74		0.62	0.55	0.75		0.79	0.75	0.85		1.08	0.97	1.15
2. Tension — perpendicular to grain (crosswise)	9				8				5				6				9			
Proportional limit stress psi		750	510	1,010		960	540	1,460		1,200	1,030	1,300		1,570	1,140	1,920		1,890	1,660	2,150
Ultimate strength...psi		1,420	1,080	1,630		1,080	820	1,460		1,440	1,250	1,710		1,980	1,490	2,540		3,300	3,120	3,660
Modulus of elasticity 1,000 psi		166	150	187		320	272	410		324	293	370		923	878	977		575	502	672
Elongation [3]....percent		1.00	0.69	1.18		0.34	0.29	0.46		0.45	0.34	0.58		0.23	0.16	0.27		0.88	0.74	1.04
3. Compression — parallel to grain (edgewise)[4]	12				12				12				12				11			
Proportional limit stress psi		6,440	5,340	7,680		8,580	7,370	9,520		9,310	7,660	10,060		16,400	14,340	17,960		9,710	8,000	12,540
Ultimate strength...psi		9,550	8,740	11,120		15,590	13,840	17,000		16,030	14,840	17,400		27,800	26,760	28,650		19,100	18,360	20,560
Modulus of elasticity 1,000 psi		2,324	2,102	2,670		2,602	2,150	2,894		2,799	2,464	3,180		3,563	3,352	3,870		4,676	4,372	5,392
4. Compression—perpendicular to grain (edgewise)[4]	12				12				12				12				12			
Proportional limit stress psi		670	610	740		1,080	900	1,270						7,980	6,260	9,760		2,620	1,400	3,450
Ultimate strength...psi		2,100	1,860	2,550		5,630	4,510	6,510		6,120	5,160	7,140		18,550	17,310	19,610		9,370	9,010	10,100
Modulus of elasticity 1,000 psi		162	147	175		301	243	344		324	289	376		788	757	836		583	456	627
5. Compression—perpendicular to grain (flatwise)[5]	12				12												7			
Proportional limit stress psi		1,030	900	1,200		2,240	1,870	3,170	3	2,290	1,890	2,580	12	8,810	6,560	11,640		5,540	4,870	6,000
Maximum crushing strength...psi						5,000	3,700	6,500	6	6,120	4,980	7,220	5	16,340	13,360	18,980		13,260	13,750	14,100
6. Flexure—grain parallel to span (flatwise)[5]	12				12				12				12				12			
Proportional limit stress psi		11,550	9,560	13,900		14,660	11,130	18,100		16,850	14,600	20,480		21,650	19,380	25,270		20,150	15,460	25,360
Modulus of rupture..psi		20,430	18,740	24,650		20,730	15,950	26,270		22,780	19,660	26,190		35,640	34,080	38,600		39,420	35,050	43,120

Modulus of elasticity 1,000 psi		2,317	1,986	2,796		2,550	2,148	2,820		2,806	2,502	3,246		3,476	3,250	3,689		4,449	4,250	4,654
Work to proportional limit. In.-lb. per cu. in.		3.21	2.32	3.84		4.71	3.11	6.61		5.65	4.40	7.42		7.52	6.01	10.14		5.18	2.85	8.05
Work to maximum load In.-lb. per cu. in.		19.9	15.2	26.1		11.1	6.8	18.1		11.9	9.3	15.1		27.1	23.8	31.9		46.1	31.2	52.9
7. Flexure—grain perpendicular to span (flatwise) [5]	12				12				10				12				12			
Proportional limit stress psi		1,030	860	1,180		1,480	910	1,890		1,320	940	1,500		4,200	3,840	4,660		3,190	2,770	4,140
Modulus of rupture psi		1,920	1,750	2,180		1,690	1,110	2,260		1,730	1,270	2,140		5,550	4,050	6,680		5,000	4,160	5,620
Modulus of elasticity 1,000 psi		153	135	170		301	246	357		326	265	406		752	708	781		602	528	666
Work to proportional limit. In.-lb. per cu. in.		0.39	0.28	0.57		0.44	0.13	0.64		0.33	0.12	0.47		1.31	1.06	1.69		0.95	0.60	1.48
Work to maximum load In.-lb. per cu. in.		1.66	1.34	2.39		0.58	0.21	0.97		0.60	0.26	0.95		2.44	1.27	3.50		3.0	1.7	4.4
8. Shear strength—parallel to grain (edgewise): [4]																				
a. Single shear across laminations psi	11	2,620	2,390	2,800	12	2,030	1,760	2,580	12	2,090	1,830	2,360	12	4,070	3,160	5,010	12	4,810	3,900	5,670
b. Johnson, double shear across liminations psi	12	2,980	2,850	3,110	6	3,460	2,640	4,460	12	4,430	3,880	4,680	12	7,370	6,780	8,270	8	6,370	6,130	6,550
c. Cylindrical double shear parallel to laminations psi	9	3,030	2,410	3,270	9	3,540	3,140	3,980					9	6,480	5,100	7,520	8	3,080	2,600	3,870
9. Modulus of rigidity (G): [6]																				
a. Plate shear (FPL test) 1,000 psi					3	217	195	236	3	207	195	220					2	385	367	403
b. Torsion method 1,000 psi	6	182	163	208					3	207	196	228	6	333	315	359				
10. Toughness (FPL test, edgewise) [4] In.-lb.	12	235	174.3	280.6	12	151.2	98.7	182	12	166	100.9	236.5	6	161.2	136.8	173.1	12	248.4	183.9	302.9
Toughness In.-lb. per in. of width		250.6	189.1	303.7		152.4	94.6	179.2		187.7	114.6	267.3		240.6	193	277.4		514.5	370.8	617.0
11. Impact strength (Izod) [5] Ft.-lb. per in. of notch	13	12.65	7.05	20.63	15	1.97	1.13	2.58	14	3.17	1.79	5.61	15	5.40	4.0	6.74	15	12.72	11.60	14.37
12. Water absorption (24-hour immersion) percent					9	7.93	5.6	10.8	9	8.93	7.5	10.6	9	0.97	0.64	1.30	9	4.33	3.90	4.8
13. Dimensional stability of thickness (t)					9				9				9				9			
Equilibrium swelling plus recovery percent						4.5	3.5	6.0		6.0	5.7	6.3		8.4	6.5	10.6		33.7	30	37
Recovery from compression percent																		4.3	1.1	8.9
Equilibrium swelling percent						4.5	3.5	6.0		6.0	5.7	6.3		8.4	6.5	10.6		20.4	28.1	31.1

[1] Veneer conditioned at 80° F. and 65 percent relative humidity prior to assembly with film glue. No other resin employed. The average moisture content of the normal laminates at test was 9.2 percent (range 8.3 to 9.9).

[2] Total resin content 49 to 54 percent, impregnating resin content 42 to 49 percent on the basis of the dry weight of the untreated veneer.

[3] Total elongation immediately before fracture measured over a 2-inch gage length.

[4] Load applied to the edge of the laminations (perpendicular to laminating-pressure direction).

[5] Load applied to the surface of the original material (parallel to laminating-pressure direction).

[6] Modulus associated with shear distortions in planes parallel to the plane of the laminations.

Table 2-17. Some properties of parallel-laminated modified wood made by the Forest Products Laboratory from 17 plies of 1/16-inch rotary-cut sweetgum veneer

Thickness at test, specific gravity, type of test and property	Normal laminated wood [1] Uninpregnated, uncompressed				Impreg [2] Impregnated, uncompressed				Semicompreg [2] Impregnated, moderately compressed				Compreg [2] Impregnated, highly compressed				Staypak [1] Unimpregnated, highly compressed			
	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
Thickness (t) of laminates.....inch		1.014	1.006	1.021		1.015	1.003	1.032		0.716	0.691	0.742		0.564	0.533	0.582		0.436	0.391	0.469
Specific gravity (based on weight and volume at test)		0.59	0.54	0.62		0.73	0.68	0.76		1.05	1.03	1.09		1.35	1.33	1.36		1.36	1.30	1.40
Test and property:																				
1. Tension—parallel to grain (lengthwise)	6				6				6				6				5			
Proportional limit stress p. s. i.		9,510	8,480	10,660		8,630	6,700	12,160		12,140	9,240	13,200		19,900	17,840	21,620		19,040	15,140	24,700
Ultimate strength..do.		14,540	12,830	15,800		10,200	8,660	12,940		19,570	17,600	21,180		26,740	23,270	28,540		32,700	28,700	35,810
Modulus of elasticity 1,000 p. s. i.		1,622	1,500	1,808		1,861	1,731	1,957		2,530	2,399	2,688		2,968	2,806	3,121		3,765	3,498	4,116
Elongation [3]....percent	5	0.93	0.90	1.02		0.56	0.45	0.67		0.80	0.73	0.86		0.92	0.79	1.00		0.96	0.79	1.04
2. Tension—perpendicular to grain (crosswise)	9				9				6				8				8			
Proportional limit stress p. s. i.		460	350	580		590	440	800		930	560	1,160		3,030	2,470	3,210		1,410	880	2,450
Ultimate strength.p. s. i.		1,000	860	1,090		700	510	880		1,150	880	1,440		3,870	3,570	5,090		2,530	2,390	3,480
Modulus of elasticity 1,000 p. s. i.		125	100	143		215	167	290		436	367	549		798	761	849		515	456	770
Elongation [3]....percent	8	0.96	0.74	1.28		0.33	0.22	0.48	4	0.24	0.16	0.35		0.48	0.44	0.67		0.84	0.65	1.10
3. Compression—parallel to grain (edgewise)[4]	12				12				12				12				12			
Proportional limit stress p. s. i.		4,510	3,880	5,180		7,550	6,160	8,600						9,320	7,900	11,760		9,030	6,470	11,120
Ultimate strength.p. s. i.		6,850	6,150	7,540		12,600	11,300	13,680		17,740	16,470	19,060		20,910	20,100	21,840		16,810	13,450	18,240
Modulus of elasticity 1,000 p. s. i.		1,693	1,541	1,842		1,873	1,695	2,049		2,616	2,420	2,799		3,350	3,030	3,820		3,852	3,380	4,228
4. Compression—perpendicular to grain (edgewise)[4]	12				12				11				11				12			
Proportional limit stress p. s. i.		340	310	420		700	600	880		2,460	2,000	2,870						2,470	1,430	3,050
Ultimate strength.p. s. i.		1,280	1,110	1,440		3,020	2,760	3,270		7,850	7,140	8,640		14,790	14,220	15,060		8,470	6,810	10,000
Modulus of elasticity 1,000 p. s. i.		120.5	105.6	133.3		199.0	173.3	219		422	316	495		666	632	698		511	367	630
5. Compression—perpendicular to grain (flatwise)[5]	12				12															
Proportional limit stress p. s. i.		780	650	950		1,820	1,570	2,070	3	2,570	2,400	2,910								
Maximum crushing strength.....p. s. i.						3,000	2,740	3,340	4	7,000	6,850	7,190	11	20,560	18,840	23,300	7	12,230	11,740	13,620
6. Flexure—grain parallel to span (flatwise)[5]	12				12				12				12				12			
Proportional limit stress p. s. i.		8,170	7,030	9,760		9,580	6,990	12,500		15,910	13,710	18,480		19,940	16,920	22,730		17,680	15,630	20,600
Modulus of rupture p. s. i.		14,330	12,420	15,720		12,930	10,880	15,370		21,010	17,280	23,960		33,540	30,200	36,150		36,390	32,530	41,080

