# DESIGN REQUIREMENTS FOR PLATE GIRDERS WITH BOLTED TRANSVERSE STIFFENERS

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**(Received: April 26, 2003 – Accepted in Revised Form: Feb. 25, 2005)**

**Abstract** The web of plate girders in bridges are usually reinforced by welded transverse stiffeners in order to improve their shear capacity. Due to problems associated with field welding and fatigue, welded stiffeners are not suitable for retrofitting existing bridges. Bolted stiffeners are a practical alternative for strengthening girders that are expected to experience shear stress in excess of their design shear capacity. This paper presents the results of an analytical study into behavior of plate girders with bolted transverse stiffeners. Based on this study new requirements are developed for designing of such girders.

**Key Words** Bridge, Building, Plate Girder, Plate Buckling, Stiffener, Shear Strength

**چکیده** سخت‌کننده‌های عرضی معمولاً به منظور افزایش ظرفیت برشی شاهتیرهای پلهای فلزی به جان تیر ورقها جوش داده می‌شوند. ولی برای مقاوم‌سازی پلهای موجود، جوشکاری در محل باعث کاهش مقاومت خستگی پل می‌شود و لذا این سخت‌کننده‌ها برای مقاوم‌سازی پلها مناسب نمی‌باشند. از این جهت نصب سخت‌کننده‌های عرضی با اتصال پیچ یک روش جایگزین مناسب و عملی برای مقاوم‌سازی شاهتیرهای پلهای فلزی می‌باشد. در این مقاله نتایج مطالعات تحلیلی بر رفتار و استقامت تیر ورقهایی که توسط سخت‌کننده عرضی با اتصال پیچ تقویت شده‌اند ارائه شده است. براساس نتایج تحلیل‌ها، ضوابط جدیدی برای طراحی این نوع سخت‌کننده‌ها نیزارائه شده است.

## 1. INTRODUCTION

Plate girder bridges are the most common type of steel bridges and are economically competitive in the span range of 20 to 90 meters. The girders in such bridges are usually fabricated with slender webs with transverse stiffeners. The stiffeners are usually welded to the web in the fabrication shop in order to increase shear capacity of the girder. However, due to problems associated with field welding and fatigue, welded stiffeners are not suitable for retrofitting existing bridge girders. For such purposes, bolted stiffeners are far more suitable than welded stiffeners because installation of bolted stiffener does not require field welding. Figure 1 shows typical bolted stiffeners installed on a plate girder. The stiffener may be any structural shape made of any engineering material like steel, wood or aluminum.

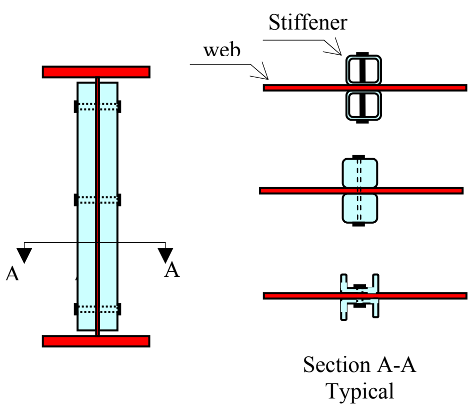

**Figure 1.** Typical bolted stiffeners

The design requirements in various building and bridge specifications are based on experimental and analytical studies of plate girders with welded transverse stiffeners. Such requirements may not be entirely applicable for girders with bolted stiffeners. This paper presents the results of an analytical study[1] sponsored by the International Institute of Earthquake Engineering and Seismology into shear strength and behavior of such plate girders.

## 2. BACKGROUND

In a transversely stiffened girder, the web resists much of the applied shear by working in diagonal tension similar to tension diagonal of a Pratt truss. The similarity between a slender web and tension diagonal of a Pratt truss was recognized since the early work on steel girders. But the original development of a comprehensive theory of diagonal tension[2,3,4] was in connection with aluminum girders with relatively rigid flanges. The theoretical and experimental work of Basler[5] resulted in a shear model for steel girders with flexible flanges. This model forms the basis of shear design provisions in AASHTO[6] and AISC[7] specifications.

