C3.3.6 Bearing For cold-formed steel beams, transverse and shear stiffeners are not frequently used. The webs of beams may cripple due to the high local intensity of the load or reaction. Figure C3.3.6(1) shows the types of failure caused by web crippling of unreinforced single webs (see Figure C3.3.6(1)(a)) and of I-beams (see Figure C3.3.6(1)(b)).

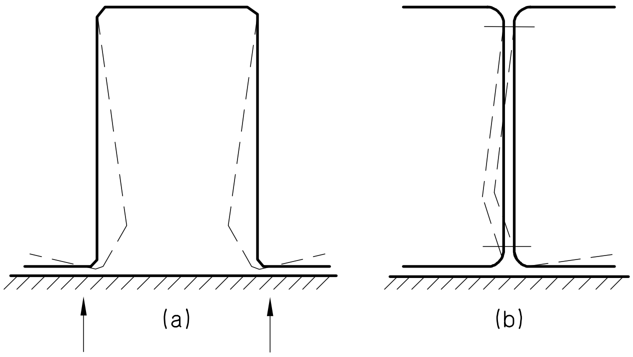

FIGURE C3.3.6(1) WEB CRIPPLING OF COLD-FORMED STEEL BEAMS

In the past, the buckling problem of separate flat rectangular plates and web crippling behaviour of cold-formed steel beam webs under locally distributed edge forces have been studied by numerous investigators (Yu, 1991) and it was found that the theoretical analysis of web crippling for cold-formed steel flexural members is rather complicated because it involves the following:

(a) Non-uniform stress distribution under the applied load and adjacent portions of the web.

(b) Elastic and inelastic stability of the web element.

(c) Local yielding in the immediate region of load application.

(d) Bending produced by eccentric load (or reaction) when it is applied on the bearing flange at a distance beyond the curved transition of the web.

(e) Initial out-of-plane imperfection of plate elements.

(f) Various edge restraints provided by beam flanges and interaction between flange and web elements.

(g) Inclined webs for decks and panels.

For these reasons, the present AISI design provisions for web crippling and the AS/NZS 4600 design provisions for bearing are based on the extensive experimental investigations conducted at Cornell University by Winter and Pian (Winter and Pian, 1946), and by Zetlin (Zetlin, 1955) in the 1940s and 1950s, and at the University of Missouri-Rolla by Hetrakul and Yu (Hetrakul and Yu, 1978). In these experimental investigations, the web crippling tests were carried out under the following four loading conditions for beams having single unreinforced webs and I-beams:

(i) End one-flange (EOF) loading.

(ii) Interior one-flange (IOF) loading.

(iii) End two-flange (ETF) loading.

(iv) Interior two-flange (ITF) loading.

All loading conditions are shown in Figure C3.3.6(2). In Figures (a) and (b), the distances between bearing plates were kept to no less than 1.5 times the web depth in order to avoid the two-flange loading action.

