

RESEARCH ARTICLE

Analysis and stiffeners' design of a steel bridge girder

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Abstract

Plate girders are designed to carry massive loads over large spans. Flanges resist moment and web resists shear forces. Shear strength of steel girders having slender webs is much less than the yielding shear capacity. It is mainly due to the buckling of the web prior to reaching the yield strength of the material. Webs are generally reinforced with transverse stiffeners to increase their buckling strength. Stiffened webs resist shear also after buckling, which is called as post buckling strength. Tension field theories explain the formation of the post buckling strength and predict the stiffened web's ultimate shear strength. Most design code provisions are set on tension field theories. There exists plenty of tension field theories proposed until today. This paper covers the design shear strength check and design flexural strength check and the stiffeners' design of a steel girder specimen which was designed intentionally to fail in shear buckling. Analysis and stiffeners' design were performed according to the provisions for load and resistance factor design (LRFD) in the ANSI/American Institute of Steel Construction (AISC) 360-16 - Specification for Structural Steel Buildings.

1. Introduction

Plate girders carry heavy loads over wide spans like floors and bridges. Plate girders are made up of a web welded to two flanges, and transverse stiffeners. Flanges stand moment and web stands shear forces. Different types of failure mechanisms, such as shear buckling of the web, lateral-torsional buckling of the girder, compression buckling of the web, flange-induced buckling of the web, local buckling and crippling of the web, local buckling of the flanges, and flexural failure of the flanges through plastic hinge formation are considered in design [1-3].

Shear strength of steel girders with slender webs is very low compared to the yielding shear capacity which typically controls the capacity of compact and non-compact webs of bridge girders. This situation is primarily because of the buckling of the web before the material reaches its yielding strength. Compact webs could reach the ultimate strength of the material without experiencing elastic buckling.

The elastic buckling of the web is directly related to the stress level induced in the web by the principal compressive forces at the high shear zone(s) of the girder. Reducing the stress level in the web at the same load level, could potentially delay the occurrence of the elastic buckling, thus enhancing the shear capacity of the girder. In order to increase the buckling strength of webs, they are frequently supported with transverse stiffeners and occasionally also with longitudinal stiffeners. Aim of utilizing transverse stiffeners is to ensure increase in shear buckling strength by decreasing panel aspect ratio.

Webs with transverse stiffeners resist shear also after buckling, and this extra strength is defined post buckling strength. Numerous studies have been performed to figure out development of post buckling strength. Truss or tension field theories clarify the mechanics of post buckling strength development and estimate the stiffened web panel's ultimate shear strength. Tension field action results from the relation between buckled web, flanges, and stiffeners. In general, tension field theory defines the mechanics of ultimate shear strength as combination of beam, tension field, and frame actions as demonstrated in Fig. 1. The main objective of the research was to perform analysis and transverse and the bearing stiffeners' design of the steel girder specimen intentionally designed to fail by shear buckling.

2. Development of tension field theories

Most of the design code provisions are based on tension field theories. There exists plenty of tension field theories proposed by now [4]. Various tension field theories for plate girders are listed in Table 1 [5].

Tension field action assumes that the post buckling shear strength develops by the tensile stresses in a diagonal field activated after elastic shear buckling [5]. Dimensions of diagonal tension fields are determined through placing of transverse stiffeners. Basic variances among different models are the supposed shape of tension diagonal in the stiffened web panel and the dispersion of web shear stresses to stiffeners and flanges after buckling [6].

Prior to consideration of post buckling strength in the design of plate girders, stiffeners were designed as having enough moment of inertia to form nodal lines during elastic buckling [7]. Timoshenko [8] built up one of the pioneering works in which he obtained flexural rigidity of transverse stiffener satisfying a linear boundary for shear web panel during elastic buckling.

Wilson [9] realized the first research on post buckling behavior of plate girders and described post buckling strength of webs with stiffeners in shear in 1886. In 1931, Wagner [10] developed uniform diagonal tension theory to identify strength of structures having very thin web panels and rigid flanges under pure shear.

Basler and Thurlimann [11] performed comprehensive work in 1950s on post buckling behavior of web panels in bending and in shear and developed an avant-garde model for plate girders. They theorized that flanges are very flexible to support a lateral load from tension field; hence, the yield band controls the shear strength [5]. In the model it was presumed that when web buckles, it loses vast majority of its compressive strength in the orthogonal direction. Thus, it was concluded that transverse stiffener should work as a compression strut between the chords [12]. Flanges show a tendency toward truss action [13]. It was supposed that flanges does not have weak axis flexural stiffness required to anchor tension forces in web diagonal [6].

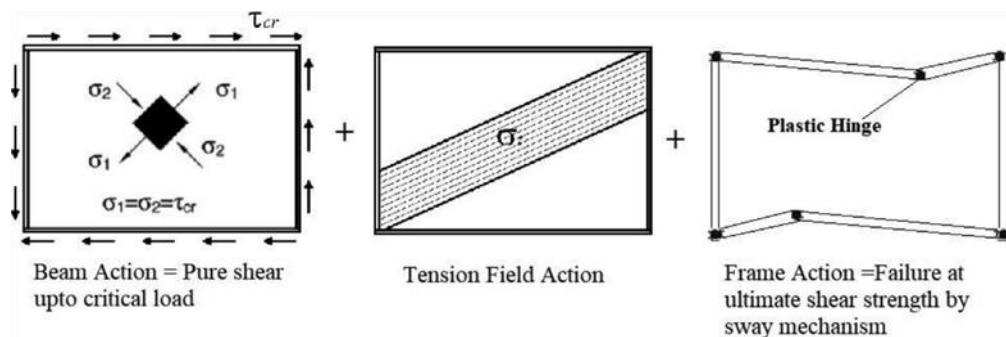


Fig. 1. Stages in tension field model [4]

