STEEL AND COMPOSITE BEAMS
WITH
WEB OPENINGS

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ABSTRACT

Three design methods, originally developed by Donahey and Darwin (1986), for determining the maximum shear capacity of composite beams with unreinforced web openings are extended to include steel and composite beams with or without reinforcement at the opening. The three design methods incorporate simplifying assumptions that permit closed-form solutions for maximum shear capacity. The first method assumes that the neutral axes for secondary bending lie in the flanges of the top and bottom tees and defines the interaction of shear and normal stresses by a linear approximation of the von Mises yield function. The second method ignores the contribution of the flanges to secondary bending moments and employs the von Mises yield function to define the interaction of shear and normal stresses. The third method ignores the contribution of the flanges to secondary bending moments and defines the interaction between shear and normal stresses with a linear approximation of the von Mises yield function. Simplified design expressions for the maximum moment capacity of steel and composite beams with web openings are presented. Six refinements of the design methods are investigated to determine their significance in predicting member strengths. Simplified design expressions developed by Darwin (1990) for determining the maximum moment capacity of steel and composite beams at web openings are summarized. The accuracy and ease of application of the design methods presented in this report (Methods I, II, and III) and applicable procedures proposed by Redwood and Shrivastava (1980), Redwood and Poumbouras (1984), and Redwood and Cho (1986) are compared with experimental results of fifty steel beams and thirty-five composite beams. Resistance factors are calculated for use in LRFD of structural steel buildings. The simplest of the design methods

presented in this report, coupled with moment-shear interaction procedures proposed by Donahey and Darwin (1986), provides excellent agreement with test results and a superior approach in terms of accuracy and ease of application. Resistance factors of 0.90 and 0.85, applied to both shear and bending, are suitable for steel and composite beams, respectively.

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1.0 INTRODUCTION

The aims of this report are to (1) extend three design methods, originally developed by Donahey and Darwin (1986), for determining the maximum shear capacity of composite beams with unreinforced web openings to cover steel and composite beams with or without reinforcement at the opening, (2) summarize simplified design expressions for the maximum moment capacity of composite and steel beams with web openings, (3) investigate the effect of the following on predicted capacities:

(a) the use of a linear approximation for the von Mises yield function by comparing two design methods that employ, respectively, the von Mises yield function, and a linear approximation of the von Mises yield function;

(b) the relative sizes of the flange and the web as a function of the design method by comparing two design methods where the only difference is whether the flanges are included or excluded in determining the secondary bending moments in a tee;

(c) reducing the tee depth to approximate the movement of the plastic neutral axis, *PNA*, with the addition of reinforcement by comparing two methods, one in which the *PNA* is constrained to the top of the flange, the other in which the *PNA* is permitted to move within the flange;

(d) limiting the normal force in the concrete at the high moment end of the opening to the axial yield capacity of the net top tee steel in a composite tee;

(e) limiting the maximum moment capacity, M_m, of reinforced steel beams to the plastic moment capacity of the unperforated section, M_p.

(f) limiting the normal force permitted in the reinforcement at the opening by the strength of the weld attaching the reinforcing steel to the web at the opening.

(4) compare the accuracy and ease of application of the three methods with procedures proposed by Redwood and Shrivastava (1980), Redwood and Poumbouras (1984), and Redwood and Cho (1986), and (5) calculate resistance factors, ϕ, for use in load and resistance factor design of structural steel buildings.

Comparisons are made with experimental results of thirty-five composite beams and fifty steel beams. The methods for shear and moment capacity found in Section 2.0 are compared with results obtained using procedures proposed by Redwood and Shrivastava (1980) for steel beams and with results published by Redwood and Shrivastava (1980), Redwood and Poumbouras (1984), and Redwood and Cho (1986) for composite beams with ribbed slabs, and composite beams with solid slabs, respectively.

2.0 STRENGTH DESIGN PROCEDURES

2.1 Overview of Design Procedures

In this section, the three design methods proposed by Donahey and Darwin (1986) for determining the maximum shear capacity, V_m, of composite beams with unreinforced web openings are modified to account for reinforcement at the opening and extended to cover steel beams. Design expressions for the maximum moment capacity, M_m, of composite and steel beams, with or without reinforcement, are also presented, as are the procedures for moment-shear interaction proposed by Donahey and Darwin (1986).

Figs. 2.1 and 2.2, respectively, illustrate web openings in steel beams and web openings in composite beams with solid and ribbed slabs. Openings are of length a_o, depth h_o, and may have an eccentricity, e, which is always taken as a positive for steel beams and positive in the upward direction for composite beams. The slab thicknesses, t_s and t_s', effective slab width, b_e, and steel section dimensions, d, b_f , t_f , t_w , s_t , s_b , b_r and t_r, are as indicated in these figures. The regions above and below the opening are referred to as the top tee and bottom tee, respectively. Definitions of variables and notation used in the report are given in Appendix A.

The procedures described in this report are based on the following assumptions:

(1) The steel will yield in tension or compression.

(2) Shear forces can be carried in the steel and the concrete at both ends of the opening.

(3) Shear forces in the steel are carried by the webs of the tees.

(4) Shear stresses are uniformly distributed over the depth of the webs.

(5) The normal forces in the concrete are applied over an area defined by an equivalent stress block.

(6) For the calculation of maximum moment capacity, the reinforcement is concentrated at the edge of the opening in the top and bottom tees.

2.2 Interaction Curve

The nominal shear and bending strengths, V_n and M_n, of a member at an opening subjected to both shear and bending moment are obtained using the interaction equation proposed by Donahey and Darwin (1986).

$$\frac{M_n^3}{M_m^3} + \frac{V_n^3}{V_m^3} = 1 \tag{2.1}$$

This continuous function, illustrated in Fig. 2.3, permits the calculation of the nominal shear and moment capacities and provides good agreement with test data (see Section 3.0).

Eq. 2.1 can be rearranged to provide a convenient expression for V_n or M_n for a given moment to shear ratio, M/V.

$$\frac{M_n^3 V_m^3}{V_n^3 M_m^3} + 1 = \frac{V_m^3}{V_n^3} \tag{2.2}$$

Setting $M_n/V_n = M/V$ and solving for V_n, gives

$$V_n = V_m \left(\frac{\dfrac{M^3}{V^3}}{\dfrac{M_m^3}{V_m^3}} + 1 \right)^{-1/3} = V_m \left(\frac{\dfrac{M^3}{M_m^3}}{\dfrac{V^3}{V_m^3}} + 1 \right)^{-1/3} \tag{2.3}$$

$$M_n = V_n \left(\frac{M}{V} \right) \tag{2.4}$$

$$M_n = M_m\left(\frac{\frac{M_m^3}{V_m^3}}{\frac{M^3}{V^3}} + 1\right)^{-1/3} = M_m\left(\frac{\frac{V^3}{V_m^3}}{\frac{M^3}{M_m^3}} + 1\right)^{-1/3} \tag{2.5}$$

2.3 Forces at the Opening

The forces acting at a web opening are shown in Fig. 2.4. Under positive bending, the top and bottom tees are each subjected to axial forces P_t and P_b, shear forces, V_t and V_b, and secondary bending moments, M_{tl}, M_{th} and M_{bl}, M_{bh}, respectively. Using equilibrium, the following relationships result.

$$P_b = P_t = P \tag{2.6}$$

$$V = V_b + V_t \tag{2.7}$$

$$V_b a_o = M_{bl} + M_{bh} \tag{2.8}$$

$$V_t a_o = M_{tl} + M_{th} \tag{2.9}$$

$$M = Pz + M_{th} + M_{bh} - \frac{Va_o}{2} \tag{2.10}$$

in which V = total shear acting at an opening;

M = primary moment acting at opening center line;

a_o = length of the opening; and

z = distance between the local neutral axes in the top and bottom tees.

2.4 Shear Capacity Equations

In this section, the three design methods, developed by Donahey and Darwin (1986) to predict the maximum shear capacity of composite beams with unreinforced web openings, are extended to cover both steel and composite beams with or without reinforcement at the opening. Theoretical differences between the methods and limitations of the methods are discussed.

A closed-form solution for the maximum shear capacity at a web opening requires the use of several simplifying assumptions. Three closed-form solutions for the maximum shear capacity are derived, each simpler than the previous one. These closed-form solutions, hereafter referred to as Methods I, II, and III, are based on the assumption that the normal forces in the top and bottom tees is zero. As discussed by Clawson and Darwin (1980) and Donahey and Darwin (1986), this load state only approximates pure shear at the opening in composite beams because the secondary bending moments at the high and low moment ends of the top tee are not equal. As a result, the total moment at the opening center line is close to, but not equal to zero. The procedure, however, does represent pure shear in steel beams and gives a close approximation of the true maximum shear maximum capacity at web openings in composite beams.

The approach that is taken in the following sections is to develop an expression for the maximum shear capacity of the most general case, a top tee in a composite beam with a reinforced opening. The capacity of the other tees, top or bottom, can be obtained from the general case by neglecting appropriate terms in the expressions. The total shear capacity at an opening is obtained by summing the shear strengths of the top and bottom tees.

2.4.1 Forces in the Concrete and Steel

Normal forces in a composite tee are illustrated in Fig. 2.5. For composite beams, the normal force in the concrete at the high moment end of the opening, P_{ch}, is limited by the compressive strength of the concrete, the shear connector capacity, and the tensile strength of the top-tee steel. These limitations are expressed as follows.

$$P_{ch} \leq 0.85 f_c' b_e t_e \tag{2.11}$$

$$P_{ch} \leq N Q_n \tag{2.12}$$

$$P_{ch} \leq F_y A_{st} \tag{2.13}$$

in which t_e = t_s for solid slabs;

t_e = t_s' for ribbed slabs with transverse ribs;

t_e = $(t_s + t_s')/2$ for ribbed slabs with longitudinal ribs;

f_c' = concrete compressive strength, ksi;

Q_n = shear connector capacity accounting for appropriate reduction factor for ribbed slabs;

A_{st} = area of top tee steel, including reinforcement; and

N = number of shear connectors from high moment end of opening to the support.

Fig. 2.5 shows the location of the concrete normal forces. Shear stresses are assumed to have no effect on the normal stresses in the concrete at the maximum load.

The concrete force at the low moment end of the tee, P_{cl}, is dependent upon the number of shear connectors over the opening, N_o, and the high moment end concrete force, P_{ch}.

$$\boldsymbol{P_{cl} = P_{ch} - N_o Q_n \geq 0} \tag{2.14}$$

N_o and N include only the shear connectors entirely within the opening. Connectors at the edge of the high-moment end of the opening are not included.

The moment arms of the high moment end and low moment end concrete forces about the top of the steel flange, d_h and d_l, respectively, are given by the following equations.

$$d_h = t_s - \frac{P_{ch}}{1.7 f_c' b_e} \tag{2.15}$$

For solid slabs,

$$d_l = \frac{P_{cl}}{1.7 f_c' b_e} \tag{2.16}$$

For ribbed slabs with transverse ribs,

$$d_l = t_s - t_s' + \frac{P_{cl}}{1.7 f_c' b_e} \tag{2.17}$$

For ribbed slabs with longitudinal ribs, d_l is the distance from the top of the flange to the centroid of the compression force in the concrete. Only the ribs that lie within the effective width, b_e, are considered for this calculation. A conservative estimate of d_l can be obtained by treating the sum of the minimum widths of the ribs that lie within the effective width of the slab as b_e.

The maximum shear in the top tee, V_{mt}, is assumed to be carried by the steel web unless V_{mt} exceeds the plastic shear capacity of the top tee web, given by

$$V_{pt} = \frac{F_{yw} t_w s_t}{\sqrt{3}} \tag{2.18}$$

This is possible only for a composite tee, not for other cases derived from the composite tee. When the plastic shear capacity of the top tee is exceeded, the top tee web will fully yield in shear and will not contribute to moment equilibrium of the tee. As will be explained, Eqs. 2.32, 2.43, and 2.54 predict maximum shear capacity in accordance with Methods I, II, and III, respectively, when the top tee web contributes to moment equilibrium. When the web fully yields in shear, these equations must be rederived, excluding any contribution of the top tee web to moment equilibrium. This results in Eq. 2.33 for Method I and Eq. 2.46 for Methods II and III. In this case, the normal force in the concrete, at high moment end of the opening, P_{ch}, is further limited based on the reduced normal force in the top tee steel.

$$P_{ch} \leq F_{yf} t_f (b_f - t_w) + P_r \tag{2.19}$$

in which P_r = normal force in the reinforcement in the top tee.

$$P_r = F_{yr} t_r (b_r - t_w) \leq \frac{F_{yw} t_w a_o}{2\sqrt{3}} \tag{2.20}$$

The term on the right side of the inequality in Eq. 2.20 represents the horizontal shear strength of the web below or above the opening. Following the determination of V_{mt}, the result must be compared to the combined shear capacity of the steel web and the concrete over the opening, $V_{t(sh)}$, given by Eq. 2.21.

$$V_{t(sh)} = V_{pt} + V_c \tag{2.21}$$

in which V_c = pure shear capacity of the concrete slab = $0.11\sqrt{f_c'}A_{vc}$, kips;

f_c' and $\sqrt{f_c'}$ are in ksi; and

A_{vc} = effective concrete shear area = $3t_s t_e$

The maximum shear capacity of the bottom tee, V_{mb} , assumed to be non-composite, may not exceed the plastic shear capacity of the web in the bottom tee, which is

$$V_{pb} = \frac{F_{yw}\, t_w\, s_b}{\sqrt{3}} \tag{2.22}$$

The maximum shear capacity of the section, V_m, is the sum of the maximum capacities of the top and bottom tees expressed as

$$V_m = V_{mt} + V_{mb} \tag{2.23}$$

2.4.2 Derivation of the Design Methods

The three design methods are developed for the most general case, a composite tee with a reinforced opening. In each of the three design methods, the von Mises yield function, or a simplification of the function, is used to model the reduced normal yield strength of the web, $\overline{F}_y$, caused by interaction with the shear stress, τ.

For a material with yield strength, F_y, the von Mises yield function is given by

$$\overline{F}_y = \sqrt{F_y^2 - 3\tau^2} \tag{2.24}$$

which is illustrated in Fig. 2.6.

The three design methods derived in the following sections employ simplifying assumptions that permit a closed-form solution for the maximum shear capacity.

2.4.2.1 Method I

The fully plastic stress distribution at an opening with zero axial force in the tees is illustrated in Fig. 2.7. Two simplifying assumptions will be made in the derivation of this method to facilitate a closed-form solution for the maximum shear capacity. First, the position of the neutral axis in the top and bottom tees for secondary bending is assumed to lie in the flanges. Second, the interaction of shear and normal stresses is defined by a linear approximation of the von Mises yield function given by

$$\bar{F}_y = \lambda F_y - \sqrt{3}\tau \tag{2.25}$$

in which $\tau \leq F_y/\sqrt{3}$

λ = a factor used to adjust the approximation to obtain an improved match with experimental results. Donahey and Darwin (1986) used $\lambda = (1 + \sqrt{2})/2 = 1.207$. As will be shown in Section 3.0, a value of $\lambda = \sqrt{2}$ appears to give better results.

The maximum shear capacity of a composite tee is found by using the moment equilibrium equation for the tee.

$$V_t a_o = M_{th} + M_{tl} \tag{2.26}$$

To determine M_{th} and M_{tl} based on the stresses in the steel and concrete, the locations of the neutral axes at the high and low moment ends of the opening, g_h and g_l, must be known. g_h and g_l are measured with respect to the outside of the flange (Fig. 2.7).

Assuming the neutral axis to be in the flange and using normal force equilibrium,

$$g_h = \frac{-P_{ch} + F_{yf}(b_f - t_w)t_f + F_y t_w s_t + P_r}{2\left[F_{yf}(b_f - t_w) + \bar{F}_y t_w\right]} \tag{2.27}$$

$$g_l = \frac{P_{cl} + F_{yf}(b_f - t_w)t_f + F_y t_w s_t + P_r}{2\left[F_{yf}(b_f - t_w) + \bar{F}_y t_w\right]} \tag{2.28}$$

in which $P_r = F_{yr} t_r (b_r - t_w)$

Substituting Eq. 2.25 for $\bar{F}_y$ in Eqs. 2.27 and 2.28 results in the following expressions for g_h and g_l.

$$g_h = \frac{-P_{ch} + F_{yf}(b_f - t_w)t_f + \lambda\bar{F}_y t_w s_t - V_{mt}\sqrt{3} + P_r}{2\left(F_{yf}(b_f - t_w) + \lambda\bar{F}_y t_w - \dfrac{\sqrt{3}\,V_{mt}}{t_w}\right)} \tag{2.29}$$

$$g_l = \frac{P_{cl} + F_{yf}(b_f - t_w)t_f + \lambda\bar{F}_y t_w s_t - V_{mt}\sqrt{3} + P_r}{2\left(F_{yf}(b_f - t_w) + \lambda\bar{F} t_w - \dfrac{\sqrt{3}\,V_{mt}}{t_w}\right)} \tag{2.30}$$

Using moment equilibrium of the tee

$$V_{mt} a_o = P_{ch} d_h - P_{cl} d_l - \frac{F_{yf}(b_f - t_w)(g_h^2 + g_l^2)}{2} + \frac{F_{yf}(b_f - t_w)(t_f^2 - g_l^2)}{2}$$

$$+ \frac{F_{yf}(b_f - t_w)(t_f^2 - g_h^2)}{2} - \frac{\bar{F}_y t_w (g_h^2 + g_l^2)}{2} + \frac{\bar{F}_y t_w (s_t^2 - g_l^2)}{2}$$

$$+ \frac{\bar{F}_y t_w (s_t^2 - g_h^2)}{2} + 2F_{yr} t_r (b_r - t_w)(s_t - y_r) \tag{2.31}$$

Substituting the resulting expressions for g_h and g_l into Eq. 2.31 and simplifying results in an equation that is quadratic in V_{mt}. This equation can be reduced to

$$V_{mt} = F_y\left(\frac{\beta - \sqrt{\beta^2 - 4\alpha\gamma}}{2\alpha}\right) \tag{2.32}$$

in which $\alpha = 3 + \dfrac{2\sqrt{3}\,a_o}{s_t}$

$$\beta = 2\sqrt{3}(b_f - t_w)\left(s_t - t_f + \frac{t_f^2}{s_t}\right) + 2\sqrt{3}\lambda t_w s_t + 2a_o[b_f + (\lambda - 1)t_w]$$

$$+ \frac{2\sqrt{3}(2P_r d_r + P_{ch}d_h - P_{cl}d_l)}{s_t F_y} + \frac{\sqrt{3}(P_{ch} - P_{cl} - 2P_r)}{F_y}$$

$$\gamma = (b_f - t_w)^2 t_f^2 + \lambda^2 t_w^2 s_t^2 + 2\lambda t_w(b_f - t_w)(s_t^2 - s_t t_f + t_f^2)$$

$$+ \frac{2[b_f + (\lambda - 1)t_w]}{F_y}(2P_r d_r + P_{ch}d_h - P_{cl}d_l)$$

$$- \frac{(2P_r^2 + P_{ch}^2 + P_{cl}^2)}{2F_y^2} + \frac{P_r(P_{ch} - P_{cl})}{F_y^2}$$

$$+ \frac{[(b_f - t_w)t_f + \lambda t_w s_t]}{F_y}(P_{ch} - P_{cl} - 2P_r)$$

For the derivation of preceding terms using the different yield strengths for the flanges, web, and reinforcement, see Appendix B.

If $V_{mt} > V_{pt}$, the web has yielded. Resolving Eq. 2.29 through Eq. 2.31 with $\overline{F}_y = 0.0$ gives

$$V_{mt} = \frac{[2P_r d_r + P_{ch} d_h - P_{cl} d_l + \frac{t_f}{2}(P_{ch} - P_{cl} - 2P_r)]}{a_o}$$

$$+ \frac{F_y}{2}(b_f - t_w)t_f^2 + \frac{2P_r(P_{ch} - P_{cl}) - 2P_r^2 - P_{ch}^2 - P_{cl}^2}{4F_y(b_f - t_w)} \geq V_{pt} \tag{2.33}$$

2.4.2.2 Method II

The primary simplification made in this method is to ignore the contribution of the flanges to the secondary bending moments. This approximation works because the contribution of the normal stresses in the flanges to the secondary moments is small when moments are calculated about the extreme edges of the flanges. Both the normal and the shear stresses are assumed to be uniform within the web. The normal stresses in the reinforcement are assumed to act at the centroid of the reinforcement. The plastic stress distribution is illustrated in Fig. 2.8. The von Mises yield function, Eq. 2.24, controls the stresses in the web.

The normal force in the web when shear is acting on a tee, P_w, is given by

$$P_w = \overline{F}_y s_t t_w \tag{2.34}$$

The shear stress, τ, is

$$\tau = \frac{V_{mt}}{s_t t_w} \tag{2.35}$$

Substituting Eq. 2.34 and Eq. 2.35 into the von Mises yield function results in the following equation for the normal force in the web.

$$P_w = \sqrt{3V_{pt}^2 - 3V_{mt}^2} \tag{2.36}$$

Taking moments about the top of the flange results in

$$V_{mt}a_o = P_w s + P_{ch}d_h - P_{cl}d_l + 2P_r d_r \tag{2.37}$$

Eq. 2.37 can be more simply represented by

$$V_{mt}a_o = P_{wt}s_t + \mu V_{pt}s_t \tag{2.38}$$

in which $\mu = \dfrac{P_{ch}d_h - P_{cl}d_l + 2P_r d_r}{V_{pt}s_t}$

Substituting Eq. 2.36 into Eq. 2.38 and solving gives,

$$V_{mt}a_o = \sqrt{3V_{pt}^2 - 3V_{mt}^2} + \mu V_{pt}s_t \tag{2.39}$$

$$(V_{mt}a_o - \mu V_{pt}s_t)^2 = 3V_{pt}^2 - 3V_{mt}^2 \tag{2.40}$$

$$V_{mt}^2 a_o^2 - 2\mu V_{pt}s_t V_{mt}a_o + (\mu V_{pt}s_t)^2 = 3V_{pt}^2 - 3V_{mt}^2 \tag{2.41}$$

$$V_{mt}^2(a_o^2 + 3) - V_{mt}(2\mu V_{pt}s_t a_o) + (\mu V_{pt}s_t)^2 - 3V_{pt}^2 = 0 \tag{2.42}$$

Substituting υ = aspect ratio of the tee = a_o/s_t into Eq. 2.42 and solving for V_{mt} gives

$$V_{mt} = V_{pt}\left(\frac{\mu\upsilon - \sqrt{3\upsilon^2 - 3\mu^2 + 9}}{\upsilon^2 + 3}\right) \tag{2.43}$$

When reinforcement is added at the edge of the opening, the plastic neutral axis, *PNA*, will shift toward the opening to maintain equilibrium in the tee. However, a key assumption made in the derivation of this method is that the *PNA* is located at the top of the flange. This assumption becomes increasingly unconservative as more reinforcement is added. An adjustment can be made to approximate the true movement of the *PNA* by reducing the effective depth of the steel tee, in the calculation of υ, by a distance which is proportional to the amount of reinforcement present.

$$\upsilon = \frac{a_o}{\overline{s}} \tag{2.44}$$

in which $\overline{s} = s - \dfrac{A_r}{2b_f}$

The procedure to approximate the movement of the PNA is discussed in greater detail in Section 2.4.3.2.

When V_{mt} exceeds the plastic shear capacity of the web, V_{pt}, an alternate determination of maximum shear capacity is necessary because the web has yielded in shear. In this case, P_w = 0.0 and Eq. 2.38 gives

$$V_{mt}a_o = \mu V_{pt}s_t \tag{2.45}$$

Solving for V_{mt} gives

$$V_{mt} = \frac{\mu V_{pt}s_t}{a_o} = \frac{\mu V_{pt}}{\upsilon} \tag{2.46}$$

in which μ is defined in Eq. 2.38 and $\upsilon = a_o/s$. <u>No adjustment</u> is necessary in s for Eq. 2.46, when reinforcement is present.

2.4.2.3 Method III

A linear solution for the maximum shear capacity is possible by adding the linear approximation for the von Mises yield function, Eq. 2.25, used in Method I to the simplified stress distribution used in Method II (see Fig. 2.8).

The normal force in the web when shear is acting on a tee is given by

$$P_w = \bar{F}_y s_t t_w \tag{2.47}$$

Substituting Eq. 2.47 into Eq. 2.25 results in

$$P_w = (\lambda F_y - \sqrt{3}\tau) s_t t_w \tag{2.48}$$

Rewriting Eq. 2.48 in terms of V_{pt} and V_{mt} results in

$$P_w = \sqrt{3}(\lambda V_{pt} - V_{mt}) \tag{2.49}$$

The maximum shear capacity in the top tee, V_{mt}, can be found by taking moments about the top of the flange.

$$V_{mt} a_o = P_w s_t + P_{ch} d_h + P_{cl} d_l + 2P_r d_r \tag{2.50}$$

Substituting Eq. 2.49 into Eq. 2.50 gives

$$V_{mt} a_o = \sqrt{3}(\lambda V_{pt} - V_{mt}) s_t + P_{ch} d_h + P_{cl} d_l + 2P_r d_r \tag{2.51}$$

Consolidating terms results in

$$V_{mt}(a_o + s_t\sqrt{3}) = \lambda\sqrt{3} V_{pt} s_t + P_{ch} d_h + P_{cl} d_l + 2P_r d_r \tag{2.52}$$

Rearranging, and using υ and μ as defined in Eqs. 2.38 and 2.44,

$$V_{mt}(\upsilon + \sqrt{3}) = V_{pt}(\lambda\sqrt{3} + \mu) \qquad (2.53)$$

$$V_{mt} = V_{pt}\left(\frac{\lambda\sqrt{3} + \mu}{\upsilon + \sqrt{3}}\right) \qquad (2.54)$$

As with Method II, the definition of υ should be altered to account for the shift in the PNA when reinforcement is added to a tee (see Eq. 2.44, also Section 2.4.3.2).

When V_{mt} exceeds the plastic shear capacity of the web, V_{pt}, the alternate determination of maximum shear capacity summarized in Eq. 2.46 applies.

2.4.3 Limitations and Differences Between Design Methods

The preceding derivations can be more fully understood by exploring the limitations of the simplifying assumptions.

In this section, the effect of the linear approximation for the von Mises yield function for secondary bending will be evaluated by comparing the predicted maximum shear capacities using Methods II and III. The effect of neglecting the flanges when determining maximum shear capacity will be established by comparing Methods I and III over the range of permissible combinations of opening length and tee depths.

Fig. 2.6 illustrates the von Mises yield function and its linear approximations when λ = 1.207 and λ = 1.414. Two concerns arise when the linear approximation of the von Mises yield function is used. First, for slender tees (high υ), it is possible that the predicted normal stress in the web, $\overline{F_y}$, will exceed the yield stress of the web, F_y. This unconservative prediction of $\overline{F_y}$

results in a less conservative and potentially unconservative prediction of the maximum shear capacity when using Methods I and III, compared to the maximum shear capacity predicted by Method II. Second, for stocky tees (low υ), it is possible that the predicted shear stress in the web, τ, will exceed the shear stress predicted by the von Mises yield function. This will also result in less conservative predictions of maximum shear capacity for Methods I and III compared to Method II.

2.4.3.1 Effect of the Linear Approximation of the von Mises Yield Function

The linear approximation of the von Mises yield function allows the normal stress in the web, $\overline{F}_y$, to be overpredicted by as much as 41% when $\lambda = 1.414$, as indicated in Fig. 2.9, which is a comparison of yield functions considering practical restraints. While this large overprediction is possible, the practical maximum stress predicted by the linear yield function is $1.236F_y$ when υ is limited to 12.0 and $\lambda = 1.414$ (see Appendix C). At this same practical maximum, for an unreinforced tee, Method III predicts a maximum shear capacity that exceeds that predicted by Method II by 24.5%, while the absolute difference between Methods II and III is 3.5% of the plastic shear capacity of the tee, V_p. For an unreinforced tee, when $\lambda = 1.207$ and $\upsilon = 12.0$, the predicted maximum shear capacities of Method II and III differ by 6.3% which translates to 0.90% of V_p. Another practical consideration that further reduces the effect of the unconservative normal stress in the web on the predicted maximum shear capacity is a restriction, p_o, placed on the size of the opening (see Appendix D). This restriction limits the value of υ for the second tee to 2.836 when $p_o = 5.6$, $a_o/h_o = 3.0$ and $\upsilon = 12.0$ for first tee. This is illustrated for $\lambda = 1.414$ and $\lambda = 1.207$ in Figs. 2.10 and 2.11. Fig 2.10 illustrates the difference between the maximum shear capacities predicted by Methods II and III for the top and bottom tees normalized on the plastic shear capacity of the perforated web versus a_o/s_t. Fig. 2.11 illustrates the ratio of Methods II and

III versus a_o/s_t. A W21X44 beam with an opening depth, h_o, equal to 50% of the overall beam depth is used for the comparisons. The curves were generated by varying the opening length, a_o. The ratio of opening length to tee depth for the bottom tee, a_o/s_b, becomes limited as the opening length increases, consequently, the difference in the combined maximum shear capacities of the top and bottom tees predicted by Methods II and III diminishes as the opening length increases. Fig. 2.11 was generated with the same beam and opening except the ratio of the maximum shear capacities predicted by Methods II and III is plotted with respect to a_o/s_t. For either value of λ, the predicted maximum shear capacity is not significantly affected by the unconservative prediction of the normal stress in the web by the linear approximation of the von Mises yield function.

For openings with a low υ, the linear approximation of the von Mises yield function can predict the shear stress in the web of a tee to be as much as 9.7% higher than that predicted by the von Mises yield function when $\lambda = 1.414$ and $\upsilon = 0.717$, as illustrated in Fig 2.9. The corresponding maximum shear capacities predicted by Methods II and III differ by 9.9%, or 9.0% of the plastic shear capacity of the tee. When $\lambda = 1.207$ and $\upsilon = 0.359$, the linear approximation overpredicts the shear stress in the web by 2.1%, and the corresponding maximum shear capacities predicted by Methods II and III differ by 2.2%, or 2.1% of the plastic shear capacity of the tee. When $\lambda = 1.414$ and $\upsilon = 0.717$, the potential difference between the maximum shear capacities predicted by Methods II and III are significant and will have the most effect on the nominal shear capacity when the opening is under high shear. Openings with $\upsilon = 0.717$ or 0.359 are very unlikely, however. Consequently, potentially unconservative predictions of maximum shear capacity by Method III are very unlikely to occur in practice.

The effect of the linear approximation of the von Mises yield function on the predicted capacities of fifty steel and thirty-five composite beams is investigated further in Section 3.4.

2.4.3.2 Effect of Reducing the Tee Depth in Proportion to Reinforcement

For an unreinforced steel tee, with $\mu = 0.0$, Method I predicts a higher maximum shear capacity than Method III over the entire range of acceptable values of $\upsilon = a_o/s$, as illustrated in Fig. 2.12. This difference is as high as 15% of the plastic shear capacity of the tee when $a_o/s = 2.00$. As reinforcement is added to a tee, the *PNA* will shift toward the opening, and the assumption made in the derivation of Methods II and III, that the *PNA* is at the top of the flange, becomes increasingly unconservative. Method I accounts for the shift of the *PNA*, so reasonably conservative predictions of shear capacity can be expected regardless of the amount of reinforcement at an opening. The unconservative difference between Methods I and III when nothing is done to account for the shift in the *PNA* is about 7.5% of the plastic shear capacity of the tee when $\mu = 9.0$ and $a_o/s = 12.0$. By reducing the depth of the tee in proportion to the reinforcement present (Eq. 2.44), the unconservative difference between Methods I and III is reduced to about 2% of V_{pt} for heavily reinforced slender tees. As shown in Fig. 2.12, with increasing quantities of reinforcement, it becomes more likely that the maximum shear capacity of a steel tee will be governed by the plastic shear capacity of the tee. The unconservative affect on predicted shear capacity by an unadjusted *PNA* location for Method III will likely be lessened in many situations because the plastic shear capacity of a tee will govern. However, reducing the tee depth in proportion to the reinforcement present to approximate the actual shift in the *PNA* permits the prediction of maximum shear capacity more in line with those predicted by Method I.

2.5 Moment Capacity Equations

The expressions for the maximum moment capacity of steel and composite beams with web openings presented in this section are applicable only to members meeting AISC (1986) criteria for compact sections. Instabilities in the compression flange or web, likely in non-compact sections, may render the expressions of this section unconservative because the full strength at the opening may not be attained.

Well established strength procedures are employed in deriving the expressions for maximum moment capacity, M_m. In all cases, fully plastic behavior is assumed for the steel section in both tension and compression.

2.5.1 Steel Beams

The maximum moment capacity of unreinforced steel beams, as derived in this section, involves no approximations. Simplified, conservative design expressions for reinforced steel beams are derived by assuming that the reinforcement is concentrated along the top and bottom edges of the opening and that the thickness of the reinforcement is small. For members with an eccentric opening, $e \neq 0$, the plastic neutral axis will be located in the reinforcement at the edge of the opening closest to the centroid of the original steel section or in the web of the deeper tee. When reinforcement is used, the maximum moment capacity, M_m, should not exceed the flexural strength of the unperforated beam, M_p.

The eccentricity of an opening, e, is always taken to be positive in steel beams. Figs. 2.13(a), 2.13(b), and 2.13(c) illustrate stress diagrams for steel sections in pure bending.

2.5.1.1 Unreinforced Openings

For members with unreinforced openings and eccentricity, e, the maximum moment capacity of a steel member can be expressed as

$$M_m = M_p - F_y t_w \left(\frac{h_o^2}{4} + e h_o \right) \tag{2.55}$$

$$M_m = M_p - F_y \Delta A_s \left(\frac{h_o}{4} + e \right) \tag{2.56}$$

in which $\Delta A_s = h_o t_w$.

2.5.1.2 Reinforced Openings

The maximum moment capacity of steel beams with reinforcement along both the top and bottom edges of the opening are derived in this section. Two simplifying assumptions are used in the following derivation so that concise, conservative expressions for M_m are possible. First, the reinforcement is assumed to be concentrated along the top and bottom edges of the opening, and second, the thickness of the reinforcement is assumed to be small. The maximum moment capacity of a perforated, reinforced, steel beam in which the *PNA* resides in the reinforcement and $e \leq F_{yr} A_r / F_y t_w$ can then be expressed as

$$M_m = M_p - F_y t_w \left(\frac{h_o^2}{4} + e h_o - e^2 \right) + F_{yr} A_r h_o \leq M_p \tag{2.57}$$

in which F_{yr} = yield strength of the reinforcement

A_r = area of reinforcement at the top or bottom of an opening

The maximum moment capacity of a perforated, reinforced, steel beam in which the *PNA* resides in the web and $e \geq F_{yr}A_r \,/\, F_y t_w$ can be expressed as

$$M_m = M_p - F_y t_w \left(\frac{h_o^2}{4} + e h_o - \frac{F_{yr}A_r}{F_y t_w}\left(2e + \frac{h_o}{2}\right)\right) + F_{yr}A_r\left(\frac{h_o}{2} - \frac{F_{yr}A_r}{F_y t_w}\right) \leq M_p \tag{2.58}$$

Further simplification is possible if Eq. 2.58 is rewritten in terms of the original unperforated cross-section.

$$M_m = M_p - F_y \Delta A_s \left(\frac{h_o}{4} + e\right) + F_{yr}\Delta A_s \frac{A_r}{2t_w} \leq M_p \tag{2.59}$$

in which $\Delta A_s = h_o t_w - \dfrac{2A_r F_{yr}}{F_y}$

2.5.2 Composite Beams

Expressions for the maximum moment capacity of composite beams (Darwin 1990) are presented in this section. Simplified design expressions (Darwin 1990) are also developed following a review of the more precise moment capacity equations. When the opening is reinforced, the maximum moment capacity, M_m, should not exceed the flexural strength of the unperforated composite section, M_{pc}. The eccentricity of the opening, *e*, is taken to be positive in the upward direction in composite beams. Figs. 2.14(a), 2.14(b), 2.14(c) illustrate stress diagrams for composite beams in pure bending.

2.5.2.1 Derivation

For a given beam and opening configuration, the force in the concrete, P_c, is limited to the lower of the concrete compressive strength, the shear connector capacity, or the tensile capacity of the net steel section.

$$P_c \leq 0.85 f_c' b_e t_e \tag{2.60}$$

$$P_c \leq N Q_n \tag{2.61}$$

$$P_c \leq T' = F_y A_{sn} \tag{2.62}$$

in which $A_{sn} = A_s - h_o t_w + \dfrac{2 A_r F_{yr}}{F_y}$

The depth of the concrete stress block, a, for solid slabs or for ribbed slabs with transverse ribs is given by

$$a = \frac{P_c}{0.85 f_c' b_e} \tag{2.63}$$

The maximum moment capacity, M_m, is dependent on the governing inequality from Eqs. 2.60, 2.61, and 2.62. If $P_c = T'$ [Eq. 2.62, Fig. 2.14(a)], the *PNA* resides at the top of the steel flange and the maximum moment capacity is expressed by

$$M_m = T' \left(\frac{d}{2} + \frac{\Delta A_s e}{A_{sn}} + t_s - \frac{a}{2} \right) \leq M_{pc} \tag{2.64}$$

in which $\Delta A_s = h_o t_w - \dfrac{2 A_r F_{yr}}{F_y}$

e = opening eccentricity, (+) upward for composite beams

Eq. 2.63 is valid for ribbed slabs if $a \leq t_s'$. If $a > t_s'$, as is possible for ribbed slabs with longitudinal ribs, the term (t_s - $a/2$) in Eq. 2.64 must be replaced with the appropriate expression for the distance between the top of the steel flange and the centroid of the concrete force.

If $P_c < T'$ (Eq. 2.60 or Eq. 2.61), the *PNA* is in the steel section, placing a portion of the steel member in compression. The *PNA* can be either in the flange or the web of the top tee, based on the inequality

$$P_c + F_y A_f \overset{>}{\underset{<}{}} F_y (A_{sh} - A_f) \tag{2.65}$$

in which A_f = flange area = $b_f\, t_f$.

If the force in the concrete and the tensile capacity of the flange (left side of Eq. 2.65) exceeds the tensile capacity of the web (right side of Eq. 2.65), the *PNA* will be in the flange (Fig. 2.14(b)) at a distance x from the top of the flange. For this case,

$$x = \frac{(A_{sn} F_y - P_c)}{2 b_f F_y} \tag{2.66}$$

The corresponding maximum moment capacity can be expressed as

$$M_m = T' \left(\frac{d}{2} + \frac{\Delta A_s e - b_f x^2}{A_{sn}} \right) + P_c \left(t_s - \frac{a}{2} \right) \leq M_{pc} \tag{2.67}$$

If the tensile capacity of the web exceeds the capacity of the concrete slab and steel flange, the *PNA* will reside in the web at a distance x from the top of the flange, as illustrated in Fig. 2.15 (c). For this case,

$$x = \frac{(A_{sn} - 2A_f)}{2 t_w} - \frac{P_c}{2 F_y t_w} + t_f \tag{2.68}$$

The corresponding maximum moment capacity can be expressed as

$$M_m = T'\left(\frac{d}{2} + \frac{\Delta A_s e - (b_f - t_w)t_f^2 - t_w x^2}{A_{sn}}\right) + P_c\left(t_s - \frac{a}{2}\right) \leq M_{pc} \tag{2.69}$$

2.5.2.2 Design Equations

Simplified design expressions (Darwin 1990) for the maximum moment capacity of perforated composite beams are developed in this section. When the *PNA* in an unperforated member resides at the top of the steel flange, Eq. 2.64, a simplified design expression is possible by assuming that $F_{yr} = F_y$, and that the internal moment arm between tensile and compressive forces is not significantly affected by the loss in steel area due to the opening or the addition of steel from the reinforcement.

Using the first assumption, Eq. 2.64 can then be rewritten as

$$M_m = A_{sn}F_y\left(\frac{d}{2} + \frac{\Delta A_s e}{A_{sn}} + t_s - \frac{a}{2}\right) \tag{2.70}$$

in which $A_{sn} = A_s - h_o t_w + 2A_r$

Rearranging,

$$M_m = A_{sn}F_y\left(\frac{d}{2} + t_s - \frac{a}{2}\right) + F_y \Delta A_s e \tag{2.71}$$

Using the second assumption, the term $(d/2 + t_s - a/2)$ is assumed to be about the same for the perforated and unperforated sections. Thus the first term of Eq. 2.71 can be expressed in terms of the maximum moment capacity of an unperforated composite section, M_{pc}.

$$M_m = M_{pc}\frac{A_{sn}}{A_s} + F_y \Delta A_s e \le M_{pc} \tag{2.72}$$

Eq. 2.72 is usually accurate within a few percent and is conservative when the steel cross-sectional area of the reinforced beam at the opening is less than that of the original unreinforced beam.

When the *PNA* in the unperforated member resides in the steel section, [Eq. 2.61 or 2.62], one design expression for M_m is possible by assuming that the term $-b_f x^2/A_{sn}$ in Eq. 2.67 and the term $[-(b_f - t_w)t_f^2 - t_w x^2]/A_{sn}$ in Eq. 2.69 are small in comparison to $d/2$ and, thus, can be ignored. The following simplified expression results.

$$M_m = F_y A_{sn}\left(\frac{d}{2} + \frac{\Delta A_s e}{A_{sn}}\right) + P_c\left(t_s - \frac{a}{2}\right) \tag{2.73}$$

Rearranging,

$$M_m = F_y A_{sn}\left(\frac{d}{2}\right) + P_c\left(t_s - \frac{a}{2}\right) + F_y \Delta A_s e \le M_{pc} \tag{2.74}$$

Eq. 2.74 is exact when the *PNA* lies at the top of the flange and can be used in place of Eq. 2.72, and it is very accurate, but slightly unconservative, when the *PNA* is in the flange. Eq. 2.74 becomes progressively more unconservative as the *PNA* moves into the web. A limitation on the application of Eq. 2.74 is then necessary to preclude overly unconservative results. This can be conservatively accomplished by limiting the magnitude of the terms neglected by Eq. 2.74

(see Eq. 2.67 and 2.69) to less than 4 percent of $d/2$ for members in which the flange area is greater than or equal to 40 percent of the web area [i.e., $(b_f - t_w)t_f \geq 0.4t_w d$]. This is accomplished by limiting the force in the concrete, P_c, to values greater than $F_y(0.75t_w d - \Delta A_s)$. The flange-to-web area ratio stipulation is conservative, and as that ratio increases, the accuracy of Eq. 2.74 improves. For members in which the *PNA* resides in the web, and either $P_c < F_y(0.75t_w d - \Delta A_s)$, or the flange-to-web area ratio is less than 0.40, M_m must be determined using Eq. 2.67 or 2.69. A derivation of the stipulation on P_c for configurations where the *PNA* is located in the web can be found in Appendix E.

2.6 Redwood Methods

In this section, the design expressions proposed by Redwood and Shrivastava (1980) for determining the maximum shear capacity, V_m, and an intermediate value of moment used for moment-shear interaction, M_v, for steel beams with and without reinforcement at the opening are altered to account for the yield strengths of the web and reinforcent. These altered expressions are used in calculating the nominal capacities of steel beams which are summarized in Tables 3.11, 3.19, and E.6. The expressions for determining moment capacity used with expressions presented in this section are those derived in Section 2.5.

The intermediate moment capacity, M_v, for an unreinforced beam at which the nominal shear capacity commences to diminish because of increasing moment at the opening is given by

$$M_v = M_p\left(1 - \frac{\dfrac{A_w F_{yw}}{4A_f F_{yf}}\left(1 - \dfrac{h_o}{d} + \dfrac{2e}{d}\right)\dfrac{2}{\sqrt{1 + \alpha_b}}}{1 + \dfrac{A_w F_{yw}}{4A_f F_{yf}}}\right)$$

The maximum shear capacity, V_m, of the top and bottom tees of an unreinforced beam is

$$V_m = \left(0.50\left(1 - \frac{h_o}{d} - \frac{2e}{d}\right)\sqrt{\frac{\alpha_t}{1 + \alpha_t}}\right) + \left(0.50\left(1 - \frac{h_o}{d} + \frac{2e}{d}\right)\sqrt{\frac{\alpha_b}{1 + \alpha_b}}\right) \tag{2.76}$$

in which

$$\alpha_t = \frac{3}{16}\left(\frac{2d}{a_o}\right)^2\left(1 - \frac{h_o}{d} - \frac{2e}{d}\right)^2$$

$$\alpha_b = \frac{3}{16}\left(\frac{2d}{a_o}\right)^2\left(1 - \frac{h_o}{d} + \frac{2e}{d}\right)^2$$

The intermediate moment capacity, M_v, for a reinforced beam at which the nominal shear capacity commences to diminish because of increasing moment at the opening is given by

$$M_v = \frac{\left(1 - \dfrac{A_r F_{yr}}{A_f F_{yf}}\right)}{\left(1 + \dfrac{A_r F_{yr}}{A_f F_{yf}}\right)} \tag{2.77}$$

The maximum shear capacity, V_m, of the top and bottom tees of a reinforced beam is

$$V_m = \sqrt{3}\left(\frac{2d}{a_o}\right)\left(\frac{A_r F_{yr}}{A_w F_{yw}}\right)\left(1 - \frac{h_o}{d}\right) \le V_{pb} + V_{pt} \tag{2.78}$$

3.0 ANALYSIS AND RESULTS

3.1 Introduction

In this section, the three design methods described in Section 2.0 are evaluated. The results from fifty steel beams and thirty-five composite beams are used for comparison. Of the fifty steel beams, nineteen are unreinforced with rectangular openings, ten are unreinforced with circular openings, and twenty-one are reinforced with rectangular openings. Of the thirty-five composite beams, twenty-two have ribbed slabs and thirteen have solid slabs. Two of the beams with solid slabs and one of the beams with ribbed slabs are reinforced at the opening. The proportioning and detailing guidelines presented in Appendix D are also discussed in this section, along with the equations used to calculate resistance factors. The results of six specific areas of investigation are presented in Sections 3.4 - 3.9. The six areas investigated are the effects of (1) varying λ, the factor used in the linear approximation of the von Mises yield function, (2) reducing the tee depth of a reinforced tee to approximate the actual movement of the plastic neutral axis with the addition of reinforcement, (3) limiting P_{ch}, the normal force in the concrete at the high moment end of the opening, by the axial yield capacity of the net steel in a composite tee, (4) limiting the normal force in the reinforcement at an opening by the capacity of the accompanying weld, (5) size of the flanges relative to the web as a function of the design method, and (6) limiting the maximum moment capacity of a perforated beam to the plastic moment capacity of the unperforated beam. These six areas are important because they are refinements, simplifications, and limitations that impact the accurate prediction of shear and moment capacity.

The comparisons made in Sections 3.4 - 3.9 are not based on tests specifically formulated to validate the refinement, simplification, or limitation in question, however. Consequently, the comparisons, in themselves, may not present a complete picture and the theoretical basis of these comparisons is of greater importance.

Dimensions and properties for the steel and composite beams included in the analysis are contained in Tables 3.1 and 3.2, respectively.

The results obtained using the expressions developed in Section 2.0 and presented in Appendix B to account for the yield strengths of the flanges, web, and reinforcement are summarized in Tables 3.3 - 3.10, and 3.13 - 3.18. Results obtained using the appropriate methods proposed by Redwood and Shrivastava (1980), Redwood and Poumbouras (1984) and Redwood and Cho (1986) are summarized in Tables 3.11, 3.12, 3.19 and 3.20. Table 3.21 is an overall summary of the results of the analysis for all of the methods considered.

3.2 Proportioning and Detailing Guidelines

Proportioning guidelines have been developed for web openings in steel and composite beams which are most recently summarized by Darwin (1990). These appear in Appendix D. The majority of the guidelines help to insure that failure of a beam, as predicted by the design methods presented in Section 2.0, does not occur prematurely.

The design limitations dictated by the proportioning and detailing guidelines for beams used in the analysis are presented in Table 3.3 for steel beams and in Table 3.4 for composite beams. Ten of the beams used in the analysis violate one or more of the proportioning guideleines which are summarized in Table 3.3(f) and Table 3.4(d) for steel and composite beams, respectively. These beams were retained in the analysis because either the violation was related more to detailing practice than to the strength of the beam and/or failure did not occur because of the violation.

Twenty-one steel beams and one composite beam tested in previous studies have been excluded from consideration in this analysis due to violations of the proportioning and detailing guidelines. Sixteen steel beams tested by Kim (1980) were excluded because of extremely conservative test/theory ratios for tests with shear acting at the opening. Dimensions and properties, design limitations and results for the design methods presented in this report and the applicable Redwood method for the excluded beams are presented in Appendix E.

3.3 Resistance Factor Determination

Resistance factors appropriate for the design methods presented in this report and design methods proposed by Redwood and Shrivastava (1980), Redwood and Poumbouras (1984), and Redwood and Cho (1986) were determined in accordance with procedures outlined in AISC (1986). The basic equation for determining the resistance factor, ϕ, is

$$\phi = \left(\frac{R_m}{R_n}\right) e^{-0.55\beta V_r} \tag{3.1}$$

in which R_m = mean resistance

R_n = nominal resistance according to expressions in Section 2.0

β = reliability index = 3.0

V_r = coefficient of variation of the resistance

The term R_m/R_n is the average test/theory ratio for a group of beams, expressed as

$$\frac{R_m}{R_n} = \left(\frac{F_{ym}}{F_{yn}}\right)\left(\frac{V_{test}}{V_n} , \frac{M_{test}}{M_n}\right) \tag{3.2}$$

in which F_{ym}/F_{yn} = mean steel strength/nominal steel strength = 1.07;

This value was determined by Galambos (1978) using a large number of test coupons from steel beams. It serves to account for the additional strength available from steel beams beyond the nominal yield strength.

V_{test} = actual shear capacity at an opening

V_n = predicted shear capacity at an opening

M_{test} = actual moment capacity at an opening

M_n = predicted moment capacity at an opening

The term V_r is the coefficient of variation resulting from several sources of variation, which is given by

$$V_r = \sqrt{V_m^2 + V_c^2 + V_{pm}^2} \tag{3.3}$$

in which V_m = coefficient of variation of F_{ym}/F_{yn} = 0.10 (Galambos 1978)

V_c = coefficient of variation of construction = 0.05 (Galambos 1978)

V_{pm} = coefficient of variation of the prediction method (obtained from comparison of predicted strengths with test results)

3.4 Effect of Varying λ

The first of six areas investigated is the effect of varying λ, the variable used in the linear approximation of the von Mises yield function. The effect of varying λ is investigated to establish a value that yields the most accurate predictions of maximum shear capacity by Methods I and III. Two values for λ are considered, 1.207 and 1.414. Donahey and Darwin (1986) used λ =

1.207 which represents the best uniform approximation of the von Mises yield function. This study uses λ = 1.414, which represents the practical upper limit for a linear approximation (Fig 2.6). The maximum shear capacities, and the predicted nominal shear and moment capacities for steel and composite beams using λ = 1.414 are presented in Tables 3.5 - 3.10, 3.13 - 3.18 and 3.21. The maximum shear capacities, and the predicted nominal shear and moment capacities for steel and composite beams using λ = 1.207 are presented in Tables G.1 - G.9.

For the fifty steel beams, when λ = 1.414, the mean test/theory ratios are 1.158, 1.213, and 1.183 and the coefficients of variation are 0.134, 0.179, and 0.150 for Methods I, II, and III, respectively. The corresponding resistance factors for the three methods are 0.929, 0.916, and 0.929. Considering the test/theory means, Method I is the most accurate followed by Method III and Method II. The fact that Method III is more accurate, for the beams considered, than Method II might not be expected considering Method III is a simplification of Method II. However, Method III, with λ = 1.414, tends to give a better match with the test data because the von Mises yield function does not account for strain hardening, which appears in virtually all of the high shear tests. The higher values of shear strength obtained with Methods I and III with λ = 1.414 take advantage of this behavior. For the same steel beams, when λ = 1.207, the mean test/theory ratios are 1.232 and 1.281 and the coefficients of variation are 0.166 and 0.193 for Methods I and III, respectively. The corresponding resistance factors for Methods I and III are 0.947 and 0.949. Method II is not influenced by λ. Considering test/theory means, coefficients of variation, and resistance factors, Method I is the most accurate followed by Method II and Method III, when λ = 1.207. In general, for the steel beams, using λ = 1.414 for Methods I and III produces lower test/theory ratios, lower coefficients of variation, and lower resistance factors. In all cases, resistance factors are higher than 0.90.

For the thirty-five composite beams, when $\lambda = 1.414$, the mean test/theory ratios are 1.024, 1.065, and 1.039 and the coefficients of variation are 0.084, 0.088, and 0.092 for Methods I, II, and III, respectively. The corresponding resistance factors for the three methods are 0.870, 0.901, and 0.876. Considering the test/theory means, coefficients of variation, and resistance factors, Method I is the most accurate followed by Methods III and II. For the same composite beams, when $\lambda = 1.207$, the mean test/theory ratios are 1.060 and 1.083 and the coefficients of variation are 0.079 and 0.086 for Methods I and III, respectively. The corresponding resistance factors for Methods I and III are 0.905 and 0.918. Method II is not influenced by λ. Considering test/theory means, coefficients of variation, and resistance factors, Method I is the most accurate followed by Method II and Method III, when $\lambda = 1.207$. In general, for the composite beams, using $\lambda = 1.414$ produces lower test/theory ratios, slightly higher coefficients of variation, and lower resistance factors. In all cases, resistance factors are higher than 0.85. Using $\lambda = 1.414$ for both steel and composite beams produces more accurate predictions of nominal capacity.

3.5 Effect of Reducing Tee Depth for Reinforcement

The effect of reducing the depth of a tee when reinforcement is present for Methods II and III is investigated to establish its significance with test data. Results obtained using Method III with no adjustment in the tee for reinforcement are compared with results obtained using Method III with an adjustment in the tee for reinforcement. The effect of reducing the depth of a tee in the calculation of υ in Eq. 2.44 is summarized in Table 3.22 for twenty-one reinforced steel beams and three reinforced composite beams. Reducing the tee depth for reinforcement does reduce the predicted maximum shear capacity and produces slightly more conservative nominal capacities for those beams affected. The overall test/theory ratio mean for the steel beams increases from 1.141 to 1.148, the coefficient of variation does not change and the resistance factor increases from 0.929 to 0.935 when the stub is reduced proportionally by the reinforcement present. The test/theory ratio for the single reinforced composite beam affected (CHO-6) increases from 1.112 to 1.118 when the stub is reduced. The other two beams have very little shear (CHO-7) or no shear (WJE-1) and are thus not affected. Reducing the tee depth by an amount proportional to the reinforcement present does not have a large affect on many other beams because the reinforcement contributes to shear capacity in excess of the maximum permitted by Section D.1.2. This restriction serves to maintain similiar conservatism available with Method I for tees with significant quantities of reinforcement.

3.6 Effect of Limiting P_{ch} by the Net Top Tee Steel

The effect of limiting P_{ch}, the normal force in the concrete slab at the high-moment end of the opening, by the normal force in the net steel in the top tee when $V_{mt} < V_{pt}$ for Methods II and III was investigated to establish if the limitation could be applied accurately and consistently with all three design methods for predicting maximum shear capacity presented in Section 2.0. The basis of comparison is the results obtained from Methods II and III with $\lambda = 1.414$ and P_{ch} not limited to the normal force in the net steel when $V_{mt} < V_{pt}$. Donahey and Darwin (1986) did not limit P_{ch} when $V_{mt} < V_{pt}$ for Methods II and III because this was thought to be unconservative and inconsistent with the assumptions made in the derivation of Methods II and III.

The results of limiting P_{ch} by the net steel when $V_{mt} < V_{pt}$ and $\lambda = 1.414$ are summarized for Method III in Table 3.23. For the D-series beams, the test/theory mean is unchanged at 0.974, the coefficient of variation increases from 0.060 to 0.067 and the resistance factor decreases from 0.845 to 0.841 when the limitation is applied to P_{ch}. For the R-series beams, the test/theory mean decreases from 1.065 to 1.050, the coefficient of variation decreases from 0.087 to 0.057, and the resistance factor increases from 0.902 to 0.913. For the C, G and CHO-series beams the test/theory mean decreases from 1.121 to 1.116, the coefficient of variation increases from 0.076 to 0.080, and the resistance factor decreases from 0.960 to 0.952. For the CHO-series beams (reinforced) the test/theory mean, the coefficient of variation and the resistance factor do not change. For the composite beams as a group, the test/theory mean decrease from 1.048 to 1.043, the coefficient of variation decreases from 0.095 to 0.091 and the resistance factor is unchanged at 0.880. For the thirty-five composite beams considered with $\lambda = 1.414$, the limitation on P_{ch} yields test/theory means closer to 1.000 and smaller coefficients of variation, though the differences are small.

3.7 Effect of Limiting P_r by Weld Strength

The effect of limiting the normal force in the reinforcement by the weld strength in determining the maximum shear capacity is checked to establish its significance on the prediction of maximum shear capacity for reinforced beams. The results of this investigation are summarized in Table 3.24. P_r in nine beams of the twenty-four reinforced beams was affected by the limitation. Of these nine beams, the maximum shear capacity of only one, CHO-6, was influenced. No change was seen in the maximum shear capacity for the other eight beams because the maximum shear capacity was limited by the plastic shear capacity of the tee even after applying the limitation.

3.8 Effect of Flanges

Because Methods II and III ignore the contribution of the flanges to the secondary bending moments, it is possible, for beams with large A_f / A_w ratios, that these two methods could significantly underpredict the maximum shear capacity when compared to Method I. Fig. 3.86 (refer to Table 3.25 for selected members and other study parameters) illustrates that, as the A_f / A_w

ratio increases, the difference between Methods I and III also increases and can be very significant. Within the typical range of A_f/A_w, 0.40 to 0.80, the difference between the two methods is never larger than 5% of V_p. For A_f/A_w ratios larger than 0.80, for sections typically used as beams, the difference between the two methods is as high as 16% (for a W12x58). A larger difference between the two methods occurs for a W14x109, but this section is not typically used as a beam. The effect of ignoring the contribution of the flanges to the secondary bending moments for sections typically used as beams with moderate flange areas is not significant. However, unnecessarily conservative predictions of shear capacity can result for some beam sections using Methods II or III, if the A_f/A_w ratio exceeds 0.80.

3.9 Effect of Limiting M_m by M_p

The effect of limiting the maximum moment capacity by the plastic moment capacity of the unperforated section is summarized in Table 3.26. All but two of the twenty-one reinforced steel beams are affected by the limitation. As a group, the test/theory ratio mean increased from 1.133 to 1.148, the coefficient of variation dropped from 0.128 to 0.122, and the resistance factor increased from 0.916 to 0.935. Insuring that $M_m \leq M_p$ provides slightly more conservative predictions of strength than when M_m is not limited to M_p.

3.10 Redwood Design Methods

For the purpose of comparison with the current work, nominal shear and moment capacities are obtained for all of the steel and composite beams considered in the report using applicable methods developed by Redwood and his coworkers. Maximum capacities are calculated for the steel beams using procedures proposed by Redwood and Shrivastava (1980) and are given in Table 3.11 for beams included in the analysis and in Table E.6 for beams not used in the analysis. Equations proposed by Redwood and Shrivastava (1980) are modified in Section 2.6 to account for the individual yield strengths of the flanges, web and reinforcement. Tables 3.11 and E.6 contain intermediate values, defined in the respective table and the respective reference, used to calculate the maximum shear capacities. Maximum shear capacities are calculated for thirteen composite beams with ribbed-slabs tested by Donahey and Darwin (1986) (D-series) using procedures presented by Redwood and Poumbouras (1984) which are given in Table 3.12. Capacities for nine composite beams with ribbed slabs tested by Redwood and Poumbouras (1984) (R-series) are taken from published values. The capacities for the remaining unreinforced composite beams with solid slabs are taken from values published by Redwood and Cho (1986). The predicted nominal shear and moment capacities for the steel and composite beams are presented in Tables 3.19 and 3.20, respectively. Capacities were not calculated or provided for beams CHO-6, CHO-7, and WJE-1 because no Redwood method has been published which accounts for reinforcement in composite beams. Several of the calculated capacities for composite beams do not agree with capacities published by Redwood. These discrepencies may be due to the way in which the shear connector capacities are calculated (see Donahey and Darwin (1986)).

