8 LOCAL BUCKLING AND SECTION CLASSIFICATION

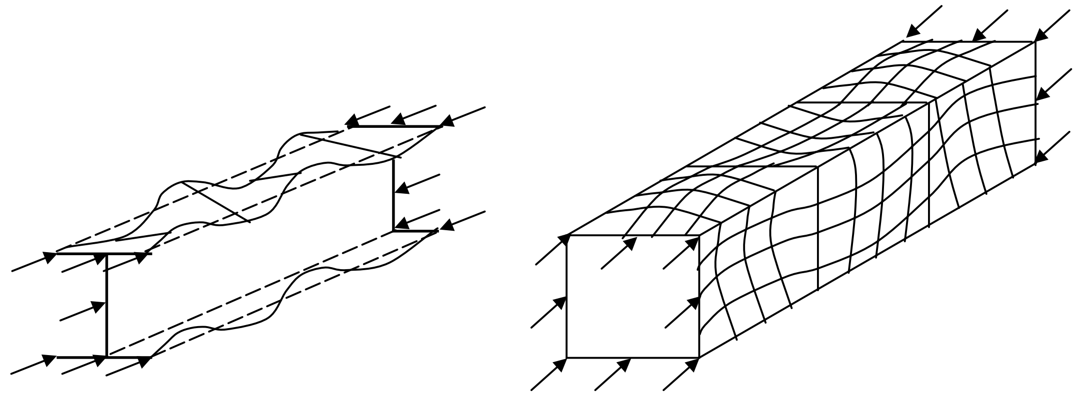
1.0 INTRODUCTION

Sections normally used in steel structures are I-sections, Channels or angles etc. which are called open sections, or rectangular or circular tubes which are called closed sections. These sections can be regarded as a combination of individual plate elements connected together to form the required shape. The strength of compression members made of such sections depends on their slenderness ratio. Higher strengths can be obtained by reducing the slenderness ratio *i.e.* by increasing the moment of inertia of the cross-section. Similarly, the strengths of beams can be increased, by increasing the moment of inertia of the cross-section. For a given cross-sectional area, higher moment of inertia can be obtained by making the sections thin-walled. As discussed earlier, plate elements laterally supported along edges and subjected to membrane compression or shear may buckle prematurely. Therefore, the buckling of the plate elements of the cross section under compression/shear may take place before the overall column buckling or overall beam failure by lateral buckling or yielding. This phenomenon is called *local buckling*. Thus, local buckling imposes a limit to the extent to which sections can be made thin-walled.

Consider an I-section column, subjected to uniform compression [Fig. 1(*a*)]. It was pointed out in the chapter on "Introduction to Plate Buckling" that plates supported on three sides (outstands) have a buckling coefficient k roughly one-tenth that for plates supported on all four sides (internal elements). Therefore, in open sections such as I-sections, the flanges which are outstands tend to buckle before the webs which are supported along all edges. Further, the entire length of the flanges is likely to buckle in the case of the axially compressed member under consideration, in the form of waves. On the other hand, in closed sections such as the hollow rectangular section, both flanges and webs behave as internal elements and the local buckling of the flanges and webs depends on their respective width-thickness ratios. In this case also, local buckling occurs along the entire length of the member and the member develops a 'chequer board' wave pattern [Fig. 1(*b*)].

In the case of beams, the compression flange behaves as a plate element subjected to uniform compression and, depending on whether it is an outstand or an internal element, undergoes local buckling at the corresponding critical buckling stress. However, the web is partially under compression and partially under tension. Even the part in compression is not under uniform compression. Therefore the web buckles as a plate subjected to in-plane bending compression.

Normally, the bending moment varies over the length of the beam and so local buckling may occur only in the region of maximum bending moment.



(a) *(b)*

Fig. 1 Local buckling of Compression Members

Local buckling has the effect of reducing the load carrying capacity of columns and beams due to the reduction in stiffness and strength of the locally buckled plate elements. Therefore it is desirable to avoid local buckling before yielding of the member. Most of the hot rolled steel sections have enough wall thickness to eliminate local buckling before yielding. However, fabricated sections and thin-walled cold-formed steel members usually experience local buckling of plate elements before the yield stress is reached.