### 2.2 AASHTO Design Requirements

The design requirements of AASHTO specification are based on the Basler tension field model. According to the AASHTO specification the ultimate shear capacity of plate girder, $V_u$ , is calculated from the following equation.

$$V_u = 0.58\sigma_y A_w \left[ C + \frac{0.87(1-C)}{\sqrt{1+\alpha^2}} \right] \qquad (1)$$

Where $C$ is the ratio of shear buckling capacity to shear yield capacity of the web. The first term in the bracket corresponds to the buckling capacity of the web and the second term corresponds to the tension field capacity of the web.

The above formula was found to be in good agreement with the test results[5]. The tests were conducted on large scale steel plate girders with dimensions similar to those used in civil engineering applications. Results of shear tests performed on composite plate girders were also found to be in good agreement with this formula.[8,9]

#### 2.2.1 Flexural Rigidity of Stiffener

The AASHTO specification requires that the moment of inertia of stiffener be equal to or greater than the following equation.

$$I = d_o t_w^3 J \qquad (2)$$

where:

$$J = 2.5\left(\frac{D}{d_o}\right)^2 - 2 \geq 0.5$$

$D$ = Depth of web

$d_o$ = stiffener spacing

#### 2.2.2 Area of Stiffener

The Basler tension field model requires that transverse stiffeners have adequate area to anchor the vertical component of the tension field force. It is also assumed that a certain width of the web tributary to stiffener and equal to $18t_w$ participate in resisting this force and the required stiffener area, $A_s$ , according to AASHTO specification is:

$$A_s = \left[ 0.15B(1-C)A_w \frac{V}{V_u} - 18t_w^2 \right] Y \qquad (3)$$

Where Y is the ratio of web yield stress to stiffener yield stress and $B$ is a factor which reflects the interaction of compression and bending stresses in the stiffener. The specification uses $B$ =1.0 for symmetrical stiffeners, $B=2.4$ for single plate stiffener and $B=1.8$ for single angle stiffener.

## 3. BOLTED TRANSVERSE STIFFENERS

The AASHTO shear strength equation and stiffener requirements are applicable to plate girders whose web is reinforced by welded transverse stiffeners. When the web is reinforced by bolted stiffeners, the shear strength and stiffener requirements are expected to be different mainly because the vertical component of the tension field would not be resisted by the stiffener. This section presents the results of the analytical study about the behavior and shear capacity of plate girders which are reinforced by bolted transverse stiffeners.

### 3.1 Analytical Study

Nonlinear Finite Element Analysis (FEA) is used for the analytical study. The study is conducted in two phases. The behavior and shear capacity of plate girders with rigid bolted stiffener are investigated in the first phase. Requirements for flexural rigidity of bolted stiffeners are developed in the second phase.

### 3.2 Description of FEA Model

Figure 2 shows the FEA representation of the analytical model. Flanges, web and welded stiffeners are modeled using plastic large strain shell elements with bilinear material property. The material yield strength and elastic modulus are respectively 336$^{MPa}$ and 204000$^{MPa}$. The inelastic modulus of the material is set equal to 0.01 times the elastic modulus. The girder was built-in at one end and laterally supported at the other end where a concentrated load is applied.

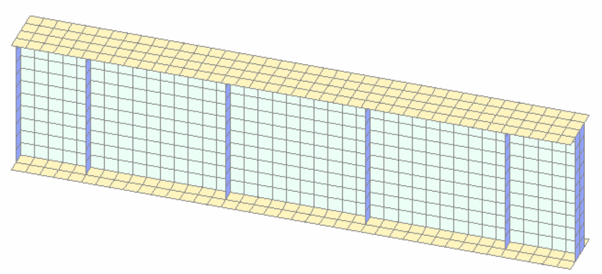

**Figure 2.** FEA representation of the analytical model

### 3.3 Rigid Bolted Stiffeners

The analyses consisted of twelve girders with two web slenderness ratios of 160 and 200 and three panel aspect ratios of 1.0, 1.5, and 2.0. Each sample is analyzed once with welded stiffeners and once with rigid bolted stiffeners. Area and stiffness of welded stiffeners satisfy the specification requirements. Rigid bolted stiffener is modeled by restraining out-of-plane displacement of the web. For each sample the welded stiffeners are replaced with rigid supports on the web to model the rigid stiffeners. Similar to bolted stiffeners, these supports only restrain the out-of-plane deflection of the web, but do not resist the vertical component of tension filed forces.