Two moment-shear interaction procedures have been proposed by Redwood and Shrivastava (1980). Both require the calculation of an intermediate value for the moment, M_v, at which interaction with shear begins to have an influence on the moment capacity, M_m. The first interaction diagram is composed of two straight lines connecting the maximum shear capacity, V_m, to M_v, and M_v to the maximum moment capacity, M_m (see Fig. 3.87). This method is referred to as Redwood(L). The second interaction procedure used by Redwood uses a straight line to connect V_m to M_v, and a circular arc to connect M_v to M_m (see Fig. 3.88). This procedure is referred to as Redwood(C). Both interaction procedures are used for the steel beams, while only Redwood(C) is used for the composite beams.

3.11 Comparison of Design Methods with Test Results

In this section the nominal shear and moment capacities obtained using the design methods discussed in Section 2.0, using $\lambda = 1.414$, and those by Redwood and Shrivastava (1980), Redwood and Poumbouras (1984) and Redwood and Cho (1986) are compared with test results. The analysis includes fifty steel beams and thirty-five composite beams. A tabular summary of results for both steel and composite beams is given in Table 3.21. Individual moment-shear interaction curves and the respective beam test values are given in Figs. 3.1 - 3.85 for the steel and composite beams. Graphical comparisons of the predicted strengths using Method III and the actual test values for the steel and composite beams are given in Figs. 3.89 and 3.90, respectively.

3.11.1 Steel Beams

Nineteen of the fifty steel beams are unreinforced with rectangular openings, ten are unreinforced with circular openings, and twenty-one are reinforced with rectangular openings. The beams with unreinforced rectangular openings have test/theory means of 1.213, 1.302, 1.250, 1.265, and 1.391 with coefficients of variation of 0.142, 0.211, 0.167, 0.191 and 0.195 for Methods I, II, III, Redwood(C), and Redwood(L), respectively. The corresponding resistance factors are 0.963, 0.939, 0.960, 0.939, and 1.027. The beams with unreinforced circular openings have test/theory means of 1.088, 1.145, 1.127, 1.111, and 1.264 with coefficients of variation of 0.119, 0.154, 0.142, 0.140 and 0.131 for Methods I, II, III, Redwood(C), and Redwood(L), respectively. The corresponding resistance factors are 0.889, 0.895, 0.895, 0.885, and 1.018. The group of beams with reinforced rectangular openings have test/theory means of 1.143, 1.166, 1.148, 1.142, and 1.362 with coefficients of variation of 0.121, 0.125, 0.122, 0.151 and 0.195 for Methods I, II, III, Redwood(C), and Redwood(L), respectively. The corresponding resistance factors are 0.932, 0.946, 0.935, 0.896, and 1.006. Overall, the fifty steel beams have test/theory means of 1.158, 1.213, 1.183, 1.183, and 1.353 with coefficients of variation of 0.134, 0.179, 0.150, 0.174 and 0.185 for Methods I, II, III, Redwood(C), and Redwood(L), respectively. The corresponding resistance factors are 0.929, 0.916, 0.930, 0.900, and 1.013. Generally, Method I provides test/theory means closest to 1.000, followed by Method III, Redwood(C), Method II, and Redwood(L). Method I gives the smallest coefficients of variation, followed by Method III, Redwood(C), Method II, and Redwood(L). Redwood(C) gives the lowest resistance factor followed by Method II, Method I, Method III, and Redwood(L).

3.11.2 Composite Beams

Of the thirty-five composite beams, twenty-one have ribbed slabs and unreinforced rectangular openings, eleven have solid slabs and unreinforced rectangular openings, one has a ribbed slab and a reinforced rectangular opening, and two have solid slabs and reinforced rectangular openings. Methods I, II and III are applied to all thirty-five beams. The Redwood(C) method (Redwood and Poumbouras (1984) and Redwood and Cho (1986)) is applied to the thirty-two beams without reinforcement. Redwood(L) is not applicable. The group of beams with ribbed slabs and unreinforced rectangular openings have test/theory means of 0.995, 1.037, 1.006, and 1.090 with coefficients of variation of 0.071, 0.069, 0.072, and 0.121 for Methods I, II, III, and Redwood(C), respectively. The corresponding resistance factors are 0.856, 0.893, 0.864, and 0.889. The beams with solid slabs and unreinforced rectangular openings have test/theory means of 1.092, 1.141, 1.116, and 1.207 with coefficients of variation of 0.066, 0.075, 0.080, and 0.124

for Methods I, II, III, and Redwood(C), respectively. The corresponding resistance factors are 0.943, 0.978, 0.952, and 0.981. The beams with ribbed and solid slabs with reinforced rectangular openings have test/theory means of 0.978, 0.985, and 0.983 with coefficients of variation of 0.110, 0.122, and 0.119 for Methods I, II, and III, respectively. The corresponding resistance factors are 0.808, 0.802, and 0.803. These low values are due to the fact that only three beams are used for this calculation, and the results are dominated by a single member (WJE-1) for which failure was controlled by shear connector capacity (Wiss et al. 1984). Thus these values are not considered to be representative of what is expected in practice.

The thirty-five composite beams [thirty-two for Redwood(C)] have test/theory means of 1.024, 1.065, 1.039, and 1.131, with coefficients of variation of 0.084, 0.088, 0.092, and 0.128 for Methods I, II, III, and Redwood(C), respectively. The corresponding resistance factors are 0.870, 0.901, 0.875, and 0.895. Overall, Method I provides test/theory means closest to 1.000, followed by Method III, Method II, and Redwood(C). Method I provides the smallest coefficients of variation, followed by Method II, Method III, and Redwood(C). Method I yields the lowest resistance factor followed by Method III, Redwood(C), and Method II.

3.11.3 Recommendations

Method III, the simplest of the design methods for determining maximum shear capacity, coupled with the cubic moment-shear interaction procedure proposed by Donahey and Darwin (1986) is recommended for design. Resistance factors of 0.90 and 0.85, applied to shear and bending, are recommended for the design of steel and composite beams, respectively. As illustrated in Figs. 3.89 and 3.90, none of the beams used for the comparisons had a strength below the product of the resistance factor and the predicted strength.

SECTION 4.0 SUMMARY AND CONCLUSIONS

4.1 Summary

Three design methods, originally developed by Donahey and Darwin (1986), for determining the maximum shear capacity of composite beams with unreinforced web openings are extended to include steel and composite beams with or without reinforcement at the opening. The three design methods incorporate simplifying assumptions that permit closed-form solutions for maximum shear capacity. The first method assumes that the neutral axes for secondary bending lie in the flanges of the top and bottom tees and defines the interaction of shear and normal stresses by a linear approximation of the von Mises yield function. The second method ignores the contribution of the flanges to secondary bending moments and employs the von Mises yield function to define the interaction of shear and normal stresses. The third method ignores the contribution of the flanges to secondary bending moments and defines the interaction between shear and normal stresses with a linear approximation of the von Mises yield function. Simplified design expressions for the maximum moment capacity of steel and composite beams with web openings are presented. Six refinements of the design methods are investigated to determine their significance in predicting member strengths. Simplified design expressions developed by Darwin (1990) for determining the maximum moment capacity of steel and composite beams at web openings are summarized. The accuracy and ease of application of the design methods presented in this report (Methods I, II, and III) and applicable procedures proposed by Redwood and Shrivastava (1980), Redwood and Poumbouras (1984), and Redwood and Cho (1986) are compared with experimental results of fifty steel beams and thirty-five composite beams. Resistance factors are calculated for use in LRFD of structural steel buildings.

4.2 Conclusions

Based on the work presented in this report, the following conclusions can be made:

1. For slender tees, the predictions of normal stress made by the linear approximation of the von Mises yield function, when $\lambda = 1.414$, can be as much as 41% higher than the normal stress predicted by the von Mises yield function. Considering practical design limitations on opening sizes (Appendix D), the normal stress is overpredicted by 26.3%. This translates into a maximum shear capacity that is overpredicted by 3.5% of the *plastic* shear capacity of a tee when considering the design limitations presented in Appendix D.

2. For stocky tees, the linear approximation of the von Mises yield function can overpredict the shear stress in a tee by as much as 9.7% when $\lambda = 1.414$ and $\upsilon = 0.717$. This translates into a difference in predicted maximum shear capacity of 9.0% of the plastic shear capacity of a tee. While this difference is signicant, such low values of υ are very unlikely to occur in practice.

3. Using $\lambda = 1.414$ with Methods I and III, instead of 1.207, for both steel and composite beams produces more accurate predictions of nominal capacity and more consistent resistance factors for different opening and slab types thus eliminating unnecessary conservatism from potential designs.

4. Unnecessarily conservative predictions of shear capacity can result for some beam sections using Methods II or III, if the ratio of the area of the flange to the area of the web, A_f/A_w, exceeds 0.80.

5. The effect of reducing the tee depth by an amount proportional to the reinforcement present, when calculating the maximum shear capacity at an opening, did not have a large effect in many of the reinforced beams considered, because the reinforcement contributed to shear capacity in excess of the maximum permitted by Section D.1.2. The procedure, however, serves

to maintain conservatism similiar to that obtained with Method I for tees with significant quantities of reinforcement.

6. For the thirty-five composite beams considered, with $\lambda = 1.414$, consistently limiting P_{ch} by the axial yield capacity of the top tee steel gives test/theory means closer to 1.0 and smaller coefficients of variation, than when P_{ch} is not limited by the net top tee steel.

7. Insuring that $M_m \leq M_p$ provides slightly more conservative predictions of moment capacity than when M_m is not limited to M_p.

8. Insuring that the normal force in the reinforcement is less than the capacity of the corresponding weld provides predictions of shear capacity that are more conservative than when the normal force in the reinforcement is not limited by the weld capacity.

9. For the steel beams, Method I provides test/theory means closest to 1.0, followed by Method III, Redwood(C), Method II, and Redwood(L). Method I gives the smallest coefficients of variation, followed by Method III, Redwood(C), Method II, and Redwood(L). For the group of steel beams, Redwood(C) yields the lowest resistance factor followed by Method II, Method I, Method III, and Redwood(L). For the composite beams, Method I provides test/theory means closest to 1.0, followed by Method II, Method III, and Redwood(C). Method I yields the smallest coefficients of variation, followed by Method II, Method III, and Redwood(C). Method I gives the lowest resistance factor followed by Method III, Redwood(C), and Method II.

10. Methods proposed by Redwood and Shrivastava (1980), Redwood and Poumbouras (1986) and Redwood and Cho (1986) for determining shear and moment capacity for steel beams, composite beams with ribbed slabs, and composite beams with solid slabs, respectively, are generally more complex than the methods in this report and do not offer any additional accuracy.

11. Method III coupled with the moment-shear interaction procedures proposed by Donahey and Darwin (1986) is easily applied and provides strength predictions that are in excellent agreement with test data.

12. Resistance factors for shear and bending of 0.90 and 0.85 are appropriate for steel and composite beams, respectively.

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Table 3.0 References Corresponding to Beam Designations

Designation	Reference
Steel Beams	
B	Bower (1968)
CL	Clawson and Darwin (1980)
CR	Congdon and Redwood (1970)
CS	Cooper and Snell (1972)
CSK	Cooper, Snell, and Knostman (1972)
DO	Doughterty (1980)
KKS	Kim (1980)
RBD	Redwood, Baranda, and Daly (1978)
RL	Lupien and Redwood (1978)
RM	Redwood and McCutcheon (1968)
Composite Beams	
C	Clawson and Darwin (1980)
D	Donahey and Darwin (1986)
CHO	Cho (1982)
G	Granade (1968)
R	Redwood and Poumbouras (1983)
WJE	Wiss, Janney, and Elstner (1984)

Table 3.1 Material and Section Properties for Steel Beams

(in inches unless noted)

	Web			Opening			Reinforcement				Top Tee				Bottom Tee			
Test[1]	d	t_w	F_{yw} (ksi)	D_o	h_o	a_o	b_r	t_r	y_r	F_{yr} (ksi)	s	b_f	t_f	F_{yf} (ksi)	s	b_f	t_f	F_{yf} (ksi)
RBD-C1	16.970	0.276	46.500	4.670	4.203	2.101					6.384	7.210	0.423	43.100	6.384	7.210	0.423	43.100
RM-1A	8.125	0.246	51.400	4.500	4.050	2.025					2.038	5.250	0.322	45.500	2.038	5.250	0.322	45.500
RM-1B	8.125	0.242	51.600	4.500	4.050	6.525					2.030	5.250	0.317	44.900	2.038	5.250	0.317	44.900
RM-2A	8.125	0.249	54.400	4.500	4.050	2.025					2.038	5.250	0.324	50.900	2.038	5.250	0.324	50.900
RM-2B	8.040	0.233	45.300	4.500	4.050	6.525					1.995	5.250	0.296	46.400	1.995	5.250	0.296	46.400
RM-2C	8.040	0.234	44.200	4.500	4.050	2.025					1.995	5.220	0.296	43.000	1.995	5.220	0.296	43.000
RM-3A	8.125	0.245	52.200	4.500	4.050	2.025					2.038	5.250	0.323	45.000	2.038	5.250	0.323	45.000
RM-4A	8.125	0.245	52.100	4.500	4.050	2.025					2.038	5.250	0.322	45.700	2.038	5.250	0.322	45.700
RM-4B	8.125	0.244	51.000	4.500	4.050	6.525					2.038	5.250	0.322	43.000	2.038	5.250	0.322	43.000
RM-4C	8.125	0.246	58.000	4.500	4.050	2.025					2.038	5.250	0.322	42.800	2.038	5.250	0.322	42.800
CR-1A	9.890	0.244	57.000		5.500	8.750	2.720	0.253	0.326	37.000	2.195	5.780	0.324	43.100	2.200	5.780	0.324	43.100
CR-2A	14.130	0.326	41.300		7.000	10.460	2.813	0.381	0.383	43.700	3.565	6.770	0.496	38.800	3.565	6.770	0.496	38.800
CR-2B	14.130	0.326	41.300		7.000	10.460	2.813	0.381	0.383	43.700	3.565	6.770	0.496	38.800	3.565	6.770	0.496	38.800
CR-2C	14.220	0.305	55.800		7.000	10.450	2.720	0.368	0.267	39.800	3.610	6.680	0.490	44.600	3.610	6.680	0.490	44.600
CR-2D	14.220	0.305	55.800		7.000	10.450	2.720	0.368	0.267	39.800	3.610	6.680	0.490	44.600	3.610	6.680	0.490	44.600
CR-3A	14.130	0.326	41.300		7.000	10.460	5.313	0.381	0.383	43.700	3.565	6.770	0.496	38.800	3.565	6.770	0.496	38.800
CR-3B	14.220	0.305	55.800		7.000	10.450	5.230	0.361	0.329	33.100	3.610	6.680	0.490	44.600	3.610	6.680	0.490	44.600
CR-4A	14.220	0.305	55.800		7.000	13.980	2.720	0.368	0.340	39.800	3.610	6.680	0.490	44.600	3.610	6.680	0.490	44.600
CR-4B	14.220	0.305	55.800		7.000	13.980	2.720	0.368	0.340	39.800	3.610	6.680	0.490	44.600	3.610	6.680	0.490	44.600
CR-5A	14.220	0.305	55.800		9.000	13.440	3.770	0.371	0.305	44.400	2.610	6.680	0.490	44.600	2.610	6.680	0.490	44.600
CR-7B	14.270	0.317	47.700		7.030	10.570	2.817	0.378	0.338	39.900	3.620	6.700	0.486	40.500	3.620	6.700	0.486	40.500
CR-7D	14.270	0.317	47.700		7.030	10.570	2.817	0.378	0.338	39.900	3.620	6.700	0.486	40.500	3.620	6.700	0.486	40.500
CSK-2	16.130	0.345	46.070		6.000	9.000	4.340	0.250	0.375	43.420	3.065	7.035	0.565	45.430	7.065	7.035	0.565	44.890
CSK-5	16.010	0.305	44.710		6.000	12.000	4.305	0.250	0.375	42.720	3.005	6.995	0.505	42.630	7.005	6.995	0.505	43.940
CSK-6	16.010	0.305	44.710		8.000	16.000	4.305	0.250	0.375	35.520	6.005	6.995	0.505	42.630	2.005	6.995	0.505	43.940
CSK-7	16.010	0.305	44.710		8.000	12.000	4.305	0.250	0.375	35.520	6.005	6.995	0.505	42.630	2.005	6.995	0.505	43.940
CS-1	12.060	0.335	37.500		6.000	9.000	4.340	0.375	0.438	34.700	3.030	8.045	0.575	33.000	3.030	8.045	0.575	33.000
CS-2	12.060	0.335	36.100		6.000	9.000	4.340	0.375	0.438	35.000	3.030	8.045	0.575	33.700	3.030	8.045	0.575	33.700
CS-3	12.060	0.335	37.400		6.000	9.000	3.340	0.500	0.625	33.100	3.030	8.045	0.575	32.200	3.030	8.045	0.575	32.200
RL-5	16.330	0.274	47.900		7.160	17.810	2.188	0.249	0.000	38.130	4.585	6.980	0.416	41.240	4.585	6.980	0.416	41.240
RL-6	16.350	0.266	49.960		10.720	26.760	2.766	0.372	0.000	44.750	2.815	6.940	0.423	40.340	2.815	6.940	0.423	40.340
B-1	15.940	0.314	44.000		7.440	9.000					4.250	7.165	0.420	36.200	4.250	7.165	0.420	36.200
B-2	15.810	0.300	40.200		7.112	9.000					4.349	7.125	0.420	35.600	4.349	7.125	0.420	35.600
B-3	15.880	0.310	37.700		7.320	9.000					4.280	7.094	0.425	33.700	4.280	7.094	0.425	33.700
B-4	15.800	0.313	43.700		7.160	7.300					4.320	7.094	0.419	35.000	4.320	7.094	0.419	35.000
CL-4B	17.875	0.343	52.000		10.813	21.625					3.000	7.500	0.485	46.390	3.060	7.500	0.485	44.890
CR-6A	14.220	0.305	55.800		7.000	10.450					3.610	6.680	0.490	44.600	3.610	6.680	0.490	44.600
CSK-1	16.130	0.345	44.790		6.000	9.000					3.065	7.035	0.565	45.980	7.065	7.035	0.565	46.060
DO-1	7.920	0.232	52.210		2.362	7.087					2.779	5.275	0.315	45.690	2.779	5.275	0.315	45.690
DO-2	7.920	0.232	52.210		4.724	9.449					1.598	5.275	0.315	45.690	1.600	5.275	0.315	45.690

Table 3.1 (continued)

Test[1]	Web: d	Web: t_w	Web: F_{yw} (ksi)	Opening: D_o	Opening: h_o	Opening: a_o	Reinforcement: b_r	Reinforcement: t_r	Reinforcement: y_r	Reinforcement: F_{yr} (ksi)	Top Tee: s	Top Tee: b_f	Top Tee: t_f	Top Tee: F_{yf} (ksi)	Bottom Tee: s	Bottom Tee: b_f	Bottom Tee: t_f	Bottom Tee: F_{yf} (ksi)
DO-3	7.920	0.232	52.210		2.362	7.087					1.598	5.275	0.315	45.690	3.960	5.275	0.315	45.690
DO-4	7.822	0.256	51.490		3.543	9.449					1.550	5.198	0.324	47.140	2.729	5.198	0.324	47.140
DO-5	7.920	0.232	52.210		4.724	4.724					1.598	5.275	0.315	45.690	1.598	5.275	0.315	45.690
RBD-R1B	16.980	0.273	46.500		2.300	6.900					7.340	7.250	0.426	43.400	7.340	7.250	0.426	43.400
RBD-R2	16.970	0.276	46.500		5.293	11.700					5.835	7.210	0.423	43.100	5.835	7.210	0.423	43.100
RM-11H	8.125	0.295	56.100		4.500	6.750					1.813	5.250	0.321	45.100	1.813	5.250	0.321	45.100
RM-21H	8.000	0.230	48.600		4.500	6.750					1.750	5.250	0.292	43.900	1.750	5.250	0.292	43.900
RM-2F	8.063	0.238	45.600		4.500	6.750					1.782	5.250	0.300	43.500	1.782	5.250	0.300	43.500
RM-4F	8.125	0.250	54.100		4.500	6.750					1.813	5.250	0.322	46.100	1.813	5.250	0.322	46.100
RM-4H	8.030	0.230	50.600		4.500	6.750					1.765	5.250	0.292	44.900	1.765	5.250	0.292	44.900

Notes:

1. Refer to Table 3.0 for key to beam designations

Table 3.2 Material and Section Properties for Composite Beams

(in inches unless noted)

(a) STEEL SECTION

	Web			Opening		Reinforcement				Top Tee				Bottom Tee			
Test(1)	d	t_w	F_{yw} (ksi)	h_o	a_o	b_r	t_r	y_r	F_{yr} (ksi)	s	b_f	t_f	F_{yf} (ksi)	s	b_f	t_f	F_{yf} (ksi)
D-1	20.630	0.358	55.400	12.380	24.750					4.178	6.510	0.440	54.600	4.101	6.500	0.430	52.300
D-2	20.630	0.357	53.100	12.380	24.750					4.094	6.500	0.427	52.300	4.094	6.510	0.448	51.200
D-3	20.630	0.358	52.500	12.380	24.750					4.105	6.570	0.423	52.600	4.097	6.560	0.435	51.700
D-5A	20.630	0.358	52.700	12.380	24.750					4.168	6.510	0.440	53.100	4.110	6.500	0.430	54.700
D-5B	20.630	0.358	52.700	14.390	24.750					4.110	6.570	0.440	53.100	2.123	6.450	0.430	54.700
D-6A	20.630	0.357	52.700	12.380	24.750					4.120	6.580	0.440	53.600	4.115	6.570	0.432	52.700
D-6B	20.630	0.357	52.700	12.380	24.750					4.120	6.580	0.440	53.600	4.115	6.570	0.432	52.700
D-7A	20.630	0.360	41.200	12.380	24.750					4.025	6.660	0.409	40.600	4.150	6.590	0.412	41.100
D-7B	20.630	0.360	41.200	12.380	24.750					4.075	6.660	0.409	40.600	4.188	6.590	0.412	41.100
D-8A	10.130	0.231	50.800	5.950	11.820					2.096	3.980	0.268	47.600	2.090	4.020	0.280	47.700
D-9A	20.630	0.365	41.200	14.750	24.750					2.960	6.670	0.425	41.100	2.960	6.610	0.429	40.600
D-9B	20.630	0.369	41.200	14.750	14.750					3.075	6.670	0.427	41.100	2.812	6.610	0.427	40.600
R-0	9.980	0.228	56.100	5.910	11.810					2.039	4.020	0.256	50.600	2.039	4.020	0.256	50.600
R-1	14.010	0.293	45.100	8.390	16.770					2.810	6.870	0.448	40.100	2.810	6.870	0.448	40.100
R-2	14.050	0.309	47.300	8.390	16.770					2.830	6.740	0.441	43.800	2.830	6.740	0.441	43.800
R-3	14.030	0.313	47.200	8.390	16.750					2.820	6.740	0.444	42.200	2.820	6.740	0.444	42.200
R-4	14.040	0.313	48.100	8.390	16.750					2.835	6.860	0.436	43.700	2.835	6.860	0.436	43.700
R-5	14.010	0.293	45.100	8.390	16.770					1.410	6.870	0.448	40.100	4.210	6.870	0.448	40.100
R-6	14.050	0.305	47.200	8.390	16.750					2.835	6.750	0.437	43.700	2.835	6.750	0.437	43.700
R-7	14.050	0.305	44.000	8.390	16.750					2.835	6.750	0.437	43.700	2.835	6.750	0.437	44.100
R-8	13.980	0.292	44.000	8.390	16.750					2.795	6.690	0.450	44.100	2.795	6.690	0.450	44.100
C-1	14.000	0.287	38.500	8.000	16.000					3.003	6.750	0.453	39.400	3.003	6.750	0.453	40.400
C-2	17.880	0.356	42.400	10.810	21.630					3.475	7.500	0.475	39.300	3.770	7.500	0.520	39.900
C-3	17.880	0.356	42.400	10.810	21.630					3.605	7.500	0.475	39.300	3.650	7.500	0.520	39.300
C-4	17.880	0.343	52.000	10.810	21.630					3.485	7.500	0.485	46.400	3.555	7.500	0.495	44.900
C-5	18.130	0.380	44.200	10.810	21.630					3.683	6.000	0.623	43.900	3.745	6.000	0.615	45.100
C-6	14.000	0.296	49.800	8.000	16.000					2.855	6.690	0.475	42.900	2.803	6.690	0.423	43.500
G-1	8.000	0.285	47.900	4.800	7.200					1.630	6.540	0.463	43.800	1.630	6.540	0.463	43.800
G-2	8.000	0.285	47.900	4.800	7.200					1.630	6.540	0.463	43.800	1.630	6.540	0.463	43.800
CHO-3	7.870	0.236	50.800	4.720	7.280					1.500	5.910	0.354	44.100	1.500	5.910	0.354	43.400
CHO-4	11.810	0.256	64.600	7.050	10.630					2.360	5.910	0.354	54.000	2.400	5.910	0.354	50.700
CHO-5	11.810	0.256	64.600	7.090	10.630					2.400	5.910	0.354	54.000	2.320	5.910	0.354	50.700
CHO-6	7.870	0.236	50.800	4.610	7.130	4.170	0.236	0.374	50.800	1.540	5.910	0.354	44.100	1.500	5.910	0.354	43.400
CHO-7	11.810	0.256	64.600	7.090	14.370	4.170	0.236	0.374	50.800	2.360	5.910	0.354	54.000	2.360	5.910	0.354	50.700
WJE-1	20.830	0.380	37.000	15.000	39.000	5.000	0.500	0.000	37.000	2.938	6.530	0.535	37.000	2.938	6.530	0.535	37.000
D-8B	10.130	0.231	50.800	6.380	18.630					2.025	3.980	0.268	47.600	1.725	4.020	0.280	47.700

Table 3.2 (continued)

(b) SLAB

Test[1]	Type	f'_c (psi)	b_e	t'_s	t_s	t_e	w_{rmx}	w_{rmn}	w_r	H_r
D-1	TRIB	4470	48.000	2.000	5.000	2.000	7.000	5.000	6.000	3.000
D-2	TRIB	4850	48.000	2.000	5.000	2.000	7.000	5.000	6.000	3.000
D-3	TRIB	5400	48.000	2.000	5.000	2.000	7.000	5.000	6.000	3.000
D-5A	TRIB	4740	48.000	2.000	5.000	2.000	7.000	5.000	6.000	3.000
D-5B	TRIB	5090	48.000	2.000	5.000	2.000	7.000	5.000	6.000	3.000
D-6A	TRIB	4020	48.000	2.000	5.000	2.000	7.000	5.000	6.000	3.000
D-6B	LRIB	4300	48.000	2.000	5.000	3.500	7.000	5.000	6.000	3.000
D-7A	LRIB	4190	48.000	2.000	5.000	3.500	7.000	5.000	6.000	3.000
D-7B	LRIB	4300	48.000	2.000	5.000	3.500	7.000	5.000	6.000	3.000
D-8A	TRIB	3940	48.000	2.500	5.500	2.500	7.000	5.000	6.000	3.000
D-9A	TRIB	4170	36.000	4.000	7.000	4.000	7.000	5.000	6.000	3.000
D-9B	TRIB	4360	48.000	4.000	7.000	4.000	7.000	5.000	6.000	3.000
R-0	TRIB	3830	48.000	2.600	5.600	2.600	6.460	5.550	6.005	3.000
R-1	TRIB	3190	39.400	2.600	5.600	2.600	6.460	5.550	6.005	3.000
R-2	TRIB	2830	47.200	2.600	5.600	2.600	6.460	5.550	6.005	3.000
R-3	TRIB	4290	47.200	2.600	5.600	2.600	6.460	5.550	6.005	3.000
R-4	TRIB	3960	47.200	2.600	5.600	2.600	6.460	5.550	6.005	3.000
R-5	TRIB	3190	47.200	2.600	5.600	2.600	6.460	5.550	6.005	3.000
R-6	TRIB	2610	39.400	2.600	5.600	2.600	6.460	5.550	6.005	3.000
R-7	TRIB	2610	39.400	2.600	5.600	2.600	6.460	5.550	6.005	3.000
R-8	TRIB	2480	39.400	2.600	5.600	2.600	6.460	5.550	6.005	3.000
C-1	SOL	7000	39.400	4.000	4.000	4.000				
C-2	SOL	4200	48.000	4.000	4.000	4.000				
C-3	SOL	4930	48.000	4.000	4.000	4.000				
C-4	SOL	4460	48.000	4.000	4.000	4.000				
C-5	SOL	4680	45.000	4.000	4.000	4.000				
C-6	SOL	4020	48.000	4.000	4.000	4.000				
G-1	SOL	3970	45.000	3.600	3.600	3.600				
G-2	SOL	3990	24.000	3.600	3.600	3.600				
CHO-3	SOL	3270	24.000	5.300	5.300	5.300				
CHO-4	SOL	3040	21.600	5.400	5.400	5.400				
CHO-5	SOL	3270	23.800	5.300	5.300	5.300				
CHO-6	SOL	3270	23.800	5.300	5.300	5.300				
CHO-7	SOL	3170	23.800	5.300	5.300	5.300				
WJE-1	TRIB	4420	110.500	3.500	6.500	3.500	7.000	5.000	6.000	3.000
D-8B	TRIB	3940	48.000	2.500	5.500	2.500	7.000	5.000	6.000	3.000

Table 3.2 (continued)

(c) SHEAR CONNECTORS

Test[1]	H_s	N_1[2]	N_2[3]	N_o[4]	N_{r1}[5]	N_{r2}[6]	F_u (ksi)	Dia.	A_{sc} (in.^{2})	R_1[7]	R_2[8]	Q_n (k)	R_1*Q_n (k)	R_2*Q_n (k)	$A_{sc}*F_u$ (k)
D-1	4.500	10		4	2		67.900	0.750	0.442	0.601	0.000	28.83	17.33	0.00	30.00
D-2	4.500	10	12	4	2	4	67.900	0.750	0.442	0.601	0.491	30.65	18.42	15.04	30.00
D-3	4.500	20		4	2		67.900	0.750	0.442	0.601	0.000	33.22	19.97	0.00	30.00
D-5A	4.500	7		2	1		67.900	0.750	0.442	0.850	0.000	30.13	25.61	0.00	30.00
D-5B	4.500	16		4	2		67.900	0.750	0.442	0.601	0.000	31.78	19.10	0.00	30.00
D-6A	4.500	12		4	2		67.900	0.750	0.442	0.601	0.000	26.62	16.00	0.00	30.00
D-6B	4.500	20		8			67.900	0.750	0.442	0.491	0.000	28.00	13.74	0.00	30.00
D-7A	4.500	22		10			67.900	0.750	0.442	0.491	0.000	27.47	13.48	0.00	30.00
D-7B	4.500	10					67.900	0.750	0.442	0.491	0.000	27.47	13.48	0.00	30.00
D-8A	5.000	8		2	2		63.200	0.625	0.307	0.801	0.000	18.21	14.60	0.00	19.39
D-8B	4.500	6		2	2		67.900	0.625	0.307	0.601	0.000	18.23	10.95	0.00	20.85
D-9A	5.500	10		4	2		63.200	0.750	0.442	1.000	0.000	27.37	27.37	0.00	27.92
D-9B	5.500	8		2	2		68.800	0.750	0.442	1.000	0.000	28.30	28.30	0.00	30.39
R-0	4.840	4		1	1		68.800	0.750	0.442	1.000	0.000	25.68	25.68	0.00	30.39
R-1	4.840	4		1	1			0.750	0.442	1.000	0.000	22.39	22.39	0.00	0.00
R-2	4.840	18		2	2			0.750	0.442	0.738	0.000	20.46	15.10	0.00	0.00
R-3	4.840	22		4	2			0.750	0.442	0.738	0.000	27.96	20.63	0.00	0.00
R-4	4.840	5		0	1			0.750	0.442	1.000	0.000	26.33	26.33	0.00	0.00
R-5	4.840	4		1	1			0.750	0.442	1.000	0.000	22.39	22.39	0.00	0.00
R-6	4.840	4		0	2			0.750	0.442	1.000	0.000	22.39	22.39	0.00	0.00
R-7	4.840	8		4	2			0.750	0.442	0.738	0.000	19.26	14.21	0.00	0.00
R-8	4.840	8		4	2			0.750	0.442	0.738	0.000	18.53	13.68	0.00	0.00
C-1	3.000	14		4				0.750	0.442	1.000	0.000	40.36	40.36	0.00	0.00
C-2	3.000	16		2				0.750	0.442	1.000	0.000	27.51	27.51	0.00	0.00
C-3	3.000	16		2				0.750	0.442	1.000	0.000	31.03	31.03	0.00	0.00
C-4	3.000	10		4				0.750	0.442	1.000	0.000	28.78	28.78	0.00	0.00
C-5	3.000	16		4				0.750	0.442	1.000	0.000	29.84	29.84	0.00	0.00
C-6	3.000	10		4				0.750	0.442	1.000	0.000	26.62	26.62	0.00	0.00
G-1	2.500	10		2				0.625	0.307	1.000	0.000	18.32	18.32	0.00	0.00
G-2	2.500	16		2				0.625	0.307	1.000	0.000	18.39	18.39	0.00	0.00
CHO-3	3.940	12		4				0.500	0.196	1.000	0.000	10.14	10.14	0.00	0.00
CHO-4	3.940	18		4				0.500	0.196	1.000	0.000	9.60	9.60	0.00	0.00
CHO-5	3.940	20		4				0.500	0.196	1.000	0.000	10.14	10.14	0.00	0.00
CHO-6	3.940	12		4				0.500	0.196	1.000	0.000	10.14	10.14	0.00	0.00
CHO-7	3.940	20		4				0.500	0.196	1.000	0.000	9.90	9.90	0.00	0.00
WJE-1	4.500	18		6	1			0.750	0.442	0.850	0.000	28.59	24.30	0.00	0.00

Table 3.2 (continued)

Notes:

(1) refer to Table 3.0 for key of beam designations
(2) N_1 = number of studs/rib in first set(*) of ribs
(3) N_2 = number of studs/rib in second set(*) of ribs
(4) N_o = number of studs over the opening
(5) N_{r1} = number of ribs in first set(*)
(6) N_{r2} = number of ribs in second set(*)
(7) R_1 = reduction factor for first set of shear connectors
(8) R_2 = reduction factor for second set of shear connectors

(*) A set of ribs is a series of ribs with the same number of stubs per rib.

Table 3.3 Design Limitation Summary for Steel Beams

(a) Local Buckling of Compression Flange (D.1.1)

(b) Web Buckling (D.1.2)

Test[1]	$b_f/2t_f$	$<65/\sqrt{F_y}$	$p_o < 6.0$	h/t	$420/\sqrt{F_y}$	$520/\sqrt{F_y}$	a_o/h_o	$a_o/h_o(max)$
RBD-C1	8.52	9.90	2.10	58.42	61.59	76.26	0.45	3.00
RM-1A	8.15	9.64	3.77	30.41	58.58	72.53	0.45	3.00
RM-1B	8.28	9.70	4.77	30.95	58.47	72.39	1.45	3.00
RM-2A	8.10	9.11	3.77	30.03	56.94	70.50	0.45	3.00
RM-2B	8.87	9.54	4.81	31.97	62.40	77.26	1.45	3.00
RM-2C	8.82	9.91	3.81	31.83	63.17	78.22	0.45	3.00
RM-3A	8.13	9.69	3.77	30.53	58.13	71.97	0.45	3.00
RM-4A	8.15	9.62	3.77	30.53	58.19	72.04	0.45	3.00
RM-4B	8.15	9.91	4.77	30.66	58.81	72.81	1.45	3.00
RM-4C	8.15	9.94	3.77	30.41	55.15	68.28	0.45	3.00
CR-1A	8.92	9.90	4.93	37.88	55.63	68.88	1.59	3.00
CR-2A	6.82	10.44	4.47	40.30	65.35	80.91	1.49	3.00
CR-2B	6.82	10.44	4.47	40.30	65.35	80.91	1.49	3.00
CR-2C	6.82	9.73	4.45	43.41	56.23	69.61	1.49	3.00
CR-2D	6.82	9.73	4.45	43.41	56.23	69.61	1.49	3.00
CR-3A	6.82	10.44	4.47	40.30	65.35	80.91	1.49	3.00
CR-3B	6.82	9.73	4.45	43.41	56.23	69.61	1.49	3.00
CR-4A	6.82	9.73	4.95	43.41	56.23	69.61	2.00	3.00
CR-4B	6.82	9.73	4.95	43.41	56.23	69.61	2.00	3.00
CR-5A	6.82	9.73	5.29	43.41	56.23	69.61	1.49	3.00
CR-7B	6.89	10.21	4.46	41.95	60.81	75.29	1.50	3.00
CR-7D	6.89	10.21	4.46	41.95	60.81	75.29	1.50	3.00
CSK-2	6.23	9.64	3.73	43.48	61.88	76.61	1.50	3.00
CSK-5	6.93	9.96	4.25	49.18	62.81	77.77	2.00	3.00
CSK-6	6.93	9.96	5.00	49.18	62.81	77.77	2.00	3.00
CSK-7	6.93	9.96	4.50	49.18	62.81	77.77	1.50	3.00
CS-1	7.00	11.32	4.49	32.57	68.59	84.92	1.50	3.00
CS-2	7.00	11.20	4.49	32.57	69.90	86.55	1.50	3.00
CS-3	7.00	11.45	4.49	32.57	68.68	85.03	1.50	3.00
RL-5	8.39	10.12	5.12	56.56	60.69	75.13	2.49	3.00
RL-6	8.20	10.23	6.43	58.29	59.42	73.57	2.50	3.00
B-1	8.53	10.80	4.01	48.09	63.32	78.39	1.21	3.00
B-2	8.48	10.89	3.96	49.90	66.24	82.01	1.27	3.00
B-3	8.35	11.20	4.00	48.48	68.40	84.69	1.23	3.00
B-4	8.47	10.99	3.74	47.80	63.53	78.66	1.02	3.00
CL-4B	7.73	9.54	5.63	49.29	58.24	72.11	2.00	3.00
CR-6A	6.82	9.73	4.45	43.41	56.23	69.61	1.49	3.00
CSK-1	6.23	9.59	3.73	43.48	62.76	77.70	1.50	3.00
DO-1	8.37	9.62	4.79	31.42	58.13	71.97	3.00	3.00
DO-2	8.37	9.62	5.58	31.42	58.13	71.97	2.00	3.00
DO-3	8.37	9.62	4.79	31.42	58.13	71.97	3.00	3.00
DO-4	8.02	9.47	5.38	28.02	58.53	72.47	2.67	3.00
DO-5	8.37	9.62	4.58	31.42	58.13	71.97	1.00	3.00
RBD-R1B	8.51	9.87	3.81	59.08	61.59	76.26	3.00	3.00
RBD-R2	8.52	9.90	4.08	58.42	61.59	76.26	2.21	3.00
RM-11H	8.18	9.68	4.82	25.37	56.07	69.43	1.50	3.00
RM-21H	8.99	9.81	4.88	32.24	60.25	74.59	1.50	3.00
RM-2F	8.75	9.86	4.85	31.36	62.20	77.01	1.50	3.00
RM-4F	8.15	9.57	4.82	29.92	57.10	70.70	1.50	3.00
RM-4H	8.99	9.70	4.86	32.37	59.04	73.10	1.50	3.00

Table 3.3 (continued)

(c) Buckling of Tee Shaped Compression Zone (D.1.3)

Test[1]	P_{crx} (k)	P_{cry} (k)	P_u (k)	M_n/M_m	a_o/s_t	Test/ Theory[2]
RBD-C1	121.24	121.22	138.48	0.552	0.33	1.258
RM-1A	57.34	57.56	92.41	1.000	0.99	1.016
RM-1B	54.90	55.69	90.34	1.000	3.20	1.002
RM-2A	64.58	64.85	73.08	0.524	0.99	1.208
RM-2B	51.88	52.65	37.80	0.275	3.27	1.394
RM-2C	49.83	50.02	61.48	0.513	1.02	1.354
RM-3A	56.80	57.02	78.61	0.782	0.99	1.031
RM-4A	57.54	57.77	87.79	0.922	0.99	1.012
RM-4B	53.31	54.07	70.26	0.730	3.20	1.000
RM-4C	53.94	54.15	85.39	0.946	0.99	0.994
CR-1A	59.36	59.87	110.38	0.812	3.99	1.007
CR-2A	100.46	100.86	134.22	0.492	2.93	1.320
CR-2B	100.46	100.86	202.88	0.813	2.93	1.211
CR-2C	111.90	112.32	183.30	0.597	2.89	1.194
CR-2D	111.90	112.32	121.86	0.360	2.89	1.294
CR-3A	100.63	100.86	152.77	0.485	2.93	1.461
CR-3B	112.08	112.32	240.50	0.845	2.89	1.154
CR-4A	111.44	112.05	128.64	0.379	3.87	1.302
CR-4B	111.44	112.05	200.08	0.692	3.87	1.140
CR-5A	102.97	104.04	127.66	0.456	5.15	1.167
CR-7B	102.25	102.62	211.96	0.804	2.92	1.174
CR-7D	102.25	102.62	114.22	0.362	2.92	1.343
CSK-2	130.34	131.11	227.17	0.670	2.94	1.161
CSK-5	108.22	109.05	185.37	0.655	3.99	1.108
CSK-6	132.08	131.79	110.11	0.445	2.66	1.002
CSK-7	132.39	132.18	133.24	0.521	2.00	1.060
CS-1	106.87	107.46	183.18	0.857	2.97	0.976
CS-2	109.13	109.74	179.24	0.842	2.97	0.963
CS-3	104.22	104.88	162.64	0.682	2.97	1.066
RL-5	98.68	98.97	239.74	1.000	3.88	1.085
RL-6	83.64	85.15	87.09	0.396	9.51	0.927
B-1	90.57	90.91	65.78	0.317	2.12	1.114
B-2	88.23	88.54	119.88	0.573	2.07	1.212
B-3	84.31	84.60	0.13	0.001	2.10	1.348
B-4	87.39	87.62	70.71	0.371	1.69	1.055
CL-4B	118.23	123.19	63.10	0.149	7.21	1.616
CR-6A	111.08	112.09	93.46	0.273	2.89	1.483
CSK-1	127.43	129.13	167.03	0.478	2.94	1.269
DO-1	60.33	60.76	56.67	0.403	2.55	1.163
DO-2	51.09	53.20	24.73	0.143	5.91	1.619
DO-3	51.86	53.34	90.61	0.761	4.43	1.093
DO-4	53.65	55.96	70.45	0.498	6.10	1.250
DO-5	52.55	53.47	98.66	1.000	2.96	1.081
RBD-R1B	129.03	128.85	124.10	0.520	0.94	1.089
RBD-R2	116.35	116.40	86.43	0.346	2.01	1.080
RM-11H	56.12	57.12	103.49	1.000	3.72	1.031
RM-21H	47.91	48.89	46.20	0.325	3.86	1.469
RM-2F	49.02	49.98	38.11	0.241	3.79	1.607
RM-4F	55.58	56.68	75.51	0.642	3.72	1.078
RM-4H	49.09	50.09	64.98	0.617	3.82	1.097

Table 3.3 (continued)

(d) Hole Restrictions (D.3.1)

Test[1]	h_o (in.) <	$0.7d$ (in.)	s_t (in.) &	s_b (in.) >	$0.15d$ (in.)	a_o/s_t &	a_o/s_b < 12.0
RBD-C1	4.67	11.88	6.38	6.38	2.55	0.33	0.33
RM-1A	4.50	5.69	2.04	2.04	1.22	0.99	0.99
RM-1B	4.50	5.69	2.04	2.04	1.22	3.20	3.20
RM-2A	4.50	5.69	2.04	2.04	1.22	0.99	0.99
RM-2B	4.50	5.63	2.00	2.00	1.21	3.27	3.27
RM-2C	4.50	5.63	2.00	2.00	1.21	1.02	1.02
RM-3A	4.50	5.69	2.04	2.04	1.22	0.99	0.99
RM-4A	4.50	5.69	2.04	2.04	1.22	0.99	0.99
RM-4B	4.50	5.69	2.04	2.04	1.22	3.20	3.20
RM-4C	4.50	5.69	2.04	2.04	1.22	0.99	0.99
CR-1A	5.50	6.92	2.20	2.20	1.48	3.99	3.98
CR-2A	7.00	9.89	3.57	3.57	2.12	2.93	2.93
CR-2B	7.00	9.89	3.57	3.57	2.12	2.93	2.93
CR-2C	7.00	9.95	3.61	3.61	2.13	2.89	2.89
CR-2D	7.00	9.95	3.61	3.61	2.13	2.89	2.89
CR-3A	7.00	9.89	3.57	3.57	2.12	2.93	2.93
CR-3B	7.00	9.95	3.61	3.61	2.13	2.89	2.89
CR-4A	7.00	9.95	3.61	3.61	2.13	3.87	3.87
CR-4B	7.00	9.95	3.61	3.61	2.13	3.87	3.87
CR-5A	9.00	9.95	2.61	2.61	2.13	5.15	5.15
CR-7B	7.03	9.99	3.62	3.62	2.14	2.92	2.92
CR-7D	7.03	9.99	3.62	3.62	2.14	2.92	2.92
CSK-2	6.00	11.29	3.07	7.07	2.42	2.94	1.27
CSK-5	6.00	11.21	3.01	7.01	2.40	3.99	1.71
CSK-6	8.00	11.21	6.01	2.01	2.40	2.66	7.98
CSK-7	8.00	11.21	6.01	2.01	2.40	2.00	5.99
CS-1	6.00	8.44	3.03	3.03	1.81	2.97	2.97
CS-2	6.00	8.44	3.03	3.03	1.81	2.97	2.97
CS-3	6.00	8.44	3.03	3.03	1.81	2.97	2.97
RL-5	7.16	11.43	4.59	4.59	2.45	3.88	3.88
RL-6	10.72	11.45	2.82	2.82	2.45	9.51	9.51
B-1	7.44	11.16	4.25	4.25	2.39	2.12	2.12
B-2	7.11	11.07	4.35	4.35	2.37	2.07	2.07
B-3	7.32	11.12	4.28	4.28	2.38	2.10	2.10
B-4	7.16	11.06	4.32	4.32	2.37	1.69	1.69
CL-4B	10.81	12.51	3.00	3.06	2.68	7.21	7.07
CR-6A	7.00	9.95	3.61	3.61	2.13	2.89	2.89
CSK-1	6.00	11.29	3.07	7.07	2.42	2.94	1.27
DO-1	2.36	5.54	2.78	2.78	1.19	2.55	2.55
DO-2	4.72	5.54	1.60	1.60	1.19	5.91	5.91
DO-3	2.36	5.54	1.60	3.96	1.19	4.43	1.79
DO-4	3.54	5.48	1.55	2.73	1.17	6.10	3.46
DO-5	4.72	5.54	1.60	1.60	1.19	2.96	2.96
RBD-R1B	2.30	11.89	7.34	7.34	2.55	0.94	0.94
RBD-R2	5.29	11.88	5.84	5.84	2.55	2.01	2.01
RM-11H	4.50	5.69	1.81	1.81	1.22	3.72	3.72
RM-21H	4.50	5.60	1.75	1.75	1.20	3.86	3.86
RM-2F	4.50	5.64	1.78	1.78	1.21	3.79	3.79
RM-4F	4.50	5.69	1.81	1.81	1.22	3.72	3.72
RM-4H	4.50	5.62	1.77	1.77	1.20	3.82	3.82

Table 3.3 (continued)

(e) One-sided Reinforcement (D.3.5)

Test[(1)]	$A_r < A_f/3$ (in.^{2})		$a_o/h_o \le 2.5$	s_t/t_w, $s_b/t_w \le$ $140/\sqrt{F_y}$			$M_u/(V_u{*}d) \le 20$
CR-7B	0.29	1.09	1.50	11.42	11.42	20.27	3.16
CR-7D	0.29	1.09	1.50	11.42	11.42	20.27	1.19
CSK-2	0.25	1.32	1.50	8.88	20.48	20.63	1.86
CSK-6	0.28	1.18	2.00	19.69	6.57	20.94	1.88
CSK-7	0.28	1.18	1.50	19.69	6.57	20.94	1.88
CS-3	0.32	1.54	1.50	9.04	9.04	22.89	3.32
RL-5	0.16	0.97	2.49	16.73	16.73	20.23	193471.52
RL-6	0.32	0.98	2.50	10.58	10.58	19.81	3.01

Table 3.3 (continued)

Test[1]	(f) Violations[3]
CR-5A	(D.1.3)
RL-5	(D.1.3), (D.3.5)
RL-6	(D.1.3)
DO-3	(D.1.3)
DO-4	(D.1.3)

Notes:

(1) refer to Table 3.0 for key to beam designations

(2) The Test/Theory ratios for Method III, $\lambda = 1.414$, are provided as some indication of the effect of a potential violation of the design parameter on the predicted capacity. If the tee-shaped compression zone were to buckle prematurely, unconservative predictions would result.

(3) Design parameters violated by the respective beams listed did not adversely affect the predicted capacities and did not contribute to premature failure.

Table 3.4 Design Limitation Summary for Composite Beams

	(a) Local Buckling of Compression Flange (D.1.1)			(b) Web Buckling (D.1.2)				
Test[1]	$b_f/2t_f$	$<65/\sqrt{F_y}$	$p_o < 6.0$	h/t	$420/\sqrt{F_y}$	$520/\sqrt{F_y}$	a_o/h_o	a_o/h_o(max)
D-1	7.57	8.80	5.60	55.17	56.43	69.86	2.00	3.00
D-2	7.25	8.99	5.60	55.39	57.64	71.36	2.00	3.00
D-3	7.55	8.96	5.60	55.26	57.97	71.77	2.00	3.00
D-5A	7.57	8.92	5.60	55.17	57.86	71.63	2.00	3.00
D-5B	7.64	8.92	5.91	55.17	57.86	71.63	1.72	3.00
D-6A	7.62	8.88	5.60	55.32	57.86	71.63	2.00	3.00
D-6B	7.62	8.88	5.60	55.32	57.86	71.63	2.00	3.00
D-7A	8.08	10.20	5.60	55.03	65.43	81.01	2.00	3.00
D-7B	8.08	10.20	5.60	55.03	65.43	81.01	2.00	3.00
D-8A	7.11	9.42	5.51	41.53	58.93	72.96	1.99	3.00
D-9A	7.77	10.14	5.97	54.19	65.43	81.01	1.68	3.00
D-9B	7.81	10.14	5.29	53.59	65.43	81.01	1.00	3.00
R-0	7.85	9.14	5.55	41.53	56.07	69.43	2.00	3.00
R-1	7.67	10.26	5.59	44.76	62.54	77.43	2.00	3.00
R-2	7.64	9.82	5.58	42.61	61.07	75.61	2.00	3.00
R-3	7.59	10.01	5.58	41.99	61.13	75.69	2.00	3.00
R-4	7.87	9.83	5.58	42.07	60.56	74.98	2.00	3.00
R-5	7.67	10.26	5.59	44.76	62.54	77.43	2.00	3.00
R-6	7.72	9.83	5.58	43.20	61.13	75.69	2.00	3.00
R-7	7.72	9.83	5.58	43.20	63.32	78.39	2.00	3.00
R-8	7.43	9.79	5.60	44.79	63.32	78.39	2.00	3.00
C-1	7.45	10.36	5.43	45.62	67.69	83.81	2.00	3.00
C-2	7.21	10.37	5.63	47.56	64.50	79.86	2.00	3.00
C-3	7.21	10.37	5.63	47.56	64.50	79.86	2.00	3.00
C-4	7.58	9.54	5.63	49.30	58.24	72.11	2.00	3.00
C-5	4.88	9.81	5.58	44.43	63.17	78.22	2.00	3.00
C-6	7.91	9.92	5.43	44.09	59.52	73.69	2.00	3.00
G-1	7.06	9.82	5.10	24.82	60.69	75.13	1.50	3.00
G-2	7.06	9.82	5.10	24.82	60.69	75.13	1.50	3.00
CHO-3	8.35	9.79	5.14	30.35	58.93	72.96	1.54	3.00
CHO-4	8.35	8.85	5.09	43.37	52.26	64.70	1.51	3.00
CHO-5	8.35	8.85	5.10	43.37	52.26	64.70	1.50	3.00
CHO-6	8.35	9.79	5.06	30.35	58.93	72.96	1.55	3.00
CHO-7	8.35	8.85	5.63	43.37	52.26	64.70	2.03	3.00
WJE-1	6.10	10.69	6.92	52.00	69.05	85.49	2.60	3.00

Table 3.4 (continued)

(c) Hole Restrictions (D.3.1)

Test[1]	h_o (in.)	< 0.7d (in.)	s_t (in.)	& s_b (in.)	> 0.15d (in.)	a_o/s_t	& a_o/s_b < 12.0
D-1	12.38	14.44	4.18	4.10	3.09	5.92	6.04
D-2	12.38	14.44	4.09	4.09	3.09	6.05	6.05
D-3	12.38	14.44	4.11	4.10	3.09	6.03	6.04
D-5A	12.38	14.44	4.17	4.11	3.09	5.94	6.02
D-5B	14.39	14.44	4.11	2.12	3.09	6.02	11.66
D-6A	12.38	14.44	4.12	4.12	3.09	6.01	6.01
D-6B	12.38	14.44	4.12	4.12	3.09	6.01	6.01
D-7A	12.38	14.44	4.03	4.15	3.09	6.15	5.96
D-7B	12.38	14.44	4.08	4.19	3.09	6.07	5.91
D-8A	5.95	7.09	2.10	2.09	1.52	5.64	5.66
D-9A	14.75	14.44	2.96	2.96	3.09	8.36	8.36
D-9B	14.75	14.44	3.08	2.81	3.09	4.80	5.25
R-0	5.91	6.99	2.04	2.04	1.50	5.79	5.79
R-1	8.39	9.81	2.81	2.81	2.10	5.97	5.97
R-2	8.39	9.83	2.83	2.83	2.11	5.93	5.93
R-3	8.39	9.82	2.82	2.82	2.10	5.94	5.94
R-4	8.39	9.83	2.84	2.84	2.11	5.91	5.91
R-5	8.39	9.81	1.41	4.21	2.10	11.89	3.98
R-6	8.39	9.83	2.84	2.84	2.11	5.91	5.91
R-7	8.39	9.83	2.84	2.84	2.11	5.91	5.91
R-8	8.39	9.79	2.80	2.80	2.10	5.99	5.99
C-1	8.00	9.80	3.00	3.00	2.10	5.33	5.33
C-2	10.81	12.52	3.48	3.77	2.68	6.22	5.74
C-3	10.81	12.52	3.61	3.65	2.68	6.00	5.93
C-4	10.81	12.52	3.49	3.56	2.68	6.21	6.08
C-5	10.81	12.69	3.68	3.75	2.72	5.87	5.78
C-6	8.00	9.80	2.86	2.80	2.10	5.60	5.71
G-1	4.80	5.60	1.63	1.63	1.20	4.42	4.42
G-2	4.80	5.60	1.63	1.63	1.20	4.42	4.42
CHO-3	4.72	5.51	1.50	1.50	1.18	4.85	4.85
CHO-4	7.05	8.27	2.36	2.40	1.77	4.50	4.43
CHO-5	7.09	8.27	2.40	2.32	1.77	4.43	4.58
CHO-6	4.61	5.51	1.49	1.45	1.18	4.78	4.91
CHO-7	7.09	8.27	2.29	2.28	1.77	6.28	6.30
WJE-1	15.00	14.58	2.76	2.76	3.12	14.12	14.12

Table 3.4 (continued)

Test[1]	(d) Violations [2]
D-9A	(D.3.1)
D-9B	(D.3.1)
R-5	(D.3.1)
WJE-1	(D.1.2), (D.3.1)

Notes:

(1) refer to Table 3.0 for key to beams designations

(2) Design paramteters violated by the respective beams did not adversely affect the predicted capacities and did not contribute to premature failure.

Table 3.5 Steel Beam Shear Capacity Summary: Method I, λ = 1.414

(values in kips)

Test	V_{mbl}	V_{pb}	V_{bl}	V_{mtl}	V_{pt}	V_{tl}	V_m	V_I
RBD-C1	57.72	47.30	47.30	57.72	47.30	47.30	82.99	82.99
RM-1A	14.89	14.88	14.88	14.89	14.88	14.88	39.15	29.76
RM-1B	7.68	14.69	7.68	7.64	14.64	7.64	38.66	15.32
RM-2A	16.10	15.94	15.94	16.10	15.94	15.94	41.94	31.88
RM-2B	6.41	12.16	6.41	6.41	12.16	6.41	32.34	12.82
RM-2C	11.88	11.91	11.88	11.88	11.91	11.88	31.69	23.76
RM-3A	15.02	15.05	15.02	15.02	15.05	15.02	39.60	30.05
RM-4A	15.02	15.02	15.02	15.02	15.02	15.02	39.52	30.04
RM-4B	7.63	14.64	7.63	7.63	14.64	7.63	38.53	15.26
RM-4C	16.35	16.79	16.35	16.35	16.79	16.35	44.17	32.71
CR-1A	14.39	17.67	14.39	14.34	17.63	14.34	52.41	28.73
CR-2A	30.36	27.71	27.71	30.36	27.71	27.71	72.49	55.42
CR-2B	30.36	27.71	27.71	30.36	27.71	27.71	72.49	55.42
CR-2C	33.00	35.47	33.00	33.00	35.47	33.00	92.22	66.01
CR-2D	33.00	35.47	33.00	33.00	35.47	33.00	92.22	66.01
CR-3A	30.36	27.71	27.71	30.36	27.71	27.71	72.49	55.42
CR-3B	39.02	35.47	35.47	39.02	35.47	35.47	92.22	70.94
CR-4A	26.64	35.47	26.64	26.64	35.47	26.64	92.22	53.27
CR-4B	26.64	35.47	26.64	26.64	35.47	26.64	92.22	53.27
CR-5A	23.67	25.65	23.67	23.67	25.65	23.67	92.22	47.34
CR-7B	31.40	31.60	31.40	31.39	31.60	31.39	82.22	62.79
CR-7D	28.47	31.60	28.47	31.39	31.60	31.39	82.22	59.86
CSK-2	80.37	64.83	64.83	31.60	28.13	28.13	97.69	92.96
CSK-5	60.91	55.15	55.15	23.62	23.66	23.62	83.19	78.78
CSK-6	10.21	15.79	10.21	41.16	47.28	41.16	83.19	51.36
CSK-7	13.02	15.79	13.02	48.93	47.28	47.28	83.19	60.30
CS-1	24.60	21.98	21.98	24.60	21.98	21.98	57.73	43.95
CS-2	23.87	21.16	21.16	23.87	21.16	21.16	55.58	42.31
CS-3	23.54	21.92	21.92	23.54	21.92	21.92	57.58	43.84
RL-5	21.30	34.74	21.30	21.30	34.74	21.30	81.67	42.60
RL-6	11.84	21.60	11.84	11.84	21.60	11.84	82.80	23.69
B-1	21.59	33.90	21.59	21.59	33.90	21.59	83.92	43.18
B-2	19.68	30.28	19.68	19.68	30.28	19.68	72.65	39.36
B-3	18.60	28.88	18.60	18.60	28.88	18.60	70.72	37.20
B-4	24.53	34.12	24.53	24.53	34.12	24.53	82.35	49.06
CL-4B	9.26	31.51	9.26	8.78	30.89	8.78	121.49	18.03
CR-6A	19.13	35.47	19.13	19.13	35.47	19.13	92.22	38.27
CSK-1	51.67	63.03	51.67	16.28	27.34	16.28	94.98	67.95
DO-1	11.27	19.43	11.27	11.27	19.43	11.27	36.56	22.53
DO-2	4.00	11.19	4.00	4.00	11.18	4.00	36.56	8.00
DO-3	19.18	27.69	19.18	5.01	11.18	5.01	36.56	24.19
DO-4	9.80	20.77	9.80	4.13	11.80	4.13	39.29	13.93
DO-5	6.72	11.18	6.72	6.72	11.18	6.72	36.56	13.45
RBD-R1B	49.59	53.80	49.59	49.59	53.80	49.59	82.14	82.14
RBD-R2	28.29	43.24	28.29	28.29	43.24	28.29	82.99	56.58
RM-11H	7.98	17.32	7.98	7.98	17.32	7.98	51.24	15.96
RM-21H	5.35	11.29	5.35	5.35	11.29	5.35	34.08	10.71
RM-2F	5.40	11.17	5.40	5.40	11.17	5.40	33.34	10.80
RM-4F	6.81	14.16	6.81	6.81	14.16	6.81	41.87	13.62
RM-4H	5.62	11.86	5.62	5.62	11.86	5.62	35.61	11.25

Notes:

refer to Table 3.0 for key to beam designations

V_{mbl}, V_{mtl} = shear capacity of bottom and top tee, respectively, using Eq. B.1.

