

GUIDE FOR

BUCKLING AND ULTIMATE STRENGTH ASSESSMENT FOR OFFSHORE STRUCTURES APRIL 2004

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The following summarizes the colors corresponding to Rule Changes, Corrigenda items and editorial changes in the Guide files which are available for download.

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BUCKLING AND ULTIMATE STRENGTH ASSESSMENT FOR OFFSHORE STRUCTURES

APRIL 2004 (Updated January 2018 – see next page)

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ABS Plaza
16855 Northchase Drive
Houston, TX 77060 USA**

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Foreword

This *Guide for the Buckling and Ultimate Strength Assessment of Offshore Structures* is referred to herein as “this Guide”. This Guide provides criteria that can be used in association with specific Rules and Guides issued by ABS for the classification of specific types of Offshore Structures. The specific Rules and Guides that this Guide supplements are the latest editions of the following.

- *Rules for Building and Classing Offshore Installations* [for steel structure only]
- *Rules for Building and Classing Mobile Offshore Drilling Units (MODUs)*
- *Rules for Building and Classing Single Point Moorings (SPMs)*
- *Rules for Building and Classing Floating Production Installations (FPIs)* [for non ship-type hulls].

In case of conflict between the criteria contained in this Guide and the above-mentioned *Rules*, the latter will have precedence.

These criteria are not to be applied to ship-type FPIs, which are being reviewed to receive a *SafeHull*-related Classification Notation. (This includes ship-type FPIs receiving the *SafeHull-Dynamic Load Approach* Classification Notation) In these vessel-related cases, the criteria based on the contents of Part 5C of the *ABS Rules for Building and Classing Steel Vessels (SVR)* apply.

The criteria presented in this Guide may also apply in other situations such as the certification or verification of a structural design for compliance with the Regulations of a Governmental Authority. However, in such a case, the criteria specified by the Governmental Authority should be used, but they may not produce a design that is equivalent to one obtained from the application of the criteria contained in this Guide. Where the mandated technical criteria of the cognizant Governmental Authority for certification differ from those contained herein, ABS will consider the acceptance of such criteria as an alternative to those given herein so that, at the Owner or Operator’s request, both certification and classification may be granted to the Offshore Structure.

ABS welcomes questions on the applicability of the criteria contained herein as they may apply to a specific situation and project.

ABS also appreciates the receipt of comments, suggestions and technical and application questions for the improvement of this Guide. For this purpose, enquiries can be sent electronically to rsd@eagle.org.



GUIDE FOR

BUCKLING AND ULTIMATE STRENGTH ASSESSMENT FOR OFFSHORE STRUCTURES

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SECTION 1 Introduction

1 General (2018)

The criteria in this Guide are primarily based on existing methodologies and their attendant safety factors. These methods and factors are deemed to provide an equivalent level of safety, reflecting what is considered to be appropriate current practice.

It is acknowledged that new methods and criteria for design are constantly evolving. For this reason, ABS does not seek to inhibit the use of an alternative technological approach that is demonstrated to produce an acceptable level of safety.

The criteria in this Guide is presented in the Working Stress Design (WSD) format, also known as the Allowable Stress (or Strength) Design (ASD) format. Alternative structural design criteria in a Load and Resistance Factor Design (LRFD) format are provided in the *ABS Guide for Buckling and Ultimate Strength Assessment of Offshore Structures (LRFD Version)*.

3 Scope of this Guide

This Guide provides criteria that should be used on the following structural steel components or assemblages:

- Individual structural members (i.e., discrete beams and columns) [see Section 2]
- Plates, stiffened panels and corrugated panels [see Section 3]
- Stiffened cylindrical shells [see Section 4]
- Tubular joints [see Section 5]

Additionally, Appendix 1 contains guidance on the review of buckling analysis using the finite element method (FEM) to establish buckling capacities.

5 Tolerances and Imperfections

The buckling and ultimate strength of structural components are highly dependent on the amplitude and shape of the imperfections introduced during manufacture, storage, transportation and installation.

Typical imperfections causing strength deterioration are:

- Initial distortion due to welding and/or other fabrication-related process
- Misalignments of joined components

In general, the effects of imperfections in the form of initial distortions, misalignments and weld-induced residual stresses are implicitly incorporated in the buckling and ultimate strength formulations. Because of their effect on strength, it is important that imperfections be monitored and repaired, as necessary, not only during construction, but also in the completed structure to ensure that the structural components satisfy tolerance limits. The tolerances on imperfections to which the strength criteria given in this Guide are considered valid are listed, for example, in *IACS Recommendation No. 47 "Shipbuilding and Repair Quality Standard"*. Imperfections exceeding such published tolerances are not acceptable unless it is shown using a recognized method that the strength capacity and utilization factor of the imperfect structural component are within proper target safety levels.

7 Gross Scantlings (2018)

The buckling and ultimate strength formulations provided in this Guide are intended to be used along with the gross scantlings of structural components.

