

Required Bending Rigidity for Transverse Stiffeners of Steel Plate Girders in Shear Loading: Numerical Investigation

Sérgio Nascimento¹; José J. Oliveira Pedro²; and Ulrike Kuhlmann³

Abstract: Transverse stiffeners provide lateral support to slender webs along their span, enhancing the shear buckling resistance of steel plate girders. Moreover, intermediate transverse stiffeners are particularly crucial, because they anchor the forces generated by tension field action, mitigate out-of-web buckling, and support the edges of the web panel. The standard EN 1993-1-5 outlines three requirements for the design of intermediate transverse stiffeners: (1) strength verification, (2) a minimum elastic bending rigidity, and (3) a torsional rigidity check. However, research has demonstrated that applying the strength verification criterion often results in overly conservative designs due to assumed internal forces significantly exceeding those observed in numerical simulations and experimental studies. Consequently, US design standards have omitted this verification, focusing solely on the minimum bending rigidity requirement. A numerical study of 13,500 plate girders with diverse geometries and stiffener configurations was conducted to evaluate the minimum bending rigidity required for intermediate transverse stiffeners to ensure the ultimate shear resistance of steel plate girders. The analysis considered the effects of steel grade, web slenderness and aspect ratio, flange-to-web area ratio, and the presence of longitudinal stiffeners. The results indicate that the required minimum bending rigidity is particularly high for plate girders with slender webs, low aspect ratios, and high flange-to-web area ratios. **DOI: 10.1061/JSENDH.STENG-15081.** © 2026 American Society of Civil Engineers.

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Introduction

Plate girders of large steel structures usually are used as the main longitudinal structural system for supporting bridges, industrial buildings, and sports complex roofs. To be competitive in terms of material efficiency and ease of fabrication, the I-shaped steel plate girders usually are fabricated by welding narrow top and bottom flanges with thin web plates, which can be strengthened by stiffening elements (Fig. 1).

The structural behavior of steel plate girders is linked closely to their web height, h_w , which must be sufficient to resist bending in high-moment areas, such as intermediate supports. However, the design often incorporates slender webs (high $\lambda_w = h_w/t_w$ ratio), because a thin web (thickness t_w) is adequate to carry shear stresses. As a result, these thin webs are susceptible to instability at relatively low loads. To address this, plate girders typically are reinforced with web stiffeners to prevent buckling, which can be double- or single-sided (Fig. 1). These stiffeners reduce the effective span of the web panels, significantly increasing the critical elastic buckling stress, σ_{cr} . The reinforcement may consist of transverse and/or longitudinal

stiffeners, depending on their orientation relative to the girder axis (Fig. 1).

As a first rule in the 1910s, the American Railway Engineering Association's Bridge Specifications recommended minimum dimensions for the design of intermediate stiffeners (AREMA 1910). The first proposal for the elastic design of the web was made by Timoshenko and Gere (1936), who defined a minimum moment of inertia for the stiffener. However, the postbuckling resistance of a web plate was studied extensively and incorporated into design guidelines only throughout the second half of the 20th century (Oliveira Pedro et al. 2024b). Emphasis was placed on the practical importance of evaluating new requirements for web transverse stiffeners, given the increased stresses they undergo from postbuckling tension field action.

Consequently, the area of the web transverse stiffener, A_{st} , required to withstand the tension field action was investigated. Strength design rules then were incorporated into the British standard, Eurocode, Swiss standard, and US standards (AASHTO and AISC), which specified that stiffeners should be designed to resist axial forces.

However, several researchers have argued that the design axial force, N_{st} , used in these standards for the strength check significantly overestimates the actual acting force in the stiffener (Rockey et al. 1981; Tang and Evans 1984; Xie and Chapman 2003; Lee et al. 2003; Johansson et al. 2007; Presta et al. 2008). As a result, the strength verification for the design of transverse stiffeners was removed from the US standards (AASHTO 2007; AISC 2010), in which design is based solely on a minimum bending rigidity that ensures that the stiffeners provide rigid support for the web panels up to the ultimate shear resistance of the girder. Recent investigations have confirmed this through experimental testing and numerical simulations (Sinur and Beg 2012; Augustyn et al. 2022; Nascimento et al. 2023, 2025a; Freitas et al. 2024).

The philosophy underlying EN 1993-1-5 (CEN 2024a) concerning stiffener design has a substantial historical background.

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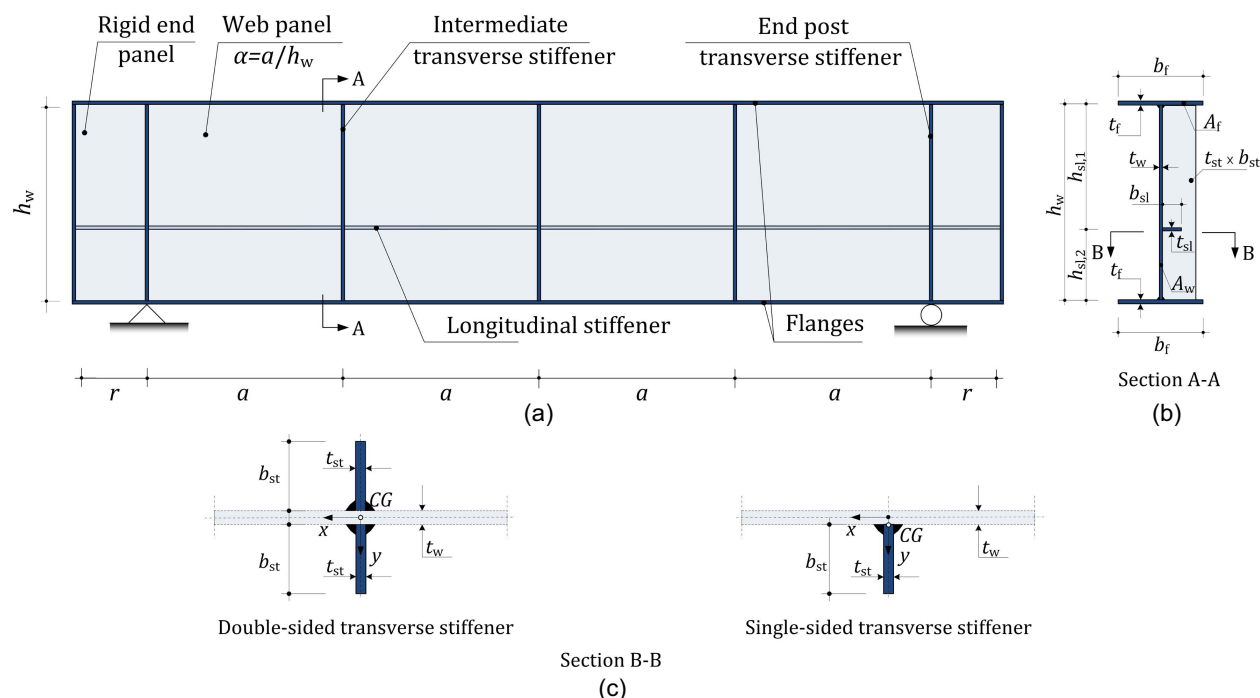


Fig. 1. Plate girder main elements and geometrical characteristics: (a) longitudinal view; (b) Section A-A; and (c) Section B-B.

Despite being deemed “probably adequate,” further investigations into current stiffener design methods have been considered “worthwhile” (Ziemian 2010). Following the assessment of axial forces in stiffeners (Nascimento et al. 2023; Freitas et al. 2024), the authors prioritized the evaluation of the required relative bending rigidity of transverse stiffeners to ensure the development of postbuckling stresses in the web, in line with recent research on this topic using numerical models (Stanway et al. 1996; Kim et al. 2007; Sinur et al. 2013; Lee et al. 2014; Kövesdi et al. 2018; Xiao et al. 2019).

Recent investigations into stiffener rigidity for the shear postbuckling range did not fully consider the influence of the flange dimensions, measured using the relative flange area compared to the web (A_f/A_w). Specifically, Kim (2004) conducted a parametric study with A_f/A_w ratios from 0.33 to 1.02, with one case of $A_f/A_w = 1.88$ for $\alpha = 3$ and $\lambda_w = 150$. Subsequently, Kim et al. (2007), Kim and White (2014) refined this work, proposing an ultimate bending rigidity formula that has been adopted as a design rule in AASHTO (2007). Furthermore, Sinur et al. (2013) conducted a detailed study on stiffener requirements for the case of $A_f/A_w = 0.3$, leading to a formula for the design of intermediate transverse stiffeners. Lee et al. (2014) performed a parametric study considering various web slenderness and aspect ratios, focusing on the case of $A_f/A_w = 0.9$.

To withstand high bending moments, stronger flanges typically are employed in structural design. Accordingly, the present paper addresses the requirements for intermediate transverse stiffeners under such conditions of high ratios of A_f/A_w .

Design Methods for Transverse Stiffeners

General Considerations

In modern plate girder design, rigid intermediate stiffeners welded vertically to a web plate are decisive for structural performance. Their essential role includes:

- Lateral support for web plates: When $\tau \leq \tau_{cr}$, the main function of a transverse stiffener—characterized by its moment of inertia

I_{st1} [or stiffener bending rigidity γ_{st1} , as defined in Eq. (1)]—is to provide lateral support to the web plate along its span and effectively divide the web into smaller subpanels (reducing the panel aspect ratio $\alpha = a/h_w$), thereby enhancing stability. This is consistent with established findings showing that increased panel aspect ratios lead to a reduction in the shear resistance of plate girders (Basler 1961; Oliveira Pedro et al. 2024b);

- Equilibrium of tension field action: When $\tau > \tau_{cr}$ the stiffener actively contributes to equilibrating the tension field action. The optimal rigidity for such conditions, denoted $I_{st2} > I_{st1}$ (or stiffener bending rigidity $\gamma_{st2} > \gamma_{st1}$), is updated to ensure postbuckling resistance of the web plate. Additionally, stiffeners prevent the inward movement of flanges toward each other, which is the primary source of axial loading on the stiffener; and
- Additional tasks: Stiffeners also facilitate the installation of bracing between girders and can serve as bearing stiffeners during erection or transportation, playing a key role in load introduction by distributing concentrated loads into the web and flange, thereby preventing local buckling.

The bending rigidity of a transverse stiffener is defined as:

$$\gamma_{st} = \frac{EI_{st}}{Db} \quad (1)$$

where

$$D = \frac{Et_w^3}{12(1-\nu^2)} \quad (2)$$

The effective width of the stiffened panel, b , varies according to standards and researchers. Two approaches exist

- Certain authors (Stanway et al. 1996) and standards, such as AASHTO (2007) and AISC 360 (2010), propose limiting the panel width to $b = \min\{a; h_w\}$, where h_w is the web height. This ensures a conservative design, particularly for panels with aspect ratios $\alpha > 1$, for which the required γ_{st} remains constant.

- However, Lee et al. (2014) argued that considering $b = \min\{a; h_w\}$ can lead to overly conservative designs. They advocated using the entire width of the stiffened panel as $b = a$. Similarly, Kim and White (2014) found that long panels require relatively less stiffener rigidity, with no significant adverse effects.

The present investigation adopts this second approach for defining the effective width of the transverse stiffeners, adopting $b = a$ also for long panels, because for larger panel aspect ratios the tension field no longer anchors primarily in the stiffeners but develops mainly between them, anchoring in the flanges instead—a behavior previously observed by Höglund (1997) and detailed by Oliveira Pedro et al. (2024b).

Historical Background

Elastic Design Methods

The first documented reference to the use of transverse stiffeners dates to 1849, when Fairbairn reported experiments on tubular bridges. He noted that “further experiments illustrated the importance of the pillars in the sides, for, with a small addition of metal to the weight of the tube, . . . the ultimate strength was increased considerably” (Fairbairn 1849). Subsequently, Wilson (1886) highlighted the significance of transverse stiffeners in his investigation into shear postbuckling resistance. His findings on the spacing of stiffeners in plate girders contributed to the understanding of plated girder shear buckling resistance (Wilson 1898).

The concept of minimum stiffener rigidity emerged with Timoshenko and Gere (1936). This work provided data on the rigidity required for ribs and plate stiffeners to effectively stabilize a web panel and achieve its critical load. Way (1936) evaluated the stability of plates under shear and bending, correlating the rigidity of transverse stiffeners with critical buckling loads. Moisseiff (1940) proposed a minimum stiffener rigidity based on work on panel design by Chwalla and Novak (1937).

With increasing emphasis on precision in aircraft structure design, Moore (1942) conducted research on the effectiveness of stiffeners in plate girders. This resulted in an empirical method for designing transverse stiffeners based on rigidity. By 1952, the rapid growth of welded construction prompted further interest in plate girder design. The British Standards Institution Committee enlisted Kerensky to develop rules for shear stresses in girder webs, leading to revisions in BS 153 (British Standards 1958) [later renamed BS 5400-3 (British Standards 1982)]. Building on Moore’s work, Kerensky et al. (1956) proposed a slightly increased value for the minimum moment of inertia of stiffeners, setting a maximum transverse stiffener spacing of $1.5 h_w$. This proposal was adopted much later in ENV 1993-1-1:1992 (CEN 1992) and maintained in the recent EN 1993-1-5:2024 (CEN 2024a), although with a revised limitation of α from 1.5 to $\sqrt{2} \approx 1.4$.

