

Review and improved method for the design of intermediate transverse stiffeners in steel plate girders

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ABSTRACT

Steel plate girders with slender webs rely on transverse stiffeners to provide web lateral support, thereby enhancing shear resistance. Intermediate stiffeners are essential not only for anchoring the compression from tension field action (TFA) but also for restraining out-of-plane buckling and supporting web panel edges. This paper reviews existing design methods for transverse stiffeners and examines key developments and challenges from elastic buckling to ultimate strength, comparing design provisions with key research findings. Based on these insights, an enhanced design approach is proposed for intermediate web stiffeners in steel plate girders, accounting for flange influence, web slenderness and aspect ratio, and material properties. The proposal is validated against finite element (FE) simulation results and full scale tests to demonstrate its consistency.

1. Introduction

Slender elements are widely used in the design of steel plate girders due to the high level of optimisation achievable in plated I-girder and box girder bridge decks. However, their slenderness makes them susceptible to instability, particularly plate buckling under combined stresses (Fig. 1). A key concern is shear buckling, which can develop in slender webs subjected to shear forces. If not properly considered, this phenomenon may reduce the girder's effective lever arm and compromise its load-carrying capacity. To avoid overly conservative assumptions that could lead to increased material use and fabrication costs, accurate prediction of buckling behaviour is essential [1,2].

The structural response of steel plate girders is strongly influenced by web height (h_w), which must be sufficient to resist bending moments, particularly in regions of high moment such as intermediate supports. In contrast, a relatively thin web (t_w) is typically adequate for resisting shear, often resulting in high slenderness ratios ($\lambda_w = h_w/t_w$), which increase susceptibility to early buckling. To address this, a reinforcement is added to reduce the effective panel span and raise the elastic critical buckling stress (σ_{cr}). This is usually achieved with transverse and longitudinal stiffeners (Fig. 2) [3,4].

The 1910 Bridge Specifications of the American Railway Engineering

Association [5] introduced minimum dimensions for intermediate stiffeners, but it was in 1936 that Timoshenko [6] first specified a minimum second moment of area. Moreover, in the second half of the 20th century, recognition of web post-buckling resistance led to changes in design codes, requiring transverse stiffeners to accommodate additional forces from post-buckling tension field action (TFA). This supported the shift towards ultimate limit state design approaches. Despite being based on decades of research, the design provisions in EN 1993-1-5:2024 [7] still make the optimal sizing of transverse stiffeners a complex task. While some researchers view the current rules as broadly sufficient, further study of existing methods is widely seen as worthwhile.

As discussed in the following sections, today's design rules stem from a century of experimental and analytical work – especially the key studies referenced in [8–11] – across a range of girder geometries. Yet, the resulting formulations have often proven overly conservative [12]. Since designs based on fictitious axial forces tend to produce such conservative results [13–16], this study focuses on defining the necessary second moment of area (I_{st2}) to enable full development of post-buckling stresses in the web. Based on this, Section 5 presents an advanced design method intended to offer a clearer, more efficient, and reliable approach to transverse stiffener sizing.

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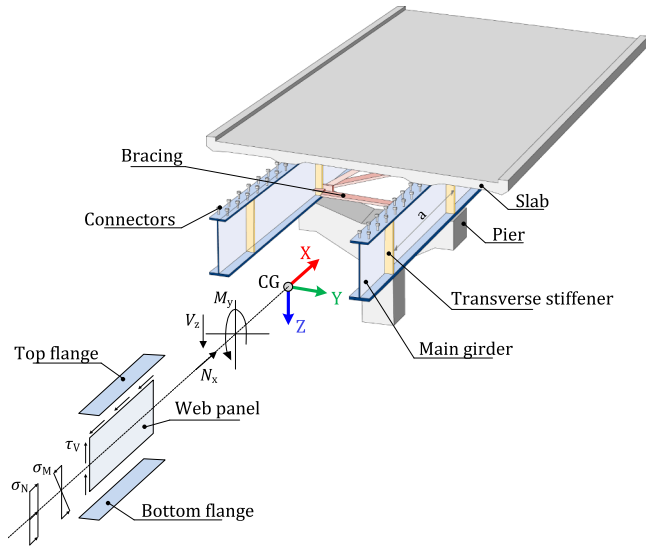


Fig. 1. Web loading of steel-concrete composite bridge deck using twin plate girders.