V_{pb}, V_{pt} = plastic shear capacity of bottom and top tee, respectively, using Eqs. 2.22, and 2.18.

V_{bl}, V_{tl} = governing shear capacity of top and bottom tees, respectively.

V_m = maximum permissible shear capacity of beam per Section D.1.2.

V_I = maximum shear capacity as predicted by Method I.

Table 3.6 Composite Beam Shear Capacity Summary: Method I, $\lambda = 1.414$

(values in kips)

Test	$V_{t(a)}$	$V_{t(b)}$	V_{pt}	V_{tsh}	V_{tl}	V_b	V_{pb}	V_{bl}	V_I
D-1	28.88	47.84	47.84	54.86	28.88	14.85	46.96	14.85	43.73
D-2	28.45	44.81	44.81	52.12	28.45	13.99	44.81	13.99	42.44
D-3	29.78	44.54	44.54	52.26	29.78	13.86	44.46	13.86	43.64
D-5A	25.94	45.40	45.40	52.63	25.94	13.39	44.77	13.39	39.33
D-5B	29.14	44.77	44.77	52.26	29.14	4.53	23.13	4.53	33.67
D-6A	26.58	44.75	44.75	51.41	26.58	13.94	44.70	13.94	40.52
D-6B	41.99	44.75	44.75	53.36	41.99	13.94	44.70	13.94	55.92
D-7A	34.48	34.47	34.47	42.96	34.47	10.93	35.54	10.93	45.40
D-7B	31.51	34.90	34.90	43.50	31.51	11.11	35.86	11.11	42.62
D-8A	20.71	17.39	14.20	23.26	17.39	4.58	14.16	4.58	21.97
D-9A	35.24	30.56	25.70	44.68	30.56	6.32	25.70	6.32	36.88
D-9B	45.29	40.04	26.99	46.40	40.04	8.96	24.68	8.96	49.00
R-0	20.90	17.02	15.06	24.52	17.02	4.76	15.06	4.76	21.78
R-1	18.51	21.44	21.44	30.07	18.51	7.18	21.44	7.18	25.70
R-2	21.76	23.88	23.88	32.01	21.76	7.95	23.88	7.95	29.71
R-3	34.93	32.52	24.05	34.07	32.52	7.94	24.05	7.94	40.46
R-4	17.98	24.64	24.64	34.26	17.98	8.18	24.64	8.18	26.16
R-5	16.04	16.14	10.76	19.39	16.14	13.71	32.12	13.71	29.85
R-6	12.93	23.56	23.56	31.37	12.93	7.87	23.56	7.87	20.80
R-7	25.30	23.80	21.97	29.78	23.80	7.38	21.97	7.38	31.19
R-8	24.32	23.05	20.73	28.35	23.05	7.08	20.73	7.08	30.13
C-1	34.32	29.51	19.16	33.21	29.51	7.01	19.16	7.01	36.52
C-2	30.45	30.28	30.28	41.17	30.28	10.87	32.85	10.87	41.15
C-3	32.12	31.42	31.42	43.21	31.42	10.42	31.81	10.42	41.84
C-4	37.75	35.89	35.89	47.11	35.89	11.90	36.61	11.90	47.78
C-5	37.47	35.71	35.71	47.21	35.71	11.69	36.32	11.69	47.40
C-6	37.33	31.75	24.30	34.95	31.75	7.91	23.86	7.91	39.65
G-1	48.90	54.53	12.85	21.42	21.42	7.27	12.85	7.27	28.69
G-2	38.74	44.97	12.85	21.44	21.44	7.27	12.85	7.27	28.71
CHO-3	50.47	55.78	10.38	27.25	27.25	4.90	10.38	4.90	32.15
CHO-4	42.62	42.13	22.53	39.41	39.41	9.82	22.92	9.82	49.24
CHO-5	45.28	43.56	22.92	39.78	39.78	9.31	22.15	9.31	49.09
CHO-6	59.80	72.61	10.66	27.53	27.53	9.99	10.38	9.99	37.52
CHO-7	45.03	48.37	22.53	39.14	39.14	17.32	22.53	17.32	56.46
WJE-1	42.13	40.88	23.85	23.85	23.85	14.00	23.85	14.00	37.85

Notes:

refer to Table 3.0 for key to beam designations

$V_{t(a)}$ = shear capacity of top tee using Eq. B.1.
$V_{t(b)}$ = shear capacity of top tee using Eq. 2.33.
V_{pt} = plastic shear capacity of top tee using Eq. 2.18
V_{tsh} = combined plastic shear capacity of top tee and concrete using Eq. 2.21.
V_{tl} = governing shear capacity of top tee.
V_b = shear capacity of bottom tee using Eq. B.1.
V_{pb} = plastic shear capacity of bottom tee using Eq. 2.22.
V_{bl} = governing shear capacity of bottom tee.
V_I = maximum shear capacity as predicted by Method I.

Table 3.7 Steel Beam Shear Capacity Summary: Method II

(values in kips)

Test	V_{mb2}	V_{pb}	V_{b2}	V_{mt2}	V_{pt}	V_{t2}	V_m	V_2
RBD-C1	46.47	47.30	46.47	46.47	47.30	46.47	82.99	82.99
RM-1A	12.91	14.88	12.91	12.91	14.88	12.91	39.15	25.81
RM-1B	6.99	14.69	6.99	6.94	14.64	6.94	38.66	13.93
RM-2A	13.82	15.94	13.82	13.82	15.94	13.82	41.94	27.65
RM-2B	5.69	12.16	5.69	5.69	12.16	5.69	32.34	11.38
RM-2C	10.28	11.91	10.28	10.28	11.91	10.28	31.69	20.56
RM-3A	13.05	15.05	13.05	13.05	15.05	13.05	39.60	26.11
RM-4A	13.03	15.02	13.03	13.03	15.02	13.03	39.52	26.06
RM-4B	6.97	14.64	6.97	6.97	14.64	6.97	38.53	13.93
RM-4C	14.56	16.79	14.56	14.56	16.79	14.56	44.17	29.12
CR-1A	14.18	17.67	14.18	14.14	17.63	14.14	52.41	28.32
CR-2A	27.19	27.71	27.19	27.19	27.71	27.19	72.49	54.38
CR-2B	27.19	27.71	27.19	27.19	27.71	27.19	72.49	54.38
CR-2C	31.63	35.47	31.63	31.63	35.47	31.63	92.22	63.27
CR-2D	31.63	35.47	31.63	31.63	35.47	31.63	92.22	63.27
CR-3A	27.02	27.71	27.02	27.02	27.71	27.02	72.49	54.04
CR-3B	34.92	35.47	34.92	34.92	35.47	34.92	92.22	69.84
CR-4A	26.56	35.47	26.56	26.56	35.47	26.56	92.22	53.12
CR-4B	26.56	35.47	26.56	26.56	35.47	26.56	92.22	53.12
CR-5A	22.67	25.65	22.67	22.67	25.65	22.67	92.22	45.34
CR-7B	29.51	31.60	29.51	29.49	31.60	29.49	82.22	59.00
CR-7D	27.50	31.60	27.50	29.49	31.60	29.49	82.22	56.99
CSK-2	64.76	64.83	64.76	27.50	28.13	27.50	97.69	92.26
CSK-5	54.58	55.15	54.58	21.96	23.66	21.96	83.19	76.55
CSK-6	9.64	15.79	9.64	40.46	47.28	40.46	83.19	50.10
CSK-7	12.17	15.79	12.17	45.11	47.28	45.11	83.19	57.28
CS-1	21.26	21.98	21.26	21.26	21.98	21.26	57.73	42.53
CS-2	20.47	21.16	20.47	20.47	21.16	20.47	55.58	40.95
CS-3	20.75	21.92	20.75	20.75	21.92	20.75	57.58	41.50
RL-5	21.40	34.74	21.40	21.39	34.74	21.39	81.67	42.79
RL-6	11.71	21.60	11.71	11.71	21.60	11.71	82.80	23.42
B-1	21.46	33.90	21.46	21.46	33.90	21.46	83.92	42.93
B-2	19.44	30.28	19.44	19.44	30.28	19.44	72.65	38.87
B-3	18.36	28.88	18.36	18.36	28.88	18.36	70.72	36.72
B-4	24.42	34.12	24.42	24.42	34.12	24.42	82.35	48.84
CL-4B	7.50	31.51	7.50	7.22	30.89	7.22	121.49	14.72
CR-6A	18.21	35.47	18.21	18.21	35.47	18.21	92.22	36.43
CSK-1	50.78	63.03	50.78	13.89	27.34	13.89	94.98	64.67
DO-1	10.92	19.43	10.92	10.92	19.43	10.92	36.56	21.84
DO-2	3.15	11.19	3.15	3.14	11.18	3.14	36.56	6.29
DO-3	19.26	27.69	19.26	4.07	11.18	4.07	36.56	23.32
DO-4	9.29	20.77	9.29	3.22	11.80	3.22	39.29	12.52
DO-5	5.65	11.18	5.65	5.65	11.18	5.65	36.56	11.30
RBD-R1B	47.28	53.80	47.28	47.28	53.80	47.28	82.14	82.14
RBD-R2	28.26	43.24	28.26	28.26	43.24	28.26	82.99	56.53
RM-11H	7.31	17.32	7.31	7.31	17.32	7.31	51.24	14.61
RM-21H	4.63	11.29	4.63	4.63	11.29	4.63	34.08	9.25
RM-2F	4.64	11.17	4.64	4.64	11.17	4.64	33.34	9.29
RM-4F	5.97	14.16	5.97	5.97	14.16	5.97	41.87	11.94
RM-4H	4.89	11.86	4.89	4.89	11.86	4.89	35.61	9.79

Notes:

refer to Table 3.0 for key to beam designations

V_{mb2}, V_{mt2} = shear capacity of bottom and top tee, respectively, using Eq. 2.43.

V_{pb}, V_{pt} = plastic shear capacity of bottom and top tee, respectively, using Eqs. 2.22 and 2.18.

V_{b2}, V_{p2} = governing shear capacity of bottom and bottom tees, respectively.

V_m = maximum permissible shear capacity of beam per Section D.1.2.

V_2 = maximum shear capacity as predicted by Method II.

Table 3.8 Composite Beam Shear Capacity Summary: Method II

(values in kips)

Test	$V_{t(a)}$	$V_{t(b)}$	V_{pt}	V_{tsh}	V_{t2}	V_b	V_{pb}	V_{b2}	V_2
D-1	29.00	47.84	47.84	54.86	29.00	12.95	46.96	12.95	41.96
D-2	29.27	44.81	44.81	52.12	29.27	12.34	44.81	12.34	41.61
D-3	30.48	44.54	44.54	52.26	30.48	12.25	44.46	12.25	42.73
D-5A	26.40	45.40	45.40	52.63	26.40	12.37	44.77	12.37	38.77
D-5B	29.93	44.77	44.77	52.26	29.93	3.40	23.13	3.40	33.33
D-6A	27.01	44.75	44.75	51.41	27.01	12.37	44.70	12.37	39.37
D-6B	41.05	44.75	44.75	53.36	41.05	12.37	44.70	12.37	53.41
D-7A	32.77	34.47	34.47	42.96	32.77	9.91	35.54	9.91	42.68
D-7B	31.36	34.90	34.90	43.50	31.36	10.09	35.86	10.09	41.45
D-8A	0.00	16.83	14.20	23.26	16.83	4.15	14.16	4.15	20.98
D-9A	0.00	29.15	25.70	44.68	29.15	5.21	25.70	5.21	34.36
D-9B	0.00	38.61	26.99	46.40	38.61	7.74	24.68	7.74	46.35
R-0	0.00	16.53	15.06	24.52	16.53	4.31	15.06	4.31	20.85
R-1	17.84	21.44	21.44	30.07	17.84	5.98	21.44	5.98	23.82
R-2	21.87	23.88	23.88	32.01	21.87	6.70	23.88	6.70	28.57
R-3	0.00	30.71	24.05	34.07	30.71	6.73	24.05	6.73	37.44
R-4	18.64	24.64	24.64	34.26	18.64	6.93	24.64	6.93	25.58
R-5	0.00	14.97	10.76	19.39	14.97	12.81	32.12	12.81	27.78
R-6	12.48	23.56	23.56	31.37	12.48	6.63	23.56	6.63	19.10
R-7	0.00	22.31	21.97	29.78	22.31	6.18	21.97	6.18	28.49
R-8	0.00	21.40	20.73	28.35	21.40	5.76	20.73	5.76	27.16
C-1	0.00	27.06	19.16	33.21	27.06	5.92	19.16	5.92	32.99
C-2	29.41	30.28	30.28	41.17	29.41	9.50	32.85	9.50	38.90
C-3	30.74	31.42	31.42	43.21	30.74	8.92	31.81	8.92	39.66
C-4	35.29	35.89	35.89	47.11	35.29	10.02	36.61	10.02	45.31
C-5	34.90	35.71	35.71	47.21	34.90	10.43	36.32	10.43	45.33
C-6	0.00	29.23	24.30	34.95	29.23	6.93	23.86	6.93	36.16
G-1	0.00	52.34	12.85	21.42	21.42	4.69	12.85	4.69	26.11
G-2	0.00	42.77	12.85	21.44	21.44	4.69	12.85	4.69	26.13
CHO-3	0.00	54.03	10.38	27.25	27.25	3.49	10.38	3.49	30.74
CHO-4	0.00	40.97	22.53	39.41	39.41	8.35	22.92	8.35	47.76
CHO-5	0.00	42.34	22.92	39.78	39.78	7.83	22.15	7.83	47.61
CHO-6	0.00	70.81	10.66	27.53	27.53	9.22	10.38	9.22	36.75
CHO-7	0.00	48.32	22.53	39.14	39.14	16.70	22.53	16.70	55.84
WJE-1	0.00	38.76	23.85	23.85	23.85	14.43	23.85	14.43	38.28

Notes:

refer to Table 3.0 for key to beam designations

$V_{t(a)}$ = shear capacity of top tee using Eq. 2.43
$V_{t(b)}$ = shear capacity of top tee using Eq. 2.46.
V_{pt} = plastic shear capacity of top tee using Eq. 2.18
V_{tsh} = combined plastic shear capacity of top tee and concrete using Eq. 2.21.
V_{t2} = governing shear capacity of top tee.
V_b = shear capacity of bottom tee using Eq. 2.43.
V_{pb} = plastic shear capacity of bottom tee using Eq. 2.22.
V_{b2} = governing shear capacity of bottom tee.
V_2 = maximum shear capacity as predicted by Method II.

Table 3.9 Steel Beam Shear Capacity Summary: Method III, $\lambda = 1.414$

(values in kips)

Test	V_{mb3}	V_{pb}	V_{b3}	V_{mt3}	V_{pt}	V_{t3}	V_m	V_3
RBD-C1	56.21	47.30	47.30	56.21	47.30	47.30	82.99	82.99
RM-1A	13.37	14.88	13.37	13.37	14.88	13.37	39.15	26.74
RM-1B	7.29	14.69	7.29	7.25	14.64	7.25	38.66	14.54
RM-2A	14.32	15.94	14.32	14.32	15.94	14.32	41.94	28.64
RM-2B	5.95	12.16	5.95	5.95	12.16	5.95	32.34	11.90
RM-2C	10.62	11.91	10.62	10.62	11.91	10.62	31.69	21.24
RM-3A	13.52	15.05	13.52	13.52	15.05	13.52	39.60	27.04
RM-4A	13.50	15.02	13.50	13.50	15.02	13.50	39.52	26.99
RM-4B	7.27	14.64	7.27	7.27	14.64	7.27	38.53	14.54
RM-4C	15.08	16.79	15.08	15.08	16.79	15.08	44.17	30.17
CR-1A	14.26	17.67	14.26	14.22	17.63	14.22	52.41	28.47
CR-2A	29.62	27.71	27.71	29.62	27.71	27.71	72.49	55.42
CR-2B	29.62	27.71	27.71	29.62	27.71	27.71	72.49	55.42
CR-2C	32.55	35.47	32.55	32.55	35.47	32.55	92.22	65.10
CR-2D	32.55	35.47	32.55	32.55	35.47	32.55	92.22	65.10
CR-3A	29.19	27.71	27.71	29.19	27.71	27.71	72.49	55.42
CR-3B	38.23	35.47	35.47	38.23	35.47	35.47	92.22	70.94
CR-4A	26.59	35.47	26.59	26.59	35.47	26.59	92.22	53.17
CR-4B	26.59	35.47	26.59	26.59	35.47	26.59	92.22	53.17
CR-5A	23.06	25.65	23.06	23.06	25.65	23.06	92.22	46.12
CR-7B	30.92	31.60	30.92	30.91	31.60	30.91	82.22	61.83
CR-7D	28.09	31.60	28.09	30.91	31.60	30.91	82.22	59.00
CSK-2	78.48	64.83	64.83	29.82	28.13	28.13	97.69	92.96
CSK-5	62.34	55.15	55.15	22.76	23.66	22.76	83.19	77.91
CSK-6	9.67	15.79	9.67	41.22	47.28	41.22	83.19	50.89
CSK-7	12.19	15.79	12.19	48.61	47.28	47.28	83.19	59.47
CS-1	22.80	21.98	21.98	22.80	21.98	21.98	57.73	43.95
CS-2	21.95	21.16	21.16	21.95	21.16	21.16	55.58	42.31
CS-3	21.90	21.92	21.90	21.90	21.92	21.90	57.58	43.80
RL-5	21.51	34.74	21.51	21.50	34.74	21.50	81.67	43.01
RL-6	11.81	21.60	11.81	11.81	21.60	11.81	82.80	23.63
B-1	21.57	33.90	21.57	21.57	33.90	21.57	83.92	43.13
B-2	19.51	30.28	19.51	19.51	30.28	19.51	72.65	39.02
B-3	18.44	28.88	18.44	18.44	28.88	18.44	70.72	36.89
B-4	24.42	34.12	24.42	24.42	34.12	24.42	82.35	48.83
CL-4B	8.77	31.51	8.77	8.46	30.89	8.46	121.49	17.23
CR-6A	18.78	35.47	18.78	18.78	35.47	18.78	92.22	37.55
CSK-1	51.35	63.03	51.35	14.35	27.34	14.35	94.98	65.70
DO-1	11.11	19.43	11.11	11.11	19.43	11.11	36.56	22.23
DO-2	3.59	11.19	3.59	3.58	11.18	3.58	36.56	7.17
DO-3	19.26	27.69	19.26	4.44	11.18	4.44	36.56	23.70
DO-4	9.79	20.77	9.79	3.69	11.80	3.69	39.29	13.48
DO-5	5.84	11.18	5.84	5.84	11.18	5.84	36.56	11.68
RBD-R1B	49.31	53.80	49.31	49.31	53.80	49.31	82.14	82.14
RBD-R2	28.33	43.24	28.33	28.33	43.24	28.33	82.99	56.67
RM-11H	7.78	17.32	7.78	7.78	17.32	7.78	51.24	15.55
RM-21H	4.95	11.29	4.95	4.95	11.29	4.95	34.08	9.90
RM-2F	4.95	11.17	4.95	4.95	11.17	4.95	33.34	9.91
RM-4F	6.36	14.16	6.36	6.36	14.16	6.36	41.87	12.71
RM-4H	5.23	11.86	5.23	5.23	11.86	5.23	35.61	10.45

Notes:
refer to Table 3.0 for key to beam designations

V_{mb3}, V_{mt3} = shear capacity of bottom and top tee, respectively, using Eq. 2.54.
V_{pb}, V_{pt} = plastic shear capacity of bottom and top tee, respectively, using Eqs. 2.22 and 2.18.
V_{b3}, V_{t3} = governing shear capacity of bottom and bottom tees, respectively.
V_m = maximum permissible shear capacity of beam per Section D.1.2.
V_3 = maximum shear capacity as predicted by Method III.

Table 3.10 Composite Beam Shear Capacity Summary: Method III, λ = 1.414

(values in kips)

Test	$V_{t(a)}$	$V_{t(b)}$	V_{pt}	V_{tsh}	V_{t3}	V_b	V_{pb}	V_{b3}	V_3
D-1	29.14	47.84	47.84	54.86	29.14	14.81	46.96	14.81	43.95
D-2	29.30	44.81	44.81	52.12	29.30	14.11	44.81	14.11	43.41
D-3	30.48	44.54	44.54	52.26	30.48	14.01	44.46	14.01	44.49
D-5A	26.59	45.40	45.40	52.63	26.59	14.14	44.77	14.14	40.73
D-5B	29.95	44.77	44.77	52.26	29.95	4.23	23.13	4.23	34.17
D-6A	27.14	44.75	44.75	51.41	27.14	14.13	44.70	14.13	41.27
D-6B	42.03	44.75	44.75	53.36	42.03	14.13	44.70	14.13	56.16
D-7A	33.93	34.47	34.47	42.96	33.93	11.31	35.54	11.31	45.24
D-7B	31.95	34.90	34.90	43.50	31.95	11.49	35.86	11.49	43.45
D-8A	0.00	16.83	14.20	23.26	16.83	4.69	14.16	4.69	21.53
D-9A	0.00	29.15	25.70	44.68	29.15	6.24	25.70	6.24	35.38
D-9B	0.00	38.61	26.99	46.40	38.61	8.66	24.68	8.66	47.28
R-0	0.00	16.53	15.06	24.52	16.53	4.90	15.06	4.90	21.43
R-1	17.97	21.44	21.44	30.07	17.97	6.82	21.44	6.82	24.79
R-2	22.39	23.88	23.88	32.01	22.39	7.64	23.88	7.64	30.02
R-3	0.00	30.71	24.05	34.07	30.71	7.68	24.05	7.68	38.38
R-4	18.66	24.64	24.64	34.26	18.66	7.90	24.64	7.90	26.56
R-5	0.00	14.97	10.76	19.39	14.97	13.76	32.12	13.76	28.73
R-6	12.67	23.56	23.56	31.37	12.67	7.55	23.56	7.55	20.22
R-7	0.00	22.31	21.97	29.78	22.31	7.04	21.97	7.04	29.35
R-8	0.00	21.40	20.73	28.35	21.40	6.57	20.73	6.57	27.98
C-1	0.00	27.06	19.16	33.21	27.06	6.65	19.16	6.65	33.71
C-2	30.75	30.28	30.28	41.17	30.75	10.77	32.85	10.77	41.53
C-3	32.34	31.42	31.42	43.21	32.34	10.17	31.81	10.17	42.52
C-4	37.24	35.89	35.89	47.11	37.24	11.47	36.61	11.47	48.71
C-5	36.73	35.71	35.71	47.21	36.73	11.85	36.32	11.85	48.58
C-6	0.00	29.23	24.30	34.95	29.23	7.85	23.86	7.85	37.08
G-1	0.00	52.34	12.85	21.42	21.42	5.12	12.85	5.12	26.54
G-2	0.00	42.77	12.85	21.44	21.44	5.12	12.85	5.12	26.56
CHO-3	0.00	54.03	10.38	27.25	27.25	3.86	10.38	3.86	31.11
CHO-4	0.00	40.97	22.53	39.41	39.41	9.11	22.92	9.11	48.52
CHO-5	0.00	42.34	22.92	39.78	39.78	8.59	22.15	8.59	48.37
CHO-6	0.00	70.81	10.66	27.53	27.53	9.40	10.38	9.40	36.93
CHO-7	0.00	48.32	22.53	39.14	39.14	16.71	22.53	16.71	55.85
WJE-1	0.00	38.76	23.85	23.85	23.85	14.46	23.85	14.46	38.31

Notes:

refer to Table 3.0 for key to beam designations

$V_{t(a)}$ = shear capacity of top tee using Eq. 2.54.
$V_{t(b)}$ = shear capacity of top tee using Eq. 2.46.
V_{pt} = plastic shear capacity of top tee using Eq. 2.18
V_{tsh} = combined plastic shear capacity of top tee and concrete using Eq. 2.21.
V_{t3} = governing shear capacity of top tee.
V_b = shear capacity of bottom tee using Eq. 2.43.
V_{pb} = plastic shear capacity of bottom tee using Eq. 2.22.
V_{b3} = governing shear capacity of bottom tee.
V_3 = maximum shear capacity as predicted by Method III.

Table 3.11 Steel Beam Shear Capacity Summary: Redwood and Shrivastava (1980)

Test	V_p (k)	α_t	α_b	$term_1$	$term_2$	$(d-2t_f)/t_w$	V_a (k)	V_m (k)	V (k)
RBD-C1	125.74	27.69	27.70	0.37	0.37	58.42	92.94	83.87	83.87
RM-1A	59.31	3.04	3.04	0.22	0.22	30.41	25.80	29.76	25.80
RM-1B	58.58	0.29	0.29	0.12	0.12	30.95	13.98	29.33	13.98
RM-2A	63.54	3.04	3.04	0.22	0.22	30.03	27.64	31.88	27.64
RM-2B	48.99	0.28	0.28	0.12	0.12	31.97	11.38	24.31	11.38
RM-2C	48.01	2.91	2.91	0.21	0.21	31.83	20.56	23.83	20.56
RM-3A	59.99	3.04	3.04	0.22	0.22	30.53	26.10	30.10	26.10
RM-4A	59.88	3.04	3.04	0.22	0.22	30.53	26.05	30.04	26.05
RM-4B	58.37	0.29	0.29	0.12	0.12	30.66	13.93	29.28	13.93
RM-4C	66.93	3.04	3.04	0.22	0.22	30.41	29.12	33.58	29.12
CR-1A	79.41	0.00	0.00	0.29	0.00	37.88	23.26	35.29	23.26
CR-2A	109.84	0.00	0.00	0.50	0.00	40.30	55.42	55.42	55.42
CR-2B	109.84	0.00	0.00	0.50	0.00	40.30	55.42	55.42	55.42
CR-2C	139.72	0.00	0.00	0.35	0.00	43.41	48.88	70.94	48.88
CR-2D	139.72	0.00	0.00	0.35	0.00	43.41	48.88	70.94	48.88
CR-3A	109.84	0.00	0.00	0.50	0.00	40.30	55.42	55.42	55.42
CR-3B	139.72	0.00	0.00	0.51	0.00	43.41	70.94	70.94	70.94
CR-4A	139.72	0.00	0.00	0.26	0.00	43.41	36.53	70.94	36.53
CR-4B	139.72	0.00	0.00	0.26	0.00	43.41	36.53	70.94	36.53
CR-5A	139.72	0.00	0.00	0.32	0.00	43.41	44.34	51.29	44.34
CR-7B	124.58	0.00	0.00	0.41	0.00	41.95	51.65	63.21	51.65
CR-7D	124.58	0.00	0.00	0.41	0.00	41.95	51.65	63.21	51.65
CSK-2	148.02	0.00	0.00	0.63	0.00	43.48	92.96	92.96	92.96
CSK-5	126.05	0.00	0.00	0.57	0.00	49.18	71.27	78.81	71.27
CSK-6	126.05	0.00	0.00	0.28	0.00	49.18	35.56	63.06	35.56
CSK-7	126.05	0.00	0.00	0.38	0.00	49.18	47.42	63.06	47.42

Table 3.11 (continued)

Test	V_p (k)	α_t	α_b	$term_1$	$term_2$	$(d\text{-}2t_f)/t_w$	V_a (k)	V_m (k)	V (k)
CS-1	87.47	0.00	0.00	0.50	0.00	32.57	43.95	43.95	43.95
CS-2	84.21	0.00	0.00	0.50	0.00	32.57	42.31	42.31	42.31
CS-3	87.24	0.00	0.00	0.50	0.00	32.57	43.84	43.84	43.84
RL-5	123.74	0.00	0.00	0.15	0.00	56.56	18.71	69.49	18.71
RL-6	125.45	0.00	0.00	0.14	0.00	58.29	17.51	43.20	17.51
B-1	127.15	0.67	0.67	0.17	0.17	48.09	42.93	67.80	42.93
B-2	110.08	0.70	0.70	0.18	0.18	49.90	38.87	60.56	38.87
B-3	107.15	0.68	0.68	0.17	0.17	48.48	36.72	57.76	36.72
B-4	124.77	1.05	1.05	0.20	0.20	47.80	48.84	68.23	48.84
CL-4B	184.07	0.06	0.11	0.04	0.07	49.29	20.16	62.40	20.16
CR-6A	139.72	0.36	0.36	0.13	0.13	43.41	36.43	70.94	36.43
CSK-1	143.90	0.35	1.85	0.10	0.35	43.48	64.67	90.38	64.67
DO-1	55.39	0.46	0.46	0.20	0.20	31.42	21.84	36.94	21.84
DO-2	55.39	0.09	0.09	0.06	0.06	31.42	6.28	22.36	6.28
DO-3	55.39	0.15	0.94	0.07	0.35	31.42	23.32	36.94	23.32
DO-4	59.53	0.08	0.25	0.05	0.16	28.02	12.52	32.56	12.52
DO-5	55.39	0.34	0.34	0.10	0.10	31.42	11.30	22.35	11.30
RBD-R1B	124.45	3.39	3.39	0.38	0.38	59.08	94.56	83.01	83.01
RBD-R2	125.74	0.75	0.75	0.22	0.23	58.42	56.58	83.87	56.58
RM-11H	77.63	0.22	0.22	0.09	0.09	25.37	14.61	34.65	14.61
RM-21H	51.63	0.20	0.20	0.09	0.09	32.24	9.25	22.59	9.25
RM-2F	50.52	0.21	0.21	0.09	0.09	31.36	9.28	22.33	9.28
RM-4F	63.45	0.22	0.22	0.09	0.09	29.92	11.94	28.31	11.94
RM-4H	53.96	0.21	0.21	0.09	0.09	32.37	9.79	23.72	9.79

Notes:

refer to Table 3.0 for key to beam designations

V_p = plastic shear capacity of unperforated web
α_t = expression used in Eq. 2.76
α_b = expression used in Eq. 2.76
$term_1$ = first part of Eq. 2.76 for unreinforced beams
Eq. 2.78/(1 - h_o/d) for reinforced beams
$term_2$ = second part of Eq. 2.76 for unreinforced beams,
0.0 for reinforced beams
V_a = $(term_1 + term_2) * V_p$
V_m = maximum permissible shear capacity per Section D.1.2 of this report
V = governing shear capacity

Table 3.12 Composite Beam Shear Capacity Summary: Redwood and Poumbouras (1984)

Test	C_o (k)	C_1 (k)	C_2 (k)	$k5$	$k6$	μ	γ	V_t (k)	V_b (k)	V_m (k)	M_v (in.-k)
D-1	364.75	173.28	103.97	0.48	0.29	2.21	5.92	29.00	12.95	41.96	2478.96
D-2	395.76	214.79	141.11	0.54	0.36	2.64	6.05	29.27	12.34	41.61	2312.13
D-3	440.64	215.37	135.50	0.49	0.31	2.86	6.03	30.48	12.25	42.73	2360.26
D-5A	386.78	179.25	128.04	0.46	0.33	2.04	5.94	26.40	12.37	38.77	2407.58
D-5B	415.34	222.68	146.27	0.54	0.35	2.74	6.02	29.93	3.40	33.33	2941.50
D-6A	328.03	192.03	128.02	0.59	0.39	2.24	6.01	27.01	12.37	39.37	2486.16
D-6B	438.60	224.28	114.33	0.51	0.26	3.44	6.01	34.00	12.37	46.37	2596.25
D-7A	427.38	164.31	29.53	0.38	0.07	4.81	6.15	31.14	9.91	41.05	1882.26
D-7B	438.60	165.05	107.79	0.38	0.25	2.91	6.07	23.94	10.09	34.03	1748.31
D-8A	401.88	72.42	43.23	0.18	0.11	8.28	5.64	14.20	4.15	18.35	494.13
D-9A	510.41	154.65	45.18	0.30	0.09	11.11	8.36	25.70	5.21	30.91	2296.02
D-9B	711.55	157.33	100.74	0.22	0.14	8.45	4.80	26.99	7.74	34.73	2256.19
D-8B	401.88	65.72	43.82	0.16	0.11	7.58	9.20	12.41	1.85	14.26	501.24

Notes:

refer to Table 3.0 for key to beam designations

C_o = full compressive resistance of the slab
C_1 = compressive force in concrete at the high moment end of the opening
C_2 = compressive force in concrete at the low moment end of the opening
$k5$ = C_1/C_o
$k6$ = C_2/C_o
μ = term relating the internal moments of compression forces at the ends of the opening and C_o to the plastic shear capacity of the tee and tee depth
γ = opening length/tee depth
V_b = bottom tee shear capacity
V_t = top tee shear capacity
V_m = maximum shear capacity without moment interaction
M_v = maximum moment capacity without shear interaction

Table 3.13 Steel Beam Capacity Summary: Method I, $\lambda = 1.414$

Test[1]	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Unreinforced							
Circular Opening							
RBD-C1	2945.79	82.99	2046.38	98.17	1626.80	78.04	1.258
RM-1A	716.71	29.76	728.13	0.00	716.71	0.00	1.016
RM-1B	711.06	15.32	712.13	0.00	711.06	0.00	1.002
RM-2A	798.33	31.88	575.54	31.96	516.63	28.69	1.114
RM-2B	660.89	12.82	295.54	16.41	227.64	12.64	1.298
RM-2C	606.23	23.76	480.54	26.69	386.99	21.49	1.242
RM-3A	713.87	30.05	619.49	20.63	624.29	20.79	0.992
RM-4A	720.71	30.04	691.77	14.38	693.16	14.41	0.998
RM-4B	695.17	15.26	553.77	11.50	566.64	11.77	0.977
RM-4C	701.33	32.71	672.77	13.98	681.79	14.17	0.987
				Mean			1.088
				Coefficient of Variation			0.119
				Resistance Factor			0.889
Rectangular Opening							
B-1	2303.02	43.18	945.00	47.22	849.48	42.45	1.112
B-2	2171.63	39.36	1704.80	42.56	1415.15	35.33	1.205
B-3	2081.90	37.20	1.80	49.74	1.35	37.20	1.337
B-4	2207.88	49.06	1003.00	50.12	954.61	47.70	1.051
CL-4B	3555.94	18.03	1000.00	27.80	647.36	18.00	1.545
CR-6A	2564.72	38.27	1212.37	55.07	832.78	37.83	1.456
CSK-1	3388.11	67.95	2358.39	78.54	1910.29	63.62	1.235
DO-1	725.03	22.53	392.95	24.94	342.11	21.71	1.149
DO-2	674.35	8.00	182.69	11.59	125.79	7.98	1.452
DO-3	691.24	24.19	622.36	19.73	574.31	18.21	1.084
DO-4	698.68	13.93	496.99	15.75	408.08	12.93	1.218
DO-5	674.35	13.45	728.74	0.00	674.35	0.00	1.081
RBD-R1B	3033.59	82.14	1718.81	85.06	1577.91	78.09	1.089
RBD-R2	2925.66	56.58	1269.70	59.86	1173.71	55.33	1.082
RM-11H	749.12	15.96	772.33	0.00	749.12	0.00	1.031
RM-21H	618.27	10.71	342.82	14.27	251.37	10.46	1.364
RM-2F	629.18	10.80	284.54	15.80	192.58	10.69	1.477
RM-4F	733.86	13.62	566.77	11.77	548.14	11.38	1.034
RM-4H	637.98	11.25	483.77	10.04	462.12	9.59	1.047
				Mean			1.213
				Coefficient of Variation			0.142
				Resistance Factor			0.963
Overall Unreinforced							
			Mean				1.170
			Coefficient of Variation				0.143
			Resistance Factor				0.928

Table 3.13 (continued)

Test[1]	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Reinforced							
Rectangular Opening							
CR-1A	1079.61	28.73	914.16	21.22	911.10	21.15	1.003
CR-2A	2362.84	55.42	1542.37	70.07	1168.65	53.09	1.320
CR-2B	2362.84	55.42	2331.35	51.74	1925.81	42.74	1.211
CR-2C	2773.20	66.01	2112.23	70.36	1786.40	59.51	1.182
CR-2D	2773.20	66.01	1404.33	82.53	1099.34	64.61	1.277
CR-3A	2362.84	55.42	1707.37	77.57	1168.60	53.09	1.461
CR-3B	2773.20	70.94	2704.85	60.10	2344.37	52.09	1.154
CR-4A	2773.20	53.27	1487.37	67.57	1144.54	52.00	1.300
CR-4B	2773.20	53.27	2313.35	51.34	2032.05	45.10	1.138
CR-5A	2773.20	47.34	1554.23	51.76	1362.83	45.39	1.140
CR-7B	2501.55	62.79	2448.35	54.34	2099.57	46.60	1.166
CR-7D	2501.55	59.86	1319.33	77.58	996.03	58.57	1.325
CSK-2	3680.73	92.96	2872.39	95.69	2473.48	82.40	1.161
CSK-5	3141.22	78.78	2309.50	76.93	2100.71	69.98	1.099
CSK-6	3043.56	51.36	1471.10	48.99	1480.76	49.31	0.993
CSK-7	3043.56	60.30	1780.10	59.29	1698.78	56.58	1.048
CS-1	2137.01	43.95	1811.25	30.08	1856.00	30.82	0.976
CS-2	2155.60	42.31	1772.25	29.43	1840.91	30.57	0.963
CS-3	2095.06	43.84	1604.00	40.03	1505.14	37.56	1.066
RL-5	2667.74	42.60	2893.50	0.00	2667.74	0.00	1.085
RL-6	2701.97	23.69	1048.89	21.36	1133.79	23.09	0.925
				Mean			1.143
				Coefficient of Variation			0.121
				Resistance Factor			0.932
Overall Steel Beams							
				Mean			1.158
				Coefficient of Variation			0.134
				Resistance Factor			0.929

Notes:

(1) refer to Table 3.0 for key to beam designations

Table 3.14 Composite Beam Capacity Summary: Method I, λ = 1.414

Test[1]	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Unreinforced							
Ribbed Slab							
D-1	5405.49	43.73	1606.00	37.80	1833.32	43.15	0.876
D-2	5967.14	42.44	3095.00	39.00	3187.38	40.16	0.971
D-3	6096.61	43.64	6075.00	11.30	6061.36	11.27	1.002
D-5A	5388.57	39.33	2768.00	34.60	2961.53	37.02	0.935
D-5B	5226.80	33.67	2568.00	32.20	2573.89	32.27	0.998
D-6A	5422.56	40.52	0.00	41.00	0.00	40.52	1.012
D-6B	5733.80	55.92	2070.00	48.90	2314.16	54.67	0.894
D-7A	4665.32	45.40	1845.00	43.50	1882.53	44.38	0.980
D-7B	4362.93	42.62	3379.00	42.60	2976.26	37.52	1.135
D-8A	1344.57	21.97	774.00	19.40	807.98	20.25	0.958
D-8B	1056.25	14.76	427.00	14.30	430.55	14.42	0.992 (2)
D-9A	4791.16	36.88	1474.00	34.50	1557.46	36.45	0.946
D-9B	4588.71	49.00	1755.00	47.30	1781.80	48.02	0.985
R-0	1288.17	21.78	752.00	18.20	816.12	19.75	0.921
R-1	2630.28	25.70	978.00	26.00	951.08	25.28	1.028
R-2	3516.43	29.71	2904.00	28.70	2557.21	25.27	1.136
R-3	3774.33	40.46	3993.00	16.40	3706.13	15.22	1.077
R-4	3022.68	26.16	3212.00	13.10	2924.05	11.93	1.098
R-5	2791.69	29.85	1038.00	27.60	1099.37	29.23	0.944
R-6	2594.94	20.80	786.00	21.20	764.39	20.62	1.028
R-7	2833.28	31.19	1134.00	30.50	1134.13	30.50	1.000
R-8	2817.84	30.15	1075.00	28.90	1098.11	29.52	0.979
			Mean				0.995
			Coefficient of Variation				0.071
			Resistance Factor				0.856
Solid Slab							
C-1	3110.10	36.52	2886.00	33.40	2486.39	28.78	1.161
C-2	4604.48	41.15	4107.00	36.80	3649.89	32.70	1.125
C-3	4624.92	41.84	5468.00	14.00	4590.49	11.75	1.191
C-4	4900.59	47.78	1723.00	47.60	1705.04	47.10	1.011
C-5	5138.23	47.40	3511.00	48.10	3165.98	43.37	1.109
C-6	3188.26	39.65	1471.00	40.40	1401.73	38.50	1.049
G-1	1734.13	28.69	791.00	32.70	679.72	28.10	1.164
G-2	1712.64	28.71	1296.00	26.50	1212.95	24.80	1.068
CHO-3	1369.30	32.15	634.00	35.70	557.83	31.41	1.137
CHO-4	2356.96	49.24	1477.00	46.70	1431.12	45.25	1.032
CHO-5	2444.36	49.09	2319.00	17.90	2399.78	18.52	0.966
			Mean				1.092
			Coefficient of Variation				0.066
			Resistance Factor				0.943
Overall Unreinforced							
			Mean				1.028
			Coefficient of Variation				0.081
			Resistance Factor				0.876

Table 3.14 (continued)

Test[1]	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Reinforced							
Ribbed Slab							
WJE-1	7782.56	37.85	7155.63	0.00	7782.56	0.00	0.919
Solid Slab							
CHO-6	1745.52	37.52	721.00	40.60	654.36	36.85	1.102
CHO-7	2983.27	56.46	2664.00	20.60	2918.36	22.57	0.913
			Mean				1.008
			Coefficient of Variation				0.133
			Resistance Factor				0.810
Overall Reinforced							
			Mean				0.978
			Coefficient of Variation				0.110
			Resistance Factor				0.808
Overall Composite Beams							
			Mean				1.024
			Coefficient of Variation				0.084
			Resistance Factor				0.870

Notes:

(1) refer to Table 3.0 for key to beam designations
(2) excluded form analysis (see Appendix E)

Table 3.15 Steel Beam Capacity Summary: Method II

Test[1]	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Unreinforced							
Circular Opening							
RBD-C1	2945.79	82.99	2046.38	98.17	1626.80	78.04	1.258
RM-1A	716.71	25.81	728.13	0.00	716.71	0.00	1.016
RM-1B	711.06	13.93	712.13	0.00	711.06	0.00	1.002
RM-2A	798.33	27.65	575.54	31.96	463.14	25.72	1.243
RM-2B	660.89	11.38	295.54	16.41	202.94	11.27	1.456
RM-2C	606.23	20.56	480.54	26.69	345.66	19.20	1.390
RM-3A	713.87	26.11	619.49	20.63	591.81	19.71	1.047
RM-4A	720.71	26.06	691.77	14.38	680.09	14.14	1.017
RM-4B	695.17	13.93	553.77	11.50	541.82	11.25	1.022
RM-4C	701.33	29.12	672.77	13.98	674.27	14.01	0.998
				Mean			1.145
				Coefficient of Variation			0.154
				Resistance Factor			0.895
Rectangular Opening							
B-1	2303.02	42.93	945.00	47.22	844.70	42.21	1.119
B-2	2171.63	38.87	1704.80	42.56	1402.40	35.01	1.216
B-3	2081.90	36.72	1.80	49.74	1.33	36.72	1.355
B-4	2207.88	48.84	1003.00	50.12	950.62	47.50	1.055
CL-4B	3555.94	14.72	1000.00	27.80	528.86	14.70	1.891
CR-6A	2564.72	36.43	1212.37	55.07	793.91	36.06	1.527
CSK-1	3388.11	64.67	2358.39	78.54	1833.36	61.06	1.286
DO-1	725.03	21.84	392.95	24.94	332.63	21.11	1.181
DO-2	674.35	6.29	182.69	11.59	99.05	6.28	1.844
DO-3	691.24	23.32	622.36	19.73	565.20	17.92	1.101
DO-4	698.68	12.52	496.99	15.75	373.66	11.84	1.330
DO-5	674.35	11.30	728.74	0.00	674.35	0.00	1.081
RBD-R1B	3033.59	82.14	1718.81	85.06	1577.91	78.09	1.089
RBD-R2	2925.66	56.53	1269.70	59.86	1172.67	55.29	1.083
RM-11H	749.12	14.61	772.33	0.00	749.12	0.00	1.031
RM-21H	618.27	9.25	342.82	14.27	218.95	9.11	1.566
RM-2F	629.18	9.29	284.54	15.80	166.21	9.23	1.712
RM-4F	733.86	11.94	566.77	11.77	504.51	10.48	1.123
RM-4H	637.98	9.79	483.77	10.04	421.11	8.74	1.149
				Mean			1.302
				Coefficient of Variation			0.211
				Resistance Factor			0.939
Overall Unreinforced							
				Mean			1.248
				Coefficient of Variation			0.203
				Resistance Factor			0.911

Table 3.15 (continued)

Test[1]	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Reinforced							
Rectangular Opening							
CR-1A	1079.61	28.32	914.16	21.22	905.81	21.03	1.009
CR-2A	2362.84	54.38	1542.37	70.07	1149.29	52.21	1.342
CR-2B	2362.84	54.38	2331.35	51.74	1908.90	42.36	1.221
CR-2C	2773.20	63.27	2112.23	70.36	1730.93	57.66	1.220
CR-2D	2773.20	63.27	1404.33	82.53	1056.38	62.08	1.329
CR-3A	2362.84	54.04	1707.37	77.57	1142.88	51.92	1.494
CR-3B	2773.20	69.84	2704.85	60.10	2329.68	51.76	1.161
CR-4A	2773.20	53.12	1487.37	67.57	1141.50	51.86	1.303
CR-4B	2773.20	53.12	2313.35	51.34	2028.51	45.02	1.140
CR-5A	2773.20	45.34	1554.23	51.76	1311.57	43.68	1.185
CR-7B	2501.55	59.00	2448.35	54.34	2043.93	45.36	1.198
CR-7D	2501.55	56.99	1319.33	77.58	951.15	55.93	1.387
CSK-2	3680.73	92.26	2872.39	95.69	2460.48	81.97	1.167
CSK-5	3141.22	76.55	2309.50	76.93	2058.36	68.56	1.122
CSK-6	3043.56	50.10	1471.10	48.99	1448.36	48.23	1.016
CSK-7	3043.56	57.28	1780.10	59.29	1627.24	54.20	1.094
CS-1	2137.01	42.53	1811.25	30.08	1834.33	30.46	0.987
CS-2	2155.60	40.95	1772.25	29.43	1817.55	30.18	0.975
CS-3	2095.06	41.50	1604.00	40.03	1452.76	36.26	1.104
RL-5	2667.74	42.79	2893.50	0.00	2667.74	0.00	1.085
RL-6	2701.97	23.42	1048.89	21.36	1121.93	22.85	0.935
				Mean			1.166
				Coefficient of Variation			0.125
				Resistance Factor			0.946
Overall Steel Beams							
			Mean				1.213
			Coefficient of Variation				0.179
			Resistance Factor				0.916

Notes:

(1) refer to Table 3.0 for key to beam designations

Table 3.16 Composite Beam Capacity Summary: Method II

Test[1]	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Unreinforced							
Ribbed Slab							
D-1	5405.49	41.96	1606.00	37.80	1761.88	41.47	0.912
D-2	5967.14	41.61	3095.00	39.00	3134.09	39.49	0.988
D-3	6096.61	42.73	6075.00	11.30	6059.10	11.27	1.003
D-5A	5388.57	38.77	2768.00	34.60	2926.49	36.58	0.946
D-5B	5226.80	33.33	2568.00	32.20	2550.56	31.98	1.007
D-6A	5422.56	39.37	0.00	41.00	0.00	39.37	1.041
D-6B	5733.80	53.41	2070.00	48.90	2216.70	52.37	0.934
D-7A	4665.32	42.68	1845.00	43.50	1776.25	41.88	1.039
D-7B	4362.93	41.45	3379.00	42.60	2919.52	36.81	1.157
D-8A	1344.57	20.98	774.00	19.40	778.78	19.52	0.994
D-8B	1056.25	14.26	427.00	14.30	416.89	13.96	1.024 (2)
D-9A	4791.16	34.36	1474.00	34.50	1454.22	34.04	1.014
D-9B	4588.71	46.35	1755.00	47.30	1690.66	45.57	1.038
R-0	1288.17	20.85	752.00	18.20	789.46	19.11	0.953
R-1	2630.28	23.82	978.00	26.00	884.35	23.51	1.106
R-2	3516.43	28.57	2904.00	28.70	2494.73	24.66	1.164
R-3	3774.33	37.44	3993.00	16.40	3689.03	15.15	1.082
R-4	3022.68	25.58	3212.00	13.10	2917.62	11.90	1.101
R-5	2791.69	27.78	1038.00	27.60	1026.99	27.31	1.011
R-6	2594.94	19.10	786.00	21.20	703.59	18.98	1.117
R-7	2833.28	28.49	1134.00	30.50	1041.39	28.01	1.089
R-8	2817.84	27.16	1075.00	28.90	995.18	26.75	1.080
				Mean			1.037
				Coefficient of Variation			0.069
				Resistance Factor			0.893
Solid Slab							
C-1	3110.10	32.99	2886.00	33.40	2356.44	27.27	1.225
C-2	4604.48	38.90	4107.00	36.80	3544.21	31.76	1.159
C-3	4624.92	39.66	5468.00	14.00	4584.59	11.74	1.193
C-4	4900.59	45.31	1723.00	47.60	1620.15	44.76	1.063
C-5	5138.23	45.33	3511.00	48.10	3058.00	41.89	1.148
C-6	3188.26	36.16	1471.00	40.40	1287.01	35.35	1.143
G-1	1734.13	26.11	791.00	32.70	621.76	25.70	1.272
G-2	1712.64	26.13	1296.00	26.50	1138.24	23.27	1.139
CHO-3	1369.30	30.74	634.00	35.70	534.82	30.12	1.185
CHO-4	2356.96	47.76	1477.00	46.70	1397.35	44.18	1.057
CHO-5	2444.36	47.61	2319.00	17.90	2395.66	18.49	0.968
				Mean			1.141
				Coefficient of Variation			0.075
				Resistance Factor			0.978
Overall Unreinforced							
				Mean			1.073
				Coefficient of Variation			0.084
				Resistance Factor			0.912

Table 3.16 (continued)

Test[1]	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Reinforced							
Ribbed Slab							
WJE-1	7782.56	38.28	7155.63	0.00	7782.56	0.00	0.919
Solid Slab							
CHO-6	1745.52	36.75	721.00	40.60	641.64	36.13	1.124
CHO-7	2983.27	55.84	2664.00	20.60	2916.28	22.55	0.913
			Mean				1.019
			Coefficient of Variation				0.146
			Resistance Factor				0.805
Overall Reinforced							
			Mean				0.985
			Coefficient of Variation				0.122
			Resistance Factor				0.802
Overall Composite Beams							
			Mean				1.065
			Coefficient of Variation				0.088
			Resistance Factor				0.901

Notes:

(1) refer to Table 3.0 for key to beam designations
(2) excluded from analysis (see Appendix E)

Table 3.17 Steel Beam Capacity Summary: Method III, λ = 1.414

Test[1]	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Unreinforced							
Circular Opening							
RBD-C1	2945.79	82.99	2046.38	98.17	1626.80	78.04	1.258
RM-1A	716.71	26.74	728.13	0.00	716.71	0.00	1.016
RM-1B	711.06	14.54	712.13	0.00	711.06	0.00	1.002
RM-2A	798.33	28.64	575.54	31.96	476.33	26.45	1.208
RM-2B	660.89	11.90	295.54	16.41	211.99	11.77	1.394
RM-2C	606.23	21.24	480.54	26.69	354.93	19.71	1.354
RM-3A	713.87	27.04	619.49	20.63	600.58	20.00	1.031
RM-4A	720.71	26.99	691.77	14.38	683.76	14.21	1.012
RM-4B	695.17	14.54	553.77	11.50	553.66	11.50	1.000
RM-4C	701.33	30.17	672.77	13.98	676.80	14.06	0.994
				Mean			1.127
				Coefficient of Variation			0.142
				Resistance Factor			0.895
Rectangular Opening							
B-1	2303.02	43.13	945.00	47.22	848.59	42.40	1.114
B-2	2171.63	39.02	1704.80	42.56	1406.27	35.11	1.212
B-3	2081.90	36.89	1.80	49.74	1.33	36.89	1.348
B-4	2207.88	48.83	1003.00	50.12	950.55	47.50	1.055
CL-4B	3555.94	17.23	1000.00	27.80	618.82	17.20	1.616
CR-6A	2564.72	37.55	1212.37	55.07	817.70	37.14	1.483
CSK-1	3388.11	65.70	2358.39	78.54	1857.84	61.87	1.269
DO-1	725.03	22.23	392.95	24.94	338.00	21.45	1.163
DO-2	674.35	7.17	182.69	11.59	112.81	7.16	1.619
DO-3	691.24	23.70	622.36	19.73	569.22	18.05	1.093
DO-4	698.68	13.48	496.99	15.75	397.53	12.60	1.250
DO-5	674.35	11.68	728.74	0.00	674.35	0.00	1.081
RBD-R1B	3033.59	82.14	1718.81	85.06	1577.91	78.09	1.089
RBD-R2	2925.66	56.67	1269.70	59.86	1175.43	55.42	1.080
RM-11H	749.12	15.55	772.33	0.00	749.12	0.00	1.031
RM-21H	618.27	9.90	342.82	14.27	233.43	9.72	1.469
RM-2F	629.18	9.91	284.54	15.80	177.10	9.83	1.607
RM-4F	733.86	12.71	566.77	11.77	525.52	10.91	1.078
RM-4H	637.98	10.45	483.77	10.04	440.82	9.15	1.097
				Mean			1.250
				Coefficient of Variation			0.167
				Resistance Factor			0.960
Overall Unreinforced							
				Mean			1.208
				Coefficient of Variation			0.165
				Resistance Factor			0.930

Table 3.17 (continued)

Test[1]	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Reinforced							
Rectangular Opening							
CR-1A	1079.61	28.47	914.16	21.22	907.83	21.07	1.007
CR-2A	2362.84	55.42	1542.37	70.07	1168.65	53.09	1.320
CR-2B	2362.84	55.42	2331.35	51.74	1925.81	42.74	1.211
CR-2C	2773.20	65.10	2112.23	70.36	1768.33	58.90	1.194
CR-2D	2773.20	65.10	1404.33	82.53	1085.19	63.77	1.294
CR-3A	2362.84	55.42	1707.37	77.57	1168.60	53.09	1.461
CR-3B	2773.20	70.94	2704.85	60.10	2344.37	52.09	1.154
CR-4A	2773.20	53.17	1487.37	67.57	1142.54	51.90	1.302
CR-4B	2773.20	53.17	2313.35	51.34	2029.72	45.05	1.140
CR-5A	2773.20	46.12	1554.23	51.76	1331.72	44.35	1.167
CR-7B	2501.55	61.83	2448.35	54.34	2086.21	46.30	1.174
CR-7D	2501.55	59.00	1319.33	77.58	982.66	57.78	1.343
CSK-2	3680.73	92.96	2872.39	95.69	2473.48	82.40	1.161
CSK-5	3141.22	77.91	2309.50	76.93	2084.50	69.44	1.108
CSK-6	3043.56	50.89	1471.10	48.99	1468.61	48.91	1.002
CSK-7	3043.56	59.47	1780.10	59.29	1679.31	55.93	1.060
CS-1	2137.01	43.95	1811.25	30.08	1856.00	30.82	0.976
CS-2	2155.60	42.31	1772.25	29.43	1840.91	30.57	0.963
CS-3	2095.06	43.80	1604.00	40.03	1504.35	37.54	1.066
RL-5	2667.74	43.01	2893.50	0.00	2667.74	0.00	1.085
RL-6	2701.97	23.63	1048.89	21.36	1131.03	23.03	0.927
				Mean			1.148
				Coefficient of Variation			0.122
				Resistance Factor			0.935
Overall Steel Beams							
			Mean				1.183
			Coefficient of Variation				0.150
			Resistance Factor				0.930

Notes:

(1) refer to Table 3.0 for key to beam designations

Table 3.18 Composite Beam Capacity Summary: Method III, $\lambda = 1.414$

Test[1]	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Unreinforced							
Ribbed Slab							
D-1	5405.49	43.95	1606.00	37.80	1842.19	43.36	0.872
D-2	5967.14	43.41	3095.00	39.00	3248.86	40.94	0.953
D-3	6096.61	44.49	6075.00	11.30	6063.33	11.28	1.002
D-5A	5388.57	40.73	2768.00	34.60	3048.68	38.11	0.908
D-5B	5226.80	34.17	2568.00	32.20	2607.65	32.70	0.985
D-6A	5422.56	41.27	0.00	41.00	0.00	41.27	0.994
D-6B	5733.80	56.16	2070.00	48.90	2323.46	54.89	0.891
D-7A	4665.32	45.24	1845.00	43.50	1876.15	44.23	0.983
D-7B	4362.93	43.45	3379.00	42.60	3015.41	38.02	1.121
D-8A	1344.57	21.53	774.00	19.40	795.00	19.93	0.974
D-8B	1056.25	14.87	427.00	14.30	433.54	14.52	0.985 (2)
D-9A	4791.16	35.38	1474.00	34.50	1496.23	35.02	0.985
D-9B	4588.71	47.28	1755.00	47.30	1722.63	46.43	1.019
R-0	1288.17	21.43	752.00	18.20	806.35	19.52	0.933
R-1	2630.28	24.79	978.00	26.00	919.08	24.43	1.064
R-2	3516.43	30.02	2904.00	28.70	2573.65	25.44	1.128
R-3	3774.33	38.38	3993.00	16.40	3694.93	15.18	1.081
R-4	3022.68	25.56	3212.00	13.10	2928.20	11.94	1.097
R-5	2791.69	28.73	1038.00	27.60	1060.45	28.20	0.979
R-6	2594.94	20.22	786.00	21.20	743.84	20.06	1.057
R-7	2833.28	29.35	1134.00	30.50	1071.24	28.81	1.059
R-8	2817.84	27.98	1075.00	28.90	1023.71	27.52	1.050
			Mean				1.006
			Coefficient of Variation				0.072
			Resistance Factor				0.864
Solid Slab							
C-1	3110.10	33.71	2886.00	33.40	2385.07	27.60	1.210
C-2	4604.48	41.53	4107.00	36.80	3666.45	32.85	1.120
C-3	4624.92	42.52	5468.00	14.00	4592.08	11.76	1.191
C-4	4900.59	48.71	1723.00	47.60	1736.49	47.97	0.992
C-5	5138.23	48.58	3511.00	48.10	3225.35	44.19	1.089
C-6	3188.26	37.08	1471.00	40.40	1317.69	36.19	1.116
G-1	1734.13	26.54	791.00	32.70	631.44	26.10	1.253
G-2	1712.64	26.56	1296.00	26.50	1151.25	23.54	1.126
CHO-3	1369.30	31.11	634.00	35.70	540.89	30.46	1.172
CHO-4	2356.96	48.52	1477.00	46.70	1414.91	44.74	1.044
CHO-5	2444.36	48.37	2319.00	17.90	2397.84	18.51	0.967
			Mean				1.116
			Coefficient of Variation				0.080
			Resistance Factor				0.952
Overall Unreinforced							
			Mean				1.044
			Coefficient of Variation				0.090
			Resistance Factor				0.882

Table 3.18 (continued)

Test[1]	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Reinforced							
Ribbed Slab							
WJE-1	7782.56	38.31	7155.63	0.00	7782.56	0.00	0.919
Solid Slab							
CHO-6	1745.52	36.93	721.00	40.60	644.63	36.30	1.118
CHO-7	2983.27	55.85	2664.00	20.60	2916.30	22.55	0.913
						Mean	1.016
						Coefficient of Variation	0.143
						Resistance Factor	0.806
Overall Reinforced							
						Mean	0.983
						Coefficient of Variation	0.119
						Resistance Factor	0.803
Overall Composite Beams							
						Mean	1.039
						Coefficient of Variation	0.092
						Resistance Factor	0.875

Notes:

(1) refer to Table 3.0 for key to beam designations
(2) excluded from analysis (see Appendix E)

Table 3.19 Steel Beam Capacity Summary: Redwood and Shrivastava (1980)

						Curvilinear			Linear		
Test[(1)]	M_m (in.-k)	V_m (k)	M_v (in.-k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory	M_n (in-k)	V_n (k)	Test/ Theory
Unreinforced											
Circular Opening											
RBD-C1	2945.79	83.87	1987.91	2046.38	98.17	1748.30	83.87	1.170	1748.30	83.87	1.170
RM-1A	716.71	25.80	487.74	728.13	0.00	716.71	0.00	1.016	716.71	0.00	1.016
RM-1B	711.06	13.98	398.60	712.13	0.00	711.06	0.00	1.002	711.06	0.00	1.002
RM-2A	798.33	27.64	553.50	575.54	31.96	497.77	27.64	1.156	497.77	27.64	1.156
RM-2B	660.89	11.38	403.67	295.54	16.41	204.94	11.38	1.442	204.94	11.38	1.442
RM-2C	606.23	20.56	422.86	480.54	26.69	370.10	20.56	1.298	370.10	20.56	1.298
RM-3A	713.87	26.10	482.17	619.49	20.63	627.02	20.88	0.988	447.88	14.92	1.383
RM-4A	720.71	26.05	489.50	691.77	14.38	668.72	13.90	1.034	484.81	10.08	1.427
RM-4B	695.17	13.93	383.82	553.77	11.50	559.48	11.62	0.990	403.76	8.38	1.372
RM-4C	701.33	29.12	441.90	672.77	13.98	665.33	13.83	1.011	491.05	10.20	1.370
						Mean		1.111			1.264
						Coefficient of Variation		0.140			0.131
						Resistance Factor		0.885			1.018
Rectangular Opening											
B-1	2303.02	42.93	965.56	945.00	47.22	859.07	42.93	1.100	859.07	42.93	1.100
B-2	2171.63	38.87	999.47	1704.80	42.56	1364.53	34.07	1.249	1004.81	25.70	1.656
B-3	2081.90	36.72	952.77	1.80	49.74	1.33	36.72	1.355	1.33	36.72	1.355
B-4	2207.88	48.84	946.42	1003.00	50.12	976.41	48.79	1.027	936.42	48.84	1.026
CL-4B	3555.94	19.79	1811.28	1000.00	27.80	711.93	19.79	1.405	711.93	19.79	1.405
CR-6A	2564.72	36.43	1252.56	1212.37	55.07	801.92	36.43	1.512	801.92	36.43	1.512
CSK-1	3388.11	64.67	1788.75	2358.39	78.54	1863.39	62.06	1.266	1501.53	49.53	1.586
DO-1	725.03	21.84	357.02	392.95	24.94	344.08	21.84	1.142	344.08	21.84	1.142
DO-2	674.35	6.28	425.78	182.69	11.59	99.04	6.28	1.845	99.04	6.28	1.845
DO-3	691.24	23.32	307.84	622.36	19.73	528.17	16.74	1.178	373.51	12.00	1.644
DO-4	698.68	12.52	344.64	496.99	15.75	373.68	11.84	1.330	294.29	9.60	1.641
DO-5	674.35	11.30	439.79	728.74	0.00	674.35	0.00	1.081	674.35	0.00	1.081
RBD-R1B	3033.59	83.01	1647.23	1718.81	85.06	1675.21	82.90	1.026	1596.88	83.01	1.025
RBD-R2	2925.66	56.58	1212.79	1269.70	59.86	1200.11	56.58	1.058	1200.11	56.58	1.058
RM-11H	749.12	14.61	370.85	772.33	0.00	749.12	0.00	1.031	749.12	0.00	1.031
RM-21H	618.27	9.25	372.26	342.82	14.27	222.29	9.25	1.542	222.29	9.25	1.542
RM-2F	629.18	9.28	385.47	284.54	15.80	167.15	9.28	1.702	167.15	9.28	1.702
RM-4F	733.86	11.94	424.03	566.77	11.77	520.77	10.81	1.088	391.77	8.66	1.360
RM-4H	637.98	9.79	378.85	483.77	10.04	438.59	9.10	1.103	338.09	7.38	1.360
						Mean		1.265			1.391
						Coefficient of Variation		0.191			0.195
						Resistance Factor		0.939			1.027
Overall Unreinforced											
						Mean		1.212			1.347
						Coefficient of Variation		0.186			0.182
						Resistance Factor		0.939			1.013

Table 3.19 (continued)

						Curvilinear			Linear		
Test[1]	M_m (in.-k)	V_m (k)	M_v (in.-k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory	M_n (in.-k)	V_n (k)	Test/ Theory
Reinforced											
CR-1A	1079.61	35.29	548.11	914.16	21.22	999.89	23.21	0.914	708.65	16.45	1.290
CR-2A	2362.84	52.38	1190.83	1542.37	70.07	1152.98	52.38	1.338	1152.98	52.38	1.338
CR-2B	2362.84	52.38	1190.83	2331.35	51.74	1681.32	37.31	1.387	1188.91	26.39	1.961
CR-2C	2773.20	68.53	1485.58	2112.23	70.36	1858.91	61.92	1.136	1395.64	46.49	1.513
CR-2D	2773.20	68.53	1485.58	1404.33	82.53	1166.02	68.53	1.204	1166.02	68.53	1.204
CR-3A	2362.84	52.38	1190.83	1707.37	77.57	1152.92	52.38	1.481	1152.92	52.38	1.481
CR-3B	2773.20	70.94	1964.94	2704.85	60.10	2168.61	48.19	1.257	1534.59	34.10	1.763
CR-4A	2773.20	51.22	1485.58	1487.37	67.57	1127.52	51.22	1.319	1127.52	51.22	1.319
CR-4B	2773.20	51.22	1485.58	2313.35	51.34	1942.77	43.12	1.191	1406.15	31.21	1.645
CR-5A	2773.20	51.29	1333.02	1554.23	51.76	1522.62	50.71	1.021	1336.70	44.52	1.163
CR-7B	2501.55	61.75	1267.75	2448.35	54.34	1883.35	41.80	1.300	1332.90	29.58	1.837
CR-7D	2501.55	61.75	1267.75	1319.33	77.58	1050.14	61.75	1.256	1050.14	61.75	1.256
CSK-2	3680.73	92.96	2191.87	2872.39	95.69	2550.28	84.96	1.126	1932.36	64.37	1.486
CSK-5	3141.22	74.59	1651.52	2309.50	76.93	2058.46	68.57	1.122	1567.85	52.23	1.473
CSK-6	3043.56	44.77	1761.75	1471.10	48.99	1344.26	44.77	1.094	1344.26	44.77	1.094
CSK-7	3043.56	59.69	1761.75	1780.10	59.29	1791.86	59.68	0.993	1766.59	58.84	1.008
CS-1	2137.01	43.95	1420.07	1811.25	30.08	2069.40	34.37	0.875	1472.57	24.45	1.238
CS-2	2155.60	42.31	1424.36	1772.25	29.43	2103.08	34.92	0.843	1513.06	25.13	1.171
CS-3	2095.06	43.84	1418.07	1604.00	40.03	1682.07	41.98	0.954	1350.37	33.70	1.188
RL-5	2667.74	23.51	1585.70	2893.50	0.00	2667.74	0.00	1.085	2667.74	0.00	1.085
RL-6	2701.97	19.55	1201.35	1048.89	21.36	960.03	19.55	1.093	960.03	19.55	1.093
						Mean		1.142			1.362
						Coefficient of Variation		0.151			0.195
						Resistance Factor		0.896			1.006
Overall Steel Beams											
						Mean		1.183			1.353
						Coefficient of Variation		0.174			0.185
						Resistance Factor		0.900			1.013

Notes:

(1) refer to Table 3.0 for key to beam designations

Table 3.20 Composite Beam Capacity Summary: Redwood et al.