However, the available theoretical solutions for stiffener rigidity were limited to closely spaced stiffeners, which often were impractical. To address this, Stein and Fralich (1949) conducted an experimental campaign in 1949 on 20 specimens with varying aspect ratios ($\alpha = 1, 2$, and 5). Their findings showed that as stiffener bending rigidity, γ_{st} , increased, the shear buckling coefficient, k_τ , also increased, but only up to an optimal value of γ_{st} , beyond which further increases had negligible effects. Building on this, Bleich (1952) proposed an optimal relative bending rigidity for stiffeners in infinitely long plates, a concept similar to current verification checks in AASHTO and AISC standards.

Rockey (1956) tested 220 web plate–stiffener combinations in plate girders made of aluminum, investigating the required γ_{st1} for single-sided and double-sided stiffeners. This work resulted in practical design proposals. Klöppel and Scheer (1960) developed charts

to determine the optimal rigidity for various loading and stiffening configurations, although they still focused on critical load design.

Škaloud et al. (1967) advanced this field by investigating the influence of initial out-of-plane imperfections on the required γ_{st} . Their second-order analysis of stiffened square panels under shear indicated that Moore’s rigidity proposals should be increased by a magnification factor ($m = 3$) for transverse stiffeners. Building on this work, Massonnet et al. (1968) proposed an optimal γ_{st1} to ensure that the stiffeners would not fail before the web reaches its critical shear buckling load.

Huang (1968) synthesized these investigations, performing comparative studies of the major research up to 1968. Stanway et al. (1993) investigated the impact of stiffener rigidity on the critical buckling load of simply supported plates with a single intermediate transverse stiffener. Lee et al. (2014) proposed an updated formula for the minimum moment of inertia required for intermediate transverse stiffeners to remain straight under critical web loading, refining earlier proposals by Bleich and AASHTO.

Ultimate Postbuckling Design Methods

During the second half of the twentieth century, postbuckling resistance of web plates began to be incorporated into design standards, including AISC (1963), AASHTO (1973), SIA 161 (SIA 1979), BS 5400-1 (BSI 1982), and ENV 1993-1-1 (CEN 1992). This shift prompted researchers to evaluate transverse stiffener requirements to safely sustain additional postbuckling loads up to the girder’s ultimate failure load. Two main perspectives emerged

1. Complementary strength verification: Adding a check to the minimum bending rigidity (γ_{st1}) by introducing a minimum area (A_{st}) to withstand the axial compressive forces generated by diagonal tension in the postbuckling range; and
2. Updating rigidity requirements: Revising the required bending rigidity (γ_{st2}) to account for the girder’s ultimate load, ensuring stiffeners provide sufficient rigidity beyond the elastic web buckling load.

Development of Strength Verification. Basler et al. (1960) conducted experiments evaluating the axial compression forces acting on transverse stiffeners. They proposed

- a formula for ultimate shear resistance; and
- an expression for axial forces developed in transverse stiffeners at maximum shear load (Basler 1961).

These findings influenced US standards, including AISC (1963) and AASHTO (1973), which recommended a minimum area (A_{st}) requirement based on Basler et al.’s model. This approach remained in place until revisions in 2007 and 2010, respectively.

Porter et al. (1975) proposed a design methodology for shear buckling resistance that incorporated stiffener axial force considerations, linking their behavior to the compression members of a Pratt truss. Rockey et al. (1981) conducted an experimental campaign complementing Porter et al.’s findings. They proposed a simplified verification method assuming constant axial compression along the stiffener, which subsequently was implemented in BS 5400-3 (BSI 1982). This methodology has persisted in European standards, from ENV 1993-1-1 (CEN 1992) to EN 1993-1-5 (CEN 2024a).

However, subsequent investigations using experimental and numerical methods have demonstrated that out-of-plane forces provide a more rational basis for web shear design. These studies concluded that axial forces acting on stiffeners are of secondary importance and that existing standards overestimate these effects (Rockey et al. 1981; Tang and Evans 1984; Xie and Chapman 2003; Lee et al. 2003; Johansson et al. 2007; Presta et al. 2008; Sinur and Beg 2012; Augustyn et al. 2022; Nascimento et al. 2023; Freitas et al. 2024; Nascimento et al. 2025a).

Evolution of Rigidity Requirements. Horne and Grayson (1983) conducted a parametric finite-element (FE) study of transverse stiffeners under shear, accounting for initial imperfections, residual stresses, and configurations with single- and double-sided stiffeners. Their research aimed to estimate the ultimate capacity of stiffeners.

Škaloud (1983) introduced the concept of ultimate stiffener rigidity (γ_{st2}), which ensured that stiffeners could support the web's ultimate shear resistance. This requirement included a multiplicative factor depending on web slenderness.

Stanway et al. (1996) recommended a deflection limit for stiffeners to ensure effective development of web postbuckling resistance. Lee et al. (2003) observed that for $V_{cr}/V_{pl} < 0.8$, a rigidity requirement 6 times greater than that prescribed by AASHTO (2005) was sufficient to achieve ultimate shear resistance.

Sinur et al. (2013) conducted an extensive parametric numerical study to evaluate ultimate stiffener requirements, and proposed a new formula.

Kim et al. (2007) and Kim and White (2014) used FE analysis to refine the rigidity (γ_{st2}) required for I-shaped plate girders to develop ultimate shear resistance. Their recommendations led to adjustments in US standards, specifically AASHTO (2007) and AISC (2010), which eliminated the minimum area (A_{st}) requirement in favor of a more practical stiffness-based design rule.

Standard Guidelines

Design According to EN 1993-1-5: 2024

Intermediate stiffeners designed in accordance with EN 1993-1-5 (CEN 2024a) are modeled as beam-columns subjected to axial compressive forces and lateral forces resulting from out-of-plane support of the web panels. To perform their structural function, these stiffeners must satisfy design criteria for bending rigidity, ultimate strength, and torsional rigidity [Figs. 2(a–c), respectively]. These criteria are summarized as follows:

- Bending rigidity: The moment of inertia I_{st1} of the effective cross section of the stiffener must meet or exceed the minimum values specified in Eq. (3). For web panel aspect ratios of $\alpha \geq \sqrt{2}$, the minimum rigidity becomes independent of the width a of the web panel

$$\begin{aligned} I_{st1} &\geq 1.5h_w^3 t_w^3 / a^2 \quad \text{for } a/h_w < \sqrt{2} \\ I_{st1} &\geq 0.75h_w t_w^3 \quad \text{for } a/h_w \geq \sqrt{2} \end{aligned} \quad (3)$$

- Ultimate strength: The ultimate resistance of the effective section of the stiffener is checked using the interaction formula for elements subjected to bending and axial compression as per Eq. (4)

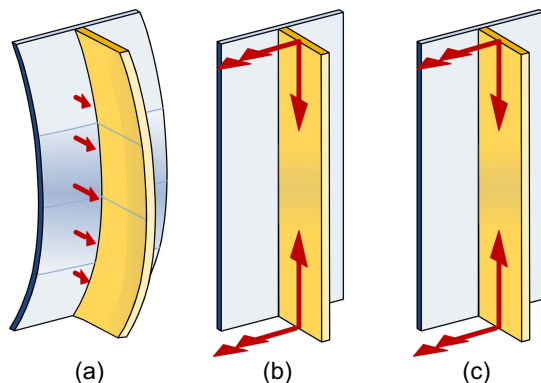


Fig. 2. Design check loads according to EN 1993-1-5.

of EN 1993-1-1 (CEN 2022). This uses the design buckling curve c and assumes a buckling length $l \geq 0.75 h_w$. In the absence of external loads, the effective axial force of the stiffener N_{st} is determined by the postbuckling tension field action of the web [Eq. (5)]. For asymmetric stiffeners, the resulting bending moment M_{st} accounts for the eccentricity, e , between the stiffener's centre of gravity and the web, plus a sinusoidal imperfection with an amplitude $\omega_0 = s/200$ [Eq. (6)] where $s = \min\{h_w; a\}$

$$\frac{N_{st}}{\chi_y N_{Rk} / \gamma_{M1}} + k_{yy} \frac{M_{st}}{\chi_{LT} M_{y,Rk} / \gamma_{M1}} \leq 1.0 \quad (4)$$

$$N_{st} = V_{Ed} - V_{cr} \quad (5)$$

$$M_{st} = N_{st}(e + \omega_0) \quad (6)$$

- Torsional rigidity: For stiffeners with little or no warping resistance, the ratio of the Saint Venant torsional constant I_T for the isolated stiffener to the polar second moment of area I_p at the point of web attachment must exceed $5.3 f_y / E$. By limiting the relative stiffener slenderness $\bar{\lambda}_{st}$ and neglecting web rotational restraint, an inferior limit to the critical stress for elastic torsional buckling, $\sigma_{cr,st}$, is obtained

$$\bar{\lambda}_{st} = \sqrt{f_{y,st} / \sigma_{cr,st}} \leq 0.70 \Leftrightarrow \sigma_{cr,st} \geq 2.04 f_{y,st} \quad (7)$$

For stiffeners with warping rigidity, the slender limit is reduced

$$\bar{\lambda}_{st} = \sqrt{f_{y,st} / \sigma_{cr,st}} \leq 0.40 \Leftrightarrow \sigma_{cr,st} \geq 6.25 f_{y,st} \quad (8)$$

The effective cross section of the transverse stiffener is defined in EN 1993-1-5 (CEN 2024a) as the gross stiffener area plus the effective width of the web plate, given by $15\epsilon t_w$ on each side of the stiffener (where $\epsilon = \sqrt{235/f_{y,w}}$) (Fig. 3).

Design According to AASHTO (2024) and AISC 360:2022

AASHTO (2024) and AISC (2022) define the required moment of inertia I_{st1} [Eq. (9)] as the rigidity needed to maintain near-zero lateral deflection along the vertical axis of the stiffener during the occurrence of the critical web shear stress, τ_{cr} . This ensures adequate lateral support to the web panel.

Additionally, Eq. (10) specifies the minimum bending rigidity (I_{st2}) when the ultimate shear stress, τ_u , of the web panel is considered in the design, accounting for the postbuckling strength of the web. For stocky webs, I_{st2} converges toward I_{st1} , reflecting the reduced significance of postbuckling effects in these cases

$$I_{st1} \geq b t_w^3 J \quad \text{and} \quad J = \frac{2.5}{(a/h_w)^2} - 2.0 \geq 0.5 \quad (9)$$

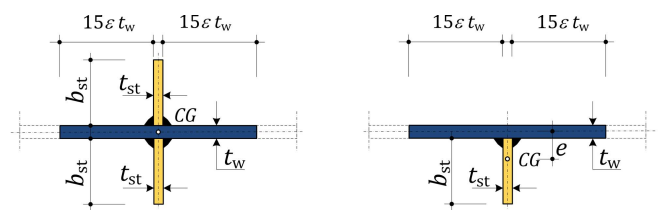


Fig. 3. Transverse stiffener effective cross section according to EN 1993-1-5.

$$I_{t2} \geq \frac{h_w^4 \rho_t^{1.3}}{40} \left(\frac{f_{y,w}}{E} \right)^{1.5} \quad \text{and} \quad b = \min\{a; h_w\};$$

$$\rho_t \max(f_{y,st}/\sigma_{cr,st}, 1.0); \quad \sigma_{cr,st} \frac{0.31 E}{(b_{st}/t_{st})^2} \leq f_{y,st} \quad (10)$$

These standards recommend that the transverse stiffener cross section for the moment of inertia calculation is taken about the edge in contact with the web for single-sided stiffeners and about the midthickness of the web for double-sided stiffeners, disregarding any effective web participation.

Numerical Model and Validation

Geometry, Loading, and Boundary Conditions

The numerical model represents a symmetric, four-panel I-shaped plate girder. Two intermediate web panels were included to investigate the behavior of an intermediate transverse stiffener and the evolution of shear buckling resistance.

To evaluate the effectiveness of the stiffeners under pure shear loading, the required rigidity (γ_{st}) of a single-sided flat intermediate transverse stiffener necessary to achieve the postbuckling resistance was assessed. The flanges were modeled explicitly to maximize the shear buckling resistance of the FE model. Moreover, as recommended by Wang et al. (2021), plate models under pure shear should include explicit elements for flanges, stiffeners and end web panels, for the best correlation with experimental results (Fig. 4). This configuration more accurately reproduces the actual rotation and displacement restraints of the web panels edges.

Furthermore, end-post stiffeners consisted of double-sided flat transverse stiffeners. Additionally, the potential use of longitudinal stiffeners was investigated.

The simulations were performed using Abaqus software v6.14, with automatic model generation via Python scripting. Postprocessing and analysis were carried out using a custom MATLAB program (MATLAB version 9.8.0 R2020a) (Dassault Systèmes 2014).

A linear S4R-type three-dimensional shell element with four nodes was employed. This element combines first-order interpolation with reduced integration. To optimize numerical accuracy and computational efficiency, a mesh convergence study was conducted, recommending a maximum element size of approximately 40×40 mm (equivalent to about $h_w/25$). The Simpson integration rule was applied with five sectional points to evaluate the shell's cross-sectional behavior.

Two numerical analyses were conducted using the same FE model: (1) linear bifurcation analysis (LBA) was used to evaluate

local geometric imperfections; and (2) geometric and material nonlinear analysis (GMNIA), with web and transverse stiffener imperfections to determine the resistance of each plate girder and the corresponding internal forces in the stiffeners.

The arc-length method (static Riks method) was employed to solve the system of nonlinear differential equations. Load control using static shell edge loads was applied to capture unstable collapse behavior and postbuckling response.