9 Loadings

Conditions representing all modes of operation of the Offshore Structure are to be considered to establish the most critical loading cases. The ABS Rules and Guides for the classification of various types of Offshore Structures typically define two primary loading conditions. In the *ABS Rules for Building and Classing Mobile Offshore Drilling Units (MODU Rules)*, they are 'Static Loadings' and 'Combined Loadings', and in the *ABS Rules for Building and Classing Offshore Installations (Offshore Installations Rules)*, the *ABS Rules for Building and Classing Single Point Moorings (SPM Rules)* and the *ABS Rules for Building and Classing Floating Production Installations (FPI Rules)* they are 'Normal Operation' and 'Severe Storm'. The component loads of these loading conditions are discussed below. The determination of the magnitudes of each load component and each load effect (i.e., stress, deflection, internal boundary condition, etc.) are to be performed using recognized calculation methods and/or test results and are to be fully documented and referenced. As appropriate, the effects of stress concentrations, secondary stress arising from eccentrically applied loads and member displacements (i.e., P-Δ effects) and additional shear displacements and shear stress in beam elements are to be suitably accounted for in the analysis.

The primary loading conditions to be considered in the *MODU Rules* are:

- i) *Static Loadings.* Stresses due to static loads only, where the static loads include operational gravity loads and the weight of the unit, with the unit afloat or resting on the seabed in calm water.
- ii) *Combined Loadings.* Stresses due to combined loadings, where the applicable static loads, as described above, are combined with relevant environmental loadings, including acceleration and heeling forces.

The primary loading conditions to be considered in the *Offshore Installations Rules*, *SPM Rules* and *FPI Rules* are:

- i) *Normal Operations.* Stresses due to operating environmental loading combined with dead and maximum live loads appropriate to the function and operations of the structure
- ii) *Severe Storm.* Stresses due to design environmental loading combined with dead and live loads appropriate to the function and operations of the structure during design environmental condition

The buckling and ultimate strength formulations in this Guide are applicable to static/quasi-static loads, Dynamic (e.g., impulsive) loads, such as may result from impact and fluid sloshing, can induce 'dynamic buckling', which, in general, is to be dealt with using an appropriate nonlinear analysis.

11 Maximum Allowable Strength Utilization Factors

The buckling and ultimate strength equations in this Guide provide an estimate of the average strength of the considered components while achieving the lowest standard deviation when compared with nonlinear analyses and mechanical tests. To ensure the safety of the structural components, maximum allowable strength utilization factors, which are the inverse of safety factors, are applied to the predicted strength. The maximum allowable strength utilization factors will, in general, depend on the given loading condition, the type of structural component and the failure consequence.

The maximum allowable strength utilization factors, η , are based on the factors of safety given in the *Offshore Installations Rules*, *MODU Rules*, *SPM Rules* and *FPI Rules*, as applicable. The maximum allowable strength utilization factors have the following values.

- i) For a loading condition that is characterized as a *static loading* of a Mobile Offshore Drilling Unit or *normal operation* of an Offshore Installation, Floating Production Installation and Single Point Mooring:

$$\eta = 0.60\psi$$

- ii) For a loading condition that is characterized as a *combined loading* of a Mobile Offshore Drilling Unit or *severe storm* of an Offshore Installation, Floating Production Installation and Single Point Mooring:

$$\eta = 0.80\psi$$

where

ψ = adjustment factor, as given in subsequent sections of this Guide.

Under the above-mentioned Rules and Guides, it is required that both of the characteristic types of loading conditions (i.e., static and combined, or normal operation and severe storm) are to be applied in the design and assessment of a structure. The loading condition producing the most severe requirement governs the design.

In the Sections that follow concerning specific structural components, different adjustment factors may apply to different types of loading (i.e., tension or bending versus pure compression). To represent the values of η applicable to the different types of load components, subscripts are sometimes added to the symbol η (e.g., in Section 2, η_1 and η_2 , apply, respectively, to axial compression or tension/bending in the individual structural member.).

SECTION 2 Individual Structural Members

1 General

This Section provides strength criteria for individual structural members. The types of members considered in this Section are tubular and non-tubular members with uniform geometric properties along their entire length and made of a single material. The criteria provided in this Section are for tubular and non-tubular elements, but other recognized standards are also acceptable.

The behavior of structural members is influenced by a variety of factors, including sectional shape, material characteristics, boundary conditions, loading types and parameters and fabrication methods.

1.1 Geometries and Properties of Structural Members

A structural member with a cross section having at least one axis of symmetry is considered. The geometries and properties of some typical cross sections are illustrated in Section 2, Table 1. For sections which are not listed in Section 2, Table 1, the required geometric properties are to be calculated based on acceptable formulations.

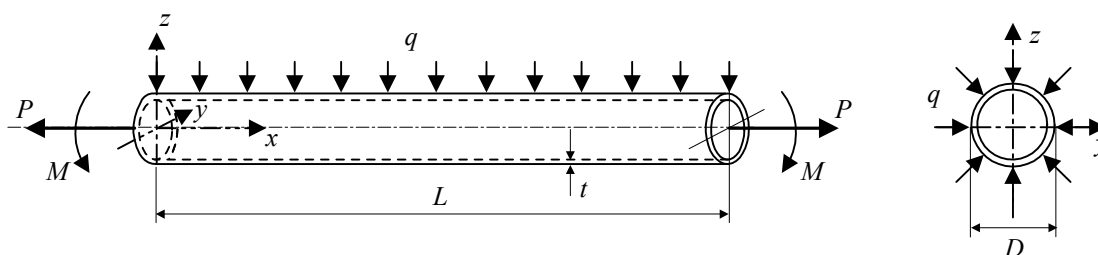
1.3 Load Application

This Section includes the strength criteria for any of the following loads and load effects:

- Axial force in longitudinal direction, P
- Bending moment, M
- Hydrostatic pressure, q
- Combined axial tension and bending moment
- Combined axial compression and bending moment
- Combined axial tension, bending moment and hydrostatic pressure
- Combined axial compression, bending moment and hydrostatic pressure

The load directions depicted in Section 2, Figure 1 are positive.