Modulus of elasticity 1,000 p. s. i.		1,586	1,346	1,760		1,859	1,782	2,024		2,458	1,922	2,781		3,168	2,960	3,398		3,805	3,416	4,473
Work to proportional limit In.-lb. per cu. in.		2.35	1.92	3.00		2.88	1.37	4.86		5.68	4.24	7.20		7.16	4.86	9.69		4.58	3.78	5.66
Work to maximum load In.-lb. per cu. in.		13.94	12.06	17.16		5.38	3.56	7.46		10.8	5.84	14.7	6	26.9	22.9	30.2		48.8	37.6	60.3
7. Flexure—grain perpendicular to span (flatwise)[5]	12				12				8				12							
Proportional limit stress p. s. i.		650	560	730		820	560	1,170		1,830	1,180	2,110		4,590	3,330	5,340	12	2,290	1,630	2,900
Modulus of rupture p. s. i.		1,280	1,060	1,470		1,180	660	1,470		2,090	1,520	2,450		6,580	5,760	8,000		4,130	3,520	5,110
Modulus of elasticity 1,000 p. s. i.		105	84.8	121		195	175	213		347	284	392		749	700	858		522	408	622
Work to proportional limit In.-lb. per cu. in.		0.23	0.20	0.28		0.20	0.09	0.40		0.57	0.20	0.77		1.61	0.86	2.24		0.57	0.28	0.86
Work to maximum load In.-lb. per cu. in.		1.14	0.78	1.39		0.44	0.13	0.71		0.84	0.32	1.46		3.50	2.68	4.36		2.35	1.61	3.31
8. Shear strength—parallel to grain (edgewise):[4]																				
a. Single shear across laminations. p. s. i.	12	2,170	1,850	2,390	12	1,880	1,450	2,260	12	2,710	2,250	3,100	12	4,060	2,630	5,080	12	4,250	3,370	4,760
b. Johnson, double shear across laminations p. s. i.	12	2,080	1,870	2,390	12	2,570	2,030	2,890	12	4,500	4,070	4,920	12	7,290	6,700	7,630	11	5,860	4,910	6,520
c. Cylindrical double shear parallel to laminations. p. s. i.	9	2,000	1,590	2,300	9	2,940	2,570	3,310					11	7,000	5,950	8,220	11	2,780	2,000	3,640
9. Modulus of rigidity (G):[6]																				
a. Plate shear (FPL test) 1,000 p. s. i.					2	153	143	163	2	168	153	183	3	362	355	368	2	372	360	384
b. Torsion method 1,000 p. s. i.	6	132	108	149																
10. Toughness (FPL test, edgewise)[4] In.-lb.	12	141.8	130.4	161.3	12	71.8	38.9	90	12	115.3	98.7	132.9	12	126.1	71.6	156.2	11	250.5	181.1	304.7
Toughness In.-lb. per in. of width		140.2	129.1	160		70.8	38.6	89.6		161.1	142.2	179.7		222.7	132.6	269.3		575.0	386.0	686.0
11. Impact strength (Izod)[5] Ft.-lb. per in. of notch	15	8.50	7.01	10.61	15	0.98	0.40	1.92	14	2.10	1.19	3.26	15	2.69	2.32	3.51	10	13.74	12.19	17.96
12. Water absorption (24-hour immersion) percent					9	14.25	10.58	16.63	9	7.44	6.68	8.05	9	1.01	0.89	1.22	9	5.95	4.16	8.89
13. Dimensional stability of thickness (t)					9				9				9				9			
Equilibrium swelling plus recovery percent						3.3	3.1	3.5		6.5	6.3	6.8		5.1	4.8	5.3		29.0	23.3	36
Recovery from compression percent						0.8	0.8	0.8										3.6	0.6	7.9
Equilibrium swelling percent						2.5	2.3	2.7		6.5	6.3	6.8		5.1	4.8	5.3		25.4	22.7	28.1

[1] Veneer conditioned at 80° F. and 65 percent relative humidity prior to assembly with film glue. No other resin employed. The average moisture content of the normal laminates at test was 10.2 percent (range 9.1 to 10.9).

[2] Total resin content 37 to 52 percent, impregnating resin content 36 to 42 percent on the basis of the dry weight of the untreated veneer.

[3] Total elongation immediately before fracture measured over a 2-inch gage length.

[4] Load applied to the edge of the laminations (perpendicular to laminating-pressure direction).

[5] Load applied to the surface of the original material (parallel to laminating-pressure direction).

[6] Modulus associated with shear distortions in planes parallel to the plane of the laminations.

Table 2–18. Some properties of parallel-laminated modified wood made by the Forest Products Laboratory from 17 plies of 1/16-inch rotary-cut Sitka spruce veneer

Thickness at test, specific gravity, type of test and property	Normal laminated wood [1] Unimpregnated, uncompressed				Impreg [2] Impregnated, uncompressed				Semicompreg [2] Impregnated, moderately compressed				Compreg [2] Impregnated, highly compressed				Staypak [1] Unimpregnated, highly compressed			
	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
Thickness (t) of laminates ..... inch		0.992	0.969	1.012		0.858	0.846	0.869		0.601	0.590	0.619		0.430	0.423	0.437		0.336	0.320	0.351
Specific gravity (based on weight and volume at test)		0.46	0.44	0.47		0.66	0.63	0.69		0.96	0.93	1.00		1.31	1.32	1.36		1.36	1.30	1.42
Test and property:																				
1. Tension—parallel to grain (lengthwise)	6				4				6				4				5			
Proportional limit stress p. s. i.		9,950	7,910	11,400										21,980	19,050	27,490		30,840	25,860	31,610
Ultimate strength p. s. i.		13,030	11,400	14,700		9,980	9,340	10,500		15,100	12,300	16,700		21,250	20,560	27,680		47,260	35,800	55,600
Modulus of elasticity 1,000 p. s. i.		1,861	1,590	2,060		2,325	2,258	2,428		3,192	3,007	3,200		4,396	4,128	4,670		5,197	4,603	5,800
Elongation [3] ... percent	4	0.78	0.70	0.91		0.42	0.40	0.44		0.47	0.41	0.51		0.55	0.48	0.62		1.05	0.75	1.25
2. Tension—perpendicular to grain (crosswise)	9				6				9				6				8			
Proportional limit stress p. s. i.		430	370	480		450	330	560		1,320	910	1,870		3,060	2,550	3,610		2,160	1,970	2,500
Ultimate strength p. s. i.		660	510	800		620	540	720		1,570	1,010	1,960		3,800	3,410	4,160		3,790	3,380	4,170
Modulus of elasticity 1,000 p. s. i.		112	102	121		208	155	286		409	370	460		779	706	874		501	357	621
Elongation [3] ... percent	4	0.64	0.6	0.75		0.36	0.32	0.40		0.42	0.31	0.53		0.50	0.43	0.56		1.07	0.91	1.29
3. Compression—parallel to grain (edgewise) [4]	12				8				10				8				12			
Proportional limit stress p. s. i.		5,220	4,360	5,950		8,520	8,060	9,080		12,700	12,140	13,490		15,430	13,340	16,460		8,490	4,300	15,660
Ultimate strength p. s. i.		6,710	6,120	7,120		11,140	9,320	12,020		17,260	16,140	18,390		26,620	23,830	28,050		18,790	15,860	20,680
Modulus of elasticity 1,000 p. s. i.		1,868	1,743	2,034		2,149	2,063	2,255		3,302	3,021	3,513		4,584	4,437	4,870		5,079	4,692	5,390
4. Compression—perpendicular to grain (edgewise) [4]	12				8				12				8				12			
Proportional limit stress p. s. i.		360	270	410		820	690	910		2,850	2,490	3,260		5,700	5,090	6,560		2,150	1,720	2,820
Ultimate strength p. s. i.		1,070	980	1,190		2,610	2,380	2,750		5,920	5,320	6,560		16,980	15,800	20,380		8,090	5,310	9,500
Modulus of elasticity 1,000 p. s. i.		103	92.8	111		182	167	198		402	383	433		807	806	1,060		566	426	678
5. Compression—perpendicular to grain (flatwise) [5]	12				8												9			
Proportional limit stress p. s. i.		307	247	370		770	600	940	3	1,170	950	1,550	3	8,630	6,900	9,680		7,430	4,760	11,750
Maximum crushing strength ..... p. s. i.						3,240	2,600	3,710	6	7,030	6,520	7,460	6	20,190	18,840	21,420		14,760	13,010	18,020
6. Flexure—grain parallel to span (flatwise) [5]	12				8				12				8				12			

Proportional limit stress p. s. i.		7,260	5,780	8,470		8,720	6,180	9,990		14,750	9,700	16,430		26,560	24,380	29,610		21,700	18,480	27,600
Modulus of rupture p. s. i.		11,960	10,960	12,580		11,560	9,990	12,540		17,860	10,940	20,880		37,040	33,370	41,190		37,140	32,540	41,090
Modulus of elasticity 1,000 p. s. i.		1,669	1,579	1,734		2,179	2,130	2,214		2,890	2,650	3,118		4,262	4,007	4,521		4,923	4,120	5,560
Work to proportional limit. In.-lb. per cu. in.		1.78	1.10	2.36		1.98	0.97	2.57		4.21	1.90	5.01		9.21	8.09	11.08		5.41	3.43	8.76
Work to maximum load In.-lb. per cu. in.		11.12	6.22	13.70		3.64	2.76	4.25		6.68	2.84	9.03		20.31	16.86	24.14		32.5	23.3	42.5
7. Flexure—grain perpendicular to span (flatwise)[3]	12				8				12				8				10			
Proportional limit stress p. s. i.		570	410	690		770	720	830		1,670	1,350	2,070		5,510	5,140	6,700		2,660	1,660	3,820
Modulus of rupture p. s. i.		780	580	970		900	780	1,140		2,010	1,580	2,620		5,980	5,270	7,100		4,680	3,580	5,900
Modulus of elasticity 1,000 p. s. i.		89.5	85	91.9		177	167	187		370	341	406		782	740	836		585	500	688
Work to proportional limit. In.-lb. per cu. in.		0.21	0.11	0.29		0.19	0.16	0.22	6	0.42	0.28	0.61		2.19	1.31	3.15		0.70	0.30	1.22
Work to maximum load In.-lb. per cu. in.		0.43	0.25	0.67		0.27	0.19	0.42	12	0.72	0.41	1.42		2.65	2.00	3.65		2.70	1.88	3.66
8. Shear strength—parallel to grain (edgewise):[4]																				
a. Single shear across laminations p. s. i.	12	1,470	1,360	1,550	8	1,540	1,330	1,720	12	2,580	2,300	2,800	7	3,570	2,620	4,410	12	4,830	4,580	5,080
b. Johnson, double shear across laminations p. s. i.	12	1,880	1,780	1,950	8	2,550	2,180	2,970	9	3,930	3,570	4,210	8	7,500	6,730	8,140	11	6,780	5,680	8,050
c. Cylindrical double shear parallel to laminations p. s. i.	8	1,420	1,140	1,700	7	1,530	1,250	2,040					8	2,720	2,060	3,820	10	1,800	1,300	2,340
9. Modulus of rigidity (G):[6]																				
a. Plate shear (FPL test) 1,000 p. s. i.					2	158	156	160	3	250	234	265	2	401	391	410	1	388		
b. Torsion method 1,000 p. s. i.	6	115	103	123																
10. Toughness (FPL test, edgewise[4]) In.-lb.	12	117.7	91.5	147.9	8	47.6	37	61.1	12	65.2	45.4	79.5	8	104	93	117.5	11	146.4	85.1	187.6
Toughness In.-lb. per in. of width		119.2	91.3	152.5		55.6	43.6	70.5		107.8	73.4	132.3		239.7	213.8	274		432	213.2	556.8
11. Impact strength (Izod)[5] Ft.-lb. per in. of notch	9	10.23	7.47	11.78	8	2.20	1.16	3.97	16	3.33	1.75	5.09	10	4.21	2.83	5.48				
12. Water absorption (24-hour immersion) percent					6	15.67	14.36	17.14	9	7.36	6.1	8.9	6	1.46	1.41	1.54	8	6.75	5.8	8.7
13. Dimensional stability of thickness (t)					6				9				6				9			
Equilibrum swelling plus recovery percent						4.7	4.3	5.1		6.2	5.9	6.5		8.4	8.2	8.7		21.8	22.8	28.7
Recovery from compression percent																		1.2	1.0	1.5
Equilibrium swelling percent						4.7	4.3	5.1		6.2	5.9	6.5		8.4	8.2	8.7		23.6	21.8	27.2

[1] Veneer conditioned at 80° F. and 65 percent relative humidity prior to assembly with film glue. No other resin employed. The average moisture content of the normal laminates at test was 8.9 percent (range 8.3 to 9.3).