The boundary conditions were selected carefully to simulate pure shear stress state on the plate girder accurately (Fig. 4). The geometry, boundary conditions, and mesh parameters aligned with recent research in the field of plate structures (Wang et al. 2021; Oliveira Pedro et al. 2024a).

Material Model

The study investigated the effects of S355 and S690 structural steels, with the material model adopting a stress-strain relationship that assumes identical behavior in tension and compression under static loading, as recommended in prEN 1993-1-14 (CEN 2024b).

An elastoplastic material model was used: (1) S355 employed a strain-hardening quad-linear material model with a Young's modulus, E , of 210 GPa; and (2) S690 utilized a two-stage Ramberg-Osgood model following an analysis of the influence of material models. The material properties for S355 and S690 steels are based on their nominal values and are detailed in Table 1 and Fig. 5 (CEN 2022; CEN 2024b). For the numerical simulations, the von Mises yield criterion was adopted.

Geometric Imperfections

To conduct the numerical studies, equivalent initial geometric imperfections were introduced in accordance with prEN 1993-1-14 (CEN 2024b). These imperfections included eigenmode-based deformations and manually defined sinusoidal functions. No imperfections were applied to the flanges or end-post stiffeners. The initial imperfections were modeled as a combination of the following:

1. Local panel or subpanel imperfections—The first eigenmode buckling shape from a linear bifurcation analysis under pure shear stress was used (Fig. 6). The maximum amplitude was defined as $\min\{a/200; h_w/200\}$.
2. Global out-of-plane sinusoidal imperfections—A half-wave sinusoidal function was applied to the intermediate transverse stiffeners and longitudinal stiffeners (Fig. 6). The maximum amplitude was $h_w/400$.

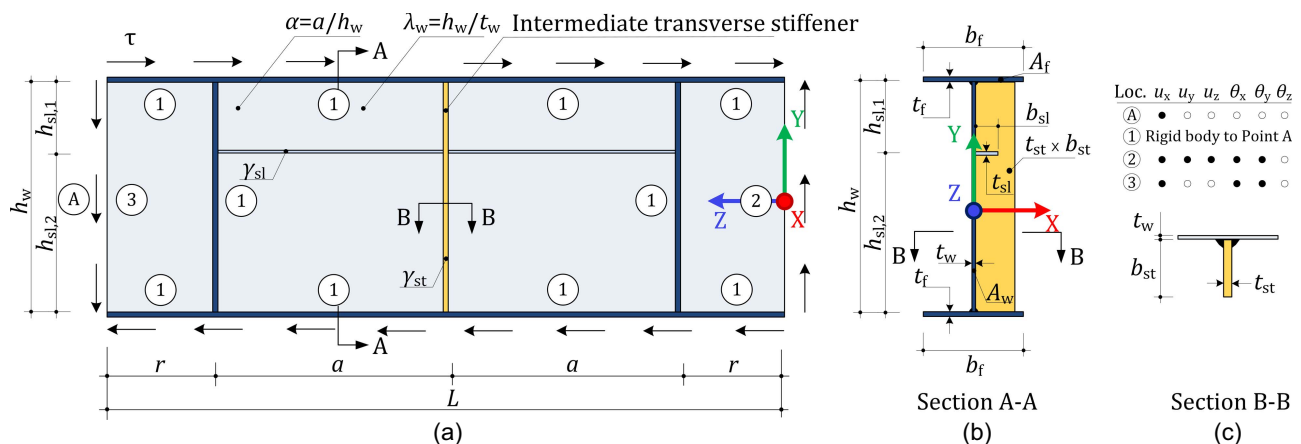
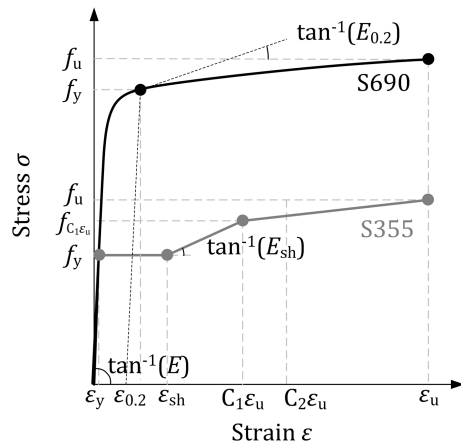
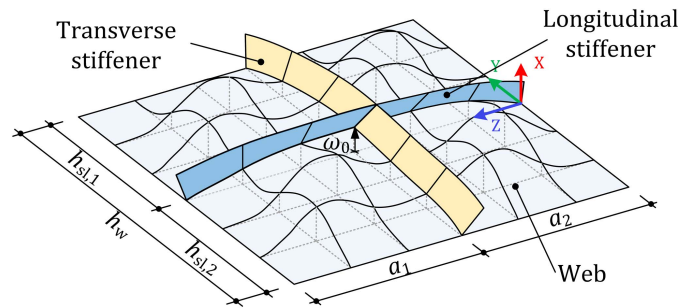


Fig. 4. Geometrical parameters of the plate girder model: (a) longitudinal view; (b) Section A-A; and (c) Section B-B.

Table 1. Material characteristics for modeling S355 and S690

Steel	E (GPa)	E_{sh} (GPa)	f_y (MPa)	$f_{C1\epsilon_u}$ (MPa)	f_u (MPa)	ϵ_y (%)	ϵ_{sh} (%)	$C_1\epsilon_u$ (%)	$C_2\epsilon_u$ (%)	$\epsilon_{0.2}$ (%)	ϵ_u (%)
S355	210	2.283	355	439.4	490	0.17	1.74	5.44	7.66	—	16.5
S690	210	—	690	—	770	—	—	—	—	0.53	14.0

Source: Data from CEN (2024b).

**Fig. 5.** Stress–strain material models for S355 and S690 steels.**Fig. 6.** Schematic initial geometric imperfections for the web and stiffeners.

- Local twist and rotation imperfections—A rotational imperfection was applied to the intermediate transverse stiffener and longitudinal stiffeners. This was modeled as a maximum twist amplitude of $1/50$ (Fig. 6).

A sensitivity analysis was conducted to evaluate the effects of amplitude and shape on the local panel or subpanel imperfection. The numerical resistance showed negligible sensitivity to changes in the amplitude of geometric imperfections when using a half-wave sinusoidal function (Table 2). Conversely, the first eigenmode-shaped web imperfection and an amplitude of $h_w/200$ were found the most appropriate amplitude when evaluating the ratio V_{FEM}/V_{pl} . This is detailed in Section “Model Validation” and expanded in Nascimento’s (2025) research.

However, due to the large number of potential combinations, leading and accompanying imperfections were not considered. Instead, the maximum amplitudes of Imperfections 1–3 were applied, ensuring that the numerical resistance values for the girders remained on the conservative side.

Failure Criterion

In line with prEN 1993-1-14 (CEN 2024b), the ultimate shear resistance, V_{ult} , was determined based on the smaller value from two

Table 2. Statistical assessment of V_{FEM}/V_{pl} ratio for equivalent local panel or subpanel imperfection

Local panel or subpanel imperfection shape	Imperfection amplitude	FE model	
		b	COV (%)
First shear buckling mode	$h_w/100$	0.988	2.60
	$h_w/200$	1.020	2.41
	$h_w/400$	1.037	2.97
Sine semiwave	$h_w/100$	1.053	3.20
	$h_w/200$	1.057	3.66
	$h_w/400$	1.057	4.12

criteria: (1) ultimate load level, i.e., the peak shear load from the load–deformation curve; and (2) the shear load corresponding to the maximum allowable equivalent strain of $\epsilon_{lim} = 5\%$ for plated structures (CEN 2024a).

Model Validation

The FE model was validated by (1) replicating Nascimento et al.’s (2025b) experimental tests, and (2) comparing the shear resistance values obtained from FE simulations, V_{FEM} , with the shear buckling resistance design curve, V_{code} , as defined in EN 1993-1-5 (CEN 2024a).

This comparison focused on squared web panels and a wide range of relative web slenderness values $0.5 \leq \bar{\lambda}_w \leq 5.0$. For this validation, the FE model configuration (Fig. 4) included a single intermediate web panel and evaluated the shape and amplitude of Imperfection 1 (applied in a local panel or subpanel), disregarding the influence of intermediate transverse stiffener (thus no Imperfections 2 and 3). The effects of flanges and longitudinal stiffeners were also excluded to isolate the web’s shear response.

The calibration results are shown in Fig. 7, and a statistical assessment of the ratio V_{FEM}/V_{pl} is summarized in Table 2, considering different imperfection shapes and amplitudes. The findings indicate that the most representative and stable results were obtained using the first eigenmode-shaped imperfection with an amplitude of $h_w/200$, which yielded an average ratio $b = 1.020$ and the lowest coefficient of variation (COV = 2.41%).

This imperfection configuration aligns with established recommendations in CEN (2024a) and the literature (Oliveira Pedro et al. 2024a, 2025), and therefore was adopted in the FE models for conducting parametric studies.

Although it is recognized that the imperfection pattern and amplitude can influence the ultimate load shear resistance—a difference of about 5% in V_{FEM} between Imperfection 1 amplitudes of $h_w/100$ and $h_w/400$ —the selected configuration was shown to reproduce experimental behavior accurately (Nascimento et al. 2025b) and to yield conservative results across the investigated slenderness range, thereby providing confidence that the adopted FE model remains reliable for other girder configurations within similar geometric and loading conditions.

Parameter Range

To address all relevant design scenarios, extensive parametric numerical research was conducted. The parameters considered and

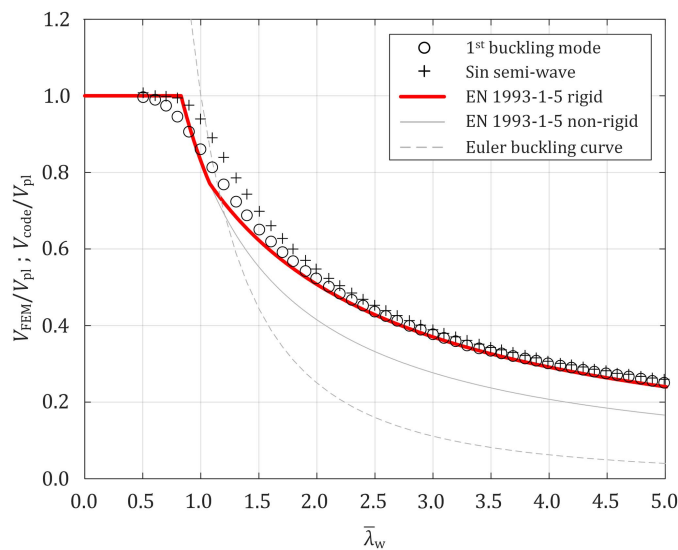


Fig. 7. Ratio V_{FEM}/V_{pl} for FE models with equivalent imperfections (first buckling mode and sin semiwave) versus relative web slenderness and comparison with the EN 1993-1-5 shear buckling design curve.

their respective ranges are summarized in Table 3. The web height was fixed at $h_w = 1,000$ mm, resulting in 13,500 different plate girder geometries. A single-sided flat open section was adopted for the longitudinal stiffener, with a relative rigidity of $\gamma_{sl} = 100$, consistent with EN 1993-1-5 rules. The open section avoided added complexity of closed sections and ensured that no premature stiffener failure influenced the shear buckling response.

However, plate girders with nonsymmetric flanges commonly are designed, in steel structures and particularly in steel–concrete composite structures (Numanović and Knobloch 2024). Because the bending rigidity of transverse stiffeners is dependent on the A_f/A_w ratio, it is important to explore the effect of adopting large flanges. Therefore, the present study evaluated Class 2 cross-section flanges (CEN 2022) with dimensions ranging from $A_f/A_w = 0.25$ to 2.50.

Numerical Results

Research Strategy

The effectiveness of the intermediate transverse stiffener was assessed by adjusting the stiffener cross-section rigidity γ_{st} for a given girder geometry. The resulting numerical shear resistance then was evaluated based on the following specific criteria.

To prevent torsional failure of the stiffener, a stiffener plate width-to-thickness ratio of $b_{st}/t_{st} = 10$ was selected. Consequently, only the bending rigidity of the stiffener was investigated numerically. The chosen relative rigidity was matched with the formula for the moment of inertia of the stiffener's effective section, considering an effective web width acting together with the outstanding stiffener plate. This allows for the definition of the intermediate transverse stiffener's dimensions, b_{st} and t_{st} .

The selected width-to-thickness ratio for the stiffener was based on the geometrical guidelines in EN 1993-1-5 to avoid torsional buckling (i.e., $b_{st}/t_{st} \leq 12.56\epsilon$) (CEN 2024a). This approach is consistent with the research by Kim et al. (2007) and Kim and White (2014), which supports the current AASHTO and AISC stiffener design standards.

Table 3. Numerical model parameter range

Parameter	Adopted values	Total
$\alpha = a/h_w$	1.0, 1.5, 2.0, 2.5, 3.0	5
$\lambda_w = h_w/t_w$	250, 167, 125, 100, 83	5
A_f/A_w	0.25, 0.5, 0.75, 1.0, 1.25, 1.5, 1.75, 2.0, 2.25, 2.5	10
$\gamma_{st,end}$	4,000	1
γ_{st}	$\approx 0, 5, 10, 25, 50, 100, 250, 500, 1,000$	9
b_{st}/t_{st}	10	1
γ_{sl}	100	1
b_{sl}/t_{sl}	10	1
$h_{sl,1}$	0, $0.25h_w$, $0.50h_w$	3
f_y (MPa)	355, 690	2

Minimum Rigidity Criterion

By adjusting its cross-sectional bending rigidity, γ_{st} , the influence of a single-sided flat intermediate transverse stiffener was evaluated numerically using the ratio $V_{max,FEM}/V_{pl}$, in which the numerical shear buckling resistance is denoted $V_{max,FEM}$.

