Task Group Report on:

Local Buckling (Width-to-thickness) Limits

Prepared by the AISC Committee on Specifications
Ad Hoc Task Group on Local Buckling (Width-to-thickness) Limits

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Final

Ad Hoc Task Group on Local Buckling Limits

Charge

The AISC Committee on Specifications has established an ad hoc task group to provide input on the following issues related to local buckling requirements (i.e., width-to-thickness ratios):

1. Recommend objective of each width-to-thickness (lambda) limit in ANSI/AISC 360, *Specification for Structural Steel Buildings* (AISC 360) and ANSI/AISC 341, *Seismic Provisions for Structural Steel Buildings* (AISC 341).
   a. Explicitly state the objective (M_p, R, θ_p, etc.) for all λ_r, λ_p, λ_{md}, λ_{hd}, etc.
      Note, objective may be system dependent, particularly in AISC 341
   b. Insure these objectives are clear to the user (through commentary, guides, user notes etc.)
2. Review all width-to-thickness (lambda) limits in AISC 360 and 341 against objective criteria
   a. Recommend updated λ limits when not aligned with objectives
   b. Document source of all existing lambda limits
3. Provide recommendation on material criteria and application in lambda limits
   a. Recommend whether nominal or expected F_y is appropriate for λ limits in 341 and 360
   b. Provide guidance on other material criteria implicit in λ limits (F_u/F_y, E_{st}, COV of F_y, etc.)
4. Recommend a consistent approach for web/flange interaction in AISC 360 and 341 λ limits
   a. Provide guidance on when web/flange interaction should be considered to meet objectives
   b. Review AISC 360 λ limits regarding web/flange interaction and provide recommendations
   c. Review ASIC 341 λ limits regarding web/flange interaction and provide recommendations
      Note, recent deep column research indicates changes will be needed at least in AISC 341
5. Provide recommendation on alternatives to use of lambda limits to achieve objectives
   a. Identify alternative means for establishing performance objectives established in 1
      i. Continuous Strength Method of Gardner et al.?
      ii. AISC 341 Appendix 1 supported analysis provisions?
      iii. Testing pathways?
   b. Insure Specifications provide users pathways to alternative means when λ is an impediment

Membership

Committee

Ben Schafer, Chairman (TC 4)
Tim Fraser (TC 6)
Subhash Goel (TC 9)
Pat McManus (TC 9)
Don Sherman (TC 4)
Tom Sabol (TC 9)
Chia-Ming Uang (TC 4)
Amit Varma (Fire, TC 5)
Brent Leu, Secretary

Additional AISC TC members and guests contributed at the in-person meetings as guests of the TG. In particular, Lou Geschwinder, Don White and Kerry Kreitman contributed substantially to this report. Also Perry Green consistently participated representing the interests of TC11 and AISI N690. All contributions to this report and efforts of the TG members and friends are acknowledged and thanked.

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Executive Summary

AISC TC4 maintains local buckling classification (width-to-thickness, *w/t*) limits for both AISC 360 and AISC 341. TC4 expressed reticence in maintaining the limits of AISC 341 and desired a deeper understanding of their background. In addition, in many cases the complete background of existing AISC 360 *w/t* limits was also not fully clear to TC members. Further compounding a need to re-examine *w/t* limits were: recent tests on deep columns that suggested existing limits were not conservative, the prospect of new higher strength materials that will be much more influenced by current *w/t* limits than conventional mild steels, and new research and specifications that employ cross-section rather than element slenderness for local buckling.

A comprehensive review of *w/t* limits with recommendations for action was required such that the standard could continue to maintain safety and efficiency for structural steel sections. Current guidance is efficient to apply in design and well-tested, but may incorrectly favor some sections over others (influencing efficiency) and may create an impediment to new materials (with higher F_y). The goal is to provide clear design criteria, with well understood objectives, that are efficient to use in design, while still providing the most material/cost efficient structural steel solution. AISC formed an ad hoc task group to address the challenges which lead to the approval of the charging statement provided in the preface to this report.

A comprehensive review of exiting *w/t* limits and the source of those limits was completed and documented in this report. No new research was performed in this review. This review provided a means to clarify the specific objectives for AISC's use of λ_r, λ_p, λ_{md}, and λ_{hd}. In some cases, stating the objectives in plain language provides clarity that (a) improvements are needed, and (b) not all *w/t* limits may be aligned with desired objectives. Specifically, in AISC 360 some λ_r limits for flexure are identified as potentially not being aligned with intended objectives. Also, in AISC 341 λ_{md} and λ_{hd} for deep wide-flange columns need additional improvements.

The review performed herein also provided a means to summarize a number of larger issues with respect to *w/t* limits: strain capacity, web-flange interaction, material property sensitivity, and more. Through this analysis it is identified that methods exist nowadays that can achieve the same objectives as current *w/t* limits, but may have broader scope in application.

In general, it is found that only minor changes are potentially needed to current *w/t* limits. Thus, in most cases, it is expected that design can continue unchanged (with the exception of the improved criteria for deep columns). To minimize change and risk, newer local buckling cross-section classification methods could be permitted as alternatives rather than used as replacements to current *w/t* limits so that advantages of the newer approaches can be utilized only when beneficial.

This report concludes with a series of recommendations (see Section 7) related to non-seismic *w/t* limits, seismic *w/t* limits, and additional related recommendations. The recommendations impact AISC 360 B4.1/Table B4.1, AISC 341 D1.1/Table D1.1, AISC 360 Appendix 1, and research priorities. Thus, the COS, TC3, TC4, TC9, and CoR are the primary intended audiences for the recommendations.

1 Introduction to local buckling (*w/t*) limits

A relatively terse introduction to local buckling is provided to establish notation and emphasize specific aspects of the behavior that are important for discussing improvements to local buckling (w/t) limits. More general treatments of local buckling can be found in the SSRC Guide (2010), Allen and Bulson (1980), Salmon et al. (2009), and others.

1.1 Background to local buckling and role of *w/t*

Classically, steel cross-sections are conceptualized as being composed of a series of connected long plates. The plates (also known as elements) of the cross-section with connection along both longitudinal edges, such as the web of an I-section, are known as stiffened elements; while plates with only connection at one longitudinal edge, such as ½ the flange of an I-section, are known as unstiffened elements.

The elastic buckling of long plates using Kirchoff thin plate theory leads to the following classical expression:

$$F_{cr} = k\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{w}\right)^2 \quad (1)$$

where F_{cr} is the elastic plate buckling stress, E and ν are material properties, t and w define the plate thickness and width respectively, and the plate buckling coefficient, k, is a function of the loading and boundary conditions. Solutions for k exist for a wide variety of conditions – and even can consider multiple attached elements to form a full cross-section (e.g. see Allen and Bulson (1980))– but commonly only the simplest values are used in design, e.g. k of 4 for a stiffened element in uniform compression, or k of 0.425 for an unstiffened element in uniform compression. See the SSRC Guide (2010) for further discussion.

If local plate buckling behaved in a manner similar to global flexural buckling (post-buckling neutral) then w/t limits would be easy to establish. However, unlike flexural buckling of a member, local buckling of a plate is not post-buckling neutral – local plate buckling is post-buckling stable. Thus, design rules do not generally use F_{cr} for the plate as directly as one would use for flexural buckling. Further, the elastic plate buckling provides no consideration for material nonlinearity in the form of Eq. 1. Nonetheless, local buckling is an important limit state in steel of cross-sections that must be considered.

1.2 Objective(s) of *w/t* limits

Typical strength objectives commonly related to w/t limits in codes and specifications include: ensure local buckling does not occur before the cross-section initiates yielding, or ensure local buckling does not occur before the cross-section develops its full plastic moment. Typical deformation objectives commonly related to w/t limits include: ensure a minimum rotation capacity for the section so that some form of redistribution can occur, or energy dissipated. It is worth stating that these strength (e.g., M_p) and deformation (e.g. θ_p) objectives are often associated with the cross-section, even though w/t limits typically only address an element within the cross-section.

It is also possible to consider w/t limits as being more directly connected to plate mechanics where the objective may be to sustain a particular stress, or strain, level in the plate. It is worth noting, even in this case, that uniform applied stress and strain on a plate are resisted by non-uniform stress and strains that include membrane as well as local plate bending strains that can far exceed the applied stress/strain and always involve transverse as well as longitudinal material properties and response.

1.3 Application of *w/t* limits in design

Structural steel design specifications world-wide use w/t limits to provide engineers guidance on the impact of local buckling on their designs. The strength and rotation capacity of beams is the archetypical case for this application and is illustrated in Figure 1. Figure 1 includes the nomenclature of AISC 360: slender, non-compact, and compact, as well as that of Eurocode: Class 4, Class 3, Class 2, and Class 1.

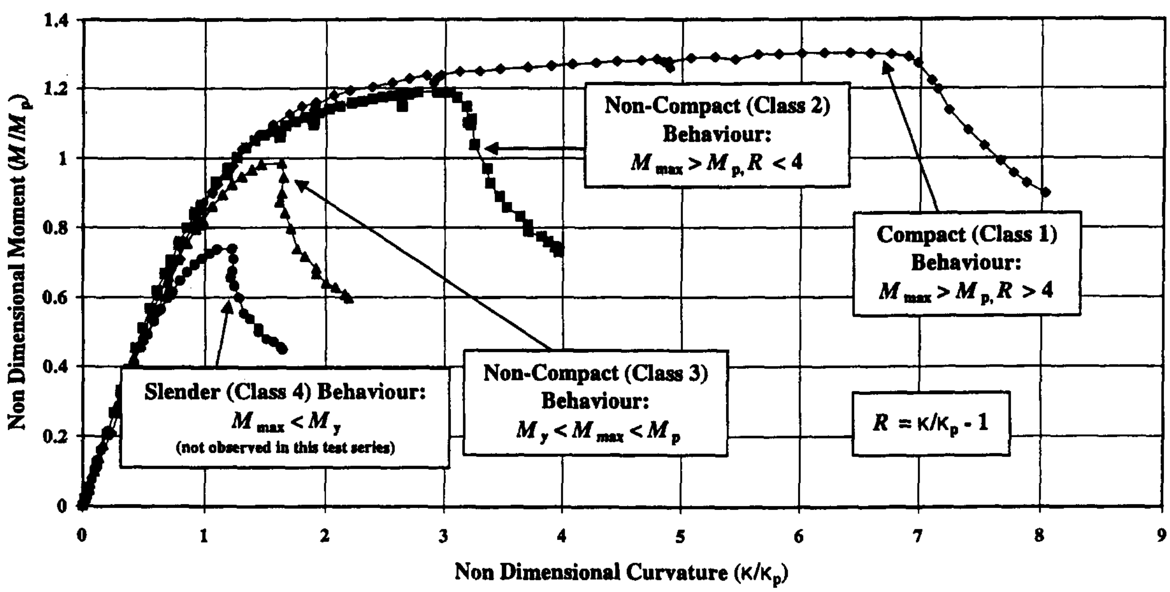


Figure 1. Moment-curvature behavior of beams with different *w/t* limits, source: Wilkinson and Hancock (1998)

The Eurocode Class 1-4 criteria are used widely in the technical literature around the world and are excerpted here for clarity.

> "Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance.
>
> Class 2 cross-sections arc those which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling.
>
> Class 3 cross-sections are those in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
>
> Class 4 cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section."
>
> Eurocode, EC3-1-1 Section 5.5.2

Again, it is worth noting that *w/t* limits, which place limits on the cross-section elements, are usually employed in efforts to achieve full cross-section behavior.

1.4 Development of *w/t* limits

Development of *w/t* limits has classically relied on comparisons to experimental testing. If one can establish a buckling stress, say at a stress of aF_y, that meets a desired objective, then the resulting *w/t* limit can be simplified as follows:

$$F_{cr} = k\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{w}\right)^2 = aF_y \tag{2}$$

$$\frac{\mathrm{w}}{\mathrm{t}} = \sqrt{\frac{\mathrm{k}\pi^2}{a12(1-\nu^2)}}\sqrt{\frac{E}{F_y}} = C\sqrt{\frac{E}{F_y}} \tag{3}$$

Note, the buckling stress in Eq. (2) is a reference stress only and does not rigorously reflect a bifurcation stress in the plate. A typical observation from experiments may be that an element with $F_{cr}\sim 2F_y$ is needed to develop first yield in a full section. If the plate buckling coefficient, k, is also assumed then the coefficient C may be found. These coefficients are tabulated in AISC 360. For example, for the flange of a rolled shape to develop the full plastic moment, AISC 360-16 provides:

$$\lambda_p = \left(\frac{\mathrm{w}}{\mathrm{t}}\right)_{\mathrm{p}} = C\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{E}{F_y}} \tag{4}$$

In much of the literature a related, but slightly different approach has been taken to finding coefficients similar to *C*. A non-dimensional slenderness is defined as $\lambda^* = \sqrt{F_y/F_{cr}}$ and this parameter is examined to determine when the desired objective is met. The methods are related:

$$\lambda^* = \sqrt{\frac{F_y}{F_{cr}}} = \sqrt{\frac{1}{a}} = \sqrt{\frac{12(1-\nu^2)}{k\pi^2 E}}\frac{\mathrm{w}}{\mathrm{t}} \tag{5}$$

For example in Winter's classical work (1947) he found that $\lambda^* = 0.673$ was an accurate boundary between elements which could develop their first yield capacity and those that required additional reductions due to local buckling.

A variety of approaches have been employed to develop w/t limits for design. The most common approach is wholly experimental; however, sometimes the experiments have been conducted on idealized elements/plates and sometimes on entire sections. In some cases researchers directly try to fit their data to the coefficient *C* of Eq. (3), in other cases the focus is on finding the a or λ^* of Eq. (2) or (5). Also, in some instances researchers have used Eq. (3) in some form to back-solve for k. This can lead to unintended consequences when such k values are re-inserted into elastic buckling expressions and used in other settings.

It is worth noting that in developing *w/t* limits, Eq. (1) has sometimes been modified to be aligned with the tangent modulus theory and/or application of plasticity reduction factors to the modulus. These approaches can be problematic. Although flexural buckling of columns may be one-dimensional, plate buckling is inherently two-dimensional and simple one-dimensional reductions to the modulus and ignoring the inherent post-buckling of the plates can lead to erroneous conclusions about strength and w/t limits.

In several instances researchers have found it useful to conceptualize Eq. (1) in terms of one-dimensional strain instead of stress, i.e.

$$\epsilon_{cr} = \frac{F_{cr}}{\mathrm{E}} = k\frac{\pi^2}{12(1-\nu^2)}\left(\frac{t}{w}\right)^2 \tag{6}$$

Eq. (6) has the desirable feature of being independent of modulus and thus, e.g., researchers and specifications in structural thermoplastics or gradual yielding materials (including steel) have preferred this form. In establishing w/t limits, instead of determining an F_{cr} in excess of F_y (i.e. aF_y) one thinks in terms of multiples of the yield strain (i.e. $a\epsilon_y$). This is more natural in inelastic cases, particularly for plastic redistribution or seismic design. It is also worth noting that in the classical literature for developing steel w/t limits it was sometimes common to consider w/t limits that achieve a certain average applied strain – a typical target was for the element to sustain a strain up to the initiation of strain hardening, or 3 or 4 times ϵ_y – the format of Eq. (6) is particularly convenient for such considerations (though one must be careful in that the critical strain is not a direct predictor of the strain that an element can sustain, but rather a parameter which is correlated with the desired strain).

1.5 Strengths and weaknesses of *w/t* formulation

Strengths of the existing AISC w/t formulation, e.g. Eq. (4) for local bucking limits include:

- the method is easy and fast to apply, and
- has a long tradition of use, and
- relatively high level of clarity.

Weaknesses of the existing w/t formulation for local buckling limits include:

- the method connects to the element, not the section, and most behavior objectives are at the section level; more specifically w/t limits provide predictions of element strain capacity, not member curvature or other section level parameters;
- for the limits to be simple, constant coefficients for *C* (Eq. 4) are commonly used; however, if web-flange interaction (i.e., simple equilibrium and compatibility within the section), stress distribution (e.g. stresses from a beam-column, difference in stresses when a flange tip is in tension/compression), or material nonlinearity is considered this breaks down and determination of *C* becomes its own quite complex process;
- by using w/t instead of the non-dimensional slenderness λ^*, i.e., $\sqrt{F_y/F_{cr}}$, the limits appear to be different for every element (i.e., lots of different *C*) while in reality only one assumption (λ^*) is typically being made – this reduces conceptual clarity.

AISC's ad hoc task group was charged with 5 basic objectives as provided in the preface to this report. The following sections provide work directed at each of these tasks and recommendations for the future.

1.6 Note on *w/t* limits for composite design

Due to time limitations the task group was not able to address *w/t* limits in composite design.

2 Task 1: Objective of local buckling (*w/t*) limits

The task group examined the application, intent, and origin of the AISC 360 and AISC 341 local buckling (w/t) limits. The current objectives of these w/t limits are summarized in what follows. Improved objectives are discussed in Chapter 3 (Task 2).

2.1 AISC 360-16 objectives for non-seismic *w/t* limits

Members under Axial Compression

λ_r: Provides the slender/non-slender limit for the section, specifically for *w/t* $\leq \lambda_r$ the cross-section can develop its squash (yield) capacity, i.e.: $P_y = A_g F_y$

Members under Flexure

λ_r: Provides the noncompact-slender limit for the section, specifically for *w/t* $\leq \lambda_r$ the compression elements remain "elastic", therefore the cross-section can develop at least its elastic limit in bending, i.e. M_r. Note, whether $M_r = M_y = SF_y$ or $M_r = SF_L \approx 0.7M_y$ varies by section and limit state, as discussed below.

> **Exception F_y vs. F_L**: Explicitly for Table B4.1b Case 11 (flanges of I-shaped built-up sections) when *w/t* = λ_r the cross-section can develop its first yield capacity considering residual stresses, i.e.: $M_r = SF_L$ where $F_L = F_y - F_r$ and F_r is the assumed level of residual stress. In addition, as discussed in Appendix 1, other elements (e.g. webs of I-shaped sections) may have implicit consideration of residual stresses in determining λ_r, but do not use F_L in the final width-to-thickness limit.
>
> **Past practice F_y vs. F_L:** Note in the 1999 LRFD edition of AISC 360 the use of $F_L = F_y - F_r$ in the flexural limits was far more pervasive. The following flexural cases used F_L: flanges of rolled I-shapes or channels; flanges of built-up I-shapes; flanges of HSS, box, or cover plates. In general F_r=10 ksi was uded for rolled shapes, and F_r=16.5 ksi was used for welded shapes.
>
> **Behavioral objective F_y vs. F_L**: Commentary to AISC 360-16 states "Non compact sections can develop partial yielding in compression elements before local buckling occurs, but will not resist inelastic local buckling at the strain levels required for a fully plastic stress distribution. Slender-element sections have one or more compression elements that will buckle elastically before the yield stress is achieved." As currently stated the noncompact-slender boundary is poorly defined and interpretation of F_y vs. F_L hinges on interpretation of plate behavior at the boundary between "buckling elastically" and resisting inelastic buckling "but not a full plastic stress". Currently, different cross-sections in Chapter F use different approaches to this issue – see Section 3.2.
>
> **Impact of Residual Stresses and F_L**: Use of F_L relaxes (liberalizes) the *w/t* limits. A justification given for this application in Salmon et al. (2009) is that for cases where key residual stresses are tensile in nature this relaxation should be allowed.