[2] Total resin content 55 to 58 percent, impregnating resin content 36 to 42 percent on the basis of the dry weight of the untreated veneer.

[3] Total elongation immediately before fracture measured over a 2-inch gage length.

[4] Load applied to the edge of the laminations (perpendicular to laminating-pressure direction).

[5] Load applied to the surface of the original material (parallel to laminating-pressure direction).

[6] Modulus associated with shear distortions in planes parallel to the plane of the laminations.

Table 2-19. Some properties of cross-laminated modified wood made by the Forest Products Laboratory from 17 plies of 1/16-inch rotary-cut yellow birch veneer

Thickness at test, specific gravity, type of test and property	Normal laminated wood [1] Unimpregnated, uncompressed				Impreg [2] Impregnated, uncompressed				Semicompreg [2] Impregnated, moderately compressed				Compreg [2] Impregnated, highly compressed				Staypak [1] Unimpregnated, highly compressed			
	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(14)	(19)	(20)	(21)
Thickness (t) of laminates.....inch..		0.955	0.933	0.980		1.040	1.020	1.051		0.939	0.889	1.009		0.669	0.622	0.720		0.497	0.465	0.531
Specific gravity (based on weight and volume at test)		0.72	0.69	0.77		0.89	0.84	0.95		0.95	0.91	0.99		1.34	1.31	1.35		1.39	1.35	1.41
Test and property:																				
1. Tension—parallel to face grain (lengthwise)	6				6				6				6				6			
Proportional limit stress p. s. i.		7,330	6,030	9,240		7,070	5,200	7,980		7,710	5,760	9,940		8,820	7,960	10,580		15,270	11,080	23,680
Ultimate strength..p. s. i.		12,300	10,030	14,700		7,990	6,760	9,160		9,200	7,770	10,260		16,510	14,300	17,850		21,500	18,660	29,190
Modulus of elasticity 1,000 p. s. i.		1,292	1,083	1,636		1,458	1,278	1,700		1,558	1,456	1,616		2,195	2,071	2,296		2,573	2,242	3,138
Elongation [3].....percent.		1.04	1.00	1.11		0.55	0.47	0.61		0.60	0.50	0.68	2	0.82	0.77	0.88		1.12	0.83	1.41
2. Tension—perpendicular to face grain (crosswise)	8				9				9				9				8			
Proportional limit stress p. s. i.		6,580	5,440	8,300		5,960	4,720	7,380		7,140	6,030	7,760		8,070	6,750	9,290		11,230	8,080	16,420
Ultimate strength..p. s. i.		12,690	10,620	15,290		7,380	6,670	8,140		8,760	6,430	10,320		12,630	10,120	14,610		25,740	22,620	29,400
Modulus of elasticity 1,000 p. s. i.		1,144	1,007	1,403		1,366	1,203	1,478		1,425	1,320	1,490		2,240	2,100	2,328		2,467	2,031	2,969
Elongation [3].....percent.		1.07	1.06	1.09		0.56	0.50	0.61		0.63	0.44	0.75	3	0.51	0.50	0.52	3	1.12	0.99	1.21
3. Compession—parallel to face grain (edgewise) [4]	12				12				12				12				11			
Proportional limit stress p. s. i.		3,330	2,520	4,660		5,280	4,370	6,900		4,900	4,190	5,240		8,710	7,840	9,690		5,230	4,190	6,130
Ultimate strength..p. s. i.		5,810	5,300	6,600		11,460	10,050	13,000		10,960	9,080	11,560		23,950	23,300	25,590		14,000	13,030	15,620
Modulus of elasticity 1,000 p. s. i.		1,362	1,164	1,592		1,505	1,363	1,744		1,532	1,414	1,633		2,327	2,182	2,530		2,701	2,462	3,173
4. Compression—perpendicular to face grain (edgewise) [4]	12				12				12				12				12			
Proportional limit stress p. s. i.		2,740	2,050	3,510										6,950	6,660	7,160		4,920	2,390	5,720
Ultimate strength..p. s. i.		5,420	4,620	6,260		10,920	10,360	11,480		10,760	9,900	11,500		19,260	18,200	20,160		13,460	13,040	14,010
Modulus of elasticity 1,000 p. s. i.		1,310	1,174	1,598		1,349	1,218	1,452		1,466	1,388	1,554		2,091	1,980	2,239		2,255	1,334	2,582
5. Compression—perpendicular to grain (flatwise) [5]	12				11								12				7			
Proportional limit stress p. s. i.		980	850	1,150		2,220	1,910	2,580	3	2,360	2,000	2,890		8,800	6,900	12,010		8,140	6,800	10,820
Maximum crushing strength.........p. s. i.						23,300	19,440	26,300	6	22,100	18,480	25,510	6	33,550	31,680	36,400		40,060	33,930	44,040
6. Flexure—face grain parallel to span (flatwise) [5]	12				12				12				12				8			
Proportional limit stress p. s. i.		6,900	5,540	8,890		8,160	4,230	11,450		9,850	6,760	11,820		14,390	12,630	17,560		11,440	9,410	14,800
Modulus of rupture p. s. i.		13,130	10,840	15,490		11,410	7,780	14,020		12,670	11,100	15,710		22,780	19,110	25,290		25,120	21,480	27,600

Modulus of elasticity 1,000 p. s. i.		1,315	1,056	1,628		1,677	1,421	1,982		1,663	1,561	1,756		2,478	2,318	2,699		2,909	2,680	3,351
Work to proportional limit In.-lb. per cu. in.		2.02	1.42	2.77		2.31	0.62	3.96		3.31	1.58	4.56		4.66	3.62	6.36		2.55	1.78	4.03
Work to maximum load In.-lb. per cu. in.		14.5	9.91	17.9		4.89	2.41	6.66		6.19	4.54	10.9		15.0	10.1	18.7		24.1	16.4	30.6
7. Flexure—face grain perpendicular to span (flatwise)[5]	12				11				12				12				8			
Proportional limit stress p. s. i.		4,890	4,280	5,930		6,430	5,000	7,720		6,750	3,960	9,050		8,120	6,450	10,120		9,170	7,200	10,140
Modulus of rupture p. s. i.		11,330	9,500	12,700		7,910	5,860	9,880		8,760	5,950	10,370		16,060	13,700	17,960		23,680	22,460	21,940
Modulus of elasticity 1,000 p. s. i.		1,045	917	1,232		1,150	1,002	1,250		1,320	1,224	1,410		1,973	1,828	2,136		2,112	2,074	2,172
Work to proportional limit In.-lb. per cu. in.		1.29	0.94	1.83		2.01	1.38	2.68		2.00	.66	3.23		1.91	1.09	2.94		2.23	1.35	2.63
Work to maximum load In.-lb. per cu. in.		14.8	11.2	17.7		3.37	1.92	5.12		3.64	1.67	5.14		9.16	6.19	11.6		33.7	25.6	40.5
8. Modulus of rigidity (G):[6]																				
a. Plate shear (FPL test) 1,000 p. s. i.					3	215	212	221	3	221	213	227					3	375	362	391
b. Torsion method 1,000 p. s. i.	8	171	163	180					2	210	208	213	6	366	331	395				
9. Toughness (FPL test, edgewise[4]) In.-lb.	12	100	89.5	109.1	12	43.8	33.7	50.3	12	43	27.7	63.1	6	71.7	61.8	82.6	12	160.5	111.6	199.8
Toughness In.-lb. per in. of width		105.5	94.6	112.9		42	32.2	56.6		45.8	30.9	62.0		107.3	99.5	116.1		321.4	212.7	400.5

[1] Veneer conditioned at 80° F. and 65 percent relative humidity prior to assembly with film glue. No other resin employed. The average moisture content of the normal laminates at test was 9.5 percent (range 8.9 to 10.2).

[2] Total resin content 49 to 54 percent, impregnating resin content 42 to 49 percent on the basis of the dry weight of the untreated veneer.

[3] Total elongation immediately before fracture measured over a 2-inch gage length.

[4] Load applied to the edge of the laminations (perpendicular to laminating-pressure direction).

[5] Load applied to the surface of the original material (parallel to laminating-pressure direction).

[6] Modulus associated with shear distortions in planes parallel to the plane of the laminations.

Table 2-20. Some properties of cross-laminated modified wood made by the Forest Products Laboratory from 17 plies of 1/16-inch rotary-cut sweetgum veneer