FIGURE 1
Load Application on a Tubular Member



1.5 Failure Modes

Failure modes for a structural member are categorized as follows:

- *Flexural buckling.* Bending about the axis of the least resistance.
- *Torsional buckling.* Twisting about the longitudinal (x) axis. It may occur if the torsional rigidity of the section is low, as for a member with a thin-walled open cross section.
- *Lateral-torsional buckling.* Synchronized bending and twisting. A member which is bent about its major axis may buckle laterally.
- *Local buckling.* Buckling of a plate or shell element that is a local part of a member

TABLE 1
Geometries, Properties and Compact Limits of Structural Members

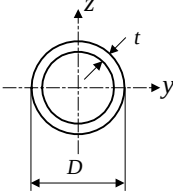
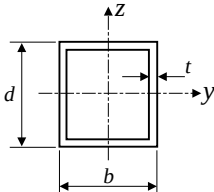
Geometry	Sectional Shape	Geometrical Parameters	Axis	Properties*	Compact Limits
1. Tubular member		D = Outer diameter t = Thickness	N/A	$A = \pi[D^2 - (D - 2t)^2]/4$ $I_y, I_z = \pi[D^4 - (D - 2t)^4]/64$ $K = \pi(D - t)^3t/4$ $I_o = \pi[D^4 - (D - 2t)^4]/32$ $\Gamma = 0$	$\frac{D}{t} \leq \frac{E}{9\sigma_0}$
2. Square or rectangular hollow section		b = Flange width d = Web depth t = Thickness	Major y-y Minor z-z	$A = 2(b + d)t$ $I_y = d^2t(3b + d)/6$ $I_z = b^2t(b + 3d)/6$ $K = \frac{2b^2d^2t}{b + d}$ $I_o = t(b + d)^3/6$ $\Gamma = \frac{b^2d^2t}{24} \frac{(d - b)^2}{b + d}$	$\frac{b}{t}, \frac{d}{t} \leq 1.5 \sqrt{\frac{E}{\sigma_0}}$

TABLE 1 (continued)
Geometries, Properties and Compact Limits of Structural Members

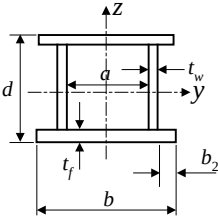
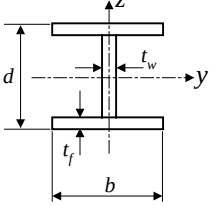
Geometry	Sectional Shape	Geometrical Parameters	Axis	Properties*	Compact Limits
3. Welded box shape		d = Web depth t_w = Web thickness b = Flange width t_f = Flange thickness b_2 = Outstand	Major y-y Minor z-z	$A = 2(bt_f + dt_w)$ $I_y = d^2(3bt_f + dt_w)/6$ $I_z = b^2(bt_f + 3dt_w)/6$ $K = \frac{2a^2d^2}{\left(\frac{a}{t_f} + \frac{d}{t_w}\right)}$ $I_o = I_y + I_z$ $\Gamma = \frac{(b^3d^2t_f - a^2d^3t_w)^2}{24(b^3d^2t_f + a^2d^3t_w)}$	$\frac{a}{t_f}, \frac{d}{t_w} \leq 1.5 \sqrt{\frac{E}{\sigma_0}}$ $\frac{b_2}{t_f} \leq 0.4 \sqrt{\frac{E}{\sigma_0}}$
4. W-shape		d = Web depth t_w = Web thickness b = Flange width t_f = Flange thickness	Major y-y Minor z-z	$A = 2bt_f + dt_w$ $I_y = d^2(6bt_f + dt_w)/12$ $I_z = b^3t_f/6$ $K = (2bt_f^3 + dt_w^3)/3$ $I_o = I_y + I_z$ $\Gamma = d^2b^3t_f/24$	$\frac{d}{t_w} \leq 1.5 \sqrt{\frac{E}{\sigma_0}}$ $\frac{b}{t_f} \leq 0.8 \sqrt{\frac{E}{\sigma_0}}$

TABLE 1 (continued)
Geometries, Properties and Compact Limits of Structural Members