λ_p: Provides the compact-noncompact limit for the section, specifically for *w/t* $\leq \lambda_p$ the cross-section can develop its ideal fully plastic capacity in bending, i.e., $M_p = ZF_y$.

Rotation Capacity: The commentary states that "compact sections … possess a rotation capacity, R_{cap}, of approximately three". In some newer cases, e.g. Table B4.1b Cases 17 and 19 for elements of HSS sections the λ_p limit was specifically selected to meet a minimum R_{cap} of 3. In other cases, as detailed in Appendix 1, there is not a direct connection between a target rotational capacity and the selected w/t limit – or the target rotational capacity was not 3; however, in available experiments on I-shaped beams which meet the λ_p criteria Appendix 1 also shows that the sections develop at least an R_{cap} of 2.9, and in many cases far in excess of this. Current λ_p limits provide strength M_p, and also supply a level of strain capacity in the element in excess of the yield strain. In many instances researchers targeted a strain capacity up to the onset of strain hardening in the material (see Appendix 1 and AISC 341-16 D1.1.1b commentary). The end result of these varied approaches is that the section typically can sustain a rotation capacity of approximately three or more.

Exceptions: In some cases, for example Case 14 of Table B4.1b: T-stems in Chapter F of AISC 360, λ_p is associated with first yield M_y instead of fully plastic M_p.

2.2 AISC 341-16 Seismic *w/t* limits

Since its first introduction in 1990, AISC *Seismic Provisions*, AISC 341, have gone through revisions to local buckling (i.e., *w/t* ratio) requirements. These seismic provisions provide limiting width-thickness ratios as part of the ductility design requirements to ensure adequate inelastic deformation capacities. The requirements in the 1990 edition were basically those from the 1988 UBC, which were based on limited research conducted in the 1970s and 1980s. The seismic local buckling requirements were also strongly influenced by the plastic design provisions in AISC 360.

The Northridge Earthquake in 1994 triggered a new wave of seismic steel research activities, not only for Special Moment Frames but also for other types of SFRS as well. Table D1.1 provides the limiting width-thickness ratios for all SFRS covered in AISC 341. Starting with the 2010 edition, this table expresses local buckling requirements in the form of λ_{hd} values for Highly Ductile Members and λ_{md} values for Moderately Ductile Members in lieu of the previously used terms Seismically Compact and Compact. This change in terminology was made because the limiting *w/t* ratios did not always reflect limit states consistent with AISC 360's use of "compact." In 2010, the limiting λ values are a function of E and F_y, but starting in 2016, these formulae were converted to a new format by replacing the nominal yield stress, F_y, by the expected yield stress, R_yF_y and modestly changing the coefficients.

Local bucking (*w/t*) limits in AISC 341 serve multiple objectives, and their application is often dependent on the Seismic Force Resisting System (SFRS). Table D1.1 provides two limits: moderately ductile λ_{md}, and highly ductile λ_{hd}; however, the limits are not only a function of the type of element in a section (e.g. stiffened vs. unstiffened) but also a function of how the section is employed in the SFRS (e.g, used as a brace, as a link beam, or as a reinforcing plate).

Focusing on I-sections in moment frames (IMF, SMF) as the prototypical application of local buckling (w/t) limits the AISC 341-16 commentary provides the basic objectives:

λ_{md}: Provides a section that can undergo plastic rotation of 0.02 radians or less.

λ_{hd}: Provides a section that can undergo plastic rotation of 0.04 radians or more.

The commentary further states that λ_{md} in AISC 341-16 is generally the same as λ_p in AISC 360-16 with the exception of HSS, stems of WT's, and webs in flexure. Further, λ_{hd} is typically stricter than λ_p, though in several cases this is relaxed. A summary of all current criteria for AISC 360-16 and AISC 341-16 is provided in Table 1.

To understand the objective in the application of the λ_{md} and λ_{hd} limits one must go through each SFRS. A summary of the application of these limits and their intended objective is provided in Table 2 and full details are provided in Table 3. From Table 2 we may observe the following objectives for λ_{md} and λ_{hd}, although not all objectives are utilized in all systems:

λ_{md} : Provide enough ductility so that the SFRS can develop its system strength (R_r), and last several cycles at that strength (n cycles), provide sufficient compactness so that a member can develop M_p or in some cases M_p and at least 0.02 radians rotation, or M_p up to and including strain hardening (M_{pe}). Application of these objectives is system dependent.

λ_{hd} : Provide enough ductility so that the SFRS can develop its system strength (R_r), and last several system cycles at that strength (n cycles), or system inter-story drift (3%ID), or provide sufficient compactness so that a member can develop M_p or in some cases M_p and at least 0.04 radian rotation (i.e., story drift angle) at a post-peak of $0.8M_p$, or M_p up to and including strain hardening (M_{pe}), or high component level strains (10-20ϵ_y) and high numbers of component cycles (n cycles). Application of these objectives is system dependent.

In general when a concern exists for seismic behavior, but limited research or knowledge is available, it is common to require λ_{md} or λ_{hd}. As a result the objectives for these criteria are sometimes clear and discrete, but more often manifold and complex.

Table 1 Summary of AISC 360-16 and AISC 341-16 *w/t* limits

AXIAL LOADS						
	360 Table B4.1a				341 Table D1.1	
	A = $\sqrt{E/F_y}$				B = $\sqrt{E/(R_y F_y)}$	
		λ_p	λ_r		λ_{hd}	λ_{md}
case						
1	rolled I flanges		0.56A		0.32B	0.40B
2	built-up I flanges		0.64A		0.32B	0.40B
3	angle legs		0.45A		0.32B	0.40B
4	Tee stems		0.75A		0.32B	0.40B
5	I webs		1.49A	braces	1.57B	1.57B
6	HSS walls		1.40A	braces	0.65B	0.76B
				columns	0.65B	1.18B
7	cover plates		1.40A		———	———
8	stiffened element		1.49A		———	———
9	Round HSS		$0.11A^2$		$0.053B^2$	$0.062B^2$
	Flanges of H-piles		———		n.a.[a]	0.48B[a]
	Webs of H-piles		———		n.a.[a]	1.57B[a]
	360 Table I1.1a					
	composite rect. HSS	2.26A	3.00A		1.48B	2.37B
	composite round HSS	$0.15A^2$	$0.19A^2$		$0.085B^2$	$0.17B^2$
FLEXURE						
	360 Table B4.1b					
10	rolled I flanges	0.38A	1.0A		0.32B	0.40B
11	built-up I flanges	0.38A	0.95A		0.32B	0.40B
12	angle legs	0.54A	0.91A		0.32B	0.40B
13	minor axis I flanges	0.38A	1.0A		———	———
14	Tee stems	0.84A	1.52A		0.32B	0.40B
15	I webs	3.76A	5.70A		$f(P_u/P_y)$	$f(P_u/P_y)$
16	singly sym. I webs	$f(h_c/h_p)$	5.70A		———	———
17	HSS flanges	1.12A	1.40A		0.65B	1.18B
18	Flange cover plates	1.12A	1.40A		———	———
19	HSS webs	2.42A	5.70A		———	———
	box webs				0.67B	1.75B
20	Round HSS	$0.07A^2$	$0.31A^2$		$0.053B^2$	$0.062B^2$
21	box flanges	1.12A	1.49A		0.65B	1.18B
	360 Table I1.1b					
	composite HSS flanges	2.26A	3.00A		1.48B	2.37B
	composite HSS webs	3.00A	5.70A		———	———
	composite round HSS	$0.09A^2$	$0.31A^2$		$0.085B^2$	$0.17B^2$

a. potentially better categorized as flexure case in AISC 341

Table 2 Summary of objectives for application of λ limits in ASCE 341-16

	System	Element	Objective
λ_{md}	IMF	Beam	M_p, 0.02 rad
		Column	R_r
	OCBF	Brace	R_r, *n* cycles
	MT-SCBF	Strut	M_{pe}
	EBF	Outside Link	R_r
		Brace	R_r
	BRBF	Beam	R_r
		Column	R_r
λ_{hd}	SMF	Beam	M_p, 0.04 rad @ $0.8M_p$
		Column	M_p, R_r, 0.04 rad
	STMF	Chord and Diagonal	3% ID
		Column	3% ID
	SCCS	Column	M_p, limit FLB
	SCBF	Beam	R_r
		Column	R_r, large Θ_p
		Brace	R_r, *n* cycles, yield 0.3%ID, 10-20ϵ_y
	MT-SCBF	Column	M_{pe}
		Brace	R_r, *n* cycles, yield 0.3%ID, 10-20ϵ_y
	EBF	Link	0.02 to 0.08 rotation
		Column	R_r
	MT-BRBF	Beam	R_r
		Column	R_r
	SPSW	Column Boundary	R_r
		Horizontal Boundary	M_p, *n* cycles

Note, R_r = Required system strength, ID=inter-story drift, FLB = Flange local buckling

Table 3 Summary of AISC 341 Member Ductility Classification and Performance Objectives

Summary of AISC 341 Member Ductility Classifications and Performance Objectives

(Generally, ductitility classifications for both web and flanges and for wide flanges and other members are listed below. If there are no differences, these have been consolidated to simplify the table.)

System	Member	341 Ref.	Member Ductility Classification	Performance Objectives	Behavior Classification	Commentary References	Remarks
Chapter D							
Moderately Ductile	N/A	Defined in Table D1.1, basis described in Comm. D1.1	Moderate	Develop and more-or-less maintain Mp to at least 0.02 rad	N/A	Comm. D1.1 and D1.2. Sawyer, 1961; Lay, 1965; Kemp, 1986; Bansal, 1971.	Table D1.1 defines moderately and highly ductile member proportions for various shapes and boundary conditions. Commentary contains broad statement about desired behavior without reference to specific systems or members, although rotational performance objective most likely only applicable to flexural members (e.g., beams in moment frames). Post-buckling behavior of SCBF is an extreme rotational behavior caused by member buckling under load. Survival of the brace through cycles is addressed by *w/t*. Values are the same as in Spec. Table B4.1b except for round and rectangular HSS, stems of WTs and webs in flexural compression. Ry term added to equations to account for expected strength and dual-certification; equations recalibrated. Numerical results are expected to be the same for common Fy values.
Highly Ductile	N/A	Defined in Table D1.1, basis described in Comm. D1.1	High	Develop and more-or-less maintain Mp to at least 0.04 rad	N/A	Comm. D1.1 and D1.2 - Primarily Dawe and Kulak 1986, Uang and Fang 2001, FEMA 2000a. Also Haaijer and Thurlimann (1958), Perlynn and Kulak (1974), and Dawe and Kulak (1986)	See above.

Chapter E							
OMF	Beam: Web	E1.5a	Compact (assumed)	Develop Mp (implied)	Ductility		Ductility requirement not explicitly specified (Comm. states "minimal") but refers to *Specification*. There may be no requirement if Section E1.6b(b) is applied (i.e., connection designed for required strength based on system limitation exception). If connection designed to E2.6 or E3.6, the beam must satisfy moderate ductility, while it is true the connection can achieve higher ductility, this doesn't require the member to achieve high ductility to achieve desired system behavior.
	Beam: Flange	E1.5a	Compact (assumed)	Develop Mp (implied)	Ductility	Comm. E1.2	Ductility requirement not explicitly specified (Comm. states "minimal") but refers to *Specification*. There may be no requirement if Section E1.6b(b) is applied (i.e., connection designed for required strength based on system limitation exception). If connection designed to E2.6 or E3.6, the beam must satisfy moderate ductility, while it is true the connection can achieve higher ductility, this doesn't require the member to achieve high ductility to achieve desired system behavior.
	Column: Web	E1.5a	Not specified	Develop Ru only?	?		Ductility requirement not specified in *Provisions*, refers to *Specification*
	Column: Flange	E1.5a	Not specified	Develop Ru only?	?		Ductility requirement not specified in *Provisions*, refers to *Specification*
IMF	Beam: WF	E2.5a	Moderate	1. Develop Mp 2. 0.02 rad interstory drift	Ductility	Comm. E2.5a; See E3.5a for references since there have been no IMF-specific tests	Engineering judgment, FEMA (2000d) and FEMA (2000f) for overall frame behavior
	Column: WF	E2.5a	Moderate	Develop Ru	Failure Consequence (assumed)		No explicit discussion

SMF	Beam: WF	E3.5a	High	1. Develop Mpe 2. 0.04 rad interstory drift 3. Maintain 0.8Mp at 0.04	Ductility	Comm 3.5a; Sawyer, 1961; Lay, 1965; Kemp, 1986; Bansal, 1971. Assumed also: Primarily Dawe and Kulak 1986, Uang and Fang 2001, FEMA 2000a. Also Haaijer and Thurlimann (1958), Perlynn and Kulak (1974), and Dawe and Kulak (1986)	See Commentary for discussion regarding 0.8Mp and issues associated with PR SMF connections.
	Column: WF Web	E3.5a	High	1. Develop Mp (implied) 2. Develop Ru	Failure Consequence (assumed)	Krawinkler, 1978; Engelhardt et al., 2000; Lee et al., 2005b; Shin and Engelhardt, 2013	No explicit discussion. References are for panel zone.
	Column: WF Flange	E3.5a	High	1. Develop Mp (implied) 2. Develop Ru 3. Develop 0.04 rad interstory drift	Failure Consequence (assumed)		No explicit discussion .
STMF	Chord (Special segment)	E4.5d	High	Undergo 3% story drfit	Ductility	Comm E4.5c Basha and Goel, 1994	Special segment only, Comm E4.5d says chord memebers are to be "compact" but Provisions require "highly ductile"
	Web(?): Flat Bar Diagonal (Special segment)	E4.5d	b/t ≤ 2.5	Undergo 3% story drfit	Ductility	Comm E4.5c Basha and Goel, 1994	Special segment only. No bt ratio specified in the *Specification* or *Provisions* for a rectangular member in compression or flexure because global buckling will occur before local buckling occurs
	Web (?): Single Angle Diagonal (Special segment)	E4.5d	High		Ductility		Special segment only. Comm E4.5d says single angles should satisfy 0.18(E/Fy)^0.5, which doesn't match Table D1.1.
	Column	E4.5a	High	Undergo 3% story drfit	Failure Consequence (assumed)	Comm E4.5c Basha and Goel, 1994	Outside of special segment

	Truss Chords and Webs (outside special segment)	Not specified	Not specified	Undergo 3% story drfit	Not specified		Outside of special segment. No discussion of truss members outside of special segment, requirements of the *Specification* apply
OCCS	Column	E5.5a	Compact (assumed)	Develop Mp (implied)	Ductility		No ductility classifications are listed. Comm. describes "minimal level of inelastic rotational capability at base of column," but no explicit performance goal listed. Limit on axial load will increase available ductillity.
SCCS	Column	E6.5a	High	Develop Mp (implied), prevent local buckling	Ductility	Comm. E6.5a	Comm. describes "limited level of inelastic rotational capability at base of column. No explicit performance goal listed other than to preclude local buckling, which is judged to be significantly detrimental to this system. Limit on axial load will increase available ductillity.
Chapter F							
OCBF	Beam	F1.5a	None listed	Develop Ru			No ductility classifications are listed.
	Brace	F1.5a	Moderate	1. Develop Ru 2. Withstand some number cycles after buckling	Ductility	Comm F1.5a	Except for KL/r ≥ 200 in tension-only braced frames where no requirement beyond the *Specification* applies. Intent is primarily to avoid brittle connection failure.
	Column	F1.5a	None listed	Develop Ru			No ductility classifications are listed.
MT-OCBF	Struts	Not specified	Not specified	Not specified	Not specified		Reduced level of ductility compared to MTBF in SCBF but amount not quantified
	Columns	Not specified	Not specified	Not specified	Not specified		Reduced level of ductility compared to MTBF in SCBF but amount not quantified.
	Braces	F1.5a	Moderate	Not specified	Ductility		Reduced level of ductility compared to MTBF in SCBF but amount not quantified. Requirement invoked because member is part of OCBF. No bt requirement for tension-only brace with slendernes ratio > 200

SCBF	Beam	F2.5a	High	Develop Ru	Ductility		Highly ductile required even if not inverted-V. Failure consequence (i.e., inability to develop forces in braces if beam deforms) also a behavior classification, although Roeder research suggests concern may be exaggerated (this work limited to wide-flange beams). Connections list performance requirements (e.g., 0.025 rad rotation for simple connections or Mpe for fixed connections).
	Brace: Other than HSS	F2.5a	High	1. Develop Ru 2. Prevent local buckling 3. Withstand unspecified number cycles after buckling? 4. Yield after 0.3 to 0.5% drfit 5. 10 to 20 times yield deformation	Ductility	Comm. F2.2a and F2.2b	"Yield deformation" not defined. Comm. states goal is to prevent local buckling, so b/t ratios less than "compact" were selected.
	Brace: HSS	F2.5a	High	1. Develop Ru 2. Prevent local buckling 3. Withstand unspecified number cycles after buckling? 4. Yield after 0.3 to 0.5% drfit 5. 10 to 20 times yield deformation	Ductility	Comm. F2.2a and F2.5b, Goel, 1992b; Goel, 1992c, Tang and Mahin, 2005,	"Yield deformation" not defined. Comm. states goal is to prevent local buckling, so bt ratios less than "compact" were selected. Discussion in commentary specifically about HSS braces notes that even members satisfying these requirements may suffer from local buckling, which would be a limit on performance.
	Column	F2.5a	High	1. Significant inelastic rotation	Failure Consequenc e (assumed)	Comm F2.5a, Tremblay, 2001, 2003, Sabelli et al., 2003	"Significant" not defined with respect to inelastic rotation.
MT-SCBF	Struts	F2.4e, F2.4b	Moderate	1. Develop strain-hardened Mpe for critical buckling direction (torsion) due to out-of-plane brace buckling	Ductility (assumed)		F2.4e references F2.4b which requires "moderately ductile" proportions when braces intersect strut away from strut-column connection.

	Columns	F2.5a	High	1. Develop strain-hardened Mpe for critical buckling direction due to out-of-plane brace buckling	Failure Consequence (assumed)		Requirement invoked because member is part of SCBF
	Braces	F2.5a	High	See SCBF	Ductility	See SCBF	Requirement invoked because member is part of SCBF
EBF	Link: I-shaped	F3.5b	High	1. Link to column configurations meet requirements of Section K2 or prescriptive connection reinforcement 2. Link rotation angle of 0.02 to 0.08 depending on link length	Ductility	Comm F3.5b	Except members with 1.6Mp/Vp may have FLANGES that are moderately ductile
	Link: Built-up boxes	F3.5b	High	See I-shaped links	Ductility	Comm F3.5b	Except built-up boxes with 1.6Mp/Vp may have WEBS that are moderately ductile
	BOL	F3.5a	Moderate	Develop Ru	Failure Consequence (assumed)	Okazaki et al., 2004a; Richards et al., 2004)	Only applies if BOL member is different from link member
	Brace	F3.5a	Moderate	Develop Ru	Failure Consequence (assumed)		No explanation provided.
	Column	F3.5a	High	Develop Ru	Failure Consequence (assumed)		No explanation provided.