Table 1. Different tension field theories for plate girders [5]

Investigator	Mechanism	Web-Buckling Edge Support	Unequal Flanges	Longitudinal Stiffener	Shear and Moment
Basler [12]			Immaterial	Yes	Yes
Takeuchi [16]			Yes	No	No
Fujii [17]			Yes	Yes	Yes
Komatsu [19]			No	Yes, at mid-depth	No
Chern and Ostapenko [18]			Yes	Yes	Yes
Porter et al. [28]			Yes	Yes	Yes
Hoglund [20, 21]			No	No	Yes
Herzog [27]		Web-buckling component neglected	Yes, in evaluating c	Yes	Yes
Sharp and Clark [25]			No	No	No
Steinhardt and Schröter [26]			Yes	Yes	Yes

Basler's [12] model for tension field action was adopted by AISC [14] for the first time and post buckling strength was utilized in plate girders' design. Additional cross-sectional area requirement was put to resist compressive axial force due to tension field action. Subsequently, AASHTO [15] also acknowledged the theory [7]. The model still serves as the basis of many design standards.

In 1964, Takeuchi [16] did the first study to make an allowance for the flange stiffness effect on the yield zone of the web. Fujii [17] proposed a tension field model in 1968 including the entire panel with beam mechanisms in each flange, interior hinge being at mid panel. The web compression orthogonal to the principal tension was assumed to be the same as the compression in that direction at the beginning of

buckling. Chern and Ostapenko [18] suggested a tension field in 1969 in which the principal band is defined by yielding, considering the stress at buckling [5].

Komatsu [19] developed formulas for four modes of failure in 1971. In the first mode, failure happens by yielding of the inner band under the joint action of the buckling stress and the post buckling tension field. In the second case, the interior hinge occurs at the mid panel and the web yields uniformly throughout the panel. In the third mode, the flanges are theorized to stay elastic while allowing fully yielding of the web. In the fourth mode, a Wagner [10] field develops together with a panel mechanism of the flanges [5].

From 1971 to 1973, Hoglund [20-22] conducted studies to develop a theory for plate girders with and without transverse stiffeners. In the theory, which later became a base for Eurocode 3 [23], the system of diagonal tension and compression bars are used in modelling the web [24].

In 1971, Sharp and Clark [25] recommended a tension field theory for aluminum girders with thin webs that contains a Basler field on which a complete tension field inclined at 45° is added. The flanges are thought to be elastic beams continuing over the stiffeners and subjected to a uniform load from the added 45° tension field. The shear strength is determined by summing up the vertical components of both tension fields and the buckling shear. Steinhardt and Schroter [26] proposed a tension field theory in 1971 for aluminum girders. The tension field band is in the panel diagonal direction and its boundaries cross the flanges' mid panel points. Herzog [27] performed a research in 1974 in which he took the limit of the tension field from the panel's mid height at the stiffeners to the flanges' plastic hinges [5].

In 1975, Porter et al. [28] developed the Cardiff model in which the tension field is composed of a single band. The researchers presumed that inclined tension fields are just present in a limited portion, but flanges participate to the post buckling resistance by absorbing normal stresses from tension fields. Consequently, failure occurs when plastic hinges occur in the flanges [24]. The model was also embraced into the British Standards [29].

The yield zone pattern hypothesis of each classical failure theory was unlike the other. However, the theories all agreed on the basic assumption that 'compressive stresses that develop in the direction perpendicular to the tension diagonal do not increase any further once elastic buckling has taken place'. Eventually, the well-known theory of 'the tension field action in plate girders with transverse stiffeners needs to be anchored by flanges and stiffeners in order for the webs to develop their full post buckling strength' arised [24].

In Fig. 2 the shear strength curve of AASHTO [30] for web panels in which post buckling strength can be utilized is given. There exists three zones in the curve as yield zone, inelastic buckling zone, and elastic buckling zone. The elastic buckling strength is greater than the shear yield strength in the yield zone. For that reason, there is no need for the stiffeners to supply nodal lines for elastic buckling.

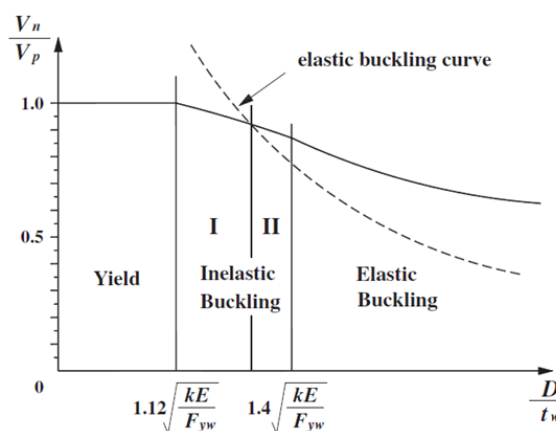


Fig. 2. AASHTO shear strength curve [30]

The stiffeners should only provide the web ability to develop shear yield strength. Hence, the flexural rigidity for this case is smaller than that for elastic buckling. Out of plane bending deformations also occur with post buckling of the web. This time greater flexural rigidity is required than that for elastic buckling in order that the stiffeners provide nodal lines and the web acquires its full post buckling strength. The inelastic buckling zone can be separated into two sub zones for deeper investigation of the necessary rigidity. The shear strength is less than the elastic buckling strength in the first zone. Whereas in the second zone, the shear strength is more because of post buckling, though not so much as in the elastic buckling case [7].

3. Steel girder properties

3.1. Girder geometry

The steel girder comprised 914x5 mm (36x3/16 in.) web and 381x13 mm (15x1/2 in.) flanges. The girder consisted of 7 shear panels 914 mm (3 ft) long. Schematic geometry of the specially designed steel girder to fail in shear buckling is presented in Fig. 3.

