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Design Curves of Shear Strength for Indian Standard Steel Sections

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Abstract:
Design tables are developed for IS sections as per IS 800:2007 for shear satisfying the codal provisions. The steel plate girders threaten high shear and low bending moment. The flanges mainly resist the moment applied, where as shear by web. Shear for members having the d/t_w ratio is <67ε, sections fail because of yielding. In shear, non-dimensional web slenderness ratio $(\lambda_w) \leq 0.8$ is safe zone and stocky beams fall during this zone will fail because of yielding and for the online slenderness ratio lies in between $0.8< (\lambda_w) <1.2$ are termed as intermediate zone fails because of inelastic bucking and if (λ_w) is quite 1.2 are termed as slender zone fails because of elastic bucking. All the IS sections represent the stocky sections zone and (d/t_w) is a smaller amount than 67 that no additional stiffeners are required. These design charts developed for IS hot rolled sections like ISJB, ISLB, ISMB, ISHB, and ISWB. Design Strength is calculated by using the simple post critical method.

Keywords: *Non-dimensional web slenderness ratio, stocky, buckling, slender, stiffeners.*

I. INTRODUCTION

The steel plate-braces are for the most part exposed to high shear and low twisting minute. The flanges primarily resist the applied moment, while the web basically opposes the shear. In the plate-girders, shear buckling of the thin web panels occurs when the applied shear approaches the critical shear stress of the panel. After buckling, the additional load is carried by a tensile membrane stress field and the flanges.

The disappointment happens when the web yields over the elastic pressure field and plastic pivots create in the top and base ribs. Since the thin snare of the plate-support experiences the shear clasping at a beginning period of stacking, the networks are fortified with transverse or longitudinal stiffeners to build their shear clasping quality.

A slight plate in shear is a basic portrayal of the prevailing stacking case in a slim web board of the plate support and can be viewed as a mix of the essential ductile and compressive in-plane worries as appeared in Figure.No. 1. The compressive part in the long run causes clasping while the elastic segment will in general control clasping. With the improvement of powerful PCs, together with condition of-craftsmanship FE (limited component) programming and easy to understand realistic interface, the FE investigations has gotten a mainstream decision to anticipate the conduct of a structure or its part exposed to the distinctive stacking and limit conditions.

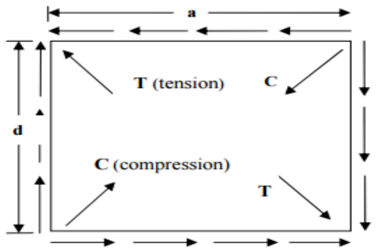

Figure.No. 1. Principle Tensile and Compressive Stresses in Thin Plates Subjected to Shear

A. *Elastic Buckling of Plates*

Nearby soundness of the compacted components of an area without transverse stiffeners can be concentrated by reference to the versatile steadiness of an interminable plate having width (b) and

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thickness (t), as appeared in Figure.No. 2. Assume, that the plate is stacked by compressive powers following up on the basically bolstered sides having width (b). The basic worry of this plate is given by

$$f_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)\left(\frac{b}{t}\right)^2} \quad (1)$$

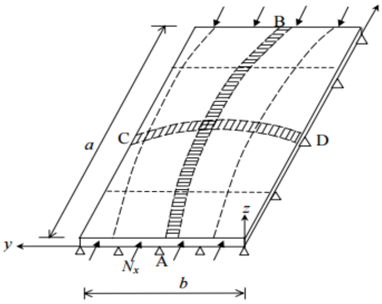

Figure.No. 2. Bending of Plate Under Uni-Axial Bending

Where μ is poisson's proportion of material, b/t is width-to-thickness proportion of the plate, k is clasping coefficient and young's modulus of unbending nature of the material is represented by E.

The estimation of the coefficient k relies upon the limitations along the non-stacked edges of the plate. The manner by which the plates clasp and furthermore the estimation of their basic clasping pressure rely upon the edge conditions, measurements, and stacking.