2. Elastic design methods

2.1. General considerations

A rigid vertical web intermediate stiffener serves the following structural functions:

- **Elastic buckling support:** When designing for the elastic critical shear stress τ_{cr} , the transverse stiffener – characterised by its second moment of area I_{st1} , and the bending rigidity γ_{st1} between the

stiffener and the web – provides lateral restraint to the web, effectively subdividing it into smaller panels. This restraint increases the web's elastic buckling resistance;

- **Post-buckling support:** Modern plate girders are typically designed to carry loads beyond the web's critical buckling load, accounting for shear-induced TFA. In such cases, the transverse stiffener must possess stronger rigidity ($I_{st2} > I_{st1}$, $\gamma_{st2} > \gamma_{st1}$) to actively equilibrate these post-buckling forces;
- **Flange restraint:** The stiffener also prevents inward displacement of the flanges, which is the principal source of axial force in the stiffener under post-buckling behaviour;
- **Secondary functions:** Additionally, stiffeners may serve as connection points for cross-bracing between girders or act as temporary bearing elements during transport and erection.

In accordance with its structural role, the behaviour of a transverse stiffener is best defined by the parameters outlined in Eqs. (1) to (5), following the notation in Fig. 2:

$$\text{stiffener slenderness} \quad \lambda_{st} = b_{st}/t_{st} \quad (1)$$

$$\text{stiffener axial rigidity} \quad \delta_{st} = \frac{A_{st}}{a t_w} \quad (2)$$

$$\text{stiffener bending rigidity} \quad \gamma_{st} = \frac{E I_{st}}{D b} \quad (3)$$

$$\text{stiffener torsional rigidity} \quad \theta_{st} = \frac{G J_{st}}{D b} \quad (4)$$

$$\text{web bending rigidity per unit width} \quad D = \frac{E t_w^3}{12 (1 - \nu^2)} \quad (5)$$

The definition of the stiffened panel width, b , varies across studies and design standards. AASHTO:2007, AISC 360:2010, and some authors

