

## The new 2005 AISC specification

(A nova norma americana do AISC 2005)

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### Resumo

No final de 2005, o American Institute of Steel Construction editou a versão mais recente da norma norte-americana para estruturas de edifícios em aço, *Specification for Structural Steel Buildings* (ANSI/AISC 360-05). Essa norma inclui prescrições para projetos com base nos métodos das tensões admissíveis (ASD) e dos estados-limites (LRFD) e incorpora, ainda, prescrições para seções tubulares e cantoneiras. Entre as mais importantes modificações destacamos a completa renovação das metodologias para a verificação da estabilidade de estruturas apertadas, novas prescrições para pilares mistos e atualização das especificações dos materiais. O presente artigo descreve essas alterações e destaca aquelas com interesse prático.

**Palavras-chave:** Projeto em aço, normas, resistência última, tensão admissível.

### Abstract

*In late 2005, the American Institute of Steel Construction issued its most recent Specification for Structural Steel Buildings (ANSI/AISC 360-05). This specification includes updated design provisions in both allowable strength design (ASD) and load and resistance factor design methods (LRFD), and incorporates the design provisions for hollow structural sections and single angles. Amongst the major changes are a complete revamping of the methodologies for assessing stability of framed structures, new provisions for composite columns and updated material requirements. This paper will describe the changes and highlight those of practical significance.*

**Keywords:** Steel design, specifications, codes, ultimate strength, allowable stress.

## 1. Introduction

For the past 20 years, the American Institute of Steel Construction (AISC) has maintained two different specifications: a limited state or ultimate strength one (AISC LRFD 1999), whose first edition dates to 1986, and an allowable or working stress design one (AISC ASD 1989), whose last edition dates to 1989. In the USA, although limited state design of reinforced concrete structures has been common since the early 1960's, design for metal structures has remained for the most part under the allowable stress approach. In this last cycle of its main specification, AISC decided that it could no longer afford to maintain two separate specifications, and decided to develop a set of unified provisions that merged both approaches. With the new Specifications, AISC intends to consolidate its design provisions into four broad documents: basic requirements (AISC 2005a/ANSI 360-05), seismic design (AISC Seismic 2005b/ANSI 341-05), nuclear design (AISC 2006/N690) and contractual provisions (Code of Standard Practice for Structural Steel Buildings, AISC 2005c).

In order to rationalize the design process along an ultimate strength design approach, the 2005 version of this specification contains a dual set of provisions. One set of provisions will follow closely the current Load and Resistance Factor Design (LRFD) format and the other set will follow an Allowable Strength Design (henceforth, NEW ASD) approach. Although the NEW ASD bears a similar name to the older Allowable Stress Design (henceforth, OLD ASD), the two are not to be confused. In the 2005 AISC Specification, a single expression will be given for the nominal resistance of a member or component ( $R_n$ ), and that resistance will be reduced by a resistance factor ( $\phi$ ) for LRFD design or divided by a safety factor ( $\Omega$ ) for the NEW ASD. The general format is:

For LRFD:

$$R_u \leq \phi R_n$$

For ASD:

$$R_a \leq \frac{R_n}{\Omega}$$

Note that the nomenclature maintains the use of the terms resistance and safety factors. While the origin of the former is clear and rooted in reliability theory, the latter is based mostly on experience and cannot be given a consistent meaning across all forms of loading. Thus, while for simple loading conditions such as tension, the NEW ASD and OLD ASD are the same, this is not a statement that can be generalized to the rest of the code.

The procedure for determining the new ASD safety factors (Duncan 2006) is based on the concept of an effective load factor. The effective load factor,  $\gamma$ , is determined by setting the ASCE 7 (ASCE, 2005) LRFD load combination - for live load ( $L$ ) and dead load ( $D$ ) only - equal to the equivalent ASD load combination. An effective load factor,  $\gamma$ , may be determined for the ASD side of the equation as follows.

$$1.2D + 1.6L = \gamma(L + D) \quad (1)$$

$$1.2 + 1.6(L/D) = \gamma((L/D) + 1) \quad (2)$$

where

$\gamma$  = effective load factor.

The AISC LRFD *Specification for Structural Steel Buildings* (1986) was originally calibrated to the OLD ASD at  $L/D = 3$ . For  $L/D = 3$ , Equation 2 yields

$$1.2 + 4.8 = \gamma \quad (4)$$

$$\gamma = 6/4 = 1.5$$

Therefore, for calibration at  $L/D = 3$  with  $\gamma = 1.5$  as the target effective load factor, and using the following inequality that the design strength ( $\phi R_n$ ) must equal or exceed the required strength ( $\gamma R$ ), and solving for the safety factor,  $\Omega$ , yields:

$$\phi R_n \geq \gamma R$$

$$\Omega = \frac{R_n}{R} \geq \frac{\gamma}{\phi} = \frac{1.5}{\phi}$$

In the new AISC Manual, color-coded tables are used to highlight the difference between LRFD and the NEW ASD. Figure 1 shows one such table for the flexural capacity of beams. The blue color for LRFD and the highlighted green background for ASD clearly separate the two design cases.

