METALLIC BRIDGES STR403

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Previous Lecture

- Introduction.
- Plate Girder Components.
- Design Considerations:
  - Bucking due to shear
  - Buckling due to bending
  - Stress calculations.
- Inelastic behavior.
- Effect of imperfections.

This lecture

- Limiting Ratios of Depth-to-Width.
- Plate Girder Components.
- Effective width
- Actual Strength:
  - Pure bending
  - Pure shear
  - Combined shear and bending.
- Example.

LIMITING RATIO - COMPRESSION

The flange plate in a plate girder cross-section is essentially a uniformly compressed long narrow plate. The elastic buckling stress may be calculated using the appropriate value for the plate buckling factor k =0.425. Furthermore, to account for the reduction in buckling strength due to residual stresses and imperfections a reduced value of $\lambda = \lambda_o = 0.74$ is used. Substituting a value of k = 0.425 and $\lambda_o = 0.74$ gives:

LIMITING RATIO - COMPRESSION

$$0.74 = \sqrt{\frac{F_y}{F_{cr}}} = \left(\frac{b}{t}\right)\sqrt{\frac{F_y}{1898\,k}}$$

Which gives: $\left(\frac{b}{t}\right)_{lim} \leq 21 \,/\, \sqrt{F_y}$

= 11 for S,

= 15.5 for S.

Whenever the width–to–thickness ratio of the plate girder compression flange exceeds the a.m. limit, the flange is considered a “slender” element whose strength is affected by local buckling as explained in the next section.

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LIMITING RATIO - BENDING

The web plate in a plate girder cross-section is essentially subjected to a linearly varying normal stress due to bending. the elastic buckling stress may be calculated using the appropriate value for the plate buckling factor k = 25.9. Furthermore, to account for the reduction in buckling strength due to residual stresses and imperfections a reduced value of $\lambda = \lambda_o = 0.90$ is used. Substituting a value of k = 25.9 and $\lambda_o = 0.90$ in the Eqn. gives:

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LIMITING RATIO - BENDING

$$0.90 = \sqrt{\frac{F_y}{F_{cr}}} = \left(\frac{d}{t}\right)\sqrt{\frac{F_y}{1898 \times 23.9}} \quad \ldots\ldots\ldots$$

which gives: $\left(\frac{d}{t}\right)_{lim} \leq 190 \;/\; \sqrt{F_y} \quad \ldots\ldots\ldots\ldots\ldots$

$= 100$ for St. 52,
$= 122$ for St. 37

Whenever the width–to–thickness ratio of the plate girder web exceeds the a/m limit, the web is considered a "slender" element whose strength is affected by local buckling.

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LIMITING RATIO - SHEAR

The elastic buckling stress for a plate under pure shear may be calculated the Eqn. using the value for the plate buckling factor k. For a narrow long plate, $\alpha \gg 1$ which gives $k_q = 5.34$. Furthermore, to account for the reduction in buckling strength due to residual stresses and imperfections a reduced value of $\lambda = \lambda_o = 0.80$ is used.
Defining the plate slenderness parameter in shear λ_q as:

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LIMITING RATIO - BENDING

$$\lambda_q = \sqrt{\frac{F_y/\sqrt{3}}{q_{cr}}} = \sqrt{\frac{F_y/\sqrt{3}}{k_q\,(1898)\left(\frac{t}{d}\right)^2}}$$

Substituting a value of kq = 5.34 and λ_o = 0.80

$$\left(\frac{d}{t}\right)_{lim} \leq 105\ /\ \sqrt{F_y}\ \ ....$$

$= 55$ for St. 52,
$= 67$ for St. 37

Whenever the width–to–thickness ratio of unstiffened plate girder webs exceeds the a/m limit, the web is considered a “slender” element whose shear strength is affected by local buckling as explained in the next section.

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EFFECT OF LARGE DISPLACEMENT

For a compressed plate loaded to its ultimate load, the stress distribution remains uniform as the loading increases until the elastic buckling stress F_{cr} is reached. Unlike one dimensional structural members, such as columns, compressed plates will not collapse when the buckling stress is reached. Further increase in load beyond the elastic buckling load corresponding to the stress F_{cr} can be achieved before failure takes place.