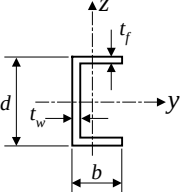
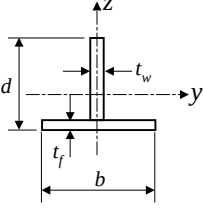
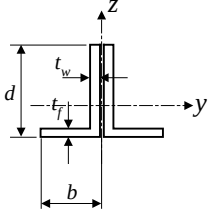
Geometry	Sectional Shape	Geometrical Parameters	Axis	Properties*	Compact Limits
5. Channel		d = Web depth t_w = Web thickness b = Flange width t_f = Flange thickness d_{cs} = Distance of shear center to centroid	Major y-y Minor z-z	$A = 2bt_f + dt_w$ $I_y = d^2(6bt_f + dt_w)/12$ $I_z = d^3t_w(bt_f + 2dt_w)/3A$ $K = (2bt_f^3 + dt_w^3)/3$ $I_o = I_y + I_z + Ad_{cs}^2$ $\Gamma = \frac{d^2b^3t_f(3bt_f + 2dt_w)}{12(6bt_f + dt_w)}$	$\frac{d}{t_w} \leq 1.5 \sqrt{\frac{E}{\sigma_0}}$ $\frac{b}{t_f} \leq 0.4 \sqrt{\frac{E}{\sigma_0}}$
6. T-bar		d = Web depth t_w = Web thickness b = Flange width t_f = Flange thickness d_{cs} = Distance of shear center to centroid	Major y-y Minor z-z	$A = bt_f + dt_w$ $I_y = d^3t_w(4bt_f + dt_w)/12A$ $I_z = b^3t_f/12$ $K = (bt_f^3 + dt_w^3)/3$ $I_o = I_y + I_z + Ad_{cs}^2$ $\Gamma = (b^3t_f^3 + 4d^3t_w^3)/144$	$\frac{d}{t_w} \leq 0.4 \sqrt{\frac{E}{\sigma_0}}$ $\frac{b}{t_f} \leq 0.8 \sqrt{\frac{E}{\sigma_0}}$

TABLE 1 (continued)
Geometries, Properties and Compact Limits of Structural Members

Geometry	Sectional Shape	Geometrical Parameters	Axis	Properties*	Compact Limits
7. Double angles		d = Web depth t_w = Web thickness b = Flange width t_f = Flange thickness d_{cs} = Distance of shear center to centroid	Major y-y Minor z-z	$A = 2(bt_f + dt_w)$ $I_y = d^3 t_w (4bt_f + dt_w) / 3A$ $I_z = 2b^3 t_f / 3$ $K = 2(bt_f^3 + dt_w^3) / 3$ $I_o = I_y + I_z + A d_{cs}^2$ $\Gamma = (b^3 t_f^3 + 4d^3 t_w^3) / 18$	$\frac{d}{t_w} \leq 0.4 \sqrt{\frac{E}{\sigma_0}}$ $\frac{b}{t_f} \leq 0.4 \sqrt{\frac{E}{\sigma_0}}$

* The formulations for the properties are derived assuming that the section is thin-walled (i.e., thickness is relatively small) where:

- A = cross sectional area, cm^2 (in^2)
- I_y = moment of inertia about y-axis, cm^4 (in^4)
- I_z = moment of inertia about z-axis, cm^4 (in^4)
- K = St. Venant torsion constant for the member, cm^4 (in^4)
- I_o = polar moment of inertia of the member, cm^4 (in^4)
- Γ = warping constant, cm^6 (in^6)

1.7 Cross Section Classification

The cross section may be classified as:

- i) *Compact.* A cross section is compact if all compressed components comply with the limits in Section 2, Table 1. For a compact section, the local buckling (plate buckling and shell buckling) can be disregarded because yielding precedes buckling.
- ii) *Non-Compact.* A cross section is non-compact if any compressed component does not comply with the limits in Section 2, Table 1. For a non-compact section, the local buckling (plate or shell buckling) is to be taken into account.

1.9 Adjustment Factor

For the maximum allowable strength utilization factors, η , defined in Subsection 1/11, the adjustment factor is to take the following values:

For axial tension and bending [to establish η_2 below]:

$$\psi = 1.0$$

For axial compression (column buckling or torsional buckling) [to establish η_1 below]:

$$\begin{aligned} \psi &= 0.87 && \text{if } \sigma_{EA} \leq P_r \sigma_0 \\ &= 1 - 0.13 \sqrt{P_r \sigma_0 / \sigma_{EA}} && \text{if } \sigma_{EA} > P_r \sigma_0 \end{aligned}$$

where

$$\begin{aligned} \sigma_{EA} &= \text{elastic buckling stress, as defined in 2/3.3, N/cm}^2 \text{ (kgf/cm}^2 \text{, lbf/in}^2\text{)} \\ P_r &= \text{proportional linear elastic limit of the structure, which may be taken as 0.6 for steel} \\ \sigma_0 &= \text{specified minimum yield point, N/cm}^2 \text{ (kgf/cm}^2 \text{, lbf/in}^2\text{)} \end{aligned}$$

For compression (local buckling of tubular members) [to establish η_x and η_θ below]:

$$\begin{aligned} \psi &= 0.833 && \text{if } \sigma_{Ci} \leq 0.55 \sigma_0 \\ &= 0.629 + 0.371 \sigma_{Ci} / \sigma_0 && \text{if } \sigma_{Ci} > 0.55 \sigma_0 \end{aligned}$$

where

$$\begin{aligned} \sigma_{Ci} &= \text{critical local buckling stress, representing } \sigma_{Ci} \text{ for axial compression, as specified in 2/9.1, and } \sigma_{C\theta} \text{ for hydrostatic pressure, as specified in 2/9.5, N/cm}^2 \text{ (kgf/cm}^2 \text{, lbf/in}^2\text{)} \\ \sigma_0 &= \text{specified minimum yield point, N/cm}^2 \text{ (kgf/cm}^2 \text{, lbf/in}^2\text{)} \end{aligned}$$

