METALLIC BRIDGES
STR403

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Lecture 5 – 9 March 2020

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Previous lectures – Plate Girder

- Plate Girder Components.
- Design Considerations.
- Imperfections and inelastic behavior.
- Effective width
- Actual Strength:
  - Shear, bending, and combined.
- Example.

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This Lecture

- Curtailment.
- Flange-to-web welds.
- Stiffeners:
  - Horizontal.
  - Vertical.
  - Bearing.

CURTAILMENT

Welded girders offer more flexibility than design with rolled sections. Since the total design moment varies along the girder span, flange plates of varying thicknesses, and sometimes of varying widths, may be butt welded to provide a section strength that closely approximates the variation in bending moment. Theoretical locations at which flange-plate thickness or width may be changed along the girder length can be determined as follows:

CURTAILMENT

1. The resisting moments of the girder with several selected flange plate areas are calculated.
2. The above values of the resisting moments are super-imposed on the graph of the total design moment. This plot is then used to determine the required length of each size flange plate.

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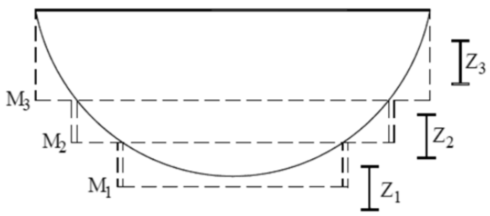
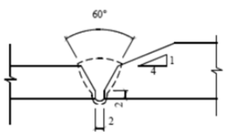
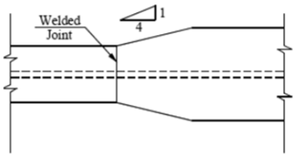
CURTAILMENT



(a) Moment of Resistant Diagram



(b) Transition in Thickness



(c) Transition in Width

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CURTAILMENT

The actual changes in flange plate thickness or width are made near theoretical locations. Although a minimum steel weight results from such changes, an excessive number of changes should be avoided since the cost of making and testing the necessary butt welds increases the over-all cost of the fabricated girder. For a simple span, the flange is usually made from three plates of two sizes; a center plate covering 40 - 60 % of the span, and two plates butt-welded to the center plate.

CURTAILMENT

When flange plates of different thicknesses are butt-welded, design codes require a uniform transition slope between the offset surfaces not exceeding 1 in 4.
If plates of different widths are joined, the wider plate must taper into the narrower plate with the same slope or with a radius of 60 cm.

FLANGE-TO-WEB CONNECTION

The connection between the flange plate and the web plate is usually executed using fillet welds on both sides of the web plate. This weld should be designed to transmit the horizontal shear flow between web and flange plate at any point along the girder plus any load applied directly to the flange.

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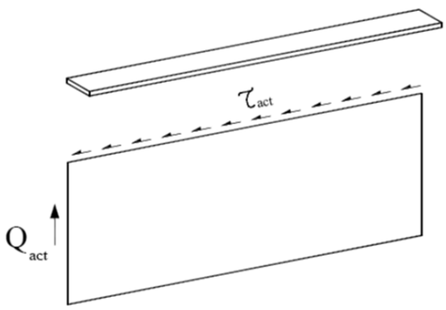
FLANGE-TO-WEB CONNECTION

Shear Effect:

The effect of horizontal shear flow between the web and the flange can be considered as follows



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FLANGE-TO-WEB CONNECTION

Horizontal shear/unit length = shear flow
$\tau_{act} = Q_{act}\ S_f\ /\ I$........................ **(5.43)**
Where
Q_{act} = shear force,
S_f = first moment of area of flange about neutral axis,
I = moment of inertia about neutral axis.

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FLANGE-TO-WEB CONNECTION

If the allowable shear stress in welds is q_w, then the weld size s can be calculated from the equation:
weld strength = $q_w * (2\ s) \geq \tau_{act}$ ... **(5.44)**
i.e., weld size $s \geq \tau_{act} / 2\ q_w$ .... **(5.45)**

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FLANGE-TO-WEB CONNECTION

Effect of Direct Loading:
In deck bridges where the wheel loads are transmitted to the girder web through the direct contact between the girder flange and the web, the flange-to-web weld is also subjected to a vertical load in addition to the horizontal shear stress.