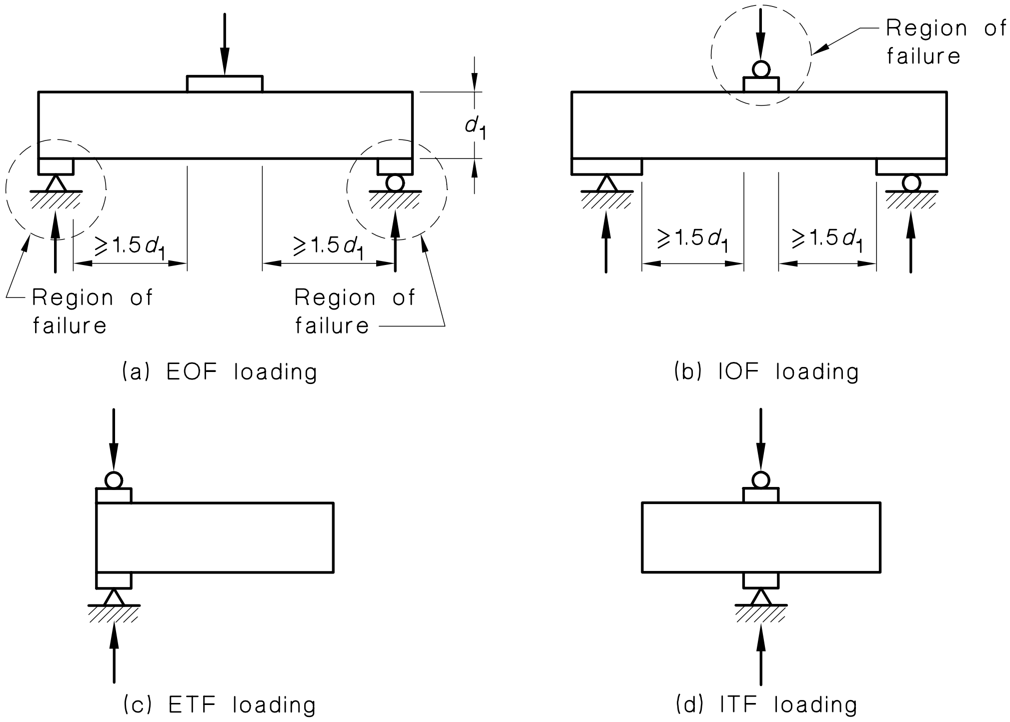

FIGURE C3.3.6(2) LOADING CONDITIONS FOR WEB CRIPPLING TESTS

Clause 3.3.6 of the Standard provides design equations to determine the web crippling strength of flexural members having flat single webs (channels, Z-sections, hat sections, tubular members, roof deck, floor deck and the like) and I-beams (made of two channels connected back to back, by welding two angles to a channel, or by connecting three channels). Different design equations are used for various loading conditions, as shown in Figure C3.3.6(3), and Tables 3.3.6(1) and 3.3.6(2) in the Standard. These design equations are based on experimental evidence (Winter, 1970; Hetrakul and Yu, 1978) and the assumed distributions of loads or reactions into the web as shown in Figure C3.3.6(4).

The assumed distributions of loads or reactions into the web, as shown in Figure C3.3.6(4), are independent of the flexural response of the beam. Due to flexure, the point of bearing will vary relative to the plane of bearing resulting in non-uniform bearing load distribution into the web. The value of R_b will vary because of a transition from the interior one-flange loading (see Figure C3.3.6(4)(b)) to the end one-flange loading condition (see Figure C3.3.6(4)(a)). These discrete conditions represent the experimental basis on which the design provisions were founded (Winter, 1970; Hetrakul and Yu, 1978).

From Tables 3.2.6(1) and 3.3.6(2) in the Standard, it can be seen that the nominal capacity for concentrated load or reaction of cold-formed steel beams depends on the ratios of d_1/t_w, l_b/t_w, r_i/t_w, the web thickness ($t(t_w)$), the yield stress (f_y), and the web inclination angle (θ).

With regard to the limit states design approach, the use of ϕ_w equal to 0.75 for single unreinforced webs and ϕ_w equal to 0.80 for I-sections provide values of safety index ranging from 2.4 to 3.8.

Recent research indicated that a Z-section having its end support flange bolted to the section's supporting member through two 12.7 mm diameter bolts will experience an increase in end-one-flange web crippling capacity (Bhakta, LaBoube and Yu, 1992; Cain, LaBoube and Yu, 1995). The increase in load-carrying capacity was shown to range from 27 to 55% for the sections under the limitations prescribed in the Standard. A lower bound value of 30% increase is permitted in Clause 3.3.6 of the Standard.