Test(1)	M_m (in.-k)	M_m' (in.-k)	V_m (k)	M_v (in.-k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory[(2)]
Unreinforced									
Ribbed Slab (Redwood and Poumbouras 1984)									
D-1	6214.50	5405.49	41.96	2478.96	1606.00	37.80	1782.74	41.96	0.901
D-2	6243.69	5967.14	41.61	2312.13	3095.00	39.00	3230.04	40.70	0.958
D-3	6409.99	6096.61	42.73	2360.26	6075.00	11.30	6096.61	11.34	0.996
D-5A	6286.92	5388.57	38.77	2407.58	2768.00	34.60	3084.93	38.56	0.897
D-5B	5736.02	5226.80	33.33	2941.50	2568.00	32.20	2658.12	33.33	0.966
D-6A	5990.69	5422.56	39.37	2486.16	0.00	41.00	0.00	39.37	1.041
D-6B	6507.28	5733.80	46.37	2596.25	2070.00	48.90	1962.90	46.37	1.055
D-7A	4937.41	4665.32	41.05	1882.26	1845.00	43.50	1741.09	41.05	1.060
D-7B	4945.59	4362.93	34.03	1748.31	3379.00	42.60	2499.73	31.51	1.352
D-8A	1523.07	1344.57	18.35	494.13	774.00	19.40	709.02	17.77	1.092
D-8B	1428.50	1056.25	14.26	501.24	427.00	14.30	425.81	14.26	1.003 (3)
D-9A	5137.87	4791.16	30.91	2296.02	1474.00	34.50	1320.62	30.91	1.116
D-9B	5169.73	4588.71	34.73	2256.19	1755.00	47.30	1288.61	34.73	1.362
R-0	1488.10	1070.69	19.07	463.42	752.00	18.20	755.73	18.29	0.995
R-1	3330.08	2432.45	23.01	1539.59	978.00	26.00	865.53	23.01	1.130
R-2	3556.57	3347.16	25.29	2235.15	2904.00	28.70	2337.04	23.10	1.243
R-3	3756.96	3756.96	29.96	1463.41	3993.00	16.40	3242.95	13.32	1.231
R-4	3837.03	2921.87	24.70	1775.19	3212.00	13.10	2921.87	11.92	1.099
R-5	3484.91	2587.36	23.52	1213.30	1038.00	27.60	884.56	23.52	1.173
R-6	3243.38	2537.64	19.04	1510.48	786.00	21.20	705.92	19.04	1.113
R-7	3243.38	2689.19	29.93	1642.66	1134.00	30.50	1112.81	29.93	1.019
R-8	3173.49	2651.24	26.20	1740.68	1075.00	28.90	974.57	26.20	1.103
						Mean			1.090
						Coefficient of Variation			0.121
						Resistance Factor			1.889
Solid Slab (Redwood and Cho 1986)									
C-1	2826.10	2543.80	46.29	1463.90	2886.00	33.40	2274.14	26.32	1.269
C-2	4339.60	3950.20	37.32	2012.70	4107.00	36.80	3076.26	27.56	1.335
C-3	4397.20	3499.70	27.86	1757.80	5468.00	14.00	3499.70	8.96	1.562
C-4	4876.00	4370.60	42.58	2157.80	1723.00	47.60	1541.29	42.58	1.118
C-5	4810.50	4302.50	40.42	2251.70	3511.00	48.10	2726.45	37.35	1.288
C-6	3176.60	2650.00	35.59	1488.70	1471.00	40.40	1295.86	35.59	1.135
G-1	1631.20	1269.20	29.23	915.20	791.00	32.70	707.06	29.23	1.119
G-2	1631.20	1503.80	30.10	922.30	1296.00	26.50	1262.97	25.82	1.026
CHO-3	1493.20	1055.00	30.10	675.30	634.00	35.70	534.55	30.10	1.186
CHO-4	2497.80	2091.50	42.83	1080.70	1477.00	46.70	1312.13	41.49	1.126
CHO-5	2497.80	2091.50	42.83	1080.70	2319.00	17.90	2091.50	16.14	1.109
						Mean			1.207
						Coefficient of Variation			0.124
						Resistance Factor			0.981
Overall Unreinforced									
						Mean			1.131
						Coefficient of Variation			0.128
						Resistance Factor			0.914

Table 3.20 Composite Beam Capacity Summary: Redwood et al. (continued)

Test[1]	M_m (in.-k)	M_m' (in.-k)	V_m (k)	M_v (in.-k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory[2]
Reinforced									
CHO-6	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
CHO-7	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
WJE-1	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a

Notes:

(1) refer to Table 3.0 for key to beam designations
(2) curvlinear interaction used
(3) excluded from analysis (see Appendix E)

M_m = moment capacity based on full composite action
M_m' = moment capacity based on actual partial composite action
V_m = maximum shear capacity
M_v = moment at which shear interaction begins to diminish moment capacity

Table 3.21 Analysis Summary, λ = 1.414 (Methods I and III)

		Mean					Coefficient of Variation					Resistance Factor				
	# Beams	I	II	III	Redwood (C)	Redwood (L)	I	II	III	Redwood (C)	Redwood (L)	I	II	III	Redwood (C)	Redwood (L)
STEEL BEAMS																
Unreinforced	29	1.170	1.248	1.208	1.212	1.347	0.143	0.203	0.165	0.186	0.182	0.928	0.911	0.930	0.907	1.013
Rectangular Opening	19	3.213	1.302	1.250	1.265	1.391	0.142	0.211	0.167	0.191	0.195	0.963	0.939	0.960	0.939	1.027
Circular Opening	10	1.088	1.145	1.127	1.111	1.264	0.119	0.154	0.142	0.140	0.131	0.889	0.895	0.895	0.885	1.018
Reinforced																
Rectangular Opening	21	1.143	1.166	1.148	1.142	1.362	0.121	0.125	0.122	0.151	0.195	0.932	0.946	0.935	0.896	1.005
OVERALL STEEL	50	1.158	1.213	1.183	1.183	1.353	0.134	0.179	0.150	0.174	0.185	0.929	0.916	0.930	0.900	1.013
COMPOSITE BEAMS																
Unreinforced	32	1.028	1.073	1.044	1.131	N/A	0.081	0.084	0.090	0.128	N/A	0.676	0.912	0.882	0.914	N/A
Ribbed Slab	21	0.995	1.037	1.006	1.090	N/A	0.071	0.069	0.072	0.121	N/A	0.856	0.893	0.864	0.889	N/A
Solid Slab	11	1.092	1.141	1.116	1.207	N/A	0.065	0.075	0.080	0.124	N/A	0.943	0.978	0.952	0.981	N/A
Reinforced	3	0.978	0.985	0.983	N/A	N/A	0.110	0.122	0.119	N/A	N/A	0.808	0.802	0.803	N/A	N/A
Ribbed Slab	1	0.919	0.919	0.919	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Solid Slab	2	1.008	1.019	1.016	N/A	N/A	0.133	0.146	0.143	N/A	N/A	0.810	0.805	0.806	N/A	N/A
OVERALL COMPOSITE	35	1.024	1.065	1.039	1.131	N/A	0.084	0.088	0.092	0.128	N/A	0.870	0.901	0.876	0.895	N/A

Table 3.22 Effect of Reducing the Tee Depth in Proportion to the Reinforcement Present, Method III, $\lambda = 1.414$

							(2)			(3)		
Test[1]	M_m (in.-k)	V_p (k)	$V_m^{(2)}$ (k)	$V_m^{(3)}$ (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory	M_n (in.-k)	V_n (k)	Test/ Theory
CR-1A	1079.61	35.29	28.95	28.47	914.16	21.22	913.82	21.21	1.000	907.83	21.07	1.007
CR-2A	2362.84	55.42	55.42	55.42	1542.37	70.07	1168.65	53.09	1.320	1168.65	53.09	1.320
CR-2B	2362.84	55.42	55.42	55.42	2331.35	51.74	1925.81	42.74	1.211	1925.81	42.74	1.211
CR-2C	2773.20	70.94	65.87	65.10	2112.23	70.36	1783.70	59.42	1.184	1768.33	58.90	1.194
CR-2D	2773.20	70.94	65.87	65.10	1404.33	82.53	1097.22	64.48	1.280	1085.19	63.77	1.294
CR-3A	2362.84	55.42	55.42	55.42	1707.37	77.57	1168.60	53.09	1.461	1168.60	53.09	1.461
CR-3B	2773.20	70.94	70.94	70.94	2704.85	60.10	2344.37	52.09	1.154	2344.37	52.09	1.154
CR-4A	2773.20	70.94	53.87	53.17	1487.37	67.57	1156.37	52.53	1.286	1142.54	51.90	1.302
CR-4B	2773.20	70.94	53.87	53.17	2313.35	51.34	2045.65	45.40	1.131	2029.72	45.05	1.140
CR-5A	2773.20	51.29	47.55	46.12	1554.23	51.76	1368.32	45.57	1.136	1331.72	44.35	1.167
CR-7B	2501.55	63.21	62.69	61.83	2448.35	54.34	2098.18	46.57	1.167	2086.21	46.30	1.174
CR-7D	2501.55	63.21	59.75	59.00	1319.33	77.58	994.35	58.47	1.327	982.66	57.78	1.343
CSK-2	3680.73	92.96	92.96	92.96	2872.39	95.69	2473.48	82.40	1.161	2473.48	82.40	1.161
CSK-5	3141.22	78.81	78.33	77.91	2309.50	76.93	2092.39	69.70	1.104	2084.50	69.44	1.108
CSK-6	3043.56	63.06	51.41	50.89	1471.10	48.99	1482.00	49.35	0.993	1468.61	48.91	1.002
CSK-7	3043.56	63.06	59.77	59.47	1780.10	59.29	1686.40	56.17	1.056	1679.31	55.93	1.060
CS-1	2137.01	43.95	43.95	43.95	1811.25	30.08	1856.00	30.82	0.976	1856.00	30.82	0.976
CS-2	2155.60	42.31	42.31	42.31	1772.25	29.43	1840.91	30.57	0.963	1840.91	30.57	0.963
CS-3	2095.06	43.84	43.84	43.80	1604.00	40.03	1505.14	37.56	1.066	1504.35	37.54	1.066
RL-5	2667.74	69.49	43.25	43.01	2893.50	0.00	2667.74	0.00	1.085	2667.74	0.00	1.085
RL-6	2701.97	43.20	24.23	23.63	1048.89	21.36	1157.61	23.57	0.906	1131.03	23.03	0.927
							Mean		1.141			1.148
							Coefficient of Variation		0.122			0.122
							Resistance Factor		0.929			0.935
CHO-6	1745.52	21.04	37.16	36.93	721.00	40.60	648.42	36.51	1.112	644.63	36.30	1.118
CHO-7	2983.27	45.07	56.29	55.85	2664.00	20.60	2917.82	22.56	0.913	2916.30	22.55	0.913
WJE-1	7782.56	47.70	39.13	38.31	7155.63	0.00	7782.56	0.00	0.919	7782.56	0.00	0.919
							Mean		0.981			0.983
							Coefficient of Variation		0.115			0.119
							Resistance Factor		0.806			0.803

Notes:

(1) refer to Table 3.0 for key to beam designations
(2) no reduction in tee depth for reinforcement
(3) tee depth reduced (as used in this study)

Table 3.23 Effect of Limiting P_{ch} by the Net Top Tee Steel Method III, $\lambda = 1.414$

						(2)			(3)		
Test[1]	M_m (in.-k)	$V_m^{(2)}$ (k)	$V_m^{(3)}$ (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory	M_n (in.-k)	V_n (k)	Test/ Theory
D-1	5405.49	42.24	43.95	1606.00	37.80	1773.29	41.74	0.906	1842.19	43.36	0.872
D-2	5967.14	40.79	43.41	3095.00	39.00	3081.37	38.83	1.004	3248.86	40.94	0.953
D-3	6096.61	42.02	44.49	6075.00	11.30	6057.19	11.27	1.003	6063.33	11.28	1.002
D-5A	5388.57	40.73	40.73	2768.00	34.60	3048.68	38.11	0.908	3048.68	38.11	0.908
D-5B	5226.80	33.72	34.17	2568.00	32.20	2576.94	32.31	0.997	2607.65	32.70	0.985
D-6A	5422.56	41.27	41.27	0.00	41.00	0.00	41.27	0.994	0.00	41.27	0.994
D-6B	5733.80	58.81	56.16	2070.00	48.90	2424.95	57.29	0.854	2323.46	54.89	0.891
D-7A	4665.32	45.78	45.24	1845.00	43.50	1897.03	44.73	0.973	1876.15	44.23	0.983
D-7B	4362.93	46.48	43.45	3379.00	42.60	3150.04	39.71	1.073	3015.41	38.02	1.121
D-8A	1344.57	21.53	21.53	774.00	19.40	795.00	19.93	0.974	795.00	19.93	0.974
D-9A	4791.16	35.38	35.38	1474.00	34.50	1496.23	35.02	0.985	1496.23	35.02	0.985
D-9B	4588.71	47.28	47.28	1755.00	47.30	1722.63	46.43	1.019	1722.63	46.43	1.019
						Mean		0.974			0.974
						Coefficient of Variation		0.058			0.067
						Resistance Factor		0.845			0.841
R-O	1288.17	21.43	21.43	752.00	18.20	806.35	19.52	0.933	806.35	19.52	0.933
R-1	2630.28	24.79	24.79	978.00	26.00	919.08	24.43	1.064	919.08	24.43	1.064
R-2	3516.43	25.19	30.02	2904.00	28.70	2289.06	22.62	1.269	2573.65	25.44	1.128
R-3	3774.33	38.38	38.38	3993.00	16.40	3694.93	15.18	1.081	3694.93	15.18	1.081
R-4	3022.68	26.56	26.56	3212.00	13.10	2928.20	11.94	1.097	2928.20	11.94	1.097
R-5	2791.69	28.73	28.73	1038.00	27.60	1060.45	28.20	0.979	1060.45	28.20	0.979
R-6	2594.94	20.22	20.22	786.00	21.20	743.84	20.06	1.057	743.84	20.06	1.057
R-7	2833.28	29.35	29.35	1134.00	30.50	1071.24	28.81	1.059	1071.24	28.81	1.059
R-8	2817.84	27.98	27.98	1075.00	28.90	1023.71	27.52	1.050	1023.71	27.52	1.050
						Mean		1.065			1.050
						Coefficient of Variation		0.087			0.057
						Resistance Factor		0.902			0.913

Table 3.23 (continued)

						(2)			(3)		
Test[1]	M_m (in.-k)	$V_m^{(2)}$ (k)	$V_m^{(3)}$ (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory	M_n (in.-k)	V_n (k)	Test/ Theory
C-1	3110.10	33.71	33.71	2886.00	33.40	2385.07	27.60	1.210	2385.07	27.60	1.210
C-2	4604.48	41.06	41.53	4107.00	36.80	3645.52	32.67	1.127	3666.45	32.85	1.120
C-3	4624.92	41.59	42.52	5468.00	14.00	4589.87	11.75	1.191	4592.08	11.76	1.191
C-4	4900.59	47.36	48.71	1723.00	47.60	1690.44	46.70	1.019	1736.49	47.97	0.992
C-5	5138.23	47.56	48.58	3511.00	48.10	3174.11	43.48	1.106	3225.35	44.19	1.089
C-5	3188.26	37.08	37.08	1471.00	40.40	1317.69	36.19	1.116	1317.69	36.19	1.116
G-1	1734.13	26.54	26.54	791.00	32.70	631.44	26.10	1.253	631.44	26.10	1.253
G-2	1712.64	26.56	26.56	1296.00	26.50	1151.25	23.54	1.126	1151.25	23.54	1.126
CHO-3	1369.30	31.11	31.11	634.00	35.70	540.89	30.46	1.172	540.89	30.46	1.172
CHO-4	2356.96	48.52	48.52	1477.00	46.70	1414.91	44.74	1.044	1414.91	44.74	1.044
					Mean			1.121			1.116
					Coefficient of Variation			0.076			0.080
					Resistance Factor			0.960			0.952
CHO-6	1745.52	36.93	36.93	721.00	40.60	644.63	36.30	1.118	644.63	36.30	1.118
CHO-7	2983.27	55.85	55.85	2664.00	20.60	2916.30	22.55	0.913	2916.30	22.55	0.913
					Mean			1.016			1.016
					Coefficient of Variation			0.145			0.145
					Resistance Factor			0.804			0.804
Overall					Mean			1.048			1.039
					Coefficient of Variation			0.095			0.091
					Resistance Factor			0.880			0.880

Notes:

(1) refer to Table 3.0 for key to beam designations
(2) P_{ch} not limited by A_{sn} x F_y
(3) P_{ch} limited by A_{sn} x F_y (as used in current study)

Table 3.24 Effect of Restricting Normal Force in Reinforcement by the Weld Strength

Test[1]	V_p (k)	(2) P_r (k)	(2) V_m (k)	(3) P_r (k)	(3) V_m (k)	(2) Test/ Theory	(3) Test/ Theory
CR-1A	35.29	23.18	28.47	23.18	28.47	1.007	1.007
CR-2A	55.42	41.41	55.42	40.65	55.42	1.320	1.320
CR-2B	55.42	41.41	55.42	40.65	55.42	1.211	1.211
CR-2C	70.94	35.37	65.10	35.37	65.10	1.194	1.194
CR-2D	70.94	35.37	65.10	35.37	65.10	1.294	1.294
CR-3A	55.42	83.03	55.42	40.65	55.42	1.461	1.461
CR-3B	70.94	58.85	70.94	51.34	70.94	1.154	1.154
CR-4A	70.94	35.37	53.17	35.37	53.17	1.302	1.302
CR-4B	70.94	35.37	53.17	35.37	53.17	1.140	1.140
CR-5A	51.29	57.08	46.12	57.08	46.12	1.167	1.167
CR-7B	63.21	37.71	61.83	37.71	61.83	1.174	1.174
CR-7D	63.21	37.71	59.00	37.71	59.00	1.343	1.343
CSK-2	92.96	43.37	92.96	41.29	92.96	1.161	1.161
CSK-5	78.81	42.72	77.91	42.72	77.91	1.108	1.108
CSK-6	63.06	35.52	50.89	35.52	50.89	1.002	1.002
CSK-7	63.06	35.52	59.47	35.52	59.47	1.060	1.060
CS-1	43.95	52.12	43.95	32.64	43.95	0.976	0.976
CS-2	42.31	52.57	42.31	31.42	42.31	0.963	0.963
CS-3	43.84	49.73	43.84	32.55	43.80	1.066	1.066
RL-5	69.49	18.17	43.01	18.17	43.01	1.085	1.085
RL-6	43.20	41.62	23.63	41.62	23.63	0.927	0.927
CHO-6	n/a	47.16	40.60	24.68	36.93	1.091	1.118
CHO-7	n/a	46.92	55.85	46.92	55.85	0.913	0.913
WJE-1	n/a	85.47	38.31	85.47	38.31	0.919	0.919

Notes:

(1) refer to Table 3.0 for key to beam designations
(2) no restriction on normal force in reinforcement
(3) normal force restricted

V_p = plastic shear capacity of the top and bottom tees
P_r = normal force in the reinforcement
V_m = naximum shear capacity as predicted by Method III

Table 3.25 Effect of Flanges, Method I versus Method III
$\lambda = 1.414$

Beam	A_f/A_w	V_1 (k)	V_3 (k)	V_1/V_3	$(V_1 - V_3)/V_p$
W21X44	0.40	19.32	19.76	0.98	-0.01
W12X22	0.54	8.88	8.75	1.01	0.00
W14X26	0.60	9.98	9.69	1.03	0.01
W16X57	0.72	21.23	19.31	1.10	0.03
W14X38	0.80	13.24	11.95	1.11	0.04
W12X35	0.91	11.97	10.25	1.17	0.06
W14X43	1.02	13.60	11.39	1.19	0.06
W12X45	1.14	14.34	11.04	1.30	0.10
W14X74	1.24	24.39	17.43	1.40	0.13
W10X39	1.35	12.30	8.54	1.44	0.14
W12X58	1.46	17.79	11.99	1.48	0.16
W10X49	1.65	14.99	9.27	1.62	0.20
W8X10	0.60	3.76	3.67	1.03	0.01
W14X109	1.67	34.38	20.55	1.67	0.22
W18X60	0.69	22.26	20.69	1.08	0.02
W21X122	0.91	42.03	35.55	1.18	0.06
W24X117	0.82	40.45	36.46	1.11	0.04
W27X114	0.60	44.08	42.51	1.04	0.01
W30X108	0.49	44.31	44.43	1.00	0.00
W30X211	0.83	74.72	65.53	1.14	0.05
W33X221	0.77	79.09	71.86	1.10	0.03
W36X150	0.50	61.28	61.23	1.00	0.00
W36X300	0.81	108.41	94.88	1.14	0.05

Notes:

Consistent relative opening dimensions calculated using:

a_o/h_o = 2.0
h_o/d = 0.60
s_t = $0.15d$
s_b = $d - h_o - s_t$
V_1 = maximum shear capacity of top and bottom tees calculated using Eq. 2.32
V_3 = maximum shear capacity of top and bottom tees calculated using Eq. 2.54
V_p = plastic shear capacity of top and bottom tees

Table 3.26 Effect of Limiting the Maximum Moment Capacity, M_m, to the Plastic Moment Capacity, M_p, Method III, $\lambda = 1.414$

						(2)			(3)		
Test[1]	M_p (in.-k)	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory	M_n (in.-k)	V_n (k)	Test/ Theory
CR-1A	1079.61	1102.64	28.47	914.16	21.22	919.11	21.33	0.995	907.80	21.07	1.007
CR-2A	2362.84	2487.76	55.42	1542.37	70.07	1175.41	53.40	1.312	1168.58	53.09	1.320
CR-2B	2362.84	2487.76	55.42	2331.35	51.74	1978.26	43.90	1.178	1925.75	42.74	1.211
CR-2C	2773.20	2812.31	65.10	2112.23	70.36	1774.62	59.11	1.190	1768.29	58.90	1.195
CR-2D	2773.20	2812.31	65.10	1404.33	82.53	1086.05	63.83	1.293	1085.16	63.77	1.294
CR-3A	2362.84	2779.13	55.42	1707.37	77.57	1187.27	53.94	1.438	1168.52	53.09	1.461
CR-3B	2773.20	2976.66	70.94	2704.85	60.10	2442.32	54.27	1.107	2344.33	52.09	1.154
CR-4A	2773.20	2812.31	53.17	1487.37	67.57	1143.55	51.95	1.301	1142.46	51.90	1.302
CR-4B	2773.20	2812.31	53.17	2313.35	51.34	2040.66	45.29	1.134	2029.63	45.05	1.140
CR-5A	2773.20	2942.26	46.12	1554.23	51.76	1339.83	44.62	1.160	1331.74	44.35	1.167
CR-7B	2501.55	2579.80	61.83	2448.35	54.34	2123.08	47.12	1.153	2086.22	46.30	1.174
CR-7D	2501.55	2579.80	59.00	1319.33	77.58	984.42	57.89	1.340	982.66	57.78	1.343
CSK-2	3690.74	3680.73	92.96	2872.39	95.69	2473.52	82.40	1.161	2473.52	82.40	1.161
CSK-5	3141.22	3165.72	77.91	2309.50	76.93	2089.13	69.59	1.105	2084.43	69.44	1.108
CSK-6	3141.22	3043.56	50.89	1471.10	48.99	1468.63	48.91	1.002	1468.63	48.91	1.002
CSK-7	3141.22	3043.56	59.47	1780.10	59.29	1679.34	55.93	1.060	1679.34	55.93	1.060
CS-1	2137.01	2336.64	43.95	1811.25	30.08	1962.33	32.59	0.923	1855.96	30.82	0.976
CS-2	2155.60	2362.15	42.31	1772.25	29.43	1942.98	32.27	0.912	1840.87	30.57	0.963
CS-3	2095.06	2280.69	43.80	1604.00	40.03	1548.59	38.65	1.036	1504.37	37.54	1.066
RL-5	2705.84	2667.74	43.01	2893.50	0.00	2667.74	0.00	1.085	2667.74	0.00	1.085
RL-6	2701.97	2766.31	23.63	1048.89	21.36	1133.14	23.08	0.926	1131.25	23.03	0.927
					Mean			1.133			1.148
					Coefficient of Variation			0.128			0.122
					Resistance Factor			0.916			0.935

Notes:

(1) refer to Table 3.0 for key to beam designations
(2) M_m not limited by M_p
(3) M_m limited by M_p

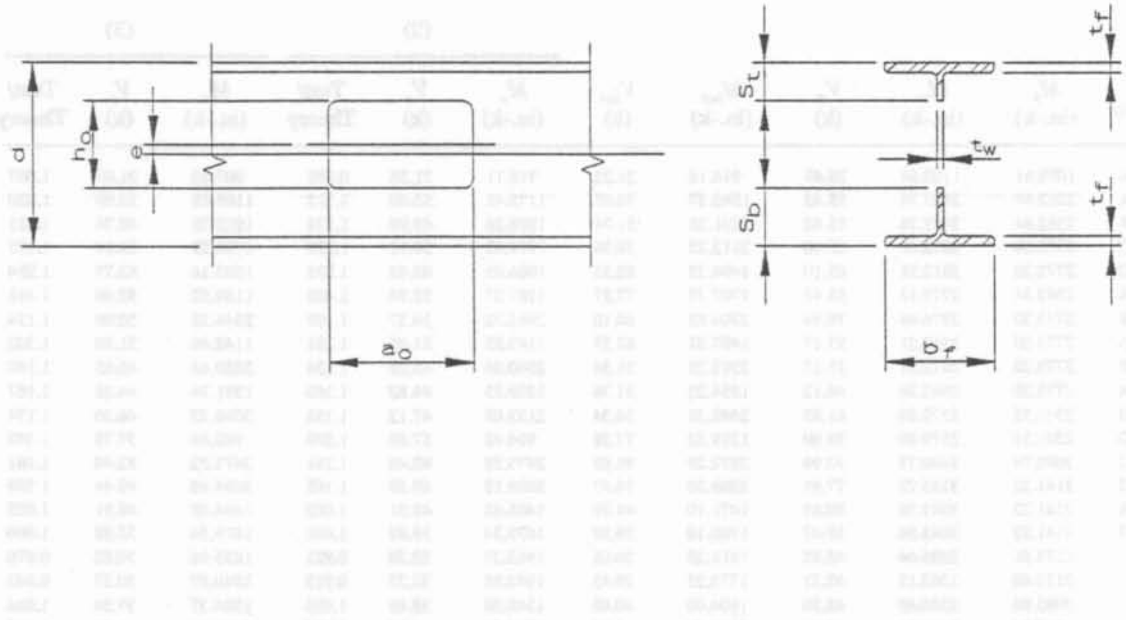

Fig. 2.1(a) Opening Configuration for an Unreinforced Steel Beam

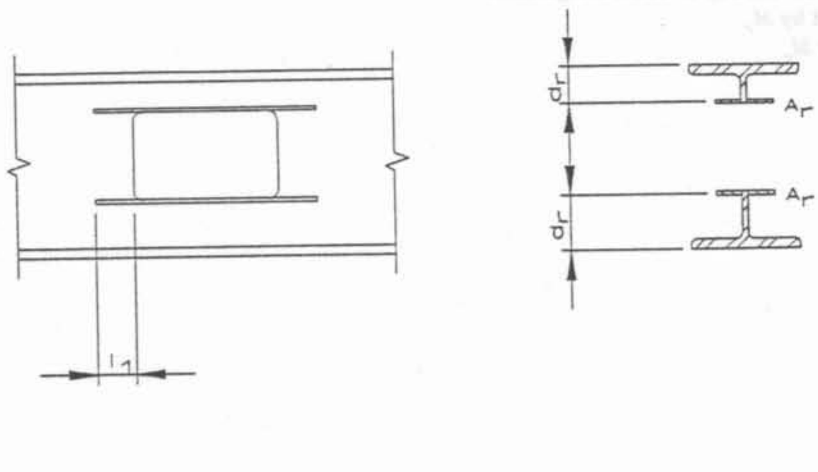

Fig. 2.1(b) Opening Configuration for a Reinforced Steel Beam

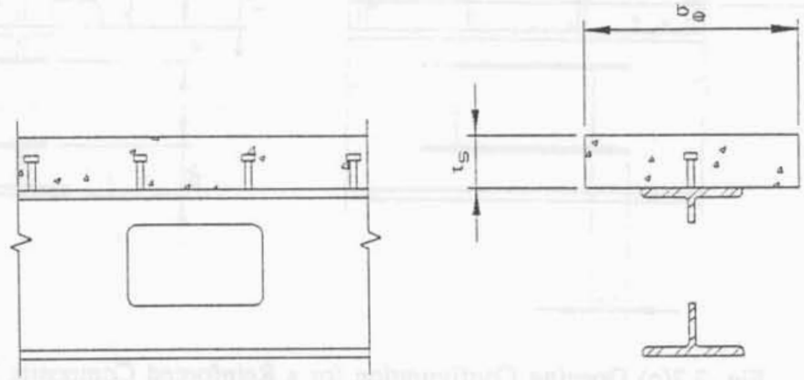

Fig. 2.2(a) Opening Configuration for an Unreinforced Composite Beam with a Solid Slab

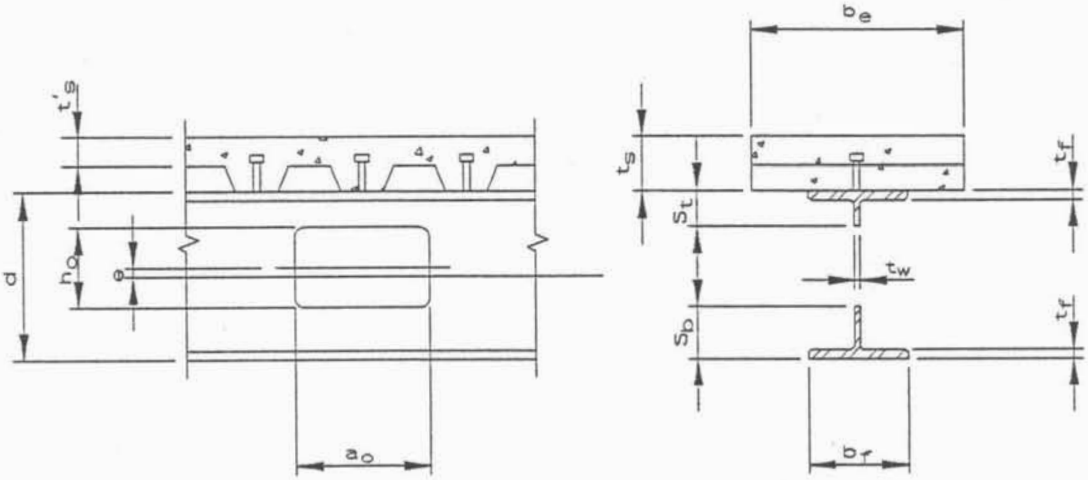

Fig. 2.2(b) Opening Configuration for an Unreinforced Composite Beam with Transverse Ribs

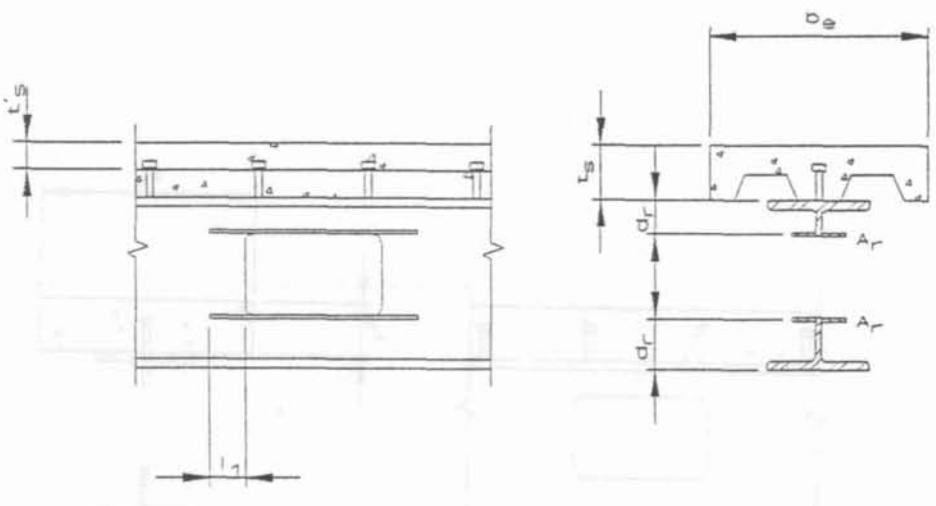

Fig. 2.2(c) Opening Configuration for a Reinforced Composite Beam with Longitudinal Ribs

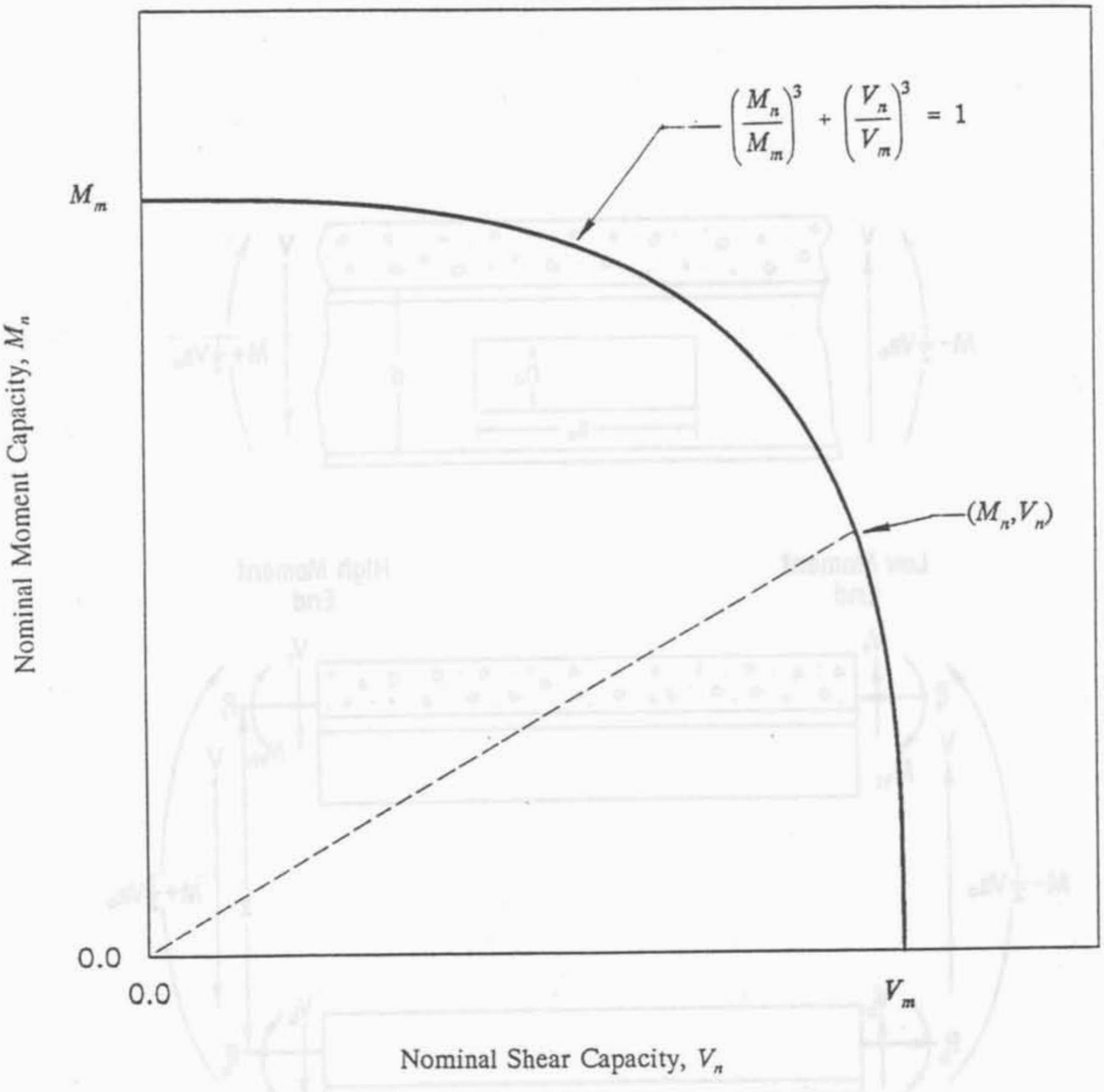

Fig. 2.3 Cubic Moment-Shear Interaction
(Darwin and Donahey 1988)

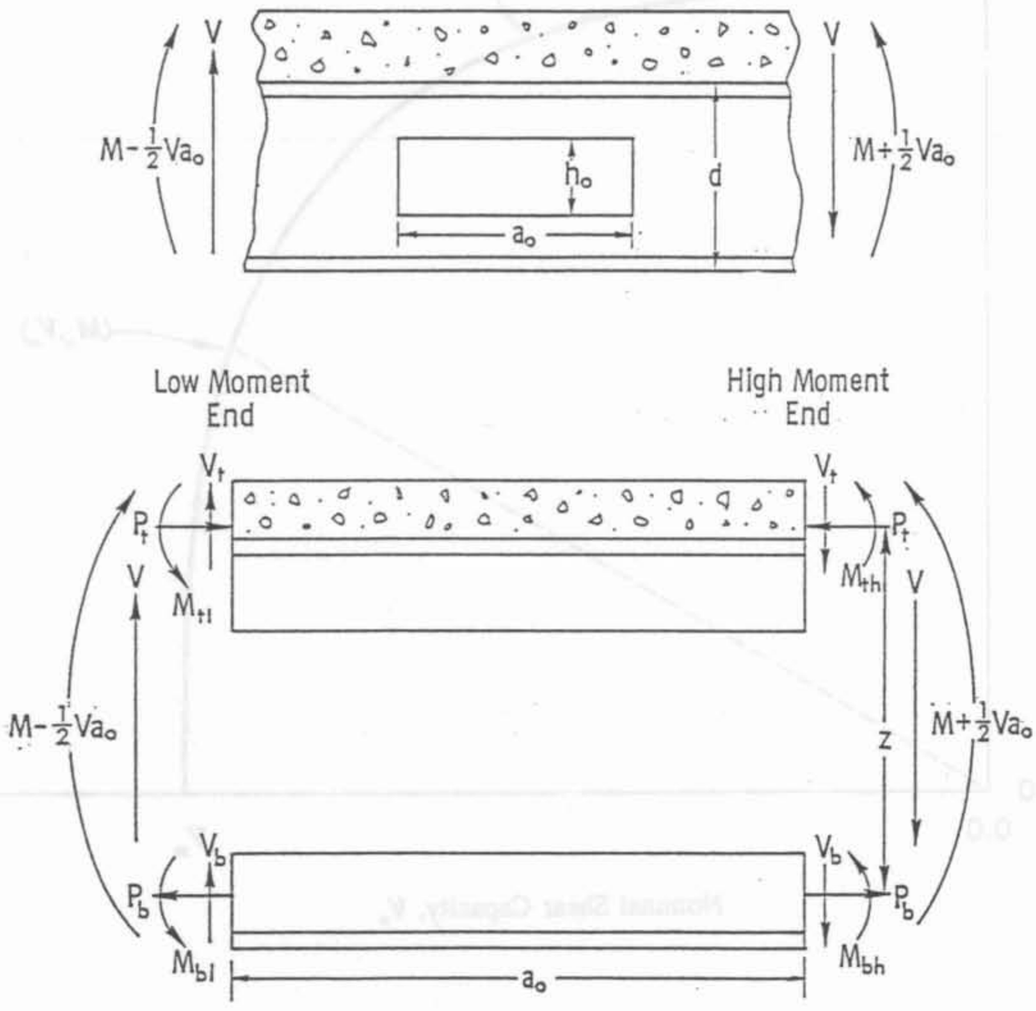

Fig. 2.4 Forces Acting at a Web Opening (Darwin 1990)

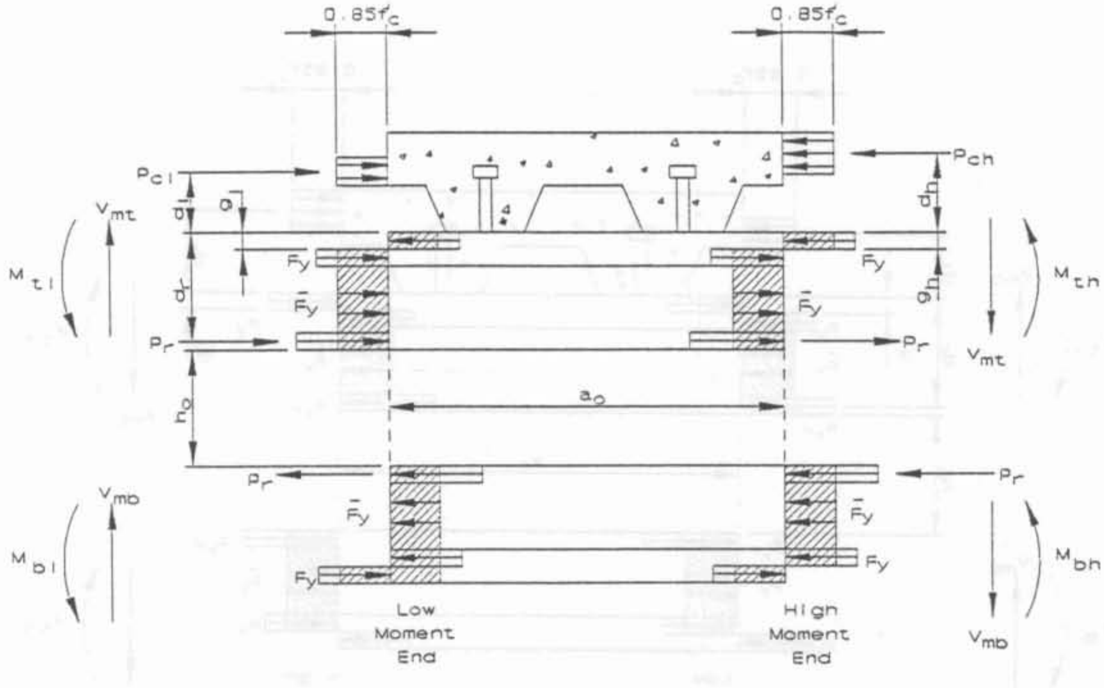

Fig. 2.5 Normal Forces in a Composite Opening

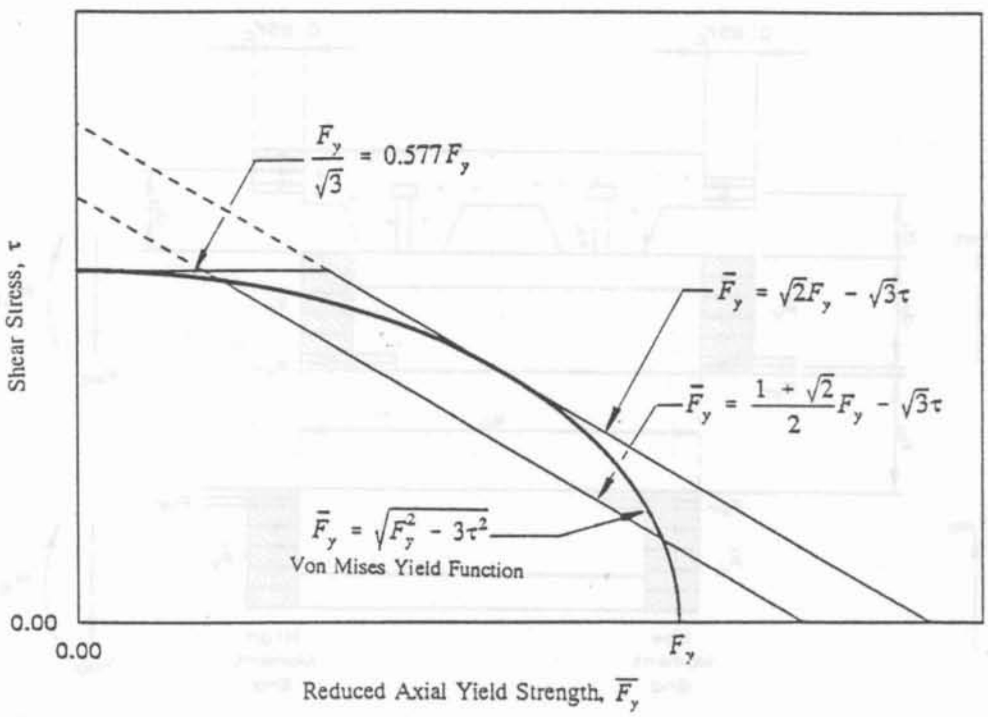

Fig. 2.6 Yield Functions for Combined Shear and Normal Stress

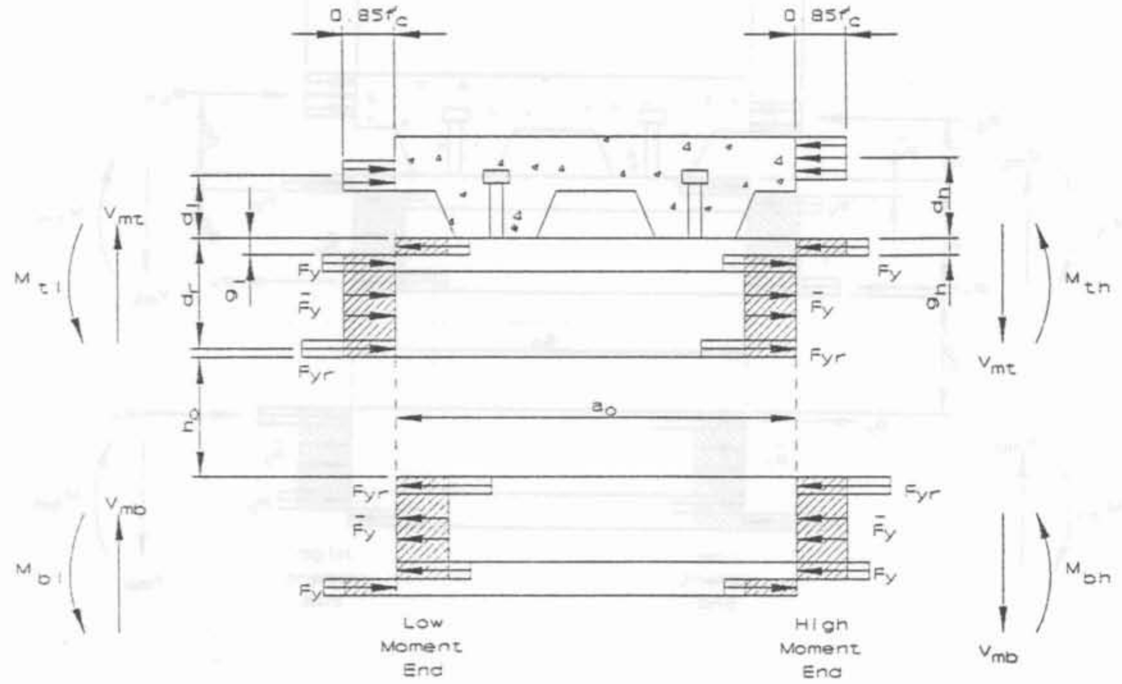

Fig. 2.7 Stress Distributions for Design Method I

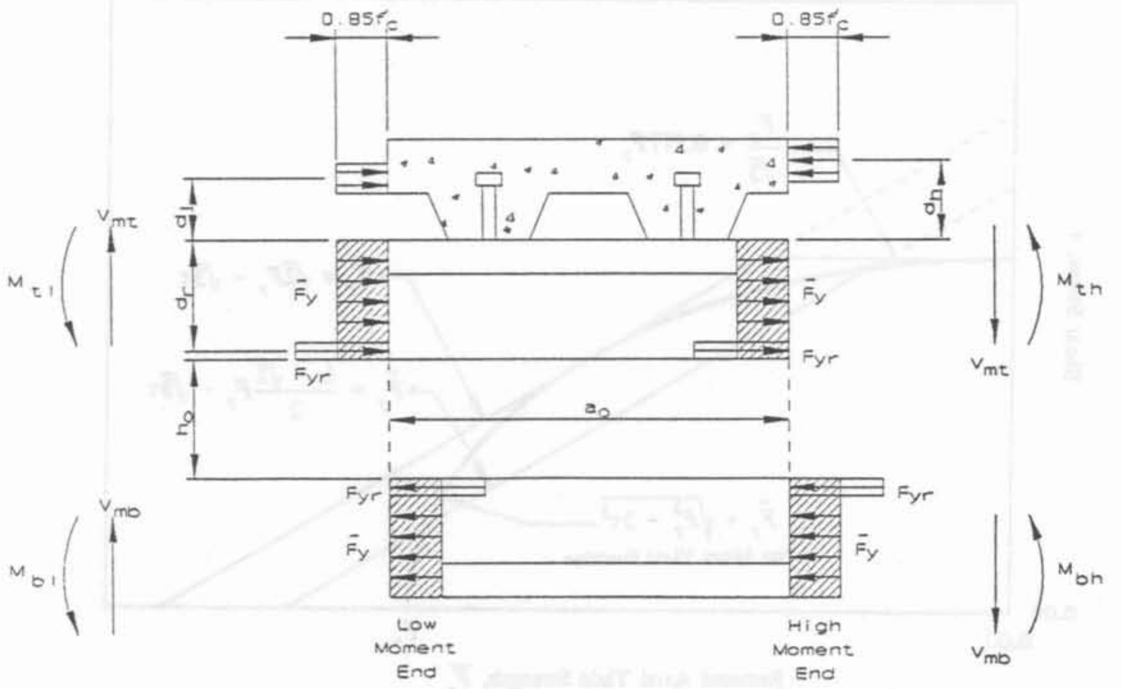

Fig. 2.8 Stress Distributions for Design Methods II and III

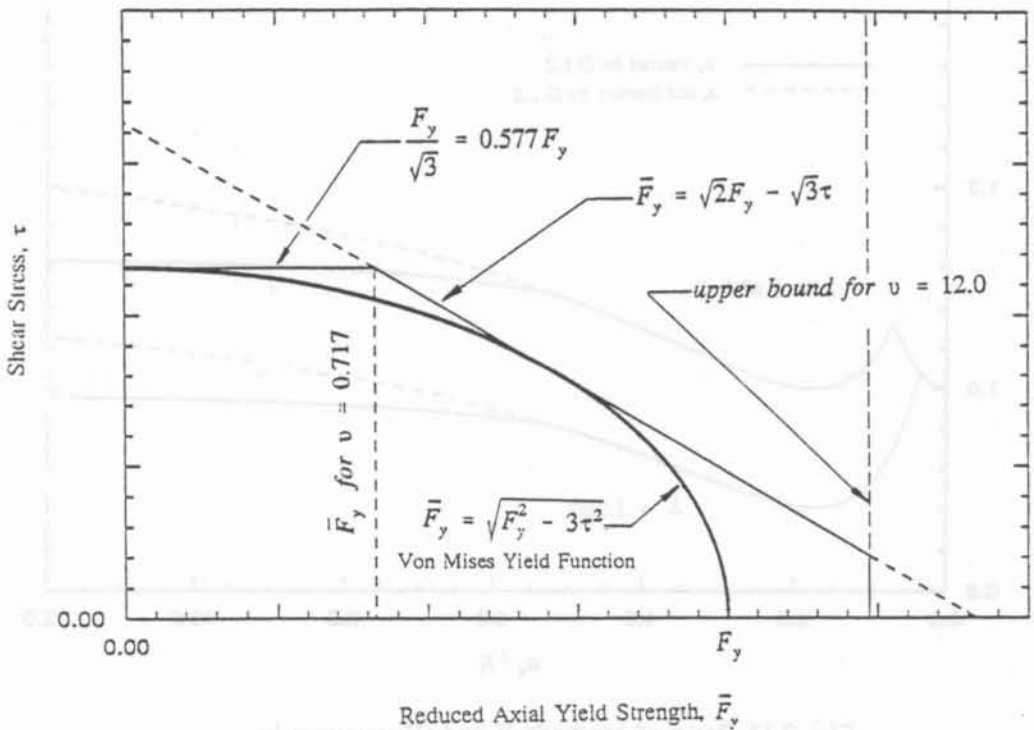

Fig. 2.9 Comparison of Yield Functions Considering Practical Constraints

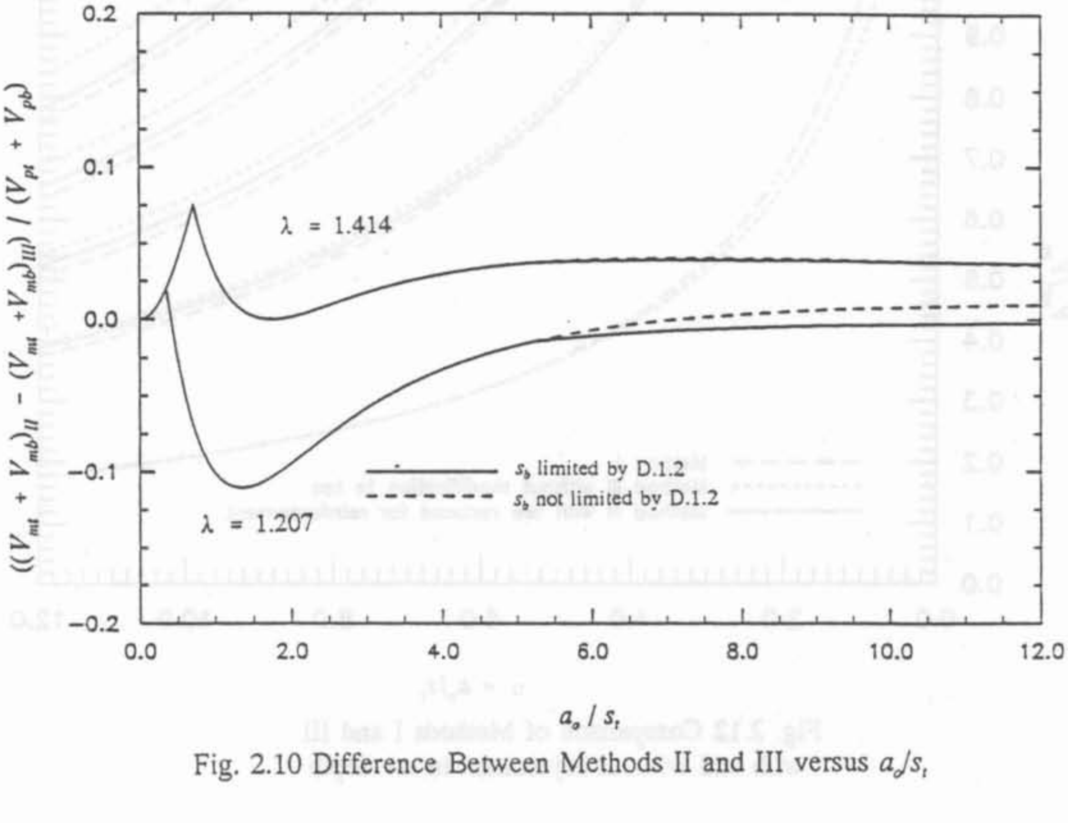

Fig. 2.10 Difference Between Methods II and III versus a_o/s_t

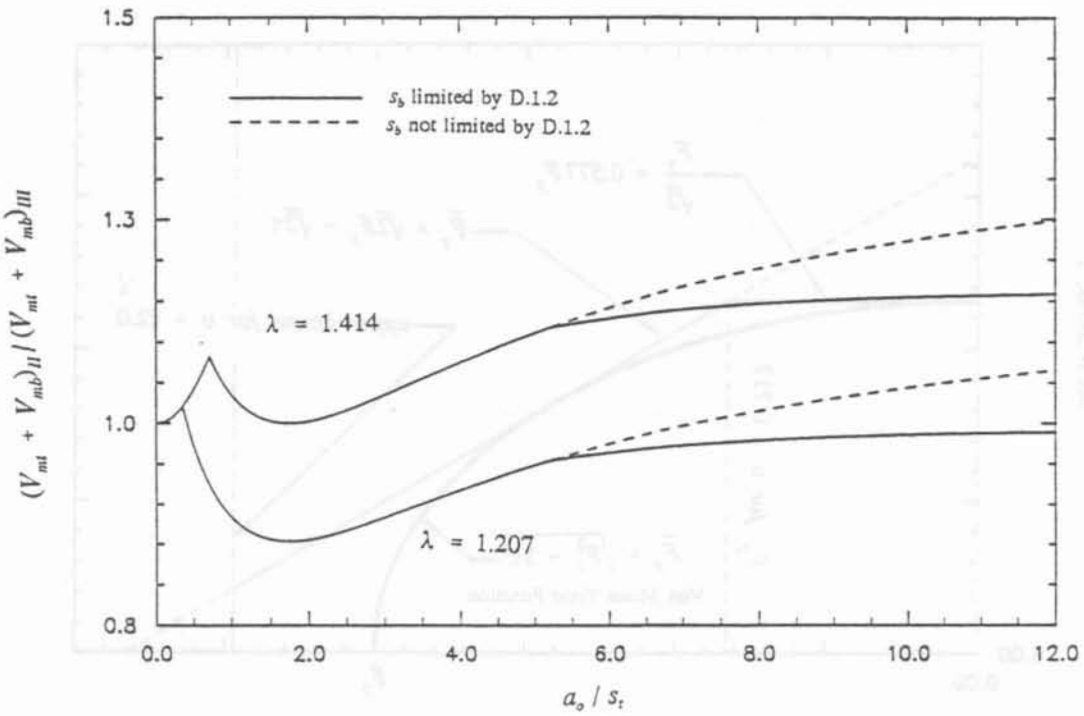

Fig. 2.11 Ratio of Methods II and III versus a_o/s_t

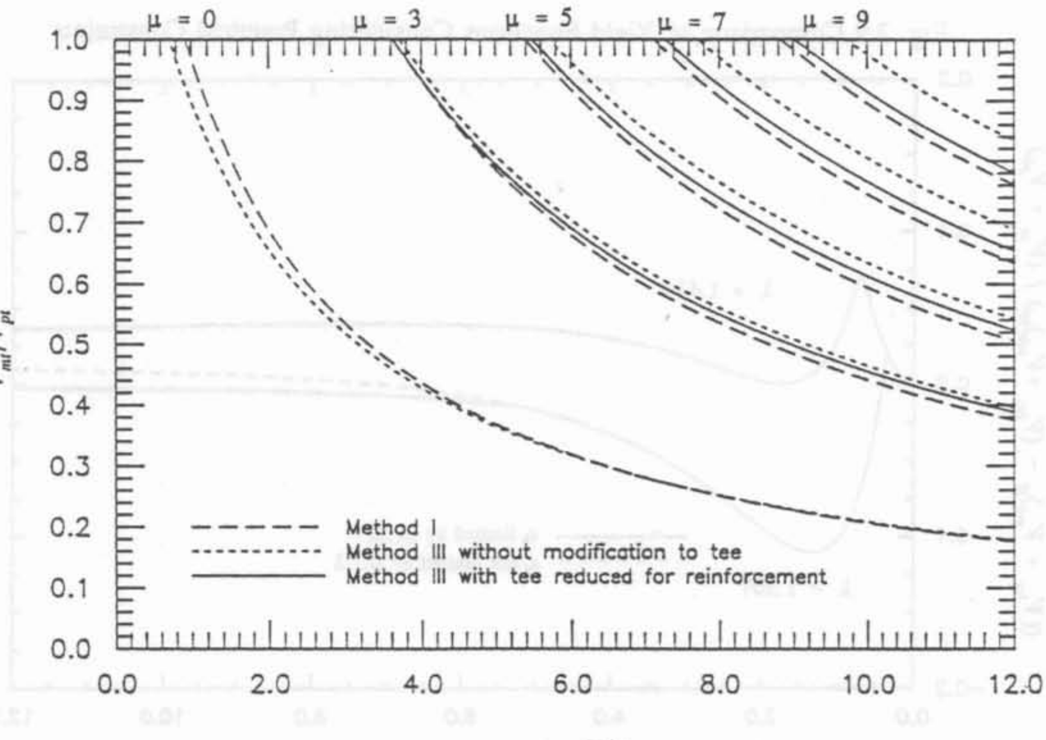

Fig. 2.12 Comparison of Methods I and III
with and without adjustment in tee depth

