Allowable stresses

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Introduction

- The actual stresses in any part of steel bridge must not exceed the elastic limit of the material otherwise permanent deformation would occur. All structural calculations are approximate even if all loads are carefully considered. In trusses neglect the secondary stresses due to the rigidity of joints.

The forces in members are determined under the assumption that the connections are hinge and the forces along the members are axial. Only the primary stresses can be calculated. In some cases the secondary stresses may reach 30 – 60 % of the primary stresses. The analyses neglect also the torsion in the main girders due to the deflections of the X-girders. The unequal distribution of stresses over the cross section due to bolts holes hasn’t taken into consideration

- The allowable stresses (maximum stresses used in the calculation) must therefore be lower than the elastic limit. The more accurate calculations of steel bridge and the better shop work, the higher allowable stresses may be taken. Also, in the calculation if all possible forces are taken into account the allowable stresses can be taken higher than in case that only D.L., L.L, and Impact are considered.

The permissible stresses for standard grade structural steel determined according to the grade of steel. Structural sections shall be classified, depending on dw/tw for web and c/tf for flanges under compression, axial bending, to compact, noncompact, and slender sections as shown Fig(4-1)

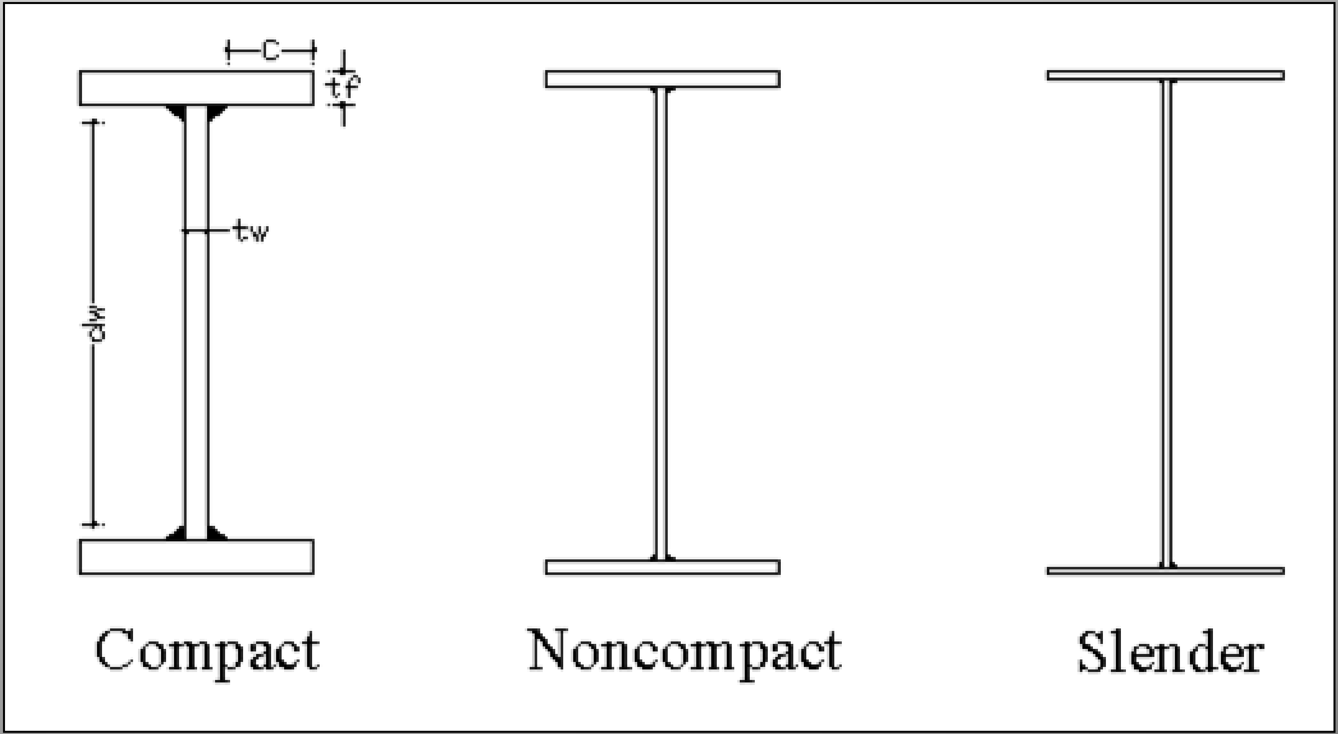

Figure-4.1

F_y and F_u (t/cm^2) depend on the thickness t(1.4-p2).

Grade of steel	t[40mm		100mm/ t >400mm	
	Fy	Fu	Fy	Fu
ST37	2.40	3.60	2.15	3.40
ST44	2.80	4.40	2.55	4.10
ST52	3.60	5.20	3.35	4.90

2.2.1(p6) **Primary + additional stresses (**wind load or earthquake loads, lateral shock, etc.**)**

2.2.3(p7) **Additional stresses**

Additional stresses (allowable) = Primary stress 31.20

2.3(p7) **Secondary stresses in truss members**