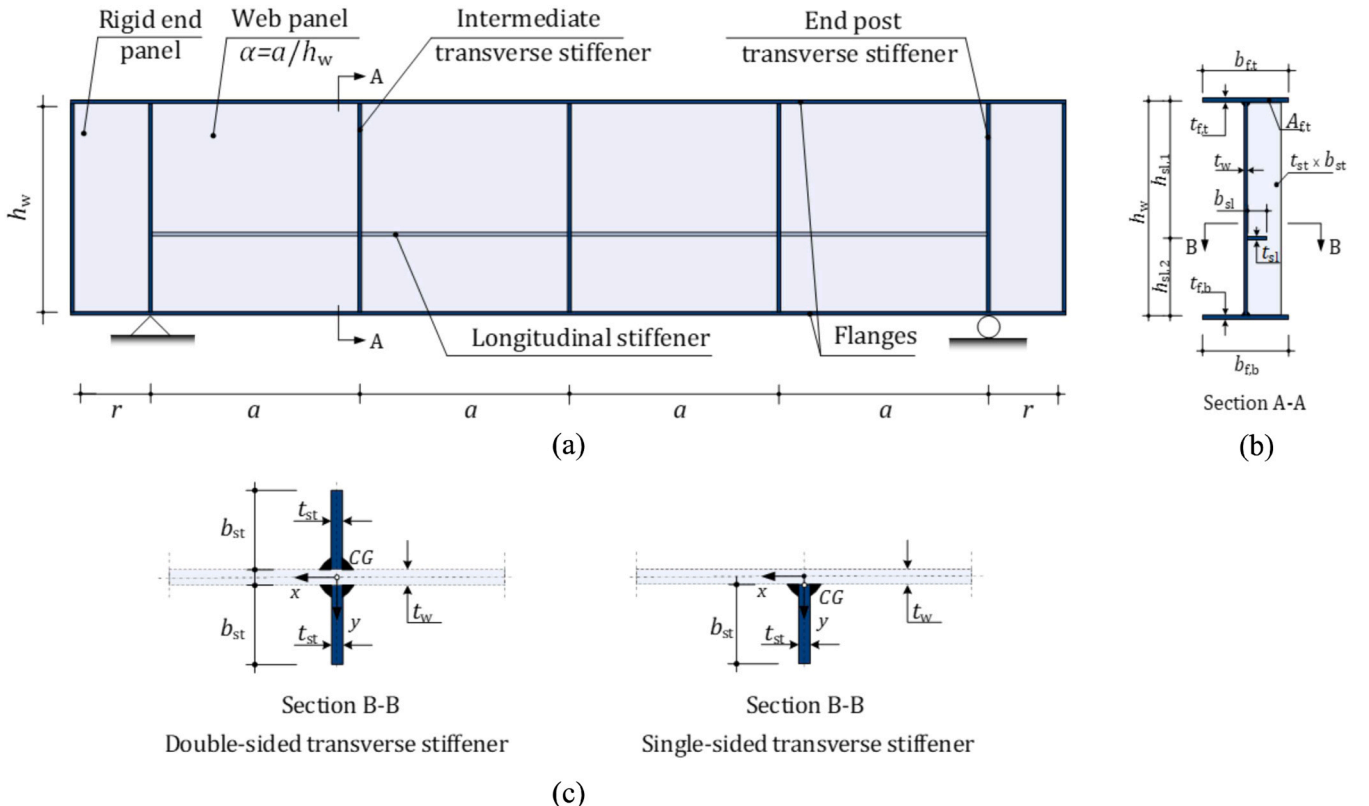


Fig. 2. Plate girder main elements and notation: (a) Elevation, (b) Section A-A, (c) Section B-B.

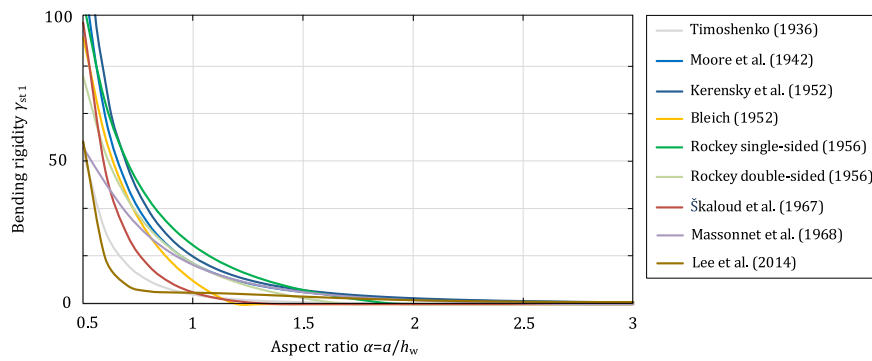


Fig. 3. Bending rigidity γ_{st1} of a transverse stiffener vs. aspect ratio α , as proposed by several authors.

[16] recommend $b = \min\{a; h_w\}$, effectively capping the width at the web height. This results in a constant stiffener requirement γ_{st} for panels with aspect ratios $\alpha = a/h_w \geq 1$. However, recent studies by Lee et al. [17] and Kim et al. [18] challenge this approach. Lee et al. advocate using $b = a$, arguing that the conventional limit is overly conservative, especially for long panels. Kim et al. similarly note that stiffener demand in such cases is generally low. Earlier 2005 editions of AASHTO and AISC 360 also followed this less restrictive interpretation. Based on this, the present study adopts $b = a$.

2.2. Bending rigidity

2.2.1. Timoshenko [6]

The concept of minimum stiffener rigidity was first introduced by Timoshenko in Theory of Elastic Stability [6], where expressions were provided to quantify the stiffness required for ribs or plate stiffeners to effectively support a web panel and achieve the subpanel's critical load (Eq. (6)).

2.2.2. Moore et al. [19]

To improve design accuracy for lightweight structural systems, particularly in aerospace applications, Moore et al. [19] conducted an experimental study on aluminium plate girders with transverse angle stiffeners. The study involved 60 girders and confirmed that transverse stiffeners increase the web's critical load and benefit panel subdivision. Based on these results, the authors proposed an empirical rigidity-based design method of Eq. (7)).

