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# Recommendations of aisc tall building study committee, August 1980

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Tall Buildings and Urban Habitat

RECOMMENDATIONS OF  
AISC TALL BUILDING STUDY COMMITTEE

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ABSTRACT

KEY WORDS: AISC, Implementation, Specification, Tall Steel Buildings

This report is part of an implementation effort by the Council on Tall Buildings and Urban Habitat. The AISC Tall Building Study Committee was formed with the purpose of determining revisions to the AISC Specification based on Monograph Volume SB. The report identifies suggestions in three categories: specific Specification changes, specific Commentary changes, and general recommendations to both the Specification and the Commentary. A total of 51 recommendations are included in the report. A ballot is provided for committee members to record their opinion. Recommendations approved by the committee will be sent to the AISC Specification Committee for its consideration.

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## 1. INTRODUCTION

This report is part of an overall implementation effort by the Council on Tall Buildings and Urban Habitat, in which the latest information and research findings are collected, incorporated into the Council's Monograph, and provided to code and specification jurisdictions under which tall buildings are planned and designed. Fritz Engineering Laboratory Report No. 440.2 gives an overview of the purpose and progress of the implementation program (Beedle, 1978).

The 1978 Specification of the American Institute of Steel Construction was chosen as the first major subject of this implementation effort. In 1978, discussion between F. R. Khan, W. A. Milek, and L. S. Beedle led eventually to the formation of the AISC Tall Building Study Committee. The role of the committee is to provide recommendations to the AISC Specification Committee regarding changes in both the AISC Specification and Commentary.

The members of the committee had been active in the preparation of Monograph Volume SB (Council on Tall Buildings, 1979), with most of them serving as editors of individual chapters. Members of the committee are as follows: F. R. Khan (Chairman), L. S. Beedle, W. F. Chen, G. C. Driscoll, M. Foreman, T. V. Galambos, E. H. Gaylord, T. R. Higgins, J. S. B. Iffland, S. H. Iyengar, L. W. Lu, W. McGuire, W. A. Milek, W. H. Munse, E. P. Popov, and I. M. Viest.

With the publication of the Monograph Volume SB in June 1979, the committee was ready to begin its work. The AISC Tall Building Study Committee held its first meeting on August 29, 1979 in Chicago. It was

agreed that the Monograph should be the primary source for suggestions, but that the committee should not necessarily restrict itself to this material. This report is the result of the meeting and of the subsequent written suggestions (AISC Tall Building Study Committee, 1979). Its purpose is to compile the recommendations of the committee members. Recommendations approved by the committee would then be forwarded to the AISC Specification Committee.

The body of the report is divided into three sections: Specification recommendations, Commentary recommendations, and other suggestions to the Specification and Commentary. The first and second sections contain recommendations for specific changes, while the third section deals with suggestions which are general in nature or require additional investigation. Each section is arranged according to AISC Specification sequence, with topics not covered in the Specification appearing at the end of the third section. The suggested revisions to the Specification proper and to the Commentary are arranged with the present provision appearing at the top of the page, and the suggested revision below it. The material is presented the way it was received from the committee members, modified only to provide clarity as to the precise suggestions being made.

The appendix contains the ballot that is provided to the committee members to record their opinion.

## 2. RECOMMENDATIONS WITH REGARD TO THE SPECIFICATION

### **1.3.5 Wind**

Proper provision shall be made for stresses caused by wind, both during erection and after completion of the building.

### Suggested Specification Change (Foreman 16Aug79)

Add a second paragraph to Specification Section 1.3.5:

The deflection of the structure due to wind should be considered, and the effect of cladding on the stiffness and strength of tall buildings should also be taken into account.

## SECTION 1.7 MEMBERS AND CONNECTIONS SUBJECT TO REPEATED VARIATION OF STRESS (FATIGUE)

### 1.7.1 General

Fatigue, as used in this Specification, is defined as the damage that may result in fracture after a sufficient number of fluctuations of stress. Stress range is defined as the magnitude of these fluctuations. In the case of a stress reversal, stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the sum of maximum shearing stresses of opposite direction at a given point, resulting from differing arrangements of live load.

Few members or connections in conventional buildings need to be designed for fatigue, since most load changes in such structures occur only a small number of times or produce only minor stress fluctuations. The occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design. However, crane runways and supporting structures for machinery and equipment are often subject to fatigue loading conditions.

### Suggested Specification Change (Munse 12Sep79)

Replace Specification Section 1.7.1 - Paragraph 2 - Line 3 with the following:

The occurrences of full design wind, thermal or earthquake loadings are rare and generally need not be considered in fatigue design.



## SECTION 1.11 COMPOSITE CONSTRUCTION

### 1.11.1 Definition

Composite construction shall consist of steel beams or girders supporting a reinforced concrete slab,\* so interconnected that the beam and slab act together to resist bending.

When the slab extends on both sides of the beam, the *effective width* of the concrete flange shall be taken as not more than  $\frac{1}{4}$  the span of the beam, and its *effective projection* beyond the edge of the beam shall not be taken as more than  $\frac{1}{2}$  the clear distance to the adjacent beam, nor more than 8 times the slab thickness. When the slab is present on only one side of the beam, the *effective projection* shall be taken as not more than  $\frac{1}{12}$  of the beam span, nor 6 times its thickness, nor  $\frac{1}{2}$  the clear distance to the adjacent beam.

Beams totally encased 2 inches or more on their sides and soffit in concrete cast integrally with the slab may be assumed to be interconnected to the concrete by natural bond, without additional anchorage, provided the top of the beam is at least  $1\frac{1}{2}$  inches below the top and 2 inches above the bottom of the slab, and further provided that the encasement has adequate mesh or other reinforcing steel throughout the whole depth and across the soffit of the beam to prevent spalling of the concrete. When shear connectors are provided in accordance with Sect. 1.11.4, encasement of the beam to achieve composite action is not required.

### Suggested Specification Change (Viest 29Aug79)

#### EFFECTIVE WIDTH

Replace Specification Section 1.11.1 - Paragraph 1 - Lines 3-9 with the following:

The effective width of the concrete slab on each side of the beam centerline shall be taken as the least of (1) one-eighth of the beam span, center-to-center of supports, (2) one-half the distance to the centerline of the adjacent beam and (3) the distance to the edge of the slab.

## SECTION 2.4 COLUMNS

Members subject to combined axial load and bending moment shall be proportioned to satisfy the following interaction formulas:

$$\frac{P}{P_{cr}} + \frac{C_m M}{\left(1 - \frac{P}{P_e}\right) M_m} \leq 1.0 \quad (2.4-2)$$

$$\frac{P}{P_y} + \frac{M}{1.18 M_p} \leq 1.0; \quad M \leq M_p \quad (2.4-3)$$

Suggested Specification Change (Driscoll 14Sep79)

- A) Replace Equations (2.4-2) and (2.4-3) with the following: (Council on Tall Buildings, 1979, pp. 255-256)

$$\left(\frac{M}{M_{ux}}\right)^\beta + \left(\frac{M}{M_{uy}}\right)^\beta = 1.0 \quad (2.4-2)$$

At a braced location

$$\beta = 1.6 - \frac{\frac{P}{P_y}}{2 \ln \left(\frac{P}{P_y}\right)} \quad (2.4-3)$$

$$M_{ux} = 1.18 M_{px} \left[ 1 - \left(\frac{P}{P_y}\right) \right] \quad (2.4-4a)$$

$$M_{uy} = 1.19 M_{py} \left[ 1 - \left(\frac{P}{P_y}\right)^2 \right] \quad (2.4-4b)$$

To check stability between braced points use

$$\left( \frac{C_{mx} M_x}{M_{ux}} \right)^\beta + \left( \frac{C_{my} M_y}{M_{uy}} \right)^\beta \leq 1.0 \quad (2.4-5a)$$

$$\beta = 0.4 + \frac{P}{P_y} + \frac{B}{D} \geq 1.0 \quad \text{when } \frac{B}{D} \geq 0.3 \quad (2.4-5b)$$

$$\beta = 1.0 \quad \text{when } \frac{B}{D} < 0.3 \quad (2.4-5c)$$

$$M_{ux} = M_m \left[ 1 - \left( \frac{P}{P_u} \right) \right] \left[ 1 - \left( \frac{P}{P_{ex}} \right) \right] \quad (2.4-6a)$$

$$M_{uy} = M_{py} \left[ 1 - \left( \frac{P}{P_u} \right) \right] \left[ 1 - \left( \frac{P}{P_{ey}} \right) \right] \quad (2.4-6b)$$

$$P_u = \left[ 1 - \frac{(\ell/r)^2}{2C_c} \right] F_y A \quad (2.4-7)$$

where

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$$

when  $\ell/r$  exceeds  $C_c$

$$P_u = \frac{\pi^2 EA}{(\ell/r)^2} \quad (2.4-8)$$

in which

$P$  = applied axial load, kips

$P_y$  = axial load at full yield condition

$P_u$  = ultimate load of axially loaded column

$P_{ex}$  = Euler buckling load about x axis of bending

$P_{ey}$  = Euler buckling load about y axis of bending

- $C_{mx}$  &  $C_{my}$  are the  $C_m$  coefficient defined in Section 1.6.1
- $M_x$  = bending moment about x axis of member.
- $M_y$  = bending moment about y axis of member
- $M_m$  = maximum moment that can be resisted by the member in the absence of axial load, kip-feet
- $M_{ux}$  = maximum end moment about x axis of member, including axial load but in absence of other moment
- $M_{uy}$  = maximum end moment about y axis of member, including axial load but in absence of other moment
- $M_{px}$  = plastic moment about x axis of member, kip-feet =  $Z_x F_y$
- $Z_x$  = plastic section modulus about x axis of member, inches<sup>3</sup>
- $M_{py}$  = plastic moment about y axis of member, kip-feet =  $Z_y F_y$
- $Z_y$  = plastic section modulus about y axis of member, inches<sup>3</sup>
- $\beta$  = exponent
- B and D = cross-sectional dimensions of the column section
- $\ell/r$  = largest slenderness ratio of the column

B) Revise equation (2.4-4) as follows and re-number to (2.4-9):

For columns braced in the weak direction:

$$M_m = M_{px}$$

For columns unbraced in the weak direction:

$$M_m = \left[ 1.07 - \frac{(\ell/r_y) \sqrt{F_y}}{3160} \right] M_{px} < M_{px} \quad (2.4-9)$$

### 3. RECOMMENDATIONS WITH REGARD TO THE COMMENTARY

#### **SECTION 1.2 TYPES OF CONSTRUCTION**

In order that adequate instructions can be issued to the shop and erection forces, the basic assumptions underlying the design must be thoroughly understood by all concerned. As heretofore, these assumptions are classified under three separate but generally recognized types of construction.