It is useful to classify sections based on their tendency to buckle locally before overall failure of the member takes place. For those cross-sections liable to buckle locally, special precautions need to be taken in design. However, it should be remembered that local buckling does not always spell disaster. Local buckling involves distortion of the cross-section. There is no shift in the position of the cross-section as a whole as in global or overall buckling. In some cases, local buckling of one of the elements of the cross-section may be allowed since it does not adversely affect the performance of the member as a whole. In the context of plate bucking, it was pointed out that substantial reserve strength exists in plates beyond the point of elastic buckling. Utilization of this reserve capacity may also be the objective of design. Therefore, local buckling may be allowed in some cases, provided due care is taken to estimate the reduction in the capacity of the section due to it and the consequences are clearly understood.

In what follows, first the basic concepts of plastic theory are introduced. Then the classification of cross-sections is described. The codal provisions limiting the width-thickness ratios of plate elements in a cross-section are given. Finally the implications in design are discussed.

2.0 BASIC CONCEPTS OF PLASTIC THEORY

Before attempting the classification of sections, the basic concepts of plastic theory will be introduced. More detailed descriptions can be found in subsequent chapters.

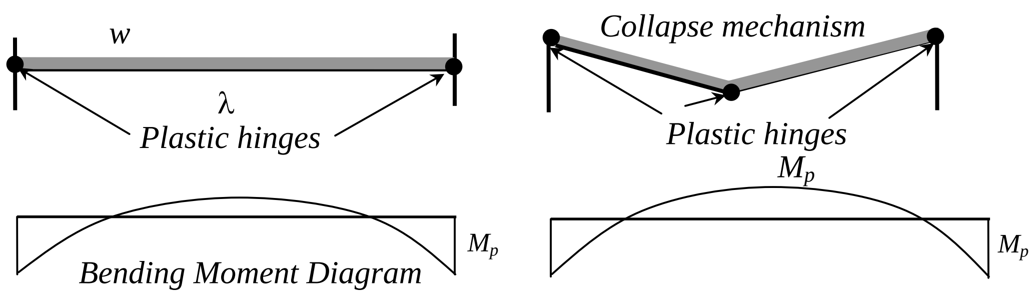


Fig. 2 Formation of a Collapse Mechanism in a Fixed Beam

Consider a beam with both ends fixed and subjected to a uniformly distributed load of *w* per meter length as shown in Fig. 2(*a*). The elastic bending moment at the ends is $w\lambda^2/12$ and at mid-span is $w\lambda^2/24$, where λ is the span. The stress distribution across any cross section is linear [Fig. 3(*a*)]. As *w* is increased gradually, the bending moment at every section increases and the stresses also increase. At a section close to the support where the bending moment is maximum, the stresses in the extreme fibers reach the yield stress. The moment corresponding to this state is called the *first yield moment* M_y, of the cross section. But this does not imply failure as the beam can continue to take additional load. As the load continues to increase, more and more fibers reach the yield stress and the stress distribution is as shown in Fig 3(*b*). Eventually the whole of the cross section reaches the yield stress and the corresponding stress distribution is as shown in Fig. 3(*c*). The moment corresponding to this state is known as the *plastic moment* of the cross section and is denoted by M_p.

The ratio of the plastic moment to the yield moment is known as the *shape factor* since it depends on the shape of the cross section. The cross section is not capable of resisting any additional moment but may maintain this moment for some amount of rotation in which case it acts like a *plastic hinge*. If this is so, then for further loading, the beam, acts as if it is simply supported with two additional moments M_p on either side, and continues to carry additional loads until a third plastic hinge forms at mid-span when the bending moment at that section reaches M_p. The beam is then said to have developed a *collapse mechanism* and will collapse as shown in Fig 2(*b*). If the section is thin-walled, due to local buckling, it may not be able to sustain the moment for additional rotations and may collapse either before or soon after attaining the plastic moment. It may be noted that formation of a single plastic hinge gives a collapse mechanism for a simply supported beam. The ratio of the ultimate rotation to the yield rotation is called the *rotation capacity* of the section. The yield and the plastic moments together with the rotation capacity of the cross-section are used to classify the sections.

3.0 SECTION CLASSIFICATION

Sections are classified depending on their moment-rotation characteristics (Fig. 4). The codes also specify the limiting width-thickness ratios $\beta = b/t$ for component plates, which enables the classification to be made.