Table 1 gives girder dimensions and ultimate shear capacities in accordance with the AASHTO specification. It also lists the shear capacity obtained from the analyses. Failures in all cases are due to shear. Yielding of the web across a tension field band in direction of panel's diagonal is the basic failure mechanism.

Figure 3 compares the shear capacities with the shear strength predicted by AASHTO specification. This figure shows that shear capacities in all cases exceed the ultimate shear strength predicted by the specification. It also shows that ultimate shear strength of girders with rigid bolted stiffeners are almost identical to the shear strength of girders with welded stiffeners. Out-of-plane deflection of the web and formation of tension field across panel's diagonal are also similar for the two cases. The results of these analyses indicate that behavior and shear strength of plate girders whose web is reinforced with rigid bolted stiffeners are almost the same as girders with welded stiffeners. In other words these results demonstrate that the vertical component of tension field force can be resisted by the web strip adjacent to the stiffener, if bolted stiffeners have sufficient flexural rigidity to maintain a nodal line along the stiffener.

**Table 1.** Girder dimensions and results of analyses

| *sample* | $D/t_w$ | *Web (cm x cm)* | *Flange (cm x cm)* | *Length (m)* | $\alpha=d_0/D$ | *Shear capacity (Ton)* | | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | *AASHTO* | *Welded stiffener* | *Bolted stiffener* |
| 1 | 200 | 1.0 x 200 | 5.0 x 80 | 8.0 | 1.0 | 266.1 | 291.3 | 284.2 |
| 2 | | | | | 1.5 | 217.3 | 251.2 | 249.8 |
| 3 | | | | | 2.0 | 182.6 | 240.2 | 240.0 |
| 4 | | 0.8 x 160 | 4.0 x 64 | 6.4 | 1.0 | 170.1 | 187.7 | 182.1 |
| 5 | | | | | 1.5 | 139.0 | 165.5 | 161.8 |
| 6 | | | | | 2.0 | 116.8 | 140.0 | 130.5 |
| 7 | 160 | 1.25 x 200 | 5.0 x 80 | 8.0 | 1.0 | 357.5 | 374.4 | 371.0 |
| 8 | | | | | 1.5 | 295.6 | 331.3 | 328.9 |
| 9 | | | | | 2.0 | 252.8 | 311.4 | 308.0 |
| 10 | | 1.0 x 160 | 4.0 x 64 | 6.4 | 1.0 | 228.8 | 241.2 | 239.6 |
| 11 | | | | | 1.5 | 189.2 | 212.5 | 210.2 |
| 12 | | | | | 2.0 | 161.8 | 199.3 | 196.6 |