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EFFECT OF LARGE DISPLACEMENT

However, the portion of the plate farthest from its side supports will deflect out of its original plane. This out-of-plane deflection violates assumption (5) of small displacements and causes the stress distribution to become non-uniform. The stresses redistribute to the stiffer edges and the redistribution becomes more extreme as buckling continues. The additional load carried thus by the plate beyond its elastic buckling stress F_{cr} is termed the “post-buckling” strength. Tests have shown that the post-buckling strength is high for large values of (b/t) and very small for low values of (b/t).

EFFECT OF LARGE DISPLACEMENT

In order to estimate the post-buckling strength, the non-uniform stress distribution can be replaced in design calculations by equivalent rectangular stress blocks over a reduced "effective width" b_e.

This equivalent uniform stress has the same peak stress and same action effect of the non-uniform stress distribution. The effective width of the element is computed from the condition that if the maximum stress is considered uniform over that width, the total section capacity will be the same.

EFFECT OF LARGE DISPLACEMENT

By applying this model, an "effective cross-section" is obtained from the original cross-section by deducting the ineffective areas where local buckling occurred. This design procedure is then the same used for sections not subjected to local buckling effect provided that the stresses are calculated using the effective section properties.

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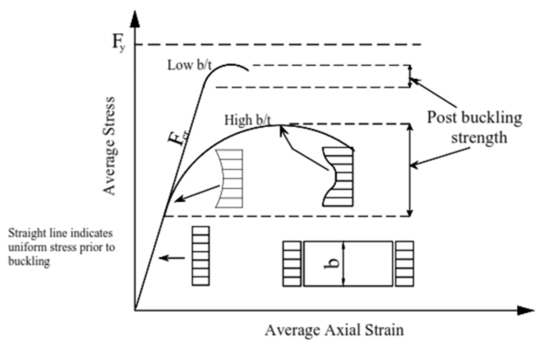
EFFECT OF LARGE DISPLACEMENT



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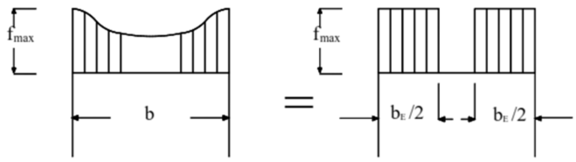
EFFECT OF LARGE DISPLACEMENT



(a) Actual Non-Uniform Stress (b) Equivalent Uniform Stress

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EFFECTIVE WIDTH

According to this procedure, the effective width can be expressed in terms of the plate slenderness λ_p defined as:

$$b_e = \rho\, b$$

where ρ = reduction factor = $(\lambda_p - 0.2) / \lambda_p^2$

For the general case where the plate is subjected to a linearly varying compression, e.g., due to bending, the reduction factor can be expressed in terms of the stress ratio ψ as:

$$\rho = (\lambda_p - 0.15 - 0.05 * \psi) / \lambda_p^2$$

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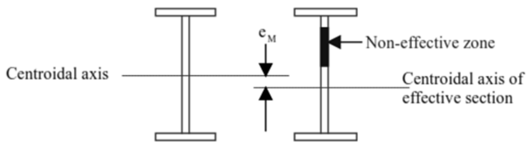
EFFECTIVE WIDTH

The following Tables give the effective width of compression elements for the case of stiffened elements, e.g., girder webs, and unstiffened elements, e.g., girder flange, respectively.



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EFFECTIVE WIDTH

For members in bending test results have shown that the effective widths may be determined on the basis of stress distributions calculated using the gross section modulus, Zx , even though the formation of "effective holes" in the compression parts will shift the neutral axis of the effective cross-section as shown in the next Fig. An iterative process is not, therefore, necessary to compute the effective section properties.