For better clarity, the provisions covering tier buildings of Type 2 construction designed for wind loading were reworded in the 1969 Specification, but without change in intent. Justification for these provisions has been discussed by Disque<sup>1</sup> and others.

#### Suggested Commentary Changes

- A) (McGuire, Iffland, Beedle 29Aug79) Add to Commentary Section 1.2:

The use of Type 2 construction is a simple way of solving a complicated problem. When stiffness under lateral load is important, then the analysis should be based, not on the simple design assumptions, but on more accurate methods (Ref: ? ).

- B) (Munse 12Sep79) Add to Commentary Section 1.2:

In the design of highly restrained welded connections care must be exercised to provide adequate ductility and flexibility, particularly when large welds are used and high shrinkage stresses are expected (AISC, 1973). Lamellar tearing occasionally has been found to occur when a high degree of restraint is built into a weldment that produces large strains in the through-thickness direction of rolled steel plates or shapes. In addition, the welding process and procedures should be selected so as to reduce to a minimum the susceptibility of a weldment to lamellar tearing (Council on Tall Buildings, 1979, p. 459).

- C) (Munse 12Sep79) Add to Commentary Section 1.2:

Designers of buildings with exposed steel members should be aware of the possibility of brittle fracture in welded members subjected to tensile stresses, both during construction and after completion of the structure. Materials of increased toughness may be desirable and increased care in the selection and design of members and connections may be necessary (Council on Tall Buildings, 1979, p. 465).

### 1.3.5 Wind

Proper provision shall be made for stresses caused by wind, both during erection and after completion of the building.

#### Suggested Commentary Change (Foreman 16Aug79)

Add to Commentary Section 1.3.5:

Excessive lateral deflection of the structure due to lateral loads can cause not only damage to the architectural features of the building, but also discomfort to the occupants. Therefore, when establishing drift criteria, both factors should be considered.

The type of cladding chosen will alter the response of the structure to wind loading.

**Effect of Type of Construction on Drift Limitation.** Masonry walls add to the stiffness of a structure; therefore an increase in the drift coefficient is allowable if the stiffening effects are not included in the deflection calculations. In the deflection index guide of Table 5.6, the deflection index decreases when a curtain wall construction is used. This is because curtain walls do not add stiffness to the structure. The value of the deflection index for the same type of building varies from  $0.0015H$  to  $0.0025H$ , depending on exposure and code wind requirements. With regard to the change of deflection index with type of construction, it is felt that while an increase in the drift limitations may be allowed in the case of a masonry building, the allowable values for curtain wall construction should not differ from those accepted in normal engineering practice.

Table 5.6 Interstory deflection-index guide

Type of Building (1)	Type of Construction		Exposure (4)	Code Wind Require- ments (5)	Deflection Index (6)
	Walls (2)	Floors (3)			
Office building	curtain wall	steel, protective ceiling	minimum	moderate	0.0025
Hotel building	masonry	concrete	maximum	extreme	0.0025
Office building	masonry	steel, concrete encased	average	moderate	0.003
Residence	masonry	concrete	minimum	extreme	0.0025
Office building	curtain wall	steel, sprayed fire proofing	maximum	minimum	0.0015

**Drift Criteria and Type of Occupancy.** The deflection-index guide (Table 5.6) distinguishes office buildings from hotels and residential buildings. For apartment buildings and hotels the allowable drift coefficient is less than for commercial or office buildings, because in the former types of building, movement and noise tend to be more disturbing (Eligator and Nasetts, 1968).

**Damage to Architectural Features.** Traditionally the primary concern of an engineer in designing a building was to provide an adequate system to support vertical loading. Then, as the use of light curtain walls, dry-wall partitions, and high-strength concrete and steel has increased, the effect of wind loads has become more significant. Horizontal movement of a building due to wind loads may result in cracking of the internal partitions and the external cladding of a structure. By limiting the deflection, that is, by setting a proper drift criterion, the engineer can design structures in which these architectural damages are minimized.

Cracking of nonstructural elements such as partitions and windows, in addition to being unsightly, may cause serious maintenance problems (loss of acoustic properties, leakage, etc.). Navy (1968) gives concise information and a good bibliography on structural cracking of concrete.

Since the tolerance to large deformations of structural materials (for example, concrete), all types of nonbearing partitions, exterior curtain walls, window units, etc., is highly indeterminate, setting a drift limitation alone might not be sufficient. Therefore, when a considerable amount of drift is expected, the structural engineer has to approach the problem of architectural integrity from a different angle. Proper detailing in design, allowing for lateral motion, can alleviate many of these damages. According to ACI Committee 442 (1971), for cases where extensive drift is expected, floating partitions with a capability of relative movement between skeleton and partition may be required. Johnston (1973) states that the exterior wall panels and window glazing should be mounted with sufficient clearance to allow for the design deflection. (Council on Tall buildings, 1979, pp. 353-354).

Section 5.8.3 of the Monograph, Volume SB, covers proposed criteria for human comfort (Council on Tall Buildings, 1979, pp. 391-394).

### SECTION 1.7 MEMBERS AND CONNECTIONS SUBJECT TO REPEATED VARIATION OF STRESS (FATIGUE)

Because most members in building frames are not subject to a large enough number of cycles of full design stress application to require design for fatigue, the provisions covering such designs have been placed in Appendix B.

When fatigue is a design consideration, its severity is most significantly affected by the number of load applications, the magnitude of the stress range, and the severity of the stress concentrations associated with the particular details. These factors are not encountered in normal building designs; however, when encountered and when fatigue is of concern, all provisions of Appendix B must be satisfied.

Members or connections subject to less than 20,000 cycles of loading will not involve a fatigue condition, except in the case of repeated loading involving large ranges of stress. For such conditions, the admissible range of stress can conservatively be taken as  $1\frac{1}{2}$  times the applicable value given in Table B3 for Loading Condition 1.

#### Suggested Commentary Change (Munse 12Sep79)

Replace Commentary Section 1.7 - Paragraph 3 with the following:

Members or connections subject to less than 20,000 cycles of loading will not involve a fatigue condition, except in the case of repeated loadings involving large ranges of stress. In general, for such conditions, the admissible range of stress can conservatively be taken as  $1\frac{1}{2}$  times the applicable value given in Table B3 for loading condition 1. However, under severe earthquake loadings special low-cycle fatigue consideration may be necessary. In addition, connections and details subjected to low-cycle fatigue must be scrutinized also with regard to the possibility of brittle fracture.

If relatively high stress ranges can be expected to occur frequently in details of low fatigue resistance as a result of wind loading and other climatic conditions, consideration should be given in design to the magnitudes of the stress ranges and the loading history expected during the projected life of the structure. In particular, the fastenings for building cladding should be examined for such loadings (Council on Tall Buildings, 1979, pp. 466-467, 471, 476).



## SECTION 1.11 COMPOSITE CONSTRUCTION

### 1.11.1 Definition

When the dimensions of a concrete slab supported on steel beams are such that the slab can effectively serve as the flange of a composite T-beam, and the concrete and steel are adequately tied together so as to act as a unit, the beam can be proportioned on the assumption of composite action.

Two cases are recognized: fully encased steel beams, which depend upon natural bond for interaction with the concrete, and those with mechanical anchorage to the slab (shear connectors), which do not have to be encased.

For composite beams with formed steel deck, studies<sup>36,37</sup> have demonstrated that the total slab thickness, including ribs, can be used in determining effective slab width.

Suggested Commentary Change (Viest 29Aug79)

### EFFECTIVE WIDTH

Replace Commentary Section 1.11.1 - Paragraph 3 with the following:

Tentative LRFD design criteria for effective width omit any limit based on slab thickness, in accord with both theoretical and experimental studies as well as current composite beam codes in other countries (Hansell et al., 1978). The same effective width rules apply to composite beams with a slab on either one side or both sides of the beam. To simplify design, effective width is based on the full span, center-to-center of supports, for both simple and continuous beams.

## SECTION 1.15 CONNECTIONS

Suggested Commentary Change (Chen 29Aug79)

STIFFNESS OF HEAVY BOLTED CONNECTIONS

Add Section 1.15.A to the Commentary:

For a structure that might be sensitive to end rotations, the slip of bolted flange plate connections reduces their stiffness.

In contrast to the behavior of moment connections with beam flanges welded to the column, moment connections with fasteners designed for bearing exhibit a slip characteristic that results in a reduction of stiffness at loads less than the plastic limit load of the beam (Standig et al., 1976). There are three distinct segments in a typical load deflection curve (Fig.1). The deflection resulting from slip of bearing bolted moment connections may be an additional factor to be considered in the analysis of the stability of frames.

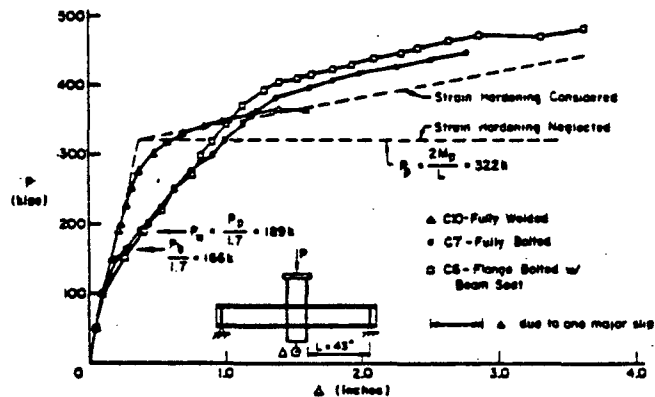


Fig. 1 Load-deflection curves

## SECTION 2.1 SCOPE

The Specification recognizes three categories of profiles, classified according to the ability to resist local buckling of elements of the cross section when subject to compressive stress. These categories are: (1) non-compact, (2) compact, and (3) plastic design. The elements of *non-compact* sections (Sect. 1.9) will not buckle locally when subject to elastic limit strains. Elements of *compact* sections (Sect. 1.5.1.4.1) are proportioned so that the cross section may be strained in bending to the degree necessary to achieve full plastification of the cross section; however, the reserve for inelastic strains is adequate only to achieve modest redistribution of moments. The elements of *plastic design* sections (Sect. 2.7) are proportioned so that they will not only achieve full plastification of the cross section, but will remain stable while being bent through an appreciable angle at a constant plastic moment up to the point where strain hardening is initiated. Thus, plastic design cross sections are capable of providing the hinge rotations that are counted upon in the plastic method of analysis.