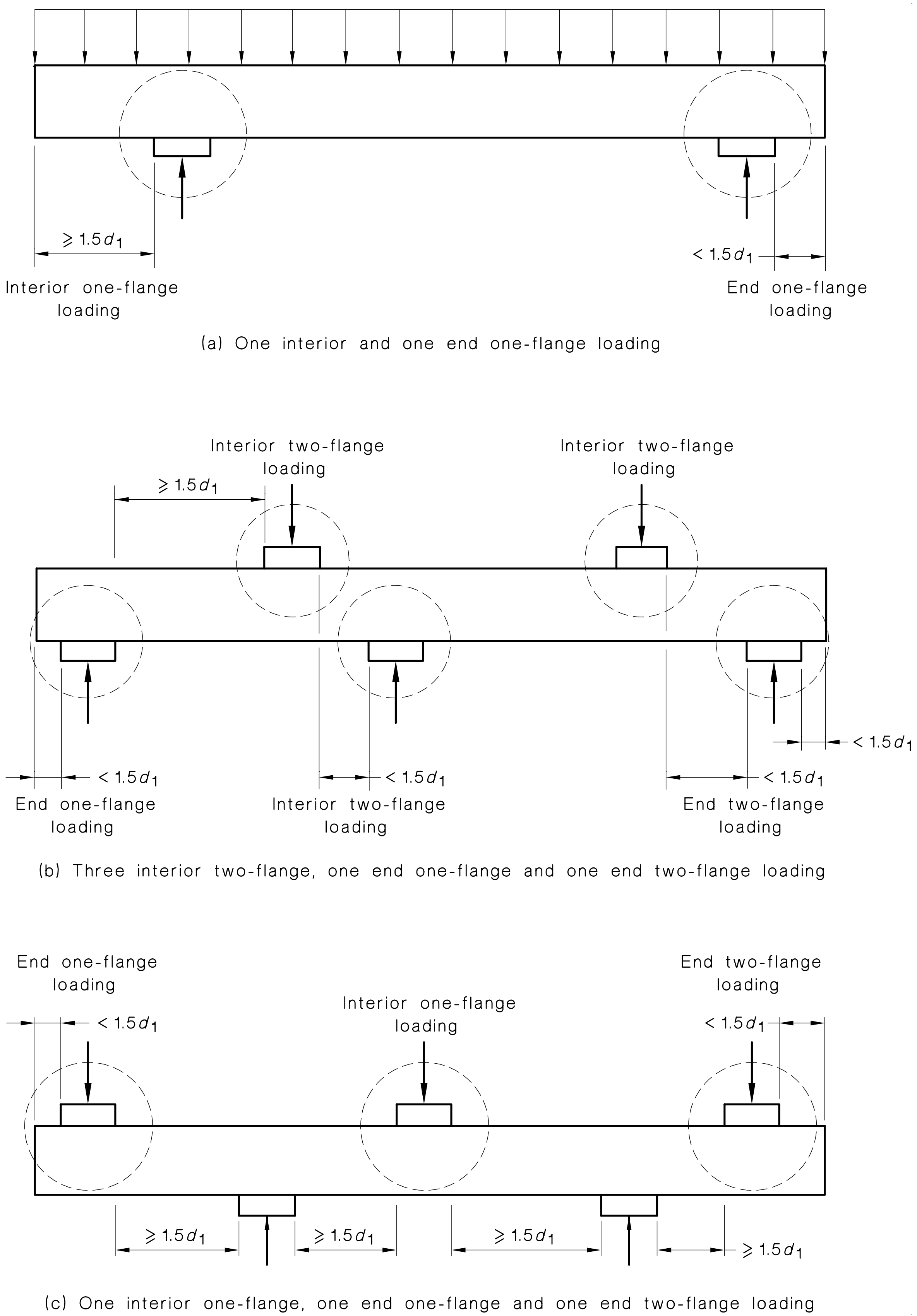

FIGURE C3.3.6(3) APPLICATION OF DESIGN EQUATIONS GIVEN IN TABLES 3.3.6(1) AND 3.3.6(2) OF THE STANDARD

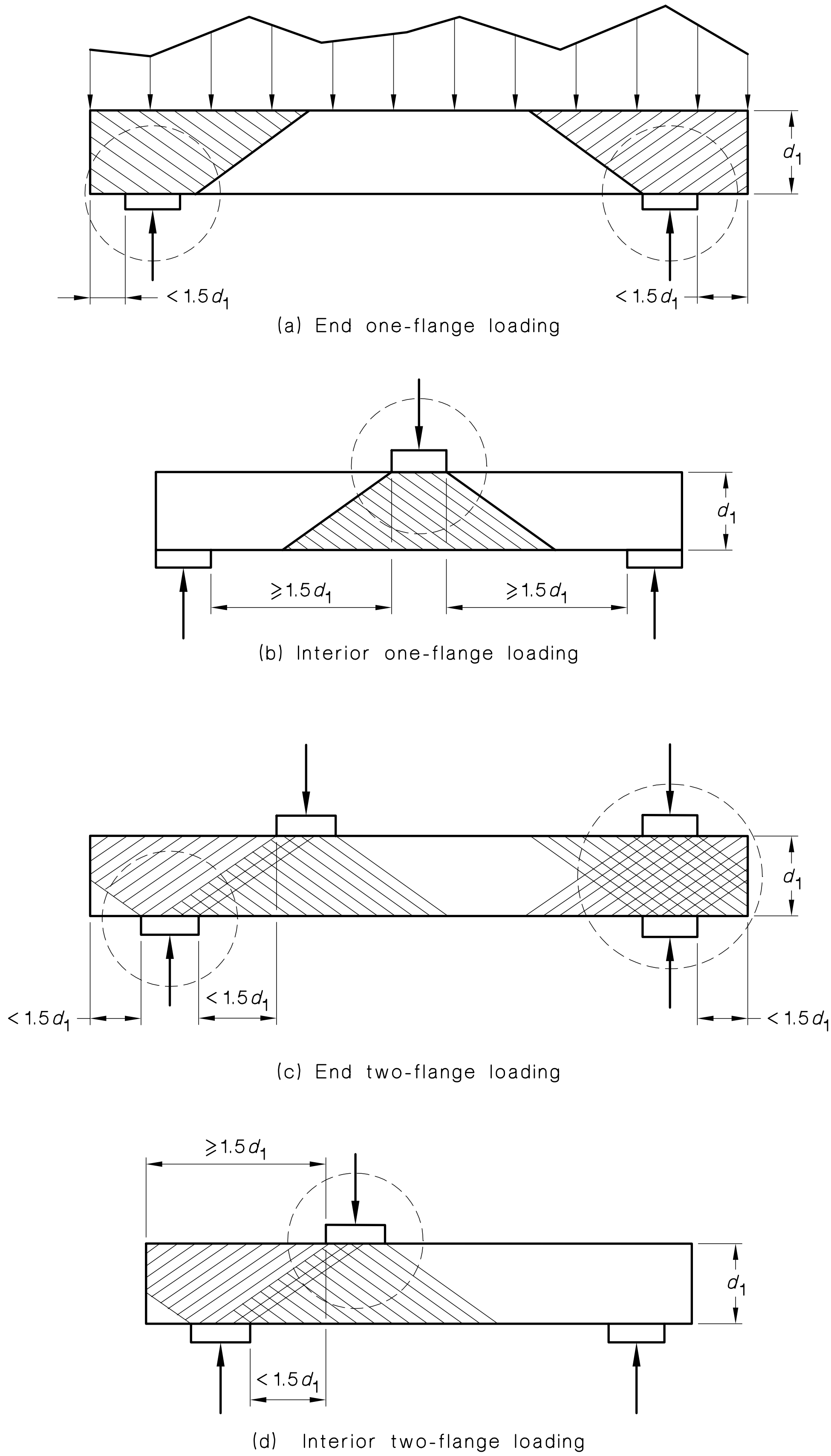

FIGURE C3.3.6(4) ASSUMED DISTRIBUTION OF REACTION OR LOAD

C3.3.7 Combined bending and bearing Clause 3.3.7 of the Standard contains interactions for the combination of bending and bearing. Clauses 3.3.7(a) and 3.3.7(b) of the Standard are based on the studies conducted at the University of Missouri-Rolla for the effect of bending on the reduction of web crippling loads with the applicable capacity factors used for bending and bearing (Hetrakul and Yu, 1978 and 1980; Yu, 1981 and 1991). Figures C3.3.7(1) and C3.3.7(2) show the correlations between the interactions and test results. For embossed webs, bearing should be determined by tests in accordance with Section 6 of the Standard.

The exception included in Clause 3.3.7 of the Standard for single unreinforced webs applies to the interior supports of continuous spans using decks and beams, as shown in Figure C3.3.7(3). Results of continuous beam tests of steel decks (Yu, 1981) and several independent studies by manufacturers indicate that, for these types of members, the post-buckling behaviour of webs at interior supports differs from the type of failure mode occurring under concentrated loads on single span beams. This post-buckling strength enables the member to redistribute the moments in continuous spans. For this reason, the interaction in Clause 3.3.7(a) of the Standard is not applicable to the interaction between bending and the reaction at interior supports of continuous spans. This exception applies only to the members shown in Figure C3.3.7(3) and similar situations explicitly described in Clause 3.3.7 of the Standard.