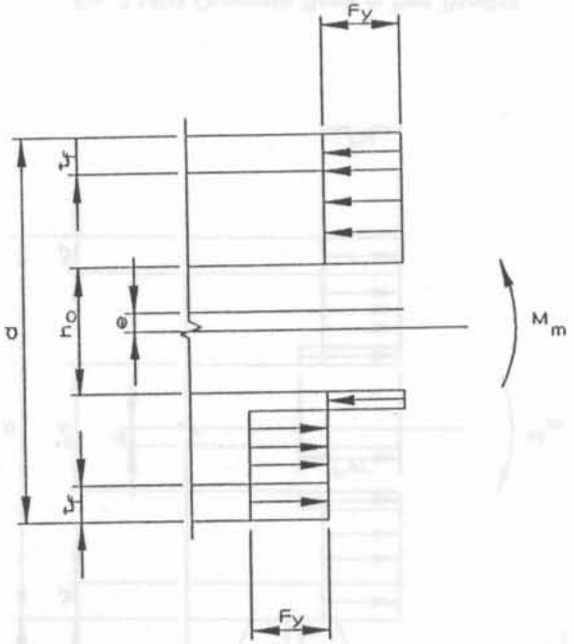

Fig. 2.13(a) Unreinforced Steel Beam in Pure Bending

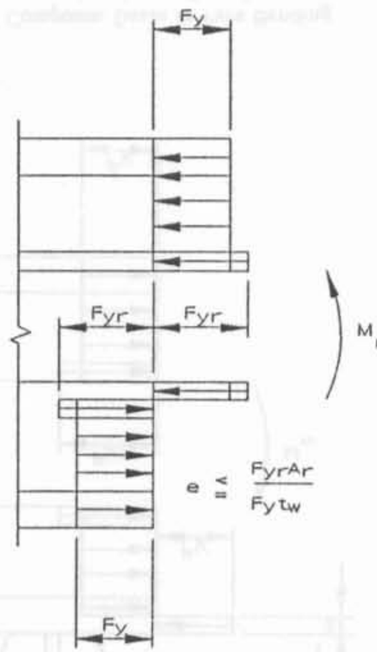

Fig. 2.13(b) Reinforced Steel Beam in Pure Bending with Neutral Axis in Reinforcement

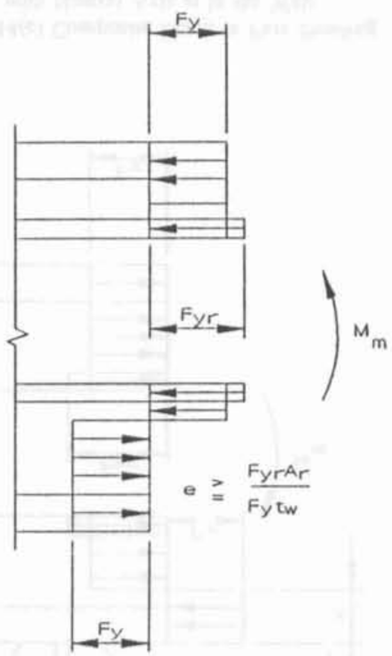

Fig. 2.13(c) Reinforced Steel Beam in Pure Bending with Neutral Axis in Web

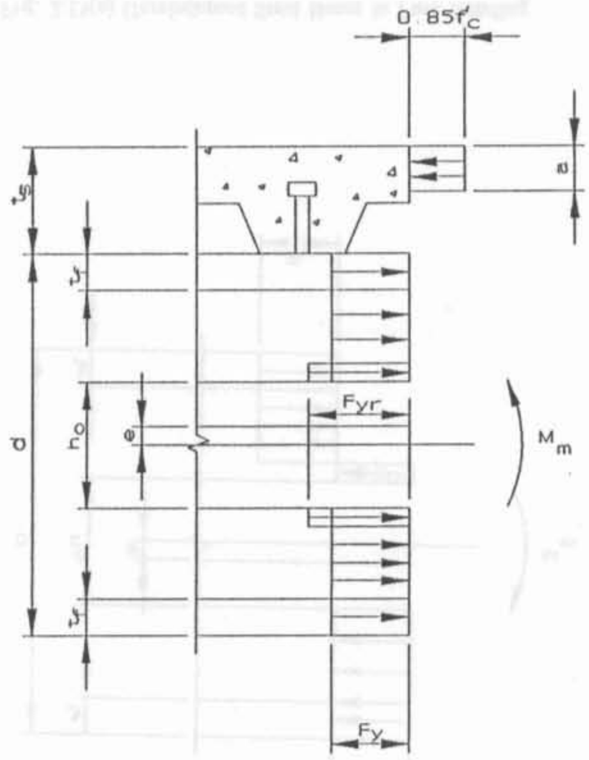

Fig. 2.14(a) Composite Beam in Pure Bending with Neutral Axis at or above Steel Flange

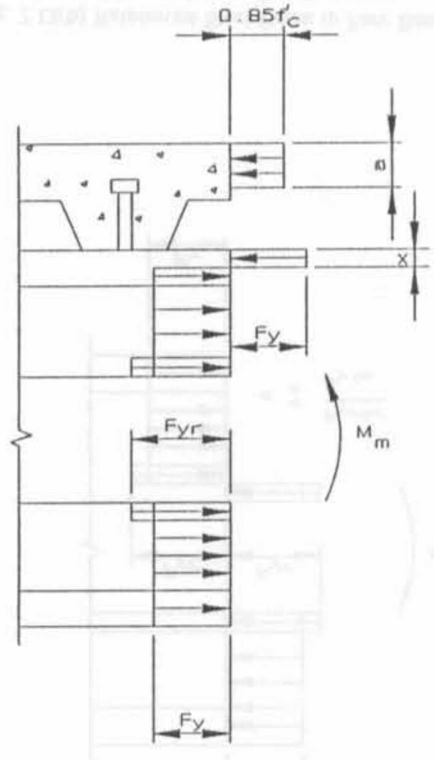

Fig. 2.14(b) Composite Beam in Pure Bending with Neutral Axis at in the Steel Flange

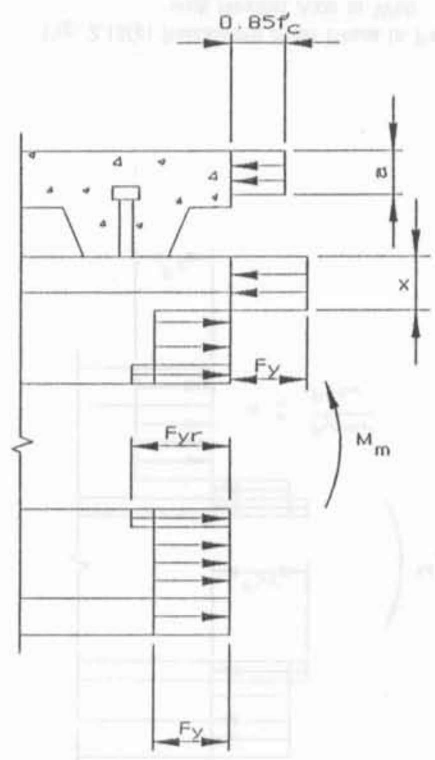

Fig. 2.14(c) Composite Beam in Pure Bending with Neutral Axis at in the Web

———— Cubic Moment-Shear Interaction Using Method III

------------ Curvlinear Moment-Shear Interaction Using Redwood & Shrivasatava (1980), Redwood & Poumbouras (1984), and Redwood & Cho (1986)