Fig. 8(a) plots the normalized FE shear resistance obtained, $V_{max,FEM}/V_{pl}$, for several stiffener bending rigidities, γ_{st} , which are represented on the horizontal axis. These results were interpolated, obtaining the solid line which represents the increase in girder resistance when a stronger transverse stiffener is adopted. The following principle is outlined:

$$\text{Criterion 1 (C1): } V_{max,FEM}(i)/V_{max,FEM}(i+1) < 0.1\% \quad (11)$$

which corresponds to a variation in girder resistance of less than 0.1‰ between two consecutive values of γ_{st} is the value at which the increase in relative resistance $V_{max,FEM}/V_{pl}$ becomes negligible with further increases in γ_{st} [Fig. 8(a)]. In other words, after a certain value of γ_{st} is reached, the relative numerical shear resistance changes very little with further increases in stiffener size, leading to a plateau in the ratio $V_{max,FEM}/V_{pl}$. A similar criterion was used by Xie (2001), Kim et al. (2007), and Kim and White (2014), although their reference shear buckling resistance was based on a characteristic value, $V_{b,Rk}$.

Fig. 8(a) also illustrates the $V_{max,FEM}/V_{pl}$ results for two plate girder examples (longitudinally unstiffened S355; $\lambda_w = 167$, $\alpha = 1$, and $A_f/A_w = 1.0$): one with a flexible transverse stiffener ($\gamma_{st} = 5$), which did not reach the maximum web panel resistance ($\approx 0.76V_{pl}$), and another with a rigid stiffener ($\gamma_{st} = 1,000$), which successfully achieved this value.

The behavior of the two plate girders examples is supported by the normalized load versus vertical displacement plot of Fig. 8(b) and the corresponding numerical representation of the failure mode in Fig. 9. Additionally, $V_{max,ref,FEM}$ represents the reference shear resistance obtained from a similar model with only one intermediate web panel [Fig. 9(a)].

Moreover, Fig. 8(a) presents the EN 1993-1-5 characteristic shear buckling resistance, $V_{b,Rk,\alpha}$, for a plate girder with a web panel stiffened with a rigid intermediate transverse stiffener. As a lower-bound reference in accordance with EN 1993-1-5, the characteristic shear buckling resistance of an unstiffened plate girder, i.e. without intermediate transverse stiffeners, was evaluated and is shown in the same figure as $V_{b,Rk,2\alpha}$ (using the web panel length L and aspect ratio of 2).

To validate the results obtained using Criterion 1, an additional minimum resistance criterion was applied, Criterion 2 (C2), which examines the influence of the intermediate transverse stiffener using the ratio $V_{max,FEM}/V_{max,ref,FEM}$, which denotes the reduction in shear capacity if a transverse stiffener is added comparing with the resistance of a reference FE model with a single web panel, $V_{max,ref,FEM}$. The following criteria are defined:

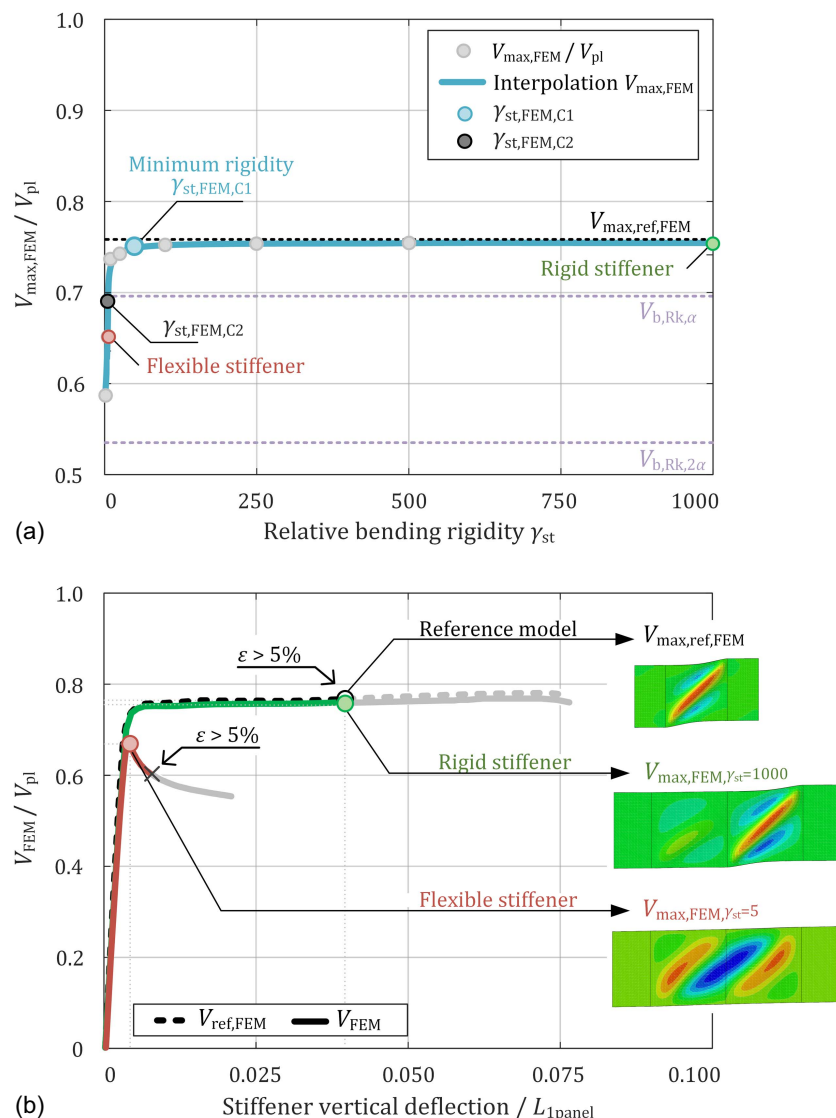


Fig. 8. Definition of the effectiveness criterion for an intermediate transverse stiffener: (a) definition of $\gamma_{st,FEM}$; and (b) load–displacement plot of two examples.

- A ratio $V_{max,FEM}/V_{max,ref,FEM} < 0.90$ indicates that the ultimate resistance of the girder with the intermediate transverse stiffener is lower than the corresponding reference girder. This suggests that the transverse stiffener is ineffective, because it may not prevent shear buckling, resulting in failure of both web panels [Figs. 8(b) and 9(c)].
- A ratio $0.90 \leq V_{max,FEM}/V_{max,ref,FEM} \leq 1.00$ indicates that a shear capacity reduction of as much as 10% is acceptable for an intermediate transverse stiffener that allows the full ultimate shear resistance of each web panel without stiffener failure [Figs. 8(b) and 9(b)].

Both Criteria 1 and 2 results are plotted in Fig. 8(a). The recommended $\gamma_{st,FEM}$ is the highest of the two values.

Required Bending Rigidity

Effect of Steel Grade

The effect of the steel grade f_y on the required relative stiffener rigidity $\gamma_{st,FEM}$ is shown in Figs. 11–15 for panels with aspect ratios $\alpha = 1.0, 1.5, 2.0, 2.5$, and 3.0 , respectively. Cases of webs with and without longitudinal stiffeners are considered. The obtained $\gamma_{st,FEM}$

values, determined using the defined minimum rigidity criterion, are presented as scatter points, expressed as a function of the web slenderness λ_w and the flange-to-web relative area ratio A_f/A_w . To provide a clearer understanding of the required stiffness behavior, a thin-plate spline interpolation surface also is included.

Adopting a higher steel grade, such as S690, than S355 steel, did not significantly impact $\gamma_{st,FEM}$. The maximum observed difference of approximately 32% occurred for the highest slender panels studied, with $\lambda_w = 250$ and strong flanges relative to the web ($A_f/A_w = 2.5$). In this case, the $\gamma_{st,FEM}$ values for S355 and S690 were 688 and 522, respectively. These findings indicate that the influence of the steel grade on the required relative stiffener rigidity $\gamma_{st,FEM}$ is less significant than suggested by the AASHTO requirements for $\gamma_{st,2}$.

Effect of Aspect Ratio

The influence of the web panel aspect ratio α was investigated within the analyzed range ($\alpha = 1.0, 1.5, 2.0, 2.5$, and 3.0). The numerically obtained required relative stiffener rigidity $\gamma_{st,FEM}$ is presented in Figs. 11–15 for each aspect ratio (using the same vertical axis scale for comparison), considering webs both with and without longitudinal stiffeners.

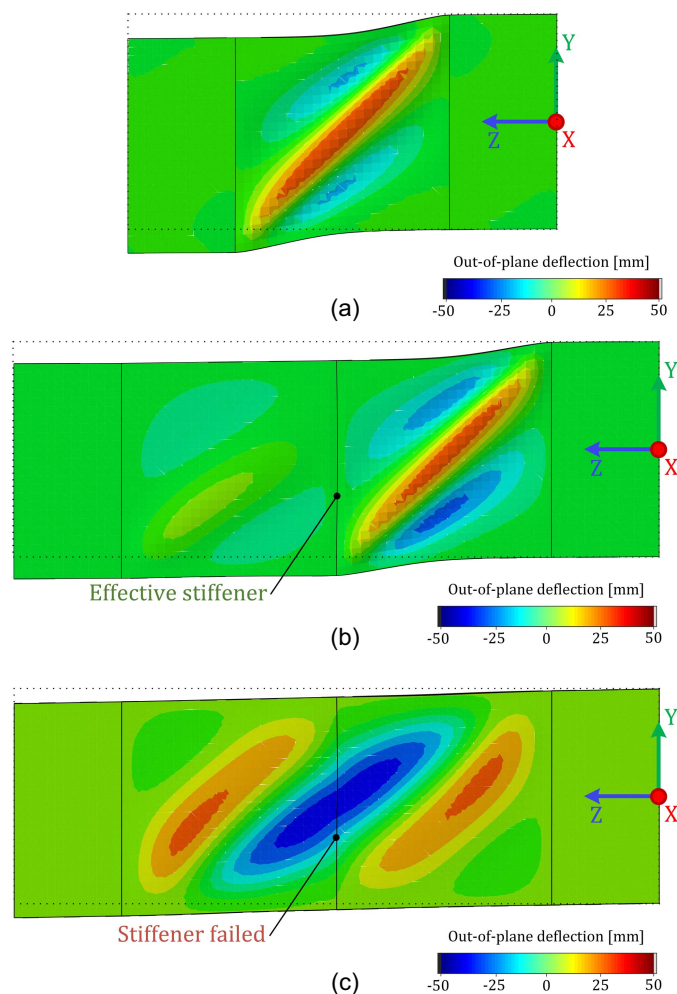


Fig. 9. FE model simulated out-of-web displacement for $V_{\max,FEM}$ for case with $\alpha = 1$, $\lambda_w = 167$, $A_f/A_w = 1.0$, S355, and no longitudinal stiffener for (a) reference model; (b) transverse stiffener $\gamma_{st} = 1,000$; and (c) transverse stiffener $\gamma_{st} = 5$.

As expected, increasing the aspect ratio significantly reduced the bending demands of the tension field action on the intermediate transverse stiffener. Between $\alpha = 1.0$ and 1.5 , the highest required $\gamma_{st,FEM}$ decreased by approximately 50%—from 688 to 363 for S355, and from 522 to 242 for S690. This reduction occurred for panels with $\lambda_w = 250$ and $A_f/A_w = 2.5$. For longer panels ($\alpha \geq 2.5$), the required $\gamma_{st,FEM}$ decreased to less than 100. In the case of very long panels ($\alpha = 3.0$), the maximum $\gamma_{st,FEM}$ was as low as 30.

Two representative girder examples are presented to illustrate the influence of the panel aspect ratio. Following the example in Section “Minimum Rigidity Criterion,” the models corresponded to $\lambda_w = 167$, $A_f/A_w = 1.0$, Steel grade S355, no longitudinal stiffener, and aspect ratios $\alpha = 1$ and 3 . A rigid intermediate transverse stiffener ($\gamma_{st} = 1,000$) was adopted in both cases. The streamline and principal tensile stress contour plots in Figs. 10(a and b) demonstrate that, at $V_{\max,FEM}$, increasing the aspect ratio caused the tension field to shift away from the transverse stiffeners and anchor predominantly in the flanges, thereby making the stiffeners less engaged in resisting shear. Fig. 10(c) presents the corresponding load–displacement response and the observed failure mode.

Effect of Web Slenderness

Web slenderness is a critical parameter in determining the numerically required relative stiffener rigidity $\gamma_{st,FEM}$. Its influence was

investigated, and the corresponding $\gamma_{st,FEM}$ values are presented in Figs. 11–15 for Steel grades S355 and S690, aspect ratios $\alpha = 1.0$ – 3.0 , and webs with and without longitudinal stiffeners.