BRBF	Beam	F4.5a	Moderate	Develop Ru	Ductility	Comm. F4.5a	ID in BF systems (even BRBF) expected to be less than MF systems. Frames tested to 2% rather than 4% ID, thus moderately ductile criteria is consistent with Table 2 objectives.
	Column	F4.5a	Moderate	Develop Ru	Failure Consequence (assumed)	Comm. F4.5a	ID in BF systems (even BRBF) expected to be less than MF systems. Frames tested to 2% rather than 4% ID, thus moderately ductile criteria is consistent with Table 2 objectives.
	Brace Core		N/A	1. Conformance with Section K3 2. Strain associated with at least 2% story drift 3. Strain associated with at least 2x design story drift	Ductility	Comm. F4.2, F4.5a; Fahnstock 2003, Sabelli, 2003, ASCE 7 and NEHRP	2% story drift based on ASCE 7 and NEHRP linear procedures. 2x design story drift based on mean drift from ground motion with POE of 10% in 50 years. Strains based on NLRHA may be used instead of either prescriptive strain limit.
MT-BRBF	Struts		Not specified	Not specified	Not specified		Not specified
	Columns	F4.5a (assumed)	High (assumed)	See BRBF	Failure Consequence (assumed)	See BRBF	Assumed that requirement is invoked because member is part of BRBF
	Braces	F4.5a (assumed)	High (assumed)	See BRBF	Ductility	See BRBF	Assumed that requirement is invoked because member is part of BRBF

SPSW	HBE	F5.5a, F5.5c	High	1. Develop Mp (assumed) at beam-column connection 2. Withstand unspecified number cycles of deformation.	Ductility	Comm F5.5a	Demand may be assumed to equal twice gravity plus web plate yielding force (assuming a simple-span beam) or twice gravity plus web plate yielding force (assuming a simple-span beam) with reduced flanges assuming c = 0.25bf. Potential for yielding suggests that member should satisfy highly ductile requirements.
	VBE	F5.5a	High	1. Develop Ru 2. Not yield in shear 3. Not yield in flexure except at base	Failure Consequence (assumed)	Comm F5.5a	Potential for yielding suggests that member should satisfy highly ductile requirements.
	Intermediate BE	F5.5a	High	See HBE or VBE, as applicable (assumed)	See HBE or VBE, as applicable	Comm F5.5a	No explicit discussion provided. See HBE or VBE, as applicable.
	Web of shear wall	Not specified	Not specified	Develop Ru	Not specified		None specified

Color Key

Minimum bt ratio based on specified lductility classification		Assumed basis for ductility classification	
[light pink]	No specific requirement beyond that in the Specification		
[yellow]	Moderately ductile requirement	[blue]	Ductile response required to achieve stated performance goal (e.g., develop Mp, attain specified level of interstory drift)
[green]	Highly ductile requirement	[orange]	Desire to avoid consequences of the member's failure, but no specific ductile behavior specified
[white]	Other		

3 Task 2: Evaluation of local buckling (*w/t*) limits

3.1 Non-seismic *w/t* limits comparison with existing standards

3.1.1 Comparison with Eurocode *w/t* limits

Due to its similar design rules with respect to local buckling and maturity with respect to application direct comparison of AISC 360 *w/t* limits to those of Eurocode is desirable. Table 5.2 in Part 1-1 of Eurocode 3 is the counterpart to Table B4.1 in AISC 360. However, the format for presenting the limits is not identical. For a typical *w/t* limit ECCS (Eurocode) and AISC may be summarized as follows:

$$\frac{w}{t} \leq C_{ECCS}\epsilon = C_{ECCS}\sqrt{\frac{235}{F_{yMPa}}}\ vs.\ C\sqrt{\frac{E_{MPa}}{F_{yMPa}}} \tag{7}$$

To convert the ECCS limit into AISC's format

$$C_{ECCS}\sqrt{\frac{235}{E_{MPa}}} = C\ or\ C = 0.0343C_{ECCS} \tag{8}$$

Comparison for compression is provided in Table 4 and for flexure in Table 5.

Table 4 AISC 360-16 vs Eurocode for Compression Only

		AISC	ECCS
	Unstiffened	λ_r	Class 3
1	Rolled Flange	$0.56\sqrt{\frac{E}{F_y}}$	$0.48\sqrt{\frac{E}{F_y}}$
2	Built-up Flange	$0.38{\sim}0.56\sqrt{\frac{E}{F_y}}$ [a]	$0.48\sqrt{\frac{E}{F_y}}$
3	Angle leg, other	$0.45\sqrt{\frac{E}{F_y}}$	$0.51\sqrt{\frac{E}{F_y}}$
4	Stem of tee	$0.75\sqrt{\frac{E}{F_y}}$	$0.48\sqrt{\frac{E}{F_y}}$
	Stiffened		
5	Rolled Web	$1.49\sqrt{\frac{E}{F_y}}$	$1.44\sqrt{\frac{E}{F_y}}$
6	HSS Wall	$1.40\sqrt{\frac{E}{F_y}}$	$1.44\sqrt{\frac{E}{F_y}}$
7	Cover plate	$1.40\sqrt{\frac{E}{F_y}}$	$1.44\sqrt{\frac{E}{F_y}}$
8	Other	$1.49\sqrt{\frac{E}{F_y}}$	$1.44\sqrt{\frac{E}{F_y}}$
	Round		
9	Round HSS/Pipe	$0.11\frac{E}{F_y}$	$0.11\frac{E}{F_y}$

a. AISC provisions a function of web h/t_w, bounds provided here, shading highlights substantial differences

Table 5 AISC 360-16 vs Eurocode for Elements in Flexural Members

		AISC 360	ECCS	ECCS	AISC 360	ECCS
	Unstiffened	λ_p	Class 1	Class 2	λ_r	Class 3
10	Rolled Flange	$0.38\sqrt{\frac{E}{F_y}}$	$0.31\sqrt{\frac{E}{F_y}}$	$0.34\sqrt{\frac{E}{F_y}}$	$1.00\sqrt{\frac{E}{F_y}}$	$0.48\sqrt{\frac{E}{F_y}}$
11	Built-up Flange [a]	$0.38\sqrt{\frac{E}{F_y}}$	$0.31\sqrt{\frac{E}{F_y}}$	$0.34\sqrt{\frac{E}{F_y}}$	$0.56\sim0.83\sqrt{\frac{E}{F_L}}$	$0.48\sqrt{\frac{E}{F_y}}$
12	Angle leg, other	$0.54\sqrt{\frac{E}{F_y}}$	$0.31\sqrt{\frac{E}{F_y}}$	$0.34\sqrt{\frac{E}{F_y}}$	$0.91\sqrt{\frac{E}{F_y}}$	$0.48\sqrt{\frac{E}{F_y}}$
13	flange in minor-axis [c]	$0.38\sqrt{\frac{E}{F_y}}$	$0.31\sim0.62\sqrt{\frac{E}{F_y}}$ $0.31\sim0.93\sqrt{\frac{E}{F_y}}$	$0.34\sim0.68\sqrt{\frac{E}{F_y}}$ $0.34\sim1.02\sqrt{\frac{E}{F_y}}$	$1.00\sqrt{\frac{E}{F_y}}$	$0.48\sim1.44\sqrt{\frac{E}{F_y}}$
14	Stem of tee [c]	$0.84\sqrt{\frac{E}{F_y}}$	$0.31\sim0.62\sqrt{\frac{E}{F_y}}$ $0.31\sim0.93\sqrt{\frac{E}{F_y}}$	$0.34\sim0.68\sqrt{\frac{E}{F_y}}$ $0.34\sim1.02\sqrt{\frac{E}{F_y}}$	$1.52\sqrt{\frac{E}{F_y}}$	$0.48\sim1.44\sqrt{\frac{E}{F_y}}$
	Stiffened					
15	Web (doubly-symm shape)	$3.76\sqrt{\frac{E}{F_y}}$	$2.46\sqrt{\frac{E}{F_y}}$	$2.84\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$	$4.25\sqrt{\frac{E}{F_y}}$
19	Web HSS & box	$2.42\sqrt{\frac{E}{F_y}}$	$2.46\sqrt{\frac{E}{F_y}}$	$2.84\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$	$4.25\sqrt{\frac{E}{F_y}}$
16	Web (singly-symm shape) [b]				$5.70\sqrt{\frac{E}{F_y}}$	
17	Flange HSS	$1.12\sqrt{\frac{E}{F_y}}$	$1.13\sqrt{\frac{E}{F_y}}$	$1.65\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$	$1.44\sqrt{\frac{E}{F_y}}$
18	Flange cover plate	$1.12\sqrt{\frac{E}{F_y}}$	$1.13\sqrt{\frac{E}{F_y}}$	$1.65\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$	$1.44\sqrt{\frac{E}{F_y}}$
21	Flange box	$1.12\sqrt{\frac{E}{F_y}}$	$1.13\sqrt{\frac{E}{F_y}}$	$1.65\sqrt{\frac{E}{F_y}}$	$1.49\sqrt{\frac{E}{F_y}}$	$1.44\sqrt{\frac{E}{F_y}}$
	Round					
20	Round HSS/Pipe	$0.07\frac{E}{F_y}$	$0.06\frac{E}{F_y}$	$0.08\frac{E}{F_y}$	$0.31\frac{E}{F_y}$	$0.11\frac{E}{F_y}$

a. AISC provisions a function of web h/t_w, bounds provided here, $F_L = 0.7F_y$

b. AISC provisions a function of ENA to PNA distances, Eurocode provisions a function of PNA for Class 1 and Class 2, ENA for Class 3 – i.e. stress gradient dependent

c. Eurocode provisions provide limit as a function of whether unsupported tip is in compression or tension and specific to the plastic or elastic stress distribution on the unstiffened element. Typical ranges provided here.

Major observations in comparing AISC 360 *w/t* limits to Eurocode:

Compression Members

- AISC stiffened element *w/t* limits are quite similar to Eurocode

- AISC unstiffened element *w/t* limits are different from Eurocode
    - AISC *w/t* limit is as 36% higher that Eurocode for the stem of a tee section
    - AISC includes web/flange interaction for flanges of built-up shapes, Eurocode does not

<u>Flexural Members</u>

- AISC differentiates between compression and flexural members, Eurocode does not
    - AISC provides no *w/t* limits for arbitrary compression+bending, Eurocode does
- AISC stiffened element *w/t* limits are generally similar to Eurocode
    - AISC's *w/t* limits for webs of rolled shapes is greater than Eurocode (note Eurocode does not distinguish between rolled and built-up shapes and thus provides no unique benefit to rotational stiffness provided from k-zones, etc.)
- AISC unstiffened element *w/t* limits are different from Eurocode
    - AISC unstiffened element flange λ_p limit is greater than even Class 2 for Eurocode (this implies that Eurocode would not predict even minimal rotational capacity for members with flanges at the AISC λ_p limit)
    - AISC's λ_r limit for unstiffened elements is significantly greater than Class 3 for Eurocode, more than double even for the simple case of a rolled flange
    - AISC's *w/t* limits for minor-axis bending of unstiffened elements do not consider the stress distribution explicitly, while Eurocode does and this can lead to stark differences (as whether or not the tip of the unstiffened element is in tension or compression changes the buckling solution)
    - AISC includes web/flange interaction for flanges of built-up shapes, Eurocode does not

3.1.2 Limited comparison with Japanese *w/t* limits

During committee deliberations additional information on Japanese *w/t* limits were assembled. A limited comparison for I-sections is provided in Table 6.

Table 6 AISC 360-16 vs AIJ 1 for I-Section Elements in Flexural Members

		AISC 360	AIJ 1	AIJ 1	AIJ 1	AISC 360	AIJ 1
	Unstiffened	λ_p	FA[a]	FB[b]	FC[c]	λ_r	FD[d]
10	Rolled Flange	$0.38\sqrt{\frac{E}{F_y}}$	$0.30\sqrt{\frac{E}{F_y}}$	$0.37\sqrt{\frac{E}{F_y}}$	$0.52\sqrt{\frac{E}{F_y}}$	$1.00\sqrt{\frac{E}{F_y}}$	–
	Stiffened						
15	Web (doubly-symm shape)	$3.76\sqrt{\frac{E}{F_y}}$	$1.46\sqrt{\frac{E}{F_y}}$	$1.52\sqrt{\frac{E}{F_y}}$	$1.63\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$	–

a. M_p and rotation capacity of 4, b. M_p and rotation capacity of 2, c. M_p, d. strength less than M_p

Similar to Eurocode the Japanese provide provisions for beams as well as beam-columns, that are not discussed here. The revised *w/t* limits when compression is present in the section can be significantly lower.

Major observations in comparing AISC 360 *w/t* limits to AIJ:

- AISC's λ_p limit for unstiffened element flanges is similar to AIJ 1's "FB" case which, different from Eurocode, implies AISC's limit provides M_p and at least a rotation capacity of 2.
- AISC's λ_p limit for stiffened element webs is much more relaxed (more than twice) that of AIJ's limits and suggests a strong difference in either the underlying data, theory, or objective in the application of this limit.
- AISC 360 does not provide a means to directly consider the impact of stress gradients other than pure compression or flexure on *w/t* limits; AIJ does provide a means to include this influence

3.1.3 Comparison with AASHTO *w/t* limits

The AASHTO *LRFD Bridge Design Specifications* (2017, 8th ed.) specify w/t limits for different steel elements throughout Chapter 6. Based on commentary provided in the AASHTO specifications, the *w/t* limits specified in the AASHTO specifications are largely based on limits provided throughout various editions of AISC 360. Consequently, the compact and noncompact *w/t* limits are very similar between AASHTO and AISC. Table 7 summarizes the differences between the AASHTO and AISC *w/t* limits.

Major observations in comparing AISC 360 *w/t* limits to AASHTO:

- For members subject to axial compression, all compact and noncompact *w/t* limits in AASHTO are identical to those in AISC 360-16.
- For members subject to flexural compression, all compact and noncompact *w/t* limits in AASHTO are identical to those in AISC 360-16 except for the cases listed in Table 7.
- As shown in the table, the only two cases for which *w/t* limits directly differ between AASHTO and AISC are (1) the noncompact limit for I-shape and channel flanges and (2) the noncompact limit for box section flanges. In each of these cases, the AASHTO limit is more restrictive than the AISC limit.
    - For I-shape and channel flanges, the AASHTO commentary states that the noncompact *w/t* limit is based on the 1999 AISC specifications, indicating that the AASHTO limit has not been updated to reflect subsequent changes that have been made by AISC.
    - For box section flanges, AASHTO does not distinguish between rectangular HSS sections and box sections and instead uses the more conservative noncompact limit for rectangular HSS sections from AISC for both types of sections.
- The remainder of the differences indicated in Table 7 consist of cases for which AASHTO does not specify *w/t* limits for various reasons.

Table 7 Differences between AASHTO and AISC *w/t* limits for members subject to flexural compression

AISC Table B4.1 Case No. and Description	AISC Compact Limit (λ_p)	AASHTO Compact Limit (λ_p)	Notes
12: Legs of single angles	$0.54\sqrt{\frac{E}{F_y}}$	None	AASHTO does not allow the use of single angles as pure flexural members.
14: Stems of tees	$0.84\sqrt{\frac{E}{F_y}}$	None	AASHTO does not provide local buckling checks for stems of tees because "lateral torsional buckling and local buckling of the stem are essentially the same phenomenon."
18: Cover plates	$1.12\sqrt{\frac{E}{F_y}}$	None	AASHTO does not provide width-to-thickness limits for cover plates in flexure. Presumably these would be considered as pure compression elements under Case No. 7.
AISC Table B4.1 Case No. and Description	**AISC Noncompact Limit (λ_r)**	**AASHTO Noncompact Limit (λ_r)**	**Notes**
10: Flanges of rolled channels	$1.0\sqrt{\frac{E}{F_y}}$	None	AASHTO requires all channel flanges to be compact.
12: Legs of single angles	$0.91\sqrt{\frac{E}{F_y}}$	None	AASHTO does not allow the use of single angles as pure flexural members.
13: Flanges of all I-shaped sections and channels in flexure about the minor axis	$1.0\sqrt{\frac{E}{F_y}}$	$0.83\sqrt{\frac{E}{F_y}}$	The AASHTO limit is based on the 1999 AISC code and appears to not have been updated to reflect changes in the AISC code.
14: Stems of tees	$1.52\sqrt{\frac{E}{F_y}}$	None	AASHTO does not provide local buckling checks for stems of tees because "lateral torsional buckling and local buckling of the stem are essentially the same phenomenon."
15: Channel webs	$5.70\sqrt{\frac{E}{F_y}}$	None	AASHTO requires all channel webs to be compact.
18: Cover plates	$1.40\sqrt{\frac{E}{F_y}}$	None	AASHTO does not provide width-to-thickness limits for cover plates in flexure. Presumably these are considered as pure compression elements under Case No. 7.
19: Rectangular HSS and box section webs	$5.70\sqrt{\frac{E}{F_y}}$	None	AASHTO does not consider the elastic local buckling limit state for rectangular HSS and box section webs.
21: Box section flanges	$1.49\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$	AASHTO does not distinguish between rectangular HSS and box sections.

It is worth noting that as of Fall 2019 AASHTO is undergoing significant revision with respect to its local buckling provisions this code cycle. The updates are based primarily on the report of White et al. (2019) for box sections and provides a number of improvements: the unified effective width method is adopted for local post-buckling; the notion of Class 2 behavior where M_p can be reached, but large rotation capacity is not necessary is introduced; improved provisions are provided for longitudinally stiffened plates; interaction of axial, bending, shear, and torsion are considered in a single framework; and other improvements.

3.1.4 Comparison with AISI S100 *w/t* limits

The cold-formed steel AISI S100-16 standard does not employ *w/t* limits in the same manner as AISC 360. However, in some instances AISI methodologies can be converted to those of AISC. Determination of the *w/t* limit follows from:

$$\lambda^* = \sqrt{\frac{F_y}{F_{cr}}} = \sqrt{\frac{12(1-\nu^2)F_y}{k\pi^2 E}}\frac{\mathrm{w}}{\mathrm{t}} \tag{9}$$

$$\frac{w}{t} = \lambda^* \sqrt{\frac{k\pi^2}{12(1-\nu^2)}}\sqrt{\frac{E}{F_y}} \tag{10}$$

For the AISC λ_r *w/t* limit AISI's assumption for a stiffened element in compression is λ^*=0.673 and *k*=4.0, and in bending λ^*=0.673 and *k*=23.9 (assuming a section symmetric about the axis of bending), for an unstiffened element in compression λ^*=0.673 and *k*=0.425 and in bending λ^*=0.673 and k=0.648~12.6 (assuming the ENA through 1/3 element depth and compression either on the unsupported edge or supported edge). AISI does not have a λ_p *w/t* limit directly parallel to AISC; however in AISI S100-16 inelastic reserve provisions were added that allow for development of M_p, these are based on cross-section slenderness, not element slenderness, at λ^*=0.086 the section is assumed to develop an extreme fiber strain of 3 times yield – consistent with M_p – this λ^* limit is converted to element slenderness here. Similar to Eurocode, AISI does not distinguish between compression or flexural members. A comparison is provided in Table 8.