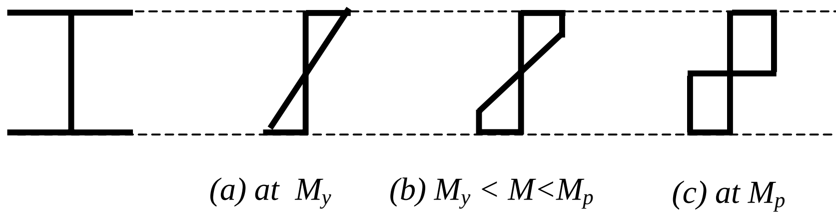


Fig. 3 Plastification of Cross-section under Bending

- Plastic cross-sections: Plastic cross-sections are those which can develop their full-plastic moment M_p and allow sufficient rotation at or above this moment so that redistribution of bending moments can take place in the structure until complete failure mechanism is formed $(b/t \le \beta_1)$ (see Fig. 5).
- Compact cross-sections: Compact cross-sections are those which can develop their full-plastic moment M_p but where the local buckling prevents the required rotation at this moment to take place $(\beta_1 < b/t < \beta_2)$.
- Semi-compact cross-sections: Semi-compact cross-sections are those in which the stress in the extreme fibers should be limited to yield stress because local buckling would prevent the development of the full-plastic moment M_p. Such sections can develop only yield moment M_y $(\beta_2 < b/t \le \beta_3)$.
- Slender cross-sections: Slender cross-sections are those in which yield in the extreme fibers cannot be attained because of premature local buckling in the elastic range $(\beta_3 < b/t)$.

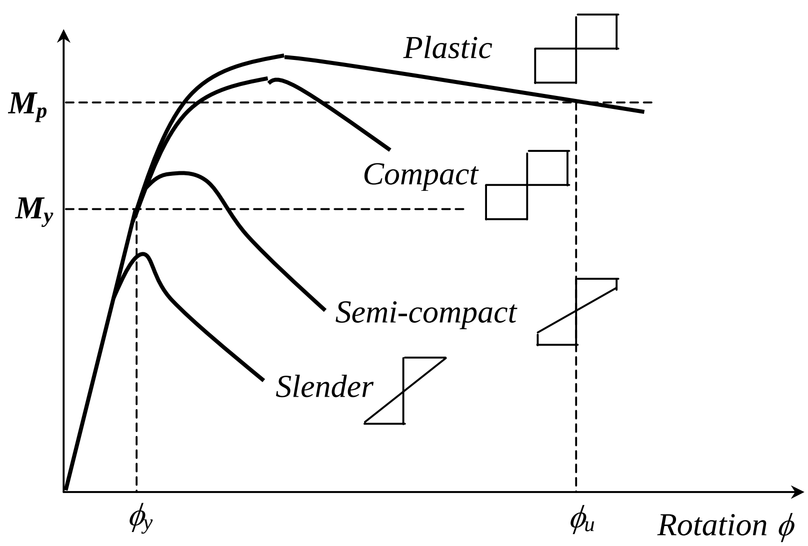


Fig. 4 Section Classification based on Moment-Rotation Characteristics

It should be remembered that even for steels with a large yield plateau, some strain hardening effects are likely to take place and the maximum moment is likely to be larger than M_p for plastic and compact sections. In such cases, the rotation capacity may be taken as the ratio of the rotation when the moment capacity drops back to M_p to the rotation at yield.

The relationship between the moment capacity M_u and the compression flange slenderness *b/t* indicating the β limits is shown in Fig. 5. In this figure, the value of M_u for semi-compact sections is conservatively taken as M_y.

In the above classification, it is assumed that the web slenderness *d/t* is such that its buckling before yielding is prevented. It should be noted that the entire web may not be in uniform compression and if the neutral axis lies in the web, a part of the web may actually be in tension. In this case, the slenderness limits are somewhat relaxed for the webs.

Since the above classification is based on bending, it cannot be used for a compression member. The only criterion required is whether the member is slender or not. However, in practice, it is considered to be prudent to use compact or plastic sections for members carrying predominantly compressive loads.