Consider a plate whose length 'a' is a lot more prominent than the width 'b'. In the event that a longitudinal strip, for example, AB in Figure.No. 2, will in general structure a solitary clasp, its ebb and flow is significantly lesser than ebb and flow of strip CD in transverse direction which opposes clasping. Which implies opposition is more prominent than propensity to clasp and quality comparing to this mode is extremely high. In this way, the plate wants to clasp to such an extent that bends of longitudinal and transverse strips are equivalent as could be expected under the circumstances. This prompts various locks in substitute ways as appeared in Figure.No. 3 with the end goal that the clasps are as square as could be expected under the circumstances. In the event that a= 2b, plate creates two clasps and for a = 3b, it creates three clasps, etc (Figure.No. 3).

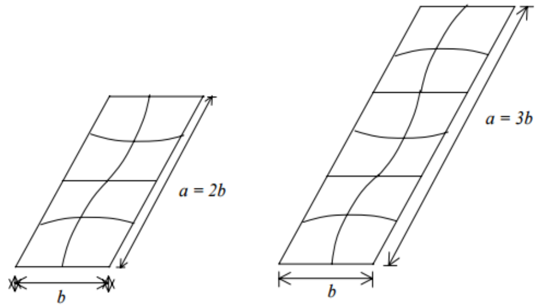

Figure.No. 3. Buckling Mode of Long Plates

B. Plates with Other Support Conditions

Up until this point, it was accepted that plate allows turning about edges in longitudinal direction. Where as other edge conditions are obviously conceivable. For instance, consider container segment madeup of four plates as appeared in Figure.No. 4 (a). On off chance that the spines are generally solid, forestall corners revolution [Figure.No. 4(b)] and web plate will carry on as though its longitudinal fixed edges. Right now, bowing opposition exhibited by transverse strips. For example, CD and clasping pressure is extensively larger for plate of just bolstered longitudinal edge. On off chance that the ribs are likewise inclined to clasping, at that point the corners will pivot as appeared in Figure.No. 4(c) and the basic clasping pressure will be equivalent for plate with just upheld longitudinal edges. Hence, clasping coefficient is component along longitudinal edges and sort stacking. It could be indicated that articulation for key clasping coefficient is as yet legitimate with the exception of the way that esteems are extraordinary. k esteems are different for basic help circumstances and stacking cases as Table.No. 1. In many moved segments, for example, I-segments/channel areas, ribs like plates with a longitudinal edge just bolstered and for other it was free. And are known outstands against plate components with all edges in longitudinal just upheld (inward components). From Table.No. 1, esteem outstands for k is 0.425 around one-tenth for inner components (k=4). Major purpose behind less worth is, transverse strip, (for example, CD in Figure.No. 2) basically pivots and offers small bowing obstruction as appeared in Figure.No. 4.

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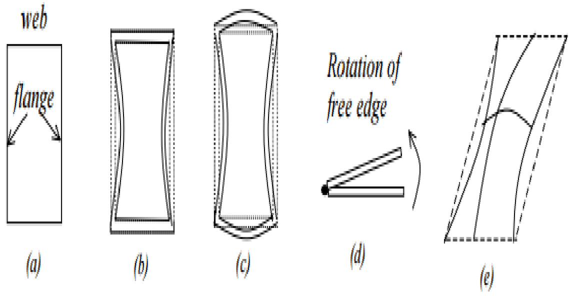

Figure.No. 4. Distinct Edge Conditions Plate Element

Table.No. 1. k Values for various Loads and End Conditions

Loading Conditions	End Conditions	Buckling Coefficient, *k*
Uniaxial Compressive Stress (σ_x)	Hinged-hinged	4.00
	Fixed-fixed	6.97
	Hinged-free	1.27
	Fixed-free	0.43
Shear stress (τ_{xy})	Hinged-hinged	5.35
	Fixed-fixed	8.99

Critical buckling stress and post-buckling behavior are affected by edge conditions. Plates of longitudinal edges (*x*-axis parallel edges in Figure.No. 1) constrict to be straight in plate's plane, transverse stresses of *CD* strip are tensile and plate stiffening effect is against lateral deflection. Free longitudinal edges are to pull -in with respect to *y*-direction [Figure.No. 4 (e)], a transverse stress of CD strip dissipates and plates are less stiff when compared to previous.

If yield stress is less than critical buckling stress, yielding cause failure of plate at support conditions than buckling. Distinct limiting values for varying b/t ratio of structural member plate elements are suggested by respective codes.