1. Chord member's depth > 1/10 of their length.

   Diagonal member's depth > 1/15 of their length.

2. Truss with sub-panel.

Reduce 20 % of the allowable stress

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2.6.1(p8) Compression element, Axial or bending

$$\left.\begin{array}{l} -\,Compact \\ -\,Noncompact \\ -\,Selender \end{array}\right\} \qquad \left(\frac{\mathrm{d_w}}{\mathrm{t_w}}\ and\ \frac{\mathrm{C_f}}{\mathrm{t_f}}\right) \le \frac{Factor}{\sqrt{F_y}}$$

Factor depends on:

1. Support of element ((One side (unstiffened element) or two sides (stiffened element))

and shape of the cross section, I, C, □, L, etc.

2. Load on element [(N) or (M) or (M+N)]

(p9,10,11-Table 2.1.a,b&c)

2.6.2(p13) Axial tension

$$F_t = 0.58\,F_y \Big\} \quad \begin{matrix} t \le 40\,mm \rightarrow F_y \\ 40\,mm < t \le 100\,mm \rightarrow F_y \end{matrix} \Bigg\} \quad \rightarrow From\ \text{clause 1.4, get}\ F_y$$

Hence for,

$$t \le 40\,mm \Rightarrow \begin{cases} F_t = 1.40\,t/cm^2 & \text{ST 37} \\ F_t = 1.60\,t/cm^2 & \text{ST 44} \\ F_t = 2.10\,t/cm^2 & \text{ST 52} \end{cases}$$

$$40\,mm < t \le 100\,mm \Rightarrow \begin{cases} F_t = 1.30\,t/cm^2 & \text{ST 37} \\ F_t = 1.50\,t/cm^2 & \text{ST 44} \\ F_t = 2.00\,t/cm^2 & \text{ST 52} \end{cases}$$

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2.6.3(p13) Allowable shear stress

q_{all}

$$q_{all} = 0.35\,F_y \Big\}\qquad \left.\begin{array}{l} t \le 40\,mm \rightarrow F_y \\ 40\,mm < t \le 100\,mm \rightarrow F_y \end{array}\right\} \rightarrow \text{From clause 1.4, get } F_y$$

Hence for,

$$\mathbf{t \le 40\,mm} \Rightarrow \begin{cases} q_{all} = 0.84\,\mathbf{t/cm^2} & \text{ST 37} \\ q_{all} = 0.98\,\mathbf{t/cm^2} & \text{ST 44} \\ q_{all} = 1.26\,\mathbf{t/cm^2} & \text{ST 52} \end{cases}$$

$$40\,\mathbf{mm} < \mathbf{t} \le 100\,\mathbf{mm} \Rightarrow \begin{cases} q_{all} = 0.75\,\mathbf{t/cm^2} & \text{ST 37} \\ q_{all} = 0.89\,\mathbf{t/cm^2} & \text{ST 44} \\ q_{all} = 1.17\,\mathbf{t/cm^2} & \text{ST 52} \end{cases}$$