In addition to supporting two different approaches, at least the following major editorial changes in the new AISC Specification should be highlighted:

- 1. Nomenclature:** An attempt has been made to coordinate the nomenclature to a standard one to be adopted by all metal specifications. This was achieved through the work of a joint committee of the American Institute of Iron and Steel (AISI) and AISC, and gives greater clarity and transparency to this code.
- 2. Mandatory language:** All non-mandatory language has been eliminated, and very few locations remain where designers are given a choice of whether to use or not a particular clause of the code.
- 3. Loading:** All references to loads have been eliminated; all loading is taken from ASCE 7-05 (ASCE 7, 2005). This means that AISC has now completely decoupled the loading and resistance part of the design process, and loads have become material independent as they should be.
- 4. New section types:** All requirements pertaining to single angles and pipe/hollow structural (HSS) sections have been incorporated into the main Specification. These two types of sections were covered in separate documents in the past. While the consolidation of all the rules into the main specification was deemed as very desirable, it has led to some very large chapters. For example, incorporation of these section types has nearly doubled the size of Chapter F - Flexure.

5. **Reorganization:** The content has been extensively reorganized, so that each topic is clearly addressed in a chapter. For example, there is now a chapter on shear (Chapter G) that groups provisions that were scattered through many chapters in previous codes (mainly section F2). In previous editions, Chapter G was very short and only addressed plate girders. In general, one can say that each chapter now addresses a specific type of failure mode.
6. **Importance:** In addition to the reorganization of the material along failure modes, the chapters are organized such that the more commonly used provisions are at the front. The intent of the Specification committee was that even though a topic may cover many pages, 90% of the everyday design requirements are covered by the first few pages of each chapter.
7. **User Notes:** The new specification makes extensive use of “user notes.” These are the equivalent of a quick commentary or clarification, and are embedded inside the specification although they are not legally part of it. The intent of these notes, which are highlighted inside a gray box in the text, is to provide designers with guidance where legal requirements force an arcane wording of the provisions. An example of its use is in Chapter D (Tension) where the limitations on the maximum slenderness have been removed from the Specification. A user note has been inserted to indicate that this limit should preferably not exceed  $L/300$ , but that this recommendation and does not apply to rods or hangers in tension.
8. **Commentary:** The commentary has been completely rewritten to provide more useful information to designers. Where possible, all historical descriptions have been eliminated and emphasis has been put on documenting why changes have been made.

In the next sections of this paper, some of the more notable changes are discussed. The sections referenced are given in parenthesis after the title; issues related to stability (Chapter C and Appendices 1 and 7) are left to the latter parts of the paper because they are the more complex. The entire provisions and commentary are downloadable free from [www.aisc.org](http://www.aisc.org). Designers are directed to the new AISC Manual and its accompanying CD, which contains hundreds of design examples and tables, for more details. Finally, it should be noted that this is the first steel specification issued under the American National Standards Institute (ANSI) approval. The ANSI process insures a more public development of the provisions, a careful documentation of the changes made, and the development of future specifications under a consistent basis.

## 2. Highlights of new 2005 AISC specification

### 2.1 Scope (Section A1)

A major change in the scope of the 2005 Specification is the elimination from this section of the traditional construction types (FR and PR for LRFD and Types 1 through 3 for ASD). Differentiation between different construction types is now embedded in appropriate sections, such as in Chapter C for issues related to stability. In addition to this major change in approach, the new Specification now explicitly recognizes its applicability for design in areas of low to moderate seismicity (Seismic Design Categories A through C). This is a significant change because as new areas of the USA become zoned at higher seismic levels, this clause intends to alert designers that traditional steel structural systems can still be used.