II. Review of Literature

Possible bending moments and shear forces interaction on plate girder's capacity. Experimental results and analytical considerations, approximations suitable for design use are suggested [3] and to develop post-buckling strength heavy flanges (plate girder) are required [5].

For all existing failure mechanisms, bending stress results of through thickness on ultimate strength were neglected. And considerable developed bending stresses at failure, low slenderness web panels results in substantial ultimate shear strength reduction [12].

Under shear, axial rigidities within diagonal compression direction at nearer sides aren't reduced severely due to buckling. Due to this lighter flange of post-buckling strength is adequately developed LCBs. LSB and LCB shear panels near the sides are similar to rectangular simply supported plate subjected to an longitudinal even compression load and can be able to develop extensive post-buckling strength [15].

The shear strength of column within the column-beam frame specimen was addicted to the degree of the destruction of the joint [17].

This study proposes design rules for Lite Steel Beams for improving shear strength. It represents main points of study along with new equations for shear strength which are developed to support direct strength method. Proposed Shear strength equations have potential to be used for other typical cold-formed steel sections (lipped channel sections) [19].

III. Methodology

I-beam is essentially consists of web plate, which may buckle by stresses due to shear. These shearing stresses are less compared to yield strength of steel at shear. When a plate is subjected to pure shear, principal stresses equivalent to shear stresses in magnitude, one in tension and another in compression, acting 45^0 to shear stresses. Waves/wrinkles inclined at 45^0 represent buckling.

Shear stress at which buckling of a perfect plate takes place is given by

$$\tau_{cr} = \frac{k_v \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2} \qquad (2)$$

Here τ_{cr}= elastic critical-shear web buckling stress and k_v =buckling coefficient. All four edges simply supported plate, k_v = 5.35. For supports with provided stiffeners,

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$$k_v = 4 + \frac{5.34}{\left(\frac{c}{d}\right)^2};\ \frac{c}{d} \leq 1 \quad (3)$$

$$k_v = 5.35 + \frac{4}{\left(\frac{c}{d}\right)^2};\ \frac{c}{d} \geq 1 \quad (4)$$

Here 'c' and'd' represents transverse stiffeners spacing, depth of web.

Limiting value of d / t_w is given by

$$\left(\frac{d}{t_w}\right)\left(\sqrt{\frac{f_y}{250}}\right) = 82 \quad (5)$$

Thus, it is seen that for f_y = 250MPa, elastic buckling due to shear will occur when the d/t ratio of the web is greater than 82.

As already indicated stress significantly increases elastic by either intermediate transverse stiffeners to decrease the aspect ratio c/d (thereby increasing the value of buckling coefficient k_v), or by longitudinal stiffeners for decreasing depth-to- thickness value d/t_w. The c/d ratio for each panel in 0.5 to 2 range is found to be more efficient.

A. IS 800: 2007 Provisions for Resistance to Shear Buckling

Shear buckling resistance will be verified if

$$\left(\frac{d}{t_w}\right) < 67\varepsilon;\ \text{for an unstiffened web} \quad (6)$$

$$\left(\frac{d}{t_w}\right) > 67\varepsilon\left(\sqrt{\frac{k_y}{5.35}}\right);\ \text{for unstiffened web} \quad (7)$$

Here k_v =shear buckling coefficient

$$\varepsilon = \sqrt{\frac{250}{f_y}}$$

B. Design for Shear Buckling

Web nominal shear strength (V_n) for with/without intermediate stiffeners governed by buckling are evaluated by theories of Simple post – critical method and tensile field. Most popular method for shear buckling design is Simple Post-Critical method.