3 Members Subjected to a Single Action

3.1 Axial Tension

Members subjected to axial tensile forces are to satisfy the following equation:

$$\sigma_t / \eta_2 \sigma_0 \leq 1$$

where

$$\begin{aligned} \sigma_t &= \text{axial tensile stress, N/cm}^2 \text{ (kgf/cm}^2 \text{, lbf/in}^2\text{)} \\ &= P/A \\ \sigma_0 &= \text{specified minimum yield point, N/cm}^2 \text{ (kgf/cm}^2 \text{, lbf/in}^2\text{)} \\ P &= \text{axial force, N (kgf, lbf)} \\ A &= \text{cross sectional area, cm}^2 \text{ (in}^2\text{)} \\ \eta_2 &= \text{allowable strength utilization factor for tension and bending, as defined in Subsection 1/11 and 2/1.9} \end{aligned}$$

3.3 Axial Compression

Members subjected to axial compressive forces may fail by flexural or torsional buckling. The buckling limit state is defined by the following equation:

$$\sigma_A / \eta_1 \sigma_{CA} \leq 1$$

where

$$\begin{aligned} \sigma_A &= \text{axial compressive stress, N/cm}^2 \text{ (kgf/cm}^2 \text{, lbf/in}^2\text{)} \\ &= -P/A \end{aligned}$$

$$P = \text{axial force, N (kgf, lbf)}$$

$$\sigma_{CA} = \text{critical buckling stress, N/cm}^2 \text{ (kgf/cm}^2 \text{, lbf/in}^2\text{)}$$

$$= \begin{cases} \sigma_{EA} & \text{if } \sigma_{EA} \leq P_r \sigma_F \\ \sigma_F \left[1 - P_r (1 - P_r) \frac{\sigma_F}{\sigma_{EA}} \right] & \text{if } \sigma_{EA} > P_r \sigma_F \end{cases}$$

$$P_r = \text{proportional linear elastic limit of the structure, which may be taken as 0.6 for steel}$$

$$\sigma_F = \sigma_0 \quad \text{specified minimum yield point for a compact section}$$

$$= \sigma_{Cx} \quad \text{local buckling stress for a non-compact section from Subsection 2/9}$$

$$\sigma_{EA} = \text{elastic buckling stress, which is the lesser of the solutions of the following quadratic equation, N/cm}^2 \text{ (kgf/cm}^2 \text{, lbf/in}^2\text{)}$$

$$\frac{I_0}{A} (\sigma_{EA} - \sigma_{E\eta}) (\sigma_{EA} - \sigma_{ET}) - \sigma_{EA}^2 d_{cs}^2 = 0$$

$$\sigma_{E\eta} = \text{Euler buckling stress about minor axis, N/cm}^2 \text{ (kgf/cm}^2 \text{, lbf/in}^2\text{)}$$

$$= \pi^2 E / (kL/r_\eta)^2$$

$$\sigma_{ET} = \text{ideal elastic torsional buckling stress, N/cm}^2 \text{ (kgf/cm}^2 \text{, lbf/in}^2\text{)}$$

$$= \frac{EK}{2.6I_0} + \left(\frac{\pi}{kL} \right)^2 \frac{E\Gamma}{I_0}$$

$$r_\eta = \text{radius of gyration about minor axis, cm (in.)}$$

$$= \sqrt{I_\eta / A}$$

$$E = \text{modulus of elasticity, } 2.06 \times 10^7 \text{ N/cm}^2 \text{ (} 2.1 \times 10^6 \text{ kgf/cm}^2 \text{, } 30 \times 10^6 \text{ lbf/in}^2\text{) for steel}$$

$$A = \text{cross sectional area, cm}^2 \text{ (in}^2\text{)}$$

$$I_\eta = \text{moment of inertia about minor axis, cm}^4 \text{ (in}^4\text{)}$$

$$K = \text{St. Venant torsion constant for the member, cm}^4 \text{ (in}^4\text{)}$$

$$I_0 = \text{polar moment of inertia of the member, cm}^4 \text{ (in}^4\text{)}$$

$$\Gamma = \text{warping constant, cm}^6 \text{ (in}^6\text{)}$$

$$d_{cs} = \text{difference of centroid and shear center coordinates along major axis, cm (in.)}$$

$$L = \text{member's length, cm (in.)}$$

$$k = \text{effective length factor, as specified in Section 2, Table 2. When it is difficult to clarify the end conditions, the nomograph shown in Section 2, Figure 2 may be used. The values of } G \text{ for each end (A and B) of the column are determined:}$$

$$G = \sum \frac{I_c}{L_c} / \sum \frac{I_g}{L_g}$$

$\sum \frac{I_c}{L_c}$ is the total for columns meeting at the joint considered and $\sum \frac{I_g}{L_g}$ is the total

for restraining beams meeting at the joint considered. For a column end that is supported, but not fixed, the moment of inertia of the support is zero, and the resulting value of G for this end of the column would be ∞ . However, in practice, unless the footing was designed as a frictionless pin, this value of G would be taken as 10. If the column end is fixed, the moment of inertia of the support would be ∞ , and the resulting value of G of this end of the column would be zero. However, in practice, there is some movement and G may be taken as 1.0. If the restraining beam is either pinned ($G = \infty$) or fixed ($G = 0$) at its far end, refinements may be made by multiplying the stiffness (I_g/L_g) of the beam by the following factors:

Sidesway prevented

Far end of beam pinned = 1.5

Sidesway permitted

Far end of beam pinned = 0.5

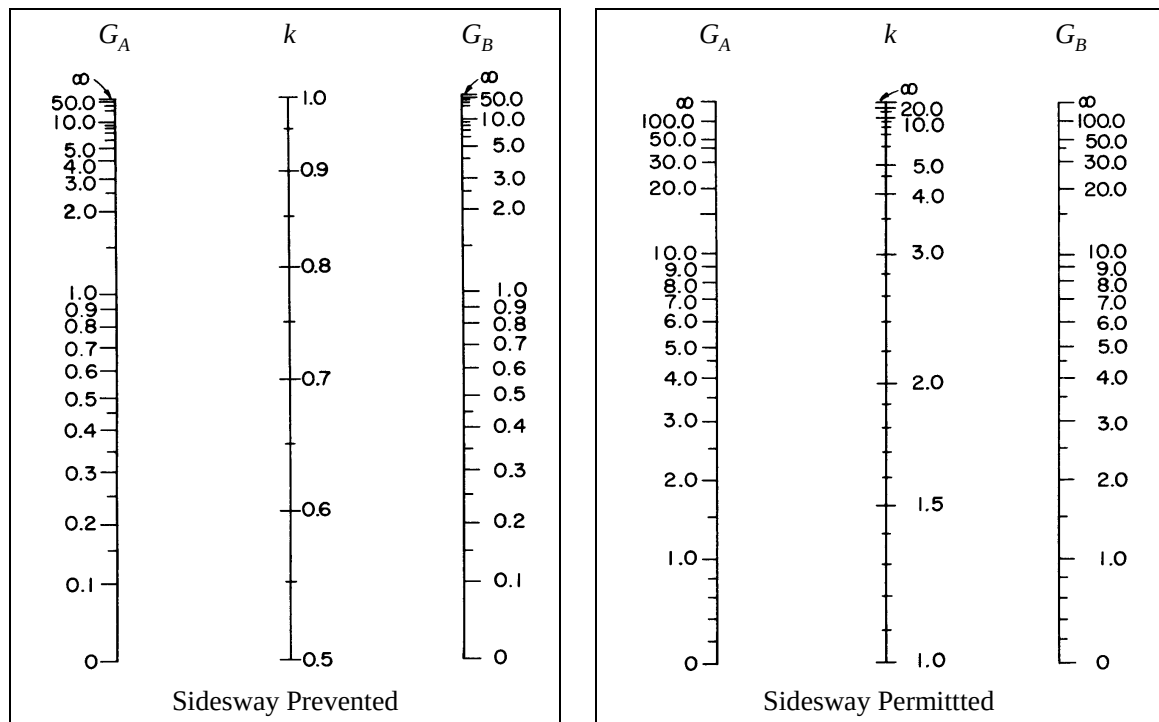
Far end of beam fixed = 2.0

η_1 = allowable strength utilization factor for axial compression (column buckling), as defined in Subsection 1/11 and 2/1.9

TABLE 2
Effective Length Factor

Buckled shape of column shown by dashed line						
Theoretical value	0.50	0.70	1.0	1.0	2.0	2.0
Recommended k value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.1	2.0
End condition notation	 	Rotation fixed. Translation fixed Rotation free. Translation fixed Rotation fixed. Translation free Rotation free. Translation free				

FIGURE 2
Effective Length Factor



Note: These alignment charts or nomographs are based on the following assumptions:

- 1 Behavior is purely elastic.
- 2 All members have constant cross section.
- 3 All joints are rigid.
- 4 For columns in frames with sidesway prevented, rotations at opposite ends of the restraining beams are equal in magnitude and opposite in direction, producing single curvature bending.
- 5 For columns in frames with sidesway permitted, rotations at opposite ends of the restraining beams are equal in magnitude and direction, producing reverse curvature bending.
- 6 The stiffness parameter $L(P/EI)^{1/2}$ of all columns is equal.
- 7 Joint restraint is distributed to the column above and below the joint in proportion to EI/L for the two columns.
- 8 All columns buckle simultaneously.
- 9 No significant axial compression force exists in the restraining beams.

Adjustments are required when these assumptions are violated and the alignment charts are still to be used. Reference is made to ANSI/AISC 360-05, Commentary C2.

3.5 Bending Moment

A member subjected to **bending moment** may fail by local buckling or lateral-torsional buckling. The buckling state limit is defined by the following equation:

$$\sigma_b / \eta_2 \sigma_{CB} \leq 1$$

where

$$\begin{aligned} \sigma_b &= \text{stress due to bending moment} \\ &= M / S M_e \end{aligned}$$

- M = bending moment, N-cm (kgf-cm, lbf-in)
 SM_e = elastic section modulus, cm³ (in³)
 η_2 = allowable strength utilization factor for tension and bending
 σ_{CB} = critical bending strength, as follows:
 i) For a tubular member, the critical bending strength is to be obtained from the equation given in 2/9.3.
 ii) For a rolled or fabricated-plate section, the critical bending strength is determined by the critical lateral-torsional buckling stress.