**Table 3 -2 (continued)**

**$F_y = 50$  ksi**

**W Shapes**

**Selection by  $Z_x$**

$Z_x$

| Shape          | $Z_x$<br>in. <sup>3</sup> | $M_{rx} / \Omega_c$ |            | $M_{rx} / \Omega_c$ |            | BF          |             | $L_p$<br>ft | $L_r$<br>ft | $I_x$<br>in. <sup>4</sup> | $V_{rx} / \Omega_c$ |            |
|----------------|---------------------------|---------------------|------------|---------------------|------------|-------------|-------------|-------------|-------------|---------------------------|---------------------|------------|
|                |                           | kip-ft              | kip-ft     | kip-ft              | kip-ft     | kips        | kips        |             |             |                           | Kips                | kips       |
|                |                           | ASD                 | LRFD       | ASD                 | LRFD       | ASD         | LRFD        |             |             |                           | ASD                 | LRFD       |
| <b>W21x55</b>  | <b>126</b>                | <b>314</b>          | <b>473</b> | <b>192</b>          | <b>289</b> | <b>10.8</b> | <b>16.3</b> | <b>6.11</b> | <b>17.4</b> | <b>1140</b>               | <b>156</b>          | <b>234</b> |
| W14x74         | 126                       | 314                 | 473        | 196                 | 294        | 5.34        | 8.03        | 8.76        | 31.0        | 795                       | 128                 | 191        |
| W18x60         | 123                       | 307                 | 461        | 189                 | 284        | 9.64        | 14.5        | 5.93        | 18.2        | 984                       | 151                 | 227        |
| W12x79         | 119                       | 297                 | 446        | 187                 | 281        | 3.77        | 5.67        | 10.8        | 39.9        | 662                       | 116                 | 175        |
| W14x68         | 115                       | 287                 | 431        | 180                 | 270        | 5.20        | 7.81        | 8.69        | 29.3        | 722                       | 117                 | 175        |
| W10x88         | 113                       | 282                 | 424        | 172                 | 259        | 2.63        | 3.95        | 9.29        | 51.1        | 534                       | 131                 | 197        |
| <b>W18x55</b>  | <b>112</b>                | <b>279</b>          | <b>420</b> | <b>172</b>          | <b>258</b> | <b>9.26</b> | <b>13.9</b> | <b>5.90</b> | <b>17.5</b> | <b>890</b>                | <b>141</b>          | <b>212</b> |
| <b>W21x50</b>  | <b>110</b>                | <b>274</b>          | <b>413</b> | <b>165</b>          | <b>248</b> | <b>12.2</b> | <b>18.3</b> | <b>4.59</b> | <b>13.6</b> | <b>984</b>                | <b>158</b>          | <b>237</b> |
| W12x72         | 108                       | 269                 | 405        | 170                 | 256        | 3.72        | 5.59        | 10.7        | 37.4        | 597                       | 105                 | 158        |
| <b>W21x48'</b> | <b>107</b>                | <b>265</b>          | <b>398</b> | <b>162</b>          | <b>244</b> | <b>9.78</b> | <b>14.7</b> | <b>6.09</b> | <b>16.6</b> | <b>959</b>                | <b>144</b>          | <b>217</b> |
| W16x57         | 105                       |                     | 394        |                     | 242        |             | 12.0        | 5.65        | 18.3        | 758                       |                     | 212        |
| W14x61         | 102                       | 254                 | 383        | 161                 | 242        | 4.96        | 7.46        | 8.65        | 27.5        | 640                       | 104                 | 156        |
| W18x50         | 101                       | 252                 | 379        | 155                 | 233        | 8.69        | 13.1        | 5.83        | 17.0        | 800                       | 128                 | 192        |
| W10x77         | 97.6                      | 244                 | 366        | 150                 | 225        | 2.59        | 3.90        | 9.18        | 45.2        | 455                       | 112                 | 169        |
| W12x65'        | 96.8                      | 237                 | 356        | 154                 | 231        | 3.60        | 5.41        | 11.9        | 35.1        | 533                       | 94.5                | 142        |
| <b>W21x44</b>  | <b>95.4</b>               | <b>238</b>          | <b>358</b> | <b>143</b>          | <b>214</b> | <b>11.2</b> | <b>16.8</b> | <b>4.45</b> | <b>13.0</b> | <b>843</b>                | <b>145</b>          | <b>217</b> |
| W16x50         | 92.0                      | 230                 | 345        | 141                 | 213        | 7.59        | 11.4        | 5.62        | 17.2        | 659                       | 124                 | 185        |
| W18x46         | 90.7                      | 226                 | 340        | 138                 | 207        | 9.71        | 14.6        | 4.56        | 13.7        | 712                       | 130                 | 195        |

Figure 1 - Example design table highlighting differences in LRFD and ASD values.

## 2.2 Materials (Sections A2-A3)

The confusing collection of material and codes referenced in Chapter A of previous specifications has been completely revamped, with standards clearly labeled and material specifications grouped by type. Given the great variety of steel grades available throughout the world and the fact that a designer is unlikely to know the source of the steel that will be used in a particular project, AISC has taken great care to limit the Specification to the more common ASTM grades, including the new ASTM 913 and 992. Perhaps the greatest changes in the materials area are (1) the allowance of materials with yield points up to 100 ksi (about 700 MPa) and (b) the requirements for a minimum impact resistance for thick sections to be welded using complete joint penetration welds (20 ft-lbs at 70° F / 27 J 21° C).

In addition, the new specification takes into account the large differences in properties due to the manufacturing process. For example, the fact that some of the hollow structural section (HSS) grades (ASTM A500, Grade A) do not meet the ductility requirements, necessitated that the usable stress be limited to 80% of its yield stress.

## 2.3 Design Requirements (Chapter B)

Several important changes have occurred in this chapter. Amongst them are:

1. Section B2 now contains no information on load combinations. For all such information, the reader is referred to ASCE 7. A user note reminds the reader of the ASCE 7 section where load combinations can be found for either LRFD or ASD design. This is important because the load combination for ASD design have been considerably expanded in the latest ASCE 7 document, effectively eliminating the large disparity between the design methods inherent in the existing LRFD and OLD ASD approaches.

2. Sections B3 and B4 give concise but complete descriptions of the dual approaches (LRFD and NEW ASD) used in the specification.

3. Section B6 now contains a clear description of connection types, which are now divided into simple and moment connections, with the latter further subdivided into fully (PR) and partially restrained (PR) ones. It allows the designer to use simple and rigid conditions in the analysis of most connections, but requires the explicit use of moment-rotation curves in the analysis of PR frames.

4. Sections 7 through 11 explicitly require that the designer account for serviceability, ponding, fatigue, fire, and corrosion in the design. These sections do not contain specific requirements for these conditions; those are given in other parts of the Specification (Chapter L for serviceability and Appendix IV for fire, for example). While a number of these have been present in the Specification for some time, they had been relegated to later Chapter or Appendices, often falling “below the radar screen” of many designers. By explicitly calling for their incorporation into the basic design requirements, the Specification now makes clear that many limit states, besides strength, need to be considered.

5. The requirements for local buckling (formerly Section B5 and now Section B4) have been considerably expanded and clarified.

6. Section B6 on evaluation of existing structures has now been reduced to a reference to the new Appendix 5, which contains all the provisions. In the past, the move to an Appendix has implied considerable interest in the topic and the possible future development of a stand-alone specification. There is now a new Appendix on fire design (Appendix 4), which may in the future also be a separate document or incorporated into the main Specification.