C. Simple Post-Critical Method as per IS 800: 2007

The simple post-critical method is a general method and is be applicable to the design of all girders. The simple post critical method will used to I- section girder webs, with/without transverse intermediate stiffener at web supports. Nominal shear strength is

$$V_n = V_{cr} \quad (8)$$

$$V_{cr} = A_v \tau_b \quad (9)$$

Where V_{cr}, τ_b are shear force and shear stress corresponding to web buckling, A_v is shear area determined as follows:

$$\text{When } \lambda_w \leq 0.8;\ \tau_b = \frac{f_{yw}}{\sqrt{3}} \quad (10)$$

$$\text{When } 0.8 < \lambda_w < 1.2;\ \tau_b = [1 - 0.8(\lambda_w)]\frac{f_{yw}}{\sqrt{3}} \quad (11)$$

$$\text{When } \lambda_w \geq 1.2;\ \tau_b = \frac{f_{yw}}{\sqrt{3\lambda_w^2}} \quad (12)$$

Where λ_w is web slenderness ratio for non-dimensional shear buckling stress and is given by

$$\lambda_w = \sqrt{\frac{f_{yw}}{3\tau_{cre}}} \quad (13)$$

If a graph is drawn between the values of shear stress corresponding to web buckling and non-dimensional web slenderness ratio, then the resulting graph is shown in Figure.No. 5.

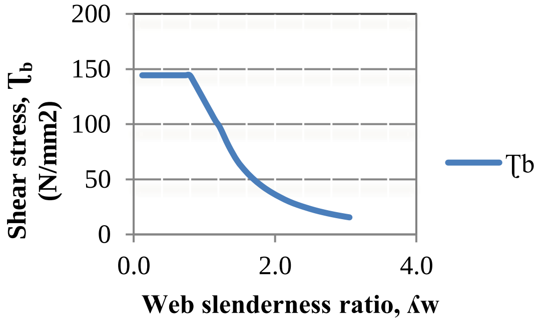

Figure.No. 5. Simple Post-Critical Shear Buckling

Table.No. 2 provides the values of shear buckling strength τ_b for various values of stiffener spacing ratio c/d and depth to thickness ratio d/t_w.

D. Design Shear Strength

Nominal plastic shear resistance under pure shear is

$$V_n = V_p \quad (14)$$

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Where $V_p = \frac{A_v f_{yw}}{\sqrt{3}}$

A_v = area of shear

f_{yw} = web's yield strength.

Design factored shear force, V of beam due to external force should satisfy

$$V_n \leq V_d \tag{15}$$

Where design strength, $V_n = \frac{V_n}{\gamma_{mo}}$

γ_{mo} = partial safety factor against shear failure.

IV. RESULTS AND DISCUSSION

Table.No. 2. Shear Buckling Strength, τ_b (N/mm^2) of Webs for f_y=250 MPa

	Stiffener spacing ratio c/d												
d/t_w	**0.40**	**0.50**	**0.60**	**0.70**	**0.80**	**0.90**	**1.00**	**1.20**	**1.40**	**1.60**	**1.80**	**2.00**	**2.50**
57	144.3	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34
58	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34
59	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34
60	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34
61	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34
62	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34
63	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34
64	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34
65	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34
66	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34
67	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34
68	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34
69	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34
70	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	143.39
71	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	142.06
72	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	143.49	140.73
73	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	142.19	139.39
74	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	142.62	140.90	138.06
75	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	143.62	141.34	139.60	136.73
76	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	142.38	140.07	138.31	135.39
77	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	141.14	138.80	137.01	134.06
78	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	139.90	137.53	135.72	132.73
79	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	138.66	136.26	134.42	131.39
80	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	137.41	134.99	133.13	130.06
81	144.34	144.34	144.34	144.34	144.34	144.34	144.34	144.34	142.86	136.17	133.71	131.83	128.73
82	144.34	144.34	144.34	144.34	144.34	144.34	144.34	142.87	141.70	134.93	132.44	130.54	127.39
83	144.34	144.34	144.34	144.34	144.34	144.34	144.34	141.72	140.54	133.69	131.17	129.24	126.06
84	144.34	144.34	144.34	144.34	144.34	144.34	144.34	140.58	139.39	132.45	129.90	127.95	124.73
85	144.34	144.34	144.34	144.34	144.34	144.34	144.34	139.43	138.23	131.21	128.63	126.65	123.40
86	144.34	144.34	144.34	144.34	144.34	144.34	144.34	138.29	137.07	129.97	127.36	125.36	122.06
87	144.34	144.34	144.34	144.34	144.34	144.34	144.34	137.14	135.91	128.73	126.08	124.06	120.73