2.6.3.1(p13) Effective web area

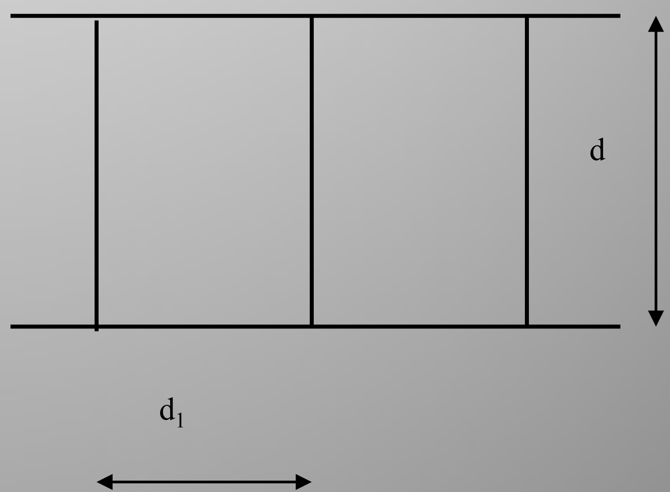
Rolled section = Total height 3t_w

Built up section = Web height 3t_w

2.6.3.2**(p14)** Shear buckling of web

$$\frac{d_w}{t_w} \le \frac{105}{\sqrt{F_y}} \qquad \alpha = \frac{d_1}{d}$$

Stiffened web



$$\alpha \le 1 \rightarrow K_q = 4 + \frac{5.34}{\alpha^2}$$

$$\alpha > 1 \rightarrow K_q = 5.34 + \frac{4}{\alpha^2}$$

Unstiffened web

$\alpha = \infty \Rightarrow K_q = 5.34$

If $\quad \dfrac{d_w}{t_w} \le 45\sqrt{\dfrac{K_q}{F_y}}, \quad \lambda_q < 0.80$

$\Rightarrow$ no web buckling occur $\quad \Rightarrow q_p = 0.35\ F_y$

If, $\quad \dfrac{d_w}{t_w} > 45\sqrt{\dfrac{K_q}{F_y}}$

$\Rightarrow$ Check web buckling

$$\lambda_q = \frac{d_w / t_w}{57}\sqrt{\frac{F_y}{K_q}}$$

(no web buckling occur)

$$(\lambda_q \le 0.80) \rightarrow q_b = 0.35F_y$$

$$(0.80 < \lambda_q < 1.20) \rightarrow q_b = (1.50 - 0.625\lambda_q) \times (0.35F_y)$$

$$(\lambda_q \ge 1.20) \rightarrow q_b = \frac{0.90}{\lambda_q} \times (0.35F_y)$$

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2.6.4(p15) Axial compression

$$\lambda = \frac{k \times l}{r}$$

$$F_c = 0.58F_y - \frac{(0.58F_y - 0.75)}{10^4}\lambda^2 \left.\begin{matrix} \Leftrightarrow t \le 40\ mm \\ \Leftrightarrow t > 40\ mm \end{matrix}\right\} \rightarrow \text{get } F_y \text{ from } 1.4$$

Grade of steel	t [40 mm	40 < t < 100 mm
	F_c (t/cm^2)	F_c (t/cm^2)
ST37	$F_c = 1.40 - 0.000065\lambda^2$	$F_c = 1.30 - 0.000055\lambda^2$
ST44	$F_c = 1.60 - 0.000085\lambda^2$	$F_c = 1.50 - 0.000075\lambda^2$
ST52	$F_c = 2.1 - 0.000135\lambda^2$	$F_c = 2.0 - 0.000125\lambda^2$

- For compact and Non-compact sections use full area(Table2.1-p9-11).
- For slender sections use effective area(Tables 2.3&2.4-p23&24).
- For one angle reduce F_c by 40 %(p15).