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FLANGE-TO-WEB CONNECTION

The direct load in railroad deck bridges, where sleepers rest on the top flange, is taken as the train wheel load (12.5 ton) plus impact distributed over one meter. For flanges carrying ballasted decks, the train wheel load may be assumed distributed over 1.5 meter. In roadway deck bridges, the truck wheel load (10 ton) plus impact is distributed over a length of 1 meter.

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FLANGE-TO-WEB CONNECTION

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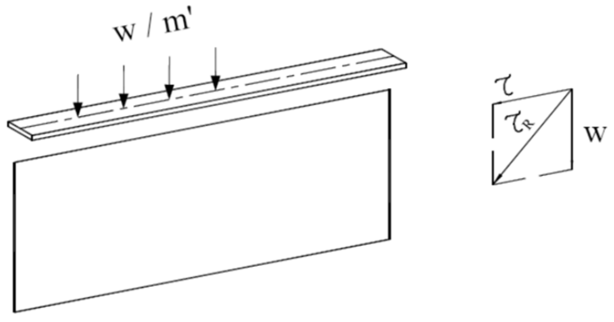
FLANGE-TO-WEB CONNECTION



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FLANGE-TO-WEB CONNECTION

The effect of these external loads should be superimposed on the a.m. shear stresses. If the external direct load per unit length of flange is w, the resultant shear on the weld shall be:

$\tau_R = \sqrt{(\tau^2 + W^2)}$............... (5.46)

and the weld size is computed from**:**

$s > \tau_R$ / 2 q_w.......................... (**5.47**)

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FLANGE-TO-WEB CONNECTION

The calculated weld size (**s**) should satisfy the following requirements:

1. The maximum size of fillet weld should not exceed the thickness of the thinner plate to be welded.
2. The minimum size of fillet welds as related to the thickness of the thicker part to be joined is shown in the following table:

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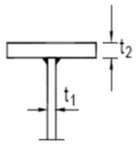
FLANGE-TO-WEB CONNECTION

t (max. of t_1 or t_2) (mm)	Size s (mm)
≤ 10	≥ 4
10-20	≥ 5
20 - 30	≥ 6
30-50	≥ 8
50-100	≥ 10



COVER PLATES

For economic design, the cross-section of the main girder is usually changed along the bridge length according to the structural requirements. A flange may comprise a series of plates joined end-to-end by full penetration welds. Three schemes can be used to accomplish changes in the flange plate areas:

a) varying the thickness of the flange plates.
b) varying the width of the flange plates.
c) adding cover plates at regions of high moment.

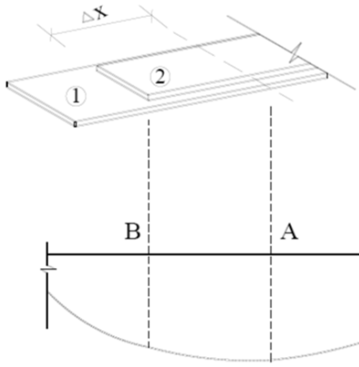
COVER PLATES

Proper connection in the region of cover plate cut-off presents a some what special case of the previous procedure. Welds connecting a cover plate to a flange should be continuous and capable of transmitting the horizontal shear between the cover plate and the flange. The “theoretical end” of the cover plate is the section at which the stress in the flange without that plate equals the allowable stress.

COVER PLATES

The “terminal distance” is the extension of the cover plate beyond the theoretical end. Welds connecting the cover plate to the flange within the terminal distance should be of sufficient size to develop the computed stress in the cover plate at its theoretical end. This distance can be calculated as follows;

COVER PLATES



COVER PLATES

Let point A be the theoretical end of the cover plate A2 with a girder having a continuous flange A1. The size of weld connecting the cover plate to the flange plate can be computed from shear flow considerations as:

Horizontal shear / unit length = $q_c = Q \times S_c / I$

where S_c = first moment of area of cover plate about neutral axis.

Weld strength = $q_w \times (2\ s) > q_c$

i.e., weld size = $s > q_c / 2\ q_w$.