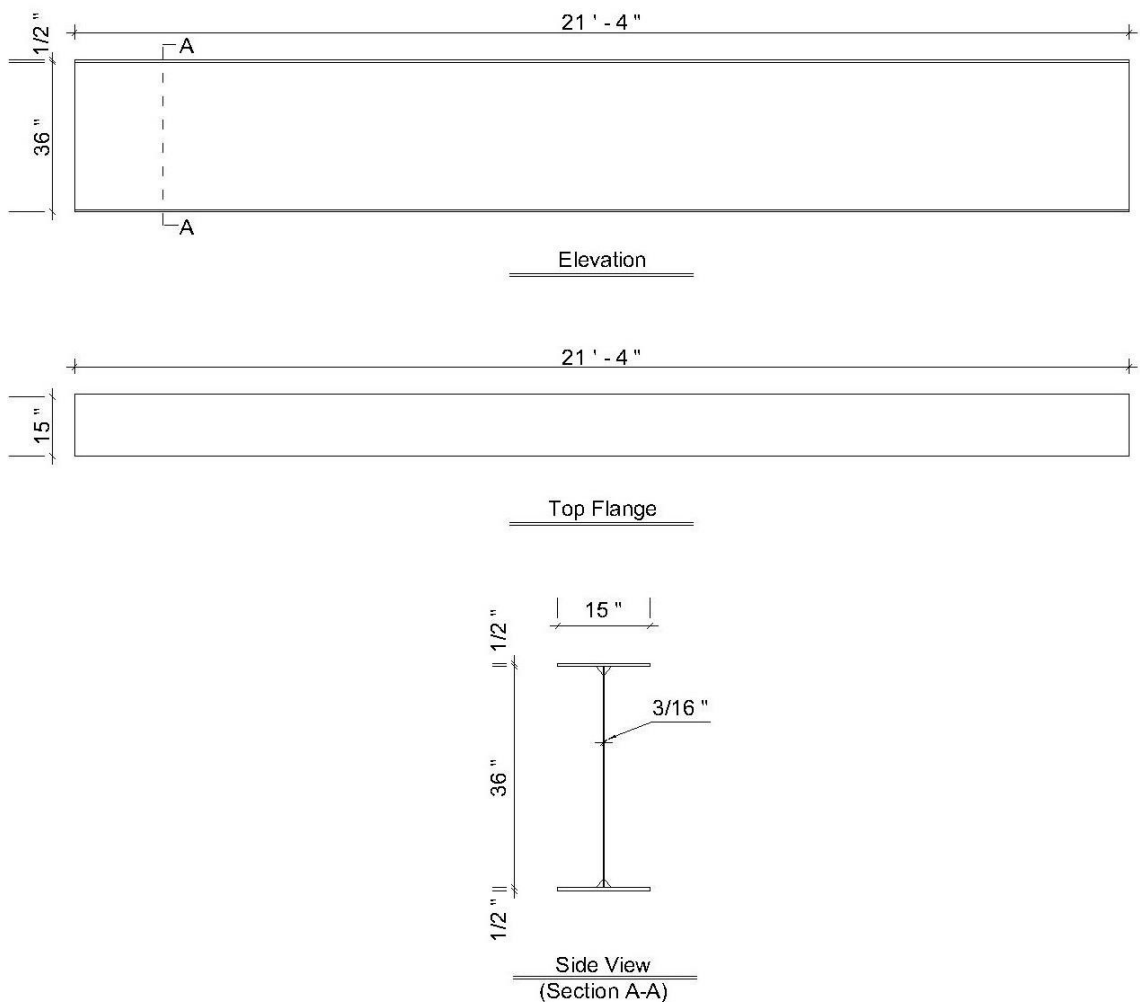


Fig. 3. Schematic geometry of the girder

3.2. Material properties

Four steel coupons from the steel girder were tested under tension according to American Society for Testing and Materials (ASTM) A370 Standard [31]. The tested coupons' geometry is shown in Fig. 4. A572 plate - grade 50 steel was utilized for the specimen, which is a high-strength, low-alloy structural steel that includes five grades as 42, 50, 55, 60, 65, and 80 with respect to their minimum yield strength. Measured average tensile properties of steel used are presented in Table 2.

4. Analysis of design strengths and stiffeners' design of the steel girder

4.1. Analysis and design basis

Analysis and stiffeners' design of the steel girder specimen were realized with respect to the provisions for load and resistance factor design (LRFD) in the ANSI/American Institute of Steel Construction (AISC) 360-16 - Specification for Structural Steel Buildings [33].

Plate girders' webs are generally slender, rarely they can be noncompact also [34]. The AISC Specification [33] examines flexural members having slender webs in Section F5, 'Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs Bent about Their Major Axis'. Flexural elements having noncompact webs are investigated in Section F4, 'Other I-shaped Members with Compact or Noncompact Webs Bent about Their Major Axis'. The section is concerned with both singly and doubly symmetric sections. In a User Note in Section F4, it is allowed to design members evaluated by Section F4 by the provisions of Section F5. Shear provisions for flexural elements are included in the AISC Specification [33] Chapter G, 'Design of Members for Shear' [35]. Additionally, stiffeners can be placed to avoid web buckling and maintain stability [34].

The loading was assumed to be a three-point bending considering simply supported boundary condition. There were 7 shear panels of 914 mm (3 ft) long in the girder. A uniaxial compressive load of 534 kN (120 kips) was applied on top of the girder at the first shear panel's end point, which is 914 mm (3 ft) from the support at the left. The set-up configuration, according to which analysis and stiffeners' design was realized, is demonstrated in Fig. 5.