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2.6.5(p16) Bending stress

1- For compact sections and the laterally unsupported length (L_u)of the compression flange is limited by:

(L_u is the smaller of)

- Box sections

$$\mathbf{L_u} \le \frac{84}{\mathbf{F_y}} \times \mathbf{b_f} \qquad \text{Or} \qquad \mathbf{L_u} \le \left(137 + 84 \times \frac{\mathbf{M}_1}{\mathbf{M}_2}\right) \times \frac{\mathbf{b_f}}{\mathbf{F_y}}$$

I-shape sections

$$\mathbf{L_u} \le \frac{20\mathbf{b_f}}{\sqrt{\mathbf{F_y}}} \qquad \text{Or} \qquad L_u \le \frac{1380\, b_f}{d \times F_y} \times C_b$$

C_b From Table2.2

- Then

· $F_b = 0.64\ 3F_y$ (M_x) Boxand I-shapes Clause **2.6.5.1**

· $F_b = 0.72\ 3F_y$ (M_y) I-shapes Clause **2.6.5.2**

· $F_b = 0.64\ 3F_y$ (M_y) Box shapes Clause **2.6.5.3**

1- 1- For Non-compact sections:

· $F_b = 0.58\ 3F_y$ (M_x & M_y) Box shapes Clause **2.6.5.4**

2- 2- For slender (Box and I-shapes) and Non-compact (I-shapes) sections:

-

Tension Clause **2.6.5.5**

· F_{bt} = 0.58 3F_y

- Compression Clause **2.6.5.5**

1- L_u [L_{all}

· F_{bc} = 0.58 3F_y

2- $L_u > L_{all}$

· i – (Shallow thick flanged section L_uxt_f / $b_f$xd >10 (P18))

For any value of

·

$$\mathbf{F_{ltb1}} = \frac{800}{\mathbf{L_u} \times \mathbf{d} / \mathbf{A_f}} \times \mathbf{C_b} \leq 0.58\mathbf{F_y}$$

(eq 2.23)

· ii - (Deep thin flanged section L_uxt_f / $b_f$xd <0.4 (P18))

$$84\sqrt{\frac{C_b}{F_y}} > \frac{L_u}{r_t}$$

$$F_{ltb2} = 0.58F_y \qquad \text{(eq2.24)}$$

$$84\sqrt{\frac{C_b}{F_y}} \le \frac{L_u}{r_t} \le 188\sqrt{\frac{C_b}{F_y}}$$

$$F_{ltb2} = \left(0.64 - \frac{(L_u / r_T)^2 \times F_y}{1.176 \times 10^5 C_b}\right) \times F_y \le 0.58F_y \qquad \text{(eq 2.25)}$$

$$\frac{L_u}{r_t} \ge 188\sqrt{\frac{C_b}{F_y}}$$

$$F_{ltb2} = \frac{12000}{(L_u / r_T)^2} \times C_b \le 0.58F_y \qquad \text{(eq2.26)}$$

$$F_{ltb} = \sqrt{F_{ltb1}^2 + F_{ltb2}^2} \le 0.58F_y \qquad \text{(eq2.27)}$$

- II - For Channels(p21) F_{ltb};

$$\mathbf{F_{ltb}} = \frac{800}{\mathbf{L_u} \times \mathbf{d} / \mathbf{A_f}} \times \mathbf{C_b} \leq 0.58 \mathbf{F_y} \quad (\mathbf{M_x}) \qquad \text{(eq2.29)}$$

- III - For slender sections use effective width (b_e) and the stress for non-compact(p21).