The exception should be interpreted to mean that the effects of combined bending and bearing need not be checked for determining load-carrying capacity. Furthermore, the positive bending resistance of the beam should be at least 90% of the negative bending resistance in order to ensure the safety specified by Clause 3.3.7 of the Standard. Using this procedure, serviceability loads may—

(a) produce slight deformations in the beam over the support;

(b) increase the actual compressive bending stresses over the support to as high as $0.8 f_y$; and

(c) result in additional bending deflection of up to 22% due to elastic moment redistribution.

If load-carrying capacity is not the primary design concern because of the above behaviour, the designer is urged to ignore the exception in Clause 3.3.7(a) of the Standard.

With regard to Clause 3.3.7(b) of the Standard, previous tests indicate that when the d_1/t_w ratio of an I-beam web does not exceed $2.33/\sqrt{f_y/E}$ and when $\lambda \leq 0.673$, the bending moment has little or no effect on the web crippling load (Yu, 1991). For this reason, the permissible reaction or concentrated load can be determined by the equations given in Clause 3.3.6 of the Standard without reduction for the presence of bending.

In the development of the limit states equations, a total of 551 tests were calibrated for combined bending and bearing. Based on ϕ_w equal to 0.75 for single unreinforced webs and ϕ_w equal to 0.80 for I-sections, the values of safety index vary from 2.5 to 3.3.

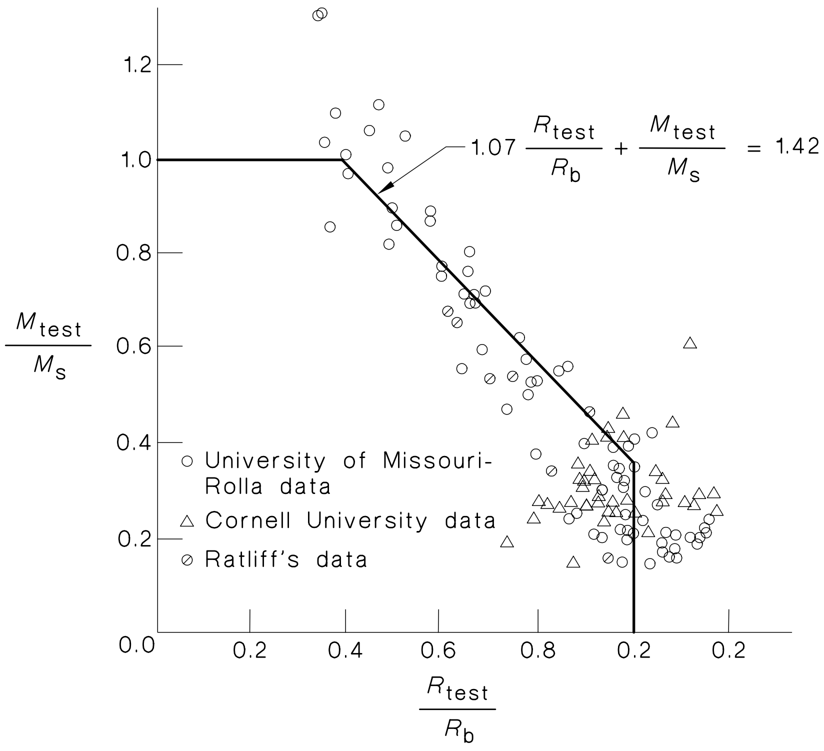

FIGURE C3.3.7(1) GRAPHIC PRESENTATION FOR WEB CRIPPLING AND COMBINED WEB CRIPPLING AND BENDING FOR SINGLE UNREINFORCED WEBS