The superior bending strength of compact sections is recognized in Part 1 of the Specification by increasing the allowable bending stress to  $0.66F_y$  and by permitting 10% redistribution of moment. By the same token, the logical load factor for plastically designed beams is given by the equation

$$F = \frac{F_y}{0.66F_y} \times (\text{shape factor})$$

For such shapes listed in the AISC *Steel Construction Manual*, the variation of shape factor is from 1.10 to 1.23, with a mode of 1.12. Then, the corresponding load factor must vary from 1.67 to 1.86, with a mode of 1.70. Such a load factor is consistent and in better balance with that inherent in the allowable working stresses for tension members and deep plate girders.

Research<sup>56</sup> on the ultimate strength of heavily loaded columns subjected to concurrent bending moments has provided data which justifies a load factor, for such members, that is the same as that provided for members subject to bending only, namely 1.7. Consistent with the  $\frac{1}{3}$  increase in allowable stress permitted in Part 1 of the Specification, the load factor to be used in designing for gravity loading combined with wind or seismic loading is 1.3.

Based on continuing research at Lehigh University on multistory framing,<sup>57,58</sup> application of the Specification provisions includes the complete design of braced and unbraced planar frames in high-rise buildings. Systematic procedures for application of plastic design in proportioning the members of such frames have been developed<sup>59,60</sup> and are available in the current literature.

### Suggested Commentary Changes (Popov 31Aug79)

- A) Add a new paragraph to Commentary Section 2.1:

Plastic methods of analysis are now well developed and include both gravity and lateral force analyses. Refer to Chapter SB-3 of the Monograph (Council on Tall Buildings, 1979).

- B) Add to Commentary Section 2.1:

It is now well documented that ductile behavior of structural systems significantly reduces the force magnitudes that develop during a strong earthquake (Newmark and Hall, 1976). Vibration of a structure behaving in a ductile manner is moderated in a manner somewhat analogous to that of viscous damping of elastic systems. Properly designed conventional moment-resisting framing using structural steel possesses these desirable characteristics. Some new framing schemes (Roeder and Popov, 1978; Popov and Roeder, 1978) attempt to combine the ductility of a moment-resisting frame with the stiffness of a diagonally braced frame.

## SECTION 2.4 COLUMNS

Formulas (2.4-2) and (2.4-3)<sup>1</sup> will be recognized as similar in type to Formulas (1.6-1a) and (1.6-1b) in Part 1, except that they are written in terms of factored loads and moments, instead of allowable stresses at service loading. As in the case of Formulas (1.6-1a) and (1.6-1b),  $P_{cr}$  is computed on the basis of  $l/r_x$  or  $l/r_y$ , whichever is larger, for any given unbraced length.<sup>1</sup>

A column is considered to be fully braced if the slenderness ratio  $l/r_y$  between the braced points is less than or equal to that specified in Sect. 2.9. When the unbraced length ratio of a member bent about its strong axis exceeds the limit specified in Sect. 2.9, the rotation capacity of the member may be impaired, due to the combined influence of lateral and torsional deformation, to such an extent that plastic hinge action within the member cannot be counted upon. However, if the computed value of  $M$  is small enough so that the limitations of Formulas (2.4-2) and (2.4-3) are met, the member will be strong enough to function at a joint where the required hinge action is provided in another member entering the joint. An assumed reduction in moment-resisting capacity is provided by using the value  $M_m$ , computed from Formula (2.4-4), in Formula (2.4-2).

Formula (2.4-4) was developed empirically\* on the basis of test observations and provides an estimate of the critical lateral buckling moment, in the absence of axial load, for the case where  $M_1/M_2 = -1.0$  (single curvature bending). For other values of  $M_1/M_2$ , adjustment is provided by using the appropriate  $C_m$  value as defined in Sect. 1.6.1.

Formula (2.4-4) is to be used only in connection with Formula (2.4-2).

Space frames containing plastically designed planar rigid frames are assumed to be supported against sidesway normal to these frames. Depending upon other conditions of restraint, the basis for determination of proper values for  $P_{cr}$  and  $P_e$  and  $M_m$ , for a plastically designed column oriented to resist bending about its strong axis, is outlined in Table C2.4.1. In each case  $l$  is the distance between points of lateral support corresponding to  $r_x$  or  $r_y$ , as applicable. When  $K$  is indicated, its value is governed by the provisions of Sect. 1.8.3 of the Specification.

TABLE C2.4.1

	Braced Planar Frames	One- and Two-Story Unbraced Planar Frames
$P_{cr}$	Use larger ratio, $\frac{l}{r_y}$ or $\frac{l}{r_x}$	<sup>1</sup> Use larger ratio, $\frac{l}{r_y}$ or $\frac{Kl}{r_x}$
$P_e$	Use $l/r_x$	<sup>1</sup> Use $Kl/r_x$
$M_m$	Use $l/r_y$	Use $l/r_y$
<sup>1</sup> Webs of columns assumed to be in plane of frame.		

Suggested Commentary Change (Driscoll 14Sep79)

## SECTION 2.4 COLUMNS

Prior editions of this specification used column formulas limiting bending to one axis and similar in type to Formulas (1.6-1a) and (1.6-1b) in Part 1, except that they are written in terms of factored loads and moments, instead of allowable stresses at service loading.

Recently, extensive theoretical studies of the behavior of steel H-columns subject to compression combined with biaxial bending have been made, using computer models (Chen and Atsuta, 1973, 1977; Santathadaporn and Chen, 1973). As a result of these studies, direct and accurate approximate formulas have been proposed as a method for design. Herein are reviewed the existing design requirements, along with the recently proposed new design procedures for biaxially loaded beam-columns.

An examination of Fig. C2.4.1 clarifies many of the premises of the present design concept. It represents, in two dimensions, what is essentially a three-dimensional surface describing the maximum strength of columns subject to axial load and biaxial bending moments. It shows a typical maximum strength interaction surface for a particular beam-column length.

If the solid lines on the mutually perpendicular planes of Fig. C2.4.1 represent the actual failure curves under the relevant restricted loading conditions, then the dotted lines represent the existing design requirements. In particular, the straight-line interaction of biaxial moment for a given axial load corresponds to the current AISC design expressions (AISC, 1969), as well as CRC Eq. 6.19 of the second edition of the CRC Guide to Design Criteria for Metal Compression Members (Johnston, 1966) and to SSRC Eq. 8.29 of the Guide to Stability Design Criteria for Metal Structures (Structural Stability Research Council, 1976). Recent research has shown that the interaction of moments about the orthogonal axes is not linear (Tebedge and Chen, 1974). On the contrary, the interaction curve resembles more closely the quadrant of a circle (see Fig. C2.4.2). It is important to note that if a member is fully loaded under axial load and bending about one axis, then there is no spare capacity to accept moment about the other axis. However, as the loading decreases slightly below the maximum, capacity rapidly develops to accept bending about the other axis.

Extensive comparisons have also been made with the results of tests on actual columns, providing final confirmation of the validity of the interaction formulas (Springfield and Hegan, 1973). Springfield's evaluation of Chen's interaction equation (Eq. 2.4-2) showed that, for Birnstiel's tests, Eq. 2.4-2 was extremely reliable while still conservative [Mean 1.01, SD (Standard Deviation) 0.074]. A further verification of Chen's equation is its good agreement with Birnstiel's incremental analytical procedure (Birnstiel and Michalos, 1963). Aside from one result, in which the error was 7% conservative, all the other values agree to within 3%. (Council on Tall Buildings, 1979, pp. 254-257).

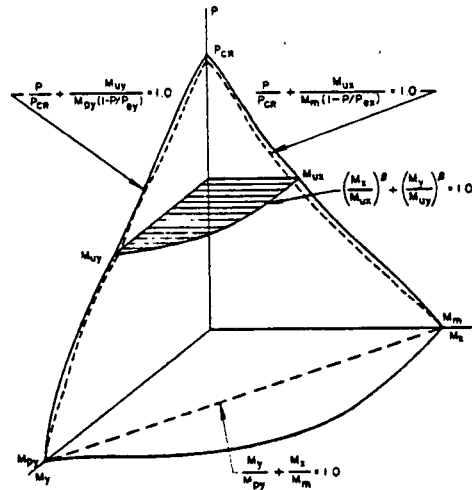


Fig. C2.4.1 Typical maximum strength interaction surface for particular column of known length subject to biaxial bending

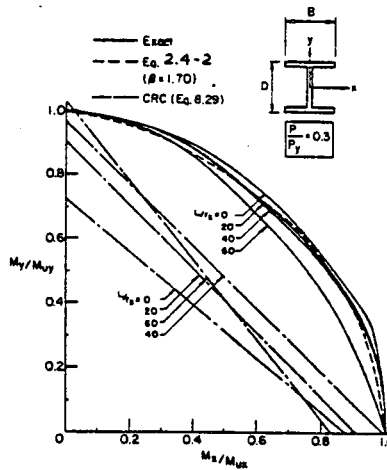


Fig. C2.4.2 Comparison of interaction curves for long columns

A column is considered to be fully braced if the slenderness ratio  $l/r_y$  between the braced points is less than or equal to that specified in Sect. 2.9. When the unbraced length ratio of a member bent about its strong axis exceeds the limit specified in Sect. 2.9, the rotation capacity of the member may be impaired, due to the combined influence of lateral and torsional deformation, to such an extent that plastic hinge action within the member cannot be counted upon. However, if the computed value of  $M$  is small enough so that the limitations of Formula 2.4-2 are met, the member will be strong enough to function at a joint where the required hinge action is provided in another member entering the joint. An assumed reduction in moment-resisting capacity is provided by using the value  $M_m$ , computed from Formula 2.4-9, in Formula (2.4-2).

Formula (2.4-9) was developed empirically on the basis of test observations and provides an estimate of the critical lateral buckling moment, in the absence of axial load, for the case where  $M_1/M_2 = -1.0$  (single curvature bending).

Formula (2.4-9) is to be used only in connection with Formula (2.4-2).

Space frames containing plastically designed planar rigid frames may be braced to be supported against sidesway normal to these frames. Depending upon other conditions of restraint, the basis for determination of proper values for  $P_{cr}$  and  $P_e$  and  $M_m$ , for a plastically designed column oriented to resist bending about its strong axis, is outlined in Table C2.4.1. In each case  $l$  is the distance between points of lateral support corresponding to  $r_x$  or  $r_y$ , as applicable. When  $K$  is indicated, its value is governed by the provisions of Sect. 1.8.3 of the Specification.