The effect of web slenderness was more pronounced when strong flanges were adopted. For stocky webs, in which inelastic or plastic buckling governs the failure mode, the stiffener demands up to the calculated maximum shear resistance $V_{\max,FEM}$ (based on a stopping criterion of 5% strain) were minimal. Conversely, for very slender webs, for which postbuckling tension field action dominates, the required $\gamma_{st,FEM}$ increased significantly.

Effect of Flange Area

Within the parameter range $A_f/A_w = 0.25$ – 2.50 , the relative flange area compared with that of the web was found to significantly influence the required relative stiffener rigidity $\gamma_{st,FEM}$. Figs. 11–15 illustrate this effect, showing the numerically obtained $\gamma_{st,FEM}$ for Steel grades S355 and S690, aspect ratios $\alpha = 1.0$ – 3.0 , and webs with and without longitudinal stiffeners.

Up to $A_f/A_w = 1.00$, the required $\gamma_{st,FEM}$ remained nearly constant. However, above $A_f/A_w = 1.00$, the flange significantly increased the rigidity demands of the intermediate transverse stiffener. For example, in the case of a slender web ($\lambda_w = 250$) made of S355 with a square web panel ($\alpha = 1.0$), $\gamma_{st,FEM}$ increased from 96 at $A_f/A_w = 1.00$ to 688 at $A_f/A_w = 2.50$, an increase of 7 times. Similarly, for the same scenario using S690, $\gamma_{st,FEM}$ increased by 8.6 times, from 61 to 522. It also was found that the influence of the flange relative area decreased as the web panel aspect ratio increased.

Effect of Longitudinal Stiffener Adoption

The impact of adopting a single-sided open-section longitudinal stiffener with $\gamma_{sl} = 100$ was analyzed, and the numerically obtained required relative stiffener rigidity $\gamma_{st,FEM}$ is presented in Figs. 11–15. Results are shown for longitudinally unstiffened webs and webs with longitudinal stiffeners positioned at $h_{sl,1} = 0.25h_w$ and $0.50h_w$ from the top flange. These findings are provided separately for Steel grades S355 and S690, with aspect ratios $\alpha = 1.0$ – 3.0 , and were evaluated as functions of web slenderness λ_w and flange relative area A_f/A_w .

When a longitudinal stiffener was positioned at $h_{sl,1} = 0.25h_w$, the required $\gamma_{st,FEM}$ decreased slightly compared with that for an unstiffened web. For the extreme case of a slender web ($\lambda_w = 250$) made of S355 with a square web panel ($\alpha = 1.0$) and $A_f/A_w = 2.50$, $\gamma_{st,FEM}$ decreased from 688 to 500, a 38% reduction in the stiffener rigidity demands; the reduction for S690 was approximately 9%.

When the longitudinal stiffener was positioned at $h_{sl,1} = 0.50h_w$, $\gamma_{st,FEM}$ decreased slightly compared with that for a web without a longitudinal stiffener. For the most critical case of a slender web ($\lambda_w = 250$) made of S355 with a square web panel ($\alpha = 1.0$) and strong flanges ($A_f/A_w = 2.50$), $\gamma_{st,FEM}$ decreased from 688 to 497, a 5% reduction in stiffener rigidity demands; the reduction for S690 was approximately 10%.

Overall, the use of longitudinal stiffeners in conjunction with transverse stiffeners appears to be justified only for plate girders with slender webs and high aspect ratios, particularly when the flanges are relatively strong. Longitudinal stiffeners help to enhance the local web shear buckling resistance and limit out-of-plane deformations of very slender and long webs (Oliveira Pedro et al. 2025).

Additional Effects

Overview

In steel plate girder design, various geometric configurations other than those parametrically analyzed may be used due to specific structural requirements. Although the typical scenarios

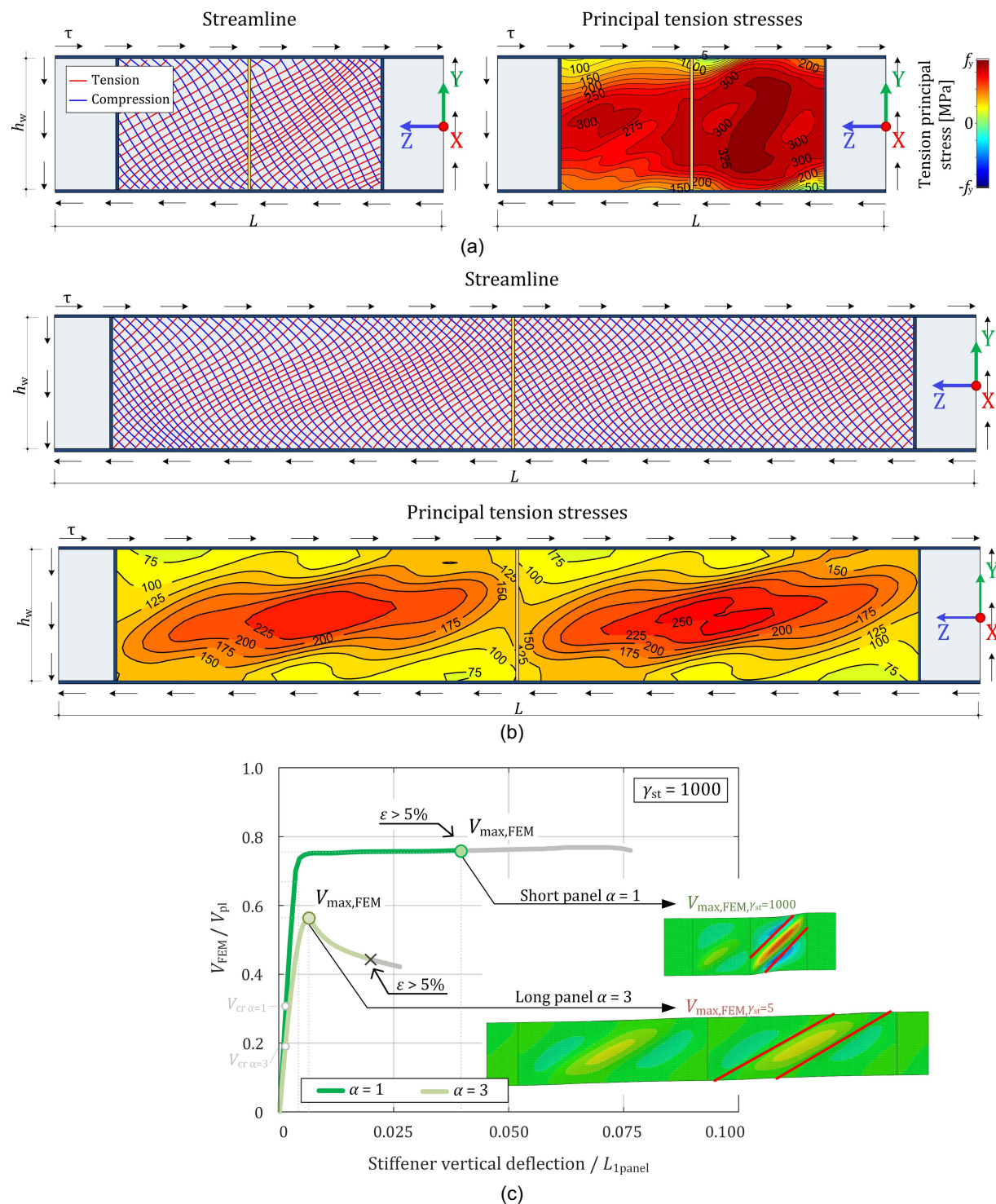


Fig. 10. (a) Effect of aspect ratio using streamline plot and membrane principal tension stresses for $\alpha = 1$; (b) effect of aspect ratio using streamline plot and membrane principal tension stresses for $\alpha = 3$; and (c) load-displacement plot of two examples.

investigated—symmetrical girders subjected to pure shear with single-sided flat intermediate transverse stiffeners—provide insight into stiffener requirements, additional cases were explored to account for other design scenarios: (1) double-sided intermediate transverse stiffeners; (2) intermediate transverse stiffeners with a T-section layout; (3) nonsymmetrical plate girders with different top and bottom flange dimensions; and (4) bending moment and shear loading (M - V interaction).

To analyze these cases, the parameters most influencing the required relative stiffener rigidity $\gamma_{st,FEM}$ were selected: $\alpha = 1$,

$\lambda_w = 250$, $A_f/A_w = 1.0$ and 2.0 , S355, and no longitudinal stiffeners. The full range of $\gamma_{st} = 1$ – $1,000$ was investigated. The evaluation followed the methodology outlined in Section “Minimum Rigidity Criterion” to assess the impact on ultimate stiffener rigidity demands.

Effect of Double-Sided Stiffeners

The influence of double-sided intermediate transverse stiffeners was studied using the selected parameters, and the geometric properties of the stiffeners calculated are shown in Fig. 3. The numerical

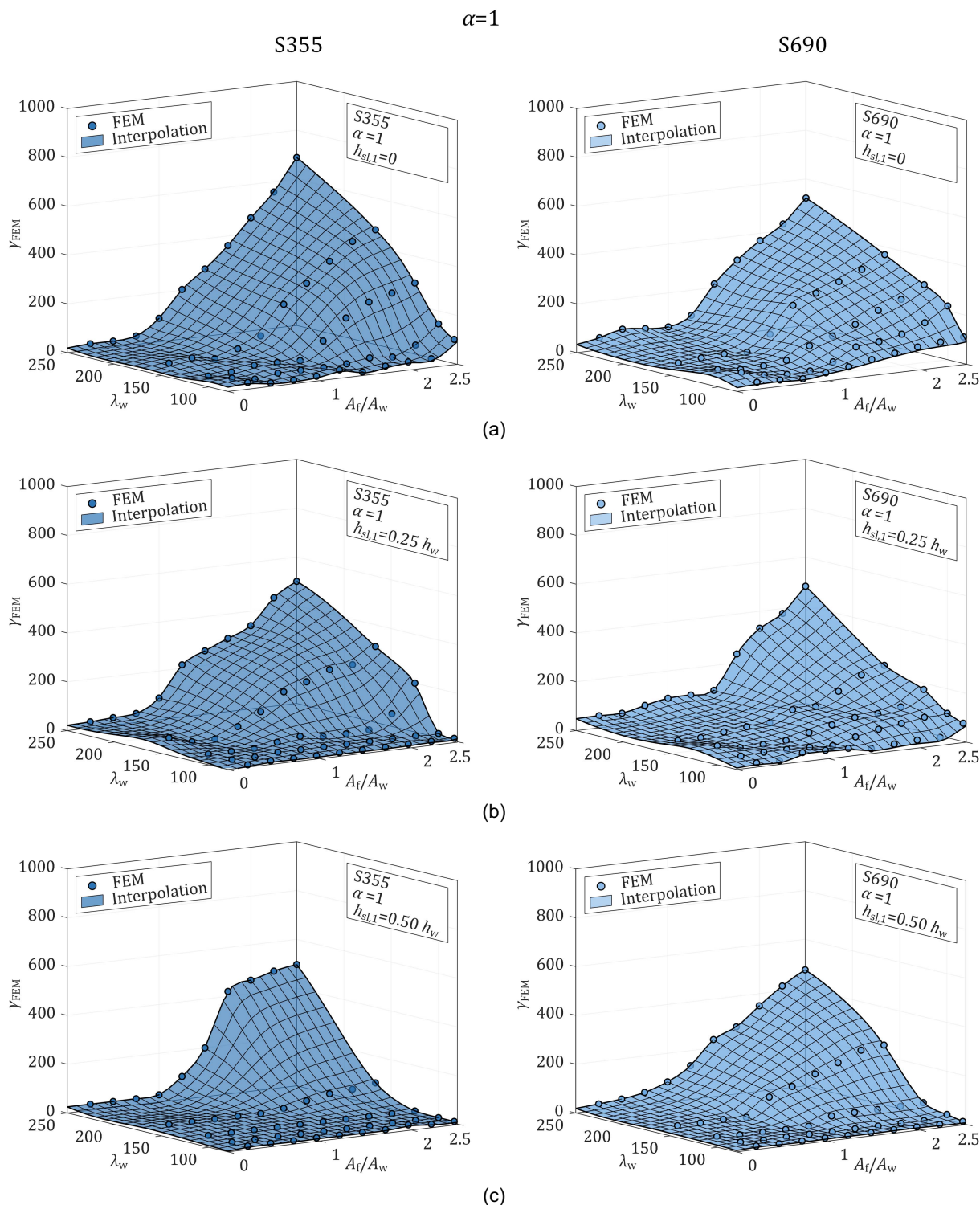


Fig. 11. Effect of steel grade on the FEM required relative stiffener rigidity $\gamma_{st,FEM}$ for $\alpha = 1.0$: (a) without longitudinal stiffener; (b) longitudinal stiffener $h_{sl,1} = 0.25h_w$; and (c) longitudinal stiffener $h_{sl,1} = 0.50h_w$.

requirement $\gamma_{st,FEM}$ was determined for double-sided stiffeners based on the $V/V_{pl}-\gamma_{st}$ relationship in Fig. 16.

These results indicate that the required rigidity $\gamma_{st,FEM}$ is not influenced by the adoption of double-sided stiffeners compared with single-sided configurations. Both types of stiffeners effectively restrained web out-of-plane displacements, provided they achieved the same relative rigidity γ_{st} as defined in EN 1993-1-5. Consequently, the choice of stiffener symmetry (single- or double-sided)

does not affect the ultimate resistance of the girder, as long as the required γ_{st} is met. This finding aligns with the conclusions of Kim et al. (2007) and Kim and White (2014).