COVER PLATES

Let Δx be the terminal distance of the cover plate extending from point A to point B. The shear force between the cover plate and the flange is equal to the resultant force in the cover plate, i.e.,

$$\Delta P_2 = P_2 = f_A * A_2 \le q_w * s * (2\Delta x) \ldots$$

$$\Delta x = \frac{f_A \, A_2}{2\, s\, q_w} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$$

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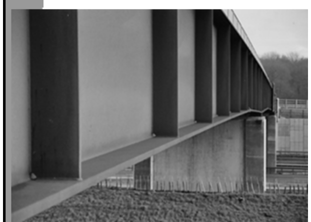
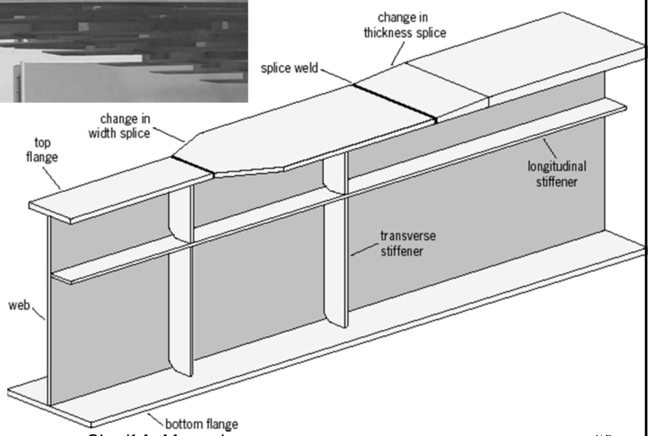
STIFFENERS





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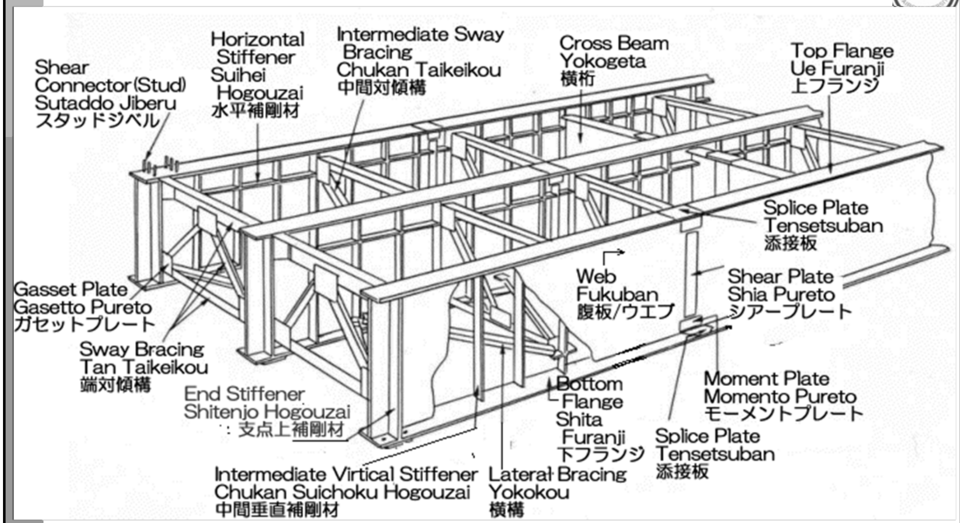
STIFFENERS



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STIFFENERS

<u>Main Function:</u> Increase buckling strength of plate girder webs.

- ✓Horizontal (Longitudinal) Stiffeners increase bend-buckling strength.
- ✓Vertical (Transversal) Stiffeners increase shear-buckling strength.
- ✓Bearing Stiffeners

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HORIZONTAL STIFFENERS

Steel Bridges

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: $\left(\frac{d}{t}\right)_{lim} \leq 190 / \sqrt{F_y}$

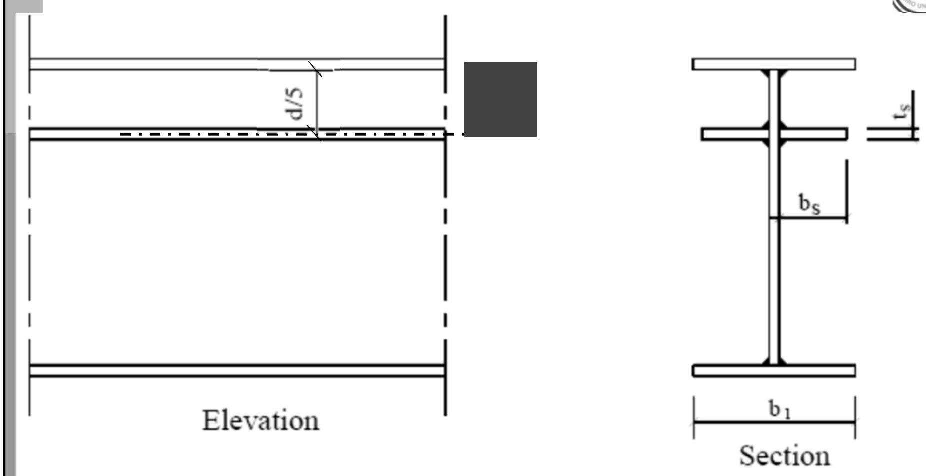
iii) For web plate under pure shear: $\left(\frac{d}{t}\right)_{lim} \leq 105 / \sqrt{F_y}$