TABLE C2.4.1

	Braced Space Frames	Unbraced Space Frames
$P_{cr}$	Use larger ratio $l/r_y$ or $l/r_x$	<sup>2</sup> Use larger ratio $l/r_y$ or $l/r_x$
$P_{ex}$	Use $l/r_x$	<sup>2</sup> Use $l/r_x$
$P_{ey}$	Use $l/r_y$	<sup>2</sup> Use $l/r_y$
$M_m$	Use $l/r_y$	Use $l/r_y$

<sup>2</sup>A frame analysis considering  $P$ -delta effects should be used in determining member forces.

4. OTHER SUGGESTIONS COVERING THE SPECIFICATION AND COMMENTARY**SECTION 1.2 TYPES OF CONSTRUCTION**

In order that adequate instructions can be issued to the shop and erection forces, the basic assumptions underlying the design must be thoroughly understood by all concerned. As heretofore, these assumptions are classified under three separate but generally recognized types of construction.

For better clarity, the provisions covering tier buildings of Type 2 construction designed for wind loading were reworded in the 1969 Specification, but without change in intent. Justification for these provisions has been discussed by Disque<sup>1</sup> and others.

Suggestion: (1.2 Commentary, McGuire 4Sep79):

An attempt should be made to explain the background of Type 2 construction and perhaps to set limits on its use.

Background Information: To me the definition of Type 2 construction presents a dilemma. On the one hand it is useful in that it legitimizes an old practice that has been found to yield economical, satisfactory results for many ordinary structures. On the other hand, it is patently irrational, and would seem to have little place in a modern specification that is attempting to place design on a rational basis. Further, there are no limits on its application. Presumably, Type 2 construction could be used for a building of any height and slenderness. I doubt that the intention is to permit it to be applied in the design of all modern tall buildings.

Because of its usefulness, I would not suggest the deletion of "Type 2 Construction" at this time. I am suggesting that an attempt be made to explain its background and perhaps to set limits on its use. The place for this is probably in the Commentary (McGuire, 1977).



### 1.3.5 Wind

Proper provision shall be made for stresses caused by wind, both during erection and after completion of the building.

A) Suggestion: (1.3.5 Specification, Gaylord 29Aug79):

Provide more adequate information for wind load in design.

Background Information: Is the information in this section adequate? It would appear that the commentary on page 103 gives the opposite approach.

B) Suggestion: (1.3.5 Specification, Khan, Viest 29Aug79):

Perhaps a committee should be set up to provide a statement and assist the designer to avoid falling into a trap, since the overall behavior of the building has often not been considered by designers.

C) Suggestion: (1.3.5 Commentary, Popov, Galambos 29Aug79):

Continuing commentary could refer to ANSI and other such groups (Popov). Also the specific limits of usefulness should be indicated. This is hard to define for Allowable-stress design (Galambos).

### 1.3.6 Other Forces

Structures in localities subject to earthquakes, hurricanes and other extraordinary conditions shall be designed with due regard for such conditions.

Suggestion: (1.3.6 Specification, Gaylord 29Aug79):

Add information on the fundamental period of lateral vibration (ATC-3, 1978, pp. 56, 372-374).

### 1.5.1.3 Compression

1.5.1.3.1 On the gross section of axially loaded compression members whose cross sections meet the provisions of Sect. 1.9, when  $Kl/r$ , the largest effective slenderness ratio of any unbraced segment as defined in Sect. 1.8, is less than  $C_c$ :

$$F_a = \frac{\left[1 - \frac{(Kl/r)^2}{2C_c^2}\right] F_y}{\frac{5}{3} + \frac{3(Kl/r)}{8C_c} - \frac{(Kl/r)^3}{8C_c^3}} \quad (1.5-1)$$

where

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$$

1.5.1.3.2 On the gross section of axially loaded compression members, when  $Kl/r$  exceeds  $C_c$ :

$$F_a = \frac{12\pi^2 E}{23(Kl/r)^2} \quad (1.5-2)$$

Suggestion: (1.5.1.3 Specification, Iffland 7Sep79):

#### MINOR AXIS BENDING, SPECIAL SECTIONS AND STEELS

Use multiple column curves.

Background Information: The column formulas in the Specification are applicable for hot-rolled shapes of moderate size subjected to bending about the major axis. They give results ranging from overconservative to underconservative for other types of columns, for columns of different steels and for bending about the minor axis. Recommendations for solving the minor axis bending have been made (Lu) but these do not solve the problem of heavy shapes, built-up columns, special cross sections, etc.

The verification of all the categories of columns by testing vertical columns is not considered important or necessary. Computer simulation of failure loads is adequate for classification of the various column types under different axis bending. The multiple column curves could be based on either the tangent modulus load or on the maximum strength. The latter procedure is recommended.

### 1.5.1.3 Compression

1.5.1.3.1 On the gross section of axially loaded compression members whose cross sections meet the provisions of Sect. 1.9, when  $Kl/r$ , the largest effective slenderness ratio of any unbraced segment as defined in Sect. 1.8, is less than  $C_c$ :

$$F_a = \frac{\left[1 - \frac{(Kl/r)^2}{2C_c^2}\right] F_y}{\frac{5}{3} + \frac{3(Kl/r)}{8C_c} - \frac{(Kl/r)^3}{8C_c^3}} \quad (1.5-1)$$

where

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$$

1.5.1.3.2 On the gross section of axially loaded compression members, when  $Kl/r$  exceeds  $C_c$ :

$$F_a = \frac{12\pi^2 E}{23(Kl/r)^2} \quad (1.5-2)$$

1.5.1.3.3 On the gross section of axially loaded bracing and secondary members, when  $l/r$  exceeds 120:\*\*

$$F_{as} = \frac{F_a \text{ [by Formula (1.5-1) or (1.5-2)]}}{1.6 - \frac{l}{200r}} \quad (1.5-3)$$

1.5.1.3.4 On the gross area of plate girder stiffeners:

$$F_a = 0.60F_y$$

1.5.1.3.5 On the web of rolled shapes at the toe of the fillet (crippling, see Sect. 1.10.10):

$$F_a = 0.75F_y$$

Suggestion: (1.5.1.3 Specification, Iffland 7Sep79):

#### END RESTRAINT AND INITIAL COLUMN CROOKEDNESS

K for braced frames (perhaps with a change in terminology) should be left in the column formulas to account for end restraint.

Background Information: Initial studies have indicated that for individual columns the effects of end restraints and initial column crookedness tend to cancel each other out (Galambos). Their influence in the column formulas can be included by use of a term in the column formulas the same way K for braced frames is presently included. The use of the concept of K for braced frames seems to give good results in accounting for end restraint. This suggestion should be subject to supporting verifications by research (the problem is currently being studied by SSRC TG 23).

### 1.5.1.4 Bending

#### 2. Compression:

- a. For members meeting the requirements of Sect. 1.9.1.2, having an axis of symmetry in, and loaded in, the plane of their web, and compression on extreme fibers of channels bent about their major axis:

The larger value computed by Formulas (1.5-6a) or (1.5-6b) and (1.5-7), as applicable\* (unless a higher value can be justified on the basis of a more precise analysis\*\*), but not more than  $0.60F_y$ .†

$$\text{When } \sqrt{\frac{102 \times 10^3 C_b}{F_y}} \leq \frac{l}{r_T} \leq \sqrt{\frac{510 \times 10^3 C_b}{F_y}}$$

$$F_b = \left[ \frac{2}{3} - \frac{F_y (l/r_T)^2}{1530 \times 10^3 C_b} \right] F_y \quad (1.5-6a)$$

$$\text{When } l/r_T \geq \sqrt{\frac{510 \times 10^3 C_b}{F_y}}$$

$$F_b = \frac{170 \times 10^3 C_b}{(l/r_T)^2} \quad (1.5-6b)$$

Or, when the compression flange is solid and approximately rectangular in cross section and its area is not less than that of the tension flange:

$$F_b = \frac{12 \times 10^3 C_b}{ld/A_f} \quad (1.5-7)$$

#### Suggestion: (1.5.1.4 Specification, McGuire 4Sep79):

Consideration should be given to a lateral buckling provision such as that appearing in the Australian Code, AS 1250 (Standards Association of Australia, 1975).

Background Information: Section 5.4.3 of AS 1250 (copy enclosed) is appealing in its approach, in that it gives one a mechanism for using an elastic flexural-torsional buckling solution in design if one chooses to do so. For those who do not so choose, a simple formula for  $F_{ob}$  is also given in the Specification (not enclosed). I would think that, in most cases, the simple formula would be used. But for many of the more complicated loading and boundary conditions that often arise, a designer could find and use a corresponding elastic solution for  $F_{ob}$ .

I haven't made a systematic comparison of the numerical results to be obtained under AISC and AS 1250. In one example I found similar results (copy enclosed). If it is decided to pursue this suggestion, I would recommend that a small research project be set up in which a Master's student could make such a systematic comparison. If the outcome of the study is that the Australian formulas yield reasonable results in standard cases, then serious consideration should be given to incorporating them in the AISC Specification.

AS 1250—1975

**5.4.3 Other Sections.** The maximum calculated stress due to bending in a beam not otherwise covered by Rules 5.4.1 and 5.4.2 shall not exceed the maximum permissible stress  $F_b$ , determined by formula 5.4.3(1) or 5.4.3(2), as appropriate:

Where  $F_{ob}$  is equal or less than  $F_Y$  —

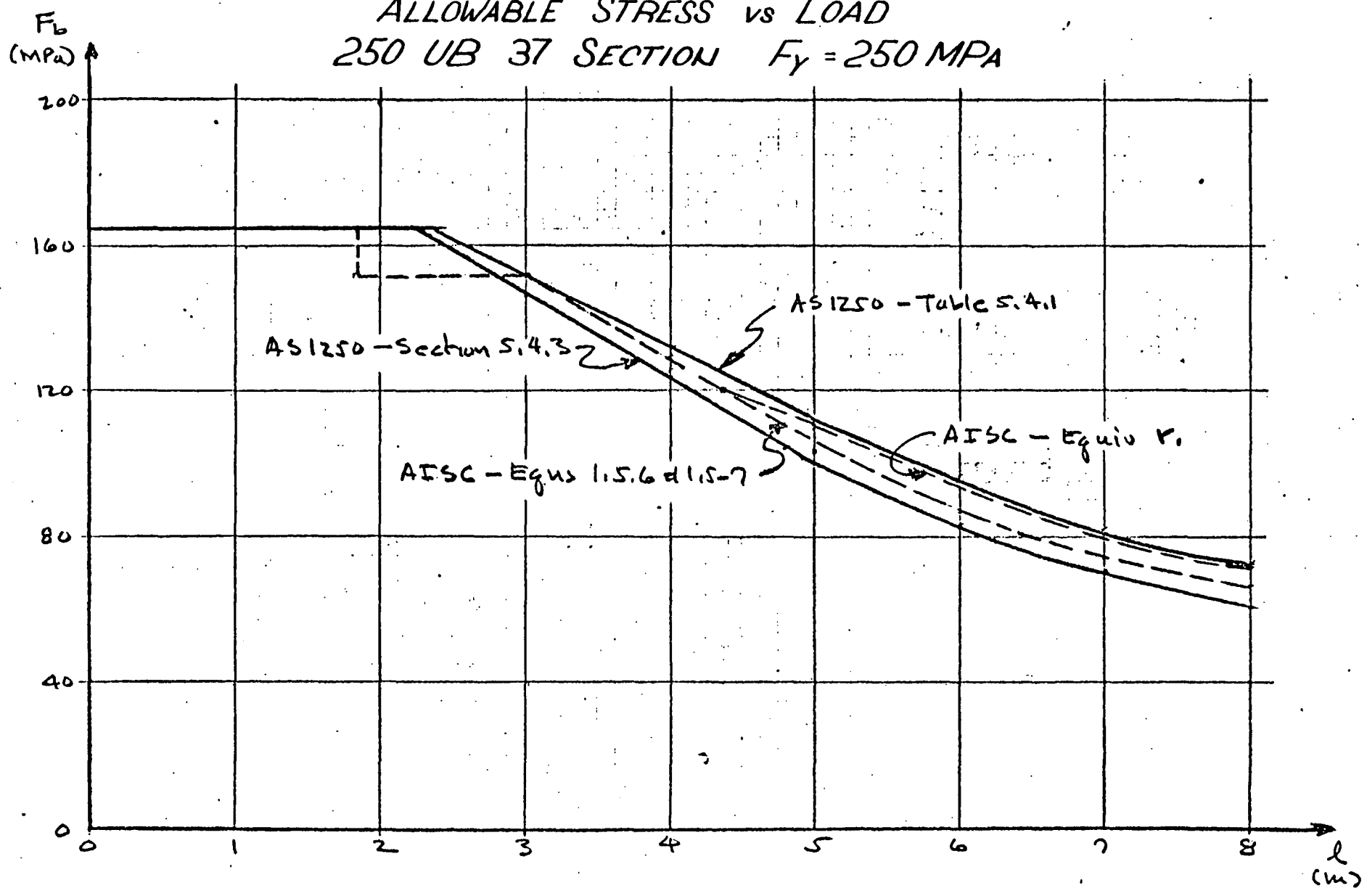
$$F_b = \left[ 0.55 - 0.10 \frac{F_{ob}}{F_Y} \right] F_{ob} \quad \dots \quad \dots \quad \dots \quad \dots \quad 5.4.3(1)$$

Where  $F_{ob}$  is equal to or greater than  $F_Y$  —

$$F_b = \left[ 0.95 - 0.50 \sqrt{\left( \frac{F_Y}{F_{ob}} \right)} \right] F_Y \quad \dots \quad \dots \quad \dots \quad \dots \quad 5.4.3(2)$$

In formulas 5.4.3(1) and 5.4.3(2) above, the maximum stress  $F_{ob}$  in the beam at elastic buckling, shall be calculated in accordance with Rule 5.5 or by an elastic flexural-torsional buckling analysis.

# ALLOWABLE STRESS vs LOAD 250 UB 37 SECTION $F_y = 250$ MPA



## 2. Compression:

- a. For members meeting the requirements of Sect. 1.9.1.2, having an axis of symmetry in, and loaded in, the plane of their web, and compression on extreme fibers of channels bent about their major axis:

The larger value computed by Formulas (1.5-6a) or (1.5-6b) and (1.5-7), as applicable\* (unless a higher value can be justified on the basis of a more precise analysis\*\*), but not more than  $0.60F_y$ .†

$$\text{When } \sqrt{\frac{102 \times 10^3 C_b}{F_y}} \leq \frac{l}{r_T} \leq \sqrt{\frac{510 \times 10^3 C_b}{F_y}}$$

$$F_b = \left[ \frac{2}{3} - \frac{F_y (l/r_T)^2}{1530 \times 10^3 C_b} \right] F_y \quad (1.5-6a)$$

$$\text{When } l/r_T \geq \sqrt{\frac{510 \times 10^3 C_b}{F_y}}$$

$$F_b = \frac{170 \times 10^3 C_b}{(l/r_T)^2} \quad (1.5-6b)$$

Or, when the compression flange is solid and approximately rectangular in cross section and its area is not less than that of the tension flange:

$$F_b = \frac{12 \times 10^3 C_b}{ld/A_f} \quad (1.5-7)$$

## SECTION 1.6 COMBINED STRESSES

## 1.6.1 Axial Compression and Bending

Members subjected to both axial compression and bending stresses shall be proportioned to satisfy the following requirements:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \leq 1.0 \quad (1.6-1a)$$

$$\frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \quad (1.6-1b)$$

When  $f_a/F_a \leq 0.15$ , Formula (1.6-2) may be used in lieu of Formulas (1.6-1a) and (1.6-1b):

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \quad (1.6-2)$$

In Formulas (1.6-1a), (1.6-1b), and (1.6-2), the subscripts  $x$  and  $y$ , combined with subscripts  $b$ ,  $m$ , and  $e$ , indicate the axis of bending about which a particular stress or design property applies, and

Suggestion: (1.6 Commentary, McGuire 4Sep79):

Add an exclusion such as the following either to the Commentary or to the definition of  $F_b$  on page 25: "Equations 1.5-6a, 1.5-6b, 1.5-7 need not be applied in determining  $F_{bx}$  and  $F_{by}$  for use in Equation 1.6-1b."

Background Information: In applying Equation 1.6-1b, is it intended that the lateral buckling equations (1.5-6a, 1.5-6b, 1.5-7) be applied in calculating  $F_{bx}$  or  $F_{by}$ ? If so, why should it be since 1.6-1b is ostensibly a check on maximum stress at a cross section and not a stability check (see Commentary page 116)? If there is a reason for using the lateral buckling formulas in Equation 1.6-1b it should be presented in the Commentary.



## SECTION 1.6 COMBINED STRESSES

### 1.6.1 Axial Compression and Bending

Members subjected to both axial compression and bending stresses shall be proportioned to satisfy the following requirements:

$$\frac{f_a}{F_a} + \frac{C_{mx}f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my}f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \leq 1.0 \quad (1.6-1a)$$

$$\frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \quad (1.6-1b)$$

When  $f_a/F_a \leq 0.15$ , Formula (1.6-2) may be used in lieu of Formulas (1.6-1a) and (1.6-1b):

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \quad (1.6-2)$$

In Formulas (1.6-1a), (1.6-1b), and (1.6-2), the subscripts  $x$  and  $y$ , combined with subscripts  $b$ ,  $m$ , and  $e$ , indicate the axis of bending about which a particular stress or design property applies, and

$F_a$  = axial compressive stress that would be permitted if axial force alone existed, kips per square inch

$F_b$  = compressive bending stress that would be permitted if bending moment alone existed, kips per square inch

$$F'_e = \frac{12 \pi^2 E}{23(Kl_b/r_b)^2}$$

= Euler stress divided by a factor of safety, kips per square inch. (In the expression for  $F'_e$ ,  $l_b$  is the actual unbraced length *in the plane of bending* and  $r_b$  is the corresponding radius of gyration.  $K$  is the effective length factor *in the plane of bending*. As in the case of  $F_a$ ,  $F_b$ , and  $0.60F_y$ ,  $F'_e$  may be increased  $\frac{1}{3}$  in accordance with Sect. 1.5.6)

$f_a$  = computed axial stress, kips per square inch

$f_b$  = computed compressive bending stress at the point under consideration, kips per square inch

$C_m$  = a coefficient whose value shall be taken as follows:

1. For compression members in frames subject to joint translation (sidesway),  $C_m = 0.85$ .
2. For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending,

$$C_m = 0.6 - 0.4 \frac{M_1}{M_2}, \quad \text{but not less than } 0.4$$

where  $M_1/M_2$  is the ratio of the smaller to larger moments at the ends of that portion of the member unbraced in the plane of bending under consideration.  $M_1/M_2$  is positive when the member is bent in reverse curvature, negative when bent in single curvature.

3. For compression members in frames braced against joint translation in the plane of loading and subjected to transverse loading between their supports, the value of  $C_m$  may be determined by rational analysis. However, in lieu of such analysis, the following values may be used:

- a. For members whose ends are restrained . . .  $C_m = 0.85$
- b. For members whose ends are unrestrained . . .  $C_m = 1.0$

Suggestion: (1.6 Specification, Iffland 7Sep79):

BIAXIAL BENDING

The Specification should either utilize non-linear biaxial bending formulas directly or permit their use by appropriate reference to additional details given in the Commentary.

Background Information: The column formulas given in the Specification could be overconservative for columns subjected to biaxial bending. Non-linear column formulas have been developed (Chen) that eliminate this overconservativeness.

## SECTION 1.8 STABILITY AND SLENDERNESS RATIOS

### 1.8.1 General

General stability shall be provided for the structure as a whole and for each compression element. Design consideration should be given to significant load effects resulting from the deflected shape of the structure or of individual elements of the lateral load resisting system, including the effects on beams, columns, bracing, connections, and shear walls.

In determining the slenderness ratio of an axially loaded compression member, except as provided in Sect. 1.5.1.3.3, the length shall be taken as its effective length  $Kl$  and  $r$  as the corresponding radius of gyration.

Suggestion: (1.8 Specification, McGuire 4Sep79):

Appoint a task committee with the general charge of looking into nonlinear computerized analysis/design methods, encouraging their development and use, and planning for their incorporation in future editions of the AISC Specification.

Background Information: Section 1.8.1 of the 1968 AISC Specification is the first real AISC specification reference to the specific consideration of second order effects in design. I believe that the desirability of nonlinear analyses will become increasingly apparent, both in tall buildings and in low, horizontally flexible structures, the use of which seems to be increasing. Further, I think that the design profession will become more receptive to them as computerized methods improve, become more practical, and are more widely understood.

I note, incidentally, that the 1978 ECCS Recommendations for Steel Construction place somewhat more emphasis on 2nd order calculations than the AISC does. Admittedly, they are still equivocal in that they combine "2nd order verifications" with "1st order theory calculations" (see enclosed ECCS Section R1.2 and accompanying comments).

Presumably, our committee is looking at things that may be considered for inclusion in the AISC Specification several years from now. Some of the current "P $\Delta$  methods" are of immediate use of course. However, I think of them more as part of a trend, and not the final answer in themselves. I have in mind an AISC sponsored task committee that could promote, influence, and guide these developments in the interest of improved analysis/design methods for steel buildings.