Table 8 AISC 360-16 vs AISI S100 for Elements in Compression and Flexural Members

		AISC 360	AISI	AISC 360	AISC 360	AISI
Case	Unstiffened	λ_p	$M_{n\ell o} = M_p$	λ_r (Flex.)	λ_r (Comp.)	$M_{n\ell o} = M_y$
1,10	Rolled Flange	$0.38\sqrt{\frac{E}{F_y}}$	$0.07\sqrt{\frac{E}{F_y}}$	$1.00\sqrt{\frac{E}{F_y}}$	$0.56\sqrt{\frac{E}{F_y}}$	$0.42\sqrt{\frac{E}{F_y}}$
2,11	Built-up Flange [a]	$0.38\sqrt{\frac{E}{F_y}}$	$0.07\sqrt{\frac{E}{F_y}}$	$0.56\sim0.83\sqrt{\frac{E}{F_L}}$	$0.38\sim0.56\sqrt{\frac{E}{F_y}}$	$0.42\sqrt{\frac{E}{F_y}}$
3,12	Angle leg, other	$0.54\sqrt{\frac{E}{F_y}}$	$0.07\sqrt{\frac{E}{F_y}}$	$0.91\sqrt{\frac{E}{F_y}}$	$0.45\sqrt{\frac{E}{F_y}}$	$0.42\sqrt{\frac{E}{F_y}}$
13	flange in minor-axis [c]	$0.38\sqrt{\frac{E}{F_y}}$		$1.00\sqrt{\frac{E}{F_y}}$	N/A	$0.52\sim2.27\sqrt{\frac{E}{F_y}}$
	Stiffened					
15	Web (doubly-symm shape)	$3.76\sqrt{\frac{E}{F_y}}$	$0.40\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$	N/A	$3.13\sqrt{\frac{E}{F_y}}$
19	Web HSS & box	$2.42\sqrt{\frac{E}{F_y}}$	$0.40\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$	N/A	$3.13\sqrt{\frac{E}{F_y}}$
17	Flange HSS	$1.12\sqrt{\frac{E}{F_y}}$	$0.16\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$	$1.49\sqrt{\frac{E}{F_y}}$	$1.28\sqrt{\frac{E}{F_y}}$
18	Flange cover plate	$1.12\sqrt{\frac{E}{F_y}}$	$0.16\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$	$1.28\sqrt{\frac{E}{F_y}}$
21	Flange box	$1.12\sqrt{\frac{E}{F_y}}$	$0.16\sqrt{\frac{E}{F_y}}$	$1.49\sqrt{\frac{E}{F_y}}$	$1.49\sqrt{\frac{E}{F_y}}$	$1.28\sqrt{\frac{E}{F_y}}$

a. AISC provisions a function of web h/t_w, bounds provided here, Case 2: $F_L = F_y$, Case 11: $F_L = 0.7F_y$

c. AISI provisions provide buckling as a function of whether unsupported tip is in compression or tension and specific to the elastic stress distribution on the unstiffened element. Typical ranges provided here.

$M_{n\ell o}$ in AISI S100 is the strength of a member with full global bracing, thus only considering local buckling

Major observations in comparing AISC 360 *w/t* limits with AISI:

- AISC's λ_r limits for flexure are fundamentally different than those in compression and significantly higher than those following AISI's approach.
- AISI's *w/t* limits for establishing M_p are more conservative than limits used in all other Specifications. (Use of strength greater than M_y is new to AISI S100-16 and the selected implementation appears highly conservative).

3.2 Plate buckling assumptions implied in AISC 360 *w/t* limits

If one considers a given *w/t* limit expressed by the coefficient *C* of Eq. (3), this coefficient may alternatively be understood as an assumption about (1) the plate buckling coefficient, k, i.e., the loading and boundary conditions of the plate, and (2) the necessary plate buckling stress (or strain) to sustain the desired load or stress/strain, i.e, either $a = F_{cr}/F_y$ or $\lambda^* = \sqrt{F_y/F_{cr}}$ per Eq. (5) must be known. More specifically:

$$\lambda_r = \left(\frac{w}{t}\right)_r = \lambda^* \sqrt{\frac{k\pi^2}{12(1-\nu^2)}} \sqrt{\frac{E}{F_y}} = C\sqrt{\frac{E}{F_y}} \qquad (10)$$

The most complete discussion of the underlying assumptions for the AISC 360 *w/t* limits can be found in Salmon et al. (2009). For the λ_r limits Salmon et al. (2009) provide the assumed λ^* and the plate buckling coefficient, k – and here we demonstrate that they match current AISC 360-16 compression λ_r exactly in Table 9.

Table 9 Assumptions underlying AISC 360-16 *w/t* limits - λ_r Compression Only

		k	$\lambda^* = \sqrt{\frac{F_y}{F_{cr}}}$	Eq. (10)	AISC
	Unstiffened			λ_r	λ_r
1	Rolled Flange	0.70[b]	0.70[a]	$0.56\sqrt{\frac{E}{F_y}}$	$0.56\sqrt{\frac{E}{F_y}}$
2	Built-up Flange	0.35~0.76	0.70[a]	$0.39\sim0.58\sqrt{\frac{E}{F_y}}$	$0.38\sim0.56\sqrt{\frac{E}{F_y}}$
3	Angle leg, other	0.425[e]	0.70[a]	$0.43\sqrt{\frac{E}{F_y}}$	$0.45\sqrt{\frac{E}{F_y}}$
4	Stem of tee	1.277[f]	0.70[a]	$0.75\sqrt{\frac{E}{F_y}}$	$0.75\sqrt{\frac{E}{F_y}}$
	Stiffened				
5	Rolled Web	5.0[c]	0.70[a]	$1.49\sqrt{\frac{E}{F_y}}$	$1.49\sqrt{\frac{E}{F_y}}$
6	HSS Wall	4.4[d]	0.70[a]	$1.40\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$
7	Cover plate	4.4[d]	0.70[a]	$1.40\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$
8	Other	5.0[c]	0.70[a]	$1.49\sqrt{\frac{E}{F_y}}$	$1.49\sqrt{\frac{E}{F_y}}$

a. non-dimensional slenderness to achieve a plate strength approaching F_y
b. ~1/2 way between pinned and fixed k values
c. ~1/3 of the way between pinned and fixed k values
d. this k factor back-calculated from λ^* and *w/t* limit
e. ideal case for simple-free longitudinal edge conditions
f. ideal case for fixed-free longitudinal edge condition

Table 10 Assumptions underlying AISC 360-16 *w/t* limits - λ_r, λ_p Flexure

		k	$\lambda_p^* = \sqrt{\frac{F_y}{F_{cr}}}$	Eq. (10)	AISC 360	k	$\lambda_r^* = \sqrt{\frac{F_y}{F_{cr}}}$	Eq. (10)	AISC 360
	Unstiffened			λ_p	λ_p			λ_r	λ_r
10	Rolled Flange	0.7[j]	0.464[k]	$0.37\sqrt{\frac{E}{F_y}}$	$0.38\sqrt{\frac{E}{F_y}}$	0.7[j]	1.0 [b**]	$0.80\sqrt{\frac{E}{F_y}}$	$1.00\sqrt{\frac{E}{F_y}}$
11	Built-up Flange	0.35 ~ 0.76[f]	0.464[k]	$0.26 \sim 0.38\sqrt{\frac{E}{F_y}}$	$0.38\sqrt{\frac{E}{F_y}}$	0.35 ~ 0.76[f]	1.0[b*]	$0.56 \sim 0.83\sqrt{\frac{E}{F_y}}$	$0.56 \sim 0.83\sqrt{\frac{E}{F_y}}$
12	Angle leg	0.90[i]	0.464[k]	$0.42\sqrt{\frac{E}{F_y}}$	$0.54\sqrt{\frac{E}{F_y}}$	0.90[i]	1.0 [b**]	$0.90\sqrt{\frac{E}{F_y}}$	$0.91\sqrt{\frac{E}{F_y}}$
13	flange in minor-axis	0.7[j]	0.464[k]	$0.37\sqrt{\frac{E}{F_y}}$	$0.38\sqrt{\frac{E}{F_y}}$	1.1[i]	1.0 [b**]	$1.00\sqrt{\frac{E}{F_y}}$	$1.00\sqrt{\frac{E}{F_y}}$
14	Stem of tee (flexure)	2.6[i]	0.464[k]	$0.71\sqrt{\frac{E}{F_y}}$	$0.84\sqrt{\frac{E}{F_y}}$	2.6[i]	1.0 [b**]	$1.53\sqrt{\frac{E}{F_y}}$	$1.52\sqrt{\frac{E}{F_y}}$
	Stiffened								
15	Web (doubly-symm)	36[a2]	0.56[h]	$3.19\sqrt{\frac{E}{F_y}}$	$3.76\sqrt{\frac{E}{F_y}}$	36[a]	1.0[b]	$5.70\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$
19	Web HSS & box	36[a2]	0.56[h]	$3.19\sqrt{\frac{E}{F_y}}$	$2.42\sqrt{\frac{E}{F_y}}$	36[a]	1.0[b]	$5.70\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$
16	Web (singly-symm)	36[a2*]	0.56[h]	$3.19\sqrt{\frac{E}{F_y}}$[g]	$3.76\sqrt{\frac{E}{F_y}}$[g]	36[a*]	1.0[b]	$5.70\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$
17	Flange HSS	4.4[e]	0.56[h]	$1.12\sqrt{\frac{E}{F_y}}$	$1.12\sqrt{\frac{E}{F_y}}$	4.4[e]	0.7[c]	$1.40\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$
18	Flange cover plate	4.4[e]	0.56[h]	$1.12\sqrt{\frac{E}{F_y}}$	$1.12\sqrt{\frac{E}{F_y}}$	4.4[e]	0.7 [c]	$1.40\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$
21	Flange box	5.0[d]	0.56[h]	$1.19\sqrt{\frac{E}{F_y}}$	$1.12\sqrt{\frac{E}{F_y}}$	5.0[d]	0.7 [c]	$1.49\sqrt{\frac{E}{F_y}}$	$1.49\sqrt{\frac{E}{F_y}}$

a. k based on symm. bending, 80% of difference from pinned (23.9) fixed (39.6) per Salmon et al. (2009)
a2. k based on elastic stress distribution, if plastic stress dist. used k pinned (10.3) k fixed (15.4), k80% (14.4)
a*. note k based on bending about symmetry axis, but k would be a function of ENA location in reality
b. For flexure AISC typ. assumes $F_{cr}=F_y$ sufficient for extreme fiber of web to reach F_y (Salmon et al. 2009)
b*. Built-up flanges also appear to use $F_{cr}=F_y$ as sufficient for λ_r^*, as Eq. 10 matches AISC exactly in this case
b**. $F_{cr}=F_y$ assumed for λ_r^*, because (i) of agreement for b*, (ii) use of b, and even fixed values for k are not high enough to give AISC slenderness limits with $\lambda_r^* = 0.7$ as was done in compression
c. For stiffened element flanges AISC uses same normalized slenderness criteria as for compression members
d. ~1/3 of the way between pinned and fixed k values for pure compression
e. this k factor back-calculated from compression λ^* and *w/t* limit, same in flexure as compression
f. k factor at the limits of expression provided in AISC 360: k=$0.35 < 4/\sqrt{h/t_w} < 0.76$
g. expression varies, value here for ENA=PNA and Mp/My=1.12 (typical rolled shape I), i.e. the symm. limit
h. 0.56 assumed, based on (a) Haaijer and Thurlimann (1960) see Salmon et al. 2009 $\lambda^* = 0.56$ onset of strain hardening in unstiffened element, and (b) CSM base curve by Gardner implies $2\epsilon_y$ at this slenderness
i. back-calculated from assumed flexure $\lambda_r^* = 1.0$
j. k based on ~1/2 way between pinned and fixed k values
k. 0.46 assumed, based on (a) Haaijer and Thurlimann (1960) see Salmon et al. 2009 $\lambda^* = 0.46$ is onset of strain hardening in unstiffened element, and (b) CSM base curve by Gardner implies $4\epsilon_y$ at this slenderness

Examination of the underlying assumptions in flexure are more complex. Nonetheless, it can be completed with some success and is provided for the λ_r and λ_p limits in Table 10. Completion of this effort reveals some key assumptions embedded within the current AISC 360 *w/t* limits. It is important to note, particularly for the λ_p limits, that the plastic strength limits are usually not derived on the basis of Eq. (10) or similar; rather, they are determined experimentally. Here we are able to observe after the fact if simple unifying methods/assumptions still exist despite the largely experimental basis.

Major observations in examining underlying plate buckling assumptions for AISC 360 *w/t* limits:

Compression Members

- For all elements $\lambda_r^* = \sqrt{F_y/F_{cr}} = 0.7$ implying $F_{cr} \cong 2F_y$ is necessary for an element to reach its yield stress, this is predicated upon certain assumptions about the plate bucking coefficient (k), but is consistent across the *w/t* limits
- AISC assumes singular k values and ignores element interaction (in all but one case), selected k values are generally between simply supported and fixed edge boundary conditions, except for stems of tees which use the maximum fully-fixed edge condition assumption.

Flexural Members

- AISC generally employs $\lambda_r^* = \sqrt{F_y/F_{cr}} = 1.0$ implying $F_{cr} = F_y$ is all that is necessary for an element to reach its target stress (i.e., F_y or F_y-F_r) at the extreme compression fiber. This is more liberal than λ_r^* used for elements in compression members.
  - o AISC extends this more liberal $\lambda_r^* = 1.0$ even for unstiffened element flanges that are part of a flexural member.
  - o AISC does not extend thid more liberal λ_r^* to stiffened element flanges that are part of a flexural member; these elements use the same λ_r^* as in compression.
  - o The use of the more liberal $\lambda_r^* = 1.0$ appears to originate in past practice for plate girder design. AISI and Eurocode do not make this assumption, leading to fairly stark difference for flexural member *w/t* limits.
  - o Buried in these comparisons are past use of F_L which liberalizes the *w/t* limit, and whether or not the limit is intended to achieve M_y or M_r. See Section 3.2.1 for detailed discussion on this point.
  - o k values follow the same overall logic as for compression members; however some cases are hard to finalize – e.g., case 10 for a rolled flange using k=0.7 and $\lambda_r^* = 1.0$ still results in a more conservative *w/t* limit than specified in AISC 360; potentially due to F_L in past use (a k=1.1 provides agreement with AISC 360). See Section 3.2.1 for detailed discussion on this point.
- AISC λ_p limits may be approximately understood as being derived from limits on the nondimensional slenderness $\lambda_p^* = \sqrt{F_y/F_{cr}}$; historically (Haaijer and Thurlimann 1960) this has been based on mechanical approximations setting $\lambda_p^* \cong 0.46$ for unstiffened

elements and $\lambda_p^* \cong 0.58$ for stiffened elements. Today, based on the work of Gardner et al. (e.g., Afshan and Gardner 2013, Zhao et al. 2017) this might be characterized as providing $4\epsilon_y$ for unstiffened elements and $2\epsilon_y$ for stiffened elements.

- Unstiffened element λ_p generally follow the $\lambda_p^* \cong 0.46$; however it is not clear why case 12 (legs of single angles) has a more relaxed λ_p limit than case 10 flanges of rolled shapes – since case 12 includes the possibility of the angle leg bent about a geometric axis that places the entire element in compression (essentially the same as case 10).
- Stiffened element λ_p when the element is in compression are generally consistent with the overall practice regarding a limiting $\lambda_p^* \cong 0.56$, this is true even for HSS where the limit was derived experimentally on full sections without direct consideration of the underlying assumptions.
- Stiffened element λ_p when the element is in flexure do not agree particularly well with the overall assumption of slenderness $\lambda_p^* \cong 0.56$. Further, if the *k* is based on the plastic stress distribution, not the elastic stress, *k* would be considerably lower, leading to even larger disagreement between assumed and actual λ_p in current AISC 360 practice for stiffened elements in flexure.

Table 9 and Table 10 provide the $k, \lambda_p^*, \lambda_r^*$ that underpin the AISC 360 *w/t* limits. Knowing these quantities that lead to current *w/t* limits is helpful, but a deeper question remains: are the selected values correct? A simple answer is that strong disagreement with testing has not been found, so the values are adequate. However, good reasons (interest in other materials, different built-up shapes, collapse prediction, etc.) exist for taking a closer look.

The assumed *k* values do not agree particularly well with elastic buckling solutions, as summarized in Seif and Schafer (2010). Thus, a deeper look at web-flange interaction is warranted – see Section 5.

The use of different λ_r^* limits for a cross-section element in a compression member ($\lambda_r^* = 0.7$) and the same element, in compression, in a flexural member ($\lambda_r^* = 1.0$) is odd at best, inconsistent for sure, and potentially an indicator that something is amiss with the objectives and application of *w/t* limits in flexure. The HSS sections use the same $\lambda_r^* = 0.7$ for compression and flexure, as does Eurocode, AISI, and AJI – this seems like where the AISC Specification should be aiming. Today, with current yield stress values, the strength impact of the *w/t* (λ_r) limit is mild, but with more sections moving from compact to non-compact or even slender with higher yield stress material, it is worth re-investigating and settling this value. See Section 3.2.1 below for further examination of this issue.

The experimentally derived λ_p limits are in reasonably good agreement with the classical derivations provided by Haaijer and Thurlimann (1960). However, modern analysis suggests that the element level strain targets for stiffened and unstiffened elements has been chosen differently (i.e. by using different λ_p^*) which potentially leads to inconsistent rotation capacity depending on the flange type. Direct strain-based methods are worthy of consideration – see Section 6.

3.2.1 Further examination of λ_r limits in flexure

The seemingly inconsistent nature of current λ_r limits in flexure warrants additional study to provide insight into why the *w/t* limits have evolved in this manner. The primary complication is alluded to in Section 2.1, i.e., λ_r in flexure is tied to either M_y or ~$0.7M_y$. A review of the application of λ_r in chapter F of AISC 360-16 is provided in Table 11. The λ_r for unstiffened elements establishes $M_r \sim 0.7M_y$, while for stiffened elements, in flexure, λ_r intends to establish $M_r = M_y$. As detailed in Table 11 for some cases the connection is explicit, while in other cases substitution of appropriate λ_r values must be completed to determine the strength that $\lambda = \lambda_r$ implies.

Table 11 Application of AISC 360-16 λ_r limits in Flexure in Chapter F

Sect.	Cross-section	Limit State	λ	M_r	Note	B4.1b Case
F3	I-doubly symm	FLB	λ_{rf}	$0.7M_y$	Explicit in F3-1	10
F4	I-singly	FLB	λ_{rf}	$0.7M_y$	or lower per S_{xt}/S_{xc}	10, 11
F5	I	FLB	λ_{rf}	$0.7M_y$	Explicit in F5-8	10, 11
F10	L	LB	λ_r	$0.86M_y$	Implicit in F10-6	12
F6	I, C, minor	FLB	λ_{rf}	$0.7M_y$	Explicit in F6-2	13
F9	Tee, 2L	FLB	λ_{rf}	$0.7M_y$	Explicit in F9-14	10
F9	Tee, 2L	LB Flexure	λ_r	$0.65M_y$	Implicit in F9-18	14
F5	I	WLB	λ_{rw}	M_y	Implicit in R_{pg} per F5-6	15, 16
F5	I	WLB-LTB	λ_{rw}	M_y	Implicit in R_{pg} per F5-6	15, 16
F7	Box, HSS	FLB	λ_{rf}	M_y	Implicit in F7-2	17, 21
F7	Box, HSS	WLB	λ_{rw}	M_y	Implicit in F7-6	19
F7	Box, HSS	WLB-LTB	λ_{rw}	M_y	Implicit in R_{pg} per F5-6	19

The use of λ_r for flanges must be understood in the context of the strength predictions of Chapter F of ASIC 360. In the prototypical Flange Local Buckling (FLB) case Figure 2 illustrates the solution. It can be observed that λ_r is an anchor point in the strength prediction, and typically tied to $0.7M_y$. AISC 360-16 does not typically account for post-buckling so for more slender elements the strength prediction transitions to the plate elastic buckling solution (even if the actual strength falls above this curve). Note, that AISI S100 anchors to M_y instead of M_r and includes post-buckling.