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:

HORIZONTAL STIFFENERS

IF, (**(d/t) > 190**/√$\mathbf{F_y}$), a longitudinal stiffener should be attached to the web at a distance d/5 from the inner surface of the compression flange measured to the center of the stiffener when it is a plate or to the gage line when it is an angle.

HORIZONTAL STIFFENERS



HORIZONTAL STIFFENERS

- Longitudinal stiffeners are usually placed on one side of the web.
- They need not be continuous, and they may be cut at their intersection with the transversal/vertical stiffeners when both are provided on the same side of the web.

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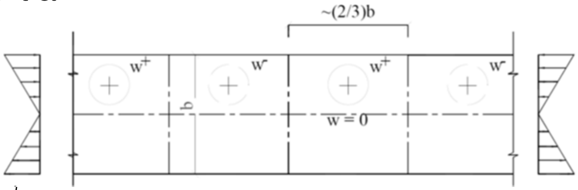
HOR. STIF. REQUIREMENTS

- A longitudinal stiffener should be sufficiently stiff to maintain a longitudinal node in the buckled web. For this reason, the stiffener should be proportioned so that it has the following minimum value of its inertia:

$\mathbf{I \geq 4\, d_w\, t^3_w}$



- If (**(d/t) > 320/√$\mathbf{F_y}$**) a second longitudinal stiffener is required, its inertia must exceed:

$\mathbf{I \geq d_w\, t^3_w}$

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HOR. STIF. REQUIREMENTS

- To avoid local buckling of the stiffener, it must meet the width-thickness limit of non-compact compression elements; i.e., $\mathbf{b_s/t_s \leq 21 /\sqrt{F_y}}$
- The computed bending stress in the stiffener should not exceed the allowable bending stress for the stiffener steel.

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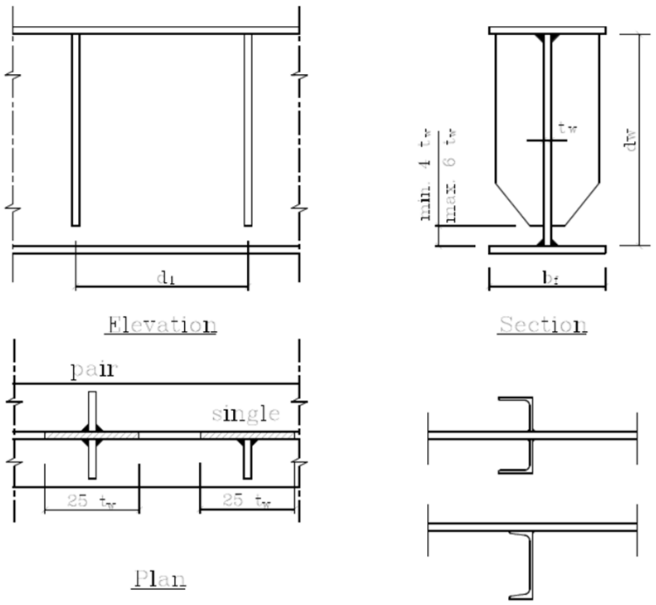
VERTICAL STIFFENER



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VERTICAL STIFFENER

- Transverse stiffeners should be used where dw / tw exceeds the value 105/ $\sqrt{F_y}$
- Transversal stiffeners are usually fabricated of plates welded to the girder web. They may be used in pairs (one stiffener welded on each side of the web) with a tight fit at the compression flange. When a concentrated load is applied on the plate girder flange, transverse stiffeners in pairs are required to prevent crippling in the web immediately adjacent to the concentrated load. These stiffeners are designed as bearing stiffeners.