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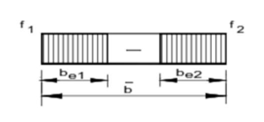
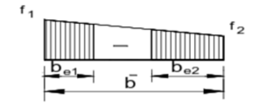
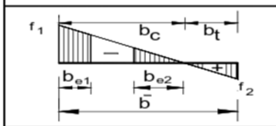
EFFECTIVE WIDTH

For $1 > \psi \geq -1$: $k_\sigma = \dfrac{16}{[(1+\psi)^2 + 0.112(1-\psi)^2]^{0.5} + (1+\psi)}$						$-1 > \psi > -2$
$\psi = f_2 / f_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	
Buckling Factor k_σ	4.0	$\dfrac{8.2}{1.05+\psi}$	7.81	$7.81 - 6.29\psi + 9.78\psi^2$	23.9	$5.98(1-\psi)^2$

Stress Distribution	Effective Width b_e for $\rho = (\bar{\lambda}_p - 0.15 - 0.05\psi) / \bar{\lambda}_p^2 \leq 1$
	$\psi = 1$: $b_e = \rho \bar{b}$ $b_{e1} = 0.5\, b_e$ $b_{e2} = 0.5\, b_e$
	$1 > \psi \geq 0$: $b_e = \rho \bar{b}$ $b_{e1} = 2\, b_e / (5-\psi)$ $b_{e2} = b_e - b_{e1}$
	$\psi < 0$: $b_e = \rho b_c = \rho \bar{b} / (1-\psi)$ $b_{e1} = 0.4\, b_e$ $b_{e2} = 0.6\, b_e$

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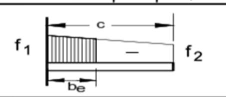
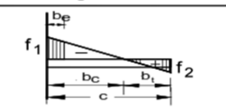
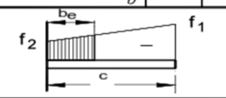
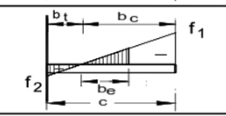
EFFECTIVE WIDTH

Stress Distribution	Effective Width b_e for $\rho = (\bar{\lambda}_p - 0.15 - 0.05\psi) / \bar{\lambda}_p^2 \leq 1$

$\psi = f_2 / f_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0.43	$\dfrac{0.578}{\psi + 0.34}$	1.70	$1.7 - 5\psi + 17.1\psi^2$	23.8

	$1 > \psi > 0$: $b_e = \rho c$
	$\psi < 0$ $b_e = \rho b_c = \rho c / (1-\psi)$

$\psi = f_2 / f_1$	1	0	-1	$1 > \psi > -1$
Buckling factor k_σ	0.43	0.57	0.85	$0.57 - 0.21\psi + 0.07\psi^2$

	$1 > \psi > 0$: $b_e = \rho c$
	$\psi < 0$: $b_e = \rho b_c = \rho c / (1-\psi)$

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EFFECTIVE WIDTH

In determining the effective width of compression elements in a given cross-section, the following assumptions can be made:

1. To determine the effective width of flange plate, the stress ratio ψ may be based on the properties of the gross cross -section.
2. To determine the effective width of the web plate, the stress ratio ψ may be obtained using the effective area of the compression flange but the gross area of the web.

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EFFECTIVE WIDTH

3. Generally the centroidal axis of the effective cross section will shift by a distance, e, measured from the centroidal axis of the gross cross section. This eccentricity should be considered when calculating the properties of the effective cross-section.
4. When the cross section is subjected to an axial force, N, the stress calculations shall take into account the additional moment $\Delta M = N * e_N$, where e_N = eccentricity of the centroidal axis when the effective cross section is subjected to uniform compression.

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EFFECTIVE WIDTH

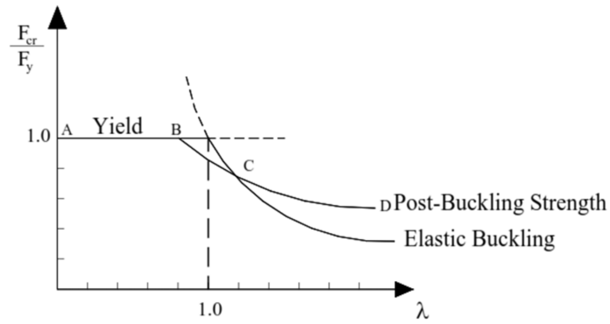
The actual plate girder strength is therefore represented by:

1. 1- For plates with low values of (b/t); i.e., $\lambda < \lambda_o$, the strength is computed directly from the yield strength divided by the appropriate safety factor.
2. For plates having higher values of (b/t), $\lambda > \lambda_o$, the strength is computed from the yield strength or the elastic buckling strength by applying the effective width concept to account for the stress reduction due to residual stresses and imperfections and the stress increase due to post-buckling.