⊙ Actual Moment-Shear Value for Beam

Fig. 3.0 Legend for Moment-Shear Interaction Curves in Figs. 3.1 - 3.85

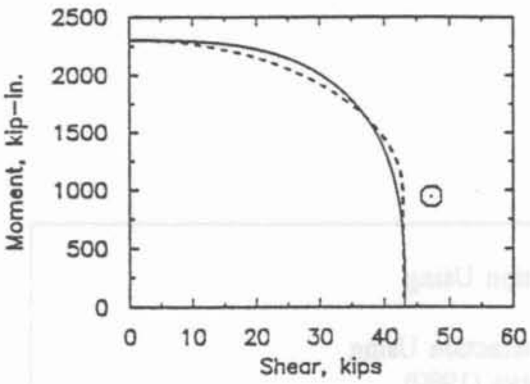

Fig. 3. 1 Interaction Curves for Test B-1

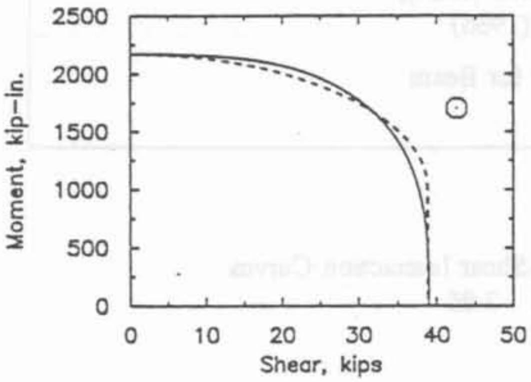

Fig. 3. 2 Interaction Curves for Test B-2

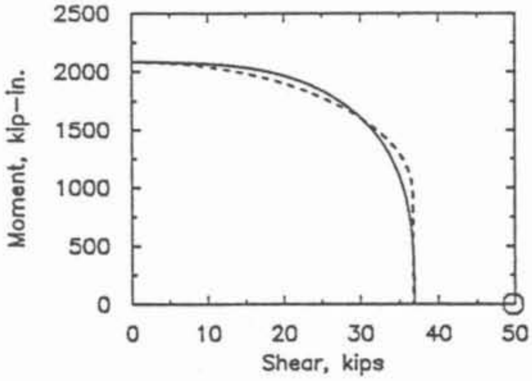

Fig. 3. 3 Interaction Curves for Test B-3

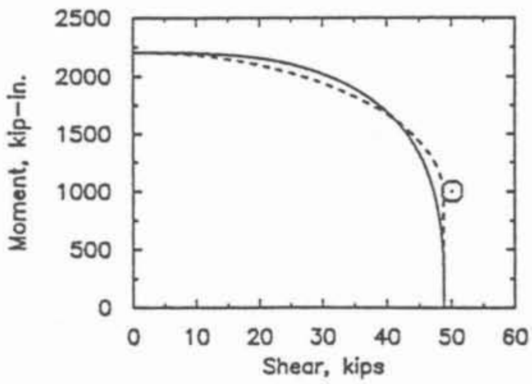

Fig. 3. 4 Interaction Curves for Test B-4

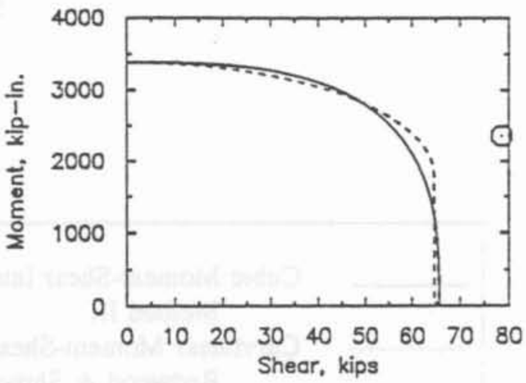

Fig. 3. 5 Interaction Curves for Test CSK-1

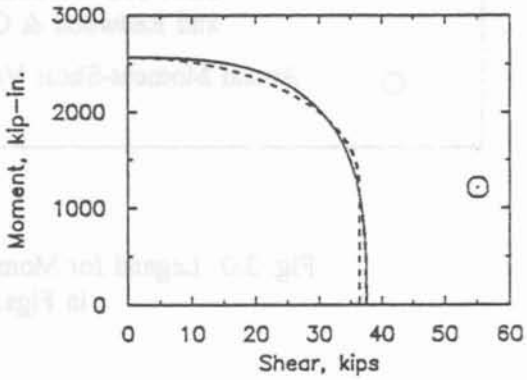

Fig. 3. 6 Interaction Curves for Test CR-6A

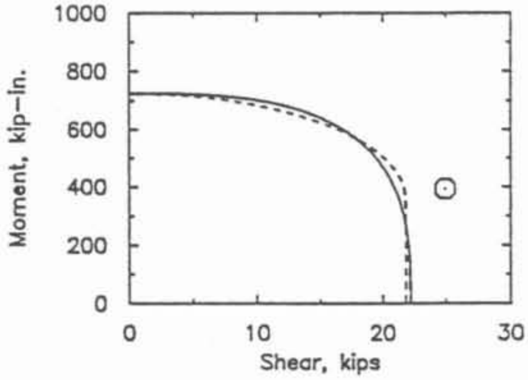

Fig. 3. 7 Interaction Curves for Test DO-1

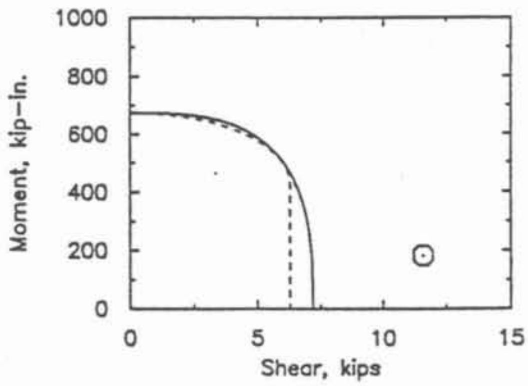

Fig. 3. 8 Interaction Curves for Test DO-2

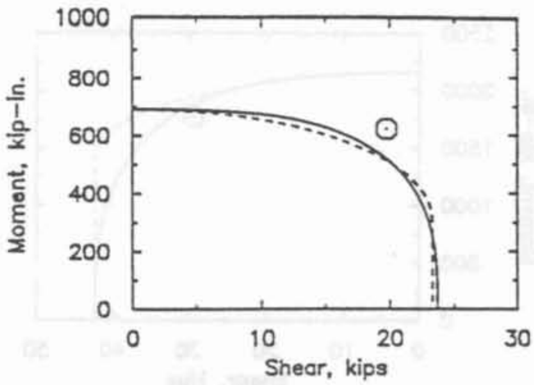

Fig. 3. 9 Interaction Curves for Test DO-3

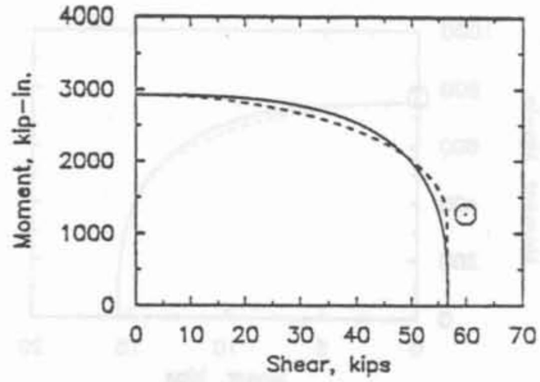

Fig. 3.13 Interaction Curves for Test RBD-R2

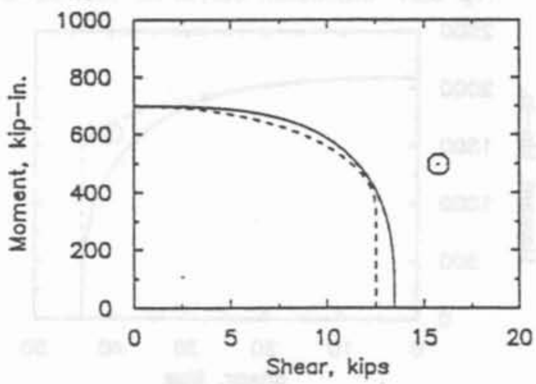

Fig. 3.10 Interaction Curves for Test DO-4

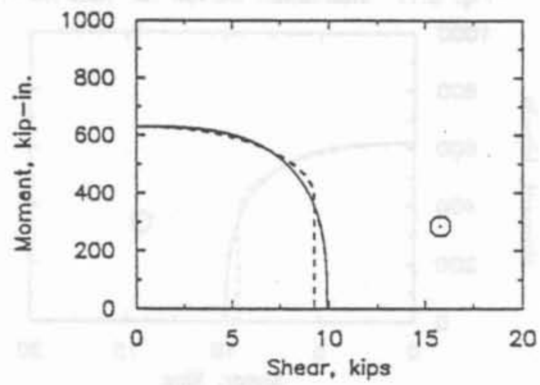

Fig. 3.14 Interaction Curves for Test RM-2F

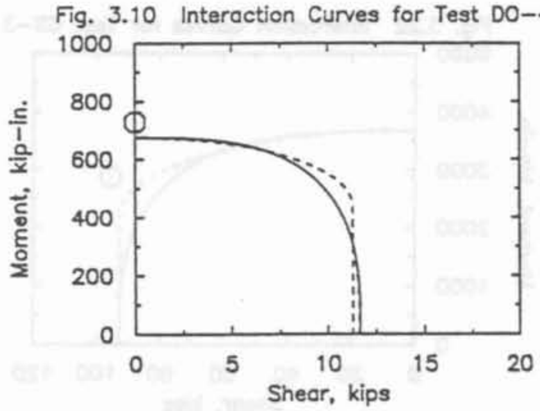

Fig. 3.11 Interaction Curves for Test DO-5

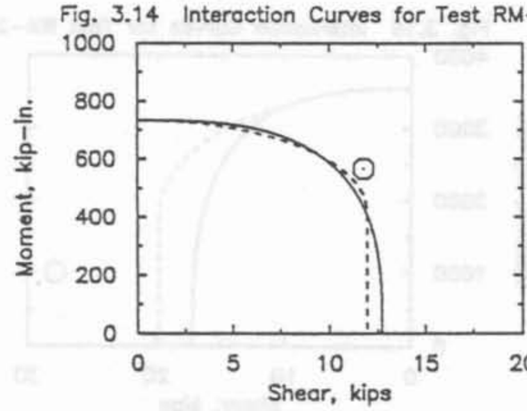

Fig. 3.15 Interaction Curves for Test RM-4F

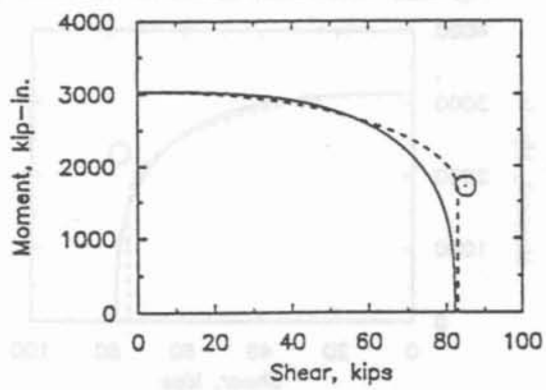

Fig. 3.12 Interaction Curves for Test RBD-R1

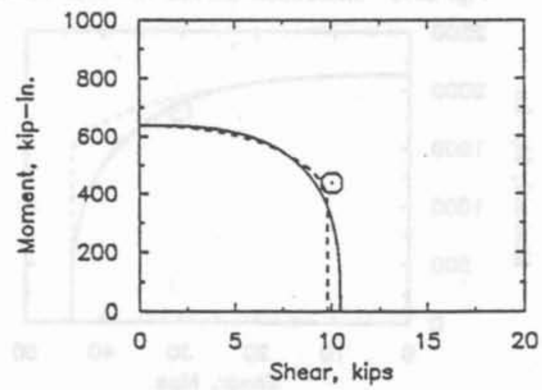

Fig. 3.16 Interaction Curves for Test RM-4H

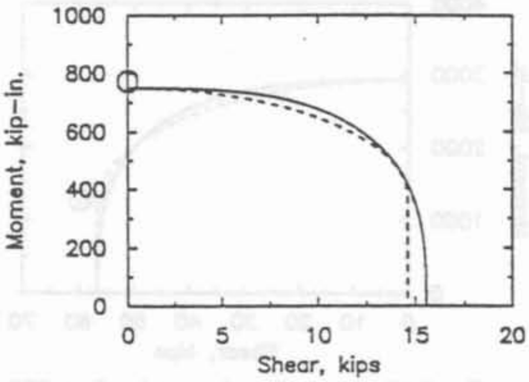

Fig. 3.17 Interaction Curves for Test RM-11H

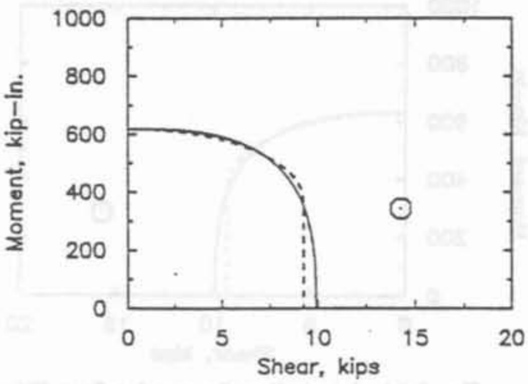

Fig. 3.18 Interaction Curves for Test RM-21H

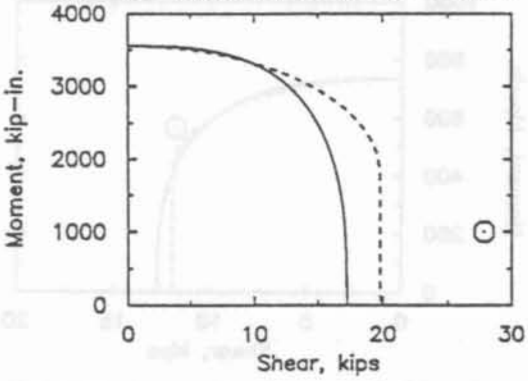

Fig. 3.19 Interaction Curves for Test CL-4B

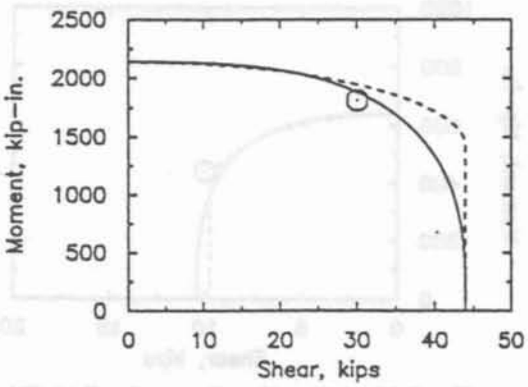

Fig. 3.20 Interaction Curves for Test CS-1

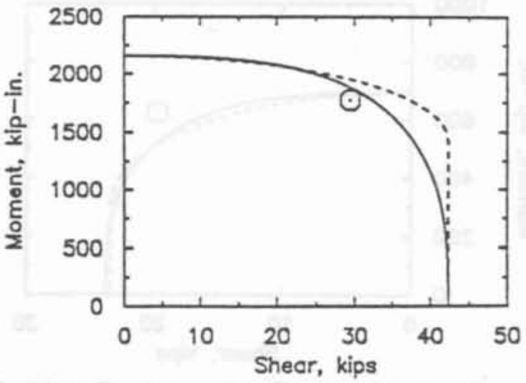

Fig. 3.21 Interaction Curves for Test CS-2

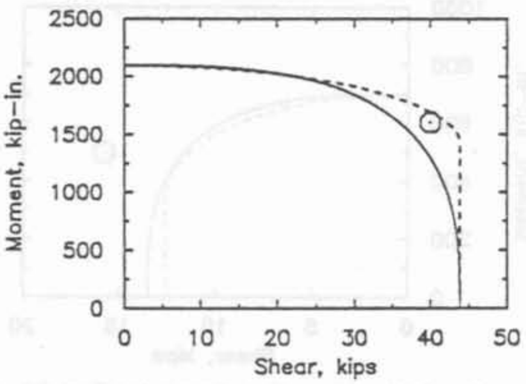

Fig. 3.22 Interaction Curves for Test CS-3

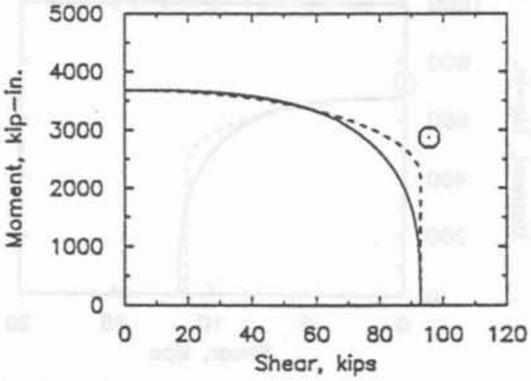

Fig. 3.23 Interaction Curves for Test CSK-2

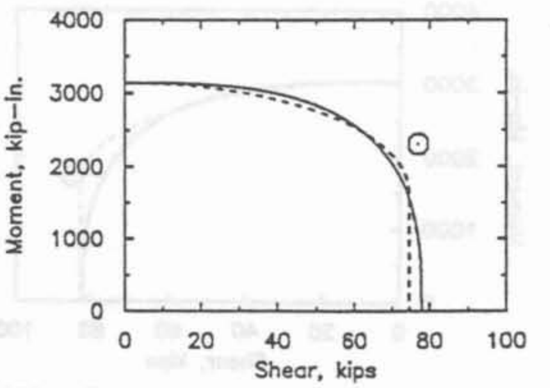

Fig. 3.24 Interaction Curves for Test CSK-5

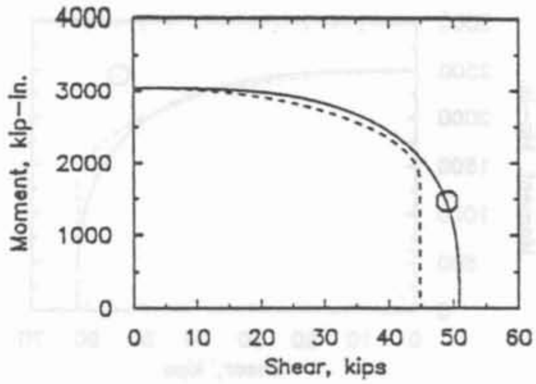

Fig. 3.25 Interaction Curves for Test CSK-6

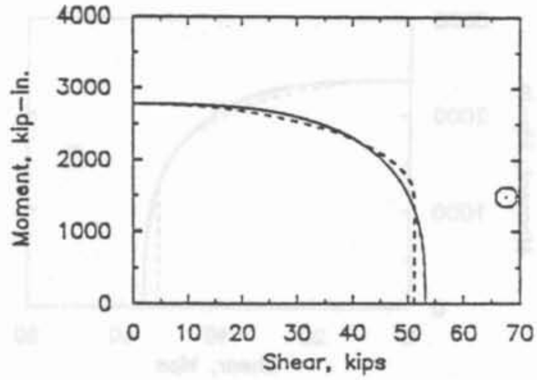

Fig. 3.29 Interaction Curves for Test CR-4A

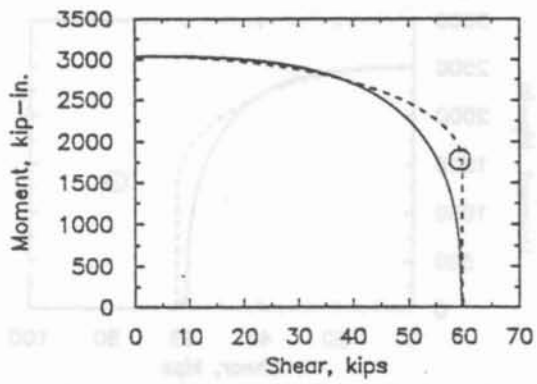

Fig. 3.26 Interaction Curves for Test CSK-7

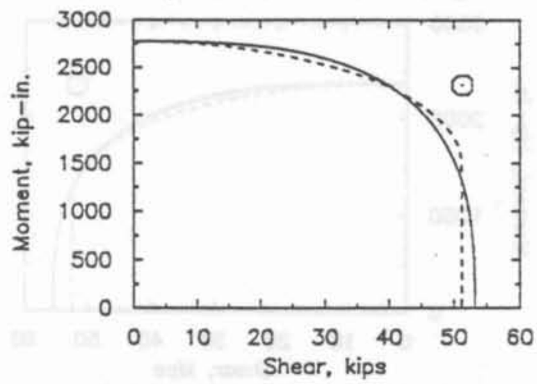

Fig. 3.30 Interaction Curves for Test CR-4B

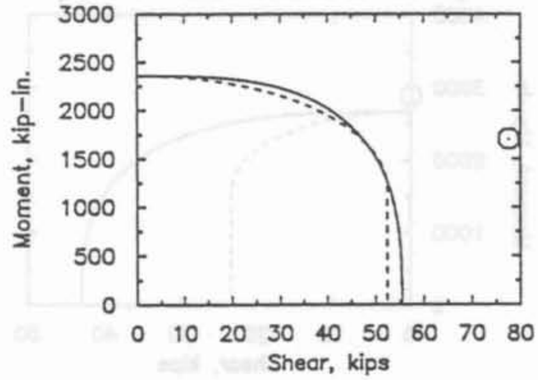

Fig. 3.27 Interaction Curves for Test CR-3A

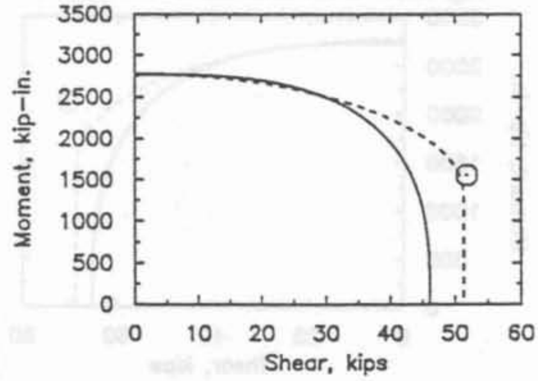

Fig. 3.31 Interaction Curves for Test CR-5A

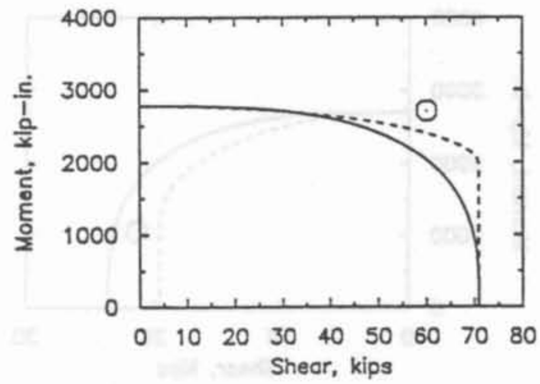

Fig. 3.28 Interaction Curves for Test CR-3B

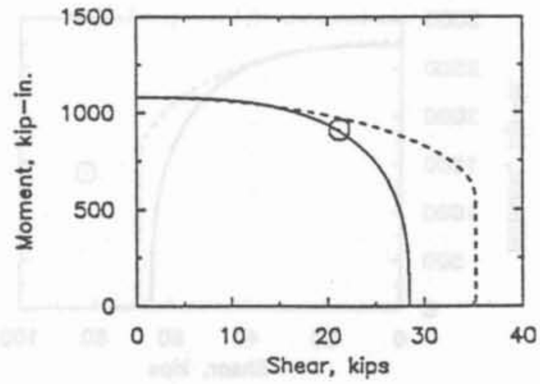

Fig. 3.32 Interaction Curves for Test CR-1A

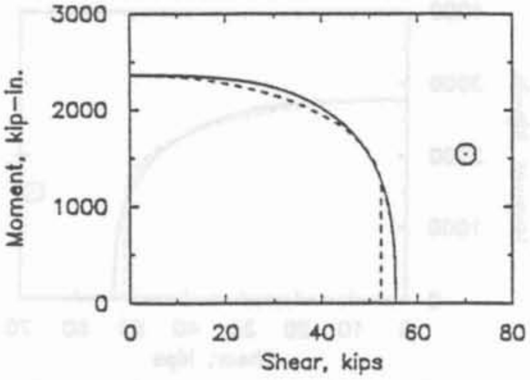

Fig. 3.33 Interaction Curves for Test CR-2A

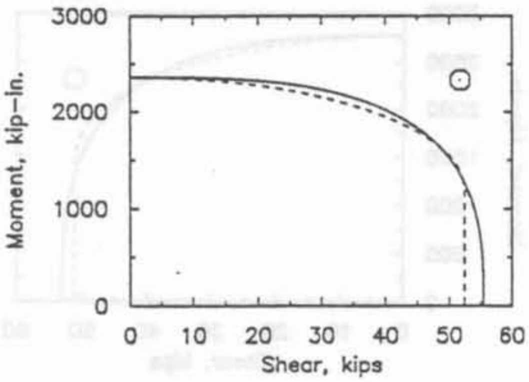

Fig. 3.34 Interaction Curves for Test CR-2B

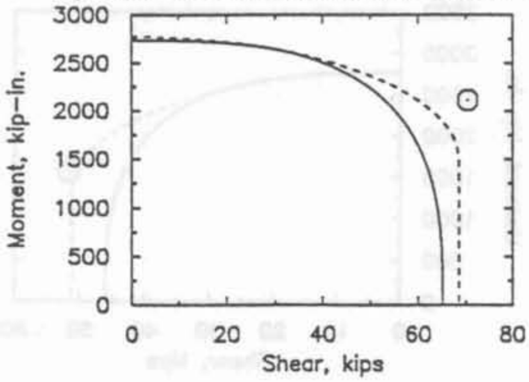

Fig. 3.35 Interaction Curves for Test CR-2C

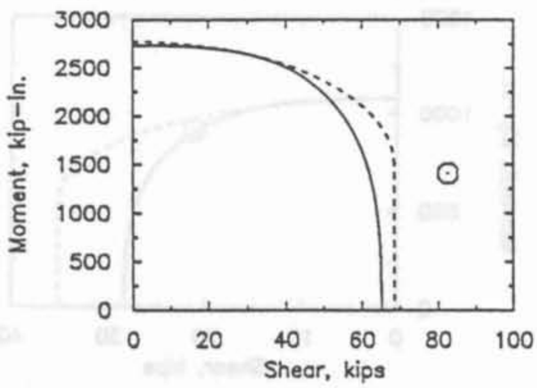

Fig. 3.36 Interaction Curves for Test CR-2D

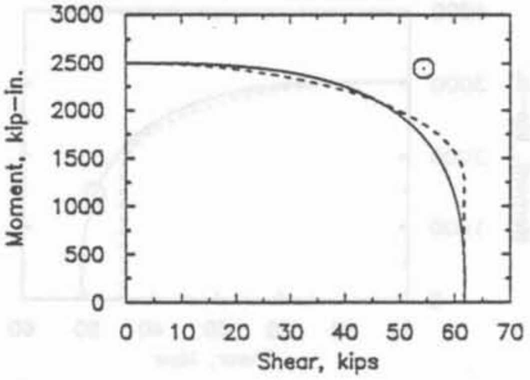

Fig. 3.37 Interaction Curves for Test CR-7B

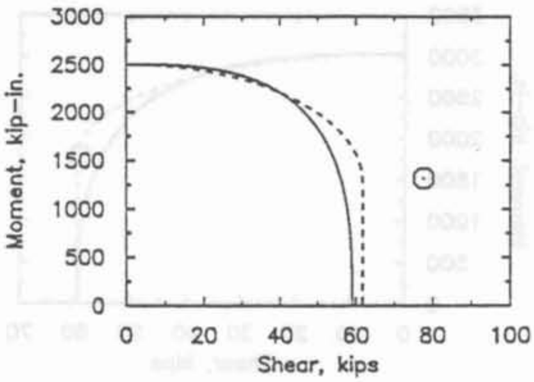

Fig. 3.38 Interaction Curves for Test CR-7D

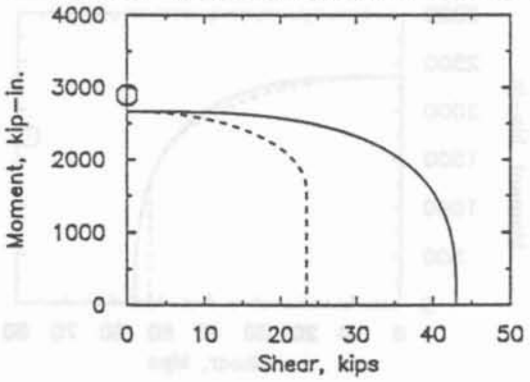

Fig. 3.39 Interaction Curves for Test RL-5

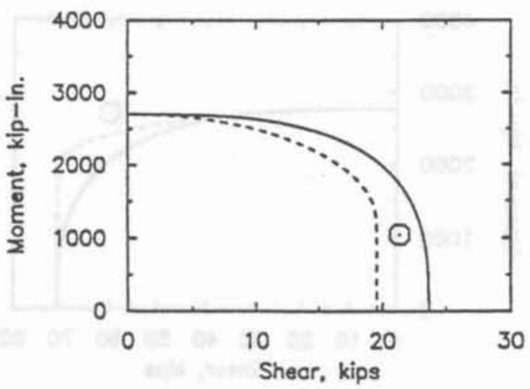

Fig. 3.40 Interaction Curves for Test RL-6

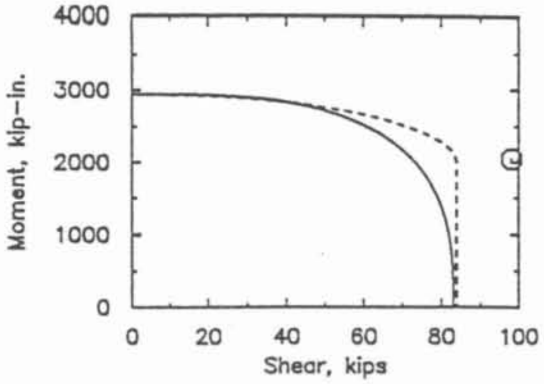

Fig. 3.41 Interaction Curves for Test RBD-C1

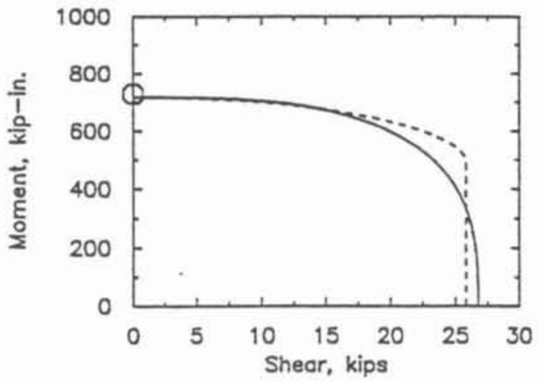

Fig. 3.42 Interaction Curves for Test RM-1A

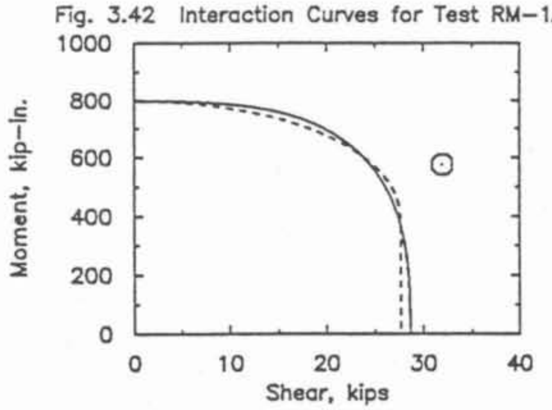

Fig. 3.43 Interaction Curves for Test RM-2A

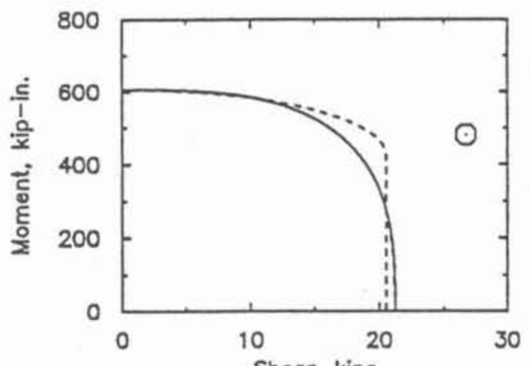

Fig. 3.44 Interaction Curves for Test RM-2C

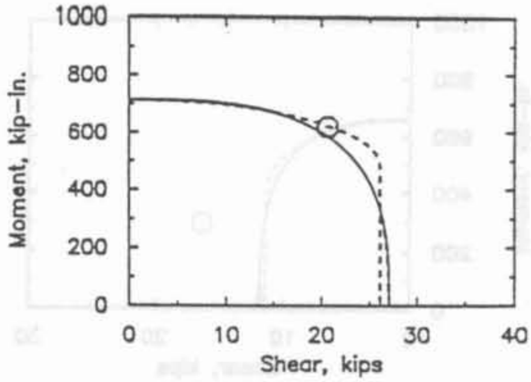

Fig. 3.45 Interaction Curves for Test RM-3A

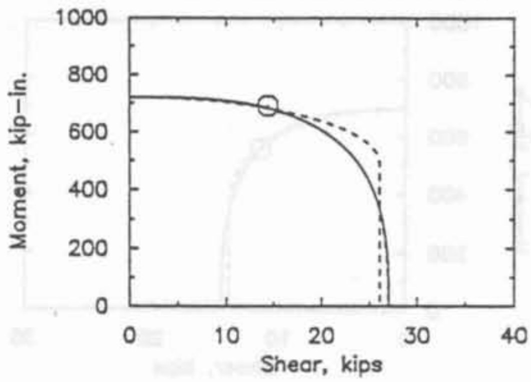

Fig. 3.46 Interaction Curves for Test RM-4A

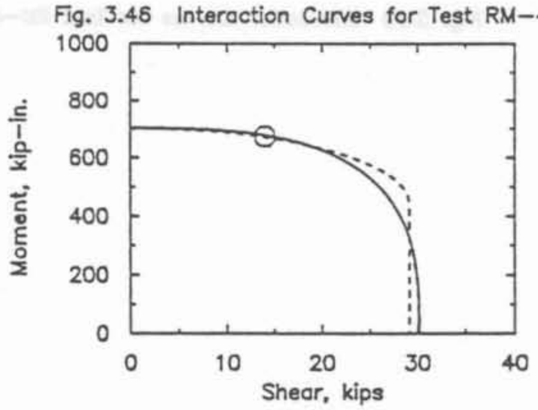

Fig. 3.47 Interaction Curves for Test RM-4C

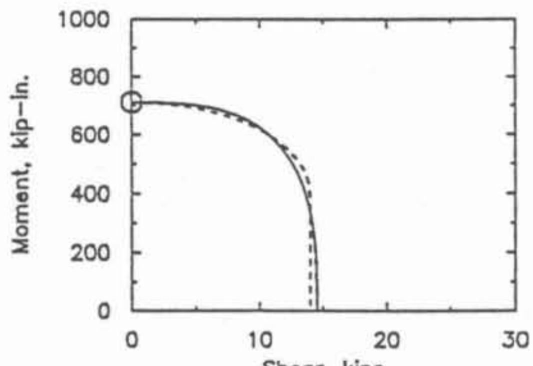

Fig. 3.48 Interaction Curves for Test RM-1B

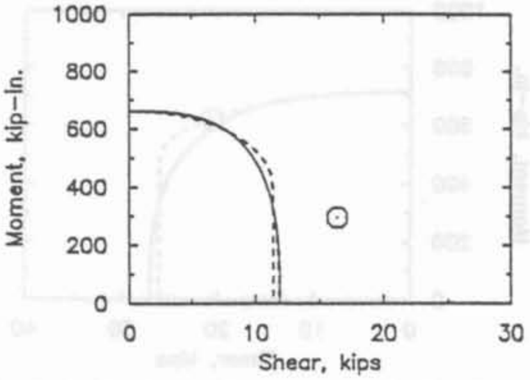

Fig. 3.49 Interaction Curves for Test RM-2B

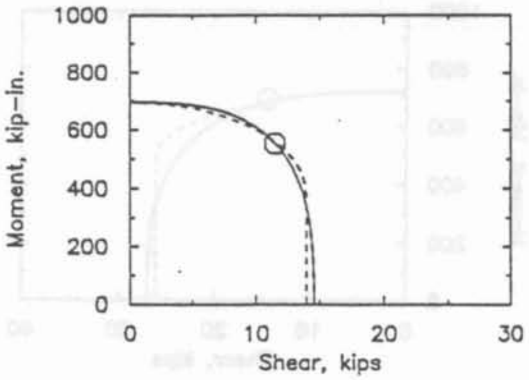

Fig. 3.50 Interaction Curves for Test RM-4B

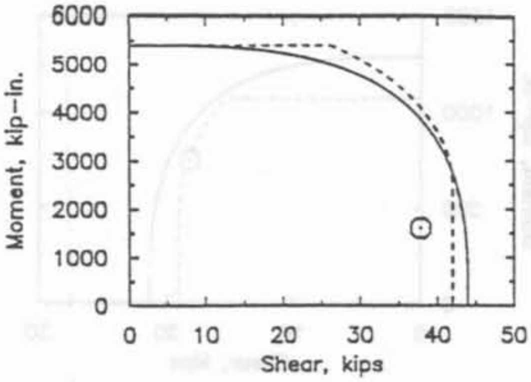

Fig. 3.51 Interaction Curves for Test D-1

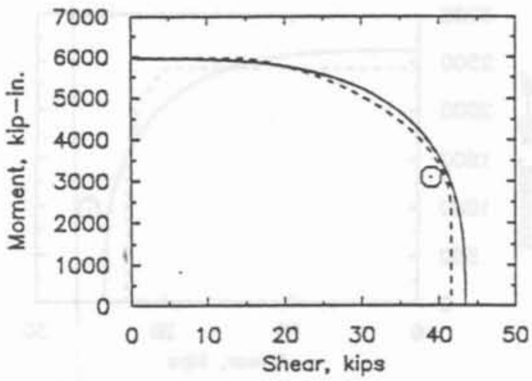

Fig. 3.52 Interaction Curves for Test D-2

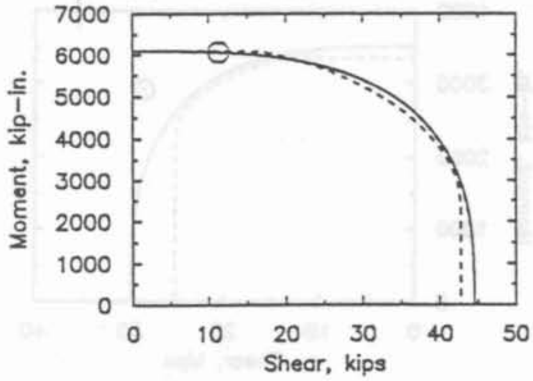

Fig. 3.53 Interaction Curves for Test D-3

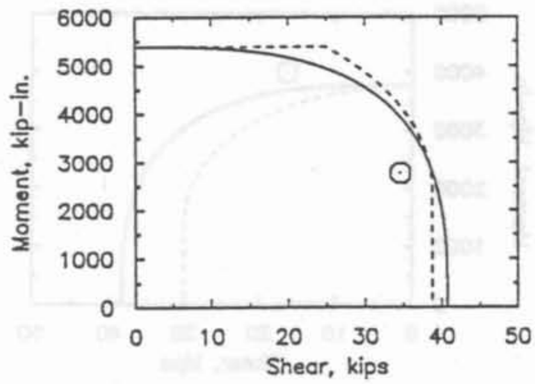

Fig. 3.54 Interaction Curves for Test D-5A

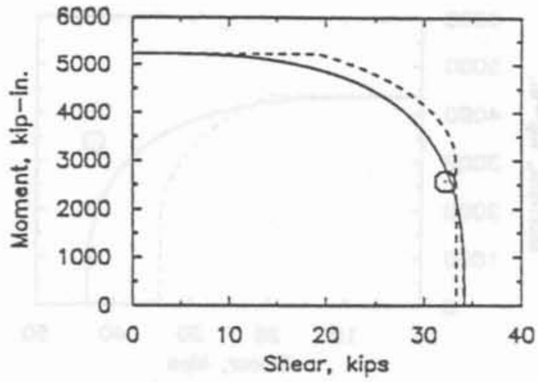

Fig. 3.55 Interaction Curves for Test D-5B

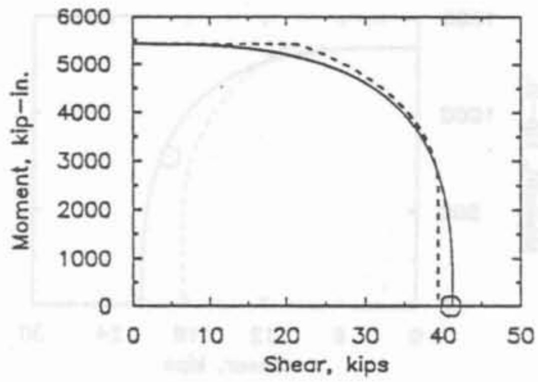

Fig. 3.56 Interaction Curves for Test D-6A

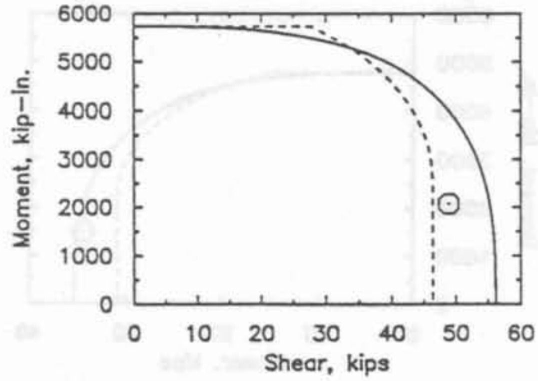

Fig. 3.57 Interaction Curves for Test D-6B

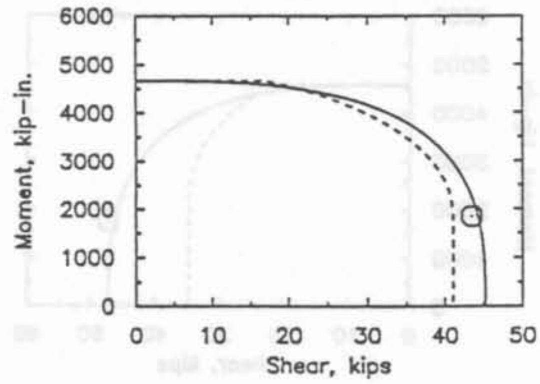

Fig. 3.58 Interaction Curves for Test D-7A

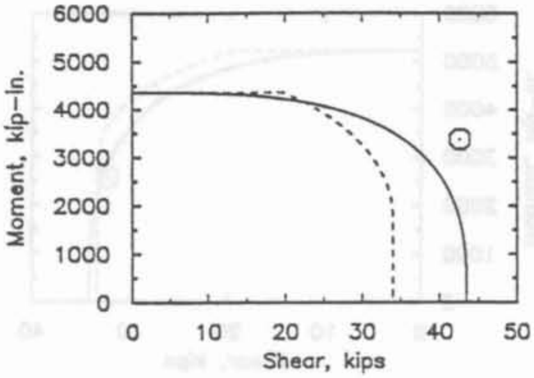

Fig. 3.59 Interaction Curves for Test D-7B

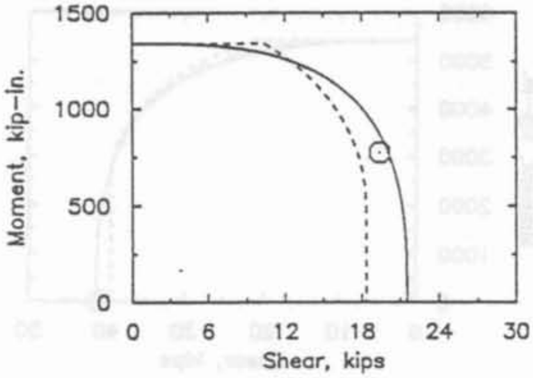

Fig. 3.60 Interaction Curves for Test D-8A

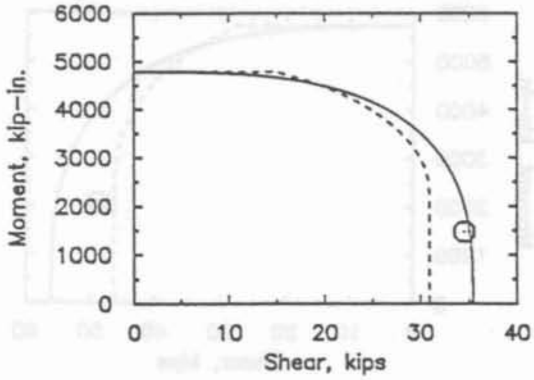

Fig. 3.61 Interaction Curves for Test D-9A

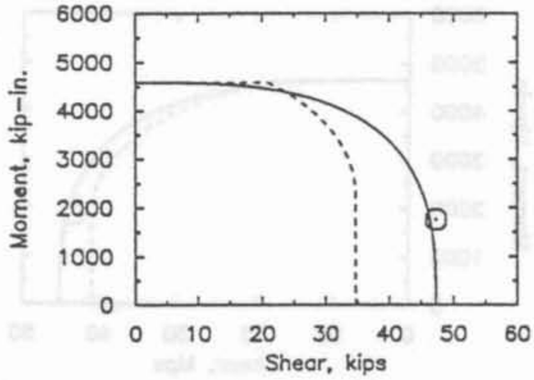

Fig. 3.62 Interaction Curves for Test D-9B

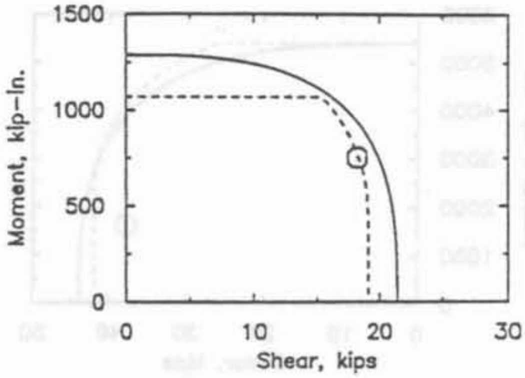

Fig. 3.63 Interaction Curves for Test R-0

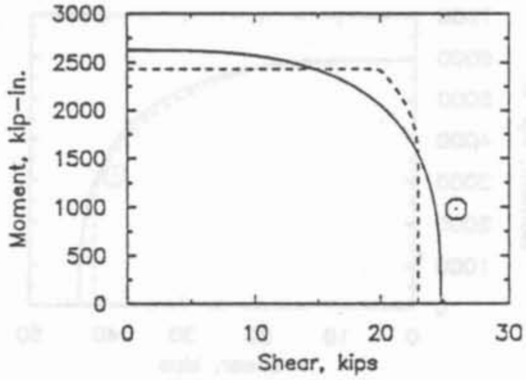

Fig. 3.64 Interaction Curves for Test R-1

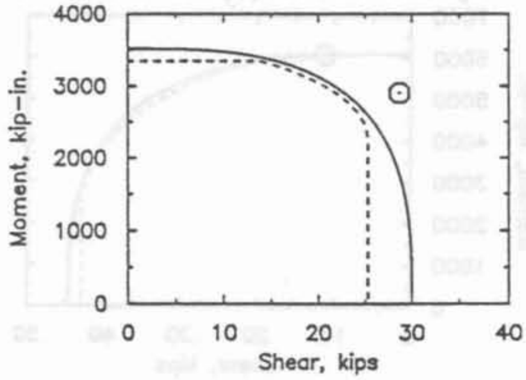

Fig. 3.65 Interaction Curves for Test R-2

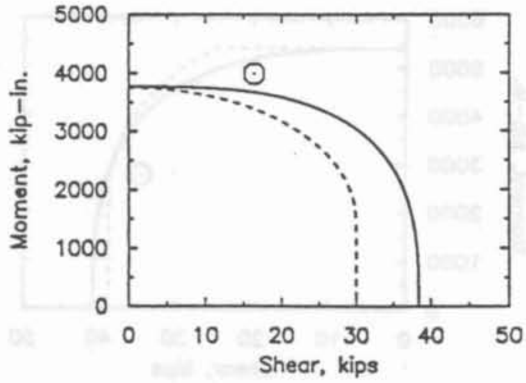

Fig. 3.66 Interaction Curves for Test R-3

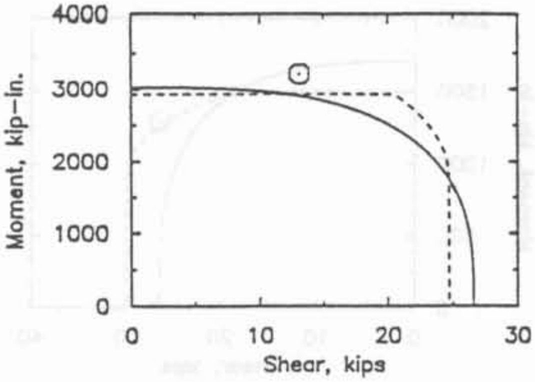

Fig. 3.67 Interaction Curves for Test R-4

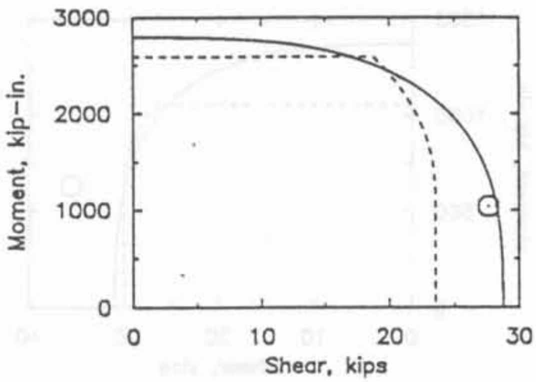

Fig. 3.68 Interaction Curves for Test R-5

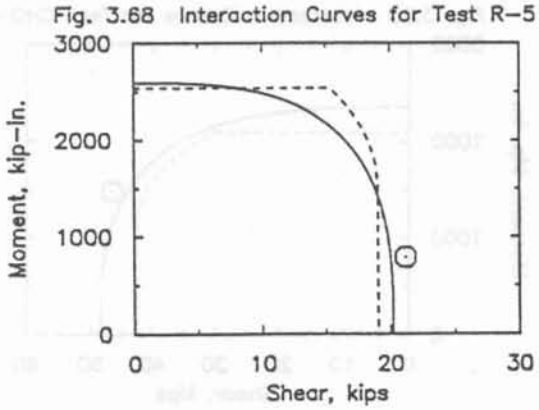

Fig. 3.69 Interaction Curves for Test R-6

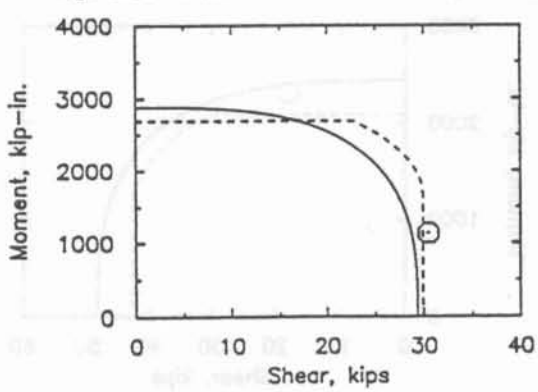

Fig. 3.70 Interaction Curves for Test R-7

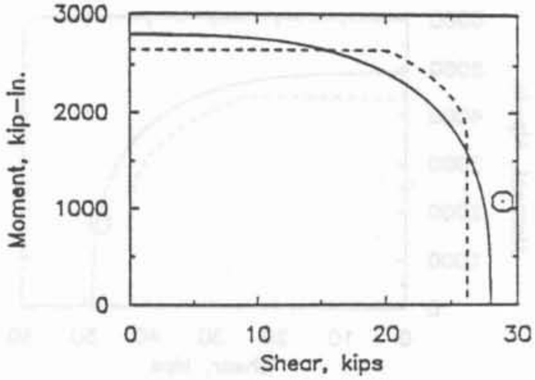

Fig. 3.71 Interaction Curves for Test R-8

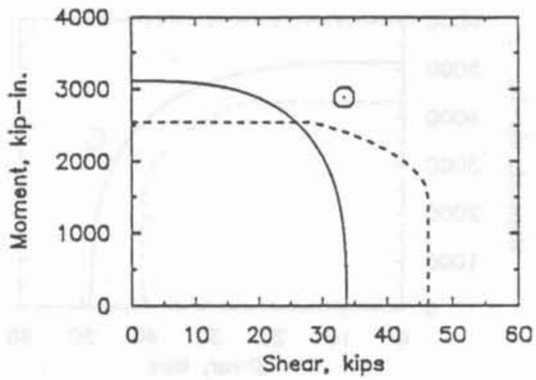

Fig. 3.72 Interaction Curves for Test C-1

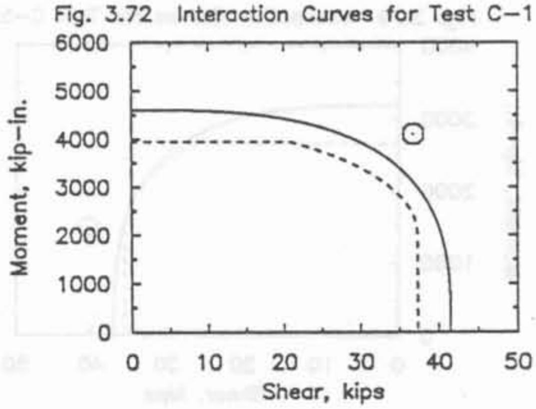

Fig. 3.73 Interaction Curves for Test C-2

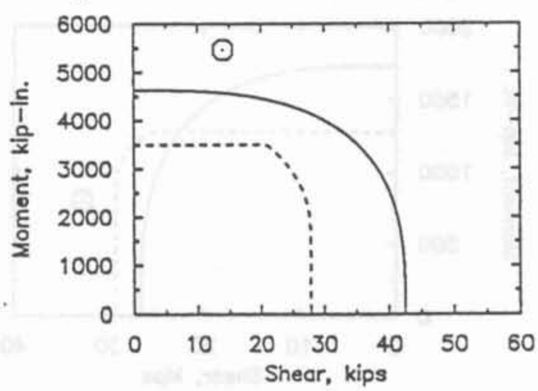

Fig. 3.74 Interaction Curves for Test C-3

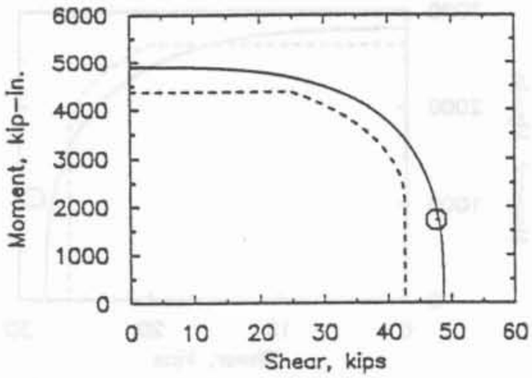

Fig. 3.75 Interaction Curves for Test C-4

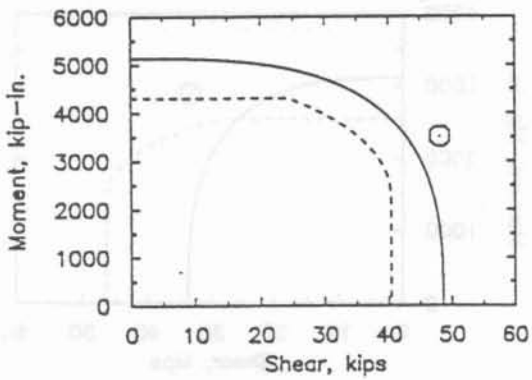

Fig. 3.76 Interaction Curves for Test C-5

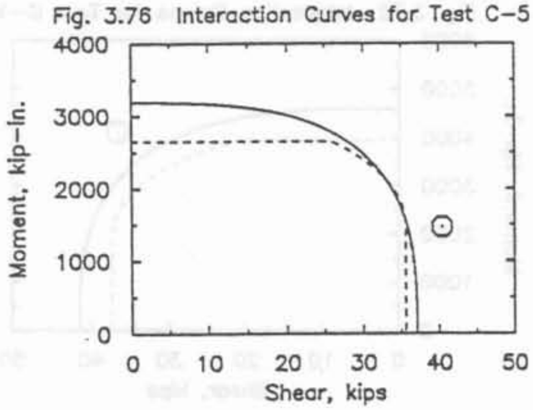

Fig. 3.77 Interaction Curves for Test C-6

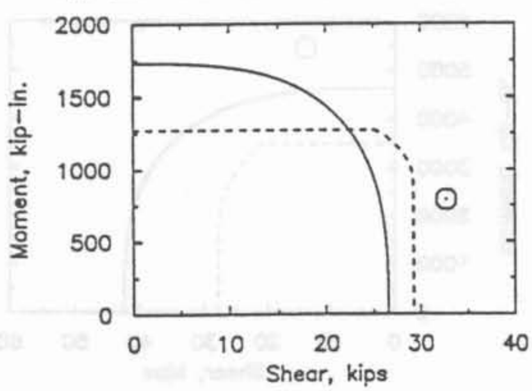

Fig. 3.78 Interaction Curves for Test G-1

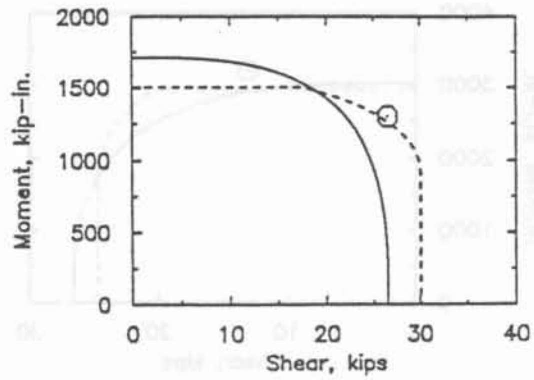

Fig. 3.79 Interaction Curves for Test G-2

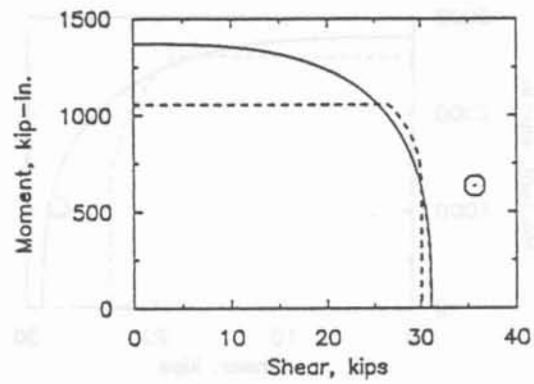

Fig. 3.80 Interaction Curves for Test CHO-3

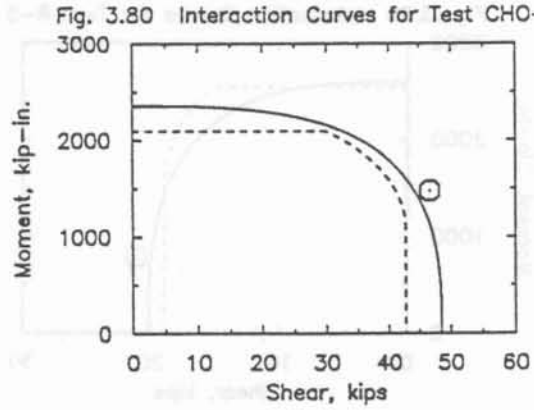

Fig. 3.81 Interaction Curves for Test CHO-4

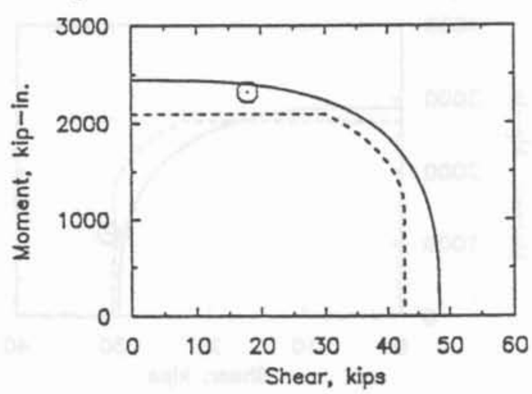

Fig. 3.82 Interaction Curves for Test CHO-5

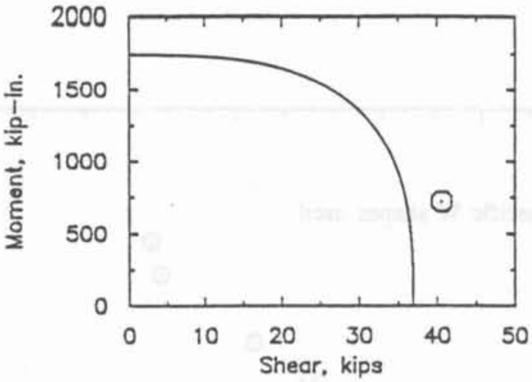

Fig. 3.83 Interaction Curves for Test CHO-6

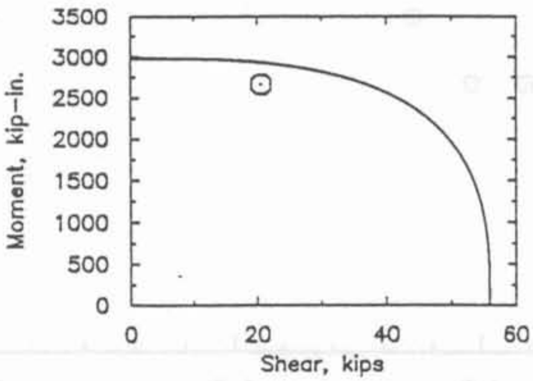

Fig. 3.84 Interaction Curves for Test CHO-7

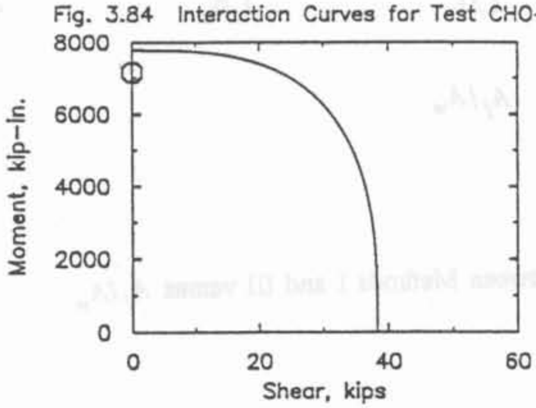

Fig. 3.85 Interaction Curves for Test WJE-1

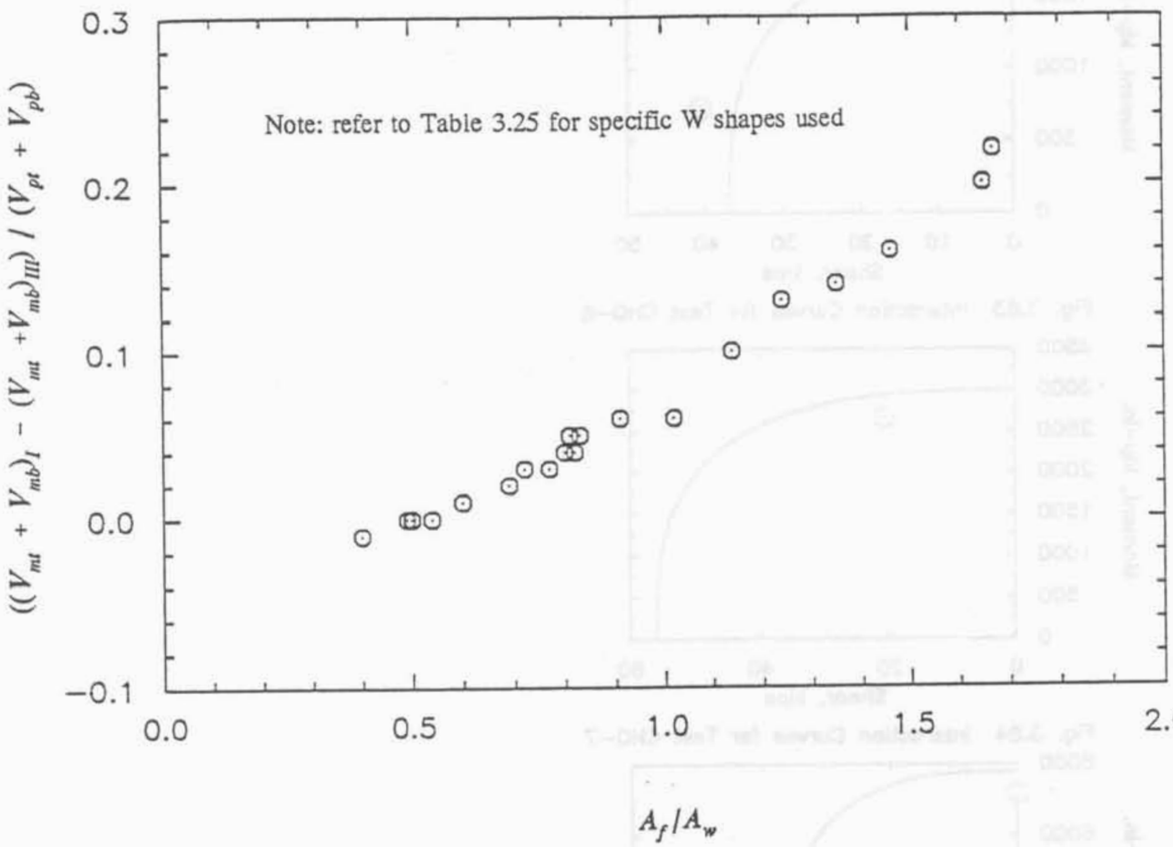

Fig. 3.86 Difference Between Methods I and III versus A_f / A_w

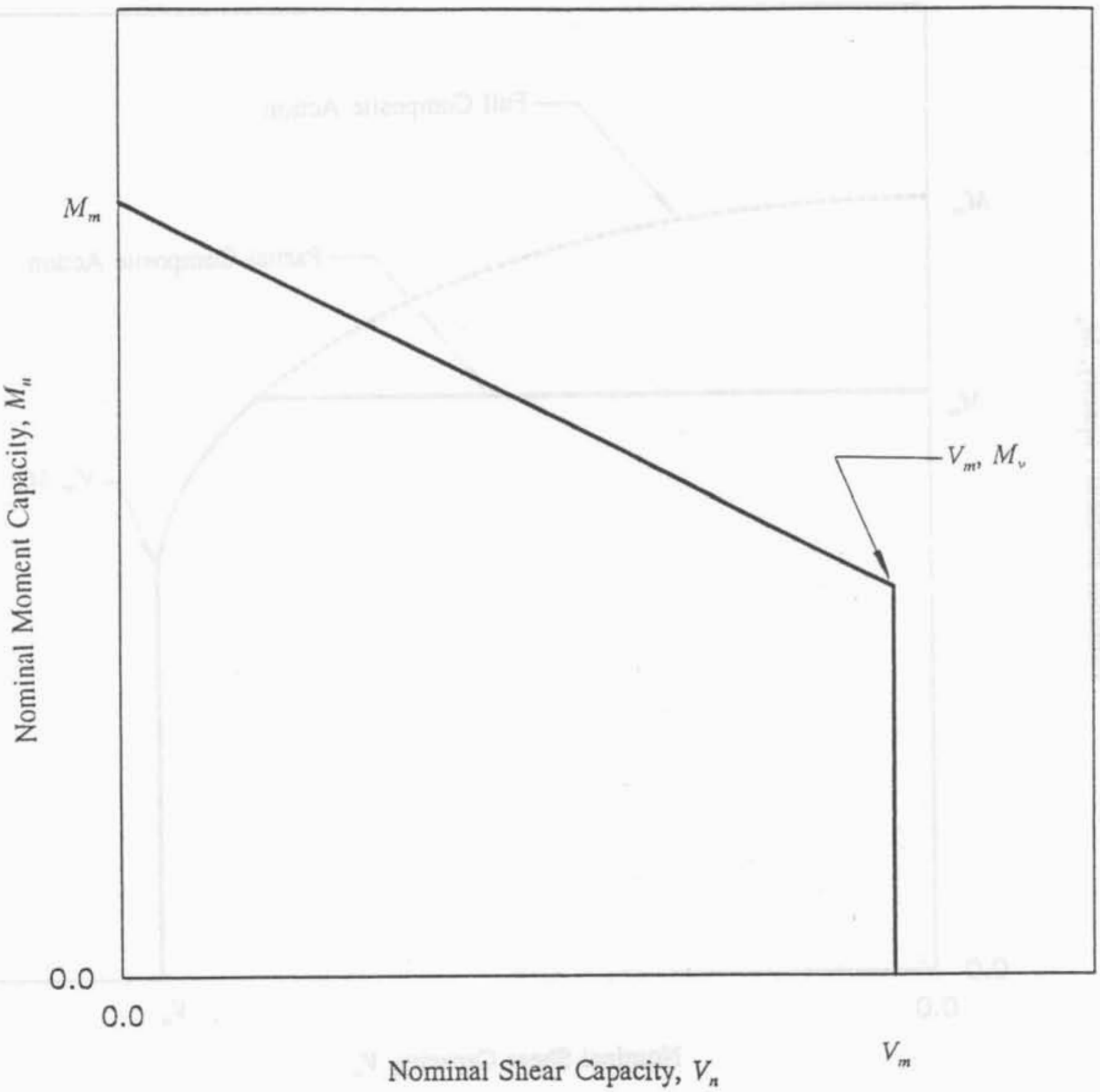

Fig. 3.87 Linear Moment-Shear Interaction Curve
(Redwood & Shrivastava 1980)

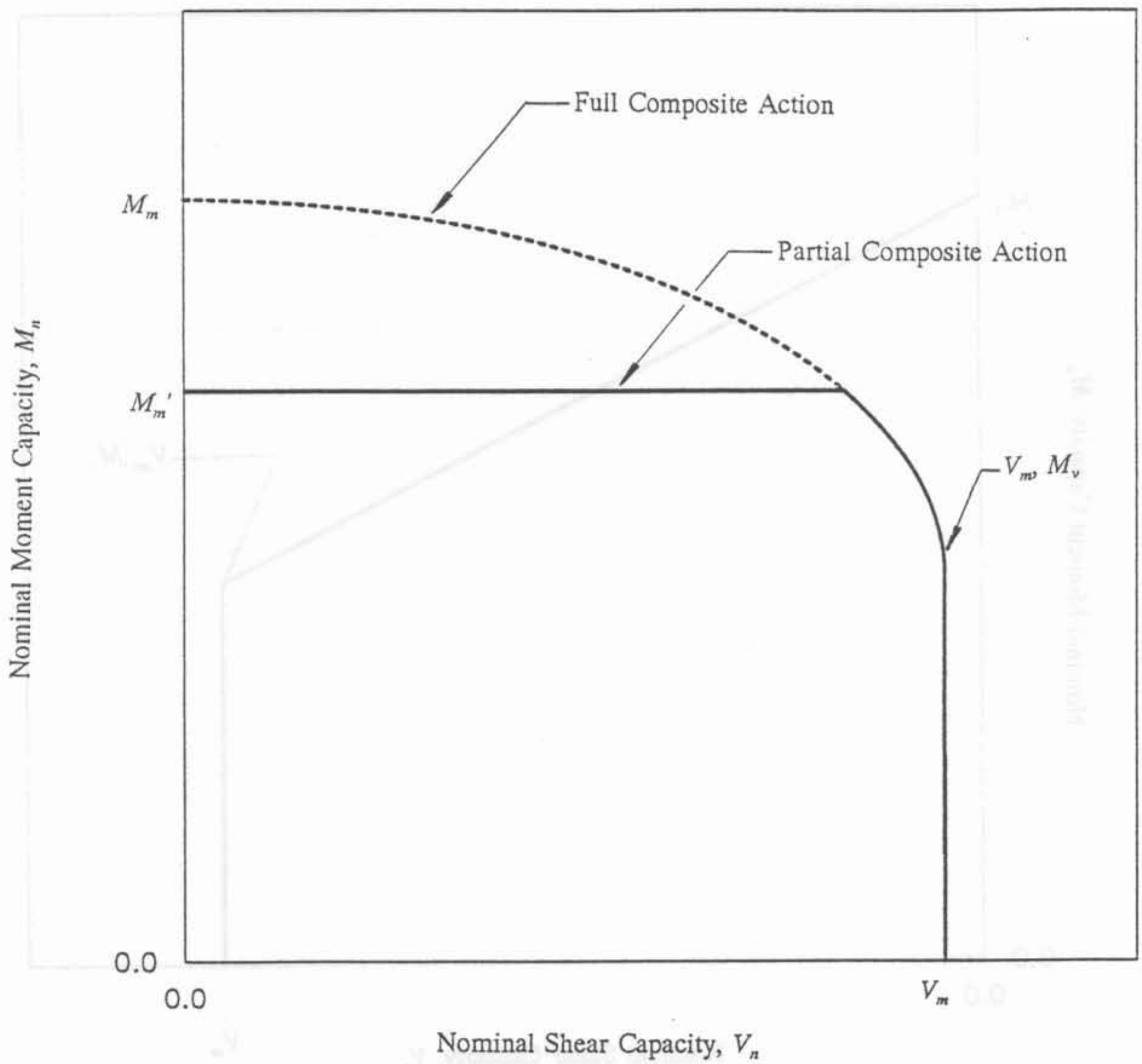

Fig. 3.88 Curvlinear Moment-Shear Interaction Curve
(Redwood & Shrivastava 1980)

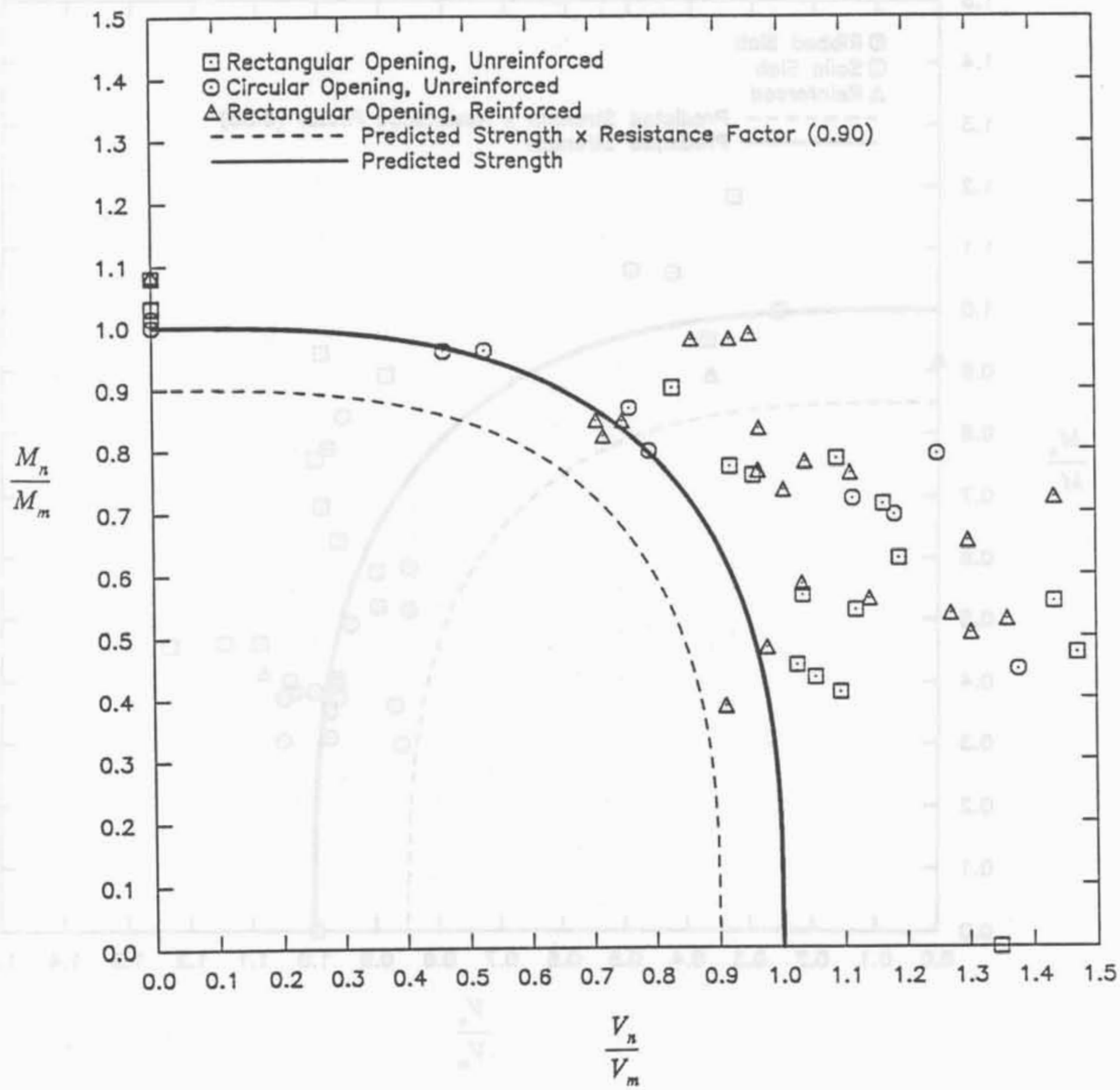

Fig. 3.89 Comparison of Method III with Test Results for Steel Beams

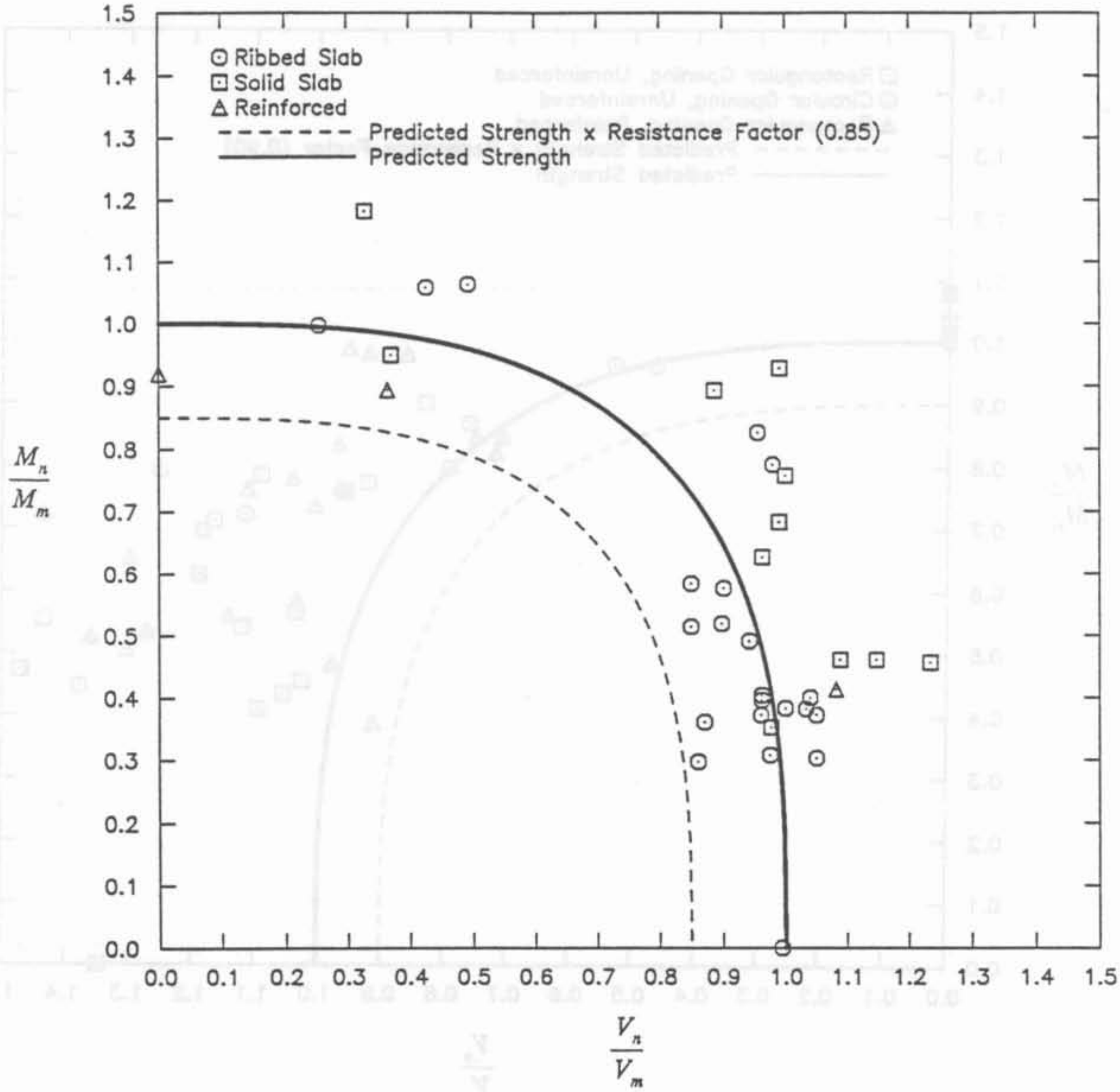

Fig. 3.90 Comparison of Method III with Test Results
for Composite Beams

APPENDIX A

DEFINITIONS AND NOTATION

A.1 Definitions (Darwin 1990)

The following terms apply to members with web openings.

bottom tee - region of a beam below an opening.

bridging - separation of the concrete slab from the steel section in composite beams. The separation occurs over an opening between the low moment end of the opening and a point outside the opening past the high moment end of the opening.

high moment end - the edge of an opening subjected to the greater primary bending moment. The secondary and primary bending moments act in the same direction.

low moment end - the edge of an opening subjected to the lower primary bending moment. The secondary and primary bending moments act in opposite directions.

opening index - parameter used to limit opening size and aspect ratio.

plastic neutral axis - position in steel section, or top or bottom tees, at which the stress changes abruptly from tension to compression.

primary bending moment - bending moment at any point in a beam caused by external loading.

reinforcement - longitudinal steel bars welded above and below an opening to increase section capacity.

reinforcement, slab - reinforcing steel within a concrete slab.

secondary bending moment - bending moment within a tee that is induced by the shear carried by the tee.

tee - region of a beam above or below an opening.

top tee - region of a beam above an opening.

unperforated member - section without an opening. Refers to properties of the member at the position of the opening.

A.2 Notation (Darwin 1990)

A	Gross transformed area of a tee
A_f	Area of flange
A_r	Cross-sectional area of reinforcement along top or bottom edge of an opening
A_s	Cross-sectional area of steel in unperforated member
A_{sc}	Cross-sectional area of shear stud
A_{sn}	Net area of steel section with opening and reinforcement
A_{st}	Net steel area of top tee
A_{vc}	Effective concrete shear area = $3t_s t_e$
D_o	Diameter of circular opening
E	Modulus of elasticity of steel
E_c	Modulus of elasticity of concrete
F_y	Yield strength of steel
$\bar{F}_y$	Reduced axial yield strength of steel; see Eqs. 2.24 and 2.25
F_{yf}	Yield strength of the flange
F_{yr}	Yield strength of opening reinforcement
F_{yw}	Yield strength of the web
M	Bending moment at center line of opening

M_{bh} , M_{bl}	Secondary bending moment at high and low moment ends of bottom tee, respectively.
M_m	Maximum nominal bending capacity at the location of an opening
M_n	Nominal bending capacity
M_p	Plastic bending capacity of an unperforated steel beam
M_{pc}	Plastic bending capacity of an unperforated composite beam
M_{th} , M_{tl}	Secondary bending moment at high and low moment ends of top tee, respectively
M_u	Factored bending moment
N	Number of shear connectors between the high moment end of an opening and the support
N_o	Number of shear connectors over an opening
P	Axial force in top or bottom tee
P_b	Axial force in top tee
P_c	Axial force in concrete for a section under pure bending
P_{ch} , P_{cl}	Axial force in concrete at high and low moment ends of opening, respectively, for a section at maximum shear capacity
PNA	Plastic neutral axis
P_r	Axial force in opening reinforcement
P_t	Axial force in top tee
Q_n	Individual shear connector capacity, including reduction factor for ribbed slabs

R	Ratio of factored load to design capacity at an opening
	$= V_u/\phi V_n$
	$= M_u/\phi M_n$
R_s	Strength reduction factor for shear studs in ribbed slabs
R_{wr}	Required strength of a weld
S	Clear space between openings
T'	Tensile force in net steel section
V	Shear at opening
V_b	Shear in bottom tee
$\bar{V}_c$	Calculated shear carried by concrete slab = $V_{pt}(\mu/\upsilon - 1) \geq 0$, or $V_{mt(sh)} - V_{pt}$, whichever is less
V_m	Maximum nominal shear capacity at the location of an opening
V_{mb} , V_{mt}	Maximum nominal shear capacity of bottom and top tees, respectively
$V_{mt(sh)}$	Pure shear capacity of top tee
V_P	Coefficient of variation on test-to-prediction ratio
V_p	Plastic shear capacity of top or bottom tee
$\bar{V}_p$	Plastic shear capacity of unperforated beam
V_{pb} , V_{pt}	Plastic shear capacity of bottom and top tees, respectively
V_R	Coefficient of variation on resistance
V_t	Shear in top tee
V_u	Factored shear
Z	Plastic section modulus

a_o	Length of opening
$\bar{a}$	Depth of concrete compressive block
b	Projecting width of flange or reinforcement
b_e	Effective width of concrete slab
b_f	Width of flange
b_r	Width of reinforcement at top or bottom of opening
d	Depth of steel section
d_h , d_l	Distance from top of steel section to centroid of concrete force at high and low moment ends of opening, respectively
d_r	Distance from outside edge of flange to centroid of opening reinforcement; may have different values in top and bottom tees
e	Eccentricity of opening; always positive for steel sections; positive up for composite sections
f_c'	Compressive (cylinder) strength of concrete
g_h , g_l	Distance from outside edge of flange to secondary bending neutral axis in top tee at high and low moment ends of opening, respectively
h_o	Depth of opening
p_o	Opening parameter $= \dfrac{a_o}{h_o} + \dfrac{6h_o}{d}$
s, s_b , s_t	Depth of a tee, bottom tee and top tee, respectively
$\overline{s}$, $\overline{s_b}$, $\overline{s_t}$	Effective depth of a tee, bottom tee and top tee, respectively, to account for movement of *PNA* when an opening is reinforced; used <u>only for calculation of υ</u>, when $\upsilon \geq \mu$

t	Thickness of flange or reinforcement
t_e	Effective thickness of concrete slab
t_f	Thickness of flange
t_s	Total thickness of concrete slab
t_s'	Thickness of concrete slab above the rib
t_w	Thickness of web
x	Distance from top of flange to plastic neutral axis in flange or web of a composite beam
z	Distance between points about which secondary bending moments are calculated
$\alpha_t, \beta_t, \gamma_t$	Variables used to calculate V_{mt}
ΔA_s	Net reduction in area of steel section due to presence of an opening and reinforcement = $h_o t_w - 2A_r$
λ	Constant used in linear approximation of von Mises yield criterion; recommended value = $\sqrt{2}$
μ	Dimensionless ratio relating the secondary bending moment contributions of concrete and opening reinforcement to the product of the plastic shear capacity of a tee and the depth of the tee $= \dfrac{2P_r d_r + P_{ch} d_h - P_{cl} d_l}{V_{pt} s_t}$
$\upsilon, \upsilon_b, \upsilon_t$	Ratio of length to depth or length to effective depth for a tee, bottom tee or top tee, respectively = a_o / s , $a_o / \overline{s}$
τ	Average shear stress

ϕ	Resistance factor

Subscripts:

b	Bottom tee
m	Maximum or mean
n	Nominal
t	Top tee
u	Factored

APPENDIX B

SHEAR CAPACITY EXPRESSIONS FOR COMPARISON WITH TEST DATA

B.1 Method I

The top and bottom tee shear capacities determined by Method I, considering different yield strengths for the web, flange, and stiffener, are calculated using the following expressions.

$$V_{mt} = \frac{\beta - \sqrt{\beta^2 - 4\alpha\gamma}}{2\alpha} \tag{B.1}$$

in which

$$\alpha = 3 + \frac{2\sqrt{3}\,a_o}{s_t}$$

$$\beta = 2a_o(F_{yf}(b_f - t_w) + \lambda F_{yw}t_w)$$

$$+ \frac{2\sqrt{3}}{s_t}F_{yf}(b_f - t_w)(s_t^2 - s_t t_f + t_f^2)$$

$$+ 2\sqrt{3}\,(\lambda F_{yw}t_w s_t - F_{yr}t_r(b_r - t_w))$$

$$+ \frac{2\sqrt{3}}{s_t}(P_{ch}d_h - P_{cl}d_l)$$

$$+ \sqrt{3}\,(P_{ch} - P_{cl})$$

$$+ \frac{4\sqrt{3}}{s_t}F_{yr}t_r(b_r - t_w)(s_t - y_r)$$

$$\gamma = (F_{yf}t_f(b_f - t_w))^2 + (\lambda F_{yw}t_w s_t)^2 - (F_{yr}t_r(b_r - t_w))^2$$

$$+ (P_{ch} - P_{cl})(F_{yf}t_f(b_f - t_w) + \lambda F_{yw}t_w s_t + F_{yr}t_r(b_r - t_w))$$

$$+ 2(P_{ch}d_h - P_{cl}d_l)(F_{yf}(b_f - t_w) + \lambda F_{yw}t_w) - \frac{P_{ch}^2}{2} - \frac{P_{cl}^2}{2}$$

$$+ 2F_{yf}(b_f - t_w)\lambda F_{yw}t_w(s_t^2 - s_t t_f + t_f^2)$$

$$+ 4F_{yr}t_r(b_r - t_w)(s_t - y_r)(F_{yf}(b_f - t_w) + \lambda F_{yw}t_w)$$

$$- 2F_{yr}t_r(b_r - t_w)(F_{yf}t_f(b_f - t_w) + \lambda F_{yw}t_w s_t)$$

B.2 Methods II and III

The yield strengths of the web and reinforcement are differentiated in Methods II and III as follows. The yield strength of the web is accounted for in the calculation of V_{pt} and V_{pb} as given by Eqs. 2.18 and 2.22.

The yield strength of the reinforcement is accounted for in the expression for μ, given by

$$\mu = \frac{P_{ch}d_h - P_{cl}d_l + 2P_r d_r}{V_{pt}s_t} \qquad \text{(B.2)}$$

in which $P_r = F_{yr}(b_r - t_w)t_r$

APPENDIX C

DERIVATION AND CALCULATION OF VALUES

FOR

THEORETICAL COMPARISON OF METHODS I, II AND III

In this appendix, calculations are presented which provide the basis for values used in comparing Methods I, II, and III in Section 2.3.4.

C.1 Overprediction of F_y by the Linear Approximation of the von Mises Yield Function

The overprediction of normal stress in a tee under low shear stress by the linear approximation of the von Mises yield function can be as high as 41% when $\lambda = 1.414$ (Method III, $\mu = 0.0$). Design considerations, however, limit υ to 12.0 (Darwin 1990). The actual effect of this overprediction, as limited by design considerations, can be determined by comparing Methods II and III, which employ the von Mises yield function and its linear approximation, respectively.

The values of V_{mt}/V_{pt} for Methods II and III when $\mu = 0.0$ and $\upsilon = 12.0$ for $\lambda = 1.207$ and $\lambda = 1.414$ follow.

$$\frac{V_{mt}}{V_{pt}}(II) = \frac{\sqrt{3\upsilon^2+9}}{\upsilon^2+3} = \frac{\sqrt{(3)(144)+9}}{144+3} = 0.143 \tag{C.1}$$

$$\frac{V_{mt}}{V_{pt}}(III) = \frac{\lambda\sqrt{3}}{\upsilon+\sqrt{3}} = \frac{(1.207)\sqrt{3}}{12+\sqrt{3}} = 0.152;\ \lambda = 1.207 \tag{C.2}$$

$$\frac{V_{mt}}{V_{pt}}(III) = \frac{\lambda\sqrt{3}}{\upsilon+\sqrt{3}} = \frac{(1.414)\sqrt{3}}{12+\sqrt{3}} = 0.178;\ \lambda = 1.414 \qquad \text{(C.3)}$$

The difference between Methods II and III is

$$V_{mt(III)} - V_{mt(II)} = (0.152 - 0.143)V_{pt} = 0.009V_{pt};\ \lambda = 1.207 \qquad \text{(C.4)}$$

$$V_{mt(III)} - V_{mt(II)} = (0.178 - 0.143)V_{pt} = 0.035V_{pt};\ \lambda = 1.414 \qquad \text{(C.5)}$$

The ratio of the maximum shear strengths using the two methods is

$$\frac{V_{mt(III)}}{V_{mt(II)}} = \frac{0.152}{0.143} = 1.063;\ \lambda{=}1.207 \qquad \text{(C.6)}$$

$$\frac{V_{mt(III)}}{V_{mt(II)}} = \frac{0.178}{0.143} = 1.245;\ \lambda{=}1.414 \qquad \text{(C.7)}$$

C.2 Overprediction of τ_{xy} by the Linear Approximation of the von Mises Yield Function

The overprediction of shear stress in the web of a tee under high shear stress by the linear approximation of the von Mises yield function can be as high as 9.7% when $\lambda = 1.414$ and $\upsilon = 0.717$ (Method III, $\mu = 0.0$). This overprediction would be even higher without the limit of $0.577F_y$ on the shear stress. A tee with such stocky dimensions is not very likely, but is possible, and is something that should be considered. The effect of this overprediction can be determined by comparing Methods II and III, which employ the von Mises yield function and its linear approximation, respectively.

The von Mises yield function can be expressed as

$$\bar{F}_y^{\,2} + 3\tau_{xy}^2 = F_y^2 \qquad \text{(C.13)}$$

Dividing Eq. C.13 by F_y^2, and rearranging gives

$$\frac{\bar{F}_y}{F_y} = \left(1 - 3\left(\frac{\tau_{xy}}{F_y}\right)^2\right)^{1/2} \tag{C.14}$$

By substituting $\tau_{xy} = V_{mt} / s_t t_w$, Eq. C.14 can be rewritten in terms of V_{mt} and V_{pt}.

$$\frac{\bar{F}_y}{F_y} = \left(1 - \left(\frac{V_{mt}}{V_{pt}}\right)^2\right)^{1/2} \tag{C.15}$$

The linear approximation of the von Mises yield function can be expressed as

$$\bar{F}_y = \lambda F_y - \sqrt{3}\,\tau_{xy} \tag{C.16}$$

Dividing Eq. C.16 by F_y and rearranging gives,

$$\frac{\bar{F}_y}{F_y} = \lambda - \frac{\sqrt{3}\,\tau_{xy}}{F_y} \tag{C.17}$$

By substituting $\tau_{xy} = V_{mt}/(s_t t_w)$ into Eq. C.17, the following expression is obtained

$$\frac{\bar{F}_y}{F_y} = \lambda - \frac{V_{mt}}{V_{pt}} \tag{C.18}$$

Eq. C.15 and C.18 are useful in comparing Methods II and III when $\tau_{xy}/F_y = 0.577$. The point at which the maximum difference occurs in the predicted shear stress in the web between the von Mises yield function and its linear approximation can now be easily predicted. This occurs when $V_{mt}/V_{pt(III)} = 1.0$ due to the maximum permissible shear stress. Eq. C.18 yields

$$\frac{\bar{F}_y}{F_y} = \lambda - 1.0 = 0.207 \;;\; \lambda = 1.207 \tag{C.19}$$

$$\frac{\bar{F}_y}{F_y} = \lambda - 1.0 = 0.414 \;;\; \lambda = 1.414 \tag{C.20}$$

The respective shear capacities can be determined by substituting the two preceding values for F_y /F_y into Eq. C.15, which gives

$$\frac{V_{mt}}{V_{pt}} = \left(1 - \left(\frac{\bar{F}_y}{F_y} \right)^2 \right)^{1/2} = \sqrt{1 - (0.414)^2} = 0.910;\ \lambda=1.414 \tag{C.21}$$

The corresponding ratios and differences between Methods II and III are

$$\frac{V_{mt(III)}}{V_{mt(II)}} = \frac{1.000}{0.978} = 1.023;\ \lambda=1.207 \tag{C.22}$$

$$\frac{V_{mt(III)}}{V_{mt(II)}} = \frac{1.000}{0.910} = 1.099;\ \lambda=1.414 \tag{C.23}$$

$$V_{mt(III)} - V_{mt(II)} = (1.000 - 0.978)V_{pt} = 0.022V_{pt};\ \lambda=1.207 \tag{C.24}$$

$$V_{mt(III)} - V_{mt(II)} = (1.000 - 0.910)V_{pt} = 0.090V_{pt};\ \lambda=1.414 \tag{C.25}$$

The ratio, τ_{xy}/F_y, for the shear capacities predicted by Methods II and III, respectively, are

$$\frac{\tau_{xy}}{F_y}(II) = \frac{1}{\sqrt{3}} \left(1 - \left(\frac{\bar{F}_y}{F_y} \right)^2 \right)^{1/2} = \frac{1}{\sqrt{3}} \sqrt{1 - (0.207)^2} = 0.565\ ;\ \frac{\bar{F}_y}{F_y} = 0.207 \tag{C.26}$$

$$\frac{\tau_{xy}}{F_y}(II) = \frac{1}{\sqrt{3}} \left(1 - \left(\frac{\bar{F}_y}{F_y} \right)^2 \right)^{1/2} = \frac{1}{\sqrt{3}} \sqrt{1 - (0.414)^2} = 0.526\ ;\ \frac{\bar{F}_y}{F_y} = 0.414 \tag{C.27}$$

$$\frac{\tau_{xy}}{F_y}(III) = \frac{1}{\sqrt{3}} = 0.577 \tag{C.28}$$

APPENDIX D

GUIDELINES FOR PROPORTIONING AND DETAILING BEAMS WITH WEB OPENINGS (Darwin 1990)

To insure that the strength provided by a beam at a web opening is consistent with the design equations presented in section 2.4, a number of guidelines must be followed. Unless otherwise stated, these guidelines apply to unreinforced and reinforced web openings in both steel and composite beams. All requirements of the AISC Specifications (1986) should be applied. The steel sections should meet the AISC requirements for compact sections in both composite and non-composite members. $F_y \leq 65$ ksi.

D.1 Stability Considerations

To insure that local instabilities do not occur, consideration must be given to local buckling of the compression flange, web buckling, buckling of the tee-shaped compression zone above or below the opening, and lateral buckling of the compression flange.

D.1.1 Local buckling of compression flange or reinforcement

To insure that local buckling does not occur, the AISC (1986) criteria for compact sections applies. The width to thickness ratios of the compression flange or web reinforcement are limited by

$$\frac{b}{t} \leq \frac{65}{\sqrt{F_y}} \tag{D.1}$$

in which b = projecting width of flange or reinforcement

t = thickness of flange or reinforcement

F_y = yield strength in ksi

For a flange of width, b_f, and thickness, t_f, Eq. D.1 becomes

D.1.2 Web Buckling

To prevent buckling of the web, two criteria should be met:

(a) The opening parameter, p_o, should be limited to a maximum value of 5.6 for steel sections and 6.0 for composite sections.

$$p_o = \frac{a_o}{h_o} + \frac{6h_o}{d} \qquad \text{(D.3)}$$

in which a_o and h_o = length and width of opening, respectively

d = depth of steel section

(b) The web width-thickness ratio should be limited as follows

$$\frac{d - 2t_f}{t_w} \leq \frac{520}{\sqrt{F_y}} \qquad \text{(D.4)}$$

in which t_w = thickness of web

If $(d - 2t_f)/t_w \le 420/\sqrt{F_y}$, the web qualifies as stocky. In this case, the upper limit on a_o/h_o is 3.0 and the upper limit on V_m (maximum nominal shear capacity) for non-composite sections is $0.67\bar{V}_p$, in which $\bar{V}_p = F_y t_w d/\sqrt{3}$, the plastic shear capacity of the unperforated web. For composite sections, this upper limit may be increased by $\bar{V}_c$ which equals $V_{pt}(\mu/\upsilon - 1) \ge 0$, or $V_{mt(sh)} - V_{pt}$, whichever is less. All standard rolled W shapes qualify as stocky members.

If $420/\sqrt{F_y} < (d - 2t_f)/t_w \le 520/\sqrt{F_y}$, then a_o/h_o should be limited to 2.2, and V_m should be limited to $0.45\bar{V}_p$ for both composite and non-composite members. The limits on opening dimensions to prevent web buckling, presented in this section are summarized graphically in Figs. D.1, D.2, and D.3. Fig. D.1 graphs a_o/h_o versus h_o/d to determine permissible opening sizes. Figs. D.2 and D.3 graph a_o/s_t versus the value a_o/s_b that meets the opening dimension requirements of this section for steel ($p_o = 5.6$) and composite ($p_o = 6.0$) beams, respectively.

D.1.3 Buckling of tee-shaped compression zone

For steel beams only: The tee which is in compression should be investigated as an axially loaded column following the procedures of AISC (1986). For unreinforced members, this is not required when the aspect ratio of the tee ($\upsilon = a_o/s$) is less than or equal to 4. For reinforced openings, this check is only required for large openings in regions of high moment.

D.1.4 Lateral Buckling

For steel beams only: In members subject to lateral buckling of the compression flange, strength should not be governed by strength at the opening (calculated without regard to lateral buckling).

In members with unreinforced openings or reinforced openings with the reinforcement placed on both sides of the web, the torsional constant, J, should be multiplied by

$$\left(1 - \left(\frac{a_o}{L_b}\right)\frac{\Delta A_s}{t_w\left(d + 2b_f\right)}\right)^2 \leq 1 \qquad \text{(D.5)}$$

in which L_b = unbraced length of compression flange

$\Delta A_s = h_o t_w - 2A_r$

In members reinforced on only one side of the web, $A_r = 0$ for the calculation of ΔA_s in Eq. D.5. Members reinforced on one side of the web should not be used for long, laterally unsupported spans. For shorter spans the lateral bracing closest to the opening should be designed for an additional load equal to 2 percent of the force in the compression flange.

D.3 Other Considerations

D.3.1 Opening and tee dimensions

Opening dimensions are restricted based on the criteria in section D.1.2. Additional criteria also apply.

The opening depth should not exceed 70 percent of the section depth ($h_o \leq 0.7d$). The depth of the top tee should not be less than 15 percent of the depth of the steel section ($s_t \geq 0.15d$). The depth of the bottom tee, s_b, should not be less than $0.15d$ for steel sections or $0.12d$ for composite sections. The aspect ratios of the tees ($\upsilon = a_o/s$) should not be greater than 12 ($a_o/s_b \leq 12$, $a_o/s_t \leq 12$).

D.3.2 Corner radii

The corners of the opening should have minimum radii at least 2 times the thickness of the web, $2t_w$, or 5/8 in., whichever is greater.

D.3.3 Concentrated loads

No concentrated loads should be placed above an opening. Unless needed otherwise, bearing stiffeners are not required to prevent web crippling in the vicinity of an opening due to a concentrated load if

$$\frac{d - 2t_f}{t_w} \leq \frac{420}{\sqrt{F_y}} \tag{D.6}$$

$$\frac{b}{t} \leq \frac{54}{\sqrt{F_y}} \tag{D.7}$$

and the load is placed at least $d/2$ from the edge of the opening.

$$\frac{d - 2t_f}{t_w} \leq \frac{520}{\sqrt{F_y}} \tag{D.8}$$

$$\frac{b}{t} \leq \frac{65}{\sqrt{F_y}} \tag{D.9}$$

and the load is placed at least d from the edge of the opening. In any case, the edge of an opening should not be closer than a distance d to a support.

D.3.4 Circular openings

Circular openings may be designed using the expressions in section 2.4 by using the following substitutions for h_o and a_o.

Unreinforced web openings

$h_o = D_o$ for bending (D.10a)

$h_o = 0.9\ D_o$ for shear (D.10b)

$a_o = 0.45\ D_o$ (D.10c)

in which D_o = diameter of circular opening.

Reinforced web openings

$h_o = D_o$ for bending and shear (D.11a)

$a_o = 0.45\ D_o$ (D.11b)

D.3.5 Reinforcement

Reinforcement should be placed as close to an opening as possible, leaving adequate room for fillet welds, if required on both sides of the reinforcement. Continuous welds should be used to attach the reinforcement bars. A fillet weld may be used on one or both sides of the bar within the length of the opening. However, fillet welds should be used on both sides of the reinforcement on extensions past the opening. The required strength of the weld within the length of the opening is,

$$R_{wr} = \phi 2P_r \qquad \text{(D.12)}$$

in which R_{wr} = required strength of the weld

ϕ = 0.90 for steel beams and 0.85 for composite beams

P_r $= F_y A_r \leq F_y t_w a_o / 2\sqrt{3}$

A_r = cross-sectional area of reinforcement above or below the opening.

The reinforcement should be extended beyond the opening by a distance $l_1 \geq a_o/4$ or $\sqrt{3A_r}/2t_w$ whichever is greater, on each side of the opening (Figs 2.1 and 2.2). Within each extension, the required strength of the weld is

$$R_{wr} = \phi F_y A_r \tag{D.13}$$

If reinforcing bars are only used on one side of the web, the section should meet the following additional requirements.

$$A_r \leq \frac{A_f}{3} \tag{D.14}$$

$$\frac{a_o}{h_o} \leq 2.5 \tag{D.15}$$

$$\frac{s_t}{t_w}, \frac{s_b}{t_w} \leq \frac{140}{\sqrt{F_y}} \tag{D.16}$$

$$\frac{M_u}{V_u d} \leq 20 \tag{D.17}$$

in which A_f = area of flange

M_u and V_u = factored moment and shear at centerline of opening, respectively.

D.3.6 Spacing of openings

Openings should be spaced in accordance with the following criteria to avoid interaction between openings.

Rectangular openings:

$$S \geq h_o \tag{D.18a}$$

Circular openings:

$$S \geq a_o \left(\frac{\frac{V_u}{\phi V_p}}{1 - \frac{V_u}{\phi V_p}} \right) \tag{D.18b}$$

$$S \geq 1.5\, D_o \tag{D.19a}$$

$$S \geq D_o \left(\frac{\frac{V_u}{\phi V_p}}{1 - \frac{V_u}{\phi V_p}} \right) \tag{D.19b}$$

in which S = clear space between openings.

In addition to the requirements in Eqs. D.18 and D.19, openings in composite beams should be spaced so that

$$S \geq a_o \tag{D.20a}$$

$$S \geq 2.0\, d \tag{D.20b}$$

D.4 Additional Criteria for Composite Beams

In addition to the guidelines presented above, composite members should meet the following criteria.