## 2.4 Compression (Chapter E)

One of the main logistical decisions in the development of the 2005 Specification hinged on the choice of the format of the equations for compression, which is the first difficult design topic to be addressed by the Specification. While the LRFD and OLD ASD curves were very similar, they were specified in different formats and units. Those for LRFD were couched mostly on a strength basis (units of load) while those for the OLD ASD were on a stress basis. The new Specification opted to adopt the stress format but using the existing LRFD equations with a non-dimensional slenderness parameter (function of  $\sqrt{E/F_y}$ ). The specification has removed all limits on slenderness ratios, includes ASD provisions for singly and non-symmetrical members, and includes all the necessary provisions for the design of slender members within the chapter.

## 2.5 Flexure (Chapter F)

The new specification has collapsed the 5 equations based on the unbraced length of the compression flange in the OLD ASD into three equations based on lateral-torsion buckling conditions similar to those in the existing LRFD. In most cases, this has resulted in increases in capacity for members designed under the ASD approach and on the removal of a number of the step functions that characterized the design of some wide flange shapes under the OLD ASD procedures (see Figure 2). The equations have also been changed so that the flexural breakpoint between inelastic and elastic buckling ( $M_r$ ) has been substituted by the much simpler expression of  $0.7S_x F_y$ .

The chapter now covers all types of members, including single angles and square and rectangular hollow sections (HSS), as well as unsymmetrical shapes. A especially useful user note, in the form of a table (Figure 3), now clearly gives designers guidance on the applicability of different failure modes and relevant clauses of the chapter to the flexural

design of all different types of members. Thus, while the Chapter itself appears as more dense and complicated in the new version, its usability has been considerably improved by careful attention to the governing failure modes.

## 2.6 Shear (Chapter G)

Chapter G, which used to cover only the design of plate girders, now has condensed all the shear design provisions (including the old section F2 which governed the shear design for most other members) into a single section. The provisions have been calibrated to the OLD ASD one, with the result that while expressions similar to those in the existing LRFD are used, the limits and resistance factors have changed in some cases.

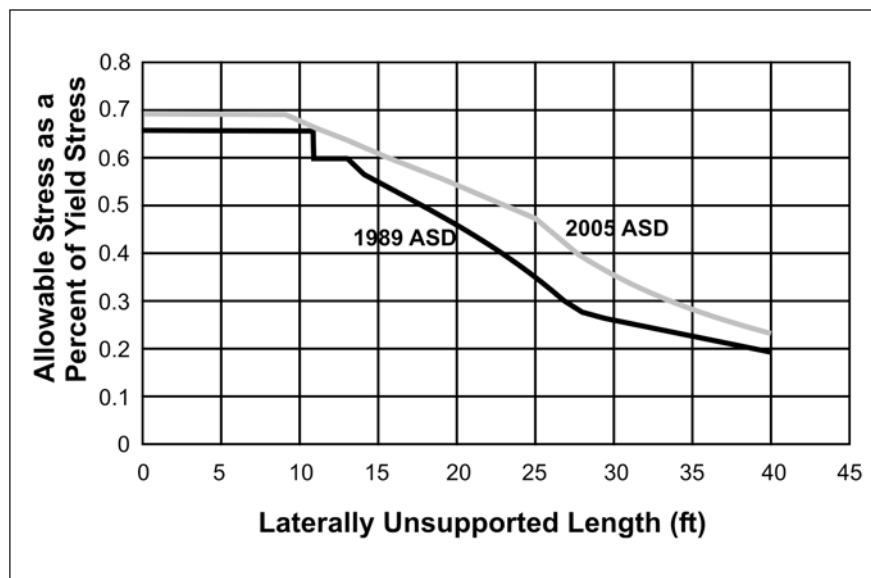
Probably the most striking change for those used to the LRFD format, is the use of a  $\phi_v = 1.00$  (and corresponding  $\Omega_v = 1.50$ ) for the shear capacity of doubly-symmetric sections and channel sections. In addition, provisions are now given for the shear strength of HSS sections as follows:

$$V_n = \frac{A_g F_{cr}}{2}$$

where  $F_{cr}$  is the larger of:

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D} \left(\frac{D}{t}\right)^4}} \quad \text{or} \quad F_{cr} = \frac{0.78E}{\left(\frac{D}{t}\right)^2}$$

and  $A_g$  is the gross area based on the design wall thickness,  $D$  is the outside diameter,  $L_v$  is the distance from maximum to zero shear force, and  $t$  is the design wall thickness (equal to the nominal wall thickness for submerged arc welded (SAW) and 0.93 for electric resistance welded (EWS) HSS).



**Figure 2** - Comparison of OLD ASD and the unified 2005 Specification for the flexural capacity of a W36 x 182,  $F_y = 50$  ksi beam (W920x271, 350 MPa)

## 2.7 Interaction (Chapter H)

Chapter H addresses members subjected to axial force and bending about one or both axes, with or without torsion, and members subject to torsion alone. The chapter basically preserves the existing LRFD approach, in which the interaction between flexure and axial load is handled by equations involving the sum of the ratios of the required to the available strengths. However, new equations have been developed in conjunction with the stability provisions of Chapter C. For an unsymmetrical section, the summation is for the stresses rather than strengths. For double-symmetric sections under uniaxial bending and axial load, the new specification permits the separation into two independent limit states (in-plane instability and out-of-plane instability/flexural torsion buckling).