EFFECTIVE WIDTH

The shown Fig. summarizes the strength of actual plates of varying slenderness. It shows the reduction in strength due to residual stress and imperfections for intermediate slender plates, region BC, and the increase due to post-buckling strength for slender plates, region CD .

EFFECTIVE WIDTH



ACTUAL STRENGTH

It has been shown in the preceding section that the strength of plates is affected by local buckling when the plate slenderness ratio exceeds a limiting value. These limiting values are:

i) For flange plate under uniform compression: $\left(\frac{b}{t}\right)_{lim} \leq 21 \;/\; \sqrt{F_y}$

ii) For web plate under pure bending: $\left(\frac{d}{t}\right)_{lim} \leq 190 \;/\; \sqrt{F_y}$

iii) For web plate under pure shear: $\left(\frac{d}{t}\right)_{lim} \leq 105 \;/\; \sqrt{F_y}$

ACTUAL STRENGTH

Whenever the width–to–thickness ratio of the girder web plate or flange plate exceeds the a.m. limit, the plate is considered a "slender" element whose strength is affected by local buckling. This effect is considered in the design of plate girder sections as follows:

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PURE BENDING

Plate girders subjected to the action of bending moment should be designed using the section modulus determined for the effective cross-sections as shown the earlier Tables. This means that the bending stress computed from the familiar bending formula $f_b = M_x / Z_{eff}$ should not exceed the allowable bending stress value:

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PURE BENDING

a) For the compression flange: the allowable bending stress is equal to 0.58 F_y if the flange is laterally supported otherwise lateral torsional buckling governs the design.

b) For the tension flange: Two checks have to be made:

   i. the maximum tensile stress should not exceed 0.58 F_y

   ii. the maximum stress range due to live load application should not exceed the allowable fatigue stress range.

PURE BENDING

According to ECP 2001; for plate girders without longitudinal stiffeners:

a- The web plate thickness of plate girders without longitudinal stiffeners (with or without transverse stiffeners) shall not be less than that detemined from:

$$t_w \geq d \sqrt{f_{bc}}/145 > d/120$$

b- Where the calculated compressive stress f_{bc} equals the allowable bending stress F_{bc}, the thickness of the web plate shall not be less than:

$$t_w \geq d \sqrt{f_{bc}}/190$$

PURE BENDING

No longitudinal stiffener

Grade of steel	Plate thickness (mm)	
	$t_w \leq 16$	$16 < t_w \leq 40$
S235	d/120	d/130
S275	d/110	d/120
S355	d/100	d/105

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PURE BENDING

If the assumed web thickness is not sufficient to resist buckling due to bending, the section strength can be increased by providing a thicker web. In plate girders with practical proportions, the flanges carry most of the applied bending moment, ~ 85 %, while the web carries all the shear force and a small part of the moment, ~ 15 %. Therefore, increasing the web thickness to resist bend-buckling is not effective.

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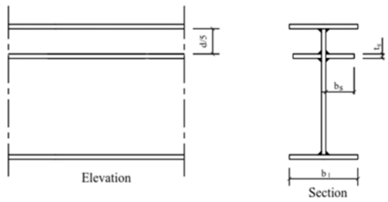
PURE BENDING

A more economic solution is usually achieved by limiting the web plate thickness to the minimum value required to resist the applied shear force. If this thickness is not sufficient for bend-buckling, the plate buckling strength is increased by providing the web plate with longitudinal stiffeners as shown



PURE BENDING

A longitudinal stiffener essentially forces the web to buckle in a higher mode by forming a nodal line in the buckled configuration, with waves much shorter than those of the longitudinally unstiffened plate. Analytically, the stiffener subdivides the plate into smaller sub-panels, thus increasing considerably the stress at which the plate will buckle.