88	144.34	144.34	144.34	144.34	144.34	144.34	142.81	136.00	134.75	127.48	124.81	122.77	119.40
89	144.34	144.34	144.34	144.34	144.34	144.34	141.75	134.86	133.59	126.24	123.54	121.47	118.06
90	144.34	144.34	144.34	144.34	144.34	144.34	140.68	133.71	132.43	125.00	122.27	120.18	116.73
91	144.34	144.34	144.34	144.34	144.34	144.34	139.61	132.57	131.28	123.76	121.00	118.89	115.40
92	144.34	144.34	144.34	144.34	144.34	144.34	138.54	131.42	130.12	122.52	119.73	117.59	114.06
93	144.34	144.34	144.34	144.34	144.34	143.51	137.48	130.28	128.96	121.28	118.45	116.30	112.73
94	144.34	144.34	144.34	144.34	144.34	142.51	136.41	129.13	127.80	120.04	117.18	115.00	111.40
95	144.34	144.34	144.34	144.34	144.34	141.51	135.34	127.99	126.64	118.80	115.91	113.71	110.06
96	144.34	144.34	144.34	144.34	144.34	140.51	134.28	126.84	125.48	117.55	114.64	112.41	108.73
97	144.34	144.34	144.34	144.34	144.34	139.50	133.21	125.70	124.32	116.31	113.37	111.12	107.40
98	144.34	144.34	144.34	144.34	144.34	138.50	132.14	124.55	123.16	115.07	112.10	109.82	106.06
99	144.34	144.34	144.34	144.34	144.34	137.50	131.07	123.41	122.01	113.83	110.82	108.53	104.73
100	144.34	144.34	144.34	144.34	144.34	136.50	130.01	122.27	120.85	112.59	109.55	107.23	103.40
101	144.34	144.34	144.34	144.34	144.34	135.49	128.94	121.12	119.69	111.35	108.28	105.94	102.06
102	144.34	144.34	144.34	144.34	144.34	134.49	127.87	119.98	118.53	110.11	107.01	104.64	100.73
103	144.34	144.34	144.34	144.34	144.34	133.49	126.81	118.83	117.37	108.87	105.74	103.35	99.40
104	144.34	144.34	144.34	144.34	144.34	132.49	125.74	117.69	116.21	107.62	104.47	102.05	98.06
105	144.34	144.34	144.34	144.34	144.34	131.49	124.67	116.54	115.05	106.38	103.19	100.76	98.21
106	144.34	144.34	144.34	144.34	144.34	130.48	123.61	115.40	113.89	105.14	101.92	99.46	96.37
107	144.34	144.34	144.34	144.34	144.34	129.48	122.54	114.25	112.74	103.90	100.65	98.17	94.58
108	144.34	144.34	144.34	144.34	144.34	128.48	121.47	113.11	111.58	102.66	99.38	98.41	92.83
109	144.34	144.34	144.34	144.34	144.34	127.48	120.40	111.96	110.42	101.42	100.18	96.62	91.14
110	144.34	144.34	144.34	144.34	144.34	126.47	119.34	110.82	109.26	100.18	98.36	94.87	89.49
115	144.34	144.34	144.34	144.34	144.34	121.46	114.00	105.10	103.47	94.45	90.00	86.80	81.88
120	144.34	144.34	144.34	144.34	144.34	116.45	108.67	99.38	99.55	86.74	82.65	79.71	75.19
125	144.34	144.34	142.80	144.34	144.34	111.44	103.33	94.03	91.74	79.94	76.17	73.46	69.30
130	144.34	144.34	139.05	144.34	144.34	106.43	97.99	86.94	84.82	73.91	70.43	67.92	64.07
135	144.34	144.34	135.29	144.34	144.34	101.42	92.74	80.62	78.66	68.54	65.31	62.98	59.41
140	144.34	144.34	131.53	144.34	144.34	97.76	86.23	74.96	73.14	63.73	60.72	58.57	55.25
145	144.34	142.84	127.78	144.34	144.34	91.14	80.39	69.88	68.18	59.41	56.61	54.60	51.50
150	144.34	139.60	124.02	144.34	144.34	85.16	75.12	65.30	63.71	55.52	52.90	51.02	48.12
155	144.34	136.37	120.26	144.34	144.34	79.76	70.35	61.16	59.67	51.99	49.54	47.78	45.07
160	144.34	133.13	116.51	144.34	144.34	74.85	66.02	57.39	56.00	48.79	46.49	44.84	42.30
165	144.34	129.89	112.75	144.34	144.34	70.38	62.08	53.97	52.65	45.88	43.72	42.16	39.77
170	144.34	126.66	108.99	144.34	144.34	66.30	58.48	50.84	49.60	43.22	41.18	39.72	37.47
175	143.40	123.42	105.24	144.34	144.34	62.57	55.19	47.98	46.81	40.79	38.86	37.48	35.36
180	140.73	120.18	101.48	144.34	144.34	59.14	52.17	45.35	44.24	38.55	36.73	35.43	33.42
185	138.06	116.94	99.62	144.34	144.34	55.99	49.38	42.93	41.88	36.50	34.78	33.54	31.64
190	135.40	113.71	94.45	144.34	144.34	53.08	46.82	40.70	39.71	34.60	32.97	31.80	29.99
195	132.73	110.47	89.66	144.34	144.34	50.39	44.45	38.64	37.70	32.85	31.30	30.19	28.48
200	130.06	107.23	85.24	144.34	144.34	47.90	42.25	36.73	35.84	31.23	29.75	28.70	27.07