Thickness at test, specific gravity, type of test and property	Normal laminated wood [1] Unimpregnated, uncompressed				Impreg [2] Impregnated, uncompressed				Semicompreg [2] Impregnated, moderately compressed				Compreg [2] Impregnated, highly compressed				Staypak [1] Unimpregnated, highly compressed			
	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
Thickness (t) of laminates....inch..		1.012	1.003	1.025		1.013	0.999	1.030		0.691	0.673	0.698		0.562	0.531	0.587		0.446	0.402	0.488
Specific gravity (based on weight and volume at test)		0.58	0.55	0.61		0.72	0.70	0.75		1.07	1.02	1.12		1.34	1.32	1.36		1.32	1.25	1.37
Test and property:																				
1. Tension—parallel to face grain (lengthwise)	12				6				6				6				6			
Proportional limit stress p. s. i..		6,620	5,420	8,860		4,780	3,560	5,800		6,320	5,120	7,200		9,080	8,340	10,030		11,520	7,800	18,110
Ultimate strength..p. s. i..		9,070	8,100	9,620		5,890	5,220	6,940		9,440	7,160	11,250		15,380	13,400	16,590		19,770	16,220	23,400
Modulus of elasticity 1,000 p. s. i..		914	853	994		1,083	1,039	1,131		1,524	1,338	1,705		1,987	1,880	2,065		2,136	1,896	2,232
Elongation [3]....percent..	6	1.04	0.97	1.11		0.56	0.50	0.64		0.64	0.54	0.77	3	0.85	0.72	1.02		0.96	0.84	1.11
2. Tension—perpendicular to face grain (crosswise)									9				9				9			
Proportional limit stress p. s. i..	9	4,430	3,770	4,780	9	3,490	2,880	4,010		6,420	5,640	7,160		7,700	6,140	8,900		9,950	6,240	12,650
Ultimate strength..p. s. i..		8,970	7,820	9,620		5,230	4,580	5,740		9,220	8,360	9,940		13,200	12,200	13,860		21,830	16,010	26,550
Modulus of elasticity 1,000 p. s. i..		862	766	936		1,055	976	1,152		1,384	1,291	1,485		1,911	1,782	2,020		1,991	1,616	2,248
Elongation [3]....percent..	2	1.05	1.03	1.07		0.51	0.43	0.58		0.70	0.65	0.77								
3. Compression—parallel to face grain (edgewise) [4]	12				12				12				12				12			
Proportional limit stress p. s. i..		2,510	2,070	2,910		4,380	3,560	4,960		4,960	4,380	5,420		7,940	6,870	8,970		4,990	4,150	5,810
Ultimate strength..p. s. i..		4,140	3,700	4,480		8,270	8,020	8,780		13,240	11,860	11,550		21,700	20,900	22,500		12,110	11,040	13,080
Modulus of elasticity 1,000 p. s. i..		970	845	1,073		1,116	1,038	1,177		1,664	1,543	1,801		2,232	2,094	2,352		2,161	1,525	2,383
4. Compression — perpendicular to face grain (edgewise) [4]	12				12				12				12				12			
Proportional limit stress p. s. i..		2,390	2,070	2,650		3,770	3,350	4,300										4,180	3,350	5,100
Ultimate strength..p. s. i..		3,910	3,520	4,320		7,570	7,380	7,840		13,060	11,980	14,060		19,950	18,750	20,700		11,520	10,280	12,600
Modulus of elasticity 1,000 p. s. i..		901	836	996		1,007	959	1,082		1,478	1,387	1,613		1,906	1,703	2,088		1,932	1,790	2,112
5. Compression — perpendicular to grain (flatwise) [5]	12				12															
Proportional limit stress p. s. i..		860	750	1,000		1,680	1,390	1,980	3	2,700	2,100	3,600								
Maximum crushing strength..........p. s. i..									6	25,730	23,800	28,100	9	31,750	29,080	36,380	7	33,560	30,600	35,900
6. Flexure—face grain parallel to span (flatwise) [5]	12				11				12				12				12			
Proportional limit stress p. s. i..		4,940	3,700	5,340		6,150	4,900	6,980		8,860	7,400	10,700		11,580	9,240	13,930		10,650	8,500	12,730
Modulus of rupture p. s. i..		9,140	8,440	9,990		8,300	6,780	9,260		14,050	11,180	15,960		18,960	14,420	22,250		24,010	21,600	26,380

Modulus of elasticity 1,000 p. s. i.		967	815	1,116		1,132	1,062	1,206		1,681	1,418	1,846		2,204	1,984	2,471		2,424	2,221	2,708
Work to proportional limit. In.-lb. per cu. in.		1.42	.85	1.81		1.89	1.13	2.50		2.62	1.90	3.54		3.47	1.92	5.11		2.67	1.45	3.71
Work to maximum load In.-lb. per cu. in.		10.13	8.32	12.43		3.66	2.25	4.72		8.41	5.71	11.0		11.8	6.0	16.1		30.67	25.22	37.30
7. Flexure—face grain perpendicular to span (flatwise) [5]	12				12				12				12				12			
Proportional limit stress. p. s. i.		3,900	3,620	4,700		4,680	3,770	5,660		7,150	4,870	8,320		9,510	7,210	11,300		7,620	5,780	9,600
Modulus of rupture p. s. i.		7,740	7,190	8,400		6,410	5,140	7,390		9,700	7,600	10,890		15,520	13,710	17,030		20,110	16,740	22,140
Modulus of elasticity 1,000 p. s. i.		735	658	804		904	846	990		1,206	1,074	1,334		1,750	1,634	1,851		1,748	1,550	1,896
Work to proportional limit. In.-lb. per cu. in.		1.20	0.94	1.60		1.36	0.93	1.81		2.38	1.23	2.97		2.93	1.72	3.93		1.89	0.98	2.71
Work to maximum load In.-lb. per cu. in.		9.35	8.22	10.74		2.76	1.66	3.95		5.12	3.46	6.62		9.8	7.2	11.4		31.18	19.87	36.42
8. Modulus of rigidity (G): [6]																				
a. Plate shear (FPL test) 1,000 p. s. i.					2	159	150	168	3	231	212	241	3	384	379	388	3	331	312	345
b. Torsion method 1,000 p. s. i.	6	120	101	138																
9. Toughness (FPL test, edgewise) [4] In.-lb.	12	68.4	58	76.4	12	30.4	22.4	37.5	12	50.6	31.9	104.4	12	42.9	31.7	61.4	12	133.3	73.8	100.6
Toughness In.-lb. per in. of width		67.8	57.3	75.7		30.1	21.9	37.1		72.9	45.8	150.6		75.5	54.3	106.2		299	152.8	426.5

[1] Veneer conditioned at 80° F. and 65 percent relative humidity prior to assembly with film glue. No other resin employed. The average moisture content of the normal laminates at test was 10.1 percent (range 9.2 to 10.8).

[2] Total resin content 37 to 52 percent, impregnating resin content 36 to 42 percent on the basis of the dry weight of the untreated veneer.

[3] Total elongation immediately before fracture measured over a 2-inch gage length.

[4] Load applied to the edge of the laminations (perpendicular to laminating-pressure direction).

[5] Load applied to the surface of the original material (parallel to laminating-pressure direction).

[6] Modulus associated with shear distortions in planes parallel to the plane of the laminations.

Table 2-21. *Some properties of cross-laminated modified wood made by the Forest Products Laboratory from 17 plies of 1/16-inch rotary-cut Sitka spruce veneer*

Thickness at test, specific gravity, type of test and property	Normal laminated wood [1] Uninpregnated, uncompressed				Impreg [2] Impregnated, uncompressed				Semicompreg [2] Impregnated, moderately compressed				Compreg [2] Impregnated, highly compressed				Staypak [1] Unimpregnated, highly compressed			
	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum	Number of tests	Average	Minimum	Maximum
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
Thickness (t) of laminates.....inch..		0.986	0.962	1.013		0.781	0.760	0.802		0.616	0.588	0.641		0.432	0.425	0.437		0.334	0.327	0.343
Specific gravity based on weight and volume at test)		0.46	0.44	0.48		0.73	0.72	0.75		0.94	0.91	0.98		1.31	1.32	1.36		1.36	1.34	1.39
Test and property:																				
1. Tension—parallel to face grain (lengthwise)	6				6								4				7			
Proportional limit stress p. s. i.		5,810	4,900	6,600										13,920	13,240	14,690		17,590	10,720	19,920
Ultimate strength..p. s. i..		7,740	7,220	8,200		5,500	4,900	5,900	6	7,440	6,620	8,890		15,600	15,060	16,650		26,640	24,660	28,920
Modulus of elasticity 1,000 p. s. i.		1,066	980	1,138		1,452	1,416	1,528	3	1,932	1,866	1,994		2,875	2,751	2,948		3,046	2,766	3,296
Elongation [3].....percent..	1	0.8				0.39	0.34	0.43		0.42	0.40	0.46		0.56	0.52	0.58		1.01	0.95	1.21
2. Tension—perpendicular to face grain (crosswise)	9				8				9				6				7			
Proportional limit stress p. s. i.		5,440	4,560	6,260		4,410	3,300	4,800						11,460	9,800	13,000		14,910	10,980	19,960
Ultimate strength..p. s. i.		7,400	6,400	8,340		4,990	4,270	5,720		6,590	5,860	7,500		13,010	11,800	14,010		23,880	21,540	25,000
Modulus of elasticity 1,000 p. s. i.		931	821	1,014		1,364	1,280	1,423		1,738	1,574	1,877		2,462	2,370	2,506		2,555	2,060	3,025
Elongation [3].....percent..	4	0.81	0.73	0.91		0.37	0.31	0.41		0.39	0.35	0.44		0.51	0.50	0.54		0.98	0.90	1.09
3. Compression—parallel to face grain (edgewise) [4]	12				12								8				12			
Proportional limit stress p. s. i.		3,040	2,670	3,500		4,640	4,220	5,270						10,040	8,720	1,440		6,510	4,760	7,860
Ultimate strength..p. s. i.		3,910	3,710	4,100		7,320	6,900	8,020	11	10,750	10,000	12,200		21,740	21,150	22,250		12,200	11,410	13,200
Modulus of elasticity 1,000 p. s. i.		1,062	968	1,148		1,434	1,360	1,488	12	1,876	1,792	1,990		2,738	2,622	2,875		2,881	2,518	3,220
4. Compression—perpendicular to face grain (edgewise) [4]	12				12								8				12			
Proportional limit stress p. s. i.		2,840	2,260	3,350		4,240	3,390	4,720	12					9,700	8,860	11,950		6,360	5,280	6,910
Ultimate strength..p. s. i.		3,530	3,290	3,790		7,070	6,410	7,780		10,860	9,860	12,660		21,740	20,940	23,020		11,860	10,820	13,090
Modulus of elasticity 1,000 p. s. i.		969	898	1,055		1,321	1,250	1,374		1,687	1,569	1,835		2,682	2,591	2,818		2,710	2,382	3,190
5. Compression—perpendicular to grain (flatwise) [5]	12				12												9			
Proportional limit stress p. s. i.		340	300	370		700	550	850	3	820	600	1,200	2	8,100	8,070	8,130		7,200	5,370	9,390
Maximum crushing strength.......p. s. i.						24,020	22,900	25,900	6	21,770	20,410	22,420	6	31,250	28,120	34,600		37,520	33,410	41,270
6. Flexure—face grain parallel to span (flatwise) [5]	12				12				11				8				8			
Proportional limit stress p. s. i.		3,950	3,680	4,240		5,850	4,670	7,050		7,740	6,720	9,120		15,440	13,400	18,840		13,150	11,290	14,320
Modulus of rupture p. s. i.		6,680	6,020	7,580		8,300	6,180	9,700		10,160	9,260	12,350		23,090	20,300	25,740		24,480	23,220	26,510

Modulus of elasticity 1,000 p. s. i.		655	599	706		1,426	1,361	1,475		1,840	1,669	2,117		2,960	2,879	3,060		2,995	2,951	3,062
Work to proportional limit In.-lb. per cu. in.		1.32	1.12	1.44	10	1.35	.87	1.92		1.82	1.36	2.26		4.55	3.26	6.66		3.22	2.39	3.83
Work to maximum load In.-lb. per cu. in.		7.40	5.04	10.90	12	2.94	1.67	4.12		3.68	2.72	5.15		11.76	8.08	15.24		25.2	22	35.1
7. Flexure—face grain perpendicular to span (flatwise) [5]	12				12				12				8				8			
Proportional limit stress p. s. i.		3,690	3,260	4,190		4,910	3,980	5,960		4,700	3,530	7,680		13,360	12,450	14,070		9,596	7,140	13,130
Modulus of rupture p. s. i.		5,750	5,390	6,290		6,250	5,220	7,400		7,670	6,810	9,920		15,240	14,610	16,360		22,075	20,840	24,200
Modulus of elasticity 1,000 p. s. i.		531	466	597		1,109	1,066	1,217		1,396	1,148	1,554		2,290	2,262	2,343		2,545	2,305	2,670
Work to proportional limit In.-lb. per cu. in.		1.43	1.19	1.74	9	1.23	0.82	1.72		0.95	0.45	2.34		4.34	3.79	4.84		2.08	1.15	3.59
Work to maximum load In.-lb. per cu. in.		6.07	4.84	7.14	11	2.16	1.41	3.13		2.58	1.81	4.00		6.34	5.47	7.48		26.4	22.6	30.4
8. Modulus of rigidity (G): [6]																				
a. Plate shear (FPL test) 1,000 p. s. i.					3	189	185	194	3	249	225	269	3	427	421	431	1	426		
b. Torsion method 1,000 p. s. i.	6	74.4	64.5	88.6																
9. Toughness (FPL test, edgewise [4]) In.-lb.	12	51.3	42.4	57.7	11	18.2	12.7	28.1	12	18.7	15.3	21.3	8	40.4	34.5	46.6	8	78.7	53.6	107.1
Toughness In.-lb. per in. of width		52.4	43.8	60.2		23.3	16.5	36.0		30.4	24.0	36.1		93.0	80.1	108.1		233.5	158.2	319.6

[1] Veneer conditioned at 80° F. and 65 percent relative humidity prior to assembly with film glue. No other resin employed. The average moisture content of the normal laminates at test was 8.7 percent (range 8.3 to 9.2).