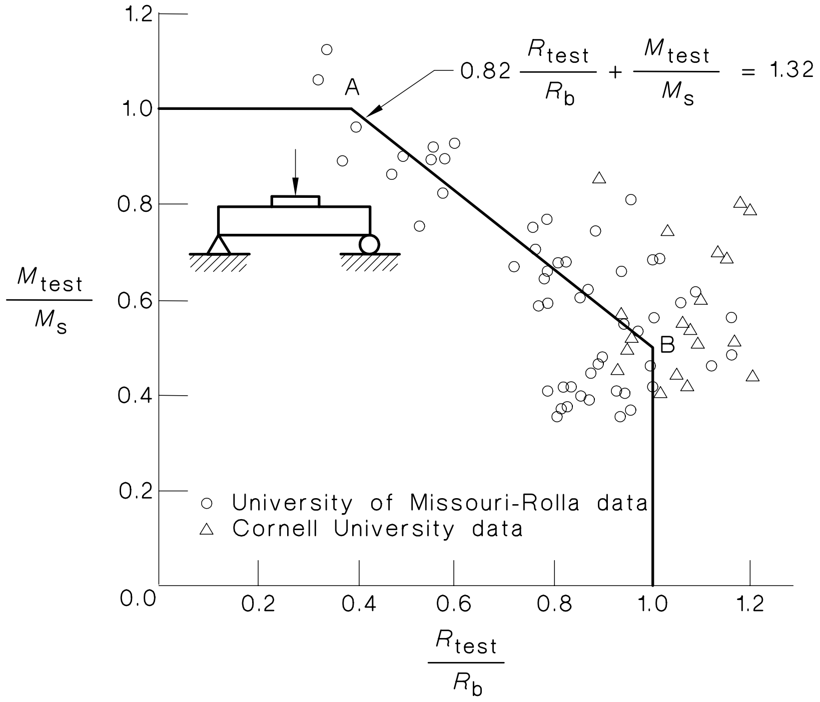

FIGURE C3.3.7(2) INTERACTION BETWEEN WEB CRIPPLING AND BENDING FOR I-BEAMS HAVING UNREINFORCED WEBS

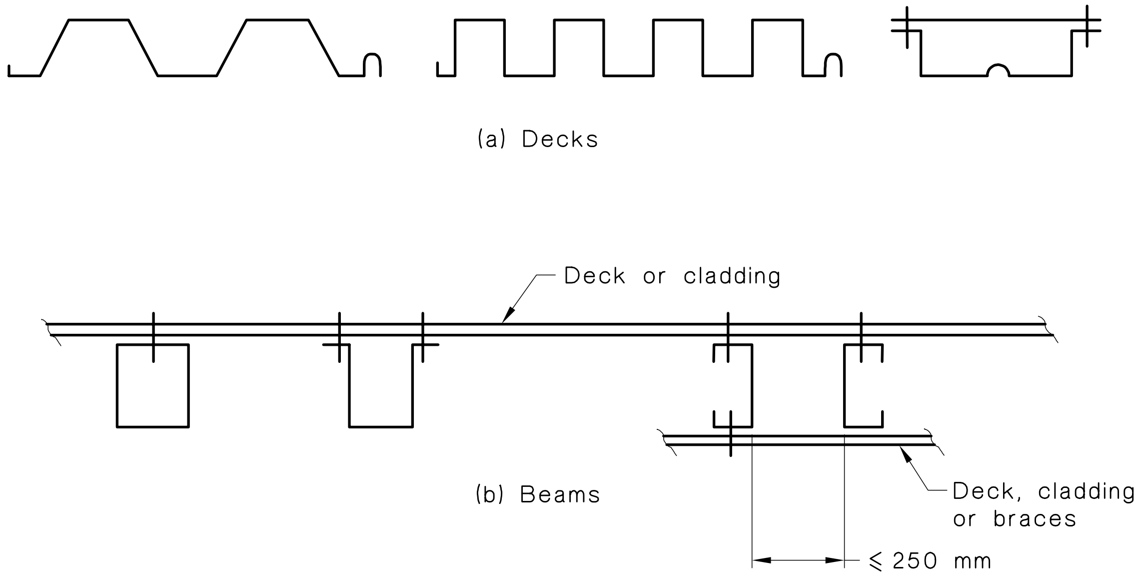

FIGURE C3.3.7(3) SECTIONS USED FOR EXCEPTION OF CLAUSE 3.3.7(a) OF THE STANDARD

C3.4 CONCENTRICALLY LOADED COMPRESSION MEMBERS Depending on the configuration of the cross-section, thickness of material, unbraced length, and end restraint, axially loaded compression members should be designed for the following ultimate limit state conditions:

(a) *Yielding* It is well known that a very short, compact column under an axial load may fail by yielding. The yield load is determined by the following equation:

$$N_y = A_g f_y \qquad \text{. . . C3.4(1)}$$

where

A_g = gross area of the column

f_y = yield stress of steel

(b) *Overall column buckling (flexural buckling, torsional buckling or flexural-torsional buckling)*

(i) *Flexural buckling of columns*

(A) *Elastic buckling stress* A slender, axially loaded column may fail by overall flexural buckling if the cross-section of the column is a doubly-symmetric shape, closed shape (square or rectangular tube), cylindrical shape, or point-symmetric shape. For singly-symmetric shapes, flexural buckling is one of the possible failure modes. Wall studs connected with sheeting material can also fail by flexural buckling.

The elastic critical buckling load for a long column can be determined by the following Euler formula:

$$N_{oc} = \frac{\pi^2 EI}{(kl)^2} \qquad \text{. . . C3.4(2)}$$

where

N_{oc} = column buckling load in the elastic range

E = modulus of elasticity

I = second moment of area

k = effective length factor

l = unbraced length

Accordingly, the elastic column buckling stress can be calculated as follows:

$$(f_{oc})_E = \frac{N_{oc}}{A_g} = \frac{\pi^2 E}{\left(\frac{kl}{r}\right)^2} = \frac{\pi^2 E}{\left(\frac{l_e}{r}\right)^2} \qquad \text{. . . C3.4(3)}$$

where

r = radius gyration of the full cross-section

l_e/r = the effective slenderness ratio

(B) *Inelastic buckling stress* When the elastic column buckling stress calculated using Equation C3.4(3) exceeds the proportional limit (f_{pr}), the column will buckle in the inelastic range. Prior to 1996, the following equation was used in the AISI Specification for calculating the inelastic column buckling stress:

$$(f_{oc})_I = f_y\left(1 - \frac{f_y}{4(f_{oc})_E}\right) \qquad \text{. . . C3.4(4)}$$

It should be noted that because Equation C3.4(4) is based on the assumption that f_{pr} is equal to $f_y/2$, it is applicable only for $(f_{oc})_E$ greater than or equal to $f_y/2$.

By using λ_c as the column slenderness parameter instead of the slenderness ratio (l_e/r), Equation C3.4(4) can be rewritten as follows:

$$(f_{oc})_I = \left(1 - \frac{\lambda_c^2}{4}\right) f_y \qquad \text{. . . C3.4(5)}$$

where

$$\lambda_c = \sqrt{\frac{f_y}{(f_{oc})_E}} = \frac{l_e}{\pi r}\sqrt{\frac{f_y}{E}} \qquad \text{. . . C3.4(6)}$$

Accordingly, Equation C3.4(5) is applicable only for λ_c less than or equal to $\sqrt{2}$.

In AS 1538—1988, the Perry curve (Ayrton and Perry (1986)) was used to define the column strength since geometric imperfections were included in the column design philosophy used in Australia. The Perry curve gives a lower column strength than Equation 3.4(5) in the Standard due to the inclusion of imperfections.

(C) *Design compressive axial force for locally stable columns* If the individual components of compression members have small b/t ratios, local buckling will not occur before the compressive stress reaches the column buckling stress and the yield stress of steel. Therefore, the nominal axial strength can be determined as follows:

$$N_c = A_g \, (f_{oc})_{\mathrm{EorI}} \qquad \ldots \text{C3.4(7)}$$

where

N_c = nominal member capacity of the member in compression

A_g = gross area of the column

$(f_{oc})_{\mathrm{EorI}}$ = column buckling stress (elastic or inelastic as appropriate)