## 2.8 Composite members (Chapter I)

The 2005 version of Chapter I includes both extensive technical and format changes as well as significant new material when compared to previous editions. The major technical change consist of new design provisions for composite columns (Section I2), which now includes new cross-sectional strength models, provisions for tension and shear design, and a liberalization of the slenderness limits for HSS tubes and pipes. Other significant technical changes are the new, more rational shear stud strengths values for design (Section I3.2d), the use of an ultimate strength model for ASD design of composite beams (I3.2), and new material limits usable for design (Section I1.2).

The new provisions require that the strength of composite sections shall be computed based on first principles of mechanics and robust constitutive models for materials. Two approaches are given to satisfy this requirement. The first is the strain compatibility approach, which provides a general method. The second is the plastic stress distribution

approach, which is a subset of the strain compatibility one. The plastic stress distribution model provides a simple and convenient calculation method for the most common design situations, and is thus treated first.

An example of the differences in design strength for encased composite columns is given in Figure 4, which contrasts the capacities given by current reinforced concrete provisions (ACI 318, 2005) and the 1999 LRFD (AISC 1999) ones with two versions proposed in the 2005 Specification (AISC 2005a). The AISC 2005 (1) curve corresponds to the polygonal approach pioneered in the Eurocodes and adopted into the 2005 specification, and the AISC 2005 (2) shows a simplified bi-linear approximation useful for design.

### 2.9 Stability (Chapter C)

Chapter C, which covers stability provisions, is probably the most radically changed chapter in the Specification. While the older methods (the effective length method for individual members, for example) remain valid, Chapter C takes an important step towards recognizing advanced analysis and design techniques rendered possible by personal computers and advanced commercial software. The chapter is the result of several years of work of a joint AISC - SSRC (Structural Stability Research Council) committee that examined a number of techniques ranging from full-advanced analysis including geometrical and material non-linearities to the nominal load concepts such as those incorporated in the Australian and European codes. The General Requirements to Chapter C state it as follows<sup>1</sup>:

*Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of second-order effects (including  $P-\Delta$  and  $P-\delta$  effects), flexural, shear and axial deformations, geometric imperfections, and member stiffness reduction due*

*to residual stresses on the stability of the structure and its elements is permitted. The methods prescribed in this Chapter and Appendix 7 satisfy these requirements. All component and connection deformations that contribute to the displacements shall be considered in the stability analysis.*

<sup>1</sup> Parts of the Specification and Commentary are included here because precise language is needed in order not to confuse the reader; thus rather than paraphrasing these parts with the possibility of distorting the meaning, parts of the introductory material to Chapter C and Appendix 7 are reproduced verbatim in Sections 2.9 and 2.10.

**Table User Note F1.1**  
**Selection Table for the Application of Chapter F Sections**

| Section in Chapter F | Cross Section                    | Flange Slenderness | Web Slenderness | Limit States     |
|----------------------|----------------------------------|--------------------|-----------------|------------------|
| F2                   |                                  | C                  | C               | Y, LTB           |
| F3                   |                                  | NC                 | C               | LTB, FLB         |
| F4                   |                                  | S                  | C               | LTB, FLB         |
| F4                   |                                  | C, NC, S           | NC              | LTB, FLB, TFY    |
| F5                   |                                  | C, NC, S           | S               | LTB, FLB, TFY    |
| F6                   |                                  | C, NC, S           | N/A             | FLB              |
| F7                   |                                  | C, NC, S           | C, NC           | Y, FLB, WLB      |
| F8                   |                                  | N/A                | N/A             | Y, LB            |
| F9                   |                                  | C, NC, S           | N/A             | Y, LTB, FLB      |
| F10                  |                                  | N/A                | N/A             | Y, LTB, LLB      |
| F11                  |                                  | N/A                | N/A             | Y, LTB           |
| F12                  | Unsymmetrical shapes             | N/A                | N/A             | All Limit States |
| F13                  | Proportions of beams and girders | N/A                | N/A             | N/A              |

Y = Yielding, LTB = Lateral-Torsional Buckling, FLB = Flange Local Buckling, TFY = Tension Flange Yielding, LLB = Leg Local Buckling, C = Compact, NC = Noncompact, S = Slender

**Figure 3** - User note from the beginning of Chapter F indicating applicability of different clauses and failure modes in design.

In structures designed by elastic analysis, individual member stability and stability of the structure as a whole are provided jointly by:

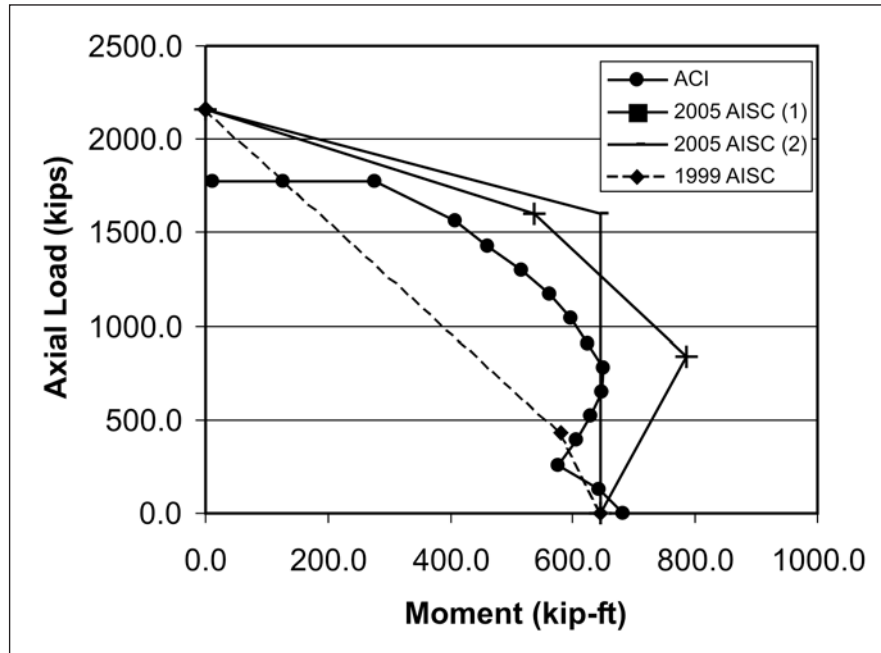
(1) Calculation of the required strengths for members, connections and other elements using one of the methods specified in Section C2.2, and