The critical lateral-torsional buckling stress is to be obtained from the following equation:

$$\sigma_{C(LT)} = \begin{cases} \sigma_{E(LT)} & \text{if } \sigma_{E(LT)} \leq P_r \sigma_F \\ \sigma_F \left[1 - P_r (1 - P_r) \frac{\sigma_F}{\sigma_{E(LT)}} \right] & \text{if } \sigma_{E(LT)} > P_r \sigma_F \end{cases}$$

P_r = proportional linear elastic limit of the structure, which may be taken as 0.6 for steel
 $\sigma_{E(LT)}$ = elastic lateral-torsional buckling stress, N/cm² (kgf/cm², lbf/in²)

$$= \sqrt{C} \frac{\pi^2 EI_\eta}{SM_c (kL)^2}$$
 I_η = moment of inertia about minor axis, as defined in Section 2, Table 1, cm⁴ (in⁴)
 SM_e = section modulus of compressive flange, cm³ (in³)

$$= \frac{I_\xi}{\xi_c}$$
 I_ξ = moment of inertia about major axis, as defined in Section 2, Table 1, cm⁴ (in⁴)
 ξ_c = distance from major neutral axis to compressed flange, cm (in.)

$$C = \frac{\Gamma}{I_\eta} + \frac{K}{I_\eta} \frac{(kL)^2}{2.6\pi^2}$$
 E = modulus of elasticity, 2.06 × 10⁷ N/cm² (2.1 × 10⁶ kgf/cm², 30 × 10⁶ lbf/in²) for steel
 σ_F = σ_0 , specified minimum yield point for a compact section
 σ_F = σ_{Cx} , local buckling stress for a non-compact section, as specified in 2/9.7
 K = St. Venant torsion constant for the member, cm⁴ (in⁴)
 Γ = warping constant, cm⁶ (in⁶)
 L = member's length, cm (in.)
 k = effective length factor, as defined in 2/3.3

5 Members Subjected to Combined Loads

5.1 Axial Tension and Bending Moment

Members subjected to combined axial tension and bending moment are to satisfy the following equations at all cross-sections along their length:

For tubular members:

$$\frac{\sigma_t}{\eta_2 \sigma_0} + \frac{1}{\eta_2} \left[\left(\frac{\sigma_{by}}{\sigma_{CBY}} \right)^2 + \left(\frac{\sigma_{bz}}{\sigma_{CBZ}} \right)^2 \right]^{0.5} \leq 1$$

For rolled or fabricated-plate sections:

$$\frac{\sigma_t}{\eta_2 \sigma_0} + \frac{\sigma_{by}}{\eta_2 \sigma_{CBY}} + \frac{\sigma_{bz}}{\eta_2 \sigma_{CBZ}} \leq 1$$

where

- σ_t = axial tensile stress from 2/3.1, N/cm² (kgf/cm², lbf/in²)
- σ_{by} = bending stress from 2/3.5 about member y-axis, N/cm² (kgf/cm², lbf/in²)
- σ_{bz} = bending stress from 2/3.5 about member z-axis, N/cm² (kgf/cm², lbf/in²)
- σ_{CBY} = critical bending strength corresponding to member's y-axis from 2/3.5, N/cm² (kgf/cm², lbf/in²)
- σ_{CBZ} = critical bending strength corresponding to member's z-axis from 2/3.5, N/cm² (kgf/cm², lbf/in²)
- η_2 = allowable strength utilization factor for tension and bending, as defined in 1/11 and 2/1.9

5.3 Axial Compression and Bending Moment

Members subjected to combined axial compression and bending moment are to satisfy the following equation at all cross sections along their length:

For tubular members:

When $\sigma_a/\sigma_{CA} > 0.15$:

$$\frac{\sigma_a}{\eta_1 \sigma_{CA}} + \frac{1}{\eta_2} \left\{ \left[\frac{1}{\sigma_{CBY}} \frac{C_{my} \sigma_{by}}{1 - \sigma_a / (\eta_1 \sigma_{Ey})} \right]^2 + \left[\frac{1}{\sigma_{CBZ}} \frac{C_{mz} \sigma_{bz}}{1 - \sigma_a / (\eta_1 \sigma_{Ez})} \right]^2 \right\}^{0.5} \leq 1$$

When $\sigma_a/\sigma_{CA} \leq 0.15$:

$$\frac{\sigma_a}{\eta_1 \sigma_{CA}} + \frac{1}{\eta_2} \left[\left(\frac{\sigma_{by}}{\sigma_{CBY}} \right)^2 + \left(\frac{\sigma_{bz}}{\sigma_{CBZ}} \right)^2 \right]^{0.5} \leq 1$$

For rolled or fabricated-plate sections:

When $\sigma_a/\sigma_{CA} > 0.15$:

$$\frac{\sigma_a}{\eta_1 \sigma_{CA}} + \frac{1}{\eta_2 \sigma_{CBY}} \frac{C_{my} \sigma_{by}}{1 - \sigma_a / (\eta_1 \sigma_{Ey})} + \frac{1}{\eta_2 \sigma_{CBZ}} \frac{C_{mz} \sigma_{bz}}{1 - \sigma_a / (\eta_1 \sigma_{Ez})} \leq 1$$