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205	127.40	104.00	81.13	144.34	144.34	45.60	40.22	34.96	34.11	29.72	28.32	27.31	25.77
210	124.73	100.76	77.31	144.34	144.34	43.45	38.33	33.32	32.51	28.32	26.99	26.03	24.55
215	122.06	99.33	73.76	144.34	144.34	41.45	36.56	31.78	31.01	27.02	25.75	24.83	23.42
220	119.40	94.87	70.44	144.34	144.34	39.59	34.92	30.36	29.62	25.81	24.59	23.72	22.37
225	116.73	90.70	67.35	144.34	144.34	37.85	33.39	29.02	28.32	24.67	23.51	22.67	21.39
230	114.06	86.80	64.45	144.34	144.34	36.22	31.95	27.77	27.10	23.61	22.50	21.70	20.47
235	111.40	83.14	61.74	144.34	144.34	34.70	30.61	26.60	25.96	22.62	21.55	20.79	19.61
240	108.73	79.71	59.19	144.34	144.34	33.27	29.34	25.51	24.89	21.69	20.66	19.93	18.80
245	106.06	76.49	56.80	144.34	144.34	31.92	28.16	24.48	23.88	20.81	19.83	19.12	18.04
250	103.40	73.46	54.55	144.34	144.34	30.66	27.04	23.51	22.94	19.99	19.04	18.37	17.32

Table.No. 3. Design Shear Strength for ISJB Section

Section	d	t_w	d/t_w	E	f_y	k_v	t_{cr}	λ_w	τ_b	Area	V_d
ISJB 150	150	3	50.00	2×10^5	250	5.35	680.45	17.7	144.3	450	59.0
ISJB 175	175	3.2	54.69	2×10^5	250	5.35	654.12	17.5	144.3	560	73.4
ISJB 200	200	3.4	58.82	2×10^5	250	5.35	623.25	17.3	144.3	680	89.2
ISJB 225	225	3.7	60.81	2×10^5	250	5.35	544.06	16.7	144.3	832.5	109.2