[2] Total resin content 55 to 58 percent, impregnating resin content 36 to 42 percent on the basis of the dry weight of the untreated veneer.

[3] Total elongation immediately before fracture measured over a 2-inch gage length.

[4] Load applied to the edge of the laminations (perpendicular to laminating-pressure direction).

[5] Load applied to the surface of the original material (parallel to laminating-pressure direction).

[6] Modulus associated with shear distortions in planes parallel to the plane of the laminations.

Tension parallel to grain (test 1, tables 2–16 to 2–21, incl.). Specimens were 1 inch wide by panel thickness t by 24 inches long, shaped to have a 2½-inch long central section ¼-inch wide. The taper followed a 60-inch radius on each edge.

Tension perpendicular to grain and parallel to laminations (test 2, tables 2–16 to 2–21, incl.). Specimens were 1 inch by t by 16 inches long, shaped to have a 2½-inch long central section ½-inch wide for tables 2–16, –17 and –18, and ¼-inch wide for tables 2–19, –20, and –21, with radii of 30 and 20 inches, respectively.

Compression parallel to grain (test 3, tables 2–16 to 2–21, incl.) and perpendicular to grain and parallel to laminations (test 4, tables 2–16 to 2–21, incl.). Specimens were 1 inch by t by 3½ to 4 inches long for the controls; impreg and semicompreg specimen lengths were approximately $4t$. Compreg and staypak specimens were 1 inch by t by 2 to 4 inches long (approx. $6t$) for proportional limit and modulus data, and 1 by t by 1 to 2 inches long (approx. $3t$) for maximum stress.

Compression perpendicular to laminations (test 5, tables 2–16 to 2–21, incl.). Specimens were 1 by 1 inch by panel thickness t, except for compreg and staypak, which consisted of two thicknesses of material, each 1 inch square, placed one upon the other. Deformations were measured between the fixed and movable heads of the testing machine.

Flexure (tests 6 and 7, tables 2–16 to 2–21, incl.). Specimens 1 inch wide by height t were tested as simple beams with center loading on spans ranging from $14t$ to $16t$.

Shear parallel to grain and perpendicular to laminations (test 8, tables 2–16 to 2–18, incl.). Notched specimens were 2 inches by t by 2½ inches (as illustrated in fig. 17 of ASTM specifications for tests of small clear timber specimens, Designation D143–50) with shearing surface 2 inches by t. Specimens tested in the Johnson-type shear tool were 1 inch by t by 3 inches (two 1-inch by t shearing surfaces). Cylindrical double-shear specimens, ⅝-inch in diameter, were tested parallel to grain and parallel to laminations in a three-plate jig by means of tensile loading.

Modulus of rigidity tests (test 9, tables 2–16, –17, and –18; and test 8, tables 2–19, –20, and –21) were conducted on panels approximately 24 inches square by full thickness of the material, using the plate shear method developed by the Forest Products Laboratory for measuring the shearing moduli of wood, as described in Forest Products Laboratory report No. 1301 and ASTM Designation D805–47. Torsion tests were conducted on rectangular specimens of width $3t$ by thickness t by 16 to 24 inches long, gripped flatwise and with a detrusion measuring device applied to their edges. Following tests on these, with torque kept within the proportional limit, specimens were cut to a width of $2t$ and the test repeated.

Toughness (test 10, tables 2–16, –17, and –18; and test 9, tables 2–19, –20, and –21) specimens ⅝ by t by 10 inches long with grain of parallel-laminated material and face grain of cross-laminated material parallel to length were tested over an 8-inch span on the Forest Products Laboratory toughness machine with plane of laminations parallel to direction of load.

Impact (Izod type) specimens (test 11, tables 2–16, –17, and –18) had the grain lengthwise and the notch in an original surface. Some of the staypak specimens were less than ½ inch thick, but the dimension from the base of the notch to the opposite face was standard.

Water absorption (test 12, tables 2–16, –17, and –18) specimens were 1 by ⅜ by 3 inches. The grain was parallel to the 1-inch dimension. One surface of each specimen was an original face sanded, and all of its other surfaces were machined. Specimens were heated for 24 hours at 122° F., cooled, weighed, and immersed in water at room temperature for 24 hours, and the percentage increase in weight during immersion was calculated.

Dimensional stability of thickness t (test 13, tables 2–16, –17, and –18) was determined by the equilibrium swelling and recovery from compression of specimens ⅛ inch by t by 2 inches long (grain parallel to the ⅛-inch dimension). Specimens were immersed in water at room temperature until equilibrium moisture content was reached, and the percentage increase in thickness (swelling plus recovery) was calculated. The specimens were then oven-dried, measured, and percentage recovery and equilibrium swelling determined.

REFERENCES FOR CHAPTER 2

(2-1) ALLIOTT, E. A.
1926. *Effect of Acids on the Mechanical Strength of Timber: A Preliminary Study.* Soc. Chem. Indus. Jour. Trans. 45:463–466, illus.

(2-2) BAILEY, C. M. AND RICHARDS, N. E.
1947. *The Effect of Temperature on the Strength of Wooden Aircraft Structures.* Coun. Sci. Ind. Res. (Aust.) Div. Aero Report SM 95.

(2-3) BROKAW, M. P. AND FOSTER, G. W.
1945. *Effect of Rapid Loading and Duration of Stress on the Strength Properties of Wood Tested in Compression and Flexure.* Forest Products Laboratory Report No. 1518.

(2-4) BROWN, C. R.
1934. *The Determination of the Ignition Tempertures of Solid Materials.* The Catholic Univ. of America, Washington, D. C.

(2-5) BROWN, G. H., HOYLER, C. N., AND BIERWIRTH, R. A.
1947. *Theory and Application of Radio-Frequency Heating.* 425 pp. D. Van Nostrand Co., Inc.

(2-6) DOYLE, D. V. AND DROW, J. T.
1946. *The Elastic Properties of Wood—Young's Moduli, Moduli of Rigidity, and Poisson's Ratio of Mahogany and Khaya.* Forest Products Laboratory Report No. 1528–C.

(2-7) ———, DROW, J. T., AND MCBURNEY, R. S.
1945. *The Elastic Properties of Wood—The Young's Moduli, Moduli of Rigidity, and Poisson's Ratios of Balsa and Quipo.* Forest Products Laboratory Report No. 1528.

(2-8) ———, MCBURNEY, R. S., AND DROW, J. T.
1946. *The Elastic Properties of Wood—The Moduli of Rigidity of Douglas-Fir at About 11 Percent Moisture Content.* Forest Products Laboratory Report No. 1528–E.

(2-9) ———, MCBURNEY, R. S., AND DROW, J. T.
1946. *The Elastic Properties of Wood—The Moduli of Rigidity of Sitka Spruce and Their Relations to Moisture Content.* Forest Products Laboratory Report No. 1528–B.

(2-10) DROW, J. T.
1945. *Effect of Moisture on the Compression, Bending, and Shear Strength and on the Toughness of Plywood.* Forest Products Laboratory Report No. 1519.

(2-11) ———
1945. *Effect of Hydraulic Equipment Oils on the Bending and Compressive Strength of Sitka Spruce.* Forest Products Laboratory Report No. 1520.

(2-12) ———, AND CLARK, M. E.
1945. *Moisture Content of Wood in Airplane Structures.* Forest Products Laboratory Report No. 1552.

(2-13) ———, CLARK, M. E., AND WILSON, T. R. C.
1946. *Distribution of Strength Values in Wood for Aircraft Construction.* Forest Products Laboratory Report No. 1515.

(2-14) ———, AND LISKA, J. A.
1943. *Effect of Moisture on the Compressive Strength of Plywood and Laminated Wood.* Forest Products Laboratory Report No. 1306.

(2-15) ———, AND MCBURNEY, R. S.
1946. *The Elastic Properties of Wood—Young's Moduli and Poisson's Ratios of Sitka Spruce and Their Relations to Moisture Content.* Forest Products Laboratory Report No. 1528–A.

(2-16) ———, AND MCBURNEY, R. S.
1946. *The Elastic Properties of Wood—Young's Moduli, Moduli of Rigidity, and Poisson's Ratios of Yellow Birch.* Forest Products Laboratory Report No. 1528–H.

(2-17) ———, AND MCBURNEY, R. S.
1946. *The Elastic Properties of Wood—Young's Moduli, Moduli of Rigidity, and Poisson's Ratios of Yellow-Poplar.* Forest Products Laboratory Report No. 1528–G.

(2-18) ERICKSON, E. C. O.
1947. *Mechanical Properties of Laminated Modified Wood.* Forest Products Laboratory Report No. R1639.

(2-19) ERICKSON, H. D. AND REES, L. W.
1940. *Effect of Several Chemicals on the Swelling and the Crushing Strength of Wood.* Jour. Agr. Research 60:593–603.

(2-20) FOREST PRODUCTS LABORATORY
1940. *Wood Handbook.* U. S. Department of Agriculture Unnumbered Publication.

(2-21) ———
1941. *Computed Thermal Conductivity of Common Woods.* Technical Note No. 248.

(2–22) FOREST PRODUCTS LABORATORY.
1941. *Moisture Content—Strength Adjustments for Wood.* Forest Products Laboratory Report No. 1313.

(2–23) ———
1943. *Design of Plywood Webs in Box Beams.* Forest Products Laboratory Report No. 1318 and Supplements.

(2–24) ———
1947. *Wood Aircraft Inspection and Fabrication.* ANC–19. (In Press).

(2–25) FREAS, A. D.
1946. *Methods of Computing the Strength and Stiffness of Plywood Strips in Bending.* Forest Products Laboratory Report No. 1304.

(2–26) GEORGE, H. O.
1933. *Effect of Low Temperature on the Strength of Wood.* New York State College of Forestry, Syracuse Univ. Tech. Pub. No. 43.

(2–27) GOODELL, H. R. AND PHILLIPS, R. S.
1944. *Bolt-bearing Strength of Wood and Modified Wood: Effects of Different Methods of Drilling Bolt Holes in Wood and Plywood.* Forest Products Laboratory Report No. 1523.

(2–28) GREENHILL, W. L.
1936. *Strength Tests Perpendicular to the Grain of Timber at Various Temperatures and Moisture Contents.* F. Coun. Sci. Ind. Res. (Aust.) Vol. 9(4):265–276.

(2–29) ———
1942. *The Damping Capacity of Timber.* (Aust.) Jour. Coun. Sci. & Ind. Res. Vol. 15, No. 2, May 1942.