## R 1.2 LIMIT STATES

There are two categories of limit states:

- the ultimate limit states,
- the serviceability limit states.

### R 1.2.1 ULTIMATE LIMIT STATES

These limit states, which correspond to the maximum load carrying capacity, should be checked either by an elastic method of analysis or by the so called "plastic design" methods of calculation.

In both cases, the limit states can be reached due to:

- loss of static equilibrium of the structure considered as a rigid body,
  - elastic or inelastic instability,
- and, depending on whether there is an elastic or a plastic calculation, due to:
- attainment, even at a single point in the structure of a conventional level of stress. This conventional level of stress is given in different items of these recommendations as calculation values of the resistance, when the stresses are calculated in the elastic field,
  - transformation of the structure into a mechanism (plastic design).

When the stresses are calculated over the initial geometry of the structure (before loading), the verifications are called of the 1st order.

The verifications are called of the 2nd order when the calculated force resultants are nonlinear with respect to the displacement of the structure.

The verifications of the 1st order are accepted only if the possible errors can be judged as being negligible.

### R 1.2.2 SERVICEABILITY LIMIT STATES

The serviceability limit states, which generally consist in deformation criteria for steel structures, are assessed by codes and/or specifications, the latter being stated to cover particular cases.

For the serviceability limit states, the calculations must always be carried out in the elastic field.

## C 1.2

*The following items must be completed for certain kinds of structures and for certain types of actions.*

*Those items should be given in particular specifications, especially in case of fatigue and dynamic actions.*

### C 1.2.1

*In a general way, the verification calculations will be of the 1st order (it's the current practice).*

*The requirements concerning the buckling, the lateral buckling and the local buckling have been set up by placing the calculations in the field of the 2nd order; they lead to formulations which will be applied to verifications based on force resultants as calculated by 1st order theory.*

*The following items give the characteristic values of the strength for different states of stress.*

*The characteristic value of the strength in case of tension is or the value of the yield point guaranteed by the steel fabricator or the mean value minus two standard deviation.*

*It is admitted for elastic calculations under bending moments at ultimate limit state to take into account a partial yielding of the cross section (see R 3.2.4).*

## SECTION 1.8 STABILITY AND SLENDERNESS RATIOS

### 1.8.1 General

General stability shall be provided for the structure as a whole and for each compression element. Design consideration should be given to significant load effects resulting from the deflected shape of the structure or of individual elements of the lateral load resisting system, including the effects on beams, columns, bracing, connections, and shear walls.

In determining the slenderness ratio of an axially loaded compression member, except as provided in Sect. 1.5.1.3.3, the length shall be taken as its effective length  $Kl$  and  $r$  as the corresponding radius of gyration.

Suggestion: (1.8 Specification, Iffland 7Sep79):

#### FRAME STABILITY VS. COLUMN STABILITY

$K$ , as a measure of frame instability, should be eliminated from the column formulas and the Specification should state clearly that the formulas given are for design of individual columns. The Specification would require that frames be checked for failure against instability. Procedures for checking (or designing against) instability could be discussed in the Commentary but the responsibility for how this is accomplished should be left up to the designer since most available procedures are only selectively applicable. The Factor of Safety against frame instability should be different for frames subjected to gravity loading alone versus frames subjected to both gravity loading and transverse loading.

It is suggested that the Commentary include details on at least one specific method of handling the problem of frame stability. The P-Delta method given in Chapter SB-4 of the Monograph (Council on Tall Buildings, 1979) is an acceptable procedure, easily understood by engineers, which, by adjustment of the Factor  $\bar{F}$ , it can be made conservative without being uneconomical.

Background Information: The use of the Effective Length Factor  $K$  in the column design formulas is a procedure for considering the stability of the entire frame in the design of a single column. Actually,  $K$ , assuming it is computed accurately, only considers the buckling of an equivalent axially loaded frame. In many practical cases the magnitudes of the P-Delta Forces are more important stability considerations. Several procedures have been suggested to include both of these effects into the column formula. (Lu, LeMessurier, Cheong-Siat-May). These procedures can be criticized for two important reasons:

- (1) They unduly complicate the column formula so that the possibility of misunderstanding and misuse is magnified while at the same time they are restricted to certain difficult to define classes and types of frames.
- (2) There are many other factors that could influence the stability of a structure (Birnstiel and Iffland) and the suggested procedures tacitly ignore these even though they could be critical. (e.g.: partially restrained joints, torsional failure, panel distortion).

The SSRC in T.M. 5 has stated that, while it may not be theoretically correct, it is not logical to try to solve the frame stability problem (for any conceivable configuration of frames with or without supplementary bracing, offset columns and other special conditions) by use of a formula used to design a single column.

## SECTION 1.10 PLATE GIRDERS AND ROLLED BEAMS

### 1.10.1 Proportions

Plate girders, coverplated beams, and rolled or welded beams shall in general be proportioned by the moment of inertia of the gross section. No deduction shall be made for shop or field rivet or bolt holes in either flange, except that in cases where the reduction of the area of either flange by such holes, calculated in accordance with the provisions of Sect. 1.14.2, exceeds 15 percent of the gross flange area, the excess shall be deducted.

Suggestion: (1.10 Specification, McGuire 4Sep79):

Form an ad hoc task group to review the results of recent plate girder research with the objective of seeing whether it provides any basis for improved plate girder proportioning provisions.

Background Information: So far as I know, the plate girder provisions in the present AISC Specifications have been satisfactory. They do, however, rest on research that was conducted twenty years ago. A lot has been done since then. In particular, I think of the work of Porter, Rockey, and Evans at Cardiff. I believe that significant work in this area has also been done in central Europe and Japan.

## SECTION 1.11 COMPOSITE CONSTRUCTION

### 1.11.1 Definition

Composite construction shall consist of steel beams or girders supporting a reinforced concrete slab,\* so interconnected that the beam and slab act together to resist bending. When the slab extends on both sides of the beam, the *effective width* of the concrete flange shall be taken as not more than  $\frac{1}{4}$  the span of the beam, and its *effective projection* beyond the edge of the beam shall not be taken as more than  $\frac{1}{2}$  the clear distance to the adjacent beam, nor more than 8 times the slab thickness. When the slab is present on only one side of the beam, the *effective projection* shall be taken as not more than  $\frac{1}{12}$  of the beam span, nor 6 times its thickness, nor  $\frac{1}{2}$  the clear distance to the adjacent beam.

Beams totally encased 2 inches or more on their sides and soffit in concrete cast integrally with the slab may be assumed to be interconnected to the concrete by natural bond, without additional anchorage, provided the top of the beam is at least  $1\frac{1}{2}$  inches below the top and 2 inches above the bottom of the slab, and further provided that the encasement has adequate mesh or other reinforcing steel throughout the whole depth and across the soffit of the beam to prevent spalling of the concrete. When shear connectors are provided in accordance with Sect. 1.11.4, encasement of the beam to achieve composite action is not required.

- A) Suggestion: (1.11 Specification, Viest 29Aug79):

Insert material on Concrete-Encased Steel Columns (Council on Tall Buildings, 1979, pp. 655-671; Task Group 20, SSRC, 1979).

- B) Suggestion: (1.11 Specification, Viest 29Aug79):

Insert material on Concrete-Filled Tubular Columns (Council on Tall Buildings, 1979, pp. 671-680; Task Group 20, SSRC, 1979).

### 1.11.2 Design Assumptions

1.11.2.1 Encased beams shall be proportioned to support, unassisted, all dead loads applied prior to the hardening of the concrete (unless these loads are supported temporarily on shoring) and, acting in conjunction with the slab, to support all dead and live loads applied after hardening of the concrete, without exceeding a computed bending stress of  $0.66F_y$ , where  $F_y$  is the yield stress of the steel beam. The bending stress produced by loads after the concrete has hardened shall be computed on the basis of the section properties of the composite section. Concrete tension stresses shall be neglected. Alternatively, the steel beam alone may be proportioned to resist, unassisted, the positive moment produced by all loads, live and dead, using a bending stress equal to  $0.76F_y$ , in which case temporary shoring is not required.

1.11.2.2 When shear connectors are used in accordance with Sect. 1.11.4, the composite section shall be proportioned to support all of the loads without exceeding the allowable stress prescribed in Sect. 1.5.1.4, even when the steel section is not shored during construction. In calculations involving composite sections in positive moment areas, the steel cross section is exempt from the compactness requirements of subparagraphs 2, 3, and 5 of Sect. 1.5.1.4.1.

Reinforcement parallel to the beam within the effective width of the slab, when anchored in accordance with the provisions of the applicable building code, may be included in computing the properties of composite sections, provided shear connectors are furnished in accordance with the requirements of Sect. 1.11.4. The section properties of the composite section shall be computed in accordance with the elastic theory. Concrete tension stresses shall be neglected. For stress computations, the compression area of lightweight or normal weight concrete shall be treated as an equivalent area of steel by dividing it by the modular ratio,  $n$ , for normal weight concrete of the strength specified when determining the section properties. For deflection calculations, the transformed section properties shall be based on the appropriate modular ratio,  $n$ , for the strength and weight concrete specified, where  $n = E_c/E$ .

In cases where it is not feasible or necessary to provide adequate connectors to satisfy the horizontal shear requirements for full composite action, the effective section modulus shall be determined as

$$S_{eff} = S_s + \sqrt{\frac{V'_{tr}}{V_h}} (S_{tr} - S_s) \quad (1.11-1)$$

where

$V_h$  and  $V'_{tr}$  are as defined in Sect. 1.11.4

$S_s$  = section modulus of the steel beam referred to its bottom flange, inches<sup>3</sup>

$S_{tr}$  = section modulus of the transformed composite section referred to its bottom flange, based upon maximum permitted effective width of concrete flange (Sect. 1.11.1), inches<sup>3</sup>



For construction without temporary shoring, stress in the steel section may be computed from the total dead plus live load moment and the transformed section modulus,  $S_{tr}$ , provided that the numerical value of  $S_{tr}$  so used shall not exceed

$$S_{tr} = \left( 1.35 + 0.35 \frac{M_L}{M_D} \right) S_s \quad (1.11-2)^*$$

In this expression for the limiting value of  $S_{tr}$ ,  $M_L$  is the moment caused by loads applied subsequent to the time when the concrete has reached 75 percent of its required strength,  $M_D$  is the moment caused by loads applied prior to this time, and  $S_s$  is the section modulus of the steel beam referred to the flange where the stress is being computed. At sections subject to positive bending moment, the stress shall be computed for the steel tension flange. At sections subject to negative bending moment, the stress shall be computed for the steel tension and compression flanges. These stresses shall not exceed the appropriate value in Sect. 1.5.1. Section 1.5.6 shall not apply to stresses in the negative moment area computed under the provisions of this paragraph.