Table.No. 4. Design Shear Strength for ISLB Section

Sections	d	t_w	d/t_w	k_v	E	τ_{cr}	λ_w	τ_b	Area	V_d
ISLB 75	75	3.7	20.27	5.35	2×10^5	65829.46	0.06	144.34	278	36.41
ISLB 100	100	4	25.00	5.35	2×10^5	64257.81	0.09	144.34	400	52.49
ISLB 125	125	4.4	28.41	5.35	2×10^5	54861.19	0.12	144.34	550	72.17
ISLB 150	150	4.8	31.25	5.35	2×10^5	46482.79	0.15	144.34	720	94.48
ISLB 175	175	5.1	34.31	5.35	2×10^5	42552.32	0.18	144.34	893	117.11
ISLB 200	200	5.4	37.04	5.35	2×10^5	38691.97	0.20	144.34	1080	141.72
ISLB 225	225	5.8	38.79	5.35	2×10^5	32706.68	0.22	144.34	1305	171.24
ISLB 250	250	6.1	40.98	5.35	2×10^5	29702.06	0.25	144.34	1525	200.11
ISLB 275	275	6.4	42.97	5.35	2×10^5	26963.65	0.28	144.34	1760	230.95
ISLB 300	300	6.7	44.78	5.35	2×10^5	24489.95	0.30	144.34	2010	263.75
ISLB 325	325	7	46.43	5.35	2×10^5	22266.76	0.32	144.34	2275	298.53
ISLB 350	350	7.4	47.30	5.35	2×10^5	19200.26	0.33	144.34	2590	339.86
ISLB 400	400	8	50.00	5.35	2×10^5	16064.45	0.37	144.34	3200	419.90
ISLB 450	450	8.6	52.33	5.35	2×10^5	13532.70	0.41	144.34	3870	507.82
ISLB 500	500	9.2	54.35	5.35	2×10^5	11481.13	0.44	144.34	4600	603.61
ISLB 550	550	9.9	55.56	5.35	2×10^5	9418.63	0.46	144.34	5445	714.49
ISLB 600	600	10.5	57.14	5.35	2×10^5	8120.07	0.49	144.34	6300	826.68

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Table.No. 5. Design Shear Strength for ISMB Section

Secions	d	t_w	d/t_w	K_v	E	f_y	t_{cr}	λ_w	τ_b	Area	Vd
ISMB 100	100	4	25.00	5.35	2×10^5	250	1547.38	0.31	144.34	400	52.49
ISMB 125	125	4.4	28.41	5.35	2×10^5	250	1198.29	0.35	144.34	550	72.17
ISMB 150	150	4.8	31.25	5.35	2×10^5	250	990.33	0.38	144.34	720	94.48
ISMB 175	175	5.5	31.82	5.35	2×10^5	250	955.27	0.39	144.34	962.5	126.30
ISMB 200	200	5.7	35.09	5.35	2×10^5	250	785.54	0.43	144.34	1140	149.59
ISMB 225	225	6.5	34.62	5.35	2×10^5	250	807.12	0.42	144.34	1462.5	191.91
ISMB 250	250	6.9	36.23	5.35	2×10^5	250	736.71	0.44	144.34	1725	226.35
ISMB 300	300	7.5	40.00	5.35	2×10^5	250	604.45	0.49	144.34	2250	295.24
ISMB 350	350	8.1	43.21	5.35	2×10^5	250	517.98	0.53	144.34	2835	372.01
ISMB 400	400	8.9	44.94	5.35	2×10^5	250	478.78	0.55	144.34	3560	467.14
ISMB 450	450	9.4	47.87	5.35	2×10^5	250	422.00	0.58	144.34	4230	555.06
ISMB 500	500	10.2	49.02	5.35	2×10^5	250	402.47	0.60	144.34	5100	669.22
ISMB 550	550	11.2	49.11	5.35	2×10^5	250	401.04	0.60	144.34	6160	808.31
ISMB 600	600	12	50.00	5.35	2×10^5	250	386.85	0.61	144.34	7200	944.78

Table.No. 6. Design Shear Strength for ISWB Section

Sections	d	t_w	d/tw	f_y	E	k_v	t_{cr}	λ_w	τ_b	Area	V_d
ISWB 150	150	5.4	27.8	250	2×10^5	5.4	1253.38	0.34	144.34	810	106.29
ISWB 175	175	5.8	30.2	250	2×10^5	5.4	1062.33	0.37	144.34	1015	133.19
ISWB 200	200	6.1	32.8	250	2×10^5	5.4	899.66	0.40	144.34	1220	160.09
ISWB 225	225	6.4	35.2	250	2×10^5	5.4	782.48	0.43	144.34	1440	188.96
ISWB 250	250	6.7	37.3	250	2×10^5	5.4	694.62	0.46	144.34	1675	219.79
ISWB 300	300	7.4	40.5	250	2×10^5	5.4	588.44	0.50	144.34	2220	291.31
ISWB 350	350	8.0	43.8	250	2×10^5	5.4	505.27	0.53	144.34	2800	367.42
ISWB 400	400	8.6	46.5	250	2×10^5	5.4	447.05	0.57	144.34	3440	451.40
ISWB 450	450	9.2	48.9	250	2×10^5	5.4	404.23	0.60	144.34	4140	543.25
ISWB 500	500	9.9	50.5	250	2×10^5	5.4	379.15	0.62	144.34	4950	649.54
ISWB 550	550	10.5	52.4	250	2×10^5	5.4	352.48	0.64	144.34	5775	757.79
ISWB600@133.7kg	600	11.2	53.6	250	2×10^5	5.4	336.99	0.65	144.34	6720	881.80
ISWB600@145.1kg	600	11.8	50.8	250	2×10^5	5.4	374.06	0.62	144.34	7080	929.04