(2–30) HEEBINK, T. B., MARCH, H. W., AND NORRIS, C. B.
1946. *Buckling of Stiffened, Flat Plywood Plates in Compression.* Forest Products Laboratory Report No. 1553.

(2–31) HUNT, P. J., GOODELL, H. R., AND PHILLIPS, R. S.
1946. *Bolt-bearing Strength of Wood and Modified Wood: Bearing Strength of Commercial Cross-Banded Compreg Under Aircraft Bolts.* Forest Products Laboratory Report No. 1523–B.

(2–32) JENKIN, C. F.
1920. *Report on Materials Used in Aircraft and Aircraft Engines.* (Gr. Brit.). Munitions-Aircraft Production Department. Aeronautical Research Committee.

(2–33) KIMBALL, A. L.
1941. *Vibration Problems.* Jour. of Applied Mechanics, Vol. 8, No. 1, March 1941, and Vol. 8, No. 3, Sept. 1941.

(2–34) KITAZAWA, G.
1947. *Relaxation of Wood Under Constant Strain.* New York State College of Forestry, Syracuse Univ. Tech. Pub. No. 67.

(2–35) KLOOT, N. H.
1940–1947. *Miscellaneous Investigations on Aircraft Timbers. The Variation of Toughness and Izod Values of Hoop and Bunya Pine, and Same for Other Species.* Aust., Coun. for Sci. & Ind. Res., Div. of Forest Products. Proj. TM 18–0.

(2–36) KOLLMAN, F.
1936. *Technologie Des Holzes.* Julius Springer, Berlin.

(2–37) ———
1940. *Die Mechanischen Eigenschaften Verschieden Feuchter Holzer in Temperaturberuch Von–200 BIS +200° C.* VDI Forschungscheft 403 B (11) 1–18, July-August 1940. Available in English as NACA Technical Memorandum No. 984, September 1941.

(2–38) KOMMERS, W. J.
1943. *The Fatigue Behavior of Wood and Plywood Subjected to Repeated and Reversed Bending Stresses.* Forest Products Laboratory Report No. 1327.

(2–39) ———
1944. *Supplement to the Fatigue Behavior of Wood and Plywood Subjected to Repeated and Reversed Bending Stresses.* Forest Products Laboratory Report No. 1327–A.

(2–40) KUENZI, E. W.
1947. *Stability of a Few Curved Panels Subjected to Shear.* Forest Products Laboratory Report No. 1571.

(2–41) LEWIS, W. C.
1946. *Fatigue of Wood and Glued Wood Constructions.* Proc. Amer. Soc. Test. Matl. Vol. 46: 814–35.

(2–42) ———, HEEBINK, T. B., AND COTTINGHAM, W. S.
1945. *Effect of Increased Moisture Content on the Shear Strength at Glue Lines of Box Beams and on the Glue-Shear and Glue-Tension Strength of Small Specimens.* Forest Products Laboratory Report No. 1551.

(2–43) LISKA, J. A.
1946. *Methods of Calculating the Strength and Modulus of Elasticity of Plywood in Compression.* Forest Products Laboratory Report No. 1315.

(2–44) MCBURNEY, R. S., DOYLE, D. V., AND DROW, J. T.
1946. *The Elastic Properties of Wood.* Forest Products Laboratory Report No. 1528–D, F.

(2–45) MCLEAN, J. D.
1941. *Thermal Conductivity of Wood.* Trans. Amer. Soc. of Heating and Vent. Engr. Vol. 47: 323–354.

(2–46) ———
1945. *Effect of Heat on the Properties and Serviceability of Wood—Experiments on Thin Wood Specimens.* Forest Products Laboratory Report No. R1471.

(2–47) McLeod, A. M.
1946. *Bolt-Bearing Strength of Wood and Modified Wood: Bearing Strength of Commercial Aircraft Plywood Under Aircraft Bolts.* Forest Products Laboratory Report No. 1523–C.

(2–48) ———, Yolton, L. A., Sanborn, W. A., and Phillips, R. S.
1945. *A Comparison of Shearing Strengths of Glued Joints of Various Grain Directions as Determined by Four Methods of Test.* Forest Products Laboratory Report No. 1522.

(2–49) McNaughton, G. C.
1944. *Ignition and Charring Temperatures of Wood.* Forest Products Laboratory Report No. R1464.

(2–50) March, H. W.
1942. *Buckling of Flat Plywood Plates in Compression, Shear, or Combined Compression and Shear.* Forest Products Laboratory Report No. 1316 and Supplements.

(2–51) ———
1942. *Flat Plates of Plywood under Uniform or Concentrated Loads.* Forest Products Laboratory Report No. 1312.

(2–52) ———
1944. *Rectangular Plywood Plates with the Grain of the Face Plies Inclined to the Edges.* Forest Products Laboratory Report No. 1507.

(2–53) ———
1944. *Stress-Strain Relations in Wood and Plywood Considered as Orthotropic Materials.* Forest Products Laboratory Report No. 1503.

(2–54) ———, Kuenzi, E. W., and Kommers, W. J.
1942. *Methods of Measuring the Shearing Moduli in Wood.* Forest Products Laboratory Report No. 1301.

(2–55) ——— and Smith, C. B.
1945. *Buckling of Flat Sandwich Panels in Compression.* Forest Products Laboratory Report No. 1525.

(2–56) Markwardt, L. J.
1938. *Form Factors and Methods of Calculating the Strength of Wooden Beams.* Forest Products Laboratory Report No. R1184.

(2–57) ——— and Wilson, T. R. C.
1935. *Strength and Related Properties of Woods Grown in the United States.* U. S. Dept. Agr. Tech. Bull. 479.

(2–58) Meyai, K. and Oshawa, M.
——— *Experiments in the Bending Strength of Frozen Wood.* Research Bull., Hokkaido Imp. Univ., Sapporo, Japan. (Text in Japanese.)

(2–59) National Advisory Committee for Aeronautics
1923. *Deflection of Beams with Special Reference to Shear Deformations.* Report No. 180.

(2–60) Nayer, A. N.
1948. *Swelling of Wood in Various Organic Liquids.* PhD. Thesis, Univ. of Minn., March 1948.

(2–61) Newlin, J. A.
1939. *Bearing Strength of Wood at an Angle to the Grain.* Engineering News-Record, May 11, 1939.

(2–62) ——— and Trayer, G. W.
1923. *Form Factors of Beams Subjected to Transverse Loading Only.* N. A. C. A. Tech. Report No. 181 (Also, Forest Products Laboratory Report No. 1310).

(2–63) ——— and Trayer, G. W.
1924. *Stresses in Wood Members Subjected to Combined Column and Beam Action.* N. A. C. A. Tech. Report No. 188 (Also, Forest Products Laboratory Report No. 1311).

(2–64) ——— and Trayer, G. W.
1930. *Design of Airplane Wing Ribs.* N. A. C. A. Tech. Report 345 (Also, Forest Products Laboratory Report No. 1307).

(2–65) Norris, C. B.
1943. *The Application of Mohr's Stress and Strain Circles to Wood and Plywood.* Forest Products Laboratory Report No. 1317.

(2–66) ——— and Kommers, W. J.
1943. *Plastic Flow (Creep) Properties of Two Yellow Birch Plywood Plates Under Constant Shear Stress.* Forest Products Laboratory Report No. 1324.

(2–67) ——— and McKinnon, P. F.
1946. *Compression, Tension, and Shear Tests on Yellow-poplar Plywood Panels of Sizes That Do Not Buckle with Tests Made at Various Angles to the Face Grain.* Forest Products Laboratory Report No. 1328 and Supplements 1328–A, 1328–B, and 1328–C.

(2–68) Sanborn, W. A., Goodell, H. R., Ely, A. W., and Phillips, R. S.
1946. *Bolt-Bearing Strength of Wood and Modified Wood: Bearing Strength of Wood Members Reinforced With Plywood and Cross-Banded Compreg Under Single and Multiple Aircraft Bolts.* Forest Products Laboratory Report No. 1523–D.

(2–69) Smith, C. B.
1944. *Effect of Elliptic or Circular Holes on the Stress Distribution in Plates of Wood or Plywood Considered as Orthotropic Materials.* Forest Products Laboratory Report No. 1510.

(2–70) Norris, C. B. and Ringelstetter, L. A.
1948. *Buckling of Stiffened, Flat, Plywood Plates in Compression.* Forest Products Laboratory Report No. 1553–C.

(2–71) Smith, C. B., Heebink, T. B., and Norris, C. B.
1946. *The Effective Stiffness of a Stiffener Attached to a Flat Plywood Plate.* Forest Products Laboratory Report No. 1557.

(2-72) STAMM, A. J.
1930. *An Electrical Conductivity Method for Determining the Moisture Content of Wood.* Forest Products Laboratory Report No. R1023.

(2-73) SULZBERGER, P. H.
1943. *The Effect of Temperature on the Strength Properties of Wood, Plywood, and Glued Joints.* Jour. Coun. Sci. & Ind. Res. (Aust.). Vol. 16, (4): 263–265.

(2-74) TAYLOR, J. P.
1943. *Heating Wood with Radio Frequency Power.* Trans. Am. Soc. Mech. Engr. Vol. 56, No. 3, April 1943.

(2-75) TIEMANN, H. D.
1944. *Wood Technology.* Pitman Publishing Co.

(2-76) THUNELL, B.
1940. *The Effect of Temperature on the Bending Strength of Swedish Pinewood.* Abst. Empire Forestry Jour. Vol. 19 (1940) P. 309.

(2-77) TRAYER, G. W.
1928. *Bearing Strength of Wood Under Steel Aircraft Bolts and Washers and Other Factors Influencing Fitting Design.* N. A. C. A. Tech. Note 296.

(2-78) ———
1930. *Wood in Aircraft Construction.* National Lumber Manufacturers Association.

(2-79) ——— AND MARCH, H. W.
1931. *Elastic Instability of Members Having Sections Common in Aircraft Construction.* N. A. C. A. Tech. Report No. 382.

(2-80) UNDERWRITERS' LABORATORIES
1940. *Wood, Ignition of at Low Temperatures.* Card Data Service Serial No. UL198.

(2-81) VORREITER, L.
1938. *Biege- und Druckfestigkeit Gefroren Fichtenholtzels.* Tharandlerforstl. Jrb. 89:-491–510.

(2-82) WEATHERWAX, R. C., AND STAMM, A. J.
1946. *Coefficient of Thermal Expansion of Wood and Wood Products.* Amer. Soc. Mech. Engin. Trans. 69(44):421–432.

(2-83) ——— AND STAMM, A. J.
1943. *The Electrical Resistivity of Resin-Treated Wood (Impreg and Compreg), Hydrolyzed Wood Sheet (Hydroxylin) and Laminated Resin-Treated Paper.* Forest Products Laboratory Report No. 1385.

(2-84) WILSON, T. R. C.
1930. *Effect of Creosote on Strength of Fir Timbers.* Timberman 31(8):50–56.

(2-85) ———
1932. *Strength-Moisture Relations for Wood.* U. S. Dept. Agr. Tech. Bull. 282.

(2-86) WOOD, L. W.
1947. *Behavior of Wood Under Continued Loading.* Engineering News-Record, Dec. 11, 1947, pp. 108–111.

(2-87) FULLER AND OBERG
1943. *Fatigue Characteristics of Natural and Resin Impregnated, Compressed, Laminated Woods.* J. Aeronautical Science, Vol. 10, March 1943, p. 81.