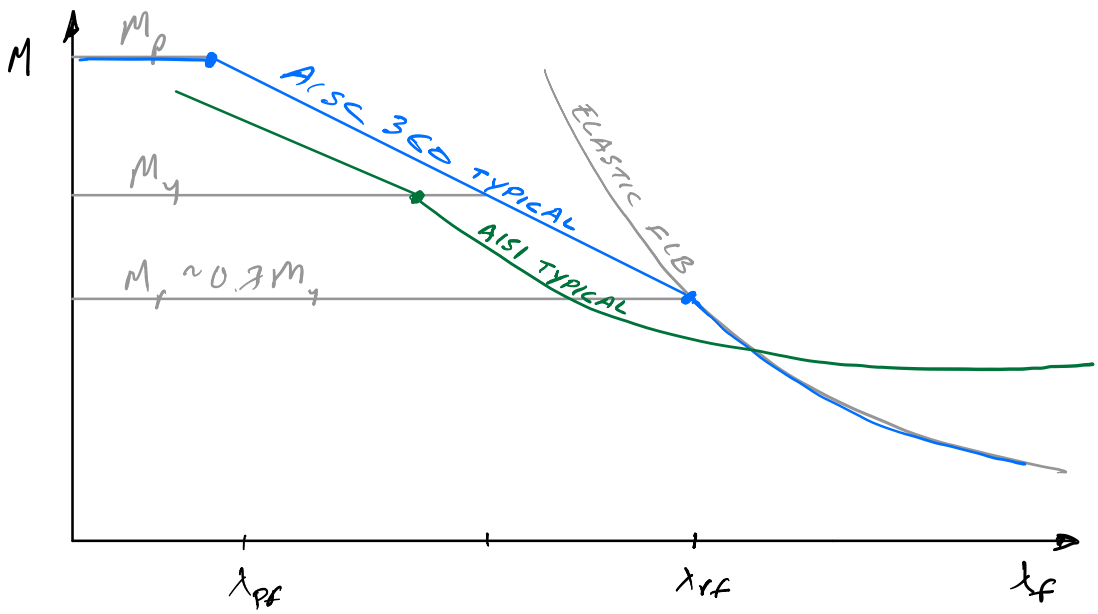


Figure 2 Typical application of λ_r in Flange Local Buckling for AISC 360

To develop $M_r = M_y$ one would expect $\lambda^* \sim 0.7$. To develop $M_r = 0.7M_y$ one would expect $\lambda^* > 0.7$ and the best evidence is that AISC360-16 uses $\lambda^* = 1.0$ for this case. As first discussed in the context of Eq. (5) for λ^*, Winter's equation provides an approximation of the effectiveness (ρ) of a slender element. Per Winter's expression if $\lambda^* \geq 0.673$, $\rho = (1 - 0.22/\lambda^*)/\lambda^*$. Considering the compression flange of a flexural member one can solve for the higher values of λ^* that would be allowed if λ_r is targeting a stress or capacity less than F_y. This exercise is completed in Table 12 and the impact of this knowledge is also applied to Case 10 (rolled flange unstiffened element) of Table 10 (Table B4.1b).

Table 12 Prediction of Winter's Equation for $M_r < M_y$, i.e. $F_r < F_y$ and Example of Impact

		Impact of assumption on Case 10		
ρ or F_r/F_y	$\lambda^* = \sqrt{F_y/F_{cr}}$	k	Eq. (10) λ_r	AISC 360 λ_r
1.0	0.673	0.7	$0.54\sqrt{\frac{E}{F_y}}$	$1.00\sqrt{\frac{E}{F_y}}$
0.86	0.868	0.7	$0.69\sqrt{\frac{E}{F_y}}$	$1.00\sqrt{\frac{E}{F_y}}$
0.78	1.000	0.7	$0.80\sqrt{\frac{E}{F_y}}$	$1.00\sqrt{\frac{E}{F_y}}$
0.7	1.155	0.7	$0.92\sqrt{\frac{E}{F_y}}$	$1.00\sqrt{\frac{E}{F_y}}$
0.65	1.272	0.7	$1.01\sqrt{\frac{E}{F_y}}$	$1.00\sqrt{\frac{E}{F_y}}$

The use of λ_r for webs in Chapter F is primarily handled through the R_{pg} reduction. Note, the connection between web slenderness limits and bending strength is particularly indirect and strongly dependent on the flange as the flange contributes much more to cross-section moment of inertia (I) and/or plastic section modulus (Z) than the web. Thus, a large error in a web slenderness limit may have only a small impact on the flexural strength prediction of many common sections. Nonetheless, the use of $\lambda_r^* = 1.0$ for the $M_r = M_y$ cases (15,16,19) is difficult to justify based on plate mechanics arguments.

3.3 Proposed AISC 360 objectives for non-seismic *w/t* limits

Based on the analyses of this Chapter and consistent with the original Task 2, the following objectives for non-seismic *w/t* limits are proposed.

Members under Axial Compression

λ_r: Provides the slender/non-slender limit for the element. Specifically for *w/t* ≤ λ_r the element can develop its yield capacity. If all elements in a cross-section have *w/t* ≤ λ_r then the cross-section can develop its yield (squash) capacity, i.e., $P_y = A_g F_y$

Members under Flexure

λ_r: Provides the noncompact-slender limit for the element. Specifically for *w/t* ≤ λ_r the element can develop its yield capacity. If all elements in a section have *w/t* ≤ λ_r then the cross-section can develop its first yield capacity in bending, i.e., $M_y = SF_y$.

> **Note on M_r**: Setting the objective that λ_r provides $M_r = M_y$ requires significant modification to AISC 360 Chapter F. However, it means different λ_r criteria will generally not be needed between compression and flexure. Alternatively, λ_r could be set to $M_r = 0.7M_y$ throughout, this is consistent and rational too, though not favored.
>
> **Note on F_y vs. F_L**: It is recommended, wherever possible, to remove the use of F_L and consideration of residual stresses in developing the λ_r width-to-thickness limits. The use of the reduced stress F_L relaxes the slenderness limit, and decreases the strain capacity of the element below that of an element which can sustain the yield strain in the element (even if it can reach the yield force due to beneficial residual stresses). The objective of λ_r is best implemented as providing M_y for the section. The use of λ_r based on F_L to establish an elastic limit (a past practice) is imprecise at best, and not recommended.

λ_{p2}: Provides the compact-noncompact limit for the element, specifically for *w/t* ≤ λ_{p2} the element can develop its fully plastic capacity. If all elements in the cross-section have *w/t* ≤ λ_{p2}, then the cross-section can develop its fully plastic capacity in bending, i.e., $M_p = ZF_y$.

λ_{p1}: Provides the inelastic-compact limit for the element, specifically for *w/t* ≤ λ_{p1} the element can develop its full plastic capacity and sustain that capacity up to approximately $4\epsilon_y$. If all elements in the cross-section have *w/t* ≤ λ_{p1} then the cross-section can develop its fully plastic capacity in bending, i.e., $M_p = ZF_y$ and a nominal inelastic rotation capacity, i.e., $R_{cap} = 3$.

> **Note on λ_{p1} and λ_{p2}:** It is conservative to use λ_{p1} criteria for λ_{p2}. AISC 360-16 essentially uses $\lambda_p = \lambda_{p1}$. Economy may exist in creating λ_{p2} criteria; however there is no safety concern driving this change. Other notation may also be logical, for example when LRFD was first introduced in 1986 λ_{pd} was used to denote a limit similar to λ_{p1}.
>
> **Note on $R_{cap} = 3$:** Element width-to-thickness limits can only influence element strain capacity. The connection between rotation capacity and element strain capacity is cross-section dependent. Nonetheless for a given cross-section and some broad dimensional limits on the section it is possible to establish minimum R_{cap}. Typically a flange element that can sustain a membrane strain of 4 times the yield strain is sufficient.

3.4 Proposed AISC 341 objectives for seismic *w/t* limits

It is proposed that the objectives for AISC 341 λ_{md} and λ_{hd} be expressed at the element level, component level, system (SFRS) level, and ancillary objectives. The ad hoc task group did not have sufficient time to develop final recommendations, but a draft of these objectives is provided for consideration:

λ_{md} Objectives

Element: For *w/t* = λ_{md} the element can develop its full plastic capacity and sustain that capacity up to approximately $4\epsilon_y$. In addition, this strain capacity can be sustained up to m cycles.

Component: A member comprised of elements with $w/t \leq \lambda_{md}$ can develop M_p and at least 0.02 radians of rotation. Further, such a member can sustain/deliver M_p up to and including strain hardening, M_{pe}.

System: Provide sufficient component ductility such that a given SFRS can develop its system strength, R_r, and last several (n) cycles at that strength, and through a specific inter-story drift.

Ancillary: -

λ_{hd} Objectives

Element: For *w/t* = λ_{hd} the element can develop its full plastic capacity and sustain that capacity up to approximately $15\epsilon_y$. In addition, this strain capacity can be sustained up to n (n>m) cycles.

Component: A member comprised of elements with $w/t \leq \lambda_{hd}$ can develop M_p and at least 0.04 radians of rotation at a post-peak strength no less than $0.8M_p$. Further, such a member can sustain/deliver M_p up to and including strain hardening, M_{pe}.

System: Provide sufficient component ductility such that a given SFRS can develop its system strength, R_r, and last several (m) cycles at that strength, and through a specific inter-story drift.

Ancillary: -

Note, Section 4.2 and Chapter 6 provide details on the specific connection between *w/t* limits and strain capacity. These strain capacities have all been developed on monotonic testing – the cyclic plate performance is needed for application to AISC 341. This effort is similar in spirit to the work on braces that established SCBF criteria.

4 Task 3: Impact of material on local buckling (*w/t*) limits

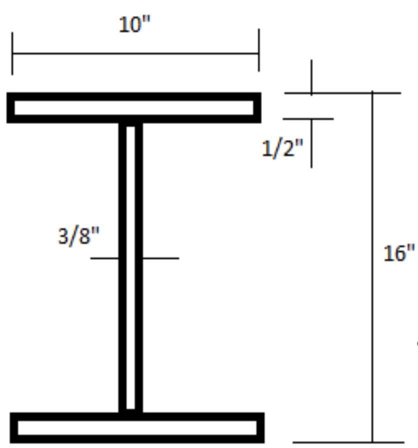
The task group was charged with (a) to recommend whether nominal or expected F_y is appropriate for λ limits in AISC 341 and AISC 360, and (b) provide guidance on other material criteria implicit in λ limits (F_u/F_y, E_{st}, COV of F_y, etc.). Definitive conclusions were completed for the first task and are provided in this chapter. An initial discussion was completed for the second task and is also summarized in this chapter.

4.1 Nominal vs. Expected F_y

The use of R_yF_y in AISC 341-16 and F_y in AISC 360-16 for the *w/t* limits creates a discrepancy for the user that requires attention and explanation. If it is important to use the best estimate of the mean F_y in seismic design (R_yF_y), why not do so in non-seismic? Also has the introduction of R_yF_y been completed properly, and does it meet the desired intent in AISC 341-16 when applied?

4.1.1 AISC 360

The task group considered if the increased yield strength modifier, R_y, that is used in AISC 341 should also be included in the AISC 360 Specification. The actual F_y is on average greater than the nominal F_y used in design. This opens the possibility that a compact section based on the nominal F_y may actually be a non-compact section since the actual λ may less than λ_p. The counter argument is that the design capacity based on F_y will be conservatively less than the capacity based on R_yF_y even if the member is no longer compact. This is illustrated in the following example.



$S_x = 88.3$ in^3

$Z_x = 98.6$ in^3

$\lambda = b/t = 5/(1/2) = 10$

For $F_y = 36$ ksi

$\lambda_P = 10.8$, Compact

$M_n = 295$ k-ft EQ. F2-1

For $F_y = 54$ ksi, $R_y = 1.5$

$\lambda_P = 8.81$ Non-compact

$\lambda_r = 23.2$

$M_n = 430$ k-ft EQ. F3-1

For the structures, loadings and margin of safety in AISC 360, large overloads are not expected and the actual mode of failure is not important. This is not true for AISC 341 where structures undergo extreme conditions. In this case the failure mode could potentially cause the energy absorbing location to shift from the intended location to an undesirable location resulting in nonductile fracture modes.

Therefore, it is <u>not</u> recommended that R_y be included in AISC 360.

4.1.2 AISC 341

The task group supports the use of R_y in the *w/t* limits for AISC 341 as it provides a more accurate prediction of the desired behavior for the particular limit. Further, it removes the perverse incentive of specifying a lower F_y, even when expected F_y is high, only so that a compactness limit or other limit related to energy dissipation can be met.

However, the implementation of the R_y factor in the existing *w/t* limits requires discussion. AISC 341-16 introduced the material factor R_y into its *w/t* limits, but in such a manner as to not actually change the limit for typical steels.

As an example for I-section flanges in AISC 341-16:

$$\lambda_{md} = 0.40\sqrt{\frac{E}{R_y F_y}} \tag{11}$$

while previously in AISC 341-10:

$$\lambda_{md} = 0.38\sqrt{\frac{E}{F_y}} \tag{12}$$

For $R_y = 1.1$, consistent with modern A992 steels, the two expressions yield the same limit:

$$\lambda_{md} = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{1.1}{R_y}}\sqrt{\frac{E}{F_y}} = 0.40\sqrt{\frac{E}{R_y F_y}} \tag{13}$$

For steels with $R_y > 1.1$ AISC 341-16 will provide a more stringent *w/t* requirement than AISC 341-10. For example A36 steel has an $R_y = 1.5$ and thus the 2016 provisions provide a much stricter *w/t* limit for that material.

The original experimental source for the λ_p limit (Lukey and Adams 1969), which λ_{md} is based on, was experimentally developed based on measured F_y, but then applied in AISC 360 and later in AISC 341 as nominal/specified F_y. If the change in 2016 for AISC 341 was intended to bring the *w/t* limit in line with the original testing, then the coefficient should not have been modified and only R_y added to the denominator. In general researchers develop *w/t* limits with measured F_y properties and code committees then implement them with specified properties.

In essence, two options for inclusion of R_y exist

$$Option\ 1: \lambda = C_{2010}\sqrt{1.1}\sqrt{\frac{E}{F_y}} \quad Option\ 2: \lambda = C_{2010}\sqrt{\frac{E}{R_y F_y}} \tag{14}$$

AISC 341-16 took Option 1, while Option 2 would align best with conducted experiments.

Case for continuing with Option 1: Closer inspection of the Lukey and Adams (1969) data that led to Eq. 12 shows that for the small tested data set, based on coupons taken from the flanges, $F_{ymean}/F_{yspecified}$ is 1.13. If one argues that AISC 341 has always included this small ~1.1 bias then modifying the coefficients to embed this bias is logical. At the same time, if the bias is bigger than 1.1 ($R_y > 1.1$), a correction would be called for; Option 1 provides this. Finally, it is worth noting that Option 1 has already been implemented in AISC-341-16 and changing the coefficients back to the 2010 values has its own costs, and the difference is relatively small.

Case for instead using Option 2: The committee was right in 2016 to bring in R_y, but the original source of the data already use expected F_y, and $R_y F_y$ is the best approximation of that value – so no modification past bringing in R_y was needed. Further, coefficients that are shared between AISC 360 and AISC 341 will be more clearly recognizable. Despite causing a small change the majority of the task group supported this option.

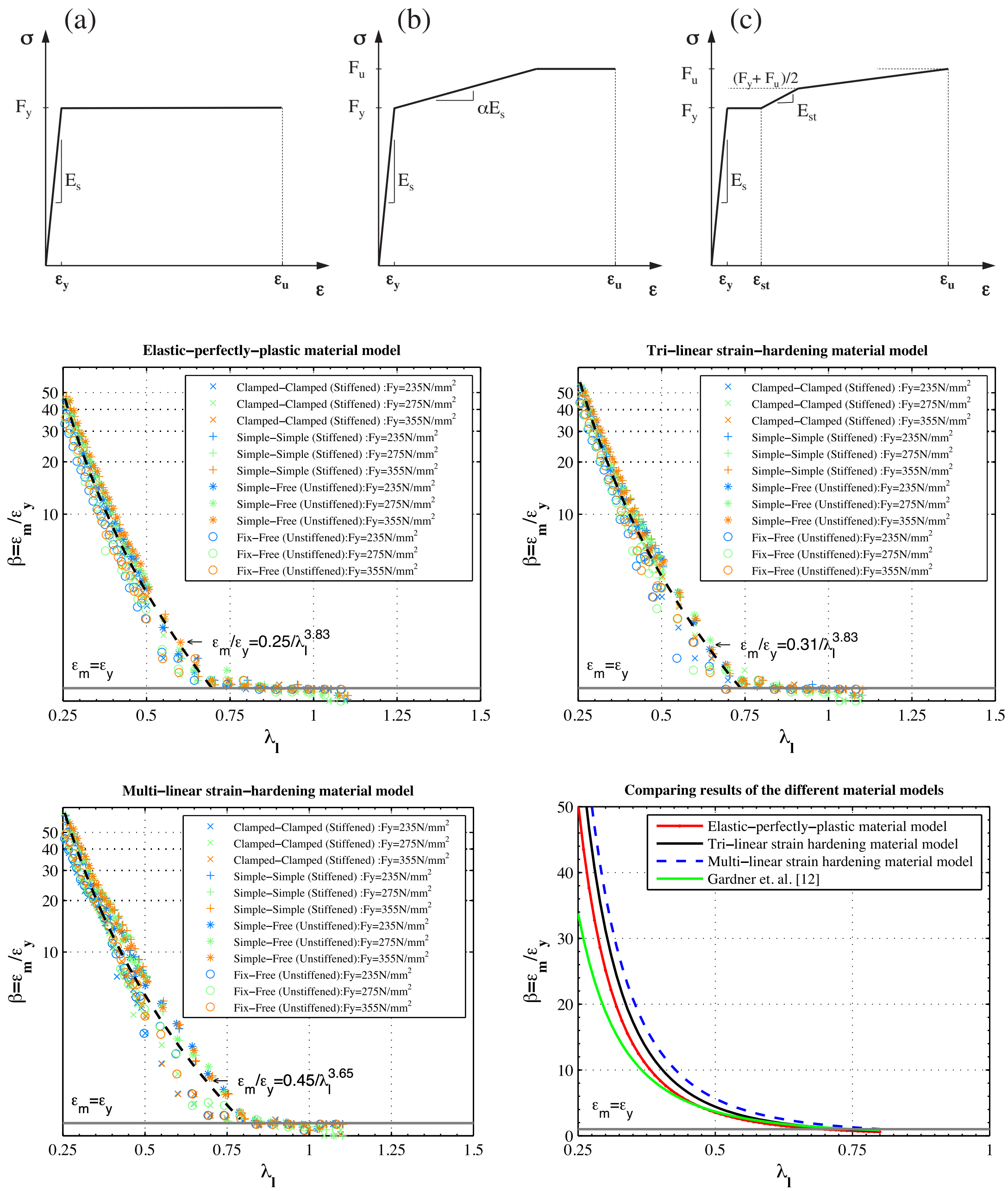
4.2 Other material property-related criteria

If one conceptualizes the *w/t* limits as establishing strain capacity against local plate buckling, as is done in the Continuous Strength Method of Gardner et al. (e.g., Afshan and Gardner 2013, Zhao et al. 2017, see Section 6.2) then the impact of material criteria can at least be partially examined.