Effect of T-Stiffener Layout

In plate girder design, it is common to use transverse stiffeners with cross-section layouts other than flat plates. Among these alternatives, the T-stiffener layout is widely adopted due to its optimized

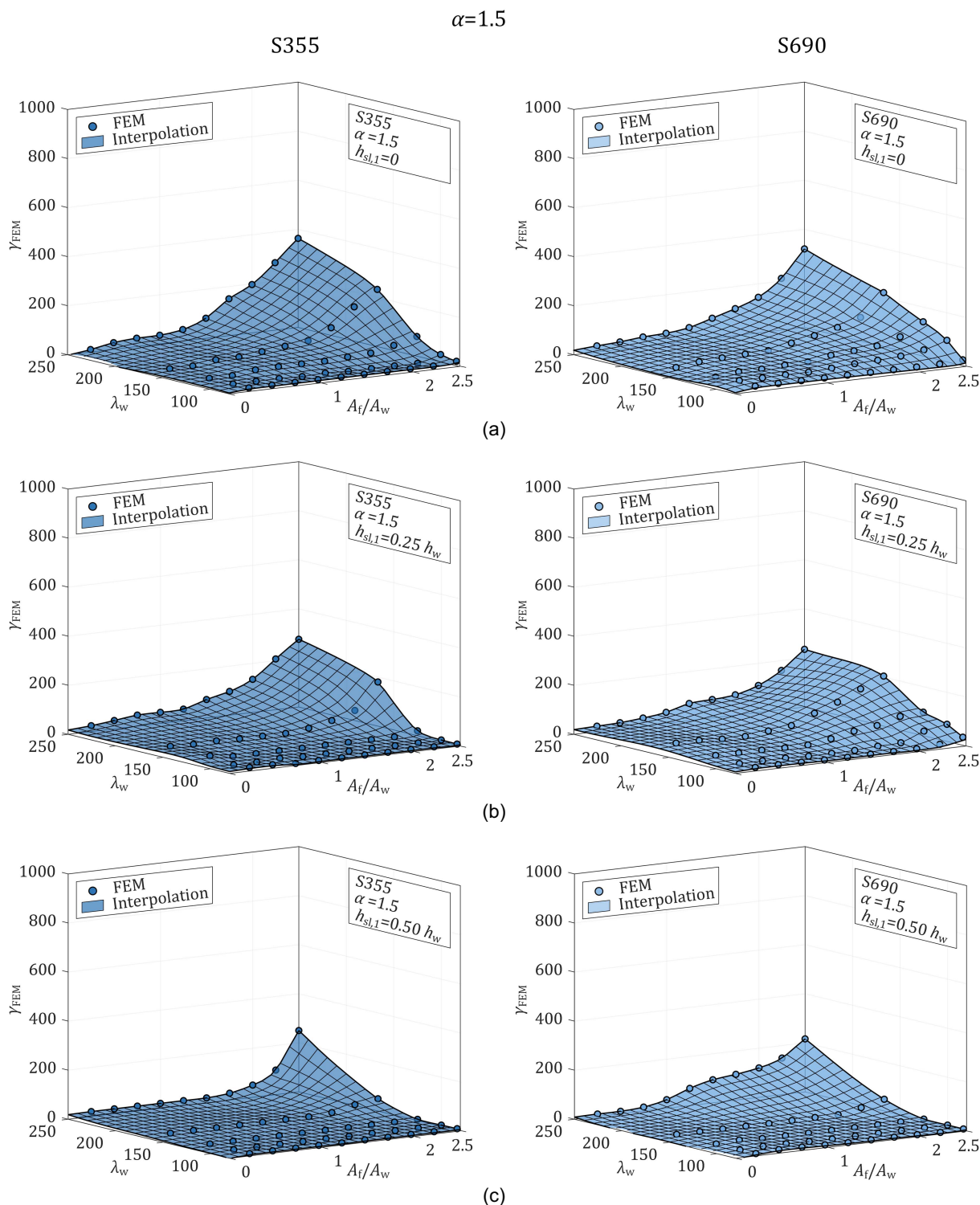


Fig. 12. Effect of steel grade on the FEM required relative stiffener rigidity $\gamma_{st,FEM}$ for $\alpha = 1.5$: (a) without longitudinal stiffener; (b) longitudinal stiffener $h_{sl,1} = 0.25h_w$; and (c) longitudinal stiffener $h_{sl,1} = 0.50h_w$.

cross-sectional area distribution, which enhances the moment of inertia. This configuration not only provides higher rigidity but also is better suited for external loading from vertical supports, transfer beams, and lateral bracing cross girders.

To evaluate the influence of a single-sided T-stiffener layout, the two configurations illustrated in Fig. 17 were analyzed. Additionally, the impact of a bottom flange cut-out detail was investigated, with dimensions of $h_{st,1} = 870$ mm and $h_{st,2} = 80$ mm, and a cut-out depth of 15 mm.

The numerical study was conducted using $\alpha = 1$, $\lambda_w = 250$, $A_f/A_w = 1.0$ and 2.0 , and S355 steel with no longitudinal stiffeners. The EN 1993-1-5 torsional requirements were applied to ensure $\bar{\lambda}_{st} \leq 0.40$. The following constraints were imposed:

- the flange width-to-thickness ratio $b_{st,f}/t_{st,f}$ was limited to 10ϵ to ensure a Class 2 cross section under compression;
- the flange width $b_{st,f}$ was $2.5b_{st}$;
- the web width-to-thickness ratio b_{st}/t_{st} was restricted to a maximum of 20; and

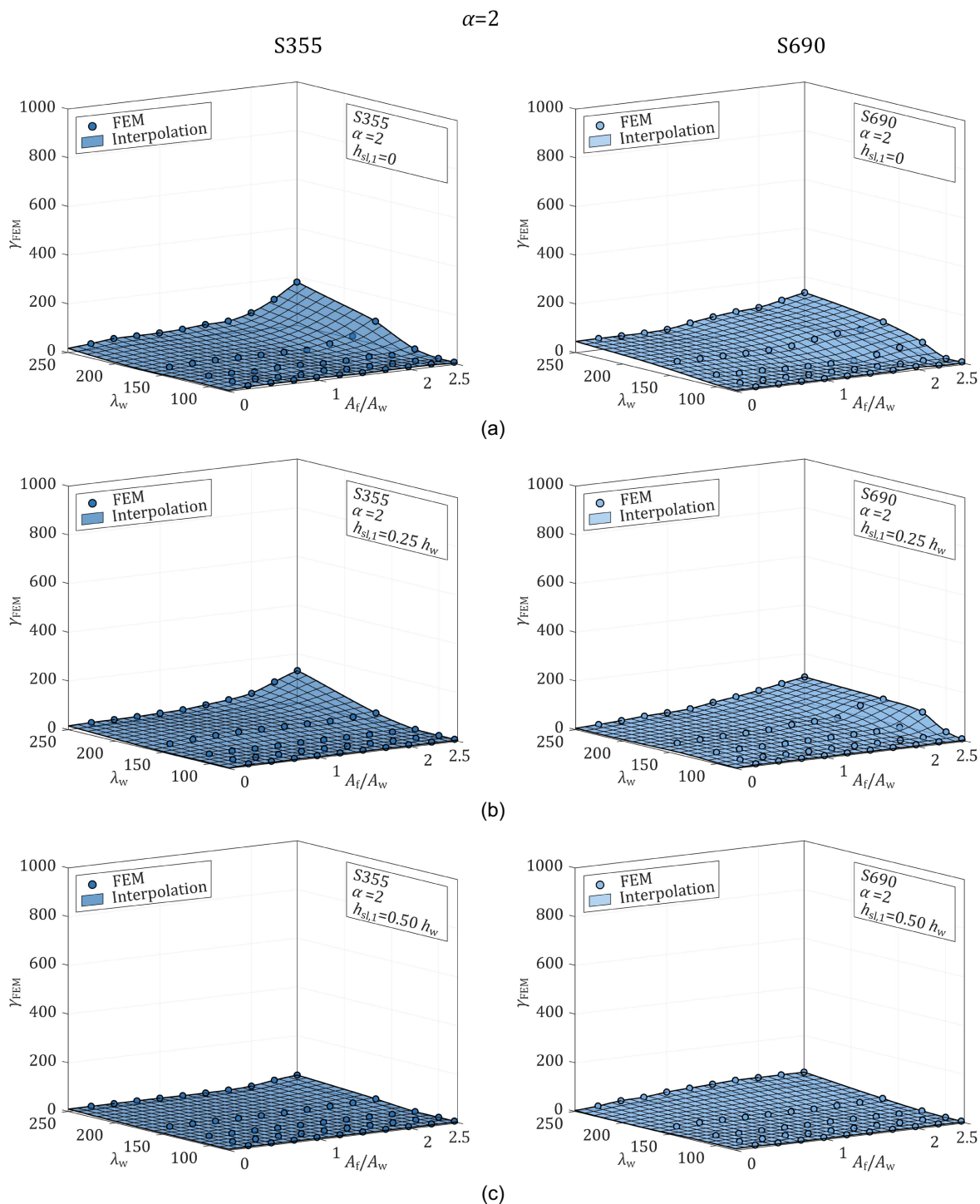


Fig. 13. Effect of steel grade on the FEM required relative stiffener rigidity $\gamma_{st,FEM}$ for $\alpha = 2.0$: (a) without longitudinal stiffener; (b) $h_{sl,1} = 0.25h_w$; and (c) $h_{sl,1} = 0.50h_w$.

- the distance b_{st} was constrained to $\leq b_f/2$, ensuring that the T-stiffener's top and bottom edges were adequately restrained by the flange.

Fig. 18 presents the numerically obtained V_{FEM}/V_{pl} versus γ_{st} curves and the corresponding $\gamma_{st,FEM}$ values for the analyzed scenarios with $\alpha = 1$, $\lambda_w = 250$, $A_f/A_w = 1.0$ and 2.0 , S355 steel, and no longitudinal stiffeners

- The required $\gamma_{st,FEM}$ was similar for flat and T-stiffener layouts, indicating no significant difference in numerical rigidity demands [Fig. 18(a)].

- There was slightly reduced shear resistance for the T-stiffener layout than for the flat stiffener [Fig. 18(b)]. This reduction was less than 10%, suggesting that although T-stiffeners may have marginally lower resistance, they still meet the required $\gamma_{st,FEM}$ as long as the same relative stiffener rigidity γ_{st} is provided.

These results are contingent on the cross-sectional ratios b_{st}/t_{st} and $b_{st,f}/t_{st,f}$ defined previously. Variations in these ratios could influence the transverse stiffener behavior and, consequently, the ultimate load capacity of the plate girder.

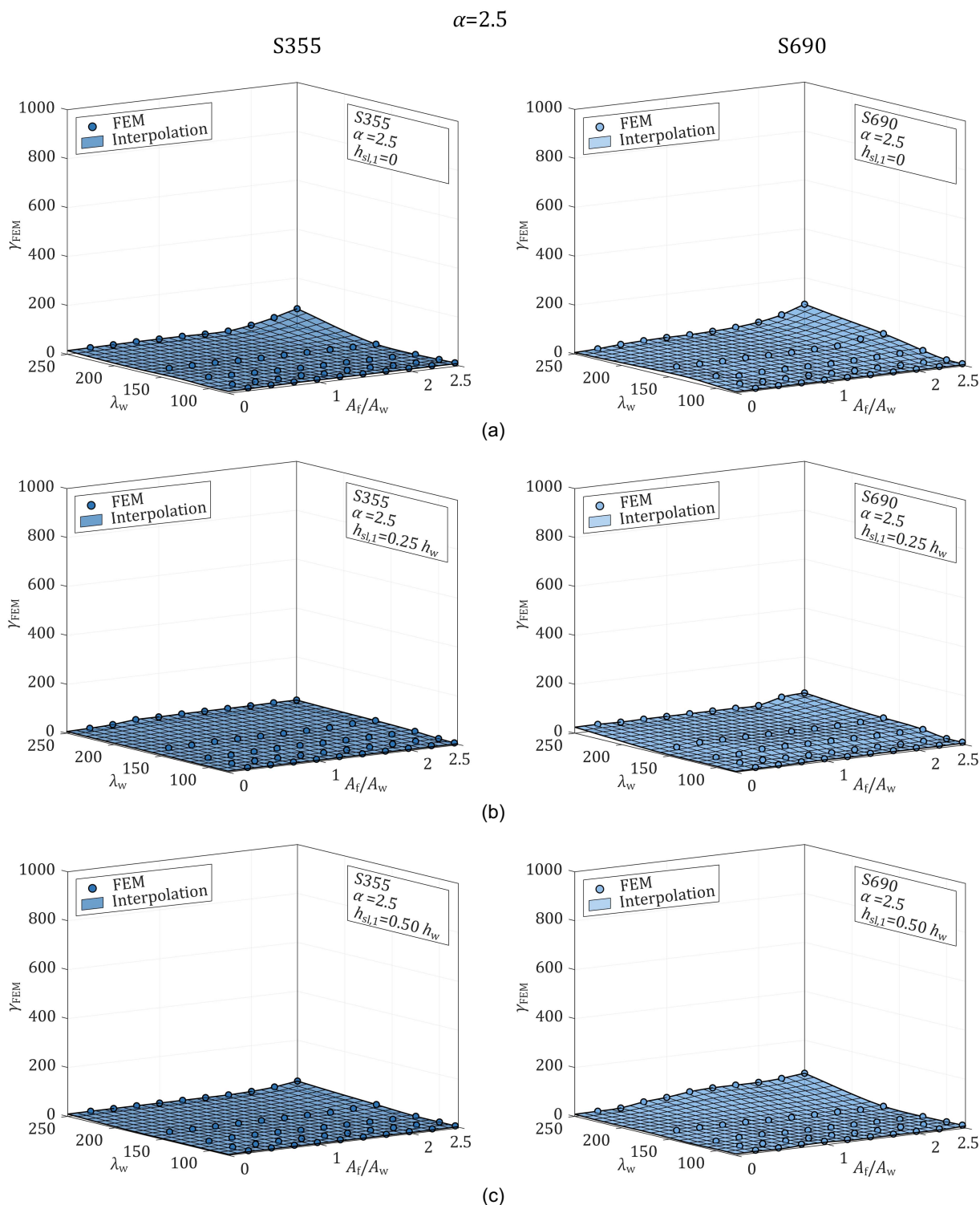


Fig. 14. Effect of steel grade on the FEM required relative stiffener rigidity $\gamma_{st,FEM}$ for $\alpha = 2.5$: (a) without longitudinal stiffener; (b) longitudinal stiffener $h_{sl,1} = 0.25h_w$; and (c) longitudinal stiffener $h_{sl,1} = 0.50h_w$.