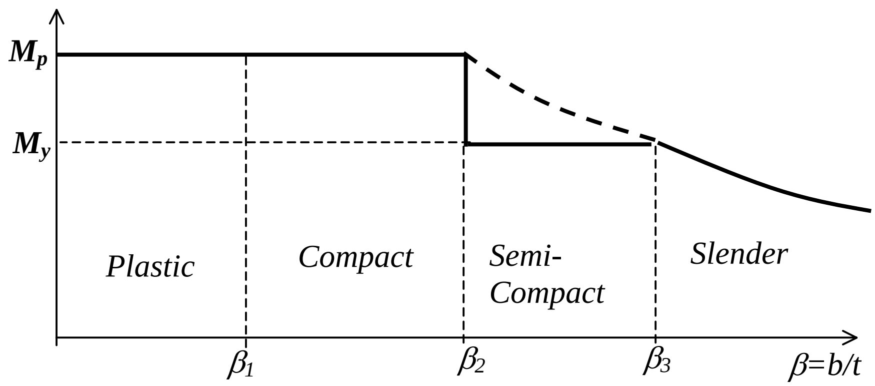


Fig. 5 Moment Capacities of Sections

4.0 LIMITS ON WIDTH-THICKNESS RATIOS

If the flanges and webs of cross-sections are considered to be plates under compression, their limiting width-thickness ratios can be obtained by equating the critical buckling stress to the yield stress. However, such an approach disregards a number of factors such as the actual support restraint provided by the adjoining plate element and the residual stresses and initial imperfections. Therefore, the limiting width-thickness ratios β_1 , β_2 and β_3 are useful for designers and are normally arrived at by validation in the testing laboratory.

The limiting width-thickness ratios for different sections as per IS: 800 are given in Table 2. The various extents of widths and thicknesses for different cross sections have been defined in Fig 6.

Local buckling can be prevented, by controlling the width-thickness ratio. One way of doing this is by adopting higher thickness of the plate. This method is adopted in rolled steel sections. However in the case of built-up sections and cold-formed sections, longitudinal stiffeners are provided which divide the total width into a number of smaller widths. The buckling of stiffened plates is beyond the scope of this chapter.

Table 1. Limits on Width to Thickness Ratio of Plate Elements

Compression element			**Ratio**	**Class of Section**		
				Plastic (β_1)	**Compact (β_2)**	**Semi-compact (β_3)**
Outstanding element of compression flange	Rolled section		b/t_f	9.4ε	10.5ε	15.7ε
	Welded section		b/t_f	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Compression due to bending		b/t_f	29.3ε	33.5ε	42ε
	Axial compression		b/t_f	Not applicable		
Web of an I-H-or box section [c]	Neutral axis at mid-depth		d/t_w	84ε	105ε	126ε
	Generally	If r_1 is negative:	d/t_w	$\frac{84\varepsilon}{1+r_1}$ but $\leq 42\varepsilon$	$\frac{105.0\varepsilon}{1+r_1}$	$\frac{126.0\varepsilon}{1+2r_2}$ but $\leq 42\varepsilon$
		If r_1 is positive :	d/t_w		$\frac{105.0\varepsilon}{1+1.5r_1}$ but $\leq 42\varepsilon$	
	Axial compression		d/t_w	Not applicable		
Web of a channel			d/t_w	42ε	42ε	42ε
Angle, compression due to bending (Both criteria should be satisfied)			b/t d/t	9.4ε $9.4\ \varepsilon$	10.5ε 10.5ε	15.7ε 15.7ε
Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)			b/t d/t $(b+d)/t$	Not applicable		15.7ε 15.7ε 25ε
Outstanding leg of an angle in contact back-to-back in a double angle member			d/t	9.4ε	10.5ε	15.7ε
Outstanding leg of an angle with its back in continuous contact with another component						
Circular tube subjected to moment or axial compression	CHS or built by welding		D/t	$44\varepsilon^2$	$55\varepsilon^2$	$88\varepsilon^2$
Stem of a T-section, rolled or cut from a rolled I-or H-section			D/t_f	8.4ε	9.4ε	18.9ε

Note 1: Section having elements which exceeds semi-compact limits are to be taken as slender cross sections

Note2: $\varepsilon = (250/f_y)^{1/2}$

Note 3: Check webs for shear buckling in accordance when d/t > 67 ε. Where, b is the width of the element may be taken as clear distance between lateral supports or between lateral support and free edge, as appropriate, t is the thickness of element, d is the depth of the web, D mean diameter of the element,

Note 4: Different elements of a cross-section can be in different classes. In such cases the section is classified based on the least favorable classification.

Note 5: The stress ratio r_1 and r_2 *are defined as*

$$r_1 = \frac{\text{actual average axial compressive stress}}{\text{design compressive stress of web alone}}, \quad r_2 = \frac{\text{actual average axial compressive stress}}{\text{design compressive stress of overall section}}$$