PURE BENDING

Theoretical and experimental studies have shown that the optimum location of one longitudinal stiffener is at 0.2d from the compression flange. The presence of this stiffener increases the plate buckling coefficient to 42.5 as compared to 23.9 for a longitudinally unstiffened web, i.e., about 280 % increase in the elastic buckling stress.

PURE BENDING

The corresponding slenderness limit for this case becomes:

$$(d/t)_{lim} \leq 320/\sqrt{F_y}$$

According to ECP 2001; for Girders Stiffened Longitudinally:

a- The web plate thickness of plate girders with longitudinal stiffeners (with or without transverse stiffeners), placed at d/5 to d/4 from compression flange, shall not be less than that determined from:

$$t_w \geq d\,\sqrt{F_{bc}}\,/240 > d/240$$

PURE BENDING

b- Where the calculated compressive stress fbc equals the allowable bending stress F_{bc}, the thickness of the web plate shall not be less than:

$$tw \geq d \sqrt{F_{bc}} / 320$$

Grade of steel	Plate thickness (mm)	
	$t_w \leq 16$	$16 < t_w \leq 40$
S235	d/206	d/218
S275	d/191	d/200
S355	d/168	d/175

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PURE SHEAR

The effect of residual stresses and imperfections on the shear buckling stress of plate girder webs is treated in a different manner. Instead of considering an effective section for the buckled plate, the critical buckling stress in shear as calculated is divided by a suitable factor of safety to give the allowable buckling shear stress. This stress is empirically modified to allow for residual stresses and imperfections. For plate girders with practical proportions, an economic solution can be obtained in most cases by using a thin web stiffened transversally by stiffeners

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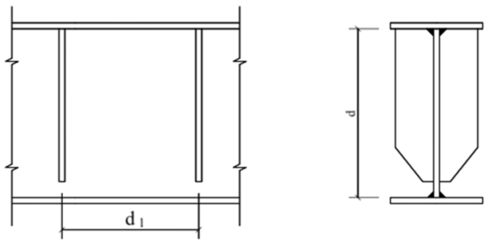
PURE SHEAR



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PURE SHEAR

Post Buckling Stress in Shear: For transversely stiffened girders where the transverse stiffener spacing lies within the range 1 < a/d < 3, full account may be taken of the considerable reserve of post-buckling resistance. This reserve arises from the development of "tension field action" within the girder.

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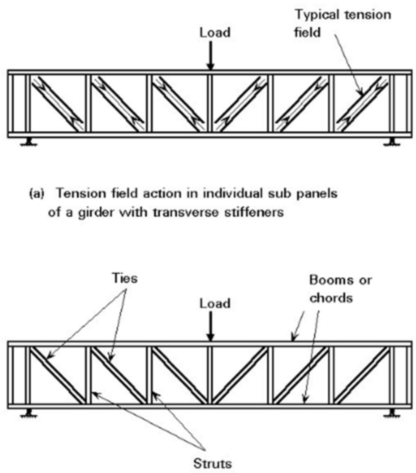
PURE SHEAR

The development of tension field action in the individual web panels of a typical girder is shown in the Fig. Once a web panel has buckled in shear, it loses its resistance to carry additional compressive stresses. In this post-buckling range, a new load-carrying mechanism is developed, whereby any additional shear load is carried by an inclined tensile membrane stress field.

PURE SHEAR



(a) Tension field action in individual sub panels of a girder with transverse stiffeners

(b) Typical N-truss for comparison

PURE SHEAR

1. thick ($\lambda_q < 0.8$, region AB) in which case the web will not buckle and the shear stress at failure will reach the shear yield stress of the web material:

$$q_b = 0.35 * F_y$$

2. intermediate ($0.8 < \lambda_q < 1.2$, region BC) which represents a transition stage from yielding to buckling action with the shear strength being evaluated empirically from the following:

$$q_b = (1.5 - 0,625\ \lambda_q)\ (0.35*F_y)$$

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PURE SHEAR

3. slender or thin ($\lambda_q > 1.2$, region CD) in which case the web will buckle before it yields and a certain amount of post-buckling action is taken into account empirically:

$$q_b = (0.9 / \lambda_q)\ (0.35*F_y)$$

In all cases the calculated shear stress q_{act} should not exceed the allowable buckling shear stress q_b.