The actual section modulus of the transformed composite section shall be used in calculating the concrete flexural compression stress and, for construction without temporary shores, this stress shall be based upon loading applied after the concrete has reached 75 percent of its required strength. The stress in the concrete shall not exceed  $0.45f'_c$ .

Suggestion: (1.11.2 Specification, Milek 29Aug79):

Include information on clustering of studs.

### 1.13.2 Vibration

Where human comfort is the criterion for limiting motion, as in the case of perceptible vibrations, the limit of tolerable amplitude is dependent on both the frequency of the vibration and the damping effect provided by components of the construction. At best, the evaluation of these criteria is highly subjective, although mathematical models<sup>42</sup> do exist which may be useful. When such vibrations are caused by running machinery, they should be isolated by effective damping devices or by the use of independent foundations.

The depth of a steel beam supporting large open floor areas free of partitions or other sources of damping should not be less than  $\frac{1}{20}$  of the span, in order to minimize perceptible transient vibration due to pedestrian traffic.

Suggestion: (1.13.2 Commentary, Foreman 13Sep79):

Include Amplitude-Frequency curves together with formulae for calculating both amplitude and frequency. Refer to material by Murray (1975), and Murray and Hendrick (1977).

## SECTION 1.15 CONNECTIONS

### 1.15.1 Minimum Connections

Connections carrying calculated stresses, except for lacing, sag bars, and girts, shall be designed to support not less than 6 kips.

### 1.15.2 Eccentric Connections

Axially stressed members meeting at a point shall have their gravity axes intersect at a point, if practicable; if not, provision shall be made for bending stresses due to the eccentricity.

### 1.15.3 Placement of Rivets, Bolts, and Welds

Except as hereinafter provided, groups of rivets, bolts, or welds at the ends of any member transmitting axial stress into that member shall have their centers of gravity on the gravity axis of the member, unless provision is made for the effect of the resulting eccentricity. Except in members subject to repeated variation in stress, as defined in Sect. 1.7, disposition of fillet welds to balance the forces about the neutral axis or axes for end connections of single angle, double angle, and similar type members is not required. Eccentricity between the gravity axes of such members and the gage lines for their riveted or bolted end connections may be neglected in statically loaded members, but should be considered in members subject to fatigue loading.

### 1.15.4 Unrestrained Members

Except as otherwise indicated by the designer, connections of beams, girders, or trusses shall be designed as flexible, and may ordinarily be proportioned for the reaction shears only.

Flexible beam connections shall accommodate end rotations of unrestrained (simple) beams. To accomplish this, inelastic action in the connection is permitted.

### 1.15.5 Restrained Members\*

1.15.5.1 Fasteners or welds for end connections of beams, girders, and trusses shall be designed for the combined effect of forces resulting from moment and shear induced by the rigidity of the connections.

Suggestion: (1.15 Specification, McGuire 4Sep79):

Appoint an ad hoc task group to investigate provisions relating to the proportioning of end plate connections in tall buildings.

Background Information: End plate connections seem to be with us more and more. They are different from T-stub hangers.

With respect to the use of end plates in tall buildings - as contrasted to their use in single story industrial frames - I think a few cautionary notes may be deduced from Dr. Krishnamurthy's discussion in the 2nd Quarter 1979 AISC Engineering Journal. He notes, for example, "For these (live and wind loads) and all other loads which would be treated as static loads in conventional analysis and design, the author's procedure is equally applicable in his opinion." Also, "Many of the proposed connections would hold the original angles virtually unchanged, within the working load levels; many would not." I don't agree that, just because we conventionally treat wind on a tall building as a static load, we can ignore the question of whether or not the bolts could loosen under fluctuating live and wind loads. Similarly, the source of any semi-rigid

behavior should be identified before relatively thin end plates are sanctioned for use as moment connections in tall buildings. If the source is permanent bolt elongation, the connection could be an undesirable one. Concerns of this sort could be considered by the ad-hoc group suggested above.

### 1.15.5 Restrained Members\*

1.15.5.1 Fasteners or welds for end connections of beams, girders, and trusses shall be designed for the combined effect of forces resulting from moment and shear induced by the rigidity of the connections.

1.15.5.2 When flanges or moment connection plates for end connections of beams and girders are welded to the flange of an I- or H-shape column, a pair of column-web stiffeners having a combined cross-sectional area,  $A_{st}$ , not less than that computed from Formula (1.15-1) shall be provided whenever the calculated value of  $A_{st}$  is positive.

$$A_{st} = \frac{P_{bf} - F_{yc}t(t_b + 5k)}{F_{yst}} \quad (1.15-1)$$

where

- $F_{yc}$  = column yield stress, kips per square inch
- $F_{yst}$  = stiffener yield stress, kips per square inch
- $k$  = distance between outer face of column flange and web toe of its fillet, if column is a rolled shape, or equivalent distance if column is a welded shape, inches
- $P_{bf}$  = the computed force delivered by the flange or moment connection plate multiplied by  $\frac{5}{3}$ , when the computed force is due to live and dead load only, or by  $\frac{4}{3}$ ,\* when the computed force is due to live and dead load in conjunction with wind or earthquake forces, kips
- $t$  = thickness of column web, inches
- $t_b$  = thickness of flange or moment connection plate delivering concentrated force, inches

1.15.5.3 Notwithstanding the requirements of Sect. 1.15.5.2, a stiffener or a pair of stiffeners shall be provided opposite the compression flange when the column web depth clear of fillets,  $d_c$ , is greater than

$$\frac{4100 t^3 \sqrt{F_{yc}}}{P_{bf}} \quad (1.15-2)$$

and a pair of stiffeners shall be provided opposite the tension flange when the thickness of the column flange,  $t_f$ , is less than

$$0.4 \sqrt{\frac{P_{bf}}{F_{yc}}} \quad (1.15-3)$$

1.15.5.4 Stiffeners required by the provisions of Sects. 1.15.5.2 and 1.15.5.3 shall comply with the following criteria:

1. The width of each stiffener plus  $\frac{1}{2}$  the thickness of the column web shall be not less than  $\frac{1}{3}$  the width of the flange or moment connection plate delivering the concentrated force.
2. The thickness of stiffeners shall be not less than  $t_b/2$ .\*\*
3. When the concentrated force delivered occurs on only one column flange, the stiffener length need not exceed  $\frac{1}{2}$  the column depth.
4. The weld joining stiffeners to the column web shall be sized to carry the force in the stiffener caused by unbalanced moments on opposite sides of the column.

1.15.5.5 Connections having high shear in the column web shall be investigated.†

\*Except where other codes may govern. For example, see Section 4(D) "Recommended Lateral Force Requirements and Commentary", Structural Engineers Assoc. of California, 1975.

\*\* See Commentary Sect. 1.15.5 for comment on width-thickness ratio of stiffeners.

† See Commentary Sect. 1.5.1.2.

Suggestion: (1.15.5 Specification, Popov 29Aug79):

Study the effect of stiffening in rectangular tubes and include in the Specification and the Monograph Volume SB.

## SECTION 1.21 COLUMN BASES

### 1.21.1 Loads

Proper provision shall be made to transfer the column loads and moments to the footings and foundations.

### 1.21.2 Alignment

Column bases shall be set level and to correct elevation with full bearing on the masonry.

### 1.21.3 Finishing

Column bases and base plates shall be finished in accordance with the following requirements:

1. Rolled steel bearing plates 2 inches or less in thickness may be used without milling,\* provided a satisfactory contact bearing is obtained; rolled steel bearing plates over 2 inches but not over 4 inches in thickness may be straightened by pressing or, if presses are not available, by milling for all bearing surfaces (except as noted in subparagraph 3 of this Section), to obtain a satisfactory contact bearing; rolled steel bearing plates over 4 inches in thickness shall be milled for all bearing surfaces (except as noted in subparagraph 3 of this Section).
2. Column bases other than rolled steel bearing plates shall be milled for all bearing surfaces (except as noted in subparagraph 3 of this Section).
3. The bottom surfaces of bearing plates and column bases which are grouted to insure full bearing contact on foundations need not be milled.

Suggestion: (1.21 Specification, Popov 29Aug79):

Include more information on column bases and anchorage.

## SECTION 2.1 SCOPE

Subject to the limitations contained herein, simple and continuous beams, braced and unbraced planar rigid frames, and similar portions of structures rigidly constructed so as to be continuous over at least one interior support,\* may be proportioned on the basis of plastic design, i.e., on the basis of their maximum strength. This strength, as determined by rational analysis, shall be not less than that required to support a factored load equal to 1.7 times the given live load and dead load, or 1.3 times these loads acting in conjunction with 1.3 times any specified wind or earthquake forces.

Rigid frames shall satisfy the requirements for Type 1 construction in the plane of the frame, as provided in Sect. 1.2. This does not preclude the use of some simple connections, provided that the provisions of Sect. 2.3 are satisfied. Type 2 construction is permitted for members between rigid frames. Connections joining a portion of a structure designed on the basis of plastic behavior with a portion not so designed need be no more rigid than ordinary seat-and-top-angle or ordinary web connections.

Where plastic design is used as the basis for proportioning continuous beams and structural frames, the provisions relating to allowable working stress, contained in Part 1, are waived. Except as modified by these rules, however, all other pertinent provisions of Part 1 shall govern.

It is not recommended that crane runways be designed continuous over interior vertical supports on the basis of maximum strength. However, rigid frame bents supporting crane runways may be considered as coming within the scope of the rules.

Suggestion: (2 Specification, Khan, Viest, Lu, Popov 29Aug79):

Make Part 2 more complete.

Background Information: Is part two sufficiently complete (Khan)? Eventually ATC-3 (1978) will force the use of plastic design in the consideration of the ultimate state (Viest). The proposed Japanese specification requires the consideration of plastic behavior (ductility) in determining the design earthquake forces (Lu). The California State Department of Architecture requires plastic analysis of certain structures (Popov).

The following additional suggestions do not refer to particular Specification or Commentary items and are simply designated by a letter.

A) LRFD

Suggestion: (Foreman 13Sep79):

LRFD should be included in the Specification as a separate part, i.e. treat it similarly to plastic design, as Part 3.

B) METHODS OF ANALYSIS

Suggestion: (Gaylord 29Aug79):

Should the Specification prescribe methods of analysis?

Background Information: Question of proportioning structure as a whole as against proportioning of members in the present AISC.

C) DYNAMIC RESPONSE

Suggestion: (Milek, Khan 29Aug79):

Include more information on dynamic response.