Torabian and Schafer (2014) examined inelastic local buckling in plates in terms of their average applied compressive strain at peak load for typical steel models from elastic-plastic to varying degrees of strain hardening as summarized in Figure 3. The analysis demonstrates that strain capacity is a robust measure across all plate boundary conditions for inelastic local buckling and that material parameters have a small but measurable impact on the average strain capacity. Most notably, assuming elastic-perfectly plastic material leads to the most conservative prediction of strain capacity and a $\lambda_r^* \sim 0.7$ (consistent with current use in AISC in compression, AISI, Eurocode and others). If strain hardening is considered $\lambda_r^* \sim 0.8$. Referring to Eq. 10 this implies that typical strain hardening could be assumed to increase w/t limits by 0.8/0.7 or 15% vs. an elastic-plastic assumption. With the advent of modern higher strength steels that have quite different stress-strain relationships from traditional mild steel it is possible to use simulation to develop improved *w/t* limits, this should be considered.

It is worth noting that classical *w/t* limits examined residual stresses and their impact in some depth. From a force standpoint the presence of tensile residual stresses was considered to be an important issue in understanding elastic limits and developed forces in plates. However, these self-equilibrating stresses have a small net cross-section influence and do not directly influence the strain/deformation capacity. As a result, residual stresses are not a central focus for material influence in current examinations of *w/t* limits.

The task group recognized that more detailed examination of the impact of material behavior and variance on local buckling should be conducted in the future.



Material	F_y	F_u	ε_{st}	ε_u	E_s	E_{st}	ν	α
	MPa	MPa	mm/mm	mm/mm	MPa	MPa	–	–
Material-1	235	360	0.014	0.14	203,000	5500	0.3	0.07
Material-2	275	430	0.015	0.12	203,000	4800	0.3	0.07
Material-3	355	510	0.017	0.11	203,000	4250	0.3	0.07

Material model	*Strain capacity*, $\beta = \varepsilon_m/\varepsilon_y$	*Limit*
Elastic-perfectly-plastic	$0.25/\lambda_l^{3.83}$	$\lambda_\ell \geq 0.70$
Trilinear strain-hardening	$0.31/\lambda_l^{3.83}$	$\lambda_\ell \geq 0.74$
Multi-linear strain-hardening	$0.45/\lambda_l^{3.65}$	$\lambda_\ell \geq 0.80$

Figure 3 Average compression strain at failure for inelastic local plate buckling in imperfect plates with different steel stress-strain relationships, excerpt from Torabian and Schafer (2014).

5 Task 4: Web-flange interaction and local buckling (*w/t*) limits

Web-flange interaction is shorthand for the phenomena that the isolated plate solutions that are typically used to predict local buckling are not actually isolated, but instead interact. Equilibrium and compatibility are, of course, maintained between elements in a cross-section even when undergoing elastic or inelastic local buckling. In this regard, the separation into flange local buckling (FLB) and web local buckling (WLB) is artificial – the web and flange of cross-sections always interact.

The primary question is: to what extent does this interaction matter? The traditional conclusion, for rolled shapes at yield stresses consistent with mild steel, is that the interaction is either weak; or otherwise does not vary much and can be approximated for standard (rolled) shapes by treating FLB and WLB as essentially constant and separate plate phenomena. This assumption is largely embedded in current *w/t* limits.

5.1 Explicit web-flange interaction in AISC 360 *w/t* limit for built-up shapes

For non-seismic *w/t* limits, the one case where web-flange interaction is explicitly considered is in the *w/t* limits for flanges of built-up I-shapes. For this case *k* is assumed as:

$$k = 0.35 < 4/\sqrt{h/t_w} < 0.76 \tag{15}$$

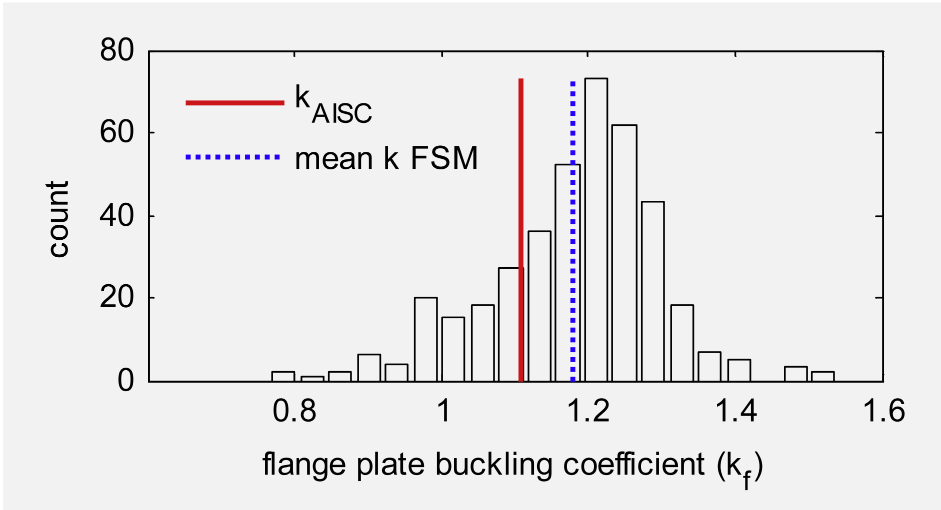
Eq. (15) is a simplification of the expression provided by Johnson (1976), where k was approximated from testing employing the basic mechanics outlined in Haaijer and Thurlimann (1960). Notably, this k is not a plate buckling coefficient in the traditional sense, and does not agree particularly well with elastic theory. A comparison was made employing the expressions in Seif and Schafer (2010), and Eq. (15) is typically higher than the elastic solution. However, White (2008) found that the expression, albeit a simplification, works generally well with available data from a strength perspective.

5.2 Implicit web-flange interaction in AISC 360 *w/t* limits

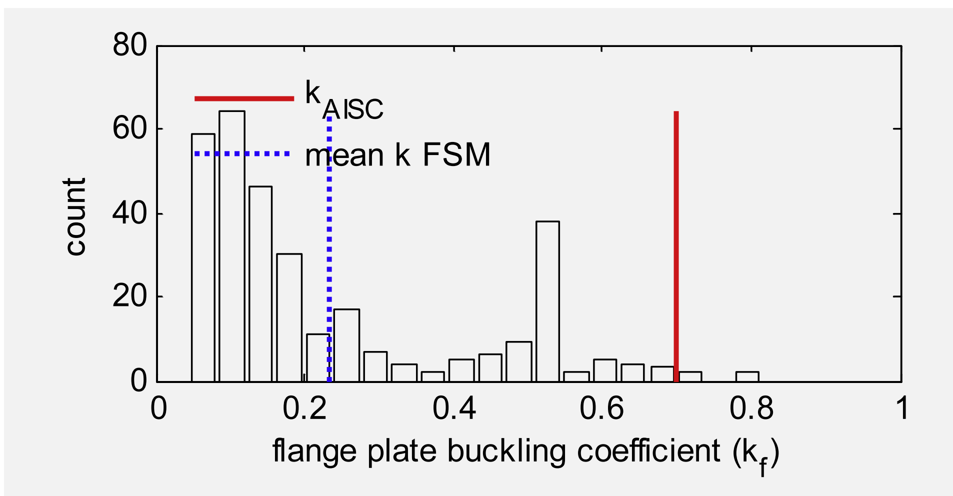
Web-flange interaction is considered for other elements in the AISC *w/t* limits, but at assumed levels of rigidity. For example, the *k* for an I-section flange in a compression member is assumed to be 0.7 which is half-way between the rigidity limits of a simply supported and a fixed longitudinal edge. This sounds rational, but when compared to the actual k for W-sections (Figure 4b) in compression this k is quite optimistic. The web of common W-sections in compression actually degrades the flange plate buckling coefficient (*k*). This is not uncommon as it is not just the rigidity, but the stress on the attached elements that influences the local cross-section stability. Thus, the *w/t* (λ_r) limit in compression is more liberal than it may seem based on *k*=0.7.

Considering all rolled W-sections in flexure, the web stability is now significantly enhanced (from the compression case) and the mean flange *k* for W-sections is as high as 1.2. Back-calculating the k from the flexural λ_r limit gives a k of 1.1, see Table 10 Case 13, and this compares favorably with the actual k as shown in Figure 4a. The notes of Table 9 and Table 10 specifically address the k value and their implicit assumptions about web-flange interaction for all *w/t* cases, comparisons are provided for nearly all cases in Seif and Schafer (2010).

It is recommended in Seif and Schafer (2010) that mean k values (determined from elastic buckling of all relevant rolled shapes) or values based on a given exceedance probability be selected so as to provide uniformity across elements even if a single k is selected. This is a reasonable suggestion, but would lead to changes in almost all λ_r limits in AISC 360.



(a) k for flange (flange in compression, web in flexure)



(b) k for flange (flange in compression, web in compression)

Figure 4 Excerpt from Seif and Schafer (2010) Example of flange buckling k for all W-sections in AISC Manual (a) flexure and (b) compression compared with k assumed in *w/t* development

5.3 Role of stress in web-flange interaction in AISC 360 *w/t* limits

As discussed in the previous section, plate buckling is a function of applied stress on the element and on its neighboring elements that form the section. This dependency of the plate buckling coefficient on the applied stress leads to another important consideration in web-flange interaction: how to handle *w/t* limits for beam-columns. Earlier additions of the AISC 360 *w/t* (λ_p) limits, e.g. the first edition of LRFD (1986) included *w/t* limits for "webs in combined flexure and axial compression" that were a function of the stress gradient captured through the ratio of the applied axial load over the squash load, i.e., P_r/P_y. These provisions were later simplified using the compression *w/t* limit throughout; however, AISC 341 has maintained a dependence on the compression load that has seen recent study as discussed in Section 5.5.

Application of stress dependent *w/t* limits presents certain challenges. AISC 360 beam-column strength design employs a calculation for the axial capacity and a separate calculation for the flexural capacity, which are then combined in an interaction equation. Thus, the expectation is that separate *w/t* limits are needed for compression and bending, and that the interaction equation reasonably combines these two *w/t* limits. If a *w/t* limit is provided that is a function of the assumed stress distribution (or load) for axial and bending combined, one wonders where the Specification expects the engineer to apply this *w/t* limit? Assuming it is applied on the flexural case, then this potentially reduces the flexural anchor point – but this is what the interaction equation itself is supposed to capture. The ad hoc task group did not form a definitive opinon on this issue, but it remains even for the proposal on deep columns in Section 5.5.

Note, that in the context of cold-formed steel, design methods have been developed to consider local stability under combined actions, but strength is also considered under the same combined actions and the interaction equation approach is abandoned, see Torabian and Schafer (2018).

5.4 Closed-form solutions for local buckling

Analytical expressions, derived from simulations, are available to provide closed-formed solutions for accurate plate buckling coefficients, k, or more directly the cross-section local buckling load, $P_{cr\ell}$, or moment, $M_{cr\ell}$: Seif and Schafer (2010) provide one set, and Fieber et al. (2019) have recently derived another set. In addition, lightweight computational methods exist for calculating cross-section local buckling and all buckling values for common shapes could be tabled in much the same way as complex section properties such as C_w.

5.5 Impact on Seismic *w/t* limits (Deep Columns)

Deep columns have seen increasing use in SFRS, particularly in moment frames due to their relative efficiency. Recent testing by Uang et al. at UCSD of λ_{hd}-compliant deep columns showed that they experience significant local buckling and axial shortening, which can negatively impact performance. Based on this work Uang has made the following proposal:

AISC 341-16 Section D1.1b (Table D1.1): For the case "Where used in beams, columns, or links, as webs in flexure, or combined axial and flexure: 1) Webs of rolled or built-up I-shaped sections or channels, 2) Side plates of boxed I-shaped sections, 3) Webs of built-up box sections," change λ_{md} and λ_{hd} as shown in Table 13:

Table 13 Uang Proposal for change to web *w/t*

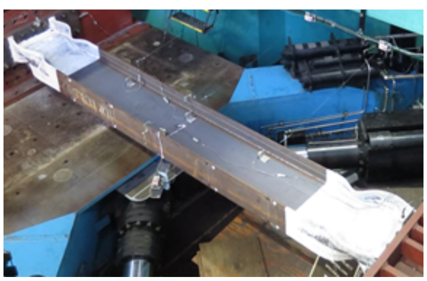
	λ_{hd} for Highly Ductile Members	λ_{md} for Moderately Ductile Members
AISC 341-16	For $C_a \le 0.114$: $2.57\sqrt{E/(R_yF_y)}(1-1.04C_a)$ For $C_a > 0.114$: $0.88\sqrt{E/(R_yF_y)}(2.68-C_a) \ge 1.57\sqrt{E/(R_yF_y)}$	For $C_a \le 0.114$: $3.96\sqrt{E/(R_yF_y)}(1-3.04C_a)$ For $C_a > 0.114$: $1.29\sqrt{E/(R_yF_y)}(2.12-C_a) \ge 1.57\sqrt{E/(R_yF_y)}$

Proposed for AISC 341-22	$2.5(1 - C_a)^{2.3}\sqrt{\frac{E}{R_y F_y}}$	$5.4(1 - C_a)^{2.3}\sqrt{\frac{E}{R_y F_y}}$

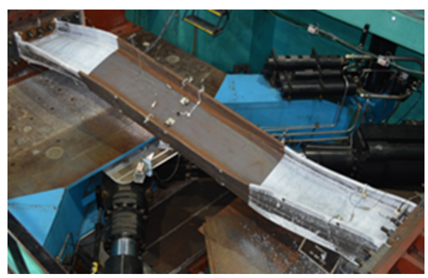
$$C_a = \frac{\alpha_s P_r}{R_y F_y A_g}$$

Steel wide-flange columns in an SMF are expected to experience flexural yielding and form plastic hinge at the column base. A total of 48 deep columns were cyclically tested in a NIST-sponsored research project at UCSD (Ozkula and Uang 2015, Chansuk et al. 2018). Because deep columns have h/t_w ratios that quite often are significantly higher than those of shallow (e.g., W14 or W12) and stocky sections, testing showed that the web was not that effective to stabilize flanges under cyclic loading. The interactive flange-web local buckling occurred earlier and caused a significant strength degradation and axial shortening. Under cyclic loading, lateral-torsional buckling together with local buckling could also occur. See Figure 5 for two typical buckling modes. Independent research conducted by both Lignos and El-Tawil/McCormick also confirmed the observed deep column phenomenon.

The proposed λ_{hd} and λ_{md} limits are based on a regression analysis of deep column responses from both testing and finite element simulation that consider the effects of boundary condition and lateral loading sequence. These limiting values h/t_w ratios are developed for constant axial loads. For exterior columns with varying axial loads due to the overturning moment effect, the proposed limits are conservative.



(a) W24×131 Column



(b) W24×176 Column

Figure 5 Typical Deep Column Buckling Mode

6 Task 5: Alternatives to local buckling (*w/t*) limits

The task group was charged with commenting on alternative means of establishing basic local buckling performance objectives and ensuring the specification provides user pathways to alternative means when current *w/t* limits may be an impediment.

6.1 Cross-section local slenderness limits and application of DSM

The Direct Strength Method (DSM) implemented in AISI S100 provides limits that are similar in spirit to AISC's *w/t* limits, but for the entire cross-section, where $P_{cr\ell}$ is the elastic axial local buckling force and $M_{cr\ell}$ is the elastic flexural local buckling moment.

$$P_n = P_y \ \ if \ \ \lambda_\ell^* = \sqrt{\frac{P_y}{P_{cr\ell}}} = 0.776 \tag{16}$$

$$M_n = M_y \ \ if \ \lambda_\ell^* = \sqrt{\frac{M_y}{M_{cr\ell}}} = 0.776 \tag{17}$$

$$M_n \cong M_p \ \ if \ \lambda_\ell^* = \sqrt{\frac{M_y}{M_{cr\ell}}} = 0.086 \tag{18}$$

Eq. (16) and (17) provide the equivalent to the λ_r limit and Eq. (18) the λ_p limit. Eq. (18) was intentionally conservative in its application for AISI S100 and would need modification for AISC 360 application.

6.2 Cross-section local slenderness limits and application of CSM

The Continuous Strength Method (CSM) developed by Gardner et al. (e.g., Afshan and Gardner 2013, Zhao et al. 2017) provides a complete strain-based alternative to local buckling classification limits – but could equally be used to provide basic limits. The CSM base curve implies the maximum strain capacity is a function of the local buckling slenderness, focusing on the range where $\epsilon \geq \epsilon_y$:

$$\epsilon = \frac{0.25}{\lambda_\ell^{*\,3.6}} \ , \ \lambda_\ell^* = \sqrt{\frac{P_y}{P_{cr\ell}}} \ or \ \sqrt{\frac{M_y}{M_{cr\ell}}} \tag{19}$$

If we set $\epsilon = \epsilon_y$ for the equivalent to the λ_r limit, and set $\epsilon = 4\epsilon_y$ for the λ_p limit:

$$P_n = P_y \ \ if \ \ \lambda_\ell^* = \sqrt{\frac{P_y}{P_{cr\ell}}} = 0.68 \tag{20}$$

$$M_n = M_y \ \ if \ \lambda_\ell^* = \sqrt{\frac{M_y}{M_{cr\ell}}} = 0.68 \tag{21}$$

$$M_n \cong M_p \ \ if \ \lambda_\ell^* = \sqrt{\frac{M_y}{M_{cr\ell}}} = 0.46 \tag{22}$$

Note $\epsilon = 15\epsilon_y$ for the λ_{hd} limit would result in $\lambda_\ell^* = 0.32$. Given the approximate nature of current element slenderness limits, it should be permitted to use more robust cross-section based slenderness limits when desired by the engineer. Note, that for some sections under some loading these limits will be more stringent than current practice, for others more lenient.

6.3 Extensions on the use of CSM and DSM

If the plate strain capacity can accurately be predicted then that strain may be converted to stress and integrated to provide axial load or moment. If this is done then one is following the basic tenets of CSM. The local slenderness must be for the cross-section, not the element for the method to provide sufficient accuracy. Both DSM and CSM also provide complementary strength solutions for higher slenderness. Also, CSM provides the ability to incorporate strain hardening in strength predictions. These approaches are being adopted for stainless steel in current drafts of ASCE 8-20 and AISC 370-22.

An additional note on Eq. (19) – the power of this expression should not be understated. Recall Eq. (6) where the elastic plate buckling strain was made independent of Young's modulus, so too is Eq. (19) and in fact has been developed considering stainless steel, aluminum, and traditional mild carbon steels. Further the limits in Eq. (20)-(22) agree quite well with Winter's insights and Haiijer and Thurlimann's insights on key slenderness ranges for first yield and plastic behavior. This generalization is attractive, and a means to leverage this insight is worthy of consideration for the AISC Specification.

6.4 AISC 360 Appendix 1 supported analysis provisions

As introduced in Section 4.2 Torabian and Schafer (2014) used a CSM-inspired approach to establish rotation capacity in addition to strength. Thus, it is possible to provide a methodology for predicting allowable rotation capacity (R_{cap} or Θ_p) for use in material nonlinear analyses, both static for AISC 360 and potentially dynamic for application to AISC 341. This could potentially be advanced in AISC Appendix 1. Recent work of Gardner et al. (2019) has extended these insights directly into line elements for use in system analysis. Also, the compactness limits for inelastic analysis (λ_{pd}) could potentially leverage the more robust methods discussed herein.