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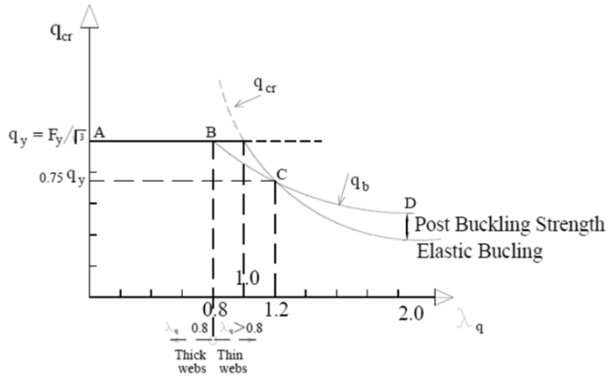
PURE SHEAR



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PURE SHEAR

Web plate without transversal stiffeners:
The web plate of a typically unstiffened plate girder has a large aspect ratio α. For such a case, the allowable buckling shear stress qb is obtained from the Eqn. 5.21 using a value of k_q = 5.34 as:

For (d/t) <159/F_y:

$q_b = [1.5 – (d/t) \sqrt{F_y}/ 212] [0.35 F_y] < 0.35F_y$

For (d/t) > 159/F_y:

$q_b = \{119 / [(d/t) \sqrt{F_y}] \} \{0.35 F_y\}$

These equations may require relatively thick webs (uneconomic design).

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PURE SHEAR

Effect of Longitudinal Stiffeners on Shear Buckling:

Both shear and bending strengths of a plate girder are increased by the presence of a longitudinal stiffener. Its location is, therefore, a key factor that affects both. Theoretical and experimental studies have shown that the optimum location of one longitudinal stiffener is at 0.2d from the compression flange for bending and 0.5d for shear.

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PURE SHEAR

The criteria for location of the stiffeners are based on elastic buckling considerations. The longitudinal stiffener may be more effective in contributing to the ultimate strength of the plate girder under combined bending and shear if placed somewhere between 0.2d and 0.5d from the compression edge of the web.

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PURE SHEAR

In bridge design practices, 0.2d has been adopted as the standard location for a longitudinal stiffener. Theoretical and experimental studies have shown that the contribution of the longitudinal stiffener placed at 0.2d to the shear buckling stress is relatively small and is usually neglected

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INTERACTION BETWEEN SHEAR AND BENDING

In general, any cross-section of a plate girder will be subjected to bending moment in addition to shear. This combination makes the stress conditions in the girder web considerably more complex. The stresses from the bending moment will combine with the shear stresses to give a lower buckling load.

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INTERACTION BETWEEN SHEAR AND BENDING

The interaction between shear and bending can be conveniently represented by the following diagram, where the allowable bending stress is plotted on the vertical axis and the allowable buckling shear stress of the girder is plotted horizontally. The interaction represents a failure envelope, with any point lying on the curve defining the co-existent values of shear and bending that the girder can just sustain.

INTERACTION BETWEEN SHEAR AND BENDING

The equation representing this interaction diagram is:

$$F_b = [\,0.8 - 0.36\,(q_{act} / q_b)]\,F_y$$

The interaction diagram can be considered in 3 regions. In region AB, the applied shear stress q_{act} is low (< 0.6 q_b) and the girder can sustain the full bending stress F_b based on the effective width b_{eff} for the compression flange.

INTERACTION BETWEEN SHEAR AND BENDING

At the other extreme of the interaction diagram in region CD, the applied shear stress is high (= q_b) then the allowable bending stress is reduced to 0.44 F_y to allow for the high shear. In the intermediate region BC the allowable bending stress is reduced linearly from 0.58 F_y to 0.44 F_y.