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VERTICAL STIFFENER

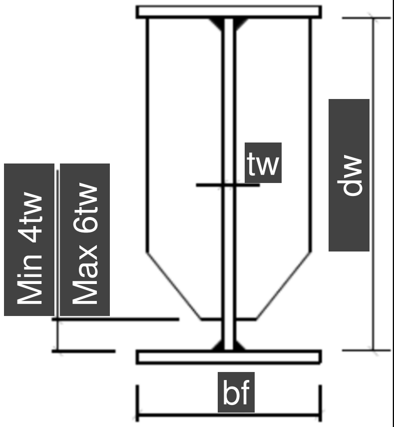
- Alternatively, transverse stiffeners may be made of single plates welded to only one side of the web plate. In this case they **must be in bearing against the compression flange (to prevent its twisting) but need not be attached to the compression flange to be effective**. When only single stiffeners are used, it is usual to place them on the inside face of the web for aesthetic reasons.

VERTICAL STIFFENER

- In some cases a stiffener may be **used as a connecting plate for a cross frame or a lateral support**, which could result in out-of-plane movement in the welded flange-to-web connection. In such cases, attachment of the stiffener to the compression flange may be necessary and the connection should be adequately designed to transmit the lateral force developed at the connection.

VERTICAL STIFFENER

- Transverse stiffeners need not be in bearing with the tension flange, but they should be terminated within a distance of **(4tw ~ 6tw)** from the tension flange. Transverse stiffeners should not be welded to the tension flange to avoid fatigue problems.



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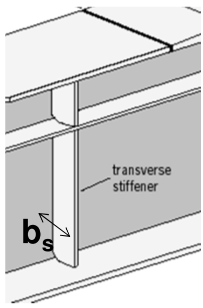
VERT. STIF. DESIGN

- Stiffeners should project a distance $\mathbf{b_s}$ from the web, where $\mathbf{b_s}$:

  a) at least $\mathbf{b_f}$ / **4**, where bf is the flange width, and

  b) at least **(d_w / 30 + 5) cm** for stiffeners on both sides of the web, or **(d_w/30 + 10) cm** for stiffeners on one side only, where d_w is the girder depth, cm.



- To avoid local buckling of the stiffener, it must meet the width-thickness limit of compression elements; i.e., $\mathbf{b_s/t_s \le 21 /\sqrt{Fy}}$

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VERT. STIF. DESIGN

- Intermediate transverse stiffeners should be designed to resist a force **Cs** equal to:

  $0.65(0.35\ F_y/\mathbf{q_b} - 1)\ Q_{act}$

- Part of the web equal to 25 times web thickness may be considered to act with the stiffener area in design of intermediate stiffener.
- Transverse stiffeners should be designed as a compression member (buckling length of $0.8d_w$).
- Connection between transverse stiffener and web should be designed on the stiffener design force such that weld in either upper or lower thirds of stiffeners transmits the design force.

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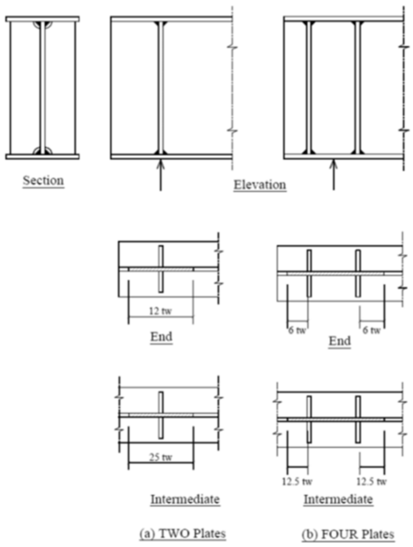
BEARING STIFFENERS



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BEARING STIFFENERS

- Bearing stiffeners are required where concentrated loads are to be transmitted to the web through flanges. Such locations are:
  - a) **End** and **Intermediate** supports of plate girders where the bottom flanges receive the reactions,
  - b) points of **Concentrated Loads** applied to the top girder flange.
- Their is to distribute reactions or concentrated loads into the web to create web shear. Additionally they prevent the possibility of local crippling and/or vertical buckling of the web.

BEARING STIFFENERS

- To effectively perform these functions, bearing stiffeners should be sufficiently stiff against buckling.
- Therefore, it is preferred to have bearing stiffeners consisting of plates provided in pairs (i.e., placed on both sides of the web), and their connection with the web should be designed to transmit the entire reaction to the bearings.

BEARING STIFFENERS

- They must bear firmly on the flanges (i.e., fit tightly against the loaded flanges) through which they receive the reaction (or the concentrated load), and extend as far possible to the outer edges of the flanges.
- The ends of bearing stiffeners must be milled to fit against the flange through which they receive their reactions.