(2-88) NORRIS, C. B., WERREN, F., AND MCKINNON, P. F.
1948. *The Effect of Veneer Thickness and Grain Direction on the Shear Strength of Plywood.* Forest Products Laboratory Report No. 1801.

(2-89) NORRIS, C. B., KOMMERS, W. J., AND MCKINNON, P. F.
1948. *Critical Buckling Strength of Stiffened Flat Plywood Plates in Compression and Shear—Closely Spaced Stiffeners.* Forest Products Laboratory Report No. 1800.

(2-90) MARCH, H. W., NORRIS, C. B., AND KUENZI, E. W.
1943. *Buckling of Long, Thin Plywood Cylinders in Axial Compression.* Forest Products Laboratory Report No. 1322 and supplements.

(2-91) KUENZI, E. W.
1948. *Effect of Length on the Buckling Stresses of Thin-walled Plywood Cylinders in Axial Compression.* Forest Products Laboratory Report No. 1514.

(2-92) ———
1944. *Thin-walled Plywood Cylinders in Bending.* Forest Products Laboratory Report No. 1502.

(2-93) MARCH, H. W., NORRIS, C. B., SMITH, C. B., AND KUENZI, E. W.
1945. *Buckling of Thin-walled Plywood Cylinders in Torsion.* Forest Products Laboratory Report No. 1529.

(2-94) KUENZI, E. W.
1944. *Thin-walled Plywood Cylinders in Bending and Torsion.* Forest Products Laboratory Report No. 1501.

(2-95) KUENZI, E. W., AND NORRIS, C. B.
1948. *Longitudinally Stiffened Thin-walled Cylinders in Axial Compression.* Forest Products Laboratory Report No. 1562.

(2-96) ———
1948. *Tortional Buckling of Longitudinally Stiffened, Thin-walled, Plywood Cylinders.* Forest Products Laboratory Report No. 1563.

(2-97) NEWLIN, J. A., AND GAHAGAN, J. M.
1930. *Tests of Large Timber Columns and Presentation of the Forest Products Laboratory Column Formula.* U. S. Dept. Agr. Tech. Bull. 167.

BIBLIOGRAPHY FOR CHAPTER 2

Elmendorf, A.
1920. *Data on the Design of Plywood for Aircraft.* N. A. C. A. Tech. Report 84. (Also Forest Products Laboratory Report No. 1302).

———
1941. *Specific Gravity-strength Relations for Wood.* Forest Products Laboratory Report No. 1303.

———
1947. *Wood Aircraft Inspection and Fabrication.* ANC-19 (On Press).

Lewis, W. C. and Dawley, E. R.
1943. *Stiffeners in Box Beams and Details of Design.* Supplement to: Design of Plywood Webs in Box Beams. Forest Products Laboratory Report No. 1318–A.

———; Heebink, T. B.; Cottingham, W. S.; and Dawley, E. R.
1943. *Buckling in Shear Webs of Box and I-Beams and the Effect Upon Design Criteria.* Supplement to: Design of Plywood Webs in Box Beams. Forest Products Laboratory Report No. 1318–B.

Lundquist, E. E.; Kotanchik, J. N.; and Zender, G. W.
1942. *A Study of the Compressive Strength of Stiffened Plywood Panels.* N. A. C. A. Advanced Tech. Note (Restricted).

March, H. W.
1941. *Summary of Formulas for Flat Plates of Plywood Under Uniform or Concentrated Loads.* Forest Products Laboratory Report No. 1300 (Revised).

———
1943. *Buckling of Long, Thin Plywood Cylinders in Axial Compression.* Forest Products Laboratory Report No. 1322 and supplements 1322–A and 1322–B.

Markwardt, L. J.
1930. *Aircraft Woods: Their Properties, Selection, and Characteristics.* N. A. C. A. Tech. Report 354. (Also Forest Products Laboratory Report No. R1079).

Newlin, J. A.
1940. *Formulas for Columns with Side Loads and Eccentricity.* Building Standards Monthly, December 1940.

Norris, C. B.
1937. *The Technique of Plywood.* Hardwood Record, October 1937 to March 1938.

Trayer, G. W.
1930. *The Design of Plywood Webs for Airplane Wing Beams.* N. A. C. A. Tech. Bull. 344.

———
1932. *The Bearing Strength of Wood Under Bolts.* U. S. Dept. Agr. Tech. Bull. 332.

CHAPTER 3

METHODS OF STRUCTURAL ANALYSIS

3.0. General

3.00. PURPOSE. It is the purpose of the Methods of Structural Analysis portion of this bulletin to present acceptable procedures for use in determining the internal stresses resulting from the application of known external loads to wood and plywood aircraft structures. The basic design procedures that have been developed for use in analyzing metal structures are generally applicable to the problem of wood structures provided that suitable modifications are made to account for the differences in physical characteristics. The designer's attention is directed to existing text material covering the treatment of common stress-analysis problems not treated herein, and to the current preparation of an Army-Navy-Civil Bulletin, ANC-4 "Methods of Structural Analysis."

It is to be emphasized that the analysis procedures described in this bulletin are not presented as required procedures but represent suggested methods that are acceptable to the Army, Navy, and Civil Aeronautics Administration. The nature, magnitude, and distribution of the loads for which the airplane structure shall be designed are defined by the applicable specification, regulation, handbook, or bulletin of the procuring or certificating agency.

Submission of a stress analysis, although such an analysis employs a method of procedure which is considered acceptable by the procuring or certificating agency, does not necessarily constitute satisfactory proof of adequate strength. The stress analysis should be supplemented by pertinent test data. Unless a structure conforms closely to a previously constructed type, the strength of which has been determined by test, a stress analysis is not considered as a sufficiently accurate and certain means of determining its strength. Most desirable is a test of the complete structure under the critical design-loads. However, tests of certain component parts and of specimens employing generally typical construction and detail design features are of great assistance both in justifying allowable stresses and in proving analysis methods. In each individual case, the extent and nature of the structural test program required to substantiate the stress analysis is specified by the procuring or certificating agency.

3.01. SPECIAL CONSIDERATIONS IN STATIC TESTING OF STRUCTURES. Since the allowable stress values given in chapter 2, tables 2–6 and 2–7, are based on a definite moisture content and method of load application, consideration should be given to these variables, both in using element tests to establish design allowable stresses and in designing structures to be statically tested as complete structures. Elements include simple structural members and details, such as panels, stiffened panels, or sections of spars. Complete structures include wing panels, center sections, fuselage, stabilizer, or other parts individually or in combination. These two types of test will be discussed separately since they are treated differently.

3.010. *Element Tests.* A comparison of the design values listed in tables 2–6 and 2–7 with the results of standard tests at 12 percent moisture content (ref. 2–57) shows that test results may be made approximately comparable to the design values by the following methods. Enough tests should be made to cover variability but the required number will be governed by various factors as discussed in the following.

Case A. When the type of element and the mode of failure are such that the results of element tests can be directly related to the physical properties of coupons cut from the materials used in the elements, the results of element tests may be corrected by the ratio of the design values in tables 2–6 and 2–7 to the test coupon values. Care should be taken that the elements and the coupons are tested at a slow rate, at the same moisture content, and under approximately the same time-loading conditions. The test element should be made of matched materials; for example all stiffeners in a stiffened panel should be made from the same stock.

Case B. When it is not practicable to correct element tests by means of related tests on coupons, the following procedure may be employed:

(1) A sufficient number of tests should be made to establish a reasonably reliable average considering the variability of the materials. Fewer tests will be required and the scatter of related tests will be reduced if the test results are corrected to the average specific gravity values listed in tables 2–6 and 2–7 by the methods of section 2.01. For the same reason, it is desirable to use material of approximately average specific gravity in test specimens.

(2) The strength should be adjusted to 12 percent moisture by factors from table 2–2 appropriate to the primary mode of failure. Should failure occur in glued or bolted fastenings, however, no upward adjustments should be made. It should be recognized that moisture adjustments are subject to error and should, therefore, be avoided whenever possible by conditioning test specimens to approximately 12 percent moisture content.

(3) In element tests it will usually be possible to arrange the test procedure so that errors due to rate and duration of load will be negligible in comparison with other experimental errors, for example:

(*a*) If the maximum load is supported for 15 seconds or more, such as in tests where the load is added by weight increments, corrections for rate and duration of load are unnecessary.

(*b*) If the specimen is loaded at a rate of strain such that the time from zero load to failure is more than 2 minutes when the testing machine is operated continuously, corrections are unnecessary. Thus, if the first stopping point is 25 percent of the expected ultimate load and the machine takes ½ minute to reach this load, the rate of strain is sufficiently low.

The time to failure after passing the limit load should be not more than 5 minutes if possible (slower loading results in lower ultimate loads) since upward corrections of test values, because of long duration, are considered inadvisable.

(4) After correction of the average test results for moisture, a correction factor to allow for variability should be applied as follows:

(*a*) 0.94 when the failure is principally the result of compression, tension, or bending stresses, or shear in 45° plywood.

(*b*) 0.80 when the failure is principally due to shear stresses parallel to the grain.

3.011. *Complete structures.*

3.0110. *Design allowances for test conditions.* When a complete structure is static tested, it is not usually possible to make the test under the conditions on which the design values of tables 2–6 and 2–7 are based. Therefore, if the purpose of the test is to prove the strength of the entire structure at a specified ultimate load regardless of test conditions (which is usually the case in order to prove joints and fittings) it is recommended that the designer investigate the effects of probable test conditions prior to designing the structure on the basis of tables 2–6 and 2–7.

If it appears that the probable test conditions will cause the strength in the test to be less than that corresponding to design values in tables 2–6 and 2–7 suitable margins of safety should be incorporated during the design.

3.0111. *Test procedure.* In complex composite structures the effects of moisture content on overall strength are uncertain. Changes in wood strength may be offset by stress concentration effects. It is, therefore, desirable that complete structures be conditioned as closely as possible to 12 percent moisture content at the time of testing.

To minimize effects of rate and duration of load, the time to failure after passing limit load should be less than 15 minutes if possible.

The ultimate load should be sustained without failure for at least 15 seconds, in order to insure the test being comparable to design values in regard to time effects.

The above procedure may be varied depending upon the purpose of the test. Agreement should be reached with the procuring or certificating agency regarding the test procedures and methods of correction, if any, prior to conducting major tests.

3.1. Wings

3.10. GENERAL. Because of the basic differences in their structural behavior, spearate stress analysis procedures are outlined for the following general types of wing structures:

(a) Two-spar wings with independent spars.

(b) Reinforced shell wings.

3.11. TWO-SPAR WINGS WITH INDEPENDENT SPARS. The methods of analysis presented under this heading are based on the assumption that the spars deflect independently in bending. Such methods are particularly applicable to two-spar fabric-covered wings with drag bracing in a single plane. They may also be applied to two-spar wings having drag bracing in two planes. In such cases, the effect of the torsional rigidity resulting from the double drag bracing, tending to equalize the deflections of the two spars, is usually neglected but may be taken into account by the methods of reference 3–7.

3.110. *Spar loadings.* The following method of determining the running loads on the spars has been developed to simplify the calculations required and to provide for certain features which cannot be accounted for in a less general method. It is equivalent to assuming that the resultant air and inertia loads at each section are divided between the spars as though the ribs were simple beams and the spars furnished the reactions. Frequently, certain items are constant over the span; then the computations are considerably simplified.