Moore defined γ_{st} using two stability-based criteria: (i) when stiffener deflection reached 25 % of the web's average deflection under critical loading, and (ii) when it reached 0.508 mm, corresponding to $h_w/1450$ and $h_w/770$ for girder heights of 392 mm and 735 mm, respectively. He also noted that for single-sided stiffeners, γ_{st} should be calculated using the second moment of area about an axis assuming an effective web width equal to 25 % of h_w [19], following earlier findings [20]. In parallel, in 1947, Wang proposed design curves for stiffened plates [21], though Sparkes' experiments [22] indicated that Moore's γ_{st1} was insufficient to properly subdivide the web – supporting earlier conclusions by Scott and Weber [23].

2.2.3. Bleich [27]

Since existing stiffness criteria were limited to closely spaced stiffeners, Stein and Fralich [26] conducted tests in 1949 on 20 specimens with aspect ratios $\alpha = 1, 2$, and 5. They examined the relationship between the shear buckling coefficient k_τ and relative stiffener rigidity γ_{st} , finding that k_τ increases with γ_{st} but approaches a plateau as $\gamma_{st} \rightarrow \infty$. Building on these findings, Bleich [27] proposed in 1952 an optimal γ_{st} for infinitely long stiffened plates, expressed in terms of the stiffener's second moment of area I_{st1} (Eq. (8)). This formulation closely aligns with current AASHTO provisions, discussed later.

2.2.4. Kerensky et al. [24]

In response to the growth of welded construction, the British Standards Institution tasked Kerensky et al. [24] in 1952 with developing design rules for shear stresses in girder webs. Their work informed the 1958 revision of BS 153 (later BS 5400–3) and built on studies by Moore et al. He proposed a modest increase in the minimum second moment of area for transverse stiffeners, raising the constant of Eq. (7) from 1.3 to 1.5 as in Eq. (9).

Based on these recommendations, BS 153 limited transverse stiffener spacing to $1.5 h_w$. Kerensky's criterion was later adopted in ENV 1993–1–1:1992 [25] and remains in EN 1993–1–5:2024 [7], with a revised α limit of $\sqrt{2} \approx 1.414$. Key clarifications from [7] and [19] include:

- The EN standard introduced the concept of an effective stiffener section, incorporating a $1.5 \varepsilon t_w$ portion of the adjacent web, without altering the Moore's formula;
- The minimum second moment of area ensures only that the web reaches its critical shear buckling load, not its ultimate shear resistance, for which further checks are required.

2.2.5. Rockey [28]

In 1956, Rockey [28] studied the link between the shear buckling coefficient k_τ and stiffener rigidity γ_{st} for aluminium plate girders, analysing 220 web-stiffener configurations. The resulting expression (Eq. (10)) defines the required γ_{st1} , incorporating a web contribution of $a/2$ on each side of the stiffener in the second moment of area calculation.

2.2.6. Škaloud et al. [29]

Škaloud et al. [29] investigated the effect of initial out-of-plane imperfections on the required stiffener rigidity γ_{st} . Through second-order analysis of stiffened square panels under shear, they proposed amplifying Moore's Eq. (7)) by a magnification factor m , recommending $m = 3$ for transverse stiffeners, leading to Eq. (11).

2.2.7. Massonnet et al. [30]

Building on Massonnet's earlier work [31], the same research group later published a study as Massonnet et al. [30], proposing an optimal stiffener rigidity γ_{st1} to prevent stiffener failure up to the web's critical shear buckling load. Based on theoretical and experimental analyses, they derived Eq. (12), valid for $0.5 \leq \alpha \leq 2.0$. This expression ensures adequate transverse stiffener design within practical aspect ratios. Subsequently, Škaloud et al. [32], incorporated initial geometric imperfections into the stiffener rigidity design.

2.2.8. Lee et al. [17]

Recently, Lee et al. [17] proposed an updated formula for the minimum second moment of area, in Eq. (13) required to keep intermediate transverse stiffeners straight under the critical web load. Their

Table 1
Summary of the key investigation on the elastic design and requirements of intermediate transverse stiffeners reported in the literature.