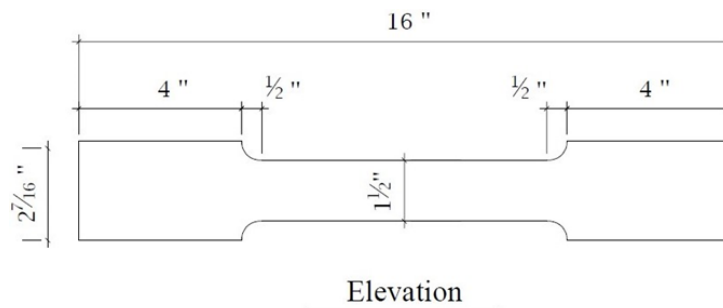


Fig. 4. Schematic geometry of the coupons

Table 2. Measured average mechanical properties of steel [32]

Nominal Thickness mm (in.)	Yield Strength MPa (ksi)	Modulus of Elasticity MPa (ksi)	Yield Strain mm/mm (in./in.)
5 (3/16)	345 (50)	200.000 (29.000)	0.00172

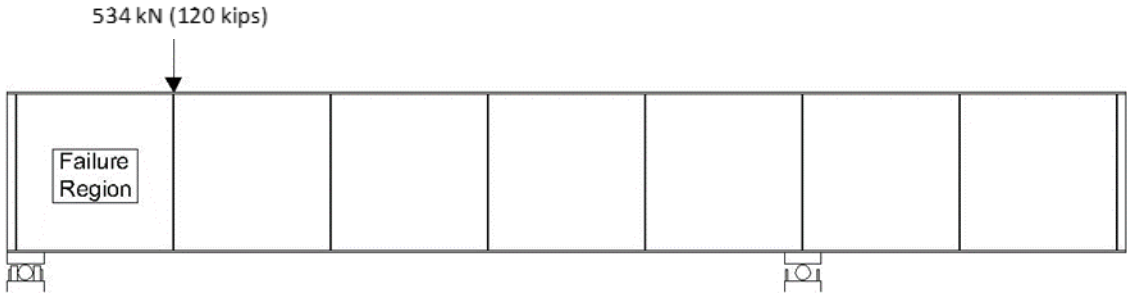


Fig. 5. Set-up configuration

4.2. Design shear strength check of the steel girder

Usage of transverse stiffeners is the most prominent property of plate girders. Stiffeners support web by increasing its strength to stand shear. A plate girder's elastic or inelastic buckling strength is not the full strength in shear. Also, after buckling post buckling strength develops through transverse stiffeners [36].

As it can be observed from Fig. 6, the shear is resisted by a diagonal tensile membrane stress field developing in the web. In the tension field, the transverse stiffeners behave as struts and the flanges behave as chords, eventually a Pratt or N-truss is created. In the post buckling range, the web plates provide strength similar to the resistance of the diagonal tie bars' in a truss [37].

- The shear strength is checked. The shear is maximum at the support at the left, but tension field action can not be used in the end panel. The shear strength must be determined from the AISC Specification [33] Eq. G2-1:

$$V_n = 0.6F_y A_w C_v \quad (1)$$

where V_n is the nominal shear strength, F_y is the yield strength, A_w is the cross sectional area of the web, C_v is the ratio of the critical web shear stress to the web shear yield stress.

If $1.37\sqrt{k_v E/F_y} < h/t_w$ then C_v is calculated from the AISC Specification [33] Eq. G2-5, which is:

$$C_v = \frac{1.51k_v E}{(h/t_w)^2 F_y} \quad (2)$$

where k_v is a plate-buckling coefficient, E is the modulus of elasticity, h is the web depth, and t_w is the web thickness. k_v is computed from the AISC Specification [33] Eq. G2-6:

$$k_v = 5 + \frac{5}{(a/h)^2} \quad (3)$$

a is the stiffener spacing.

For $h = 914$ mm (36 in), $t_w = 5$ mm (3/16 in), $a = 914$ mm (36 in):

$$1.37\sqrt{k_v E/F_y} = 104 < h/t_w = 192$$

then the AISC Specification [33] Equation G2-5 is used for calculating C_v , eventually:

$$V_n = 214 \text{ kN (48 kips)}$$

According to LRFD, design shear strength is $\phi_v V_n$ where $\phi_v = 0.9$ is the resistance factor for shear. Thus, the design shear strength is

$$\phi_v V_n = 193 \text{ kN (43 kips)} < 427 \text{ kN (96 kips)} \text{ (the maximum shear in the end panel) (N.G.)}$$

- It should be determined whether tension field action can be used in regions other than the end panels referring to the conditions of the AISC Specification [33] Section G3.1:

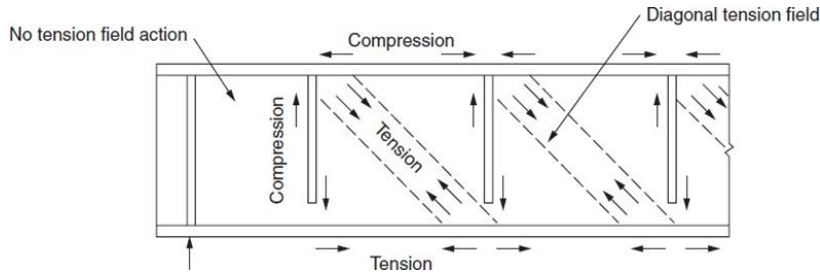


Fig. 6. Tension field action [37]

$$a) \quad \frac{a}{h} \leq 3.0 \quad \text{or} \quad [260/(h/t_w)]^2$$

$$b) \quad \frac{2A_w}{(A_{fc} + A_{ft})} \leq 2.5$$

$$c) \quad \frac{h}{b_{fc}} \quad \text{or} \quad \frac{h}{b_{ft}} \leq 6.0$$

where A_{fc} is the area of compression flange, A_{ft} is the area of tension flange, b_{fc} is the width of compression flange, and b_{ft} is the width of tension flange.