The net running load on each spar, in pounds per inch run, can be obtained from the following equations:

$$y_f=[\{C_N(r-a)+C_{M_a}\}q+n_2e\,(r-j)]\,\frac{C'}{144b} \tag{3:1}$$

$$y_r=[\{C_N(a-f)-C_{M_a}\}q+n_2e\,(j-f)]\,\frac{C'}{144b} \tag{3:2}$$

where:

y_f=net running load on front spar, in pounds per inch

y_r=net running load on rear spar, in pounds per inch

a, b, f, j, and r are shown in figure 3–1 and are *all expressed as fractions of the chord* at the station in question. The value of a must agree with the value on which C_{Ma} is based.

q=dynamic pressure for the condition being investigated.

C_N and C_{Ma} are the airfoil normal force and moment coefficients, respectively, at the section in question.

C' is the wing chord, in *inches.*

e is the average unit weight of the wing, in pounds per square foot, over the chord at the station in question. It should be computed or estimated for each area included between the wing stations investigated, unless the unit wing weight is substantially constant, in which case a constant value may be assumed. By properly correlating the values of e and j, the effects of local weights, such as fuel tanks and nacelles, can be directly accounted for.

n_2 is the net limit-load factor representing the inertia effect of the whole airplane acting at the center of gravity. The inertia load always acts in a direction opposite to the net air load. *For positively accelerated conditions n_2 will always be*

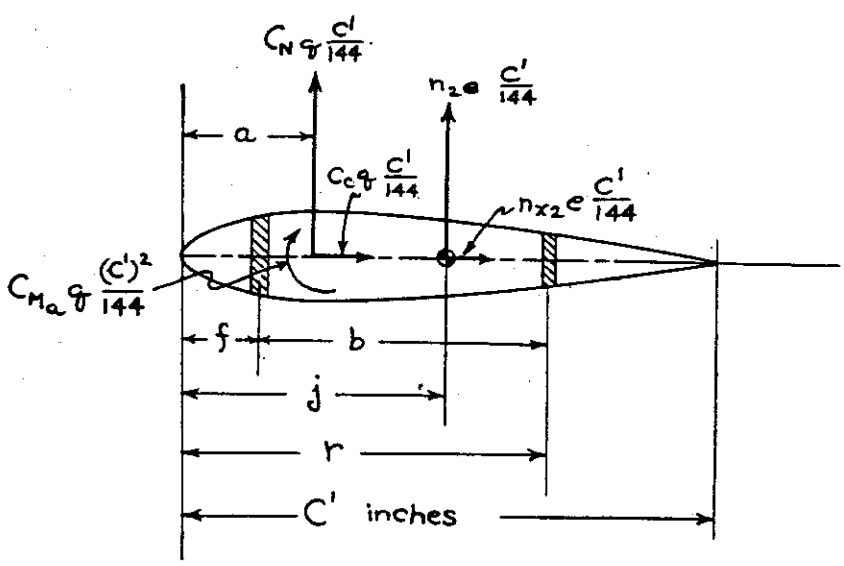


Figure 3–1. Unit section of a conventional 2-spar wing. All vectors are shown in positive sense.

Table 3-1. Computation of net unit loadings (constants)

		Stations Along Span				
1	Distance from root, inches					
2	C'/144 = (chord in inches) /144					
3	f, fraction of chord					
4	r, " " "					
5	b = r - f = ④-③					
6	a, fraction of chord (a.c.)					
7	j, " " " *					
8	e = unit wing wt., lbs/sq.ft.*					
9	r - a = ④-⑥					
10	a - f = ⑥-③					
11	r - j = ④-⑦					
12	j - f = ⑦-③					
13	C'/144 b = ②/⑤					

* These values will depend on the amount of disposable load carried in the wing.

negative, and vice versa. Its value and sign are obtained in the balancing of the airplane.

If it is desired to compute the airloading and inertia loadings separately, formulas (3:1) and (3:2) may be modified by omitting terms containing n_2 for the airloading, and omitting terms containing q for the inertia loading. Then the inertia loading, shear, and moment curves need be computed for only one condition (say, $n_2=1.0$), the values for any other condition being obtained by multiplying by the proper load factor.

The computations required in using the preceding method are outlined in tables 3-1 and 3-2, in a form which is convenient for making calculations and for checking.

The following modifications and notes apply to tables 3-1 and 3-2:

(*a*) When the curvature of the wing tip prevents the spars from extending to the extreme tip of the wing, the effect of the tip loads on the spar can easily be accounted for by extending the spars to the extreme span as hypothetical members. In such cases, the dimesion f will become negative, as the leading edge will lie behind the hypothetical front spar.

(*b*) The local values of C_N, item 14, are determined from the design values of C_N in accordance with the proper span-distribution curve.

(*c*) Item 15 provides for a variation in the local value of C_M. When a design value of center-of-pressure coefficient is specified, the value of C_M should be determined by the following equation, using item numbers from tables 3-1 and 3-2.

$$C'_{M_a}=⑭[⑥-CP'] \qquad (3:3)$$

(*d*) When conditions with deflected flaps are investigated, the value of C_{M_a} over the flap portion should be properly modified. For most conditions, C_{M_a} will have a constant value over the span.

(*e*) The gross running loads on the wing structure can be obtained by assuming e to be zero; then, items ⑲, ㉕, and ㉚ become zero, y_f becomes ⑱×⑬, y_r becomes ㉔×⑬, and y_c becomes ㉙×②.

3.111. *Chord loading.* The net chord loading, in pounds per inch run, can be determined from the following equation:

$$y_c=\frac{[C_cq+n_{x2}e]\,C'}{144} \qquad (3:4)$$

Table 3–2. Computation of net unit loadings (variables)

CONDITION ----

q	C_{NI} (etc)	C'_C	C'_M or C.P.	n_z	n_{x_z}

		(Refer also to Table 3-4)	Distance b from root					
	14	C_{N_b} = (variation with span)						
	15	C_{M_a} (variation with span)						
Front Spar	16	(14) x (9)						
	17	(16) + (15)						
	18	(17) x q						
	19	n_z x (8) x (11)						
	20	(18) + (19)						
	21	y_f = (20) x (13), lbs/inch						
Rear Spar	22	(14) x (10)						
	23	(22) - (15)						
	24	(23) x q						
	25	n_z x (8) x (12)						
	26	(24) + (25)						
	27	y_r = (26) x (13), lbs/inch						
Chord Load	28	C'_C (variation with span)						
	29	(28) x q						
	30	n_{x_z} x (8)						
	31	(29) + (30)						
	32	y_c = (31) x (2), lbs/inch						

where:

y_c=running chord load, in pounds per inch.

C_c=airfoil chord force coefficient at each station. The proper sign should be retained throughout the computations.

n_{x2}=net limit chord-load factor approximately representing the inertia effect of the whole airplane in the chord direction. The value and sign are obtained in the balancing of the airplane. *When C_c is negative, n_{x2} will be positive.*

q, e, and C' are the same as in section 3.110.

The computations for obtaining the chord load are outlined in table 3–2, items 28 to 32. The following points should be noted:

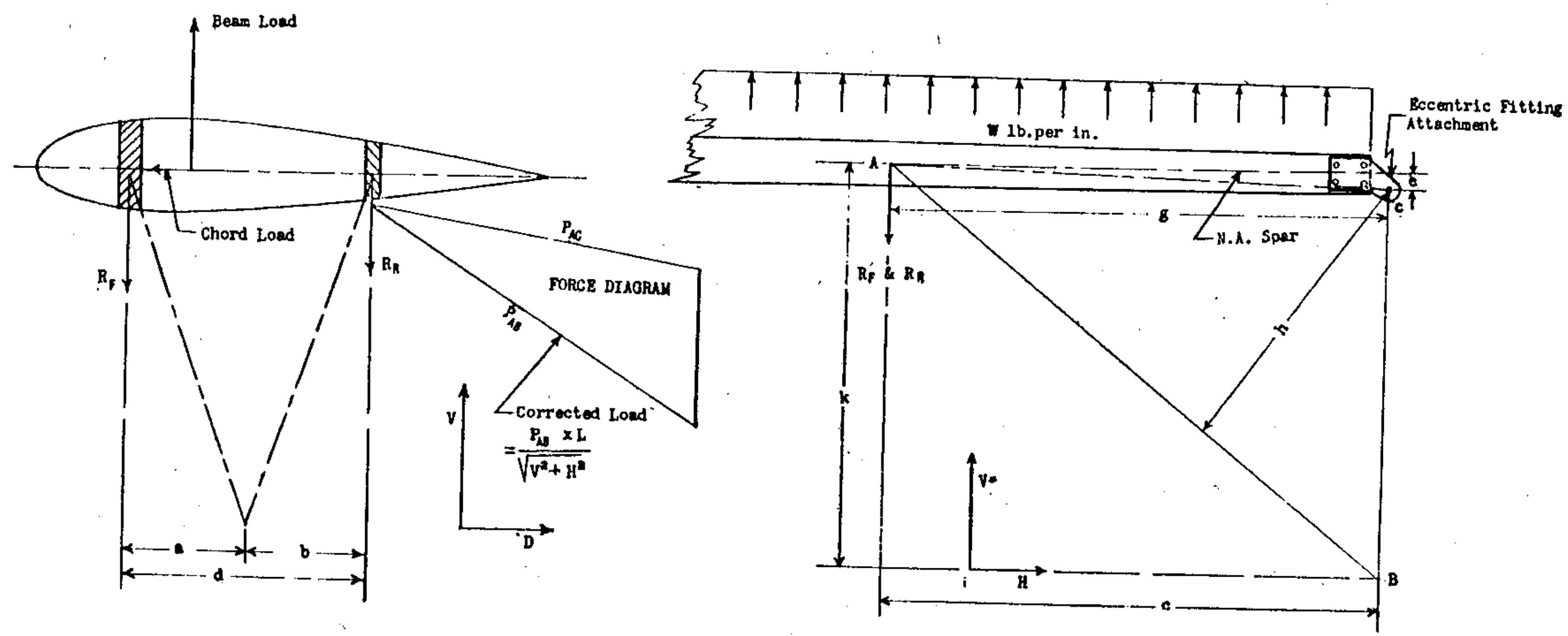
(*a*) The value of C_c, item 28, usually can be assumed to be constant over the span. The only variation required is in the case of partial-span wing flaps or similar devices.

(*b*) The relative location of the wing spars and drag truss will affect the drag-truss loading produced by the chord and normal air forces. This can easily be accounted for by correcting the value of C_c (sec. 3.1121).

It is often necessary to consider the local loads produced by the propeller thrust and by the drag of items attached to the wing. The drag of nacelles built into the wing is usually so small that it safely can be neglected. The drag of independent nacelles and that of wing-tip floats can be computed by using a rational drag coefficient or drag area in conjunction with the design speed. In general, the effects of nacelles or floats can be computed separately and added to the loads obtained in the design conditions.

3.112. *Lift-truss analysis.*

3.1120. *General.* In considering a lift-truss system for either a monoplane or a biplane and,



* For simplicity V and H axis taken perpendicular and parallel to spar N.A. respectively.

Member	V	H	D	V^2	H^2	D^2	$L^2=V^2+D^2+H^2$	L	$\sqrt{V^2+H^2}$ Proj. length V-H plane
Front Strut	k	c	a	k^2	c^2	a^2	$k^2+c^2+a^2$	$\sqrt{k^2+c^2+a^2}$	$\sqrt{k^2+c^2}$
Rear Strut	k	c	b	k^2	c^2	b^2	$k^2+c^2+b^2$	$\sqrt{k^2+c^2+b^2}$	$\sqrt{k^2+c^2}$

Figure 3–2. Strut-braced monoplane.