Additionally, the adoption of a bottom cut-out detail in the T-stiffener reduced the required bending rigidity by approximately 8%, emphasizing the need to account for such details in practical design.

Effect of Nonsymmetric Plate Girders

The design of nonsymmetric plate girders is a common practice in steel plate girders as well as in steel-concrete composite girders in

which a concrete slab contributes to the effective flange area. Given the dependency of the design proposal on the flange-to-web area ratio A_f/A_w , it was crucial to evaluate the impact of varying flange dimensions on the required relative stiffener rigidity $\gamma_{st,FEM}$.

To investigate this effect, a girder with a nonsymmetric flange configuration ($A_{f,t}/A_w = 2.0$ for the top flange and $A_{f,b}/A_w = 1.0$ for the bottom flange) was analyzed. The parametric study considered $\alpha = 1$, $\lambda_w = 250$, $A_f/A_w = 1.0$ and 2.0 , S355 steel, and no

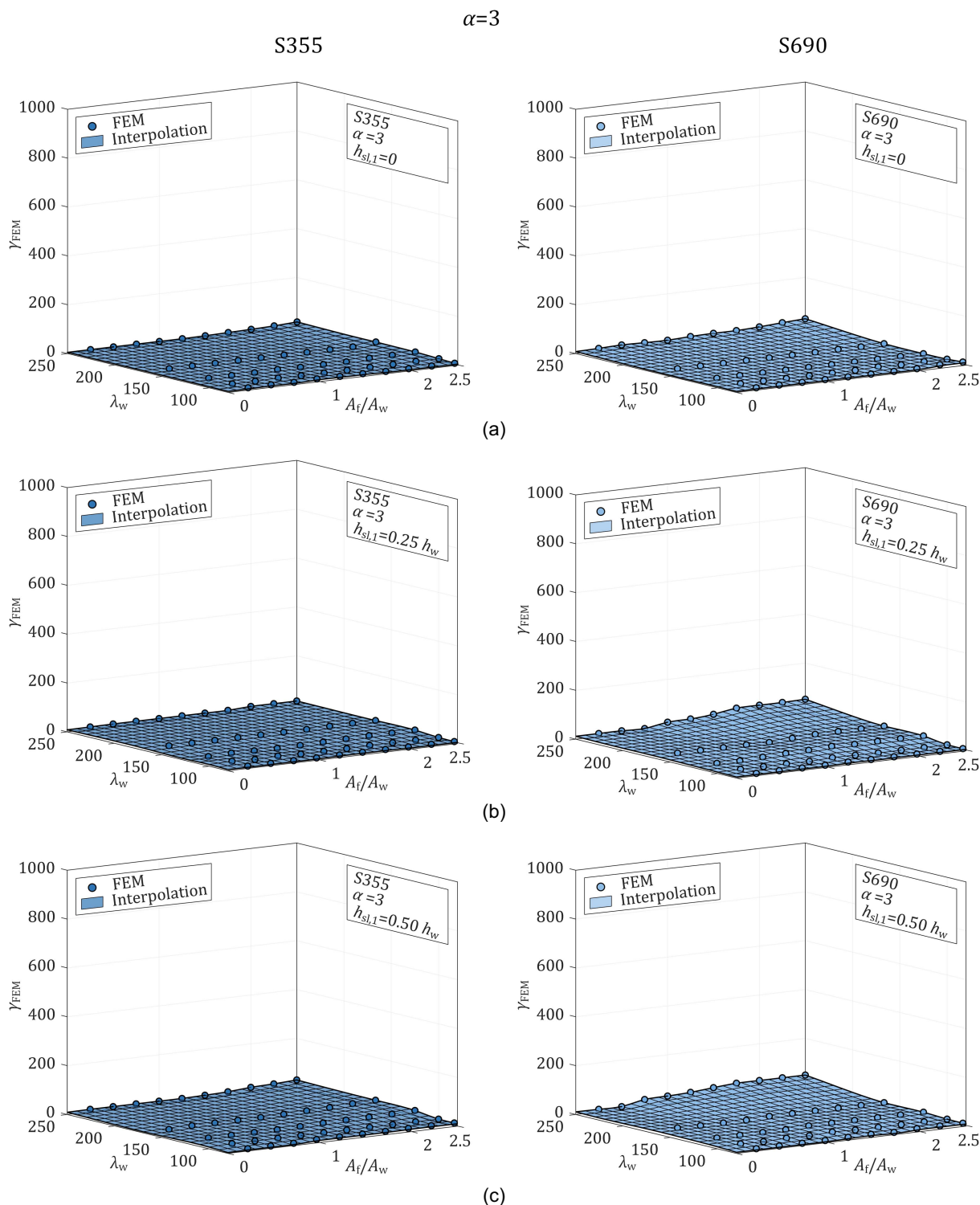


Fig. 15. Effect of steel grade on the FEM required relative stiffener rigidity $\gamma_{st,FEM}$ for $\alpha = 3.0$: (a) without longitudinal stiffener; (b) longitudinal stiffener $h_{sl,1} = 0.25h_w$; and (c) longitudinal stiffener $h_{sl,1} = 0.50h_w$.

longitudinal stiffeners. The results are presented in Fig. 19, which shows the relationship between V/V_{pl} and γ_{st} for the nonsymmetric girder. The required $\gamma_{st,FEM}$ for the nonsymmetric case ($A_{f,t}/A_w = 2.0$, and $A_{f,b}/A_w = 1.0$) was 235. This value is between the requirements for symmetric girders with $A_f/A_w = 1.0$ ($\gamma_{st,FEM} = 96$) and $A_f/A_w = 2.0$ ($\gamma_{st,FEM} = 463$).

The findings indicate that the required stiffener rigidity for nonsymmetric girders is higher (approximately double) than for the

smaller A_f/A_w ratio but lower (about 50% less) than the requirement for the larger A_f/A_w ratio. However, although these results are between the extremes of the symmetric cases, the limited scope of this study does not allow for a definitive conclusion that using an average flange area $A_{f,avg}/A_w$ would be a safe design approach for nonsymmetric girders.

In the absence of additional data, such as results from detailed FE analyses, it is recommended to adopt the higher A_f/A_w ratio when

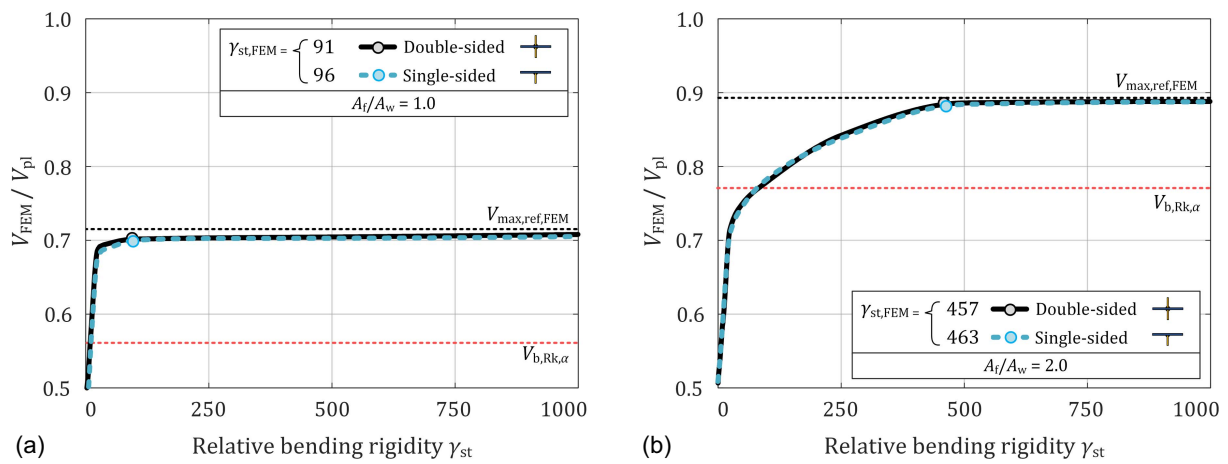


Fig. 16. Transverse single-sided and double-sided stiffeners ultimate rigidity $\gamma_{st,FEM}$ for (a) $A_f/A_w = 1.0$; and (b) $A_f/A_w = 2.0$.

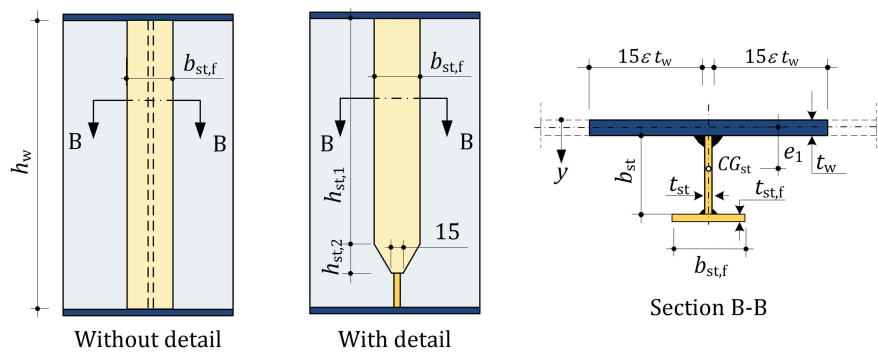


Fig. 17. Geometry of the T-stiffener layout (dimensions in mm).

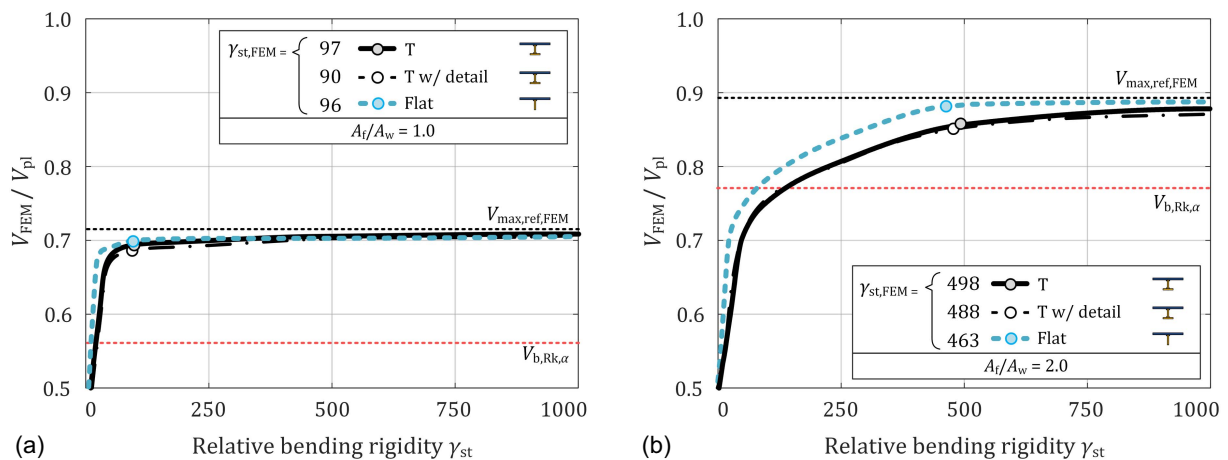


Fig. 18. Transverse stiffener ultimate rigidity $\gamma_{st,FEM}$ for T- and flat stiffeners with (a) $A_f/A_w = 1.0$; and (b) $A_f/A_w = 2.0$.

designing nonsymmetric girders using the proposed method for calculating $I_{st,2}$. This conservative approach ensures compliance with design criteria and provides a margin of safety for the ultimate load capacity.

Effect of Bending Moment–Shear (M – V) Interaction

The influence of combined bending moments and shear stresses on the behavior of intermediate transverse stiffeners was investigated numerically. In line with Oliveira Pedro et al. (2024a), to ensure a

continuous distribution the bending stresses along the web, shell edge loads were applied using the FE model described in Section “Numerical Model and Validation,” guaranteeing a consistent plate buckling behavior and interaction with shear stresses.

For this analysis, the case study with $\alpha = 1$, $\lambda_w = 250$, $A_f/A_w = 1.0$, S355 steel, and no longitudinal stiffeners was selected. The aim was to assess how the interaction between bending moment (M) and shear stresses (V) affects the intermediate stiffener requirements.