Author	Year	Experimental	FEA	Design proposal	Formula	Eq.
Timoshenko [6]	1936	No	No	Stiffness	$\gamma_{st1} \geq 0.3/(a/h_w)^4$	(6)
Moore et al. [19]	1942	Yes	No	Stiffness	$\gamma_{st1} \geq 14/(a/h_w)^3$	(7)
Bleich [27]	1952	No	No	Stiffness	$\gamma_{st1} \geq [28/(a/h_w)^2] - 20$	(8)
Kerensky et al. [24]	1956	No	No	Stiffness	$\gamma_{st1} \geq 16.38/(a/h_w)^3$	(9)
Rockey [28]	1956	Yes	No	Stiffness	$\gamma_{st1} \geq \begin{cases} [27.75/(a/h_w)^2] - 7.5 & \text{for single - sided stiffeners} \\ [21.50/(a/h_w)^2] - 7.5 & \text{for double - sided stiffeners} \end{cases}$	(10)
Škaloud et al. [29]	1967	No	No	Stiffness	$\gamma_{st1} \geq m \frac{2.7}{(a/h_w)^2} \left(\frac{1}{8} + \frac{5}{8(a/h_w)^2} - 1 \right)$ with $m = 3$	(11)
Massonnet et al. [30]	1968	Yes	No	Stiffness	$\gamma_{st1} \geq \frac{5.4}{a/h_w} \left(\frac{2}{a/h_w} + \frac{2.5}{(a/h_w)^2} - 1 \right)$	(12)
Lee et al. [17]	2014	No	Yes	Stiffness	$\gamma_{st1} \geq 10.92 J_1$ with $J_1 \geq \begin{cases} \frac{2.99}{(a/h_w)^4} - \frac{9.38}{(a/h_w)^3} + \frac{11.15}{(a/h_w)^2} - \frac{4.64}{(a/h_w)} + 0.70 & \text{for } a/h_w \leq 3 \\ \frac{1.20}{(a/h_w)^2} - \frac{0.38}{(a/h_w)} + 0.04 & \text{for } a/h_w > 3 \end{cases}$	(13) (14)

expression, based on AASHTO's J -value (detailed later), expressed by Eq. (14).

2.2.9. Supplementary research

To evaluate the required stiffener rigidity for achieving buckling loads, Way [33] studied plate stability under shear and bending in 1936, considering the influence of transverse stiffeners. Moisseiff [34] later proposed a minimum rigidity based on Chwalla's earlier work [35]. In 1961, Klöppel et al. [36] developed charts for optimal rigidity under various loading and stiffening conditions, still within a critical load framework. Huang [37] provided a comparative summary of key research up to 1968. During the 1990s, Stanway et al. [38,39] used finite element analysis (FEA) to examine elastic shear buckling in simply supported plates with a single stiffener, quantifying the effect of stiffener flexural rigidity on the web's critical load.

2.2.10. Comparison between bending rigidity design models

Fig. 3 compares the key proposals for the optimal transverse stiffener rigidity γ_{st1} in elastic web design. As the stiffener bending rigidity γ_{st1} depends solely on the aspect ratio α , this is the only variable shown on the horizontal axis. For short panels, Timoshenko, Škaloud et al. and Lee et al. design methods yield smaller rigidity requirements than the other design methods. Furthermore, for $\alpha > 1.5$, generally, all models lead to similar stiffener bending requirements. Also, the elastic design method proposed by Bleich shows reduced requirements on the range of $1 \leq \alpha \leq 2$.

Table 1 summarises the key investigation on the elastic design and requirements of intermediate transverse stiffeners reported in the literature, namely the elastic bending stiffness criterion.

3. Ultimate design methods

3.1. Strength

From the mid-20th century, the development of numerical methods and a better understanding of nonlinear behaviour in structures, led to the incorporation of the post-buckling resistance of web plates in design standards such as AISC:1963 [40], AASHTO:1973 [41], SIA 161:1979 [42], BS 5400:1982 [43], and ENV 1993-1-1:1992 [25] the post-buckling resistance of web plates. This shift prompted research into the required cross-sectional area (A_{st}) of transverse stiffeners to resist compressive forces generated by diagonal tension in the post-buckling phase – complementing the bending rigidity needed to maintain simply supported edge conditions after buckling [44].