Table.No. 7. Design Shear Strength for ISHB Section

Sections	t_w	d	d/t_w	f_y	E	k_v	t_{cr}	λ_w	τ_b	Area	V_d
ISHB 150@27.1kg	5.4	150	27.8	250	2×10^5	5.4	1253.4	0.3	144.3	810	106.29
ISHB 150@30.6kg	8.4	150	17.9	250	2×10^5	5.4	3032.9	0.2	144.3	1260	165.34
ISHB 150@34.6kg	11.8	150	12.7	250	2×10^5	5.4	5984.9	0.2	144.3	1770	232.26
ISHB 200@37.3kg	6.1	200	32.8	250	2×10^5	5.4	899.7	0.4	144.3	1220	160.09
ISHB 200@40kg	7.8	200	25.6	250	2×10^5	5.4	1471.0	0.3	144.3	1560	204.70

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ISHB 225@43.1kg	6.5	225	34.6	250	2×10^5	5.4	807.1	0.4	144.3	1462.5	191.91
ISHB 225@46.8kg	8.6	225	26.2	250	2×10^5	5.4	1412.9	0.3	144.3	1935	253.91
ISHB 250@51kg	6.9	250	36.2	250	2×10^5	5.4	736.7	0.4	144.3	1725	226.35
ISHB 250@54.7kg	8.8	250	28.4	250	2×10^5	5.4	1198.3	0.3	144.3	2200	288.68
ISHB 300@58.8kg	7.6	300	39.5	250	2×10^5	5.4	620.7	0.5	144.3	2280	299.18
ISHB 300@63kg	9.4	300	31.9	250	2×10^5	5.4	949.5	0.4	144.3	2820	370.04
ISHB 350@67.4kg	8.3	350	42.2	250	2×10^5	5.4	543.9	0.5	144.3	2905	381.19
ISHB 350@72.4kg	10.1	350	34.7	250	2×10^5	5.4	805.4	0.4	144.3	3535	463.86
ISHB 400@77.4kg	9.1	400	44.0	250	2×10^5	5.4	500.5	0.5	144.3	3640	477.64
ISHB 400@82.2kg	10.6	400	37.7	250	2×10^5	5.4	679.2	0.5	144.3	4240	556.37
ISHB 450@87.2kg	9.8	450	45.9	250	2×10^5	5.4	458.7	0.6	144.3	4410	578.68
ISHB 450@92.5kg	11.3	450	39.8	250	2×10^5	5.4	609.8	0.5	144.3	5085	667.25

V. Conclusion

1. Shear strength of view,the web slenderness ratio (λ_w) ≤ 0.8 is considered to be safe zone and stocky beams fall in this zone will fail due to yielding and for the web slenderness ratio lies in between 0.8<(λ_w) <1.2 are termed as intermediate zone fails due to inelastic bucking and if (λ_w) is more than 1.2 are termed as slender zone fails due to elastic bucking. All the **IS sections** fall into the stocky sections zone where the (d/t_w) is less than 67 for which no additional stiffeners are required.
2. Easily understood from the Table.No. 8.

Table.No. 8. Zone and Type of Failure

Zone/Failure	Stocky	Intermediate	Slender
Shear	$\lambda_w \leq 0.8$	$0.8 < \lambda_w < 1.2$	$\lambda_w > 1.2$
Failure criteria	Yielding	Inelastic buckling	Elastic buckling

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