(2) Satisfaction of the member and connection design requirements in this specification based upon those required strengths.

In structures designed by inelastic analysis, the provisions of Appendix I shall be satisfied.

The commentary says that:

The stability of structures must be considered from the standpoint of the structure as a whole, including not only compression members, but also beams, bracing systems, and connections. The stability of individual components also must be provided. Considerable attention has been given in the technical literature to this subject, and various methods are available to assure stability (Galambos, 1998). In all approaches, the method of analysis and the equations for checking of component resistances are inextricably interlinked. Traditionally, the effects of unavoidable geometric imperfections (within fabrication and erection tolerances) and distributed yielding at strength limit states (including residual stress effects) are addressed solely within member resistance equations. Correspondingly, the structural analysis is conducted using the nominal structure geometry and elastic stiffness. The Specification addresses this traditional approach, termed the Effective Length Method in this commentary, as well as a new approach which is termed the Direct Analysis Method and is addressed in Appendix 7. The Direct Analysis Method includes



**Figure 4** - Comparison of the design strength for a 24 in. x 24 in. encased column ( $f'_c = 5$  ksi) with 4 # 8 bars and a Grade 50 W10x49 section (610 mm x 610mm concrete column (35 MPa) with 4 D25 bars and a W250x73 column (350MPa).

nominal geometric imperfection and stiffness reduction effects directly within the structural analysis. In either the Effective Length or the Direct Analysis approaches, the structural analysis by itself is not sufficient to ensure the stability of the structure as a whole. The overall stability of the structure as well as the stability of individual elements is ensured by the combined calculation of the required strengths by structural analysis and the satisfaction of the member and connection design provisions of the Specification.

In general, it is essential that an accurate second-order analysis of the structure be performed. The analysis should in general consider the influence of second-order effects (including  $P-\Delta$  and  $P-\delta$  effects as shown in Figure C-C1.1), flexural, shear, and axial deformations. More rigorous analysis methods allow formulation of simpler limited state models. One such example can be found in Appendix 7 where the new Direct Analysis Method is presented

as an alternative method to improve and simplify design for stability. In this case, the inclusion of nominal geometric imperfection and member stiffness reduction effects directly in the analysis allows the use of  $K = 1.0$  in calculating the in-plane column strength  $P_n$  within the beam-column interaction equations of Chapter H. This simplification comes about because the Direct Analysis Method provides a better estimate of the true internal forces within the structure. The Effective Length Method, in contrast, includes the above effects indirectly within the member resistance equations.

## 2.10 Direct Design Method (Appendix 7)

The commentary to Appendix 7 says:

The Direct Analysis Method, addresses a new method for the stability analysis and design of structural steel systems comprised of moment frames, braced frames, shear

walls or combinations thereof (AISC-SSRC, 2003a). While the precise formulation of the method is unique to the AISC Specification, some features of it have similarities to other major design specifications around the world including the Eurocodes, the Australian Standard, the Canadian Standard, and ACI 318.

The Direct Analysis Method has been developed with the goal to more accurately determine internal forces in the structure in the analysis stage and to eliminate the need for calculating the effective buckling length (K factor) for columns in the first term of the beam-column interaction equations. This method is, therefore, a major step forward in the design of steel moment frames from past versions of the specification. In addition, it can be used for the design of braced frames and combined frame systems. Thus, this one method can be used for the design of all types of steel framed structures used in practice. The method can be expanded in the future beyond its use as a second-order elastic analysis tool as presented here. For example, it can be applied with inelastic or plastic analysis. Also, it can be used in the analysis of composite and hybrid

structures, although this application is not explicitly addressed in this edition.

Chapter C requires that the Direct Analysis Method, as described herein, be used wherever the value of the sidesway amplification  $\Delta_{2nd\ order} / \Delta_{1st\ order}$  (or  $B_2$  from Equation C2-3), determined from a first-order analysis of the structure, exceeds 1.5. It may also be used in lieu of the methods described in Chapter C for the analysis and design of any lateral load resisting frame in a steel building.

Some of the most important assumptions embedded in the direct design method are:

1. Initial member out-of-straightness is  $L/1000$ , where  $L$  is the length of the member.
2. The initial frame out-of-plumbness for a story is assumed as  $H/500$ , where  $H$  is the story height of the building.
3. The total out-of-plumbness of the structure is bounded by the limits given in the Code of Standard Practice.
4. The limit states considered include cross section yielding, local

buckling, flexural buckling, and lateral-torsion or torsion-flexure buckling.

5. For the in-plane buckling check, the effective length can be assumed to be equal to 1.
6. Residual stresses are assumed to be linearly distributed and have a maximum value of  $0.3 F_y$  at the flange tips.