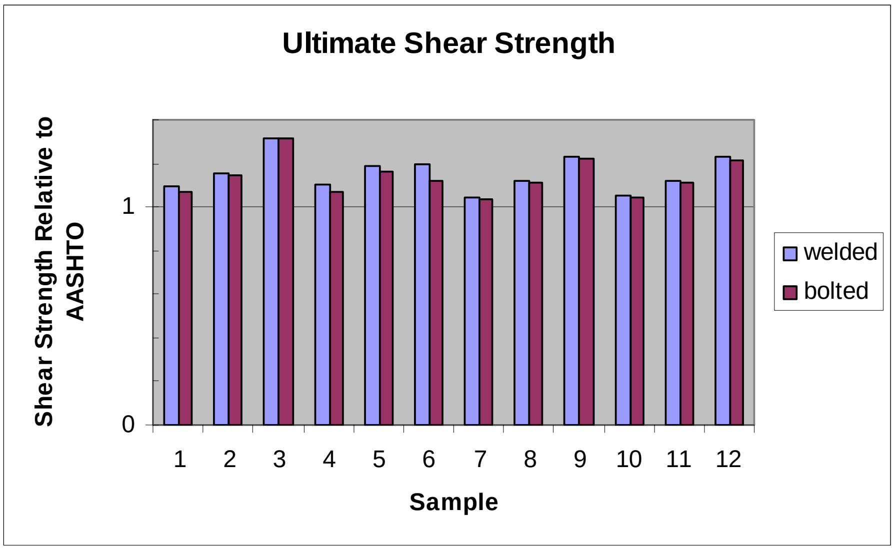


**Figure 3.** Shear strength - welded stiffener vs. rigid bolted stiffener

### 3.4 Flexural rigidity of bolted stiffener

The flexural rigidity of the stiffener can be estimated based on required stiffness to adequately brace the web under the vertical component of the tension filed force. In this approach, the web strip adjacent to the stiffener and the bolted stiffeners are idealized as a simply supported column with an unbounded sandwich cross section as shown in Figure 4. The web strip and stiffeners are free to move axially relative to each other, but are constrained to move laterally with each other. The stiffeners are not under any axial load and are only subjected to the bending moment caused by lateral deflection of the web strip

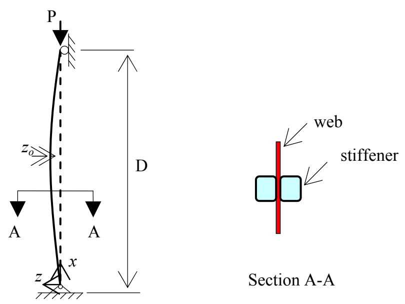


**Figure 4.** Idealized column - bolted stiffener

The critical buckling load of the idealized column is the sum of buckling load of each element making up the column. Assuming negligible buckling capacity for the web strip, the critical buckling load of the idealized column is the sum of buckling load of each stiffener.

$$P_{cr} = \sum P_{cr-stiffener} = \frac{\pi^2 \sum (EI)_{stiffener}}{D^2} \quad (4)$$

Since the section is unbounded, each element bends with respect to its own neutral axis and $\Sigma(EI)_{stiffener}$ in the above equation, corresponds to the sum of flexural rigidity of each stiffener with respect to its own bending axis.

The critical buckling load should be greater than the maximum axial force which is carried by the web strip multiplied by an appropriate load factor γ.

$$P_{cr} \geq \gamma F_v \quad (5)$$

The maximum axial force which is directly applied to the web strip corresponds to vertical component of tension filed force. Assuming the Pratt truss analogy, the maximum axial force which is carried by the web strip is equal to the tension field contribution to the shear capacity.

$$F_v = \frac{\sigma_y (1-C) A_w}{2\sqrt{1+\alpha^2}} \quad (6)$$

The required flexural rigidity of the bolted stiffeners can be calculated as follows.

$$P_{cr} \geq \gamma F_v$$

$$\sum (EI)_{stiffener} \geq \gamma \frac{\sigma_y (1-C) A_w D^2}{2\pi^2 \sqrt{1+\alpha^2}} \quad (7)$$

The load factor, γ , should be chosen to assure that the girder reaches its ultimate shear capacity prior to buckling of the stiffener. Since the ultimate shear strength of plate girders usually exceeds the predicted capacity, a load factor of 1.5 is recommended for design of bolted stiffeners. The recommended requirement for flexural rigidity of transverse bolted stiffener is:

$$\sum (EI)_{stiffener} \geq \frac{3\sigma_y (1-C) A_w D^2}{4\pi^2 \sqrt{1+\alpha^2}} \quad (8)$$

where:

$$C = \frac{\tau_{cr}}{0.58\sigma_y}$$

α = aspect ratio of panel
$\sigma_y$ = web yield stress
$A_w$ = web area

$D$ = Depth of web

It is also recommended that the flexural rigidity of bolted stiffener meet the minimum stiffness requirement of the AASHTO specification to insure stability of stiffener prior to formation of tension field.

**3.4.1 Flexible Bolted Stiffeners**

In order to evaluate the requirements for flexural rigidity of bolted stiffeners, some of the previous samples are analyzed again with flexible bolted stiffeners. Beam elements, whose lateral displacement is coupled with the corresponding out-of-plane displacement of the web, are used to model the stiffener. Similar to bolted stiffener, the beam elements restrain the out-of-plane deflection of the web, but do not resist the vertical component of the tension filed force. Rectangular steel sections are used for the stiffeners. The analyses are performed for stiffeners with the proposed flexural rigidity and stiffeners with moment of inertia equal to that required by the specification.