As early as the 1950s, Kerensky et al. [24] acknowledged axial forces in stiffeners under post-buckling shear but advised their omission when $V_{Ed}/V_{cr} < 2$, typical of standard plate girders. They concluded that “the direct load can be safely carried by all practical stiffeners”.

3.1.1. Basler [45,46]

Through strain gauge testing, Basler [45,46] quantified the axial compression force N_{st} , in transverse stiffeners under shear and proposed both an ultimate shear resistance model and a closed-form expression present in Eq. (20).

This formulation informed design provisions in AISC:1963 [40] and AASHTO:1973 [41], which introduced a minimum stiffener area A_{st} . However, in 2007, this axial force requirement was removed from American codes in favour of a minimum bending rigidity check (γ_{st}) for both pre- and post-buckling conditions [47]. The Swiss standard SIA 161 [42] also adopted Basler's approach, which remains part of the current SIA 263:2013 [48].

3.1.2. Rockey et al. [50]

Building on Porter et al.'s [49] analogy to N-type trusses developed in 1975, Rockey et al. [50] developed a more refined axial force model

for transverse stiffeners under shear buckling, supported by experimental testing. While their main formulation was complex, they also proposed a simplified expression assuming constant axial compression as in Eq. (21). This simplified approach was later validated by Tang and Evans (1984) [51], who found it practical and conservative. Rockey's model was incorporated into BS 5400-3 (1982–2000) [43,52], ENV 1993-1-1 (1992) [25], and now in current EN 1993-1-5:2024 [7].

The method relies on the difference between ultimate and critical shear forces and is sensitive to the shear buckling model used to estimate V_{ult} . However, it primarily addresses in-plane axial forces from TFA and neglects out-of-plane bending effects. This leads to conservative results, particularly for panels with small aspect ratios, where stiffeners are sized mainly for axial resistance [53].

3.1.3. Xie [53]

Xie [53] conducted experimental and numerical studies on the behaviour of intermediate transverse stiffeners in plate girders subjected to combined shear and direct stresses. A finite element (FE) study was performed on two-panel I-girders with λ_w ranging from 80 to 300, and $\alpha = 0.5$ and 1.0, made from S355 steel. Transverse stiffeners with $b_{st}/t_{st} = 10$ were examined. Based on his findings, Xie proposed an expression for the stiffener axial force N_{st} . An approximate form is given in Eq. (22), while an improved version incorporates the axial rigidity of the stiffener, δ_{st} , as shown in Eq. (23) [53], calibrated against the FE results and depend on the panel aspect ratio α . The term δ_{st} refers to the extensional rigidity of the stiffener, defined in Eq. (2).

3.1.4. Supplementary research

Recognising the post-buckling resistance of web panels, Huang [37] reviewed developments in stiffener design in 1968, while 4 years later Richmond [54] proposed modelling transverse and longitudinal stiffeners as pin-ended columns under axial compression from shear-induced forces. However, subsequent studies by Stanway et al. [16,38,39] argued that out-of-plane forces, rather than axial compression, are more relevant for stiffeners in shear-loaded webs. FEA have since challenged the necessity of axial force resistance, criticising traditional criteria as overly conservative and based on questionable assumptions [9]. Bjorhovde [55], in a 1980 ASCE report, similarly observed that standard provisions overestimated stiffener cross-sectional demands and called for more rational design methods. In 2003, Lee et al. [9] later confirmed that stiffeners do not need to equilibrate diagonal compression forces but instead require sufficient bending rigidity to resist post-buckling demands.

Both experimental and FE studies have consistently shown that current standards overestimate axial forces in transverse stiffeners [10, 11,13–16,44,50,51,56–59]. Some findings [57,59] even demonstrated that substituting axial restraint with lateral springs yielded comparable critical and ultimate capacities, reinforcing the limited role of axial compression in stiffener performance.

3.2. Bending rigidity

Conversely, with the design of steel girder web panels acknowledging web post-buckling resistance, the requirements for transverse stiffeners to develop the diagonal tension field warranted further investigation. Consequently, the stiffener bending rigidity, γ_{st2} , which ensures the transverse stiffener remains rigid up to the ultimate girder capacity, became the focus of additional research by several investigators.