When $\sigma_a/\sigma_{CA} \leq 0.15$:

$$\frac{\sigma_a}{\eta_1 \sigma_{CA}} + \frac{\sigma_{by}}{\eta_2 \sigma_{CBY}} + \frac{\sigma_{bz}}{\eta_2 \sigma_{CBZ}} \leq 1$$

where

- σ_a = axial compressive stress from 2/3.3, N/cm² (kgf/cm², lbf/in²)
 σ_{by} = bending stress from 2/3.5 about member y-axis, N/cm² (kgf/cm², lbf/in²)
 σ_{bz} = bending stress from 2/3.5 about member z-axis, N/cm² (kgf/cm², lbf/in²)
 σ_{CA} = critical axial compressive strength from 2/3.3, N/cm² (kgf/cm², lbf/in²)
 σ_{CBy} = critical bending strength corresponding to member y-axis from 2/3.5, N/cm² (kgf/cm², lbf/in²)
 σ_{CBz} = critical bending strength corresponding to member z-axis from 2/3.5, N/cm² (kgf/cm², lbf/in²)
 σ_{Ey} = Euler buckling stress corresponding to member y-axis, N/cm² (kgf/cm², lbf/in²)
 $\quad = \pi^2 E / (k_y L / r_y)^2$
 σ_{Ez} = Euler buckling stress corresponding to member z-axis, N/cm² (kgf/cm², lbf/in²)
 $\quad = \pi^2 E / (k_z L / r_z)^2$
 E = modulus of elasticity, 2.06×10^7 N/cm² (2.1×10^6 kgf/cm², 30×10^6 lbf/in²) for steel
 r_y, r_z = radius of gyration corresponding to the member y- and z-axes, cm (in.)
 k_y, k_z = effective length factors corresponding to member y- and z-axes from 2/3.3
 C_{my}, C_{mz} = moment factors corresponding to the member y- and z-axes, as follows:
 - i) For compression members in frames subjected to joint translation (sidesway):

$$C_m = 0.85$$
 - ii) For restrained compression members in frames braced against joint translation (sidesway) and with no transverse loading between their supports:

$$C_m = 0.6 - 0.4 M_1 / M_2$$

but not less than 0.4 and limited to 0.85, where M_1 / M_2 is the ratio of 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.
 - iii) For compression members in frames braced against joint translation in the plane of loading and subject 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.
For members whose ends are restrained:

$$C_m = 0.85$$
For members whose ends are unrestrained:

$$C_m = 1.0$$

η_1 = allowable strength utilization factor for axial compression (column buckling), as defined in Subsection 1/11 and 2/1.9
 η_2 = allowable strength utilization factor for tension and bending, as defined in Subsection 1/11 and 2/1.9

7 Tubular Members Subjected to Combined Loads with Hydrostatic Pressure

Appropriate consideration is to be given to the *capped-end actions* on a structural member subjected to hydrostatic pressure. It should be noted that the equations in this Subsection do not apply unless the criteria of 2/9.5 are satisfied first.

7.1 Axial Tension, Bending Moment and Hydrostatic Pressure

The following equation is to be satisfied for tubular members subjected to combined axial tension, bending moment and hydrostatic pressure:

$$\frac{\sigma_{tc}}{\eta_2 \sigma_{T\theta}} + \frac{\sqrt{\sigma_{by}^2 + \sigma_{bz}^2}}{\eta_2 \sigma_{CB\theta}} \leq 1$$

where

- σ_{tc} = calculated **axial tensile** stress due to forces from actions that include the capped-end actions due to hydrostatic pressure, N/cm² (kgf/cm², lbf/in²)
- $\sigma_{T\theta}$ = axial tensile strength in the presence of hydrostatic pressure, N/cm² (kgf/cm², lbf/in²)
= $C_q \sigma_0$
- $\sigma_{CB\theta}$ = bending strength in the presence of hydrostatic pressure, N/cm² (kgf/cm², lbf/in²)
= $C_q \sigma_{CB}$
- σ_{CB} = critical bending strength excluding hydrostatic pressure from 2/3.5
- C_q = $[\sqrt{1 + 0.09B^2} - B^{2\xi} - 0.3B]$
- B = $\sigma_\theta / (\eta_\theta \sigma_{C\theta})$
- ξ = $5 - 4\sigma_{C\theta} / \sigma_0$
- σ_θ = hoop stress due to hydrostatic pressure from 2/9.5, N/cm² (kgf/cm², lbf/in²)
- $\sigma_{C\theta}$ = critical hoop buckling strength from 2/9.5, N/cm² (kgf/cm², lbf/in²)
- σ_0 = specified minimum yield point, N/cm² (kgf/cm², lbf/in²)
- η_2 = allowable strength utilization factor for tension and bending, as defined in Subsection 1/11 and 2/1.9
- η_θ = allowable strength utilization factor for local buckling in the presence of hydrostatic pressure, as defined in Subsection 1/11 and 2/1.9

7.3 Axial Compression, Bending Moment and Hydrostatic Pressure

Tubular members subjected to combined compression, bending moment and external pressure are to satisfy the following equations at all cross sections along their length.

When $\sigma_{ac} / \sigma_{CA\theta} > 0.15$ and $\sigma_{ac} > 0.5\sigma_\theta$:

$$\frac{\sigma_{ac} - 0.5\sigma_\theta}{\eta_1 \sigma_{CA\theta}} + \frac{1}{\eta_2 \sigma_{CB\theta}} \left\{ \left[\frac{C_{my} \sigma_{by}}{1 - \frac{\sigma_{ac} - 0.5\sigma_\theta}{\eta_1 \sigma_{Ey}}} \right]^2 + \left[\frac{C_{mz} \sigma_{bz}}{1 - \frac{\sigma_{ac} - 0.5\sigma_\theta}{\eta_1 \sigma_{Ez}}} \right]^2 \right\}^{0.5} \leq 1$$