6.5 Testing pathways

For new steel materials, new cross-sections, or novel built-up shapes, AISC does not have a clear process for establishing *w/t* limits. However, if the objectives for the λ limits are clearly stated as in this report, then the test objectives become equally clear and a test-based path becomes reasonably clear. It should be noted that current provisions are not typically based on extensive testing, and a small number of controlled tests with complementary analysis/simulation may well be sufficient.

7 Recommendations

7.1 Non-seismic *w/t* limits – AISC 360

- Rewrite the Table B4.1 commentary: provide objectives per Section 3.3 (aligned with Specification not aspirational), make the role of non-dimensional slenderness λ^* clear and provide finalized versions of Table 9 and Table 10 in the commentary or through reference to an archival publication.
- Provide a "shall be permitted" pathway for the use of cross-section elastic buckling analysis that includes web-flange interaction as an alternative to current *w/t* limits. Set $\lambda_r^* = 0.7$ or 1.0 as appropriate and $\lambda_p^* = 0.5$ for these alternative provisions. Additional research may have to be conducted on this topic for acceptance by TCs and COS.
- Establish a small research project to improve λ_r for flexure.
    - In lieu of such a project, it is recommended at a minimum that current λ_r for flexure values be re-cast to make it explicitly clear why $\lambda_r^* = 1.0$ not $\lambda_r^* = 0.7$. This would explain the discrepancy in Table B4.1b between (a) stiffened elements in compression, and (b) unstiffened elements in compression and stiffened elements in flexure; and explain the discrepancy between compression elements in Table B4.1a and b. This would also explain a significant discrepancy between current AISC practice and other international standards.
    - If project funding is not available and only committee efforts are possible it is recommended that $\lambda_r^* = 0.7$ be used throughout and Chapter F modified to accommodate this change. This would remove the discrepancy in Table B4.1b between (a) stiffened elements in compression, and (b) unstiffened elements in compression and stiffened elements in flexure; and remove the discrepancy between compression elements in Table B4.1a and b. This would also remove a significant discrepancy between current AISC practice and other international standards.
- Align λ_p Case 12 (angle) with that of Case 10 (rolled flange in compression) or make it explicit that Case 12 only applies to the angle leg under stress gradient.
- Align λ_p Case 15 (I-section web) with that of Case 19 (box-section web) or provide evidence that I-section webs can have more liberal *w/t* limits than box-section webs (even beyond that of assuming a fully fixed edge boundary condition for the I-section web).
- Remove the use of residual stress (F_L vs. F_y) in the Table B4.1 limits. If justified correct limits after removal to insure new limits are not unduly conservative.
- Fund a small research project to create λ_{p1} and λ_{p2} consistent with Class 1 and Class 2 that provide (1) M_p with minimum rotation (2) M_p, respectively. This will provide improved efficiency in some cases and will provide needed rotation capacity only where necessary (for example in inelastic analysis with moment redistribution of Appendix 1 of AISC 360).
    - Note, it is recommended that for simplicity implementation in Chapter F need only use λ_{p2} since this establishes M_p, while Appendix 1 could reference the use of λ_{p1} for plastic design and/or material nonlinear analyses with redistribution.

7.2 Seismic *w/t* limits – AISC 341

- Rewrite the Table D1.1 commentary: provide objectives per Section 3.4 (aligned with Specification not aspirational) and finalized version of Table 2 in the commentary or reference to archival publication. Note the commentary should describe intent and not imply specific values that are met by the *w/t* (lambda) limits.
- Correct the λ_{md} and λ_{hd} limits back to their 2010 coefficients (and include R_y). Note, TC9 formally supported this recommendation at the November 2019 meetings; however, there is still concern about final application (e.g. for SCBF braces and H-piles) that will need to be addressed in final implementation.
- Provide a "shall be permitted" pathway for the use of cross-section elastic buckling analysis that includes web-flange interaction as an alternative to current *w/t* limits. Set $\lambda_{md}^{*} = 0.5$ and $\lambda_{hd}^{*} = 0.32$ for these alternative provisions.
- Ballot the proposed provisions for deep columns of Section 5.5.

7.3 Additional recommendations

- Establish a research project to take advantage of the findings from the Continuous Strength Method research and bring these advantages into the AISC 360 and AISC 341 standard. Active work in the development of AISC 370, Stainless Steel Specification, may be utilized in this regard.
- Establish a research project to determine cyclic degradation in the strain capacity of plate elements subjected to local buckling such that AISC 341 *w/t* criteria can be improved, and where possible aligned with reality. Recent advances in cyclic fracture models of ductile steels can be leveraged as a mechanical basis for this effort and the results have the potential to widely influence λ_{md} and λ_{hd} and their future application.
- Develop a test standard for establishing *w/t* limits (for AISC 360 and AISC 341) consistent with past practice and current application. This recommendation provides a pathway for alternative built-up shapes and new materials (steels) that may be impeded by current design rules.
- Extend Appendix 1 of AISC 360: provide alternative means for meeting λ_{pd} criteria based on cross-section slenderness, provide discussion/guidance on member rotational demands coming from nonlinear analysis and how to calculate member rotational capacity based on local cross-section slenderness.

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Note, Structural standards: AISC 360/341, AISI S100, ECCS etc. are not provided in this list

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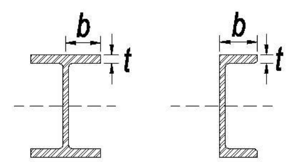
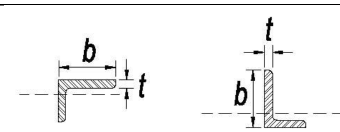
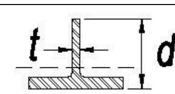
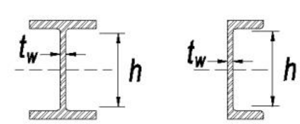
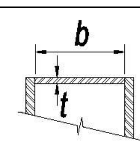
Appendix 1: Background to AISC 360 λ_p limits and R_{cap}=3

Discussion of width-to-thickness limitations
Louis Geschwindner
February 12, 2018

AISC Specification Width-to-Thickness Limits

Work item 1a for the AISC TG on Width-to-Thickness Limits is to state specific objectives for these limits. To begin the process, a look at the history of some of these limits may be useful. Table 1 gives the compact member limits, identified as λ_p in the AISC Specifications since 1986, for 5 of the cases identified in AISC 360-16. The values for 1963 and 1969 were converted for comparison purposes. The Spec. Range in the table shows that for rolled I-shaped sections, cases 10 and 15, and flanges of box section, case 21, the requirements have been the same since 1978. Case 12 was added in AISC 360-05 and case 14 was added in AISC 360-10.

Table 1: Compact Width-to-Thickness Limit for Flexure

	Case	Description		1963 [a]	1969 [a]	λ_p [a]	Spec. Range	
Unstiffened Elements	10	Flanges of rolled I-shaped sections, channels, and tees	b/t	$0.30\sqrt{\frac{E}{F_y}}$	$0.31\sqrt{\frac{E}{F_y}}$	$0.38\sqrt{\frac{E}{F_y}}$ [b]	1978-2016	
	12	Legs of single angles	b/t			$0.54\sqrt{\frac{E}{F_y}}$	2005-2016	
	14	Stems of tees	d/t			$0.84\sqrt{\frac{E}{F_y}}$	2010-2016	
Stiffened Elements	15	Webs of doubly symmetric I-shaped sections and channels	h/t_w [c]	$2.47\sqrt{\frac{E}{F_y}}$	$2.42\sqrt{\frac{E}{F_y}}$	$3.76\sqrt{\frac{E}{F_y}}$ [b]	1978-2016	
	21	Flanges of box sections	b/t	$1.11\sqrt{\frac{E}{F_y}}$	$1.12\sqrt{\frac{E}{F_y}}$	$1.12\sqrt{\frac{E}{F_y}}$	1978-2016	

[a] 1963 converted from $1/\sqrt{F_y}$ in psi, 1969 – 1993 converted from $1/\sqrt{F_y}$ in ksi, 1999 – 2016 in terms of $\sqrt{E/F_y}$.
[b] These changes were actually implemented with Supplement 3 to the 1969 Specification, June, 1974.
[c] These limits were given in terms of d/t through the 1999 ASD Specification and in terms of h/t_w from 1986 LRFD to 2016.

Basis of Width-to-Thickness Limits

The Commentary for the 1986 LRFD Specification references Galambos (1976) and Yura et al. (1978) when discussing the limits in cases 10 and 15. Thus, it should be helpful to review what those two papers present.

Yura et al, (1978) indicates that

> "Current rules in plastic design are not based on any consistent rotation capacity requirements. Some recommendations (ASCE, 1971) are based on rotation capacities up to about 10, while others are based on flange strain reaching levels of four times the yield strain, which corresponds to $R = 3$. In 1974, the AISC Specification adopted changes in the allowable stress provisions for compact beams, i.e., beams in the plastic zone where moment redistribution is permitted. These rules for controlling instability were based on the ability of the cross section to reach rotation capacities of three or greater (or stress four times the elastic limit strain)."

This quote appears to be the first place in the literature where a statement is made that the rules were "based on" the ability of the cross section to reach a rotation capacity of three or greater. However, reference to (ASCE, 1971) does not reveal any confirmation of that basis. ASCE, 1971 does refer to Lukey and Adams (1969) for their discussion of rotation capacity. This work will be addressed in the following.

Yura et al. (1978) also state that

> "For local flange buckling, Lukey and Adams (1969) developed an experimental relationship between $b_f/2t_f$ and R. For $R = 3.0$, this relationship is
>
> $$\frac{b_f}{2t_f}\sqrt{\frac{F_y E}{44E_{st}}} \le 78 \qquad (1)^{1}$$
>
> The mean value of E_{st} is 600 ksi with a standard deviation of 150 ksi (Galambos and Ravindra, 1978). Using a value for E_{st} of one standard deviation below the mean (450 ksi) because of the large variation and with $E = 29{,}000$ ksi gives
>
> $$\frac{b_f}{2t_f} \le \frac{65}{\sqrt{F_y}} \qquad (2)$$
>
> which is the current provision in Part I of the AISC Specification."

Figure 1 presents the data from Lukey and Adams (their Figure 13) as a plot of the rotation capacity, R, vs. a nondimensionalized flange slenderness given as

[1] Equation numbers are for this paper only. They are not part of the quoted papers.

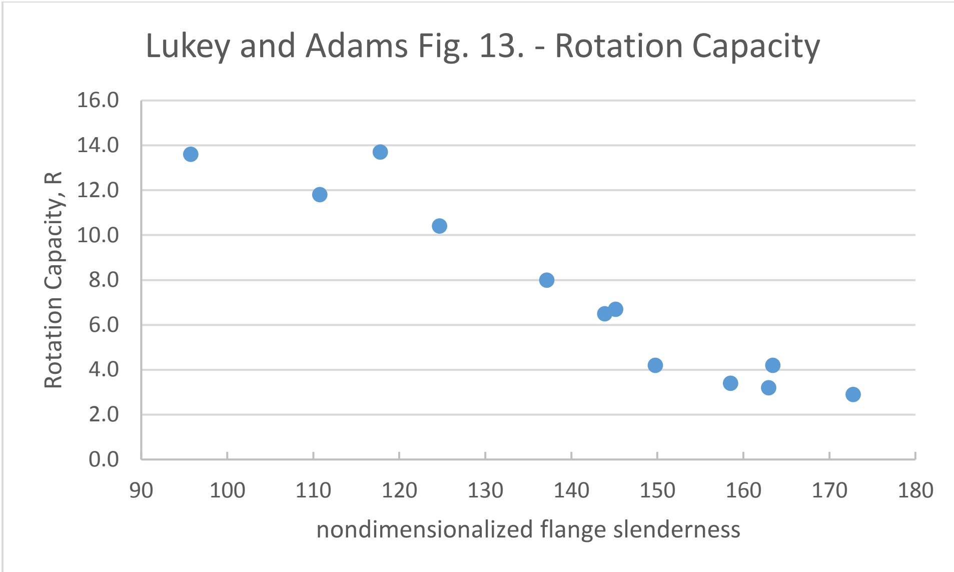


Figure 1: Rotation Capacity vs. Flange Slenderness

$$\frac{b}{t}\sqrt{\frac{F_y E}{44E_{st}}} \tag{3}$$

Close examination of the Lukey and Adams (1969) paper reveals a somewhat different interpretation of rotation capacity than that given by Yura et al. Their conclusion states

> "For a structure designed by the allowable stress technique, a compact section is required only to reach M_p. All sections tested did reach M_p, however, to provide a nominal amount of inelastic rotation capacity (e.g. $R = 2.5$) the factor
>
> $$\frac{b}{t}\sqrt{\frac{F_y E}{44E_{st}}} \tag{4}$$
>
> would be limited to 160."

For their work, they defined the width-to-thickness ratio as the flange width divided by the flange thickness. Thus, to use the normal AISC definition, $b_f/2t_f$, the limit would be set to 80 and the resulting equation would become

$$\frac{b_f}{2t_f}\sqrt{\frac{F_y E}{44E_{st}}} \le 80 \tag{5}$$

Using a value for E_{st} = 442 ksi and E = 29,600 ksi, as the original authors did, yields

$$\frac{b_f}{2t_f} \le \frac{64.8}{\sqrt{F_y}} \tag{6}$$

Thus, either way, the same limit, the current width-to-thickness ratio, is obtained. But what is different if we look directly at Lukey and Adams (1969) without the filter of Yura et al. (1978) is that they say the current limit is for a rotation capacity of R = 2.5, not 3.0.

Definition of Rotation Capacity

Figure 2 illustrates the moment-rotation relationship for the beams in the Lukey and Adams (1969) tests.

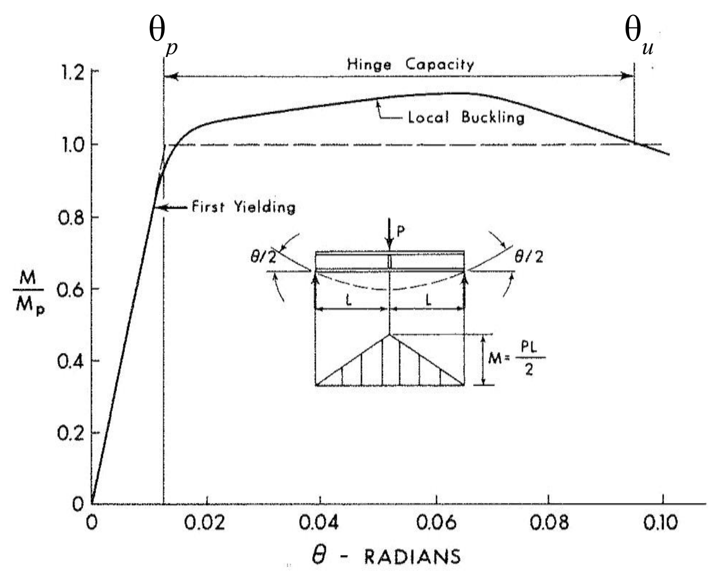


Figure 2: Moment-Rotation Relationship

Rotation capacity is defined as

$$R = \frac{\theta_u}{\theta_p} - 1 \tag{7}$$

where θ_u is the rotation at which the moment drops below M_p on the unloading branch and θ_p is the rotation that corresponds to just reaching M_p on the loading branch of the moment-rotation curve. It is important to note that the actual magnitude of the rotation is not needed. Some researchers have defined the rotation as shown here where the sum of the rotation at each end of the beam is θ while others have defined the rotation at each end of the beam as θ. The definition of θ is important when comparing rotations from different test programs but not when considering rotation capacity.

Initiation of Local Buckling

Local buckling will commence at the onset of strain hardening. *Plastic Design in Steel: A Guide and Commentary* (ASCE, 1971) looks to define the flange slenderness at which local buckling will initiate. From the work of several researchers, for beams with uniform moment, they conclude that this occurs when

$$\frac{b}{t} = \frac{1.78}{\sqrt{F_y/E}}\sqrt{\frac{1}{1+\dfrac{E}{5.2E_{st}}}} \tag{8}$$

where E_{st} is the strain hardening modulus of the material. They use E_{st} = 800 ksi and determine that the limit for F_y = 36 ksi is b/t = 17.9. However, for beams with a "moment gradient, strain hardening tends to occur very early, and it has been found that buckling normally occurs at a stress level higher than yield stress." To account for this, they conclude, from Lay and Galambos (1967), that

$$\frac{b}{t} = \frac{3.56}{\sqrt{F_y/E}}\sqrt{\frac{1}{\left(3+\dfrac{F_u}{F_y}\right)\left(1+\dfrac{E}{5.2E_{st}}\right)}} \tag{9}$$

For F_y = 36 ksi, this results in b/t = 16.7. It is interesting to note that the difference between these two equations, Equations 8 and 9, is less than the difference between using E_{st} = 800 ksi (ASCE, 1971) and E_{st} = 600 ksi (Galambos and Ravindra, 1978). ASCE (1971) concludes that a value between these limits should be used and therefor recommend that b/t = 17 be used for F_y = 36 ksi steel. This is the limit found in Part 2 of the ASD Specifications through 1989, $b_f/2t_f$, = 8.5.

Rotation Capacity

To judge the rotation capacity when the limit of Equation 9 is applied, the Lukey and Adams data is given in Figure 3 as it was presented in Figure 6.15 from ASCE (1971). The nondimensionalized flange slenderness is taken as

$$\frac{b}{t}\sqrt{\frac{F_y}{E}\left(3+\frac{F_u}{F_y}\right)} \tag{10}$$

The vertical dashed line represents the limit proposed by Equation 9 for a beam with a moment gradient and using E_{st} = 800 ksi, given as

$$\frac{b}{t}\sqrt{\frac{F_y}{E}\left(3+\frac{F_u}{F_y}\right)} = 1.26 \tag{11}$$

For the members tested by Lukey and Adams, this limit corresponds to a rotation capacity greater than 10. This may be the limiting rotation capacity of 10 referred to by Yura et al. (1978).

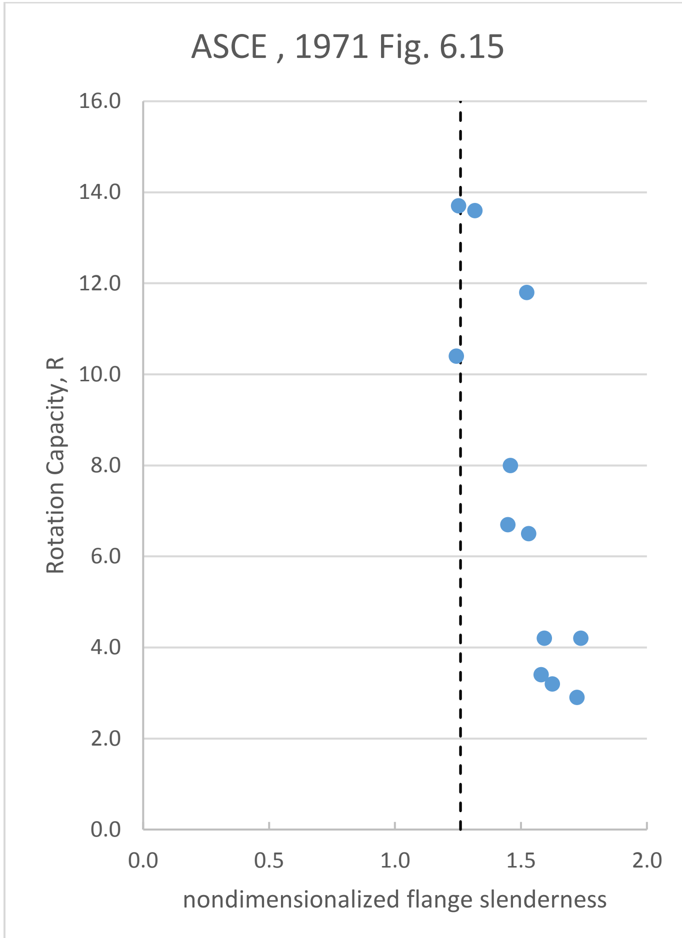


Figure 3: Relationship between Rotation Capacity, *R* and the Nondimensionalized Flange Slenderness

Lukey and Adams point out that all of their test specimens were able to reach M_p. The test specimen exhibiting the least rotation capacity, $R = 2.9$, had

$$\frac{b_f}{2t_f}\sqrt{\frac{F_y E}{44E_{st}}} = 86.5 \tag{12}$$

which results in

$$\frac{b_f}{2t_f} = \frac{70.1}{\sqrt{F_y}} \tag{13}$$

This limit is higher than the current compact flange limit in use since the 1978 Specification. Thus, it does not appear that the goal of the compact flange limit was ever really $R = 3$.