The direct design method requires that:

1. A rigorous second-order analysis be conducted that accounts for both  $P-\Delta$  and  $P-\delta$  effects.
2. The application of a notional load  $N_i = 0.002 Y_i$  where  $Y_i$  is the gravity load from the appropriate load combination acting on level  $i$ .
3. That the analysis be based on a reduced stiffness ( $EI^* = 0.8\tau_b EI$  and  $EA^* = 0.8EA$ ) in the structure.

Figure 5 shows a comparison of the in-plane beam-column interaction checks using the two main different methods allowed in Chapter C. On the left part (Figure 5(a)), the conventional effective length approach is shown. In this case, the yield strength ( $P_y$ ) is reduced by the conventional approach (kL effect) and the usual interaction equations applied. On the right side of the figure (Figure

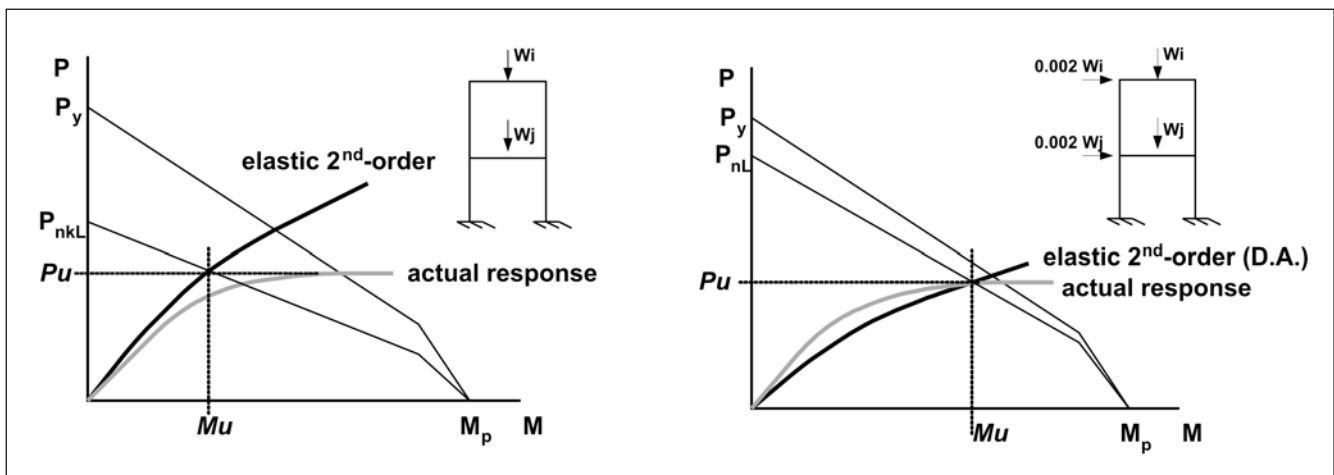


Figure 5 - Comparison of in-plane interaction checks for (a) the Effective Length Method and (b) the Direct Analysis Method (Figure C-A-7.1 in AISC 2005a).



5(b)), the direct analysis method is shown. Note that for this case, because of the reductions on the axial and flexure stiffness of the members, the elastic second-order response falls below the actual response for much of the load path.

## 2.11 Inelastic Analysis and Design (Appendix 1)

*Inelastic analysis* is permitted for LRFD but not ASD design. As the design is governed by the ductility of the plastic hinge zones, the *specified minimum yield stress* for members undergoing plastic hinging shall not exceed 65 ksi (450 MPa) and the compactness criteria are those of Table B4.1 for  $\lambda_p$  modified as follows:

(a) For webs of doubly-symmetric wide flange members and rectangular HSS in combined flexure and compression

(i) for  $P_u/\phi_b P_y \leq 0.125$

$$h/t_w \leq 3.76 \sqrt{\frac{E}{F_y} \left( 1 - \frac{2.75 P_u}{\phi_b P_y} \right)}$$

(ii) for  $P_u/\phi_b P_y > 0.125$

$$h/t_w \leq 1.12 \sqrt{\frac{E}{F_y} \left( 2.33 - \frac{P_u}{\phi_b P_y} \right)} \geq 1.49 \sqrt{\frac{E}{F_y}} \quad (\text{A-1-2})$$

(b) For flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression, flange *cover plates* and *diaphragm plates* between lines of *fasteners* or *welds*:

$$b/t \leq 0.94 \sqrt{E/F_y}$$

(c) For circular hollow sections in flexure:

$$D/t \leq 0.045 E/F_y$$

The laterally *unbraced length*,  $L_p$ , of the compression flange adjacent to *plastic hinge* locations shall not exceed  $L_{pd}$ , determined as follows.

(a) For doubly symmetric and singly symmetric I-shaped members with the compression flange equal to or larger than the tension flange loaded in the plane of the web:

$$L_{pd} = \left[ 0.12 + 0.076 \frac{M_1}{M_2} \right] \frac{E}{F_y} r_y$$

where

$M_1$  = smaller moment at end of *unbraced length* of beam, kip-in. (N-mm)

$M_2$  = larger moment at end of *unbraced length* of beam, kip-in. (N-mm)

$r_y$  = radius of gyration about minor axis, in. (mm)

$(M_1/M_2)$  is positive when moments cause *reverse curvature* and negative for *single curvature*.