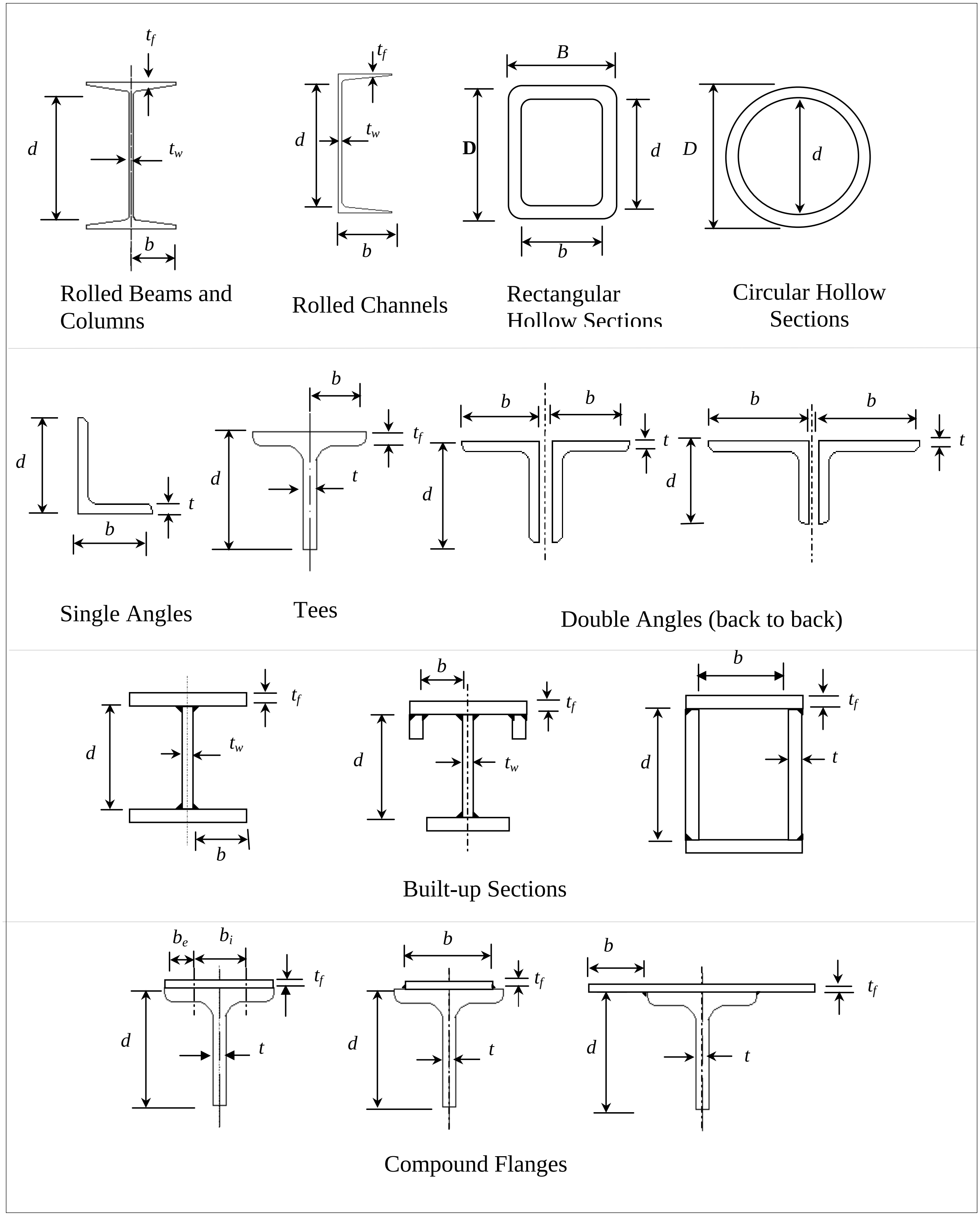


Fig 6 Dimensions of Sections

It may be noted that semi-compact and slender members cannot be used in plastic design. In fact, only plastic sections can be used in indeterminate frames forming plastic collapse mechanisms while compact sections can be used in simply supported beams failing after reaching M_p at one section. In elastic design, semi-compact sections may be used with the understanding that they will fail at M_y. Slender sections also have a stiffness problem and are normally not preferred in hot-rolled structural steel work. However, they are extensively used in cold-formed members and the manufacturer's literature may be consulted while using them. Plate girders are usually designed taking advantage of the tension field approach to achieve economy.

5.0 CONCLUDING REMARKS

Local buckling is discussed as a phenomenon controlling the strength of compression and bending members. The cross-sections are classified into plastic, compact, semi-compact and slender depending upon their moment-rotation characteristics. The limits on the width-thickness ratios of plate elements are provided to classify the section under a particular class. Only plastic and compact sections can be used if limit state design is followed and only plastic sections can be used in mechanism-forming indeterminate frames. Slender sections are to be avoided even in elastic design but are invariably used in cold-formed construction for reasons of economy. In this case, caution is required in predicting their ultimate capacities.

6.0 REFERENCES

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