For $b_f = 381$ mm (15 in), $t_f = 13$ mm (1/2 in), $A_{fc} = A_{ft} = b_f t_f = 4953$ mm² (7.5 in²), and the steel girder satisfies all three conditions. Tension field action can be used.

To calculate the nominal shear strength with tension field action, except for the end panels, the AISC Specification [33] Eq. G3-2 is utilized:

$$V_n = 0.6F_y A_w \left(C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}} \right) \quad (4)$$

Thus,

$$V_n = 636 \text{ kN (143 kips)}$$

The design shear strength is:

$$\phi_v V_n = 573 \text{ kN (129 kips)} > 107 \text{ kN (24 kips)}.$$

Therefore, the shear strength is adequate where tension field action is allowed. (OK.)

4.3. Design flexural strength check of the steel girder

For a plate girder having a noncompact or slender web, the nominal moment capacity, M_n , will be smaller than its plastic moment capacity, M_p , due to a few limit states observed prior to achieving its plastic moment capacity. Probable limit states can be defined as follows:

- Compression flange yielding
- Lateral torsional buckling
- Compression flange local buckling
- Tension flange yielding

The design moment capacity of the plate girder is based on the compactness or slenderness of the flanges and it will be the lowest strength got from the mentioned limit states [34].

From the AISC Specification [33] Section B4, Table B4.1b, the web of a doubly symmetric I-shaped section is slender if:

$$\frac{h}{t_w} > 5.70 \sqrt{\frac{E}{F_y}} \quad (5)$$

From Eq. (5),

$$\frac{h}{t_w} = 192 > 5.70 \sqrt{\frac{E}{F_y}} = 137$$

thus, the web is slender and the provisions of the AISC Specification [33] Section F5 are applied.

4.3.1. Compression flange yielding

The compression flange nominal strength is calculated from the AISC Specification [33] Equation F5-1:

$$M_n = R_{pg} F_y S_{xc} \quad (6)$$

where M_n is the nominal flexural strength, R_{pg} is the bending strength reduction factor, S_{xc} is the elastic section modulus of the compression side. R_{pg} is obtained from the AISC Specification [33] Eq. F5-6 and the value of a_w in the R_{pg} equation is calculated from the AISC Specification [33] Eq. F4-12:

$$R_{pg} = 1 - \frac{a_w}{1200 + 300a_w} \left(\frac{h_c}{t_w} - 5.7 \sqrt{\frac{E}{F_y}} \right) \leq 1.0 \quad (7)$$

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} \leq 10 \quad (8)$$

The bending strength reduction factor of the steel girder is computed as $R_{pg} = 0.97$. The elastic section modulus of the compression side is: $S_{xc} = \frac{I_x}{y_c} = 5.10^6 \text{ mm}^3$ (308 in³). Thus, the nominal flexural strength is: $M_n = 1682.10^3 \text{ kN.mm}$ (14887 kips.in).

The design flexural strength is $\phi_b M_n$ where $\phi_b = 0.9$ is the resistance factor for flexure. Therefore, the design flexural strength is:

$$\phi_b M_n = 1514.10^3 \text{ kN.mm} (13398 \text{ kips.in})$$

4.3.2. Lateral-torsional buckling

The AISC Specification [33] Equation F5-2 gives the nominal lateral-torsional buckling strength as:

$$M_n = R_{pg} F_{cr} S_{xc} \quad (9)$$

where F_{cr} is the critical stress.

The occurrence of lateral-torsional buckling depends on the lateral support value, i.e. the unbraced length L_b . The unbraced length of the steel girder is, $L_b = 4572 \text{ mm}$ (180 in).

The length parameters, L_p , limiting laterally unbraced length for the yielding state, and L_r , limiting laterally unbraced length for the inelastic lateral-torsional buckling state are defined by the AISC Specification [33] Eqs. F4-7 and F5-5, respectively:

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} \quad (10)$$

$$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}} \quad (11)$$

The AISC Specification [33] Eq. F4-11 (simplified) is used to determine the value of r_t which is the effective radius of gyration for lateral-torsional buckling.

$$r_t = \frac{b_{fc}}{\sqrt{12 \left(1 + \frac{1}{6} a_w\right)}} \quad (12)$$

For the steel girder, $L_p = 2717$ mm (107 in) and $L_r = 9271$ mm (365 in).

Since $L_p < L_b < L_r$, the failure type is inelastic lateral-torsional buckling and Eq. F5-3 from the AISC Specification [33] is utilized to find out the F_{cr} value.

$$F_{cr} = C_b \left[F_y - (0.3F_y) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq F_y \quad (13)$$

where C_b is the lateral-torsional buckling modification factor for nonuniform moment diagrams and it is equal to 1.0 for cantilevers or overhangs. From Eq. (13),

$$F_{cr} = 316 \text{ MPa (46 ksi)} < F_y = 345 \text{ MPa (50 ksi)}.$$

Accordingly, the nominal lateral-torsional buckling strength is:

$$M_n = 1539.10^3 \text{ kN.mm (13625 kips.in)}$$

The design flexural strength is:

$$\phi_b M_n = 1385.10^3 \text{ kN.mm (12262 kips.in)}$$

4.3.3. Compression flange local buckling

The compression flange nominal strength is obtained from the AISC Specification [33] Eq. F5-7:

$$M_n = R_{pg} F_{cr} S_{xc} \quad (14)$$

F_{cr} is specified according to the flange's compactness or slenderness. The AISC Specification [33] classifies the slenderness parameters as; λ : the slenderness parameter, λ_{pf} : the limiting slenderness parameter for a compact flange, λ_{rf} : the limiting slenderness parameter for a noncompact flange. The slenderness parameters are defined in the AISC Specification [33] Table B4.1b as

$$\lambda = \frac{b_f}{2t_f} \quad (15)$$

$$\lambda_p = 0.38 \sqrt{\frac{E}{F_y}} \quad (16)$$

$$\lambda_r = 0.95 \sqrt{\frac{k_c E}{F_L}} \quad (17)$$

$k_c = \frac{4}{\sqrt{h/t_w}}$, ($0.35 \leq k_c \leq 0.76$) is the coefficient for slender unstiffened elements.