(D) *Design compressive axial force for locally unstable columns* For cold-formed steel compression members with large b/t ratios, local buckling of individual component plates may occur before the applied load reaches the nominal axial strength determined by Equation C3.4(7). The interaction effect of the local and overall column buckling may result in a reduction of the overall column strength. From 1946 through 1986, the effect of local buckling on column strength was considered in the AISI Specification and AS 1538—1988 by using a form factor (Q) in the determination of the permissible stress for the design of axially loaded compression members (Winter, 1970; Yu, 1991). Even though the Q-factor method was used successfully for the design of cold-formed steel compression members, research work conducted at Cornell University and other institutions has shown that this method is capable of improvement. On the basis of the test results and analytical studies of DeWolf, Peköz, Winter, and Mulligan (DeWolf, Peköz and Winter, 1974; Mulligan and Peköz, 1984) and Peköz's development of a unified approach for the design of cold-formed steel members (Peköz, 1986b), the Q-factor method was eliminated in the 1986 edition of the AISI Specification. In order to reflect the effect of local buckling on the reduction of column strength, the nominal axial strength is determined by the critical column buckling stress and the *effective area* (A_e) instead of the full sectional area. For a more in depth discussion of the background for these provisions, see Peköz (1986b). Therefore, the nominal member capacity of cold-formed steel compression members can be determined by the following equation:

$$N_c = A_e (f_{oc})_{\mathrm{EorI}} \qquad \ldots \text{C3.4(8)}$$

where $(f_{oc})_{\mathrm{EorI}}$ is the column buckling stress (elastic or inelastic as appropriate).

An exception for Equation C3.4(7) is for C-shapes and Z-shapes, and single-angle sections with unstiffened flanges. For these cases, the nominal axial strength is also limited by the following capacity, which is determined by the local buckling stress of the unstiffened element and the area of the full cross-section as follows:

$$N_c = \frac{A\pi^2 E}{25.7\left(\frac{b}{t}\right)^2} \qquad \ldots \text{C3.4(9)}$$

Equation C3.4(9) was included in Section C4(b) of the 1986 edition of the AISI Specification when the unified design approach was adopted. A recent study conducted by Rasmussen at the University of Sydney (Rasmussen, 1994) indicated that the design provisions of Section C4(b) of the 1986 AISI Specification lead to unnecessarily and excessively conservative results. This conclusion was based on the analytical studies carefully validated against test results as reported by Rasmussen and Hancock (1992). Consequently, Section C4(b) of the AISI Specification (Equation C-C4(9)) was deleted in 1996 and is not included in the Standard.

In the AISI Specification, the design equations for calculating the critical stress have been changed from those given by Equations C3.4(3) and C3.4(4) to those used in the AISC LRFD Specification (AISC, 1993). As given in Clause 3.4.1 of the Standard, these design regulations are as follows:

For $\lambda_c \leq 1.5$: $$f_n = \left(0.658^{\lambda_c^2}\right) f_y \qquad \text{. . . C3.4(10)}$$

For $\lambda_c > 1.5$: $$f_n = \left[\frac{0.877}{\lambda_c^2}\right] f_y \qquad \text{. . . C3.4(11)}$$

where

f_n = critical stress which depends on the value of $\lambda_c = \sqrt{f_y/(f_{oc})_E}$

$(f_{oc})_E$ = elastic flexural buckling stress calculated using Equation C3.4(3).

Consequently, the equation for determining the nominal axial strength can be written as follows:

$$N_c = A_e f_n \qquad \text{. . . C3.4(12)}$$

which is Equation 3.4.2(2) of the Standard with f_{oc} equal to $(f_{oc})_E$.

The reasons for changing the design equations from Equation C3.4(4) to Equation C3.4(10) for inelastic buckling stress and from Equation C3.4(3) to Equation C3.4(11) for elastic buckling stress are as follows:

(1) The revised column design equations (Equations C3.4(10) and C3.4(11) were shown to be more accurate by Peköz and Sumer (1992). In this study, 299 test results on columns and beam-columns were evaluated. The test specimens included members with component elements in the post-local buckling range as well as those that were locally stable. The test specimens included members subject to flexural buckling as well as flexural-torsional buckling.

(2) Because the revised column design equations represent the maximum strength with due consideration given to initial crookedness and can provide a better fit to test results, the required factor of safety can be reduced. In addition, the revised equations enable the use of a single capacity factor for all λ_c values even though the nominal member capacity of columns decreases as the slenderness increases because of initial out-of-straightness.