D.4.1 Slab reinforcement

Transverse and longitudinal slab reinforcement ratios should be a minimum of 0.0025, based on the <u>gross</u> area of the slab, within a distance d or a_o, whichever is greater, of the opening. For beams with longitudinal ribs, the transverse reinforcement should be below the heads of the shear connectors.

D.4.2 Shear connectors

In addition to the shear connectors used between the high moment end of the opening and the support, a minimum of two studs per foot should be used for a distance d or a_o, whichever is greater, from the high moment end of the opening toward the direction of <u>increasing</u> moment.

D.4.3 Construction loads

If a composite beam is to be constructed without shoring, the section at the web opening should be checked for adequate strength as a <u>non-composite</u> member under factored dead and construction loads.

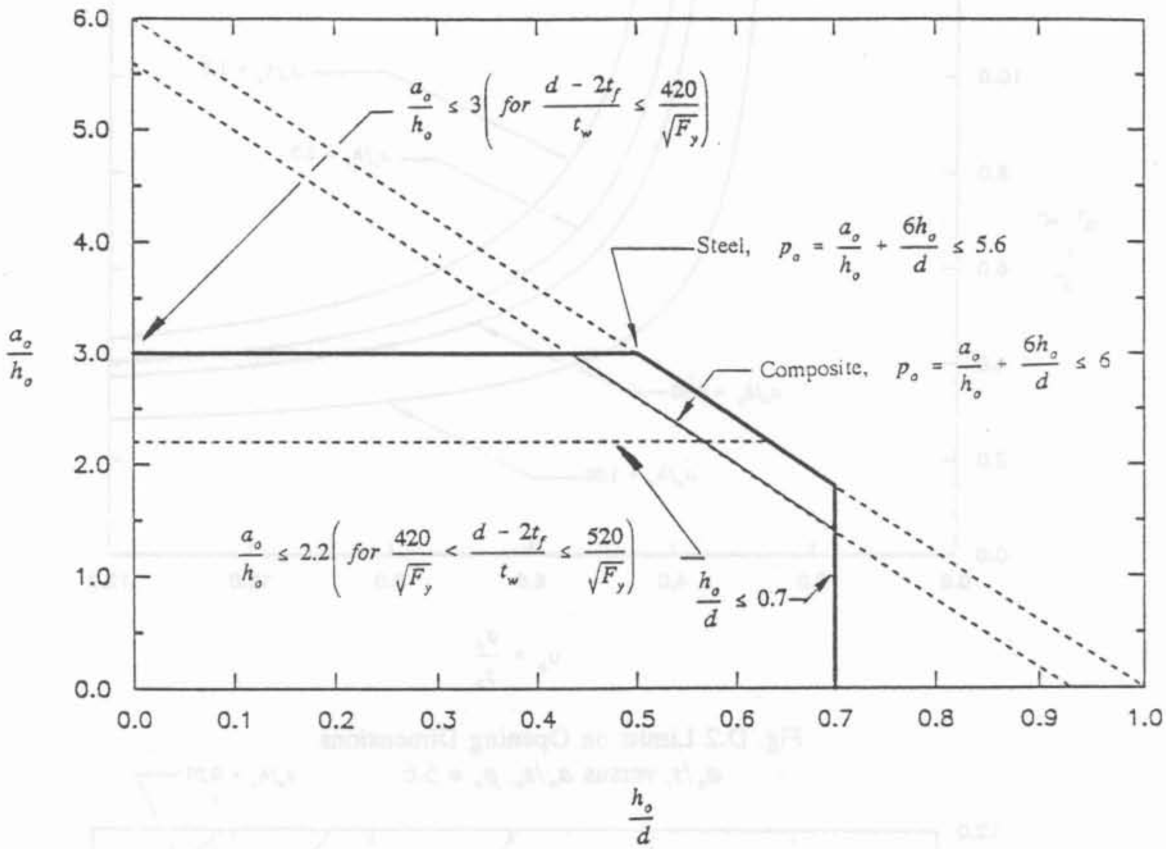

Fig. D.1 Limits on Opening Dimensions
a_o/h_o versus h_o/d (Darwin 1990)

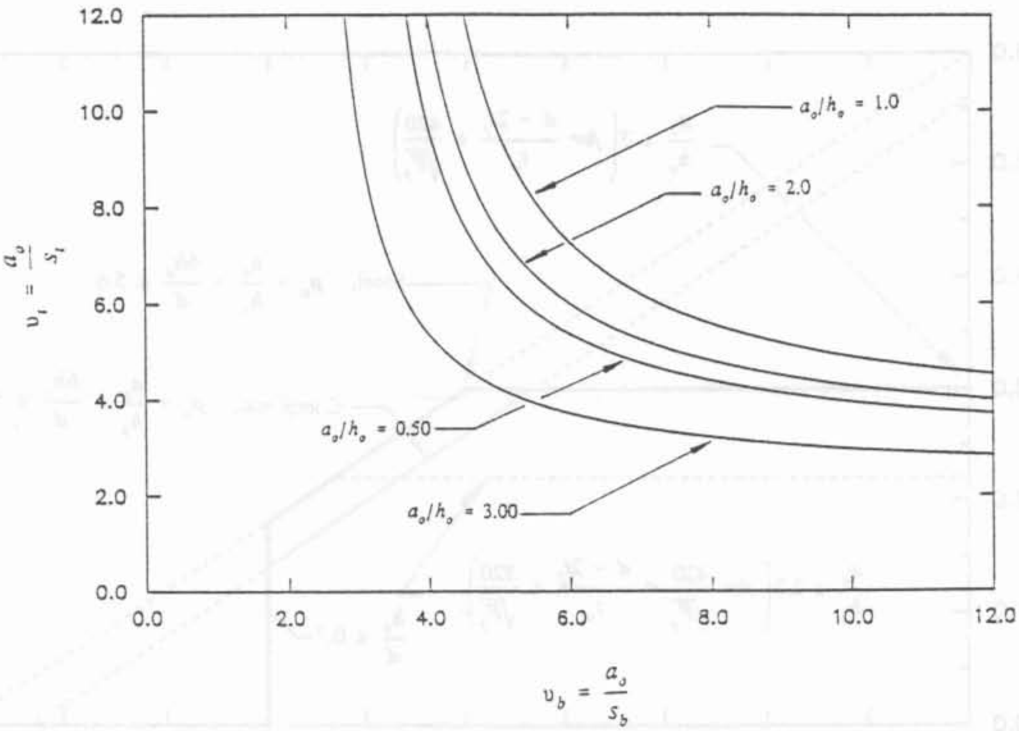

Fig. D.2 Limits on Opening Dimensions
a_o/s_t versus a_o/s_b, $p_o = 5.6$

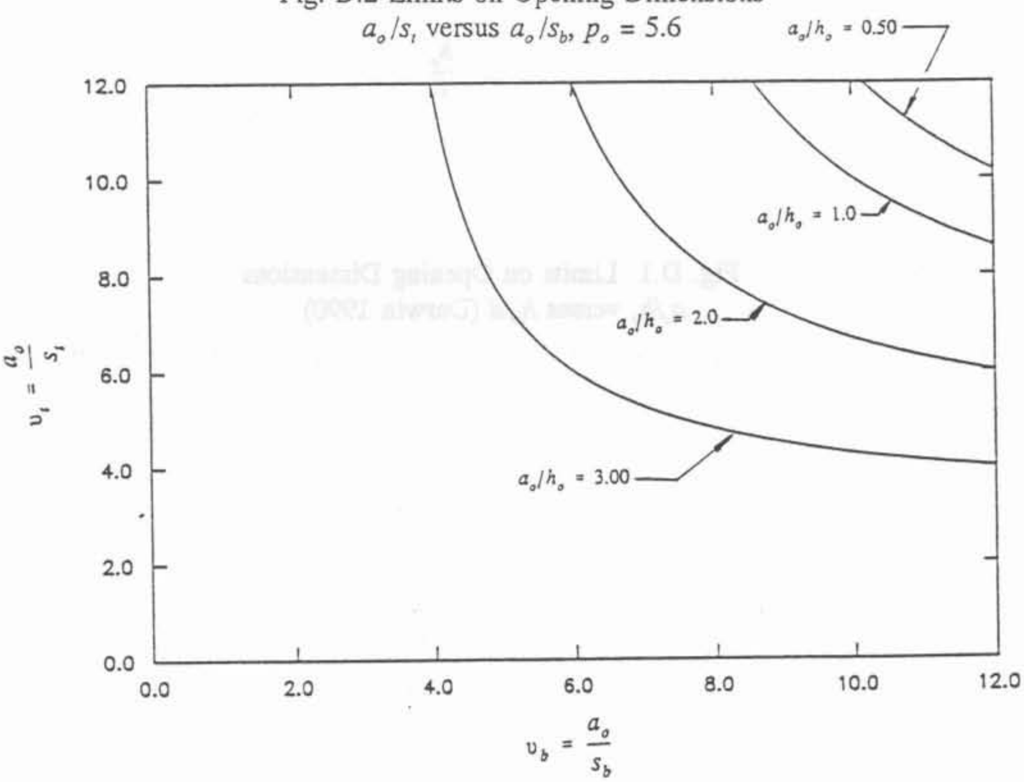

Fig. D.3 Limits on Opening Dimensions
a_o/s_t versus a_o/s_b, $p_o = 6.0$

APPENDIX E

SUMMARY OF BEAMS NOT MEETING DESIGN LIMITATIONS

A total of thirty-eight steel and composite beams available from previous research were excluded from consideration in determining resistance factors because of one or more violations of design limitations presented in Appendix D. Tables containing material and section properties, design limitation summaries, and capacity summaries and figures showing shear and moment interaction plots for the excluded beams follow (Tables E.1 - E.6 and Figs. E.0 - E.38).

Most of the excluded beams violated limitations pertaining to local buckling of the compression flange and/or the web. These violations contributed most significantly to premature failure of the beams, as illustrated by the results for beams RBD-UG2, RL-3, and RL-4. With the exception of RL-3, the predicted capacities for beams resisting high moment at the opening agreed reasonably well with test data. The predicted capacities for beams resisting high shear at the opening generally did not agree very well with test data.

Five beams, RM-1D, RM-2D, RM-4D, RM-21G, and RM-4G had closely spaced openings which, in three cases (RM-2D, RM-21G, and RM-4G), failed as a unit (Redwood and McCutcheon 1968). However, the predicted capacities of all five beams were conservative. Beams RL-1, RL-2, RL-3, and RL-4 were reinforced on one side of the web and violated associated design limitations. Beam RL-3 exhibited very premature failure with a test/theory ratio of 0.455. Reasonable strength predictions were obtained for the other four beams.

Sixteen beams tested by Kim (1980), (KKS-series), were excluded from the analysis although they met all of the design limitations. Without exception, the beams subjected to any amount of shear were unusually strong when compared to predicted capacities.

The predicted capacity of KKS-2HRC was the most conservative with a test/theory ratio of 2.022. These conservative results may well be due to strain hardening which is not accounted for by the prediction methods.

Table E.1 Material and Section Properties for Excluded Steel Beams

(in inches unless noted)

STEEL SECTION

	Web			Opening			Reinforcement				Top Tee				Bottom Tee			
Test	d	t_w	F_{yw} (ksi)	D_o	h_o	a_o	b_r	t_r	y_r	F_{yr} (ksi)	s	b_f	t_f	F_{yf} (ksi)	s	b_f	t_f	F_{yf} (ksi)
RBD-HB1A	20.750	0.257	45.700	7.000	6.300	3.150					6.880	7.220	0.381	43.300	6.870	7.220	0.381	43.300
RBD-UG2	20.770	0.251	58.000	11.000	9.900	4.950					4.890	7.340	0.385	54.100	4.880	7.340	0.385	54.100
RBD-UG2A	20.770	0.251	58.000	13.000	11.700	5.850					3.890	7.340	0.385	54.100	3.880	7.340	0.385	54.100
RBD-UG3	20.710	0.257	59.400	11.000	9.900	4.950					4.860	7.250	0.388	52.800	4.850	7.250	0.388	52.800
RM-1D	8.016	0.235	49.600	4.500	4.050	2.025					1.983	5.250	0.297	46.200	1.983	5.250	0.297	46.200
RM-2D	8.125	0.248	53.800	4.500	4.050	2.025					2.038	5.250	0.321	45.400	2.038	5.250	0.321	45.400
RM-4D	8.125	0.248	53.800	4.500	4.050	2.025					2.038	5.250	0.326	48.000	2.038	5.250	0.326	48.000
RL-1	20.560	0.256	60.870		9.000	22.560	2.510	0.242	0	47.070	5.780	7.060	0.392	55.480	5.780	7.060	0.392	55.480
RL-2	20.560	0.256	58.670		13.500	33.750	2.534	0.368	0	43.310	3.530	6.880	0.392	54.170	5.780	7.060	0.392	55.480
RL-3	20.630	0.252	58.260		9.000	22.500	2.533	0.368	0	43.780	5.815	7.060	0.392	53.230	5.815	7.060	0.392	53.230
RL-4	15.540	0.255	67.000		6.780	16.940	2.751	0.251	0	47.070	4.380	5.500	0.355	53.450	4.380	5.500	0.355	53.450
RBD-EH1	15.940	0.317	47.900		6.540	12.940					8.510	6.940	0.488	46.600	0.890	6.940	0.488	46.600
RBD-HB1	20.750	0.257	45.700		13.000	13.000					3.875	7.220	0.381	43.300	3.875	7.220	0.381	43.300
RBD-HB2	20.740	0.254	45.700		13.000	13.000					3.870	7.200	0.372	43.200	3.870	7.200	0.372	43.200
RBD-HB3	20.720	0.254	45.700		7.000	7.000					6.860	7.190	0.379	43.200	6.860	7.190	0.379	43.200
RBD-HB3A	20.720	0.254	45.700		7.000	14.000					6.860	7.190	0.379	43.200	6.860	7.190	0.379	43.200
RBD-HB4	20.750	0.258	45.700		7.000	7.000					6.875	7.230	0.378	43.200	6.875	7.230	0.378	43.200
RBD-HB5	20.750	0.255	57.600		11.000	22.000					4.875	7.220	0.374	55.800	4.875	7.220	0.374	55.800
RBD-HB5A	20.750	0.255	57.600		11.000	11.000					4.875	7.220	0.374	55.800	4.875	7.220	0.374	55.800
RM-21G	8.020	0.238	47.200		4.050	6.750					1.760	5.250	0.300	40.200	1.760	5.250	0.300	40.200
RM-4G	8.125	0.251	54.100		4.050	6.750					1.813	5.250	0.321	47.400	1.813	5.250	0.321	47.400
KKS-1HSC	7.090	0.157	40.000		3.150	3.150					1.970	3.540	0.236	40.000	1.970	3.540	0.236	40.000
KKS-1HRC	7.090	0.157	40.000		3.150	4.720					1.970	3.540	0.236	40.000	1.970	3.540	0.236	40.000
KKS-1HS10	7.090	0.157	40.000		3.150	3.150					1.260	3.540	0.236	40.000	2.680	3.540	0.236	40.000
KKS-1HR10	7.090	0.157	40.000		3.150	4.720					1.260	3.540	0.236	40.000	2.680	3.540	0.236	40.000
KKS-2HSC	7.090	0.157	40.000		3.150	3.150					1.970	3.540	0.236	40.000	1.970	3.540	0.236	40.000
KKS-2HRC	7.090	0.157	40.000		3.150	4.720					1.970	3.540	0.236	40.000	1.970	3.540	0.236	40.000
KKS-2HSE	7.090	0.157	40.000		3.150	3.150					1.260	3.540	0.236	40.000	2.680	3.540	0.236	40.000
KKS-2HRE	7.090	0.157	40.000		3.150	4.720					1.260	3.540	0.236	40.000	2.680	3.540	0.236	40.000
KKS-3HRC2	7.090	0.157	40.000		3.150	4.720					1.970	3.540	0.236	40.000	1.970	3.540	0.236	40.000
KKS-3HSC3	7.090	0.157	40.000		3.150	3.150					1.970	3.540	0.236	40.000	1.970	3.540	0.236	40.000
KKS-3HSC2	7.090	0.157	40.000		3.150	3.150					1.970	3.540	0.236	40.000	1.970	3.540	0.236	40.000
KKS-3HRC3	7.090	0.157	40.000		3.150	4.720					1.970	3.540	0.236	40.000	1.970	3.540	0.236	40.000
KKS-3HS10	7.090	0.157	40.000		3.150	3.150					1.260	3.540	0.236	40.000	2.680	3.540	0.236	40.000
KKS-3HR10	7.090	0.157	40.000		3.150	4.720					1.260	3.540	0.236	40.000	2.680	3.540	0.236	40.000
KKS-3HS5E	7.090	0.157	40.000		3.150	3.150					1.620	3.540	0.236	40.000	2.320	3.540	0.236	40.000
KKS-3HR5E	7.090	0.157	40.000		3.150	4.720					1.620	3.540	0.236	40.000	2.320	3.540	0.236	40.000

Notes:
refer to Table E.0 for key to beam designations

Table E.2 Design Limitation Summary for Excluded Steel Beams

	(a) Local Buckling of Compression Flange (D.1.1)			(b) Web Buckling (D.1.2)				
Test[1]	$b_f/2t_f <$	$65/\sqrt{F_y}$	$p_o < 6.0$	h/t	$420/\sqrt{F_y}$	$520/\sqrt{F_y}$	a_o/h_o	a_o/h_o(max)
RBD-HB1A	9.48	9.88	2.47	77.77	62.13	76.92	0.45	2.20
RBD-UG2	9.53	8.84	3.63	79.68	55.15	68.28	0.45	2.20
RBD-UG2A	9.53	8.84	4.21	79.68	55.15	68.28	0.45	2.20
RBD-UG3	9.34	8.95	3.64	77.56	54.49	67.47	0.45	2.20
RM-1D	8.84	9.56	3.82	31.58	59.64	73.84	0.45	3.00
RM-2D	8.18	9.65	3.77	30.17	57.26	70.89	0.45	3.00
RM-4D	8.05	9.38	3.77	30.13	57.26	70.89	0.45	3.00
RL-1	9.01	8.73	5.13	77.25	53.83	66.65	2.51	2.20
RL-2	8.78	8.83	6.44	77.25	54.83	67.89	2.50	2.20
RL-3	9.01	8.91	5.12	78.75	55.03	68.13	2.50	2.20
RL-4	7.75	8.89	5.12	58.16	51.31	63.53	2.50	2.20
RBD-EH1	7.11	9.52	4.44	47.21	60.69	75.13	1.98	3.00
RBD-HB1	9.48	9.88	4.76	77.77	62.13	76.92	1.00	2.20
RBD-HB2	9.68	9.89	4.76	78.72	62.13	76.92	1.00	2.20
RBD-HB3	9.49	9.89	3.03	78.59	62.13	76.92	1.00	2.20
RBD-HB3A	9.49	9.89	4.03	78.59	62.13	76.92	2.00	2.20
RBD-HB4	9.56	9.89	3.02	77.50	62.13	76.92	1.00	2.20
RBD-HB5	9.65	8.70	5.18	78.44	55.34	68.52	2.00	2.20
RBD-HB5A	9.65	8.70	4.18	78.44	55.34	68.52	1.00	2.20
RM-21G	8.75	10.25	4.70	31.18	61.13	75.69	1.67	3.00
RM-4G	8.18	9.44	4.66	29.81	57.10	70.70	1.67	3.00
KKS-1HSC	7.50	10.28	3.67	42.15	66.41	82.22	1.00	3.00
KKS-1HRC	7.50	10.28	4.16	42.15	66.41	82.22	1.50	3.00
KKS-1HS10	7.50	10.28	3.67	42.15	66.41	82.22	1.00	3.00
KKS-1HR10	7.50	10.28	4.16	42.15	66.41	82.22	1.50	3.00
KKS-2HSC	7.50	10.28	3.67	42.15	66.41	82.22	1.00	3.00
KKS-2HRC	7.50	10.28	4.16	42.15	66.41	82.22	1.50	3.00
KKS-2HSE	7.50	10.28	3.67	42.15	66.41	82.22	1.00	3.00
KKS-2HRE	7.50	10.28	4.16	42.15	66.41	82.22	1.50	3.00
KKS-3HRC25	7.50	10.28	4.16	42.15	66.41	82.22	1.50	3.00
KKS-3HSC35	7.50	10.28	3.67	42.15	66.41	82.22	1.00	3.00
KKS-3HSC25	7.50	10.28	3.67	42.15	66.41	82.22	1.00	3.00
KKS-3HRC35	7.50	10.28	4.16	42.15	66.41	82.22	1.50	3.00
KKS-3HS10E25	7.50	10.28	3.67	42.15	66.41	82.22	1.00	3.00
KKS-3HR10E25	7.50	10.28	4.16	42.15	66.41	82.22	1.50	3.00
KKS-3HS5E25	7.50	10.28	3.67	42.15	66.41	82.22	1.00	3.00
KKS-3HR5E25	7.50	10.28	4.16	42.15	66.41	82.22	1.50	3.00
D-8B	7.11	9.42	6.70	41.53	58.93	72.96	2.92	3.00

Table E.2 Design Limitation Summary for Excluded Steel Beams

(c) Buckling of Tee Shaped Compression Zone (D.1.3)

Test[1]	P_{crx} (k)	P_{cry} (k)	P_a (k)	M_n/M_m	a_o/s_t	Test/ Theory[2]
RBD-HB1A	114.63	114.57	169.52	0.921	0.46	0.957
RBD-UG2	127.71	127.91	88.80	0.454	1.01	1.063
RBD-UG2A	119.17	119.72	72.05	0.330	1.50	1.293
RBD-UG3	124.83	125.01	194.00	0.928	1.02	1.032
RM-1D	53.88	54.10	87.35	1.000	1.02	1.033
RM-2D	57.17	57.39	67.54	0.523	0.99	1.622
RM-4D	61.15	61.39	90.36	0.947	0.99	1.014
RL-1	135.82	135.59	279.08	1.000	3.90	1.009
RL-2	109.33	111.05	258.92	1.000	9.56	1.078
RL-3	130.19	129.96	127.55	0.999	3.87	0.455
RL-4	94.20	93.95	167.21	0.513	3.87	0.954
RBD-EH1	164.64	163.74	139.54	0.715	1.52	1.077
RBD-HB1	92.96	93.86	177.74	0.992	3.35	1.128
RBD-HB2	90.67	91.54	45.06	0.213	3.36	1.589
RBD-HB3	112.78	112.66	105.53	0.459	1.02	1.202
RBD-HB3A	112.23	111.96	80.97	0.440	2.04	1.114
RBD-HB4	113.77	113.64	211.93	0.999	1.02	1.098
RBD-HB5	125.11	126.12	42.56	0.205	4.51	1.126
RBD-HB5A	127.13	127.57	73.72	0.318	2.26	1.252
RM-21G	45.21	46.08	37.79	0.400	3.84	1.297
RM-4G	57.03	58.17	76.66	0.708	3.72	1.137
KKS-1HSC	26.36	26.46	84.51	1.000	1.60	1.042
KKS-1HRC	26.24	26.40	84.51	1.000	2.40	1.033
KKS-1HS10	23.52	23.81	84.92	1.000	2.50	1.087
KKS-1HR10	23.30	23.75	84.92	1.000	3.75	1.081
KKS-2HSC	26.36	26.46	84.51	0.000	1.60	1.942
KKS-2HRC	26.24	26.40	84.51	0.000	2.40	2.022
KKS-2HSE	23.52	23.81	84.92	0.000	2.50	1.909
KKS-2HRE	23.30	23.75	84.92	0.000	3.75	1.932
KKS-3HRC25	26.24	26.40	84.51	0.293	2.40	1.794
KKS-3HSC35	26.36	26.46	84.51	0.492	1.60	1.667
KKS-3HSC25	26.36	26.46	84.51	0.360	1.60	2.013
KKS-3HRC35	26.24	26.40	84.51	0.405	2.40	1.824
KKS-3HS10E25	23.52	23.81	84.92	0.389	2.50	1.752
KKS-3HR10E25	23.30	23.75	84.92	0.321	3.75	1.852
KKS-3HS5E25	24.99	25.15	84.58	0.372	1.94	1.988
KKS-3HR5E25	24.83	25.10	84.58	0.303	2.91	1.837

Table E.2 Design Limitation Summary for Excluded Steel Beams

(d) Hole Restrictions (D.3.1)

Test[1]	h_o < (in.)	$0.7d$ (in.)	s_t & (in.)	s_b > (in.)	$0.15d$ (in.)	a_o/s_t	a_o/s_b < 12.0
RBD-HB1A	7.00	14.52	6.88	6.87	3.11	0.46	0.46
RBD-UG2	11.00	14.54	4.89	4.88	3.12	1.01	1.01
RBD-UG2A	13.00	14.54	3.89	3.88	3.12	1.50	1.51
RBD-UG3	11.00	14.50	4.86	4.85	3.11	1.02	1.02
RM-1D	4.50	5.61	1.98	1.98	1.20	1.02	1.02
RM-2D	4.50	5.69	2.04	2.04	1.22	0.99	0.99
RM-4D	4.50	5.69	2.04	2.04	1.22	0.99	0.99
RL-1	9.00	14.39	5.78	5.78	3.08	3.90	3.90
RL-2	13.50	14.39	3.53	3.53	3.08	9.56	9.56
RL-3	9.00	14.44	5.82	5.82	3.09	3.87	3.87
RL-4	6.78	10.88	4.38	4.38	2.33	3.87	3.87
RBD-EH1	6.54	11.16	8.51	0.89	2.39	1.52	14.54
RBD-HB1	13.00	14.52	3.88	3.88	3.11	3.35	3.35
RBD-HB2	13.00	14.52	3.87	3.87	3.11	3.36	3.36
RBD-HB3	7.00	14.50	6.86	6.86	3.11	1.02	1.02
RBD-HB3A	7.00	14.50	6.86	6.86	3.11	2.04	2.04
RBD-HB4	7.00	14.52	6.88	6.88	3.11	1.02	1.02
RBD-HB5	11.00	14.52	4.88	4.88	3.11	4.51	4.51
RBD-HB5A	11.00	14.52	4.88	4.88	3.11	2.26	2.26
RM-21G	4.05	5.61	1.76	1.76	1.20	3.84	3.84
RM-4G	4.05	5.69	1.81	1.81	1.22	3.72	3.72
KKS-1HSC	3.15	4.96	1.97	1.97	1.06	1.60	1.60
KKS-1HRC	3.15	4.96	1.97	1.97	1.06	2.40	2.40
KKS-1HS10	3.15	4.96	1.26	2.68	1.06	2.50	1.18
KKS-1HR10	3.15	4.96	1.26	2.68	1.06	3.75	1.76
KKS-2HSC	3.15	4.96	1.97	1.97	1.06	1.60	1.60
KKS-2HRC	3.15	4.96	1.97	1.97	1.06	2.40	2.40
KKS-2HSE	3.15	4.96	1.26	2.68	1.06	2.50	1.18
KKS-2HRE	3.15	4.96	1.26	2.68	1.06	3.75	1.76
KKS-3HRC25	3.15	4.96	1.97	1.97	1.06	2.40	2.40
KKS-3HSC35	3.15	4.96	1.97	1.97	1.06	1.60	1.60
KKS-3HSC25	3.15	4.96	1.97	1.97	1.06	1.60	1.60
KKS-3HRC35	3.15	4.96	1.97	1.97	1.06	2.40	2.40
KKS-3HS10E25	3.15	4.96	1.26	2.68	1.06	2.50	1.18
KKS-3HR10E25	3.15	4.96	1.26	2.68	1.06	3.75	1.76
KKS-3HS5E25	3.15	4.96	1.62	2.32	1.06	1.94	1.36
KKS-3HR5E25	3.15	4.96	1.62	2.32	1.06	2.91	2.03
D-8B	6.38	7.09	2.03	1.73	1.52	9.20	10.80

Table E.2 Design Limitation Summary for Excluded Steel Beams

(e) One-sided Reinforcement (D.3.5)

Test[1]	$A_r < A_f/3$ (in.^{2})		$a_o/h_o \le 2.5$	s_t/t_w	$s_b/t_w \le$	$140/\sqrt{F_y}$	$M_u/(V_u{*}d) \le 20$
RL-1	0.20	0.92	2.51	22.58	22.58	17.94	223164.88
RL-2	0.31	0.90	2.50	13.79	13.79	18.28	221842.90
RL-3	0.30	0.92	2.50	23.08	23.08	18.34	19.06
RL-4	0.32	0.65	2.50	17.18	17.18	17.10	25.30

Table E.2 Design Limitation Summary for Excluded Steel Beams

Test[1]	(f) Violations
RBD-HB1A	(D.1.1), (D.1.3)
RBD-UG2	(D.1.1)
RBD-UG2A	(D.1.1)
RBD-UG3	(D.1.1), (D.1.3)
RM-1D	(D.1.3)
RM-2D	(D.1.3)
RM-4D	(D.1.3)
RL-1	(D.1.1), (D.1.2), (D.1.3), (D.3.5)
RL-2	(D.1.2), (D.1.3), (D.3.5)
RL-3	(D.1.1), (D.1.2), (D.3.5)
RL-4	(D.1.2), (D.1.3), (D.3.5)
RBD-EH1	(D.1.3), (D.3.1)
RBD-HB1	(D.1.3)
RBD-HB2	
RBD-HB3	
RBD-HB3A	
RBD-HB4	(D.1.3)
RBD-HB5	(D.1.1)
RBD-HB5A	(D.1.1)
RM-21G	
RM-4G	(D.1.3)
KKS-1HSC	(D.1.3)
KKS-1HRC	(D.1.3)
KKS-1HS10	(D.1.3)
KKS-1HR10	(D.1.3)
KKS-2HSC	(D.1.3)
KKS-2HRC	(D.1.3)
KKS-2HSE	(D.1.3)
KKS-2HRE	(D.1.3)
KKS-3HRC25	(D.1.3)
KKS-3HSC35	(D.1.3)
KKS-3HSC25	(D.1.3)
KKS-3HRC35	(D.1.3)
KKS-3HS10E25	(D.1.3)
KKS-3HR10E25	(D.1.3)
KKS-3HS5E25	(D.1.3)
KKS-3HR5E25	(D.1.3)
D-8B	(D.1.1)

Notes:

(1) refer to Table E.0 for key to beam designations

(2) The test/theory ratios for Method III with $\lambda = 1.414$ are provided as some indication of the effect of a potential violation of the design parameter on the predicted capacity. If the tee-shaped compression zone were to buckle prematurely, unconservative predictions would result.

Table E.3 Excluded Beam Capacity Summary: Method I, λ = 1.414

Test[1]	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
RBD-HB1A	3431.79	63.32	3025.09	36.45	3161.91	38.10	0.957
RBD-UG2	4052.31	65.00	1691.25	65.10	1649.78	63.50	1.025
RBD-UG2A	3842.82	44.83	1405.65	54.10	1154.19	44.42	1.218
RBD-UG3	3994.94	67.28	3680.39	44.30	3601.26	43.35	1.022
RM-1D	659.40	23.19	681.13	0.00	659.40	0.00	1.033
RM-2D	721.51	27.27	531.94	37.80	366.22	26.02	1.453
RM-4D	763.06	27.68	711.77	14.79	720.49	14.97	0.988
RL-1	4546.63	56.15	4588.27	0.00	4546.63	0.00	1.009
RL-2	4229.85	23.86	4561.09	0.00	4229.85	0.00	1.078
RL-3	4437.05	62.38	2016.89	5.13	4428.32	11.26	0.455
RL-4	2542.72	52.19	1141.72	48.61	1183.22	50.38	0.965
RBD-EH1	2751.06	51.03	1900.56	47.63	1817.83	45.56	1.046
RBD-HB1	3107.81	22.34	3459.23	8.31	3070.09	7.38	1.127
RBD-HB2	3031.42	21.98	876.40	33.77	569.07	21.93	1.540
RBD-HB3	3413.62	62.49	1881.42	72.60	1565.52	60.41	1.202
RBD-HB3A	3413.62	50.82	1443.55	55.70	1292.88	49.89	1.117
RBD-HB4	3444.06	63.56	3778.78	9.08	3441.53	8.27	1.098
RBD-HB5	4098.45	27.72	807.73	31.10	718.60	27.67	1.124
RBD-HB5A	4098.45	44.09	1399.11	53.90	1136.36	43.78	1.231
RM-21G	590.97	9.46	263.72	10.97	223.34	9.29	1.181
RM-4G	749.67	12.12	540.77	11.23	513.19	10.66	1.054
KKS-1HSC	282.23	9.63	294.20	0.00	282.23	0.00	1.042
KKS-1HRC	282.23	7.66	291.60	0.00	282.23	0.00	1.033
KKS-1HS10E	268.18	9.98	291.60	0.00	268.18	0.00	1.087
KKS-1HR10E	268.18	8.04	289.90	0.00	268.18	0.00	1.081
KKS-2HSC	282.23	9.63	0.13	17.41	0.07	9.63	1.808
KKS-2HRC	282.23	7.66	0.13	14.63	0.07	7.66	1.909
KKS-2HSE	268.18	9.98	0.13	17.65	0.07	9.98	1.768
KKS-2HRE	268.18	8.04	0.13	14.60	0.07	8.04	1.816
KKS-3HRC25	282.23	7.66	127.01	12.91	74.91	7.61	1.696
KKS-3HSC35	282.23	9.63	200.64	14.55	128.50	9.32	1.561
KKS-3HSC25	282.23	9.63	175.80	17.87	93.59	9.51	1.878
KKS-3HRC35	282.23	7.66	179.40	13.01	103.87	7.53	1.727
KKS-3HS10E25	268.18	9.98	157.39	15.99	96.70	9.82	1.628
KKS-3HR10E25	268.18	8.04	136.78	13.90	78.43	7.97	1.744
KKS-3HS5E25	275.30	9.71	174.74	17.76	94.28	9.58	1.853
KKS-3HR5E25	275.30	7.75	131.36	13.35	75.74	7.70	1.734

Notes:

(1) refer to Table E.0 for key to beam designations

Table E.4 Excluded Beam Capacity Summary: Method II

Test[1]	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
RBD-HB1A	3431.79	63.32	3025.09	36.45	3161.91	38.10	0.957
RBD-UG2	4052.31	70.88	1691.25	65.10	1787.15	68.79	0.946
RBD-UG2A	3842.82	49.29	1405.65	54.10	1265.16	48.69	1.111
RBD-UG3	3994.94	73.75	3680.39	44.30	3682.09	44.32	1.000
RM-1D	659.40	22.99	681.13	0.00	659.40	0.00	1.033
RM-2D	721.51	27.24	531.94	37.80	365.83	26.00	1.454
RM-4D	763.06	27.24	711.77	14.79	718.61	14.93	0.990
RL-1	4546.63	62.67	4588.27	0.00	4546.63	0.00	1.009
RL-2	4229.85	24.84	4561.09	0.00	4229.85	0.00	1.078
RL-3	4437.05	68.80	2016.89	5.13	4430.54	11.27	0.455
RL-4	2542.72	58.31	1141.72	48.61	1304.79	55.55	0.875
RBD-EH1	2751.06	56.99	1900.56	47.63	1958.71	49.09	0.970
RBD-HB1	3107.81	24.11	3459.23	8.31	3077.64	7.39	1.124
RBD-HB2	3031.42	23.77	876.40	33.77	615.21	23.71	1.425
RBD-HB3	3413.62	62.49	1881.42	72.60	1565.52	60.41	1.202
RBD-HB3A	3413.62	59.50	1443.55	55.70	1497.31	57.77	0.964
RBD-HB4	3444.06	63.56	3778.78	9.08	3441.53	8.27	1.098
RBD-HB5	4098.45	29.63	807.73	31.10	767.77	29.56	1.052
RBD-HB5A	4098.45	50.34	1399.11	53.90	1293.01	49.81	1.082
RM-21G	590.97	9.40	263.72	10.97	221.84	9.23	1.189
RM-4G	749.67	11.99	540.77	11.23	509.34	10.58	1.062
KKS-1HSC	282.23	10.50	294.20	0.00	282.23	0.00	1.042
KKS-1HRC	282.23	8.37	291.60	0.00	282.23	0.00	1.033
KKS-1HS10E	268.18	10.64	291.60	0.00	268.18	0.00	1.087
KKS-1HR10E	268.18	8.73	289.90	0.00	268.18	0.00	1.081
KKS-2HSC	282.23	10.50	0.13	17.41	0.08	10.50	1.659
KKS-2HRC	282.23	8.37	0.13	14.63	0.07	8.37	1.748
KKS-2HSE	268.18	10.64	0.13	17.65	0.08	10.64	1.658
KKS-2HRE	268.18	8.73	0.13	14.60	0.08	8.73	1.672
KKS-3HRC25	282.23	8.37	127.01	12.91	81.67	8.30	1.555
KKS-3HSC35	282.23	10.50	200.64	14.55	138.77	10.06	1.446
KKS-3HSC25	282.23	10.50	175.80	17.87	101.63	10.33	1.730
KKS-3HRC35	282.23	8.37	179.40	13.01	112.89	8.19	1.589
KKS-3HS10E25	268.18	10.64	157.39	15.99	102.75	10.44	1.532
KKS-3HR10E25	268.18	8.73	136.78	13.90	84.99	8.64	1.609
KKS-3HS5E25	275.30	10.53	174.74	17.76	101.80	10.35	1.717
KKS-3HR5E25	275.30	8.45	131.36	13.35	82.44	8.38	1.593

Notes:

(1) refer top Table E.0 for key to beam designations

Table E.5 Excluded Beam Capacity Summary: Method III, λ = 1.414

Test[1]	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
RBD-HB1A	3431.79	63.32	3025.09	36.45	3161.91	38.10	0.957
RBD-UG2	4052.31	62.53	1691.25	65.10	1591.08	61.24	1.063
RBD-UG2A	3842.82	42.17	1405.65	54.10	1087.27	41.85	1.293
RBD-UG3	3994.94	65.02	3680.39	44.30	3567.30	42.94	1.032
RM-1D	659.40	20.27	681.13	0.00	659.40	0.00	1.033
RM-2D	721.51	24.08	531.94	37.80	327.94	23.30	1.622
RM-4D	763.06	24.08	711.77	14.79	701.80	14.58	1.014
RL-1	4546.63	56.51	4588.27	0.00	4546.63	0.00	1.009
RL-2	4229.85	23.80	4561.09	0.00	4229.85	0.00	1.078
RL-3	4437.05	62.54	2016.89	5.13	4428.39	11.26	0.455
RL-4	2542.72	52.85	1141.72	48.61	1196.68	50.95	0.954
RBD-EH1	2751.06	48.95	1900.56	47.63	1763.96	44.21	1.077
RBD-HB1	3107.81	21.60	3459.23	8.31	3066.16	7.37	1.128
RBD-HB2	3031.42	21.30	876.40	33.77	551.66	21.26	1.589
RBD-HB3	3413.62	62.49	1881.42	72.60	1565.52	60.41	1.202
RBD-HB3A	3413.62	50.95	1443.55	55.70	1295.90	50.00	1.114
RBD-HB4	3444.06	63.56	3778.78	9.08	3441.53	8.27	1.098
RBD-HB5	4098.45	27.68	807.73	31.10	717.59	27.63	1.126
RBD-HB5A	4098.45	43.34	1399.11	53.90	1117.30	43.04	1.252
RM-21G	590.97	8.57	263.72	10.97	203.26	8.45	1.297
RM-4G	749.67	10.89	540.77	11.23	475.53	9.88	1.137
KKS-1HSC	282.23	8.97	294.20	0.00	282.23	0.00	1.042
KKS-1HRC	282.23	7.23	291.60	0.00	282.23	0.00	1.033
KKS-1HS10E	268.18	9.24	291.60	0.00	268.18	0.00	1.087
KKS-1HR10E	268.18	7.56	289.90	0.00	268.18	0.00	1.081
KKS-2HSC	282.23	8.97	0.13	17.41	0.07	8.97	1.942
KKS-2HRC	282.23	7.23	0.13	14.63	0.06	7.23	2.022
KKS-2HSE	268.18	9.24	0.13	17.65	0.07	9.24	1.909
KKS-2HRE	268.18	7.56	0.13	14.60	0.07	7.56	1.932
KKS-3HRC25	282.23	7.23	127.01	12.91	70.80	7.20	1.794
KKS-3HSC35	282.23	8.97	200.64	14.55	120.35	8.73	1.667
KKS-3HSC25	282.23	8.97	175.80	17.87	87.32	8.88	2.013
KKS-3HRC35	282.23	7.23	179.40	13.01	98.34	7.13	1.824
KKS-3HS10E25	268.18	9.24	157.39	15.99	89.83	9.13	1.752
KKS-3HR10E25	268.18	7.56	136.78	13.90	73.86	7.51	1.852
KKS-3HS5E25	275.30	9.03	174.74	17.76	87.89	8.93	1.988
KKS-3HR5E25	275.30	7.31	131.36	13.35	71.53	7.27	1.837

Notes:

(1) refer to Table E.0 for key to beam designations

Table E.6 Excluded Beam Capacity Summary: Redwood and Shrivastava (1980)

						Curvlinear			Linear		
Test[1]	M_m (in.-k)	V_m (k)	M_v (in.-k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory	M_n (in.-k)	V_n (k)	Test/ Theory
RBD-HB1A	3431.79	63.32	2430.47	3025.09	36.45	3414.09	41.14	0.886	2882.52	34.73	1.049
RBD-UG2	4052.31	78.56	2309.55	1691.25	65.10	2040.88	78.56	0.829	2040.88	78.56	0.829
RBD-UG2A	3842.82	61.16	2184.18	1405.65	54.10	1589.06	61.16	0.885	1589.06	61.16	0.885
RBD-UG3	3994.94	82.14	2172.12	3680.39	44.30	3933.19	47.34	0.936	3152.78	37.95	1.167
RM-1D	659.40	22.99	444.85	681.13	0.00	659.40	0.00	1.033	659.39	0.00	1.033
RM-2D	721.51	27.23	468.45	531.94	37.80	383.15	27.23	1.388	383.15	27.23	1.388
RM-4D	763.06	27.23	510.34	711.77	14.79	758.49	15.76	0.938	639.68	13.29	1.113
RL-1	4546.63	26.31	1665.36	4588.27	0.00	4546.63	0.00	1.009	4546.52	0.00	1.009
RL-2	4229.85	15.19	1435.59	4561.09	0.00	4229.85	0.00	1.078	4229.68	0.00	1.078
RL-3	4437.05	37.99	1448.10	2016.89	5.13	4378.94	11.14	0.461	3697.19	9.40	0.546
RL-4	2542.72	30.50	916.39	1141.72	48.61	716.34	30.50	1.594	716.34	30.50	1.594
RBD-EH1	2751.06	56.99	1019.12	1900.56	47.63	2396.94	60.07	0.793	1561.65	39.14	1.217
RBD-HB1	3107.81	24.11	1525.91	3459.23	8.31	3088.51	7.42	1.120	2684.64	6.45	1.289
RBD-HB2	3031.42	23.77	1469.69	876.40	33.77	616.93	23.77	1.421	616.93	23.77	1.421
RBD-HB3	3413.62	62.49	1902.17	1881.42	72.60	1619.35	62.49	1.162	1619.35	62.49	1.162
RBD-HB3A	3413.62	59.50	1081.96	1443.55	55.70	2368.12	91.37	0.610	1358.85	52.43	1.062
RBD-HB4	3444.06	63.56	1906.43	3778.78	9.08	3441.47	8.27	1.098	3254.87	7.82	1.161
RBD-HB5	4098.45	29.63	1609.71	807.73	31.10	769.46	29.63	1.050	769.46	29.63	1.050
RBD-HB5A	4098.45	50.34	1815.78	1399.11	53.90	1306.83	50.34	1.071	1306.83	50.34	1.071
RM-21G	590.97	11.77	303.08	263.72	10.97	282.92	11.77	0.932	282.92	11.77	0.932
RM-4G	749.67	14.90	390.50	540.77	11.23	711.66	14.78	0.760	499.56	10.37	1.082
KKS-1HSC	282.23	10.50	162.80	294.20	0.00	282.23	0.00	1.042	282.23	0.00	1.042
KKS-1HRC	282.23	8.37	151.50	291.60	0.00	282.23	0.00	1.033	282.23	0.00	1.033
KKS-1HS10E	268.18	10.64	155.48	291.60	0.00	268.18	0.00	1.087	268.18	0.00	1.087
KKS-1HR10E	268.18	8.73	137.86	289.90	0.00	268.18	0.00	1.081	268.18	0.00	1.081
KKS-2HSC	282.23	10.50	162.80	0.13	17.41	0.08	10.50	1.659	0.08	10.50	1.659
KKS-2HRC	282.23	8.37	151.50	0.13	14.63	0.07	8.37	1.748	0.07	8.37	1.748
KKS-2HSE	268.18	10.64	155.48	0.13	17.65	0.08	10.64	1.658	0.08	10.64	1.658
KKS-2HRE	268.18	8.73	137.86	0.13	14.60	0.08	8.73	1.672	0.08	8.73	1.672
KKS-3HRC25	282.23	8.37	151.50	127.01	12.91	82.34	8.37	1.543	82.34	8.37	1.543
KKS-3HSC35	282.23	10.50	162.80	200.64	14.55	144.74	10.50	1.386	144.74	10.50	1.386
KKS-3HSC25	282.23	10.50	162.80	175.80	17.87	103.26	10.50	1.702	103.26	10.50	1.702
KKS-3HRC35	282.23	8.37	151.50	179.40	13.01	115.41	8.37	1.554	115.41	8.37	1.554
KKS-3HS10E25	268.18	10.64	155.48	157.39	15.99	104.75	10.64	1.503	104.75	10.64	1.503
KKS-3HR10E25	268.18	8.73	137.86	136.78	13.90	85.91	8.73	1.592	85.91	8.73	1.592
KKS-3HS5E25	275.30	10.53	158.68	174.74	17.76	103.58	10.53	1.687	103.58	10.53	1.687
KKS-3HR5E25	275.30	8.45	144.12	131.36	13.35	83.19	8.45	1.579	83.19	8.45	1.579

Notes:

(1) refer to Table E.0 for key to beam designations

———— Cubic Moment-Shear Interaction Using Method III

------------ Curvlinear Moment-Shear Interaction Using Redwood & Shrivasatava (1980), Redwood & Poumbouras (1984), and Redwood & Cho (1986)