F_L , the magnitude of flexural stress in compression flange at which flange local buckling or lateral-torsional buckling is influenced by yielding, is equal to $0.7F_y$ for girders with slender webs. Since the steel girder web is slender, the AISC Specification [33] Eq. F4-6a is used:

$$F_L = 0.7F_y \quad (18)$$

Using $k_c = 0.29$ and $F_L = 241$ MPa (35 ksi) into Eqs. (15-19), $\lambda = 15$, $\lambda_p = 9.15$, $\lambda_r = 16.18$. Since $\lambda_p < \lambda < \lambda_r$, the flange is noncompact. Inelastic flange local buckling controls. For sections with noncompact flanges F_{cr} is calculated from the AISC Specification [33] Eq. F5-8.

$$F_{cr} = \left[F_y - (0.3F_y) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (19)$$

Thus, $F_{cr} = 259$ MPa (38 ksi).

The nominal flexural strength for the compression flange is

$$M_n = 1262.10^3 \text{ kN.mm (11171 kips.in)}$$

Thus, the design flexural strength is

$$\phi_b M_n = 1136.10^3 \text{ kN.mm (10054 kips.in)}$$

4.3.4. Tension flange yielding

According to the AISC Specification [33], when $S_{xt} \geq S_{xc}$, the limit state of tension flange yielding does not exist. Since $S_{xt} = S_{xc}$ for the steel girder, there is no probability of tension flange yielding.

- Consequently, the lowest nominal flexural strength, M_n , according to the limit states is: $M_n = 1262.10^3$ kN.mm (11171 kips.in).

The design flexural strength, $\phi_b M_n = 1136.10^3$ kN.mm (10054 kips.in) > the maximum factored load moment, $M_u = 391.10^3$ kN.mm (3456 kips.in) (OK.)

4.4. Stiffeners' design of the steel girder

4.4.1. Intermediate stiffeners' design

To determine a trial width for stiffeners, the available space is investigated. The maximum feasible width is: $\frac{b_f - t_w}{2} = 178$ mm (7 in) and $b = 102$ mm (4 in) is selected.

The requirements for stiffeners where tension field action is used are given in the AISC Specification [33] by Equations G3-3 and G3-4 as follows:

$$(b/t)_{st} \leq 0.56 \sqrt{\frac{E}{F_{yst}}} \quad (20)$$

$$I_{st} \geq I_{st1} + (I_{st2} - I_{st1}) \left[\frac{V_r - V_{c1}}{V_{c2} - V_{c1}} \right] \quad (21)$$

where $(b/t)_{st}$ is the width to thickness ratio of the stiffener, F_{yst} is the specified minimum yield stress of the stiffener material, I_{st} is the moment of inertia of the transverse stiffeners, I_{st1} is the moment of inertia required for the development of the web shear buckling resistance, I_{st2} is the moment of inertia required for the development of the buckling plus post buckling shear strength, V_r is the larger of the required shear strengths in the adjacent web panels, V_{c1} is the smaller of the available shear strengths in the adjacent panels calculated with no tension field action, V_{c2} the smaller of the available shear strengths in the adjacent panels calculated with tension field action.

The minimum required thickness is calculated from the AISC Specification [33] Equation G3-3 as:

$$t \geq 8 \text{ mm (0.3 in)}$$

The values of I_{st1} and I_{st2} are obtained from the AISC Specification [33] Equations G2-7 and G3-5.

$$I_{st1} = bt_w^3 j \quad (22)$$

where b is the smaller of the dimensions a and h and j is defined by the Equation G2-8 of the AISC Specification [33] as:

$$j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5 \quad (23)$$

$$I_{st2} = \frac{h^4 \rho_{st}^{1.3}}{40} \left(\frac{F_{yw}}{E} \right)^{1.5} \quad (24)$$

where F_{yw} is the specified minimum yield stress of the web material and ρ_{st} is the larger of F_{yw}/F_{yst} and 1.0.

For the intermediate stiffeners of the steel girder, $I_{st1} = 5.10^4 \text{ mm}^4$ (0.1 in⁴), $I_{st2} = 125.10^4 \text{ mm}^4$ (3 in⁴).

Utilizing the Steel Construction Manual of AISC [38] Table 3-17a where tension field action is not included, for $h/t_w = 192$ and $a/h = 1$

$$\frac{\phi_v V_n}{A_w} = 45 \text{ MPa (6.6 ksi)}$$

by interpolation. Thus, $\phi_v V_n = V_{c1} = 196 \text{ kN (44 kips)}$.

From the Steel Construction Manual of AISC [38] Table 3-17b where tension field action is included, for $h/t_w = 192$ and $a/h = 1$

$$\frac{\phi_v V_n}{A_w} = 132 \text{ MPa (19.2 ksi)}$$

by interpolation. Hence, $\phi_v V_n = V_{c2} = 577 \text{ kN (130 kips)}$. Required strength is: $V_r = V_u = 427 \text{ kN (96 kips)}$.