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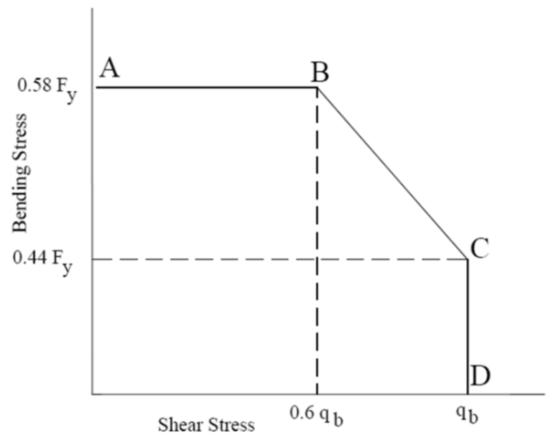
INTERACTION BETWEEN SHEAR AND BENDING



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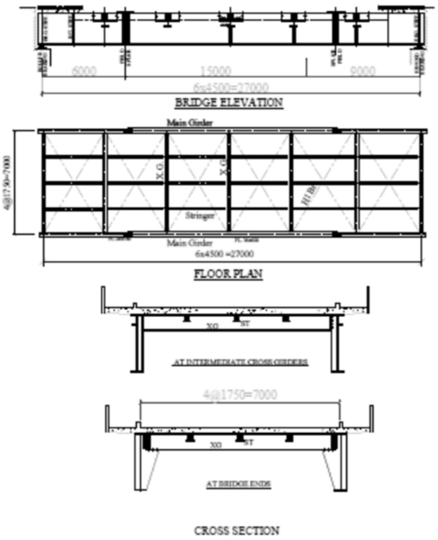
EXAMPLE

Consider a two-lane plate girder roadway bridge. The span measures 27 m between the centers of bearings. The bridge cross section provides for a clear roadway having two 3-m-wide traffic lanes in addition to 1.00 m wide median and two 1.5 m side walks. The bridge is to be designed according to the Egyptian Code of Practice ECP2001 using steel grade S355.

EXAMPLE

An elevation, plan, and a cross-section of the bridge is shown in Fig. 5.55. The bridge deck consists of a 22 cm reinforced concrete slab covered by an 8 cm asphalt wearing surface. The deck is carried by four main girders spaced at 1.75 meters center-to-center.

EXAMPLE



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EXAMPLE

The straining actions on an intermediate main girder due to dead loads and live loads plus impact at the critical sections are shown in the following table:

Action / Load Case	At Support		6 m from support		Mid section	
	Q (t)	M (m.t.)	Q (t)	M (m.t.)	Q (t)	M (m.t.)
Dead Load DL1	62	0	35	250	0	385
Add. Dead Load DL2	18	0	10	75	0	115
Live Load LL+I	100	0	60	460	25	700
Sum	180	0	105	785	25	1200

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EXAMPLE

Main Girder Design

First: Girder depth, d:

An estimate of the girder depth is obtained from Eq. 5.12 as:

$$d = (0.25\sim0.30)\ (M/F_b)^{1/3}$$

Assuming $F_b < 0.58\ Fy \cong (0.58 \times 3.6) = 2.1\ t/cm^2$

the required girder depth is

$d = (0.25\sim0.30)\ (1200/2)^{1/3} = 2.1 – 2.5\ m$

Use d = 2.25 m

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EXAMPLE

Second: Web thickness, t_w

Transverse stiffeners may be omitted if the actual shear stress does not exceed the allowable shear stress given by Eqns. 5.40 , 5.41. Usually $d/t > 1.59 \sqrt{F_y}$ gives: $q_b = (\ 119 / (d/t) \sqrt{F_y}\ )\ (0.35\ F_y\ )$

With the actual shear stress given by $q_{act} = Q / (d * t)$, the minimum thickness for a web without transverse stiffeners is obtained from: $t^2 = Q / (41.65 \sqrt{F_y}\ )$

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EXAMPLE

Substituting Q = 180 at support gives:
$t^2 = 180 / 41.65 \sqrt{3.6}\ ) = 2.27$
i.e., t = 1.51 cm.
Either use t = 16 mm (next even integer) without transverse stiffeners, usually an uneconomic solution, or a smaller value t = 14 mm with transverse stiffeners.
Stiffener spacing d_1 is controlled by:
a) cross girders spacing = 4.50 m
b) aspect ratio $\alpha = d_1/d = \sim 1$
A suitable value for d_1 would be 2.25 m