Plate Buckling

The theoretical elastic buckling stress for a plate is the well-known equation

$$F_{cr} = k \frac{\pi^2 E}{12\left(1-\nu^2\right)\left(b/t\right)^2} \tag{14}$$

where k is the plate buckling coefficient that is a function of the type of stress, edge conditions, and length-to-width ratio of the plate. Table 2 gives the minimum k values for four different edge support conditions from Salmon and Johnson (1990).

Table 2 Theoretical Buckling Coefficients

Longitudinal Edge Support	*k*
Simple – Simple	4.00
Fixed - Simple	5.42
Fixed - Fixed	6.97
Fixed - Free	1.277
Simple - Free	0.425

If F_{cr}/F_y is defined as $1/\lambda_c^2$, Equation 14 becomes

$$\lambda_c = \frac{b}{t}\sqrt{\frac{F_y\left(12\right)\left(1-\nu^2\right)}{\pi^2 E k}} \tag{15}$$

and $\lambda_c = 1.0$ when $F_{cr} = F_y$. Figure 4 shows a plot of the ratio F_{cr}/F_y vs. λ_c taken from Salmon and Johnson (1990).

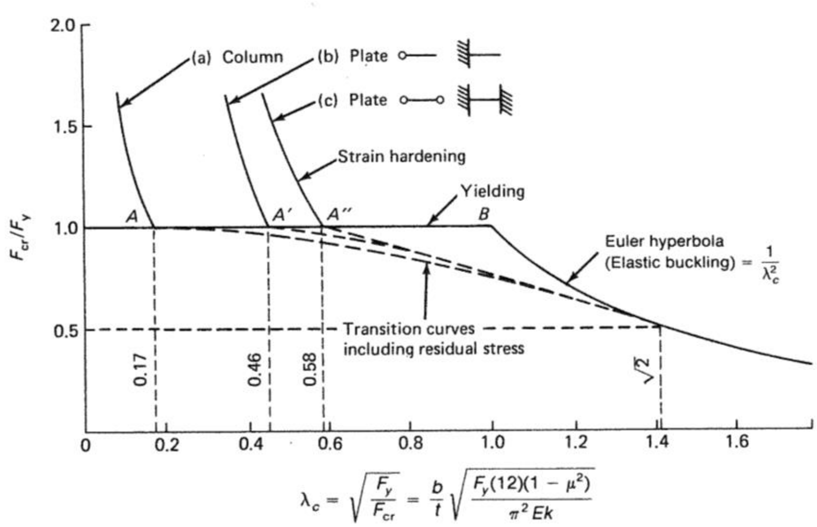


Figure 4: Plate Buckling

Points *A*, *A*', and *A*" represent points where the inelastic buckling curve ends and the strain hardening curve begins; *A* for columns, *A*' for plates supported along one edge, and *A*" for plates supported along two edges, defined as λ_0. Haaijer and Thürlimann (1958) determined λ_0 for the five cases given in Table 3. From that data it is apparent that the important factor in determining λ_0 is whether the plate is supported along one edge or two. The degree of edge restraint, hinged or fixed, essentially has no effect. Thus, for one edge supported, unstiffened elements, λ_0 can be taken as 0.46 and for both edges supported, stiffened elements, λ_0 can be taken as 0.58. These values are shown in Figure 4.

Table 3: Nondimensionalized Slenderness Ratio for Initiation of Strain Hardening

Plate Type	λ_0
Columns	0.173
Hinged Flanges	0.455
Fixed Flanges	0.461
Hinged Webs	0.588
Fixed Webs	0.579

For a beam to attain the plastic moment without buckling, the width-to-thickness ratio must not exceed λ_0. Setting Equation 15 equal to λ_0 and solving for *b*/*t* yields

$$\frac{b}{t} \le \lambda_0 \sqrt{\frac{\pi^2 Ek}{F_y(12)(1-\nu^2)}} = 0.95\lambda_0 \sqrt{\frac{kE}{F_y}} \qquad (16)$$

Thus, for unstiffened elements

$$\frac{b}{t} \le 0.95(0.46)\sqrt{\frac{kE}{F_y}} = 0.437\sqrt{\frac{kE}{F_y}} \qquad (17)$$

and for stiffened elements

$$\frac{b}{t} \le 0.95(0.58)\sqrt{\frac{kE}{F_y}} = 0.551\sqrt{\frac{kE}{F_y}} \qquad (18)$$

Flanges of I-shapes and Box Sections with Bending

For the flange of an I-shape, the web will provide restraint somewhere between fixed, $k = 1.277$ and simple, $k = 0.425$, based on Table 2. Salmon and Johnson say that $k = 0.7$ is selected as being about midway between simply supported and fixed along the web. Thus, Equation 17 becomes

$$\frac{b}{t} \le 0.437\sqrt{\frac{(0.7)E}{F_y}} = 0.37\sqrt{\frac{E}{F_y}} \qquad (19)$$

which is very close to λ_p given in Table 1 for case 10, flanges of rolled I-shapes, $0.38\sqrt{E/F_y}$.

Next consider the flange of a box section, case 21 in Table 1. If it is assumed that the webs provide little rotational support to the flange, the simple – simple support condition with k = 4.0 can be used. Thus, Equation 18 becomes

$$\frac{b}{t} \leq 0.551\sqrt{\frac{(4.0)E}{F_y}} = 1.10\sqrt{\frac{E}{F_y}} \tag{20}$$

which is very close to λ_p given in Table 1 for case 21, flanges of box sections, $1.12\sqrt{E/F_y}$. In fact, when you go back to 1963 and recognize that the limit had been given in terms of the overall width of the flange, not as currently defined by AISC, and all the rounding that has taken place as the limit was changed from psi to ksi and then to a non-dimensional equation, this is even closer to the 1963 limit of$1.11\sqrt{E/F_y}$.

Thus, it appears that the only requirement for a flange to be classified as a compact element is that the flange element reach strain hardening without buckling. There is nothing said about a minimum rotation capacity.

Webs of I-shapes with Bending

Haaijer and Thürlimann (1958) considered stiffened plates under uniform compression, similar to the flange plate of a box section just discussed. They also considered the plate as the web of an I-shape under bending where the plate would be partially in compression and partially in tension. By equating the work of the external forces to the dissipation of energy at the moment of buckling, they determined the plate buckling coefficient, k, as a function of the location of the plastic neutral axis. For pure bending at the plastic moment, k = 10.5. They then proceeded to determine the point at which strain hardening will commence, λ_0, for different values of the ratio of maximum strain in the flange to yield strain, $\varepsilon_m/\varepsilon_y$. This ratio also established the corresponding rotation capacity which is also given in Table 4.

Table 4: Initiation of Strain Hardening, λ_0, for Strain Ratio

$\varepsilon_m/\varepsilon_y$	***R***	λ_0
12	11	0.58
8	7	0.60
4	3	0.69

Thus, using the values from Table 4 along with the new plate buckling coefficient, k = 10.5, Equation 16 results in three possible d/t limits.

$$\frac{d}{t} \le 0.95\lambda_0 \sqrt{\frac{kE}{F_y}} = 0.95(0.58)\sqrt{\frac{10.5E}{F_y}} = 1.79\sqrt{\frac{E}{F_y}} \quad (21a)$$

$$\frac{d}{t} \le 0.95\lambda_0 \sqrt{\frac{kE}{F_y}} = 0.95(0.60)\sqrt{\frac{10.5E}{F_y}} = 1.85\sqrt{\frac{E}{F_y}} \quad (21b)$$

$$\frac{d}{t} \le 0.95\lambda_0 \sqrt{\frac{kE}{F_y}} = 0.95(0.69)\sqrt{\frac{10.5E}{F_y}} = 2.12\sqrt{\frac{E}{F_y}} \quad (21c)$$

Haaijer and Thürlimann recommend using Equation 21c except in cases requiring large rotations. However, Equation 21c does not compare well to λ_p given in Table 1 for case 15 from 1963 to 1969, $2.47\sqrt{E/F_y}$ and $2.42\sqrt{E/F_y}$, and it is significantly less than the values in Specifications after that, which are based on h/t_w. Thus, if the limit given in Equation 21c is based on $R = 3$, and the current limit is much greater than that, $R = 3$ is not the basis of the compact section limit for the web.

Peolynn and Kulak (1974) determined that there was no satisfactory correlation between the Haaijer and Thürlimann theory and their beam-column tests. Thus, they decided to find some other way to predict web buckling. They combined their data with that of Haaijer and Thürlimann (1958), Lukey and Adams (1969), and Holtz and Kulak (1973) to determine the limit required to preclude web plate buckling. They determined that λ_0 at the initiation of strain hardening for the stiffened web plate could again be taken as 0.58 as originally proposed by Haaijer and Thürlimann, even for the case of combined loading. They then concluded that

$$\frac{h}{t_w} \le \frac{520}{\sqrt{F_y}}\sqrt{1 - 0.695\left(P/P_y\right)^{0.3846}} \quad (22)$$

For bending only this gives

$$\frac{h}{t_w} \le \frac{520}{\sqrt{F_y}} = 3.05\sqrt{\frac{E}{F_y}} \quad (23)$$

which is greater than the limit proposed by Haaijer and Thürlimann but less than the limit in the AISC Specifications since 1969.

The goal of Peolynn and Kulak (1974) was to show that the Canadian web slenderness limit, $h/t_w \le 420/\sqrt{F_y}$, was much too conservative. They appear to have shown that. The AISC limit at that time was $d/t \leq 412/\sqrt{F_y}$, close to the Canadian limit which was based on h/t_w and less than the proposal of Peolynn and Kulak. Their proposed limit was between the limit given in the 1969 AISC Specification and that given in the 1974 Supplement 3 to the 1969 AISC Specification. Galambos (1976) suggests that the Peolynn and Kulak recommendation was liberalized before adoption into the AISC Specification based on the tests conducted by Croce (1970) and Costley (1970).

Costley (1970) conducted tests on 11 specimens, two on members with noncompact flanges and four on members with flanges right at the compact limit. He did not address slenderness of the web. Thus, other than the general conclusion that all the tests exceeded the predicted plastic capacity, this work cannot be used to justify the liberalization suggested by Galambos (1967).

Croce (1970) specifically addressed the case of slender web girders. He conducted eight tests, one on a girder with a web slenderness less than the compact limit and seven on girders with web slenderness greater than the compact limit by from 9 to 89%. In all cases the members reached the plastic moment capacity and formed a mechanism. Based on his tests, he recommended a compact web limit d/t = 125 for A36 steel. This converts to

$$\frac{d}{t} = \frac{750}{\sqrt{F_y}} = 4.4\sqrt{\frac{E}{F_y}} \qquad (24)$$

This limit is significantly greater than the h/t_w limit recommended by Peolynn and Kulak (1974). Based on Galambos (1967) it is the bases for a liberalization implemented in Supplement No. 3 to the 1969 Specification. Thus, the limit

$$\frac{d}{t} \leq \frac{640}{\sqrt{F_y}} = 3.76\sqrt{\frac{E}{F_y}} \qquad (25)$$

between Equations 23 and 24 can be attributed to a compromise decision of the committee.

Thus, even with the difference between d/t and h/t_w, it is clear from a comparison between the current limit and that of Equation 21c that the minimum rotation capacity for a compact web element is not and never was R = 3.

Noncompact/Slender Width-to-Thickness Limit, λ_r

The limiting width-to-thickness ratio defining the boundary between a noncomapct and slender element is the point at which the elastic buckling stress equation, Equation 14, no longer correctly predicts the strength of the element, it no longer behaves elastically. This limit is a function of the residual stress and can be found by setting the elastic buckling stress equal to the yield stress minus the residual stress, currently presented in the Specification as $F_L = \left(F_y - F_r\right)$, where F_r is the residual stress. This nomenclature will be used here.

Haaijer and Thürlimann (1958) said that the residual stress could be conservatively taken as $0.5F_y$. Their studies were based on A7 steel, F_y = 33 ksi, but they extended it to other strength steel by multiplying by the ratio of $\sqrt{33/F_y}$. Thus, they felt this would be conservative for other steels as well. For steel, with ν = 0.3, and setting Equation 14 equal to F_L, the limiting width-to-thickness ratio becomes

$$\frac{b}{t} \le 0.95\sqrt{\frac{kE}{F_L}} \tag{26}$$

With $F_L = 0.5F_y$, the limit is

$$\frac{b}{t} \le 1.34\sqrt{\frac{kE}{F_y}} \tag{27}$$

For the flange of an I-shape, with k = 0.7 as used earlier,

$$\frac{b}{t} \le 1.34\sqrt{\frac{kE}{F_y}} = 1.34\sqrt{\frac{0.7E}{F_y}} = 1.12\sqrt{\frac{E}{F_y}} \tag{28}$$

This is higher than the limit of $\lambda_r = 1.0\sqrt{E/F_y}$ used in the Specification since 2005.

Since 2005, the Specifications have used a residual stress of $0.3F_y$ which results in $F_L = 0.7F_y$. Thus, from Equation 26, with k = 0.7 and $F_L = 0.7F_y$ the limit for a flange would be

$$\frac{b}{t} \le 0.95\sqrt{\frac{0.7E}{0.7F_y}} = 0.95\sqrt{\frac{E}{F_y}} \tag{29}$$

This is fairly close to the limit, $\lambda_r = 1.0\sqrt{E/F_y}$ used in the Specification since 2005 and it could be concluded that this is how that limit was established.

For the web of an I-shape, with k = 10.5 as used earlier and $F_L = 0.7F_y$, Equation 26 becomes

$$\frac{b}{t} \le 0.95\sqrt{\frac{kE}{F_L}} = 0.95\sqrt{\frac{10.5E}{0.7F_y}} = 3.68\sqrt{\frac{E}{F_y}} \tag{30}$$

which is quite far from the web limit of $\lambda_r = 5.7\sqrt{E/F_y}$ in use since 2005, recognizing again the difference between d/t and h/t_w. This might reasonably be expected since the approach of Haaijer and Thürlimann (1958) did not prove useful for the compact limit of webs either.

Unfortunately the work of Croce (1970) and Costley (1970) did not investigate the behavior of beams with webs in this region. There do not appear to be any references to point to how the λ_r limit was established. Since no rolled I-shapes come even close to the λ_r limit in Case 15, it may not be particularly important to identify exactly where that limit came from. One more factor that must be remembered is that, as λ_r increases, the strength predicted becomes more conservative, unlike for λ_p, where, as the limit increases the predicted strength increases.

Web-Flange Interaction

A second work item for the TG, item 4b, was to look at the web-flange interaction provisions. The flange local buckling criteria at the noncompact-slender boundary were modified to include web-flange interaction for the 1993 LRFD Specification based on the work of Johnson (1985). As discussed earlier, based on Table 2, the plate buckling coefficient for the flange would be between $k = 0.425$ for simple-free arrangement and $k = 1.277$ for fixed-free arrangement. AISC uses an intermediate value of $k = 0.7$. Since the pinned edge gives $k = 0.425$ it would appear that this is the minimum value. However, for cases with very slender webs, the web may actually buckle and cause the flange to twist. Thus providing even less support and resulting in an effective buckling coefficient less than 0.425. Bleich (1952) reports on work published in 1939 that illustrated this condition for column sections and showed that k could be less than 0.425. Johnson (1985) conducted two sets of tests to investigate this condition for beams. He concluded that the flange plate buckling coefficient, as a function of the web slenderness, should be taken as

$$k = \frac{4.05}{\left(h/t_w\right)^{0.46}} \tag{31}$$

AISC conservatively simplifies this to

$$k_c = \frac{4}{\left(h/t_w\right)^{0.5}} \tag{32}$$

and also puts on limits such that k_c shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

Web-flange interaction was introduced in the 1993 LRFD Specification. The Commentary to the 1993 Specification states that "the maximum limit of 0.763 corresponds to $F_{cr} = 20{,}000/\left(b/t\right)^2$ which was used as the local buckling strength in earlier editions of both the LRFD and ASD Specifications." Although this limit does correspond to that strength equation, web-flange interaction only applies to built-up sections and for built-up sections the strength is given as, $F_{cr} = 11{,}200/\left(b/t\right)^2$ in those past Specifications. Using the strength equation for built-up members, the upper limit would be $k_c = 0.427$ to produce the local buckling strength of the past Specifications.

Additionally, the Commentary says that the other limit, $k_c = 0.35$, corresponds to $h/t_w = 970/\sqrt{F_y}$ which is λ_r. But this cannot always be the case since λ_r is a function of F_y and k_c

is only a function of h/t_w. This equivalence only occurs for F_y = 55 ksi. It is interesting to note that of the 19 data points in the Johnson study, only four of them are in the range applicable to the limits set by AISC.

Johnson (1985) also proposes a new set of equations for determining the critical stress and a new set of bounds for those equations;

$$\lambda_p = 95\sqrt{\frac{k_c}{F_y}} = 0.558\sqrt{\frac{k_c E}{F_y}} \tag{33}$$

and

$$\lambda_r = 195\sqrt{\frac{k_c}{F_y}} = 1.14\sqrt{\frac{k_c E}{F_y}} \tag{34}$$

Neither of these limits compare particularly well to the Specification when k_c is introduced.

Thus, it appears that the web-flange interaction proposed by Johnson (1985) was adopted in the 1993 LRFD Specification but the slenderness limits were not.

Conclusion

The study of element slenderness and its impact on axial strength goes back at least to studies reported in 1939. Most of the limits found in current AISC Specifications seem to have at least some fundamental link to the work of Haaijer and Thürlimann (1958).

- For flanges, the only requirement for an element to be compact is that the element is capable of reaching strain hardening. There is no foundation to statements that indicate a minimum rotation capacity of R = 3 was a basis for compact elements.

- For webs, the compact limit appears to be between the limits recommended by Peolynn and Kulak (1974) and that recommended by Croce (1970). However, there is no indication how the committee came to the conclusion they reached for this limit. It is also clear from Haaijer and Thürlimann (1958) that R = 3 was not a requirement.

- For slender flanges, it appears that the limit comes from the work of Haaijer and Thürlimann (1958) using a residual stress of $0.3F_y$.

- For slender webs the limit given in the Specification does not appear to match with any of the research identified in this study.

- For web-flange interaction, the work of Johnson (1985) appears to be the sole source. However, the limits he proposed were never used. Thus, as stated above, the limits do not appear to match with any of the research identified here.

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