The required moment of inertia of the stiffeners is established from the AISC Specification [33] Equation G3-4:

$$I_{st} \geq 79.10^4 \text{ mm}^4 (1.9 \text{ in}^4)$$

If two 3/8 in×4 in plates are examined:

$$I_{st} = 716.10^4 \text{ mm}^4 (17.2 \text{ in}^4) > 79.10^4 \text{ mm}^4 (1.9 \text{ in}^4) \text{ (OK)}$$

The specified size is used for all of the intermediate stiffeners of the steel girder. In order to identify the stiffeners' length, priorly the stiffener to web weld and web to flange weld distances should be calculated. According to the AISC Specification [33] Section G2.2, the weld should terminate from the bottom flange a minimum distance of four times and a maximum distance of six times the web thickness.

$$\text{Minimum distance} = 4t_w = 19 \text{ mm (0.8 in)}$$

$$\text{Maximum distance} = 6t_w = 28 \text{ mm (1.1 in)}$$

If a flange to web weld size of 5 mm (3/16 in) and 20 mm (0.8 in) between welds are taken, then the estimated stiffener length is:

$$h - \text{weld size} - 20 \text{ mm (0.8 in)} = 890 \text{ mm (35 in)}$$

Two plates of 10×102×890 mm (3/9×4×35 in) are utilized for the intermediate stiffeners of the steel girder.

Compression capacity – Intermediate stiffener: ($d = 25t_w$)

With respect to the AISC Specification [33] Section J10.8, the length of web acting with the stiffener plates to form a compression member is $25t_w$ for an intermediate stiffener and $12t_w$ for an end stiffener. Moreover, the effective length factor, K , is stated in the same section of the AISC Specification [33] as $K = 0.75$. The slenderness ratio is defined as:

$$KL/r = Kh/r \quad (25)$$

where r is the radius of gyration of the column about the web axis and h is the clear distance between the flanges. It is stated in the AISC Specification [33] by Eq. J4-6 that the nominal compressive strength, P_n , of

the elements when the slenderness ratio is not greater than 25 is $F_y A_g$, where A_g is the gross area of the element.

$$P_n = F_y A_g \quad (26)$$

In the AISC Specification [33] Section J4.4, the available compressive strength, P_c , is indicated as:

$$P_c = \phi P_n \quad (27)$$

$\phi = 0.9$ is the resistance factor for compression.

For the intermediate stiffeners of the steel girder, $KL/r = 12.9 < 25$,

$$P_n = 863 \text{ kN (194 kips)},$$

$$P_c = 777 \text{ kN (175 kips)} > P = 534 \text{ kN (120 kips)} \text{ (OK)}$$

4.4.2. Bearing stiffeners' design:

Bearing stiffeners are supplied at the supports and at the concentrated load location.

Two 10×102 mm (3/8 in×4 in) stiffeners are tried:

The AISC Specification [33] Eq. J7-1 represents the nominal bearing strength of the bearing stiffeners as:

$$R_n = 1.8 F_y A_{pb} \quad (28)$$

where A_{pb} is the stiffener bearing area. For LRFD, the resistance factor is $\phi = 0.75$

In the AISC Specification [33] Section J7, the available bearing strength, R_c , is signified as:

$$R_c = \phi R_n \quad (29)$$

where $\phi = 0.75$ is the resistance factor for bearing.

For the bearing stiffeners of the steel girder,

$$R_n = 901 \text{ kN (203 kips)}$$

$$R_c = 676 \text{ kN (152 kips)} > P = 534 \text{ kN (120 kips)} \text{ (OK)}$$

Compression capacity – End stiffener: ($d = 12t_w$)

For the bearing stiffeners of the steel girder, $KL/r = 12.1 < 25$,

$$P_n = 761 \text{ kN (171 kips)}$$

$$P_c = 685 \text{ kN (154 kips)} > P = 427 \text{ kN (96 kips)} \text{ (OK)}$$

Two plates of 10×102×914 mm (3/8×4×36 in) are used for the bearing stiffeners of the steel girder.