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EXAMPLE

Third: Check of web buckling due to shear:
Using transversal stiffeners at a distance d_1 = 2.25 m, then Aspect ratio**:** $\alpha = d_1 / d$ = 225/225 = 1 Buckling Coefficient K_q = $4 + 5.34 / \alpha^2 = 9.34$
Plate Slenderness:

$$\lambda_q = \sqrt{\frac{F_y/\sqrt{3}}{K_q\,(1898)\left(\frac{t}{d}\right)^2}} = \frac{d/t}{57.34}\sqrt{\frac{F_y}{K_q}}$$

$$= \frac{225/1.4}{57.34}\sqrt{\frac{3.6}{9.34}} = 1.74 > 1.2$$

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EXAMPLE

Buckling Shear Stress:
q_b = 0.9/1.74 x (0.35x3.6) = 0.652 t/cm^2
Actual Shear Stress:
q_{act} = 180/225/1.4 = 0.571 t/cm^2 < q_b OK
i.e., Web Plate is safe against buckling due to shear at support.
Since shear decreases away from support, the location where the transverse stiffeners are not needed can be found from the unstiffened web equation:
t2= Q / (41.65 $\sqrt{F_y}$)

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EXAMPLE

Substituting t = 1.4 cm gives Q = 154 t. This value is located at ~ 2 m from the support so that the transverse stiffener is only needed between the support and the first cross girder. Note that transverse stiffeners are always used at cross girder locations where the concentrated reaction of the cross girder is transmitted to the main girder.

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EXAMPLE

Fourth: Flange Plate

$A_f = (M / (F_b\ d)) – A_w / 6 =$
$1200/2/2.25 – 225x1.4/6 = 214.167\ cm^2$.
Assume flange width $b_f = (0.2 \sim 0.3)\ d =$ $(48 \sim 72)$ cm Use $b_f = 60$ cm and calculate the required flange thickness as: $t_f = 214.167/ 60 = 3.56$ cm
Provide two 600 × 36 mm flanges,
Check the b/t ratio for compression flange local buckling for S335:
$b_f / 2t_f < 21 / \sqrt{F_y} = 11$

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EXAMPLE

Actual $b_f / 2t_f = 600 / (2\times 36) = 8.333 < 11$
O.K. for non-compact flange.
Check d/t ratio for web buckling due to bending S335:

$$d / t < 190 / \sqrt{F_y} = 100$$

actual $d / t = 225 / 1.4 = 160.714 > 100$ i.e., web is slender It is therefore necessary to use longitudinal stiffeners to prevent web buckling due to bending. First longitudinal stiffener at $d/5 = 225/5 = 45$ cm from compression flange (top).

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EXAMPLE

Note:
No need for another longitudinal stiffener at d/2 since d/t = 160.74 < 320 / $\sqrt{F_y}$ = 168.65.

EXAMPLE

Fifth: Check of Bending Stresses:
Inertia I_x =1.4 $(225)^3/12+2*(60\times3.6)(114.3)^2$ = 6973232 cm^4
Modulus Zx = 6973232 / 116.1 = 60062 cm^3
Actual bending stress fb = M_{max} / Z_x =
1200 × 100 / 60062 = 1.998 t/cm^2
a) Check of Bending Compression:
Since the girder compression flange is supported laterally by deck slab, the allowable bending stress in compression F_b = 0.583 F_y = 2.10 t/cm^2. Girder is safe in bending compression.

EXAMPLE

The lateral stability of the girder during erection (before the deck slab hardens) should be also checked. For this case:

Dead load bending stress f_{DL} = 385*100/ 60062 = 0.641 t/cm^2

The allowable lateral torsional buckling stress is computed as:

L_u /r_t = 2700/15.5 = 177.42

F_{ltb} = 12000 / (177.42)2= 0.381 t/cm^2

Since f_{DL} > F_{ltb} then the girder must be supported laterally during erection using upper wind bracings.

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