Background Information: When the drift is greater than 1/500 it is an indication that one should consider a wind tunnel test.

D) DRIFT CONTROL

Suggestion: (Popov 29Aug79):

Drift control needs to be added for drift limit and design ultimate load.

E) FACTOR OF SAFETY FOR DYNAMIC LOADING

Suggestion: (Iffland 29Aug79):

The present one-third increase for factor of safety for dynamic loading needs to be studied. It is not the right approach.



F) PERFORMANCE

Suggestion: (Galambos, Khan 29Aug79):

Add a separate appendix to handle the topic of performance.

Background Information: The present Specification does not speak directly to performance. Are we concerned about it (Khan)? The consensus was "yes". The Canadians handle this by a separate appendix (Galambos).

G) CLADDING

Suggestion: (Munse 29Aug79):

Include in the Commentary whether the failures in cladding are due to fracture or fatigue.

H) NIL-DUCTILITY

Suggestion: (Khan 29Aug79):

Include information on nil-ductility.

Background Information: There is always a question about this in cold areas and construction during winter. Also fractures of big weldments after welding.

I) LAMELLAR TEARING

Suggestion: (Driscoll 29Aug79):

Insert in the Commentary material on lamellar tearing on page 459 and refer to pages 554-557 of the Monograph Vol. SB (Council on Tall Buildings, 1979).

**Design.** Lamellar tearing generally results when a high degree of restraint is built into a weldment and produces large strains in the through-thickness direction of rolled steel plates or shapes. Therefore, care must be exercised in design to provide flexibility that will relieve the strains that might develop as a result of weld shrinkage, particularly in a highly restrained weldment. In addition, the welding processes and procedures should be selected so as to reduce to a minimum the susceptibility of a weldment to lamellar tearing.

**Recommendations.** Farrar et al. (1969) made suggestions for reducing the risk of lamellar tearing, which involve decohesion at inclusions or inclusion clusters, followed by linkage of the decohesed regions by shear or by normal ductile fracture for smaller inclusions. To reduce the risk of lamellar tearing of a corner joint, they propose the redesign shown in Fig. 6.63, because the fusion boundary is no longer parallel to the plane of the plate. Some other remedial measures that can be taken to reduce the risk of lamellar tearing are shown in Fig. 6.64. They are: (1) The use of low-strength weld metals; (2) modified run procedure; (3) buttering; and (4) balanced welding (Farrar et al., 1969). Further recommendations can be found in a commentary prepared by the American Institute of Steel Construction (AISC, 1973).

Heuschkel (1971) showed that decohesion cracking parallel to the plate surfaces occurred most commonly in corner and tee joints when welded under conditions of high restraint, whereas minimum weldment susceptibility to decohesion cracking occurred in clean, ductile, tough steels, and where the designs and welding procedures involved minimum rigidity and the lowest residual stresses. (Council on Tall Buildings, 1979, p. 459).

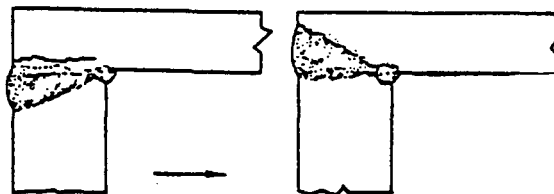


Fig. 6.63 Possible modification of corner joint to reduce risk of tearing

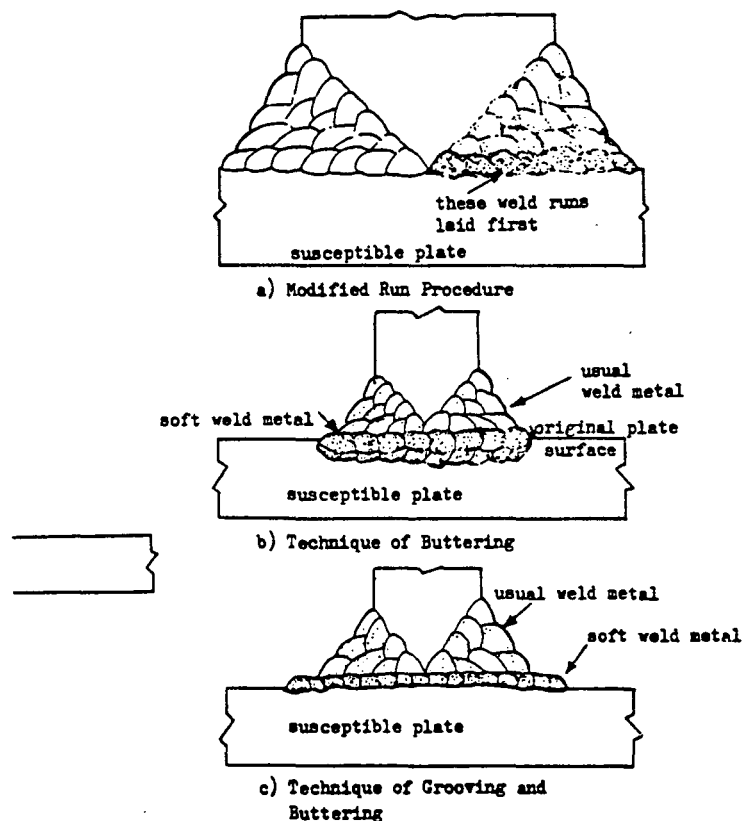


Fig. 6.64 Precautions or techniques to reduce risk of lamellar tearing

J) STEEL/CONCRETE CONNECTIONS

Suggestion: (Milek 29Aug79):

Include information on connecting steel beams to concrete columns and walls.

K) DIFFERENTIAL MOVEMENT OF COLUMNS

Suggestion: (Khan 29Aug79):

Include in the Commentary information on differential movement of columns.

Background Information: When steel and concrete columns are intermixed, consideration must be given to the differential movement (because of concrete shrinkage). Example: One Shell Square.

L) MIXED CONSTRUCTION

Suggestion: (McGuire 4Sep79):

Should there be a new section in the Specification on Mixed Construction?

Background Information: I just raise the general question of whether or not it is time to have some provisions for mixed construction (particularly columns) in the AISC Specification.

Note: Suggestions are expected and will be assembled as a supplement to this report.

M) COMMENTARY FOR TALL BUILDINGS

Suggestion: (Higgins 29Aug79):

Perhaps there should be a separate, short, concise statement in the Commentary about items that are particularly unique to tall buildings.

N) SCOPE

Suggestion: (Gaylord 29Aug79):

Should the Specification go beyond individual members and deal with frames?

O) SEISMIC PROVISIONS

Suggestion: (Driscoll 29Aug79):

Include in the Specification new seismic information from the Monograph Vol. SB (Council on Tall Buildings, 1979).

## 5. APPENDIX

BALLOT

Page	Item No.	Topic	By	<u>Recommend to Main Committee</u>		
				Yes	No*	<u>Further Committee Discussion Needed*</u>
<u>SPECIFICATION CHANGES</u>						
3	1.3.5	Wind	Foreman	—	—	—
4	1.7.1	Fatigue	Munse	—	—	—
5	1.11.1	Effective Width	Viest	—	—	—
6	2.4(A)	Columns		—	—	—
		(Eqs. 2.4-2 & 3)	Driscoll	—	—	—
8	2.4(B)	Columns (Eq. 2.4-4)	Driscoll	—	—	—
<u>COMMENTARY CHANGES</u>						
9	1.2(A)	Type 2 Construction	McGuire, Iffland, Beedle	—	—	—
9	1.2(B)	Lamellar Tearing	Munse	—	—	—
9	1.2(C)	Brittle Fracture	Munse	—	—	—
10	1.3.5	Wind	Foreman	—	—	—
12	1.7	Fatigue	Munse	—	—	—
13	1.11.1	Effective Width	Viest	—	—	—
14	1.15.A	Connections (Heavy Bolted)	Chen	—	—	—
15	2.1(A)	Plastic Analysis	Popov	—	—	—
15	2.1(B)	Ductile Behavior	Popov	—	—	—
16	2.4	Columns	Driscoll	—	—	—
<u>OTHER SUGGESTIONS</u>						
20	1.2	Type 2 Construction	McGuire	—	—	—
21	1.3.5(A)	Wind	Gaylord	—	—	—
21	1.3.5(B)	Wind	Khan, Viest	—	—	—
21	1.3.5(C)	Wind	Popov, Galambos	—	—	—
22	1.3.6	Earthquakes & Hurricanes	Gaylord	—	—	—
23	1.5.1.3	Minor Axis Bending	Iffland	—	—	—
24	1.5.1.3	End Restraint & Initial Column Crookedness	Iffland	—	—	—
25	1.5.1.4	Lateral Buckling	McGuire	—	—	—
28	1.6	Combined Stresses	McGuire	—	—	—
29	1.6	Biaxial Bending	Iffland	—	—	—

\*In case of any "No" or "Further Discussion" votes, comment is to be made on the attached page.

Page	Item No.	Topic	By	Recommend to Main Committee		
				Yes	No	Further Committee Discussion Needed
31	1.8	Nonlinear Analysis	McGuire	—	—	—
33	1.8	Frame vs. Column Stability	Iffland	—	—	—
34	1.10	Plate Girders	McGuire	—	—	—
35	1.11(A)	Concrete-Encased Steel Columns	Viest	—	—	—
35	1.11(B)	Concrete-Filled Tubular Columns	Viest	—	—	—
36	1.11.2	Clustering of Studs	Milek	—	—	—
38	1.13.2	Vibration	Foreman	—	—	—
39	1.15	End Plate Connections	McGuire	—	—	—
41	1.15.5	Rectangular Tube Stiffening	Popov	—	—	—
42	1.21	Column Bases	Popov	—	—	—
43	2	Completeness	Khan, Viest, Lu	—	—	—
44	A	LRFD	Foreman	—	—	—
44	B	Methods of Analysis	Gaylord	—	—	—
44	C	Dynamic Response	Milek, Khan	—	—	—
44	D	Drift Control	Popov	—	—	—
44	E	Factor of Safety for Dynamic Loading	Iffland	—	—	—
45	F	Performance	Galambos, Khan	—	—	—
45	G	Cladding	Munse	—	—	—
45	H	Nil-Ductility	Khan	—	—	—
45	I	Lamellar Tearing	Driscoll	—	—	—
47	J	Steel/Concrete Connections	Milek	—	—	—
47	K	Differential Movement of Columns	Khan	—	—	—
47	L	Mixed Construction	McGuire	—	—	—
47	M	Commentary for Tall Buildings	Higgins	—	—	—
47	N	Scope	Gaylord	—	—	—
47	O	Seismic Provisions	Driscoll	—	—	—

Name

Date

COMMENTS

Item No.

Comment

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