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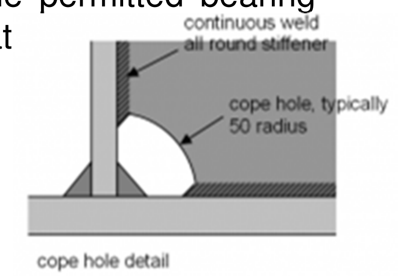
BEARING STIFFENERS

- To provide space for continuous fillet welds at the girder web-flange connection, the side corner on one edge of the stiffeners must be clipped to ensure tight fit against the flange. This results in a reduced contact area between the stiffener and the loaded flange. This reduced contact area of the stiffener should be adequate to transmit the reaction without exceeding the permitted bearing stress on either the flange mat material.



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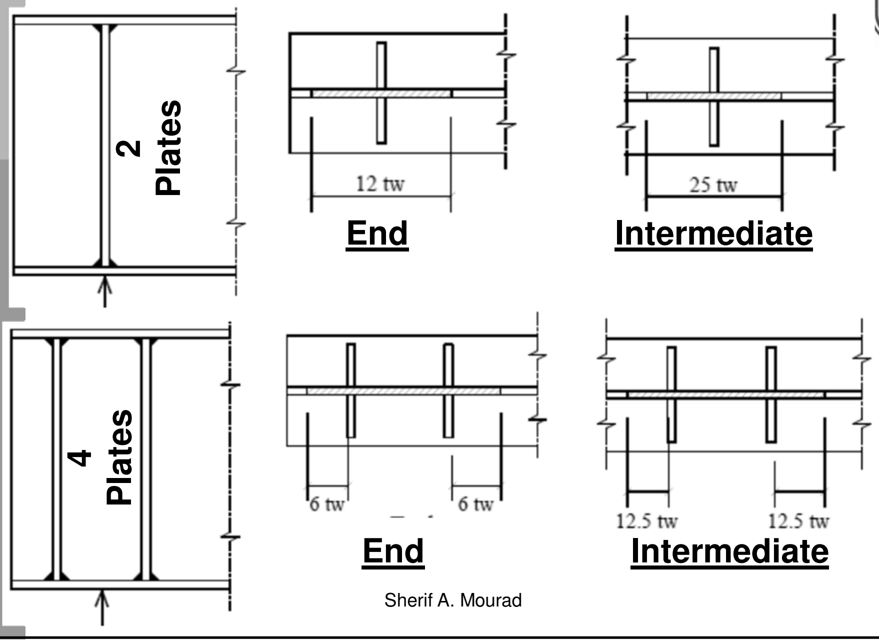
BEARING STIFFENER DESIGN

- Bearing stiffeners are designed as concentrically loaded columns. A portion of the web extending longitudinally on both sides of the bearing stiffeners is considered participating in carrying the reaction. Depending on the magnitude of the reaction to be transmitted, the design may require two (one on each side of the web) or four or more stiffeners (symmetrically placed about the web). The cross sectional area of the fictitious column is defined as follows:

BEARING STIFFENER DESIGN

- When two stiffener plates are provided, the column section consists of the two stiffener plates and a centrally loaded strip of the web equal to 12 t_w for bearing stiffeners at girder ends and 25 t_w for bearing stiffeners at interior supports
- If there are four or more stiffener plates, the column section consists of the areas of all stiffener plates and a centrally loaded strip of the web plate whose width is equal to that enclosed by the stiffener plates plus a width equal to 12 t_w for bearing stiffeners at girder ends and 25 t_w for bearing stiffeners at interior supports.

BEARING STIFFENER DESIGN



BEARING STIFFENER DESIGN

a) **Buckling Check:** The actual compressive stress in the fictitious column should not exceed the allowable buckling stress of the stiffener cross section considered to act as a column with a buckling length of 0.8 d_w. The radius of gyration of the section is computed about the axis through the center of the web.

b) **Compression Check:** The compressive stress in the stiffener plate alone should be less than the allowable stress in compression for the stiffener steel.

c) **Bearing Check:** The calculated stress on the actual contact area between the stiffener and the bottom flange should not exceed the allowable bearing stress. According to ECP: $F_{bearing} = 2\ F_t$, where F_t is the allowable tensile stress of the material.

Summary

- Curtailment.
- Flange-to-web welds.
- Stiffeners:
  - Horizontal.
  - Vertical.
  - Bearing.