- Effective width b_e for slender sections(Table 2.3& 2.4 – p23&24);

$$\Psi = \frac{f_2}{f_1}$$

$$\mathbf{K_\sigma} = \frac{16}{\left[(1+\Psi)^2 + 0.112(1-\Psi)^2\right]^{0.5} + (1+\Psi)} \qquad (\textbf{Table } 2.3)$$

- For any value of Ψ get $\mathbf{K_\sigma}$ from tables 2.3, and 2.4 for stiffened and unstiffened elements respectively.

Calculate $\bar{\lambda}_\rho = \dfrac{\bar{b}/t}{44}\sqrt{\dfrac{F_y}{K_\sigma}}$ (plate slenderness)

Calculate $\rho = \left(\bar{\lambda}_\rho - 0.15 - 0.05\Psi\right) / \bar{\lambda}_\rho^2 \leq 1.0$

$b_e = \rho \times \bar{b}$

Summary Table for Lateral Torsional Buckling ($L_u > L_{all}$)

$F_{ltb1} = \dfrac{800}{L_u \times d / A_f} \times C_b$	
$84\sqrt{\dfrac{C_b}{F_y}} > \dfrac{L_u}{r_t}$	$F_{ltb2} = 0.58 F_y$
$84\sqrt{\dfrac{C_b}{F_y}} \le \dfrac{L_u}{r_t} \le 188\sqrt{\dfrac{C_b}{F_y}}$	$F_{ltb2} = \left(0.64 - \dfrac{(L_u / r_T)^2 \times F_y}{1.176 \times 10^5 C_b}\right) \times F_y$
$\dfrac{L_u}{r_t} \ge 188\sqrt{\dfrac{C_b}{F_y}}$	$F_{ltb2} = \dfrac{12000}{(L_u / r_T)^2} \times C_b$
For all $F_{ltb} \le 0.58\, F_y$	$F_{ltb} = \sqrt{F_{ltb1}^2 + F_{ltb2}^2}$

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2.6.6 (p22)Allowable crippling stress in web q_{all}

$$F_{crp} = 0.75\, F_y \Big\} \begin{matrix} t \le 40 \text{ mm} \\ t > 40 \text{ mm} \end{matrix} \quad \rightarrow \text{From clause 1.4, get } F_y$$

In tension members we get smaller cross sections by using high tensile stresses St. 52. While in compression members we get smaller section if l/i is less than 100 but if l/i is more than 100 we get same section for all kinds of steel.

2.6.7 Combined stresses

In a continuous beam we have a state of combined shear and bending

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$$F_{1,2} = \frac{f}{2} \pm \sqrt{\left(\frac{f}{2}\right) + q^2} \leq f_{pt} \leq f_{pc}$$

This stress may be greater than the bending stress in the outside fibers.

The modern theory of equivalent structure is given by;

$$F_e = \sqrt{(f)^2 + 3q^2} \leq 1.10 \times f_{pt}$$

2.6.7.1- Axial Compression And Bending

$$\frac{f_{ca}}{F_c} + \frac{f_{bcx}}{F_{bcx}} \mathrm{A}_1 + \frac{f_{bcy}}{F_{bcy}} \mathrm{A}_2 \leq 1.0$$

$$When \ \frac{f_{ca}}{F_c} \prec 0.15 \qquad \rightarrow \quad \mathrm{A}_1 = \mathrm{A}_2 = 1.0$$

$$\mathrm{A}_1 = \frac{C_{mx}}{\left(1 - \frac{f_{ca}}{F_{EX}}\right)} \quad , \qquad \mathrm{A}_2 = \frac{C_{my}}{\left(1 - \frac{f_{ca}}{F_{EY}}\right)}$$

$$F_{EX} = \frac{7500}{\lambda^2} \quad , \quad \mathrm{F}_{\mathrm{EY}} = \frac{7500}{\lambda^2}$$

C_{mx} , C_{my} from code

2.6.7.2- Axial Tension And Bending

$$\frac{f_{ta}}{F_t} + \frac{f_{btx}}{F_{btx}} + \frac{f_{bty}}{F_{bty}} \leq 1.0$$