(b) For solid rectangular bars and symmetric box *beams*:

$$L_{pd} = \left[ 0.17 + 0.10 \frac{M_1}{M_2} \right] \frac{E}{F_y} r_y \geq 0.10 \frac{E}{F_y} r_y \quad (\text{A-1-6})$$

There is no limit on  $L_b$  for members with circular or square cross sections or for any *beam* bent about its minor axis.

The required axial strength of members subjected to plastic hinging in combined flexure and axial compression shall not exceed  $0.7A_g F_y$ . The column slenderness ratio  $L/r$  of members subjected to combined flexure and axial compression shall not exceed  $4.71 \sqrt{E/F_y}$ .

*Connections* shall be designed with sufficient strength and ductility to sustain the *forces* and *deformations* imposed under the required *loads*.

**Stability:** Second-order effects may be neglected in frames where the ratio of the elastic buckling load to the loading requirements specified in Section B3.3

$$\lambda_{cr} \geq 10 \quad (\text{A1-7})$$

For  $5 \leq \lambda_{cr} \leq 10$ , second-order effects may be neglected provided that the design load effects are amplified by the factor:

$$AF = \frac{0.9}{1 - \frac{1}{\lambda_{cr}}}$$

For  $\lambda_{cr} < 5$ , the design shall be based on a second-order inelastic analysis. Two options are given: direct elastic-plastic hinge analysis or distributed plasticity.

**Direct analysis:** The requirements for direct elastic-plastic hinge analysis are governed by the provisions of Appendix 7, with the following modifications:

- (1) Inelastic redistribution is permitted in members satisfying the provisions of Section 1.2. Moments shall be redistributed based on the assumption of elastic-perfectly plastic hinge response at the member resistance defined by the provisions of Appendix 7.
- (2) The *notional load*  $N_i$  shall be applied as an additive lateral load for all *load combinations*.

**Distributed plasticity:** A second-order inelastic analysis that takes into account the relevant material properties, geometric imperfections, residual stresses, partial yielding of

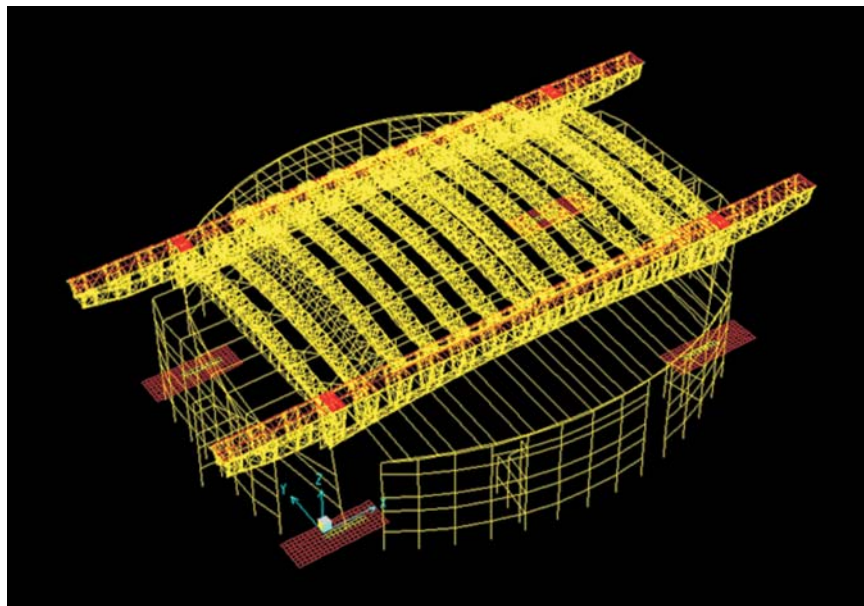
member cross sections and connection force-deformation characteristics may be used to assess the design strength of structural frames, provided that the analysis is shown to model the behavior and the following requirements are satisfied:

- (1) All contributions to the elastic stiffness of the structure shall be reduced by a factor of 0.9.
- (2) The material strength used in the analysis shall be reduced by a factor of 0.9.
- (3) Unless member torsion-flexure instabilities are captured by the analysis, doubly-symmetric I-section members shall satisfy Eq. H1-2 and all other members shall satisfy Eq. H1-1 for the limited state of out-of-plane buckling.

Moments shall be redistributed based on the assumption of elastic-perfectly plastic hinge response at the member resistance defined by the provisions of Section H1, with the nominal *column* strengths,  $P_n$ , determined using  $K = 1.0$ . For members where the required axial strength is less than  $0.15P_y$ , moments may be redistributed based on the assumption of elastic-perfectly plastic hinge response at a member resistance of  $\phi_b M_p$ , where  $\phi_b = 0.9$ .

### 3. Conclusions

The 2005 AISC Specification represents a significant improvement over previous editions from both the editorial and technical standpoint. From the editorial standpoint, the nomenclature has been tightened and the organization of the material follows more closely how designers would use the specification in practice. From the technical standpoint, the 2005



**Figure 6** - Computer model for the 300m long (200 m clear span) trusses of the retractable roof of the Houston Reliant Energy Stadium (SAP 2000 model by Walter P. Moore and Assoc., Houston, TX)

Specification presents a unified treatment of both the ASD and LRFD approach, and incorporates the latest knowledge in material, member, connection and structural system behavior. In particular, through the new material in Chapters C and Appendices 1 and 7, the new Specification brings in advanced analysis methods into the design of steel structures. It is expected that this will lead to more safe and reliable designs, more economical steel structures, more efficient design and use of steel in even more daring structures (Figure 6).

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