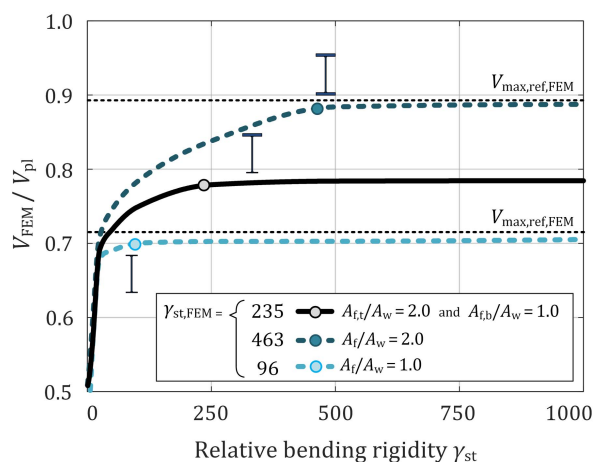


Fig. 19. Transverse stiffener numerical relative ultimate rigidity $\gamma_{st,FEM}$ for nonsymmetric plate girders with $A_{f,t}/A_w = 2.0$ and $A_{f,b}/A_w = 1.0$.

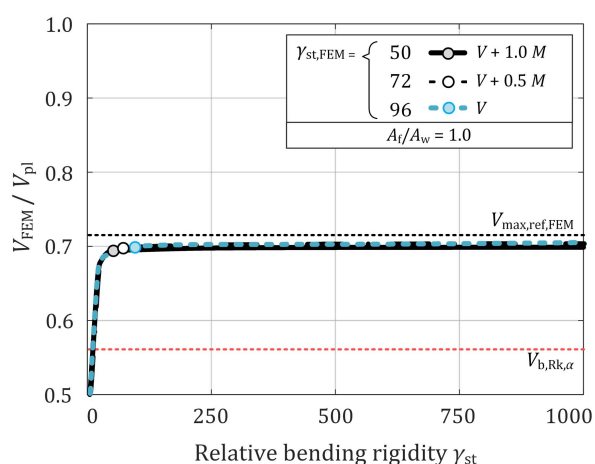


Fig. 20. Transverse stiffener numerical relative ultimate rigidity $\gamma_{st,FEM}$ for combined bending and shear (M - V) loadings.

Two bending moment levels, $M = 0.5 M_{f,pl}$ and $1.0 M_{f,pl}$, were applied with shear stresses. The plastic bending moment of the flange, $M_{pl,f}$, is defined using the web height measured at the mid-section of the flange

$$M_{f,pl} = M_{f,Rk} = (h_w + t_f) b_f t_f f_{yf} \quad (12)$$

The required intermediate transverse stiffener relative bending rigidity for the girder to attain the ultimate load under shear and bending stresses is plotted using the V/V_{pl} - γ_{st} curve in Fig. 20 and compared with the correspondent case of pure shear stresses. The maximum shear resistance for a plate girder under both shear and bending stresses was slightly lower than that for the corresponding case under pure shear stresses. Consequently, if M - V interaction is present, the required bending rigidity of the intermediate transverse stiffener is reduced.

The bending moment decreases the flanges' availability to contribute to the shear postbuckling resistance, reducing the overall girder shear resistance. Consequently, the transverse stiffener requirement is slightly reduced when the girder is under M - V loading. Thus, the bending rigidity requirement for pure shear remains applicable for plate girders subjected to shear and bending moments applied together.

Conclusions

The bending rigidity (γ_{st}) required for intermediate transverse stiffeners in steel plate girders was investigated extensively, with reference to key standards and a detailed numerical parametric study. The main conclusions are summarized as follows:

1. EN 1993-1-5 requirements

- The minimum relative bending rigidity ($\gamma_{st,1}$) defined in EN 1993-1-5 is based on an empirical relationship between γ_{st} and the web panel aspect ratio (α). This relationship ensures only that the critical shear stress (τ_{cr}) of the web is reached while limiting stiffener deflections.
- The required $\gamma_{st,1}$ is independent of web slenderness (λ_w) and steel grade (f_y), and it becomes less demanding as the aspect ratio (α) increases.
- The standard's minimum bending rigidity requirement ($I_{st,1}$) is not calibrated to achieve the ultimate shear resistance of the plate girder. Therefore additional verifications concerning strength and torsional rigidity are recommended for the design of the transverse stiffeners. Alternatively, a higher bending rigidity requirement should be defined in the standard for cases of aspect ratios $\alpha \leq 1.0$.

2. Comparison with AASHTO and AISC standards

- US standards (AASHTO and AISC) distinguish between the bending rigidity required to reach the critical load (τ_{cr}) and the postbuckling resistance (τ_u), denoted $I_{st,1}$ and $I_{st,2}$, respectively.
- These standards simplify stiffener design by calculating properties based solely on the stiffener plate, excluding the web's contribution to the stiffener cross-section properties.

3. Numerical Findings

A comprehensive parametric study demonstrated clear dependencies of $\gamma_{st,FEM}$ on key design parameters

- Steel grade (f_y): The required stiffener rigidity ($\gamma_{st,FEM}$) is influenced only marginally by the material strength. A maximum 30% increase in $\gamma_{st,FEM}$ was observed for S355 steel compared with S690 steel. Thus, $\gamma_{st,FEM}$ increases slightly as f_y decreases.
- Aspect ratio (α): As the aspect ratio decreases, the required $\gamma_{st,FEM}$ increases. For example, $\gamma_{st,FEM}$ decreases by approximately 50% when α increases from 1.0 to 1.5;
- Web slenderness (λ_w): Increased web slenderness increases the required $\gamma_{st,FEM}$ due to the dominant postbuckling tension field action. This effect is more pronounced when strong flanges are used.
- Flange area ratio (A_f/A_w): The flange significantly impacts $\gamma_{st,FEM}$, especially as A_f/A_w increases above 1.0. For A_f/A_w values up to 1.0, $\gamma_{st,FEM}$ remains nearly constant. However, increasing A_f/A_w from 1.0 to 2.5 increases $\gamma_{st,FEM}$ by as much as 20%. This influence decreases as the web panel aspect ratio (α) increases.
- Longitudinal stiffeners: Introducing longitudinal stiffeners can reduce the required rigidity of transverse stiffeners, particularly if the longitudinal stiffener is located at the midheight of the web panel. However, this reduction is moderate for most cases, becoming significant only for very slender webs with high aspect ratios and strong flanges, so the adoption of longitudinal stiffeners may not always be the most economical solution for improving the web shear buckling resistance.

Rather than emphasizing the area requirement [which is still present in the European standard and is "known to be conservative" (Lee et al. 2002)], this investigation evaluated and quantified the required rigidity of the transverse web stiffener necessary to maintain the stiffener's effectiveness up to the girder's ultimate shear resistance. Design standards such as AASHTO (since 2007) and AISC

(since 2010) have eliminated the stiffener area verification in favor of a minimum bending rigidity requirement, a shift widely regarded as providing a more reliable and practical basis for design.

Nonetheless, it is clear that, within the framework of the EN 1993-1-5, the rules developed here concerning the bending rigidity of intermediate transverse stiffeners—ensuring the postbuckling resistance of plate girders without requiring an explicit strength verification of the stiffeners—still need to be incorporated into normative rules.

To advance this goal, several avenues for future research are identified. These include extending the evaluation of the required bending rigidity of stiffeners to cases of directly loaded stiffeners (e.g., subjected to bracing forces or support reactions), hybrid or composite girders, and nonuniform geometries such as tapered or curved webs. Further investigations into alternative stiffener cross sections and the influence of combined M – N – V interactions also are recommended to establish a unified, stiffness-based design framework for transverse stiffeners.

Data Availability Statement

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

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Author Contributions

Sérgio Nascimento: Formal analysis; Investigation; Methodology; Validation; Writing – original draft. José J. Oliveira Pedro: Formal analysis; Investigation; Supervision; Writing – review and editing. Ulrike Kuhlmann: Validation; Writing – review and editing.

Notation

The following symbols are used in this paper:

A_f = area of flange (mm^2);
 $A_{f,avg}$ = average area of flange (mm^2);
 $A_{f,b}$ = area of bottom flange (mm^2);
 $A_{f,t}$ = area of top flange (mm^2);
 A_{st} = area of effective transverse stiffener (mm^2);
 A_w = area of web (mm^2);
 a = length of stiffened web (mm);
 b = width of stiffened web (mm)—average of sample;
 b_f = width of flange (mm);
 b_{sl} = width of longitudinal stiffener outstand (mm);
 b_{st} = width of transverse stiffener outstand (mm);
 $b_{st,f}$ = width of transverse stiffener outstand flange (mm);
 D = bending rigidity of web (Nmm);
 E = modulus of elasticity (N/mm^2);

e = design eccentricity calculated between effective stiffener and web plate (mm);
 E_{sh} = strain hardening modulus for hot-rolled steels (N/mm^2);
 $E_{0.2}$ = yielding tangent modulus for cold-formed and stainless steels (N/mm^2);
 $f_{C1\varepsilon_u}$ = stress for hot-rolled steels (N/mm^2);
 $f_{C2\varepsilon_u}$ = stress for hot-rolled steels (N/mm^2);
 f_u = ultimate stress (N/mm^2);
 f_y = yielding stress (N/mm^2);
 $f_{y,f}$ = yielding stress of flange (N/mm^2);
 $f_{y,st}$ = yielding stress of transverse stiffener (N/mm^2);
 $f_{y,w}$ = yielding stress of web (N/mm^2);
 $h_{sl,i}$ = depth of web subpanel i (mm);
 $h_{st,i}$ = depth of transverse stiffener outstand flange (mm);
 h_w = clear depth of web between flanges (mm);
 I_p = polar moment of inertia (mm^4);
 I_{st} = moment of inertia of effective transverse stiffener (mm^4);
 I_{st1} = elastic buckling second moment of inertia of effective transverse stiffener (mm^4);
 I_{st2} = ultimate second moment of inertia of effective transverse stiffener (mm^4);
 I_T = Saint Venant torsional constant (mm^4);
 J = elastic rigidity parameter for transverse stiffener according to AASHTO or AISC;
 k_{yy} = interaction factor;
 k_τ = shear buckling factor of web;
 L = element length (mm);
 l = element buckling length (mm);
 M = bending moment (kNm);
 $M_{f,pl}$ = plastic bending moment resistance of flanges (kNm);
 $M_{f,Rk}$ = characteristic bending moment of resistance of flanges (kNm);
 M_{st} = transverse stiffener bending moment (kNm);
 $M_{y,Rk}$ = characteristic y – y bending moment (kNm);
 m = factor according to Škaloud et al. (1967);
 N_{Rk} = characteristic axial force (kN);
 N_{st} = axial force of transverse stiffener (kN);
 r = length of end web panel (mm);
 s = maximum between a and wh (mm);
 t_f = thickness of flange (mm);
 t_{sl} = thickness of longitudinal stiffener (mm);
 t_{st} = thickness of transverse stiffener outstand (mm);
 $t_{st,f}$ = thickness of transverse stiffener outstand flange (mm);
 t_w = thickness of web (mm);
 V, V_{Ed} = shear force (kN);
 $V_{b,Rk}$ = characteristic shear force (kN);
 V_{code} = shear buckling resistance according to EN 1993–1–5 (kN);
 V_{cr} = elastic buckling shear load (kN);
 V_{FEM} = FEM shear force (kN);
 $V_{max,FEM}$ = maximum FEM shear force (kN);
 $V_{max,ref,FEM}$ = maximum reference FEM shear force (kN);
 V_{pl} = plastic web shear force (kN);
 V_{ult} = ultimate shear force (kN);
 α = aspect ratio of web panel;
 γ_{M1} = partial factor for buckling;

γ_{sl} = relative bending rigidity of longitudinal stiffener;
 γ_{st} = relative bending rigidity of effective transverse stiffener;
 $\gamma_{st,end}$ = relative bending rigidity of end effective transverse stiffener;
 $\gamma_{st,FEM}$ = required FEM bending rigidity of effective transverse stiffener;
 γ_{st1} = relative elastic buckling bending rigidity of effective transverse stiffener;
 γ_{st2} = relative ultimate bending rigidity of effective transverse stiffener;
 ε = strain;
 ε_{lim} = limit strain;
 ε_{sh} = strain hardening for hot-rolled steels;
 ε_u = ultimate strain;
 ε_y = yielding strain;
 $\varepsilon_{0.2}$ = total strain at 0.2% proof stress for Ramberg–Osgood type material model cold-formed steel and stainless steels;
 $\bar{\lambda}_{st}$ = relative transverse stiffener slenderness;
 $\bar{\lambda}_w$ = relative web slenderness;
 ν = Poisson's ratio;
 ρ_t = parameter according to AASHTO and AISC;
 σ_{cr} = elastic buckling stress of web (N/mm²);
 $\sigma_{cr,st}$ = elastic buckling stress of transverse stiffener (N/mm²);
 τ = shear stress (N/mm²);
 τ_{cr} = elastic buckling shear stress (N/mm²);
 τ_u = ultimate yielding stress (N/mm²);
 τ_y = shear yielding stress (N/mm²);
 χ_{LT} = reduction coefficient for lateral buckling;
 χ_y = reduction coefficient for bending; and
 ω_0 = initial imperfection amplitude (mm).

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