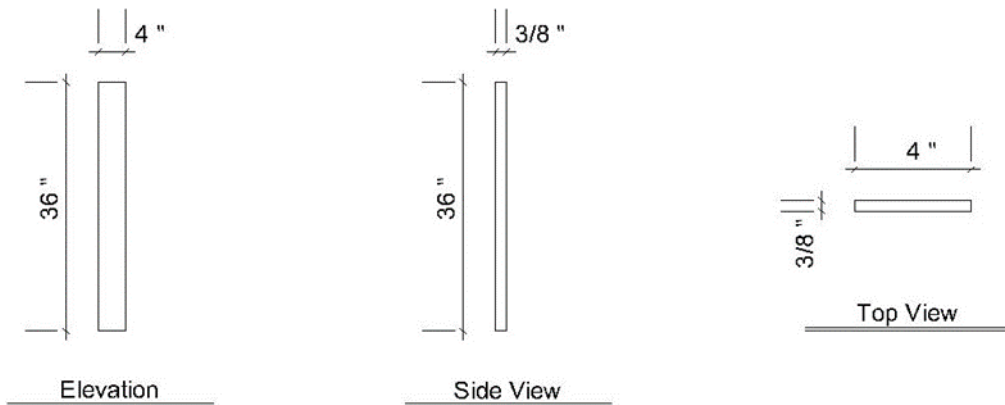


Fig. 7. Schematic geometry of the stiffeners

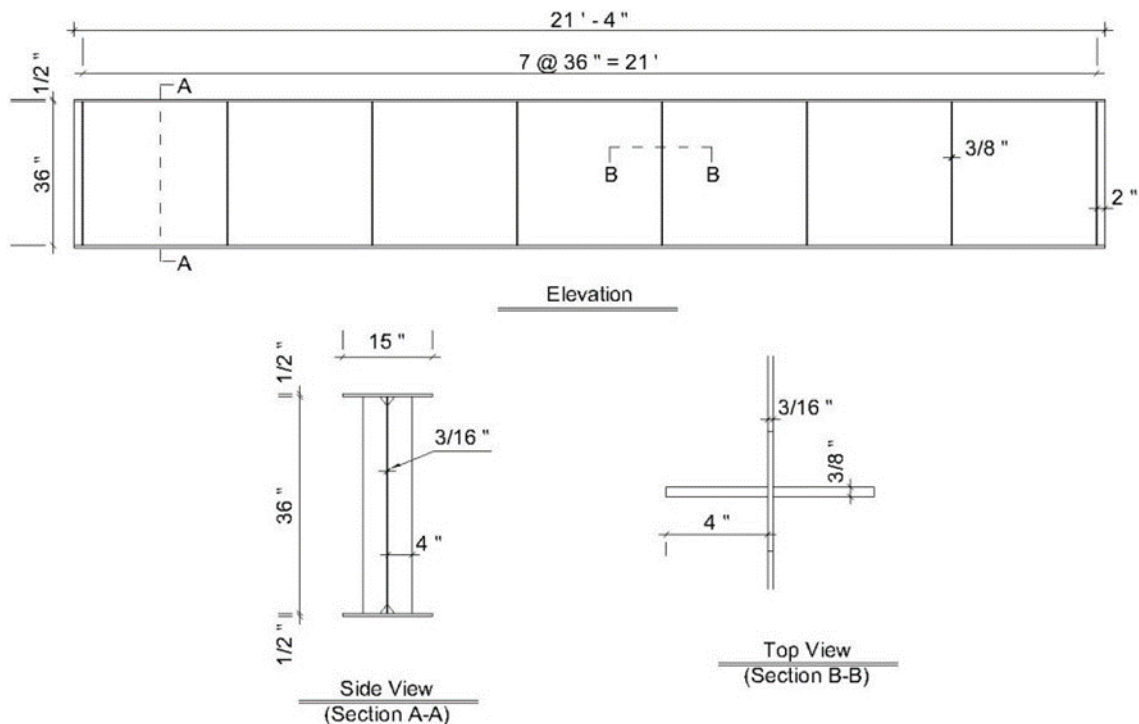


Fig. 8. Schematic geometry of the specimen

Consequently, in order to have single size, $10 \times 102 \times 914$ mm ($3/8 \times 4 \times 36$ in) are utilized for the intermediate and the bearing stiffeners of the steel girder. In Fig. 7, the schematic geometry of the stiffeners designed is presented. Furthermore, the schematic geometry of the whole specimen, i.e. the steel girder with the stiffeners, is demonstrated in Fig. 8.

5. Discussion

The analysis and stiffeners' design of the steel girder specimen, which was designed intentionally to fail in shear buckling, were performed in the study. The setup was presumed to be a three-point bending considering simply supported boundary condition. There were 7 shear panels of 914 mm (3 ft) long in the girder. An axial compressive load of 534 kN (120 kips) was applied on top of the girder at the first shear panel's end point.

The steel girder properties in terms of girder geometry and measured average mechanical properties of the steel used were presented. Analysis of design strengths and stiffeners' design of the steel girder specimen was done according to the provisions for LRFD in the AISC Specification [33].

In the AISC Specification [33], flexural members having slender webs are examined in Section F5. Flexural elements having noncompact webs are included in Section F4. It is permitted to design members mentioned in Section F4 by the provisions of Section F5. Shear provisions for flexural elements are given in the Chapter G of the AISC Specification [33].

Design shear strength check of the steel girder was performed. It was determined that the shear strength of the steel girder was satisfactory where tension field action was allowed.

Afterwards, design flexural strength check of the steel girder was realized. The nominal flexural strength of a plate girder is identified by one of the limit states of compression flange yielding, lateral-torsional buckling, compression flange local buckling, or tension flange yielding. The design moment capacity of the

plate girder is the lowest strength obtained from the limit states. For the steel girder, the lowest nominal flexural strength, M_n , was found from compression flange local buckling. The design flexural strength was higher than the maximum factored load moment.

At last, the design stages of the intermediate stiffeners and bearing stiffeners of the steel girder were accomplished. Also, the compression capacities of the intermediate stiffeners and the end stiffeners were checked and determined to be convincing. Consequently, a single size was specified for the intermediate and the bearing stiffeners of the steel girder.

6. Concluding remarks

Plate girders are made to carry severe loads over wide spans. In the design of plate girders various failure mechanisms, e.g. shear buckling of the web, lateral-torsional buckling of the girder, compression buckling of the web, local buckling of the web, local buckling of the flanges, are taken into account. Shear strength of steel girders having slender webs is too low in proportion to the yielding shear capacity. This is principally observed because the web buckles before the material reaches its yield strength. For this reason, webs are reinforced with transverse stiffeners to improve their buckling strength. Webs with stiffeners stand shear also after buckling, which is known as post buckling strength. Tension field theories explain the mechanics of the post buckling strength generation and propose the ultimate shear strength of the stiffened web. Tension field action supposes that the post buckling shear strength forms by the tensile stresses in a diagonal field activated after elastic shear buckling. Most of the design code provisions are set on tension field theories. The paper included the design shear strength and design flexural strength checks and stiffeners' design of the steel girder specimen purposely designed to have shear buckling failure. Analysis and stiffeners' design were realized with respect to the provisions for LRFD in the AISC Specification [33].

Declaration of conflicting interests

The author(s) declared no potential conflicts of interest with respect to the research, authorship, and/or publication of this article.

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