⊙ Actual Moment-Shear Value for Beam

Fig. E.0 Legend for Moment-Shear Interaction Curves in Figs. E.1 - E.38

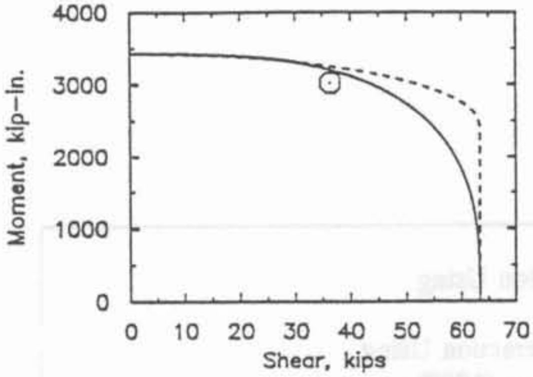

Fig. E. 1 Interaction Curves for Test RBD-HB1A

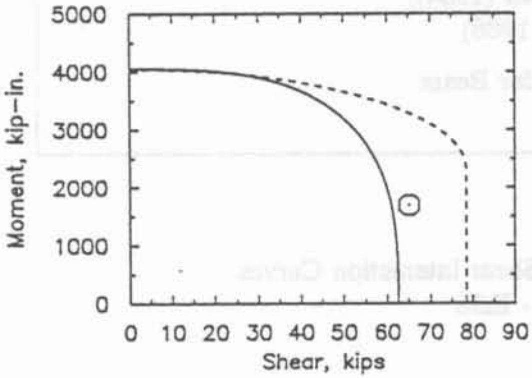

Fig. E. 2 Interaction Curves for Test RBD-UG2

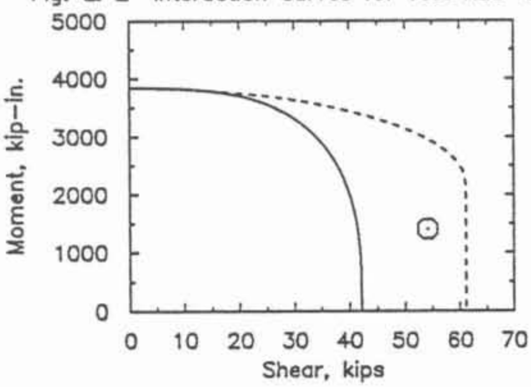

Fig. E. 3 Interaction Curves for Test RBD-UG2A

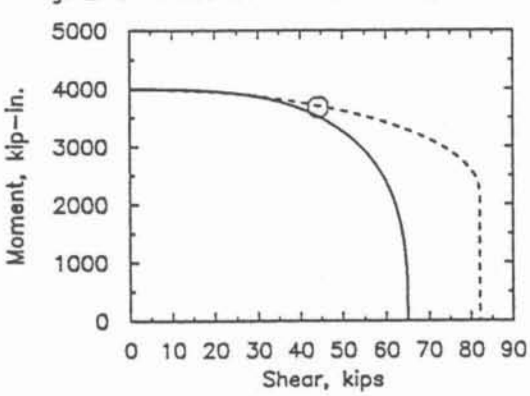

Fig. E. 4 Interaction Curves for Test RBD-UG3

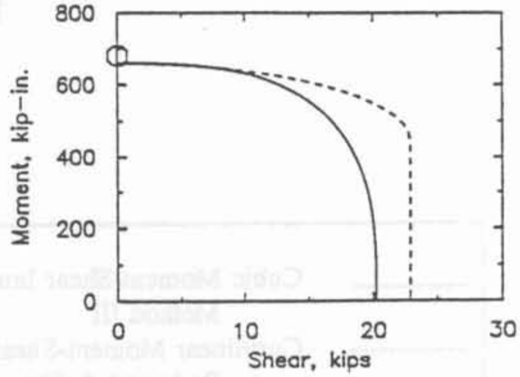

Fig. E. 5 Interaction Curves for Test RM-1D

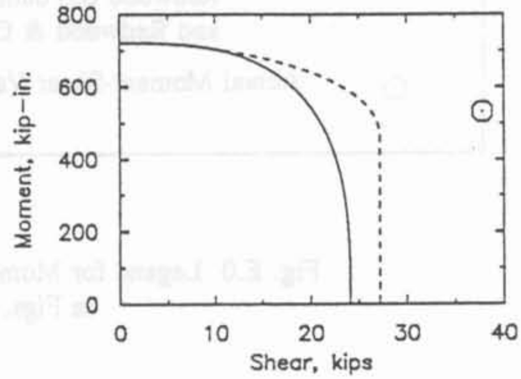

Fig. E. 6 Interaction Curves for Test RM-2D

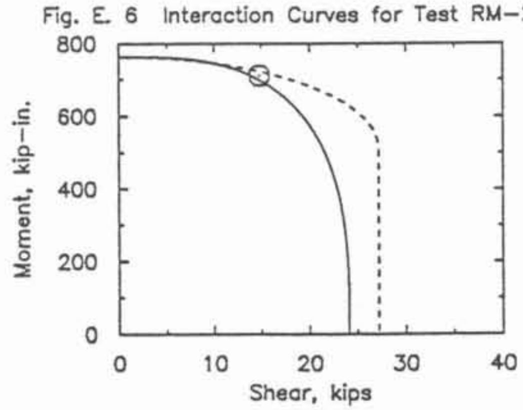

Fig. E. 7 Interaction Curves for Test RM-4D

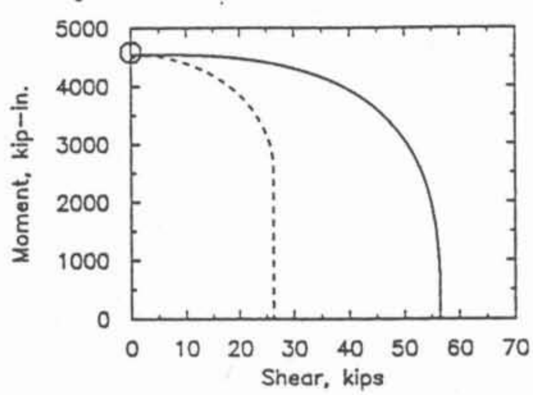

Fig. E. 8 Interaction Curves for Test RL-1

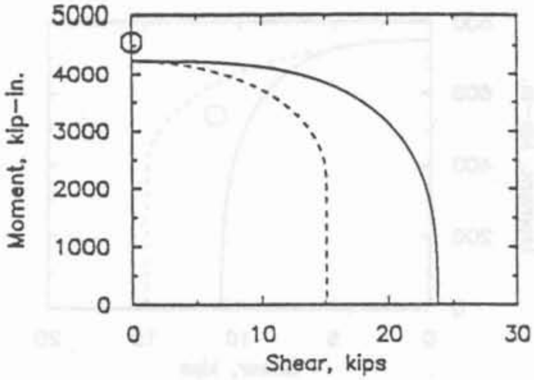

Fig. E. 9 Interaction Curves for Test RL-2

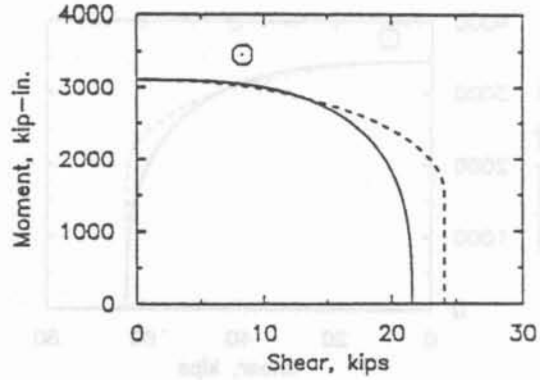

Fig. E.13 Interaction Curves for Test RBD-HB1

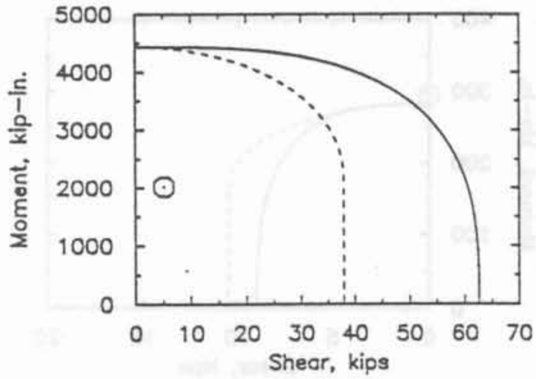

Fig. E.10 Interaction Curves for Test RL-3

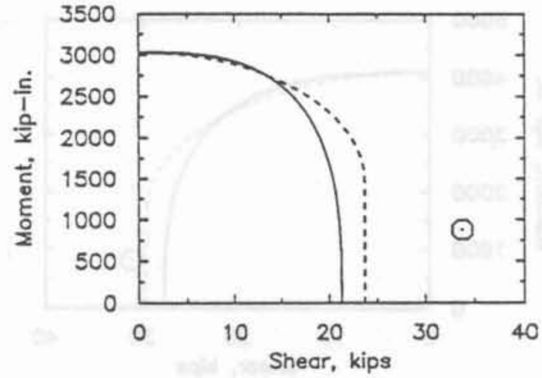

Fig. E.14 Interaction Curves for Test RBD-HB2

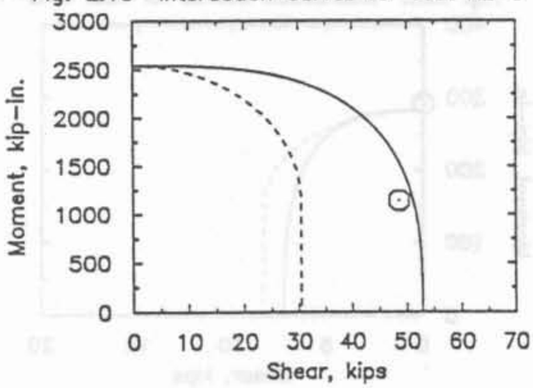

Fig. E.11 Interaction Curves for Test RL-4

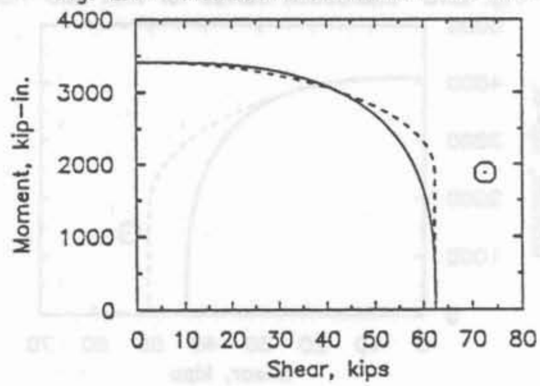

Fig. E.15 Interaction Curves for Test RBD-HB3

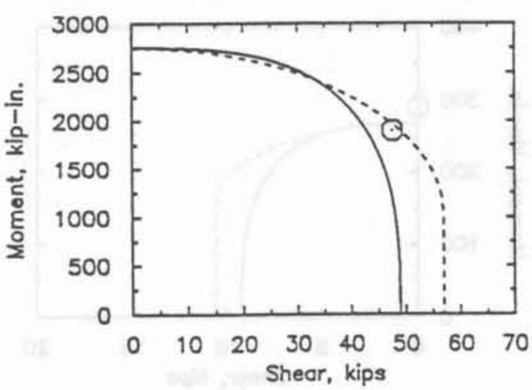

Fig. E.12 Interaction Curves for Test RBD-EH1

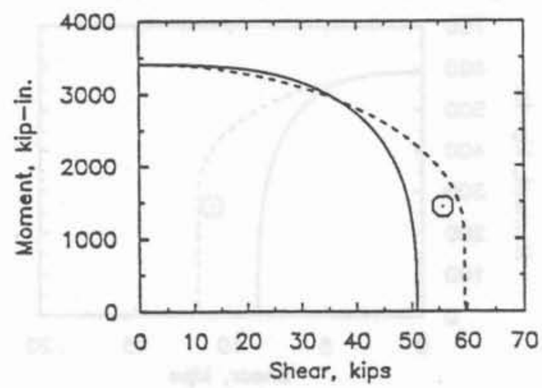

Fig. E.16 Interaction Curves for Test RBD-HB3A

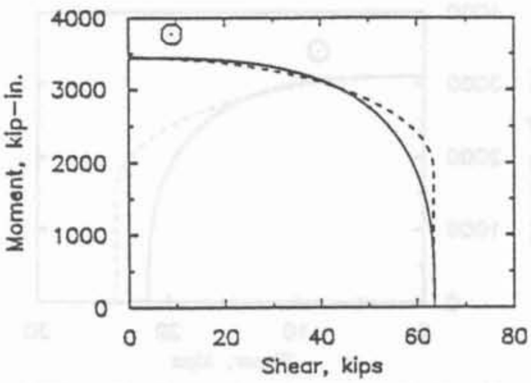

Fig. E.17 Interaction Curves for Test RBD-HB4

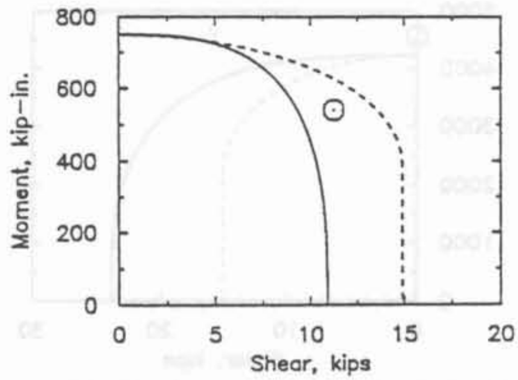

Fig. E.21 Interaction Curves for Test RM-4G

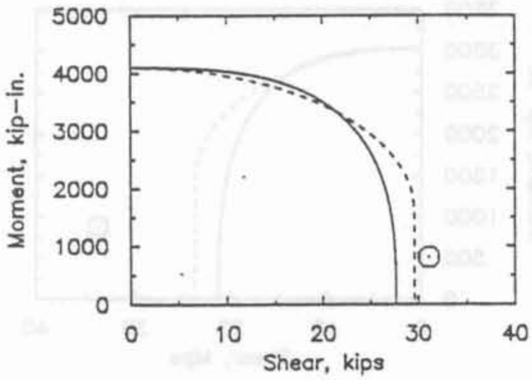

Fig. E.18 Interaction Curves for Test RBD-HB5

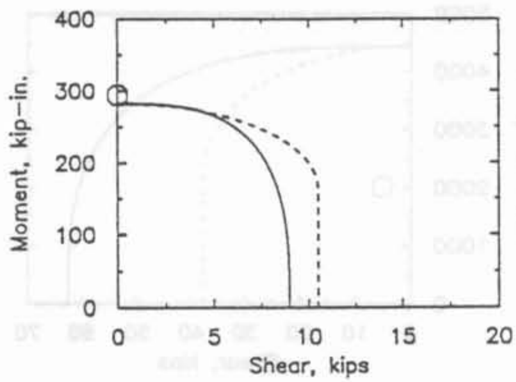

Fig. E.22 Interaction Curves for Test KKS-1HSC

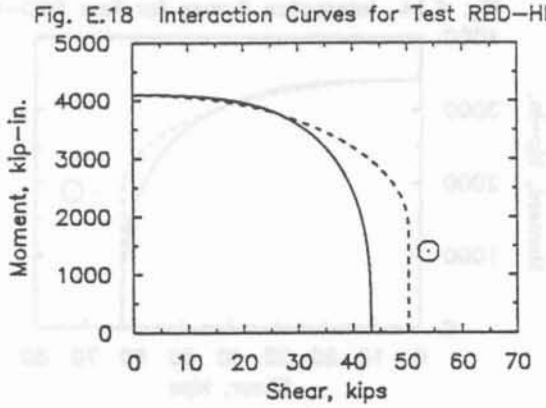

Fig. E.19 Interaction Curves for Test RBD-HB5A

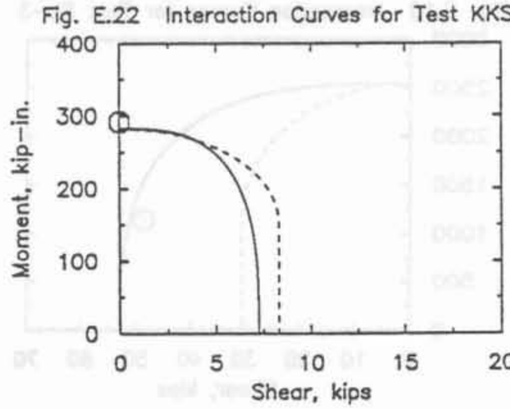

Fig. E.23 Interaction Curves for Test KKS-1HRC

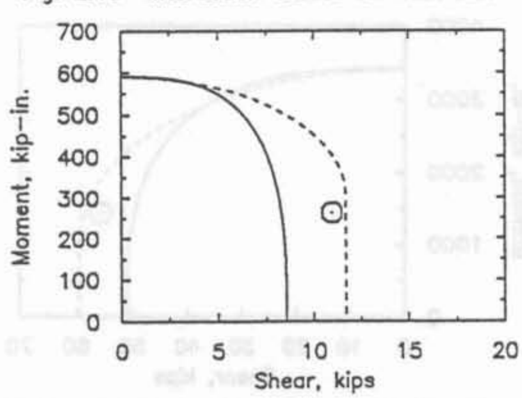

Fig. E.20 Interaction Curves for Test RM-21G

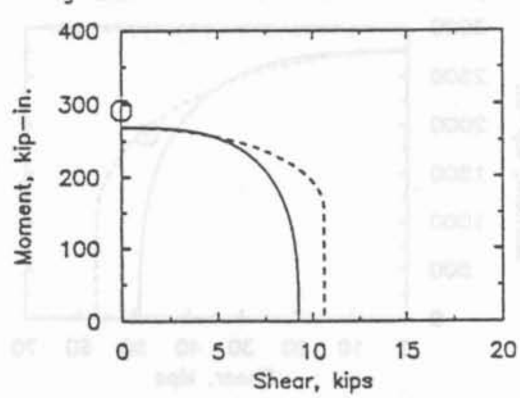

Fig. E.24 Interaction Curves for Test KKS-1HS10E

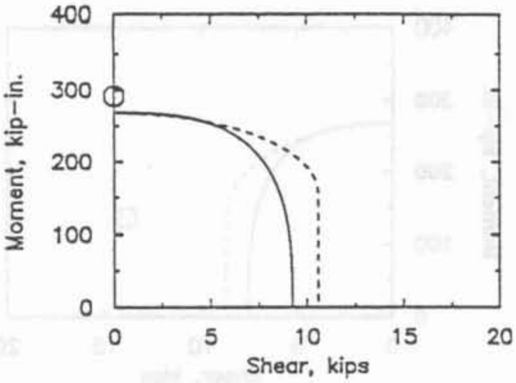

Fig. E.24 Interaction Curves for Test KKS-1HS10E

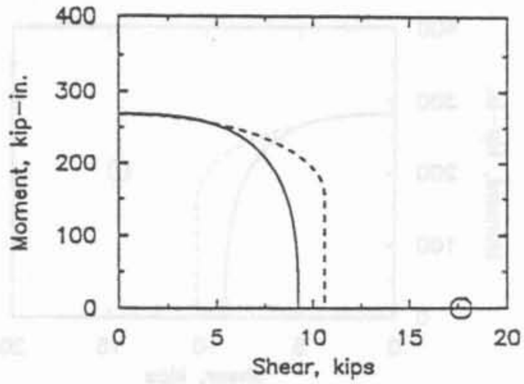

Fig. E.28 Interaction Curves for Test KKS-2HSE

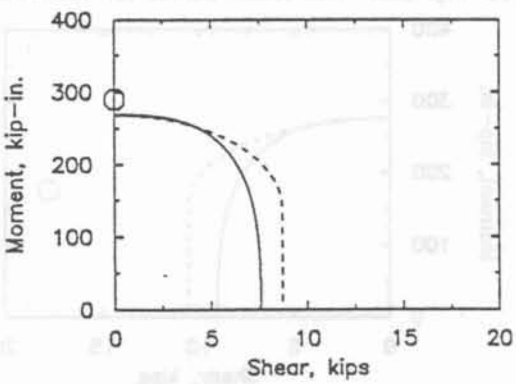

Fig. E.25 Interaction Curves for Test KKS-1HR10E

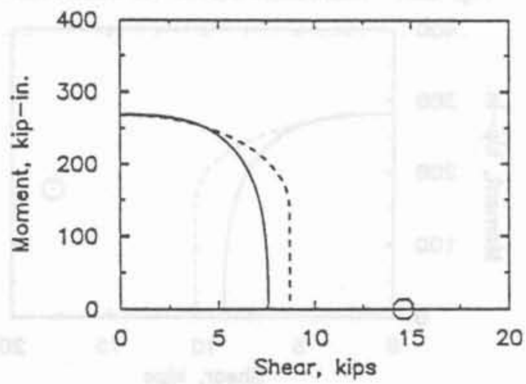

Fig. E.29 Interaction Curves for Test KKS-2HRE

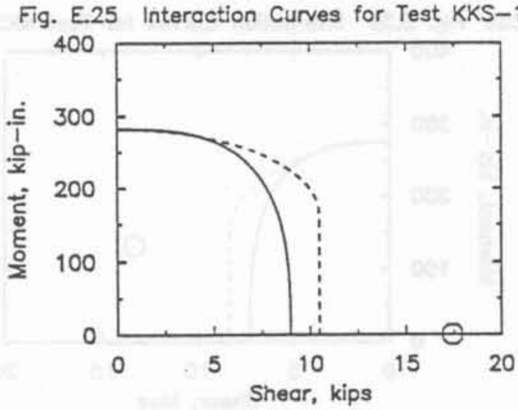

Fig. E.26 Interaction Curves for Test KKS-2HSC

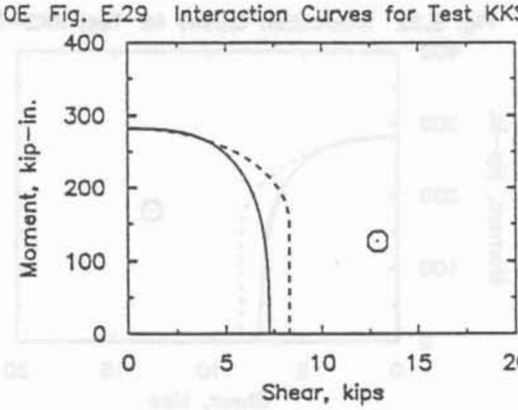

Fig. E.30 Interaction Curves for Test KKS-3HRC25

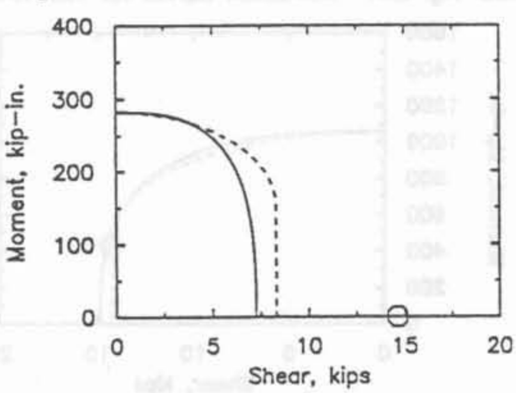

Fig. E.27 Interaction Curves for Test KKS-2HRC

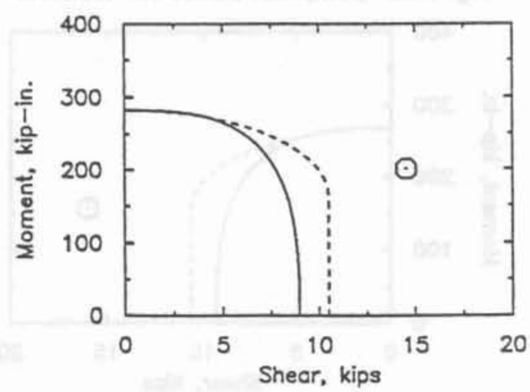

Fig. E.31 Interaction Curves for Test KKS-3HSC35

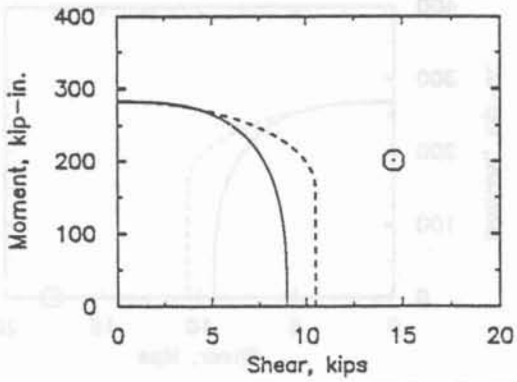

Fig. E.31 Interaction Curves for Test KKS-3HSC35

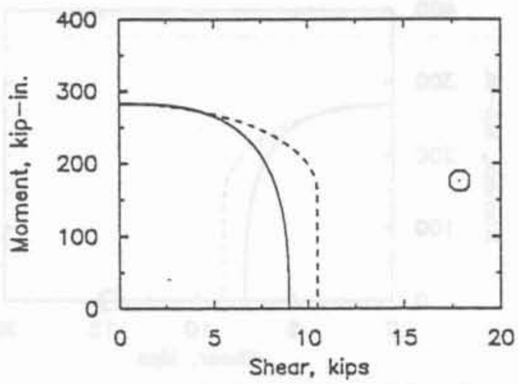

Fig. E.32 Interaction Curves for Test KKS-3HSC25

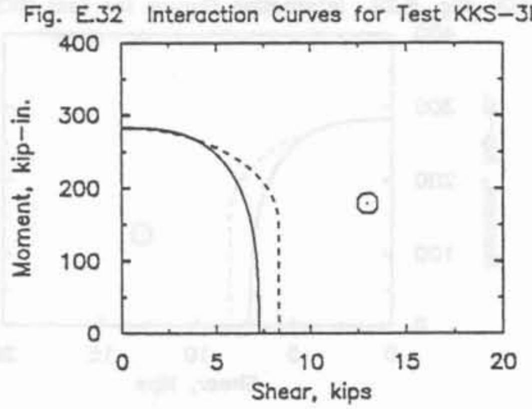

Fig. E.33 Interaction Curves for Test KKS-3HRC35

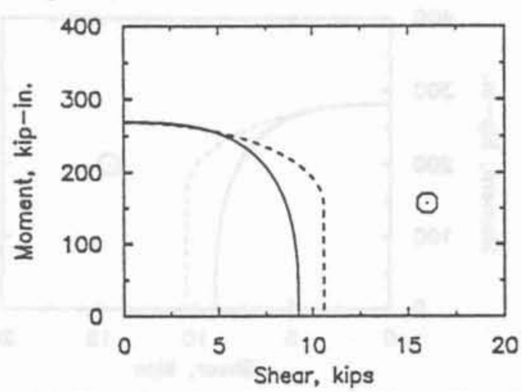

Fig. E.34 Interaction Curves for Test KKS-3HS10E25

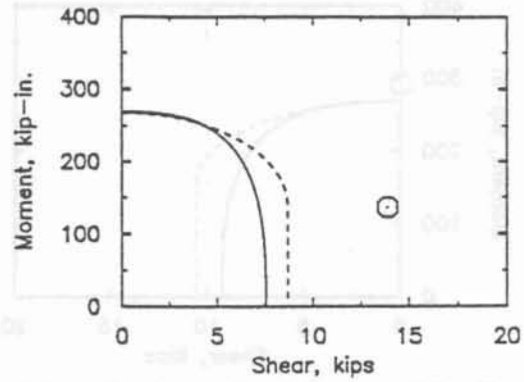

Fig. E.35 Interaction Curves for Test KKS-3HR10E25

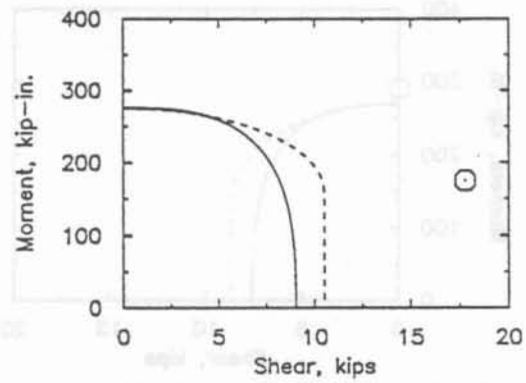

Fig. E.36 Interaction Curves for Test KKS-3HS5E25

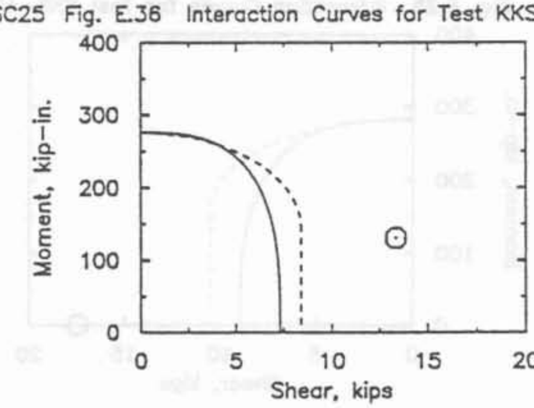

Fig. E.37 Interaction Curves for Test KKS-3HR5E25

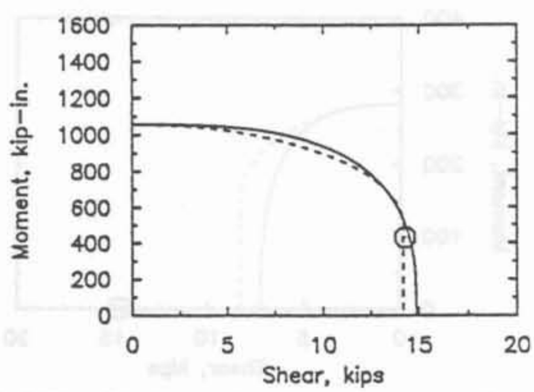

Fig. E.38 Interaction Curves for Test D-8B

APPENDIX F

DERIVATION OF $P_{c(min)}$ FOR COMPOSITE BEAM SIMPLIFIED MOMENT EQUATION

When the *PNA* resides in the steel section, a simplified expression for the maximum moment capacity of a composite beam, Eq. 2.74, can be used. As the *PNA* moves into the web, Eq. 2.74 becomes increasingly unconservative. In this appendix, the limit on P_c is derived for applying the approximation for M_m if the *PNA* is located in the web of a perforated composite beam.

The approximate equation is

$$M_m = F_y A_{sn}\frac{d}{2} + P_c\left(t_s - \frac{a}{2}\right) + F_y \Delta A_s e \tag{F.1}$$

The first term of equation F.1 is an approximation for the correct terms given in Eqs. 2.67 and 2.69. The first term of Eq. 2.69 can be rewritten as

$$F_y\left(A_{sn}\frac{d}{2} - (b_f - t_w)t_f^2 - t_w x^2\right) \tag{F.2}$$

The object of the derivation will be to determine what the lower bound for P_c is, such that the approximate term differs from the more precise term by a small percentage. This is expressed by

$$(b_f - t_w)t_f^2 + t_w x^2 \leq \alpha A_{sn}\frac{d}{2} \tag{F.3}$$

in which α is some small number.

The neutral axis location in a perforated composite beam, where the neutral axis is located in the web, is determined by

$$x = \frac{A_{sn} - 2A_f}{2t_w} - \frac{P_c}{2F_y t_w} + t_f \tag{F.4}$$

in which x is measured from the top of the flange of the steel section. Solving for x in terms of the inequality expressed by Eq. F.3 gives

$$x \leq \sqrt{\frac{\alpha A_{sn} d - 2A_f' t_f}{2t_w}} \tag{F.5}$$

in which $A_f' = (b_f - t_w)t_f$

Solving for P_c in Eq. F.4 gives

$$P_c = F_y(A_{sn} - 2A_f - 2t_w(x - t_f)) \tag{F.6}$$

Eq. F.6 can be more simply expressed as

$$P_c = F_y(t_w(d - h_o) - 2t_w x) \tag{F.7}$$

Substituting the expression for x in Eq. F.5 into Eq. F.7 results in

$$P_c = F_y\left(t_w(d - h_o) - \sqrt{2}\sqrt{\alpha A_{sn} d t_w - 2A_f' t_f t_w}\right) \tag{F.8}$$

By substituting $2A_f' + dt_w - h_o t_w$ for A_{sn} in Eq. F.8, the expression under the radical can be arranged to give

$$2A_f' t_w(\alpha d - t_f) + \alpha(d - h_o)t_w^2 d \tag{F.9}$$

Setting $A_f' = \beta A_w = \beta t_w d$, in which β is some fraction results in the following expression.

$$2\beta d t_w^2(\alpha d - t_f) + \alpha(d - h_o)t_w^2 d \tag{F.10}$$

Rearranging gives,

$$d t_w^2(\alpha((2\beta + 1)d - h_o) - 2\beta t_f) \tag{F.11}$$

h_o is typically between 0.3d and 0.7d, so if h_o is assumed to 0.5d, and if t_f is conservatively assumed to be 0.02d, Eq. F.11 can be rewritten as

$$d^2 t_w^2(\alpha(2\beta + 0.5) - 0.04\beta) \tag{F.12}$$

Substituting equation F.11 into equation F.8, and rearranging gives,

$$P_{c(\min)} = F_y\left[t_w(d - h_o) - t_w d\sqrt{2\alpha(2\beta + 0.5) - 0.08\beta}\right] \tag{F.13}$$

For $\alpha = 0.04$ (i.e. a 4% maximum error in the first term in Eq. F.1), the following table is obtained for different values of β:

β	$P_{c(min)}$
0.00	$F_y t_w(d - h_o)$
0.40	$F_y t_w(0.732d - h_o)$
0.50	$F_y t_w(0.717d - h_o)$
1.00	$F_y t_w(0.654d - h_o)$

As seen from the table, $P_{c(min)} = F_y t_w (d - h_o)$ is always safe, however, $P_{c(min)} = F_y t_w (0.75d - h_o)$ is safe and reasonable for building construction because β, the ratio of the flange area to the web area, is rarely below 0.40.

APPENDIX G

STEEL AND COMPOSITE BEAM RESULTS FOR METHODS I AND III

WITH $\lambda = 1.207$

This appendix contains nine tables summarizing shear capacities and analysis results for steel and composite beams obtained using Methods I and III with $\lambda = 1.207$. These results were used to calculate the resistance factors corresponding to $\lambda = 1.207$.

Table G.1 Steel Beam Shear Capacity Summary: Method I, $\lambda = 1.207$

(values in kips)

Test	V_{mbl}	V_{pb}	V_{bl}	V_{mtl}	V_{pt}	V_{tl}	V_m	V_t
RBD-C1	49.59	47.30	47.30	49.59	47.30	47.30	82.99	82.99
RM-1A	13.06	14.88	13.06	13.06	14.88	13.06	39.15	26.13
RM-1B	6.74	14.69	6.74	6.70	14.64	6.70	38.66	13.44
RM-2A	14.14	15.94	14.14	14.14	15.94	14.14	41.94	28.27
RM-2B	5.64	12.16	5.64	5.64	12.16	5.64	32.34	11.27
RM-2C	10.42	11.91	10.42	10.42	11.91	10.42	31.69	20.85
RM-3A	13.17	15.05	13.17	13.17	15.05	13.17	39.60	26.35
RM-4A	13.18	15.02	13.18	13.18	15.02	13.18	39.52	26.35
RM-4B	6.70	14.64	6.70	6.70	14.64	6.70	38.53	13.39
RM-4C	14.29	16.79	14.29	14.29	16.79	14.29	44.17	28.58
CR-1A	13.43	17.67	13.43	13.40	17.63	13.40	52.41	26.83
CR-2A	28.49	27.71	27.71	28.49	27.71	27.71	72.49	55.42
CR-2B	28.49	27.71	27.71	28.49	27.71	27.71	72.49	55.42
CR-2C	30.57	35.47	30.57	30.57	35.47	30.57	92.22	61.15
CR-2D	30.57	35.47	30.57	30.57	35.47	30.57	92.22	61.15
CR-3A	28.49	27.71	27.71	28.49	27.71	27.71	72.49	55.42
CR-3B	36.61	35.47	35.47	36.61	35.47	35.47	92.22	70.94
CR-4A	24.67	35.47	24.67	24.67	35.47	24.67	92.22	49.33
CR-4B	24.67	35.47	24.67	24.67	35.47	24.67	92.22	49.33
CR-5A	22.60	25.65	22.60	22.60	25.65	22.60	92.22	45.19
CR-7B	29.25	31.60	29.25	29.24	31.60	29.24	82.22	58.49
CR-7D	26.31	31.60	26.31	29.24	31.60	29.24	82.22	55.55
CSK-2	72.90	64.83	64.83	29.76	28.13	28.13	97.69	92.96
CSK-5	55.63	55.15	55.15	22.37	23.66	22.37	83.19	77.52
CSK-6	9.78	15.79	9.78	37.59	47.28	37.59	83.19	47.37
CSK-7	12.47	15.79	12.47	44.67	47.28	44.67	83.19	57.14
CS-1	23.17	21.98	21.98	23.17	21.98	21.98	57.73	43.95
CS-2	22.48	21.16	21.16	22.48	21.16	21.16	55.58	42.31
CS-3	22.11	21.92	21.92	22.11	21.92	21.92	57.58	43.84
RL-5	19.29	34.74	19.29	19.28	34.74	19.28	81.67	38.57
RL-6	11.28	21.60	11.28	11.28	21.60	11.28	82.80	22.56
B-1	18.66	33.90	18.66	18.66	33.90	18.66	83.92	37.33
B-2	17.02	30.28	17.02	17.02	30.28	17.02	72.65	34.04
B-3	16.09	28.88	16.09	16.09	28.88	16.09	70.72	32.18
B-4	21.18	34.12	21.18	21.18	34.12	21.18	82.35	42.37
CL-4B	8.13	31.51	8.13	7.73	30.89	7.73	121.49	15.86
CR-6A	16.68	35.47	16.68	16.68	35.47	16.68	92.22	33.36
CSK-1	44.50	63.03	44.50	14.45	27.34	14.45	94.98	58.95
DO-1	9.78	19.43	9.78	9.78	19.43	9.78	36.56	19.56
DO-2	3.57	11.19	3.57	3.56	11.18	3.56	36.56	7.13
DO-3	16.51	27.69	16.51	4.46	11.18	4.46	36.56	20.98
DO-4	8.51	20.77	8.51	3.69	11.80	3.69	39.29	12.20
DO-5	5.99	11.18	5.99	5.99	11.18	5.99	36.56	11.97
RBD-R1B	42.56	53.80	42.56	42.56	53.80	42.56	82.14	82.14
RBD-R2	24.36	43.24	24.36	24.36	43.24	24.36	82.99	48.72
RM-11H	7.01	17.32	7.01	7.01	17.32	7.01	51.24	14.01
RM-21H	4.73	11.29	4.73	4.73	11.29	4.73	34.08	9.45
RM-2F	4.77	11.17	4.77	4.77	11.17	4.77	33.34	9.54
RM-4F	6.01	14.16	6.01	6.01	14.16	6.01	41.87	12.02
RM-4H	4.96	11.86	4.96	4.96	11.86	4.96	35.61	9.92

Notes:

refer to Table 3.0 for key to beam designations

V_{mbl}, V_{mtl} = shear capacity of bottom and top tee, respectively, using Eq. B.1.

V_{pb}, V_{pt} = plastic shear capacity of bottom and top tee, respectively, using Eqs. 2.22, and 2.18.

V_{bl}, V_{pl} = governing shear capacity of top and bottom tees, respectively.

V_m = maximum permissible shear capacity of beam per Section D.1.2.

Table G.2 Composite Beam Shear Capacity Summary: Method I, $\lambda = 1.207$

(values in kips)

Test	$V_{t(a)}$	$V_{t(b)}$	V_{pt}	V_{tsh}	V_{tl}	V_b	V_{pb}	V_{bl}	VI
D-1	26.83	47.84	47.84	54.86	26.83	12.86	46.96	12.86	39.68
D-2	26.55	44.81	44.81	52.12	26.55	12.12	44.81	12.12	38.67
D-3	27.88	44.54	44.54	52.26	27.88	12.01	44.46	12.01	39.89
D-5A	24.01	45.40	45.40	52.63	24.01	11.60	44.77	11.60	35.61
D-5B	27.23	44.77	44.77	52.26	27.23	4.04	23.13	4.04	31.27
D-6A	24.69	44.75	44.75	51.41	24.69	12.07	44.70	12.07	36.76
D-6B	40.04	44.75	44.75	53.36	40.04	12.07	44.70	12.07	52.11
D-7A	32.97	34.47	34.47	42.96	32.97	9.46	35.54	9.46	42.44
D-7B	30.01	34.90	34.90	43.50	30.01	9.61	35.86	9.61	39.62
D-8A	20.05	17.39	14.20	23.26	17.39	3.99	14.16	3.99	21.38
D-9A	34.37	30.56	25.70	44.68	30.56	5.53	25.70	5.53	36.09
D-9B	43.88	40.04	26.99	46.40	40.04	7.84	24.68	7.84	47.89
R-0	20.23	17.02	15.06	24.52	17.02	4.14	15.06	4.14	21.16
R-1	17.63	21.44	21.44	30.07	17.63	6.33	21.44	6.33	23.96
R-2	20.75	23.88	23.88	32.01	20.75	6.99	23.88	6.99	27.75
R-3	33.88	32.52	24.05	34.07	32.52	6.98	24.05	6.98	39.50
R-4	16.96	24.64	24.64	34.26	16.96	7.19	24.64	7.19	24.15
R-5	15.82	16.14	10.76	19.39	16.14	11.90	32.12	11.90	28.03
R-6	11.97	23.56	23.56	31.37	11.97	6.92	23.56	6.92	18.89
R-7	24.36	23.80	21.97	29.78	23.80	6.51	21.97	6.51	30.31
R-8	23.45	23.05	20.73	28.35	23.05	6.25	20.73	6.25	29.30
C-1	33.37	29.51	19.16	33.21	29.51	6.18	19.16	6.18	35.70
C-2	29.19	30.28	30.28	41.17	29.19	9.51	32.85	9.51	38.70
C-3	30.78	31.42	31.42	43.21	30.78	9.13	31.81	9.13	39.90
C-4	36.24	35.89	35.89	47.11	35.89	10.40	36.61	10.40	46.28
C-5	35.92	35.71	35.71	47.21	35.71	10.26	36.32	10.26	45.98
C-6	36.21	31.75	24.30	34.95	31.75	6.95	23.86	6.95	38.69
G-1	48.19	54.53	12.85	21.42	21.42	6.67	12.85	6.67	28.09
G-2	38.05	44.97	12.85	21.44	21.44	6.67	12.85	6.67	28.11
CHO-3	49.90	55.78	10.38	27.25	27.25	4.43	10.38	4.43	31.68
CHO-4	41.40	42.13	22.53	39.41	39.41	8.59	22.92	8.59	48.01
CHO-5	44.01	43.56	22.92	39.78	39.78	8.15	22.15	8.15	47.94
CHO-6	59.20	72.61	10.66	27.53	27.53	9.53	10.38	9.53	37.06
CHO-7	44.09	48.37	22.53	39.14	39.14	16.42	22.53	16.42	55.56
WJE-1	41.63	40.88	23.85	23.85	23.85	13.58	23.85	13.58	37.42

Notes:

refer to Table 3.0 for key to beam designations

$V_{t(a)}$ = shear capacity of top tee using Eq. B.1.
$V_{t(b)}$ = shear capacity of top tee using Eq. 2.33.
V_{pt} = plastic shear capacity of top tee using Eq. 2.18
V_{tsh} = combined plastic shear capacity of top tee and concrete using Eq. 2.21.
V_{tl} = governing shear capacity of top tee.
V_b = shear capacity of bottom tee using Eq. B.1.
V_{pb} = plastic shear capacity of bottom tee using Eq. 2.22.
V_{bl} = governing shear capacity of bottom tee.
V_I = maximum shear capacity as predicted by Method I.Notes:

Table G.3 Steel Beam Shear Capacity Summary: Method III, $\lambda = 1.207$

(values in kips)

Test	V_{mb3}	V_{pb}	V_{b3}	V_{mt3}	V_{pt}	V_{t3}	V_m	V_3
RBD-C1	47.98	47.30	47.30	47.98	47.30	47.30	82.99	82.99
RM-1A	11.41	14.88	11.41	11.41	14.88	11.41	39.15	22.82
RM-1B	6.23	14.69	6.23	6.19	14.64	6.19	38.66	12.41
RM-2A	12.22	15.94	12.22	12.22	15.94	12.22	41.94	24.45
RM-2B	5.08	12.16	5.08	5.08	12.16	5.08	32.34	10.16
RM-2C	9.07	11.91	9.07	9.07	11.91	9.07	31.69	18.13
RM-3A	11.54	15.05	11.54	11.54	15.05	11.54	39.60	23.08
RM-4A	11.52	15.02	11.52	11.52	15.02	11.52	39.52	23.04
RM-4B	6.20	14.64	6.20	6.20	14.64	6.20	38.53	12.41
RM-4C	12.88	16.79	12.88	12.88	16.79	12.88	44.17	25.75
CR-1A	13.17	17.67	13.17	13.13	17.63	13.13	52.41	26.29
CR-2A	27.53	27.71	27.53	27.53	27.71	27.53	72.49	55.05
CR-2B	27.53	27.71	27.53	27.53	27.71	27.53	72.49	55.05
CR-2C	29.83	35.47	29.83	29.83	35.47	29.83	92.22	59.67
CR-2D	29.83	35.47	29.83	29.83	35.47	29.83	92.22	59.67
CR-3A	27.12	27.71	27.12	27.12	27.71	27.12	72.49	54.25
CR-3B	35.53	35.47	35.47	35.53	35.47	35.47	92.22	70.94
CR-4A	24.35	35.47	24.35	24.35	35.47	24.35	92.22	48.69
CR-4B	24.35	35.47	24.35	24.35	35.47	24.35	92.22	48.69
CR-5A	21.76	25.65	21.76	21.76	25.65	21.76	92.22	43.53
CR-7B	28.52	31.60	28.52	28.50	31.60	28.50	82.22	57.02
CR-7D	25.69	31.60	25.69	28.50	31.60	28.50	82.22	54.19
CSK-2	70.79	64.83	64.83	27.69	28.13	27.69	97.69	92.52
CSK-5	56.64	55.15	55.15	21.31	23.66	21.31	83.19	76.46
CSK-6	9.10	15.79	9.10	37.39	47.28	37.39	83.19	46.49
CSK-7	11.48	15.79	11.48	44.09	47.28	44.09	83.19	55.57
CS-1	21.16	21.98	21.16	21.16	21.98	21.16	57.73	42.32
CS-2	20.38	21.16	20.38	20.38	21.16	20.38	55.58	40.75
CS-3	20.27	21.92	20.27	20.27	21.92	20.27	57.58	40.53
RL-5	19.30	34.74	19.30	19.30	34.74	19.30	81.67	38.60
RL-6	11.14	21.60	11.14	11.14	21.60	11.14	82.80	22.28
B-1	18.41	33.90	18.41	18.41	33.90	18.41	83.92	36.82
B-2	16.65	30.28	16.65	16.65	30.28	16.65	72.65	33.31
B-3	15.74	28.88	15.74	15.74	28.88	15.74	70.72	31.49
B-4	20.84	34.12	20.84	20.84	34.12	20.84	82.35	41.69
CL-4B	7.49	31.51	7.49	7.22	30.89	7.22	121.49	14.71
CR-6A	16.03	35.47	16.03	16.03	35.47	16.03	92.22	32.06
CSK-1	43.84	63.03	43.84	12.25	27.34	12.25	94.98	56.08
DO-1	9.49	19.43	9.49	9.49	19.43	9.49	36.56	18.98
DO-2	3.06	11.19	3.06	3.06	11.18	3.06	36.56	6.12
DO-3	16.44	27.69	16.44	3.79	11.18	3.79	36.56	20.23
DO-4	8.36	20.77	8.36	3.15	11.80	3.15	39.29	11.51
DO-5	4.98	11.18	4.98	4.98	11.18	4.98	36.56	9.97
RBD-R1B	42.09	53.80	42.09	42.09	53.80	42.09	82.14	82.14
RBD-R2	24.19	43.24	24.19	24.19	43.24	24.19	82.99	48.37
RM-11H	6.64	17.32	6.64	6.64	17.32	6.64	51.24	13.28
RM-21H	4.22	11.29	4.22	4.22	11.29	4.22	34.08	8.45
RM-2F	4.23	11.17	4.23	4.23	11.17	4.23	33.34	8.46
RM-4F	5.43	14.16	5.43	5.43	14.16	5.43	41.87	10.85
RM-4H	4.46	11.86	4.46	4.46	11.86	4.46	35.61	8.92

Notes:

refer to Table 3.0 for key to beam designations

V_{mb3}, V_{mt3} = shear capacity of bottom and top tee, respectively, using Eq. 2.54.
V_{pb}, V_{pt} = plastic shear capacity of bottom and top tee, respectively, using Eqs. 2.22 and 2.18.
V_{b3}, V_{t3} = governing shear capacity of bottom and bottom tees, respectively.
V_m = maximum permissible shear capacity of beam per Section D.1.2.
V_3 = maximum shear capacity as predicted by Method III.

Table G.4 Composite Beam Shear Capacity Summary: Method III, $\lambda = 1.207$

(values in kips)

Test	$V_{t(a)}$	$V_{t(b)}$	V_{pt}	V_{tsh}	V_{t3}	V_b	V_{pb}	V_{b3}	V3
D-1	26.90	47.84	47.84	54.86	26.90	12.64	46.96	12.64	39.54
D-2	27.24	44.81	44.81	52.12	27.24	12.04	44.81	12.04	39.28
D-3	28.43	44.54	44.54	52.26	28.43	11.96	44.46	11.96	40.38
D-5A	24.47	45.40	45.40	52.63	24.47	12.07	44.77	12.07	36.54
D-5B	27.87	44.77	44.77	52.26	27.87	3.61	23.13	3.61	31.49
D-6A	25.06	44.75	44.75	51.41	25.06	12.06	44.70	12.06	37.13
D-6B	39.96	44.75	44.75	53.36	39.96	12.06	44.70	12.06	52.02
D-7A	32.36	34.47	34.47	42.96	32.36	9.65	35.54	9.65	42.01
D-7B	30.35	34.90	34.90	43.50	30.35	9.81	35.86	9.81	40.16
D-8A	0.00	16.83	14.20	23.26	16.83	4.01	14.16	4.01	20.84
D-9A	0.00	29.15	25.70	44.68	29.15	5.32	25.70	5.32	34.47
D-9B	0.00	38.61	26.99	46.40	38.61	7.40	24.68	7.40	46.01
R-0	0.00	16.53	15.06	24.52	16.53	4.18	15.06	4.18	20.72
R-1	16.97	21.44	21.44	30.07	16.97	5.82	21.44	5.82	22.79
R-2	21.27	23.88	23.88	32.01	21.27	6.52	23.88	6.52	27.79
R-3	0.00	30.71	24.05	34.07	30.71	6.55	24.05	6.55	37.26
R-4	17.51	24.64	24.64	34.26	17.51	6.74	24.64	6.74	24.25
R-5	0.00	14.97	10.76	19.39	14.97	11.75	32.12	11.75	26.72
R-6	11.56	23.56	23.56	31.37	11.56	6.45	23.56	6.45	18.01
R-7	0.00	22.31	21.97	29.78	22.31	6.01	21.97	6.01	28.32
R-8	0.00	21.40	20.73	28.35	21.40	5.61	20.73	5.61	27.01
C-1	0.00	27.06	19.16	33.21	27.06	5.67	19.16	5.67	32.74
C-2	29.39	30.28	30.28	41.17	29.39	9.20	32.85	9.20	38.59
C-3	30.89	31.42	31.42	43.21	30.89	8.68	31.81	8.68	39.57
C-4	35.61	35.89	35.89	47.11	35.61	9.79	36.61	9.79	45.41
C-5	35.05	35.71	35.71	47.21	35.05	10.11	36.32	10.11	45.16
C-6	0.00	29.23	24.30	34.95	29.23	6.70	23.86	6.70	35.93
G-1	0.00	52.34	12.85	21.42	21.42	4.37	12.85	4.37	25.79
G-2	0.00	42.77	12.85	21.44	21.44	4.37	12.85	4.37	25.81
CHO-3	0.00	54.03	10.38	27.25	27.25	3.30	10.38	3.30	30.54
CHO-4	0.00	40.97	22.53	39.41	39.41	7.78	22.92	7.78	47.19
CHO-5	0.00	42.34	22.92	39.78	39.78	7.33	22.15	7.33	47.12
CHO-6	0.00	70.81	10.66	27.53	27.53	8.84	10.38	8.84	36.37
CHO-7	0.00	48.32	22.53	39.14	39.14	15.70	22.53	15.70	54.84
WJE-1	0.00	38.76	23.85	23.85	23.85	13.92	23.85	13.92	37.77

Notes:

refer to Table 3.0 for key to beam designations

$V_{t(a)}$ = shear capacity of top tee using Eq. 2.54.
$V_{t(b)}$ = shear capacity of top tee using Eq. 2.46.
V_{pt} = plastic shear capacity of top tee using Eq. 2.18
V_{tsh} = combined plastic shear capacity of top tee and concrete using Eq. 2.21.
V_{t3} = governing shear capacity of top tee.
V_b = shear capacity of bottom tee using Eq. 2.43.
V_{pb} = plastic shear capacity of bottom tee using Eq. 2.22.
V_{b3} = governing shear capacity of bottom tee.
V_3 = maximum shear capacity as predicted by Method III.

Table G.5 Steel Beam Capacity Summary: Method I, λ = 1.207

Test	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Unreinforced							
Circular Opening							
RBD-C1	2945.79	82.99	2046.38	98.17	1626.80	78.04	1.258
RM-1A	716.71	26.13	728.13	0.00	716.71	0.00	1.016
RM-1B	711.06	13.44	712.13	0.00	711.06	0.00	1.002
RM-2A	798.33	28.27	575.54	31.96	471.46	26.18	1.221
RM-2B	660.89	11.27	295.54	16.41	201.06	11.16	1.470
RM-2C	606.23	20.85	480.54	26.69	349.60	19.42	1.375
RM-3A	713.87	26.35	619.49	20.63	594.13	19.79	1.043
RM-4A	720.71	26.35	691.77	14.38	681.31	14.16	1.015
RM-4B	695.17	13.39	553.77	11.50	530.34	11.01	1.044
RM-4C	701.33	28.58	672.77	13.98	672.83	13.98	1.000
				Mean			1.144
				Coefficient of Variation			0.152
				Resistance Factor			0.897
Rectangular Opening							
B-1	2303.02	37.33	945.00	47.22	738.68	36.91	1.279
B-2	2171.63	34.04	1704.80	42.56	1266.53	31.62	1.346
B-3	2081.90	32.18	1.80	49.74	1.16	32.18	1.546
B-4	2207.88	42.37	1003.00	50.12	832.41	41.60	1.205
CL-4B	3555.94	15.86	1000.00	27.80	569.84	15.84	1.755
CR-6A	2564.72	33.36	1212.37	55.07	728.72	33.10	1.664
CSK-1	3388.11	58.95	2358.39	78.54	1693.13	56.39	1.393
DO-1	725.03	19.56	392.95	24.94	300.65	19.08	1.307
DO-2	674.35	7.13	182.69	11.59	112.14	7.11	1.629
DO-3	691.24	20.98	622.36	19.73	536.43	17.01	1.160
DO-4	698.68	12.20	496.99	15.75	365.57	11.59	1.359
DO-5	674.35	11.97	728.74	0.00	674.35	0.00	1.081
RBD-R1B	3033.59	82.14	1718.81	85.06	1577.91	78.09	1.089
RBD-R2	2925.66	48.72	1269.70	59.86	1018.74	48.03	1.246
RM-11H	749.12	14.01	772.33	0.00	749.12	0.00	1.031
RM-21H	618.27	9.45	342.82	14.27	223.44	9.30	1.534
RM-2F	629.18	9.54	284.54	15.80	170.64	9.48	1.667
RM-4F	733.86	12.02	566.77	11.77	506.83	10.53	1.118
RM-4H	637.98	11.25	483.77	10.04	462.12	9.59	1.047
				Mean			1.340
				Coefficient of Variation			0.174
				Resistance Factor			1.019
Overall Unreinforced							
			Mean				1.272
			Coefficient of Variation				0.182
			Resistance Factor				0.957

Table G.5 (continued)

Test	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Reinforced							
Rectangular Opening							
CR-1A	1079.61	26.83	914.16	21.22	885.11	20.55	1.033
CR-2A	2362.84	55.42	1542.37	70.07	1168.65	53.09	1.320
CR-2B	2362.84	55.42	2331.35	51.74	1925.81	42.74	1.211
CR-2C	2773.20	61.15	2112.23	70.36	1686.25	56.17	1.253
CR-2D	2773.20	61.15	1404.33	82.53	1022.77	60.11	1.373
CR-3A	2362.84	55.42	1707.37	77.57	1168.60	53.09	1.461
CR-3B	2773.20	70.94	2704.85	60.10	2344.37	52.09	1.154
CR-4A	2773.20	49.33	1487.37	67.57	1065.03	48.38	1.397
CR-4B	2773.20	49.33	2313.35	51.34	1935.46	42.95	1.195
CR-5A	2773.20	45.19	1554.23	51.76	1307.87	43.56	1.188
CR-7B	2501.55	58.49	2448.35	54.34	2035.79	45.18	1.203
CR-7D	2501.55	55.55	1319.33	77.58	928.30	54.59	1.421
CSK-2	3680.73	92.96	2872.39	95.69	2473.48	82.40	1.161
CSK-5	3141.22	77.52	2309.50	76.93	2077.04	69.19	1.112
CSK-6	3043.56	47.37	1471.10	48.99	1377.05	45.86	1.068
CSK-7	3043.56	57.14	1780.10	59.29	1623.96	54.09	1.096
CS-1	2137.01	43.95	1811.25	30.08	1856.00	30.82	0.976
CS-2	2155.60	42.31	1772.25	29.43	1840.91	30.57	0.963
CS-3	2095.06	43.84	1604.00	40.03	1505.14	37.56	1.066
RL-5	2667.74	38.57	2893.50	0.00	2667.74	0.00	1.085
RL-6	2701.97	22.56	1048.89	21.36	1083.56	22.07	0.968
				Mean			1.176
				Coefficient of Variation			0.128
				Resistance Factor			0.951
Overall Steel Beams							
			Mean				1.232
			Coefficient of Variation				0.166
			Resistance Factor				0.947

Notes:

(1) refer to Table 3.0 for key to beam designations

Table G.6 Steel Beam Capacity Summary: Method III, λ = 1.207

Test	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Unreinforced							
Circular Opening							
RBD-C1	2945.79	82.99	2046.38	98.17	1626.80	78.04	1.258
RM-1A	716.71	22.82	728.13	0.00	716.71	0.00	1.016
RM-1B	711.06	12.41	712.13	0.00	711.06	0.00	1.002
RM-2A	798.33	24.45	575.54	31.96	418.11	23.22	1.377
RM-2B	660.89	10.16	295.54	16.41	181.72	10.09	1.626
RM-2C	606.23	18.13	480.54	26.69	311.04	17.28	1.545
RM-3A	713.87	23.08	619.49	20.63	558.14	18.59	1.110
RM-4A	720.71	23.04	691.77	14.38	664.65	13.82	1.041
RM-4B	695.17	12.41	553.77	11.50	507.20	10.53	1.092
RM-4C	701.33	25.75	672.77	13.98	663.46	13.79	1.014
				Mean			1.208
				Coefficient of Variation			0.193
				Resistance Factor			0.895
Rectangular Opening							
B-1	2303.02	36.82	945.00	47.22	728.99	36.43	1.296
B-2	2171.63	33.31	1704.80	42.56	1244.53	31.07	1.370
B-3	2081.90	31.49	1.80	49.74	1.14	31.49	1.580
B-4	2207.88	41.69	1003.00	50.12	819.73	40.96	1.224
CL-4B	3555.94	14.71	1000.00	27.80	528.58	14.69	1.892
CR-6A	2564.72	32.06	1212.37	55.07	700.86	31.84	1.730
CSK-1	3388.11	56.08	2358.39	78.54	1620.25	53.96	1.456
DO-1	725.03	18.98	392.95	24.94	292.30	18.55	1.344
DO-2	674.35	6.12	182.69	11.59	96.35	6.11	1.896
DO-3	691.24	20.23	622.36	19.73	525.85	16.67	1.184
DO-4	698.68	11.51	496.99	15.75	347.59	11.02	1.430
DO-5	674.35	9.97	728.74	0.00	674.35	0.00	1.081
RBD-R1B	3033.59	82.14	1718.81	85.06	1577.91	78.09	1.089
RBD-R2	2925.66	48.37	1269.70	59.86	1011.69	47.70	1.255
RM-11H	749.12	13.28	772.33	0.00	749.12	0.00	1.031
RM-21H	618.27	8.45	342.82	14.27	200.63	8.35	1.709
RM-2F	629.18	8.46	284.54	15.80	151.60	8.42	1.877
RM-4F	733.86	10.85	566.77	11.77	471.50	9.79	1.202
RM-4H	637.98	8.92	483.77	10.04	393.37	8.16	1.230
				Mean			1.415
				Coefficient of Variation			0.202
				Resistance Factor			1.034
Overall Unreinforced							
			Mean				1.343
			Coefficient of Variation				0.210
			Resistance Factor				0.970

Table G.6 (continued)

Test	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Reinforced							
Rectangular Opening							
CR-1A	1079.61	26.29	914.16	21.22	876.97	20.36	1.042
CR-2A	2362.84	55.05	1542.37	70.07	1161.73	52.78	1.328
CR-2B	2362.84	55.05	2331.35	51.74	1919.83	42.61	1.214
CR-2C	2773.20	59.67	2112.23	70.36	1654.31	55.11	1.277
CR-2D	2773.20	59.67	1404.33	82.53	999.23	58.72	1.405
CR-3A	2362.84	54.25	1707.37	77.57	1146.69	52.10	1.489
CR-3B	2773.20	70.94	2704.85	60.10	2344.37	52.09	1.154
CR-4A	2773.20	48.69	1487.37	67.57	1052.00	47.79	1.414
CR-4B	2773.20	48.69	2313.35	51.34	1918.77	42.58	1.206
CR-5A	2773.20	43.53	1554.23	51.76	1264.36	42.11	1.229
CR-7B	2501.55	57.02	2448.35	54.34	2011.66	44.65	1.217
CR-7D	2501.55	54.19	1319.33	77.58	906.68	53.32	1.455
CSK-2	3680.73	92.52	2872.39	95.69	2465.42	82.13	1.165
CSK-5	3141.22	76.46	2309.50	76.93	2056.65	68.51	1.123
CSK-6	3043.56	46.49	1471.10	48.99	1353.83	45.08	1.087
CSK-7	3043.56	55.57	1780.10	59.29	1585.73	52.82	1.123
CS-1	2137.01	42.32	1811.25	30.08	1831.00	30.41	0.989
CS-2	2155.60	40.75	1772.25	29.43	1814.07	30.12	0.977
CS-3	2095.06	40.53	1604.00	40.03	1429.73	35.68	1.122
RL-5	2667.74	38.60	2893.50	0.00	2667.74	0.00	1.085
RL-6	2701.97	22.28	1048.89	21.36	1070.93	21.81	0.979
			Mean				1.194
			Coefficient of Variation				0.129
			Resistance Factor				0.964
Overall Steel Beams							
			Mean				1.281
			Coefficient of Variation				0.193
			Resistance Factor				0.949

Notes:

(1) refer to Table 3.0 for key to beam designations

Table G.7 Composite Beam Capacity Summary: Method I, λ = 1.207

Test	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Unreinforced							
Ribbed Slab							
D-1	5405.49	39.68	1606.00	37.80	1669.31	39.29	0.962
D-2	5967.14	38.67	3095.00	39.00	2941.32	37.06	1.052
D-3	6096.61	39.89	6075.00	11.30	6050.60	11.25	1.004
D-5A	5388.57	35.61	2768.00	34.60	2720.95	34.01	1.017
D-5B	5226.80	31.27	2568.00	32.20	2409.69	30.21	1.066
D-6A	5422.56	36.76	0.00	41.00	0.00	36.76	1.115
D-6B	5733.80	52.11	2070.00	48.90	2165.57	51.16	0.956
D-7A	4665.32	42.44	1845.00	43.50	1766.68	41.65	1.044
D-7B	4362.93	39.62	3379.00	42.60	2827.11	35.64	1.195
D-8A	1344.57	21.38	774.00	19.40	790.80	19.82	0.979
D-8B	1056.25	14.76	427.00	14.30	430.55	14.42	0.992
D-9A	4791.16	36.09	1474.00	34.50	1525.13	35.70	0.966
D-9B	4588.71	47.89	1755.00	47.30	1743.61	46.99	1.007
R-0	1288.17	21.16	752.00	18.20	798.46	19.32	0.942
R-1	2630.28	23.96	978.00	26.00	889.50	23.65	1.099
R-2	3516.43	27.75	2904.00	28.70	2447.86	24.19	1.186
R-3	3774.33	39.50	3993.00	16.40	3701.23	15.20	1.079
R-4	3022.68	24.15	3212.00	13.10	2899.47	11.83	1.108
R-5	2791.69	28.03	1038.00	27.60	1036.07	27.55	1.002
R-6	2594.94	18 89	786.00	21.20	695.81	18.77	1.130
R-7	2833.28	30.31	1134.00	30.50	1104.11	29.70	1.027
R-8	2817.84	29.30	1075.00	28.90	1069.76	28.76	1.005
			Mean				1.045
			Coefficient of Variation				0.070
			Resistance Factor				0.899
Solid Slab							
C-1	3110.10	35.70	2886.00	33.40	2458.30	28.45	1.174
C-2	4604.48	38.70	4107.00	36.80	3534.04	31.67	1.162
C-3	4624.92	39.90	5468.00	14.00	4585.32	11.74	1.193
C-4	4900.59	46.28	1723.00	47.60	1653.61	45.68	1.042
C-5	5138.23	45.98	3511.00	48.10	3092.00	42.36	1.136
C-6	3188.26	38.69	1471.00	40.40	1370.48	37.64	1.073
G-1	1734.13	28.09	791.00	32.70	666.40	27.55	1.187
G-2	1712.64	28.11	1296.00	26.50	1196.46	24.46	1.083
CHO-3	1369.30	31.68	634.00	35.70	550.21	30.98	1.152
CHO-4	2356.96	48.01	1477.00	46.70	1403.01	44.36	1.053
CHO-5	2444.36	47.94	2319.00	17.90	2396.60	18.50	0.968
			Mean				1.111
			Coefficient of Variation				0.065
			Resistance Factor				0.960
Overall Unreinforced							
			Mean				1.068
			Coefficient of Variation				0.073
			Resistance Factor				0.917

Table G.7 (continued)

Test	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Reinforced							
Ribbed Slab							
WJE-1	7782.56	37.42	7155.63	0.00	7782.56	0.00	0.919
Solid Slab							
CHO-6	1745.52	37.06	721.00	40.60	646.74	36.42	1.115
CHO-7	2983.27	55.56	2664.00	20.60	2915.29	22.54	0.914
			Mean				1.015
			Coefficient of Variation				0.140
			Resistance Factor				0.808
Overall Reinforced							
			Mean				0.983
			Coefficient of Variation				0.117
			Resistance Factor				0.805
Overall Composite Beams							
			Mean				1.060
			Coefficient of Variation				0.079
			Resistance Factor				0.905

Notes:

Refer to Table 3.0 for key to beam designations

Table G.8 Composite Beam Capacity Summary: Method III, λ = 1.207

Test	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Unreinforced							
Ribbed Slab							
D-1	5405.49	37.83	1606.00	37.80	1593.53	37.51	1.008
D-2	5967.14	39.28	3095.00	39.00	2981.87	37.57	1.038
D-3	6096.61	40.38	6075.00	11.30	6052.26	11.26	1.004
D-5A	5388.57	36.54	2768.00	34.60	2782.43	34.78	0.995
D-5B	5226.80	31.49	2568.00	32.20	2424.53	30.40	1.059
D-6A	5422.56	37.13	0.00	41.00	0.00	37.13	1.104
D-6B	5733.80	52.02	2070.00	48.90	2161.99	51.07	0.957
D-7A	4665.32	42.01	1845.00	43.50	1750.04	41.26	1.054
D-7B	4362.93	40.16	3379.00	42.60	2855.00	35.99	1.184
D-8A	1344.57	20.84	774.00	19.40	774.60	19.42	0.999
D-8B	1056.25	14.87	427.00	14.30	433.54	14.52	0.985
D-9A	4791.16	34.47	1474.00	34.50	1458.75	34.14	1.010
D-9B	4588.71	46.01	1755.00	47.30	1678.74	45.24	1.045
R-0	1288.17	20.72	752.00	18.20	785.65	19.01	0.957
R-1	2630.28	22.79	978.00	26.00	847.76	22.54	1.154
R-2	3516.43	27.79	2904.00	28.70	2450.20	24.22	1.185
R-3	3774.33	37.26	3993.00	16.40	3687.85	15.15	1.083
R-4	3022.68	24.25	3212.00	13.10	2900.85	11.83	1.107
R-5	2791.69	26.72	1038.00	27.60	989.64	26.31	1.049
R-6	2594.94	18.01	786.00	21.20	664.03	17.91	1.184
R-7	2833.28	28.32	1134.00	30.50	1035.52	27.85	1.095
R-8	2817.84	27.01	1075.00	28.90	990.08	26.62	1.086
			Mean				1.065
			Coefficient of Variation				0.066
			Resistance Factor				0.920
Solid Slab							
C-1	3110.10	32.74	2886.00	33.40	2346.28	27.15	1.230
C-2	4604.48	38.59	4107.00	36.80	3528.34	31.62	1.164
C-3	4624.92	39.57	5468.00	14.00	4584.33	11.74	1.193
C-4	4900.59	45.41	1723.00	47.60	1623.42	44.85	1.061
C-5	5138.23	45.16	3511.00	48.10	3048.64	41.77	1.152
C-6	3188.26	35.93	1471.00	40.40	1279.56	35.14	1.150
G-1	1734.13	25.79	791.00	32.70	614.44	25.40	1.287
G-2	1712.64	25.81	1296.00	26.50	1128.26	23.07	1.149
CHO-3	1369.30	30.54	634.00	35.70	531.65	29.94	1.193
CHO-4	2356.96	47.19	1477.00	46.70	1384.08	43.75	1.067
CHO-5	2444.36	47.12	2319.00	17.90	2394.17	18.48	0.969
			Mean				1.147
			Coefficient of Variation				0.076
			Resistance Factor				0.982
Overall Unreinforced							
			Mean				1.093
			Coefficient of Variation				0.078
			Resistance Factor				0.934

Table G.8 (continued)

Test	M_m (in.-k)	V_m (k)	M_{test} (in.-k)	V_{test} (k)	M_n (in.-k)	V_n (k)	Test/ Theory
Reinforced							
Ribbed Slab							
WJE-1	7782.56	37.77	7155.63	0.00	7782.56	0.00	0.919
Solid Slab							
CHO-6	1745.52	36.37	721.00	40.60	635.33	35.78	1.135
CHO-7	2983.27	54.84	2664.00	20.60	2912.73	22.52	0.915
			Mean				1.025
			Coefficient of Variation				0.152
			Resistance Factor				0.803
Overall Reinforced							
			Mean				0.990
			Coefficient of Variation				0.127
			Resistance Factor				0.801
Overall Composite Beams							
			Mean				1.083
			Coefficient of Variation				0.086
			Resistance Factor				0.918

Notes:

refer to Table 3.0 for key to beam designations

Table G.9 Analysis Summary, λ = 1.207 (Methods I and III)

Mean

	# Beams	I	II	III	Redwood (C)	Redwood (L)
STEEL BEAMS						
Unreinforced	29	1.272	1.248	1.343	1.212	1.347
Rectangular Opening	19	1.340	1.302	1.415	1.265	1.391
Circular Opening	10	1.144	1.145	1.208	1.111	1.264
Reinforced						
Rectangular Opening	21	1.176	1.166	1.194	1.142	1.362
OVERALL STEEL	50	1.232	1.213	1.281	1.183	1.353
COMPOSITE BEAMS						
Unreinforced	32	1.068	1.073	1.093	1.131	N/A
Ribbed Slab	21	1.045	1.037	1.065	1.090	N/A
Solid Slab	11	1.111	1.141	1.147	1.207	N/A
Reinforced	3	0.983	0.985	0.990	N/A	N/A
Ribbed Slab	1	0.919	0.919	0.919	N/A	N/A
Solid Slab	2	1.015	1.019	1.025	N/A	N/A
OVERALL COMPOSITE	35	1.060	1.065	1.083	1.131	N/A

Coefficient of Variation

	I	II	III	Redwood (C)	Redwood (L)
STEEL BEAMS					
Unreinforced	0.182	0.203	0.210	0.186	0.182
Rectangular Opening	0.174	0.211	0.202	0.191	0.195
Circular Opening	0.152	0.154	0.193	0.140	0.131
Reinforced					
Rectangular Opening	0.128	0.125	0.129	0.151	0.195
OVERALL STEEL	0.166	0.179	0.193	0.174	0.185
COMPOSITE BEAMS					
Unreinforced	0.073	0.084	0.078	0.128	N/A
Ribbed Slab	0.070	0.069	0.066	0.121	N/A
Solid Slab	0.065	0.075	0.076	0.124	N/A
Reinforced	0.117	0.122	0.127	N/A	N/A
Ribbed Slab	N/A	N/A	N/A	N/A	N/A
Solid Slab	0.140	0.146	0.152	N/A	N/A
OVERALL COMPOSITE	0.079	0.088	0.086	0.128	N/A

Resistance Factor

	I	II	III	Redwood (C)	Redwood (L)
STEEL BEAMS					
Unreinforced	0.957	0.911	0.970	0.907	1.013
Rectangular Opening	1.019	0.939	1.034	0.939	1.027
Circular Opening	0.897	0.895	0.895	0.885	1.018
Reinforced					
Rectangular Opening	0.951	0.946	0.964	0.896	1.005
OVERALL STEEL	0.947	0.916	0.949	0.900	1.013
COMPOSITE BEAMS					
Unreinforced	0.917	0.912	0.934	0.914	N/A
Ribbed Slab	0.899	0.893	0.920	0.889	N/A
Solid Slab	0.960	0.978	0.982	0.981	N/A
Reinforced	0.805	0.802	0.801	N/A	N/A
Ribbed Slab	N/A	N/A	N/A	N/A	N/A
Solid Slab	0.808	0.805	0.803	N/A	N/A
OVERALL COMPOSITE	0.905	0.901	0.918	0.895	N/A