Table 2 lists the moment of inertia of the stiffeners and the ultimate shear capacity of the samples. This table also lists the shear capacity associated with rigid stiffeners and shear strength predicted by AASHTO specification.

Figure 5 compares the shear capacities of samples with the shear strength predicted by AASHTO specification. It also shows the shear capacities when girders are reinforced by rigid bolted stiffeners. This figure indicates that when bolted stiffeners with AASHTO required moment of inertia used, the shear strength may be less than the shear strength predicted by the specification. For such cases the failure is generally due to yielding of stiffener prior to full development of

**Table 2**. Results of analyses for flexible bolted stiffeners

| Sample | Web (cm x cm) | $D/t_w$ | $\alpha = d_o/D$ | Moment of inertia ($cm^4$) | | Shear capacity (Tons) | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | AASHTO | Proposed | *AASHTO* | *Bolted stiffener* | | |
| | | | | | | | *Rigid* | $I=I_{AASHTO}$ | $I=I_{Proposed}$ |
| 1 | 1.0 x 200 | 200 | 1.0 | 100 | 553 | 266.1 | 284.2 | 187.0 | 283.0 |
| 2 | | | 1.5 | 150 | 462 | 217.3 | 249.8 | 182.0 | 248.0 |
| 3 | | | 2.0 | 200 | 381 | 182.6 | 240.0 | 190.0 | 240.0 |
| 4 | 1.25 x 200 | 160 | 1.0 | 195 | 578 | 357.5 | 371.0 | 340.0 | 370.0 |
| 5 | | | 1.5 | 293 | 511 | 295.6 | 328.9 | 320.0 | 328.0 |
| 6 | | | 2.0 | 390 | 429 | 252.8 | 308.0 | 308.0 | 308.0 |

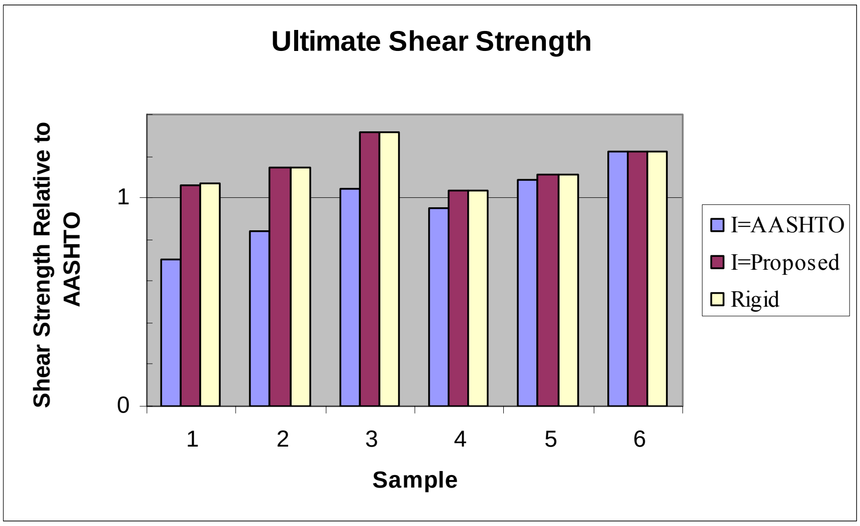


**Figure 5.** shear strength of girders reinforced by bolted stiffeners

tension field action. This indicates that the specification requirement for minimum moment of inertia is not adequate for bolted stiffeners. This figure also shows that when bolted stiffeners with proposed flexural rigidity are used, shear strength excees the ultimate shear strength predicted by the specification. For such cases the shear strength is almost the same as the shear strength of the girder with rigid stiffeners. Failure in these cases is due to the yielding of the web across a tension field band in direction of panel's diagonal.

## 4. CONCLUSION

The results of the analytical study indicate that behavior and shear strength of plate girders with bolted stiffeners are almost identical to girders with welded stiffeners provided that the stiffener has sufficient flexural rigidity. For such cases the current AASHTO shear strength formula gives a conservative estimate of the shear capacity. However, the specification requirement for flexural rigidity of stiffener is not adequate for bolted stiffeners. A new requirement is proposed for minimum flexural rigidity of bolted transverse stiffeners. The proposed requirement needs to be verified by experimental investigation. Further work is also needed to establish the design requirements for connecting bolts.

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