3.2.1. Horne and Grayson [60]

Due to the complexity of calculating internal forces in stiffeners induced by the tension field, Horne and Grayson [60] carried out a parametric FE study on the shear response of transverse stiffeners. The study accounted for initial imperfections, residual stresses, and both single- and double-sided stiffeners. The ultimate capacities of stiffened

webs were compared with those of fully simply supported panels. This investigation led to the expression for the ultimate bending rigidity given in Eq. (24).

3.2.2. Škaloud et al. [61]

In the same publication, *Optimum Rigidity of Stiffeners of Webs and Flanges*, Škaloud et al. [61] introduced the optimum ultimate bending rigidity of stiffeners, γ_{st2} , ensuring that no stiffener would fail prior to the ultimate resistance of the web plate. To achieve this, the transverse stiffener should be designed according to Eq. (25), incorporating a multiplicative factor k_{st2} in Eq. (26), which depends on the web slenderness $\lambda_w = h_w/t_w$. This factor can be linearly interpolated between 1 and 3 as: $k_{st2} = 0.0267(h_w/t_w - 1)$. The value of I_{st1} and γ_{st1} are taken as proposed previously by Massonnet et al. [30] (Eq. (12))

3.2.3. Stanway et al. [16]

Building on earlier research [38,39], Stanway et al. [16] investigated the development of post-buckling resistance in webs and its implications for the required transverse stiffener rigidity, γ_{st2} . They concluded that, at ultimate shear resistance, the stiffeners are primarily subjected to bending, resulting from their restraint of lateral web deflections. FEA led to a deflection limit criterion, expressed by Eq. (27), which must be verified against the permissible stiffener deflection, ω_{st} . The stiffener bending moment M_{st} is calculated following the method in Stanway et al. [16]; $b = \min\{a; h_w\}$, $\omega_{w,ult}$ is the maximum out-of-plane web deflection at ultimate load assuming rigid stiffeners, and k_λ is given by Eq. (28).

3.2.4. Lee et al. [9,44]

Lee et al. [9,44] examined the transverse stiffener requirements in the context of shear buckling resistance. For cases where the web slenderness results in $V_{cr}/V_{pl} \leq 0.8$, they found that a stiffener second moment of area of about six times greater than the one of AASHTO:2005 [64] (i.e. $I_{t1} \geq a t_w^3 J_1$, whereas the current is presented later in Eq. (42), was sufficient to ensure that the web reaches its ultimate shear resistance, as in Eq. (29), without the need for additional strength checks. The non-dimensional coefficient J_1 , given by Eq. (30) was used. Also, Lee et al. also mentioned that, for webs with $V_{cr}/V_{pl} > 0.8$, the AASHTO:2005 existing provisions of I_{st1} were deemed adequate, without any additional stiffener area requirement.

3.2.5. Sinur et al. [62]

In 2013, Sinur et al. [62] evaluated the minimum bending rigidity and area requirements prescribed by EN 1993-1-5 and proposed a unified expression for the ultimate design of intermediate transverse stiffeners in plate girders. This formulation was calibrated through an extensive parametric FE study but limited to cases where the flange-to-web area ratio was $A_f/A_w = 0.3$. The proposed design formula, applicable to single-sided stiffeners with $b_{st}/t_{st} = 10$, is presented in Eq. (31).

3.2.6. Kim et al. [11,18]

In 2004, Kim [63] conducted a FE study to determine the ultimate stiffener bending rigidity, γ_{st2} , required for I-shaped plate girders to reach their ultimate shear resistance. Web slenderness values of 150 and 300, with aspect ratios of 0.5, 1, 2, and 3, were considered. Both single- and double-sided stiffeners were analysed, using $b_{st}/t_{st} = 10$. The study included horizontally curved girders but excluded longitudinally stiffened webs. The models featured 485 MPa webs and stiffeners, combined with S690 flanges to reduce flange size while keeping maximum bending moment below $0.75 M_{pl,f}$. This resulted in flange-to-web area ratios A_f/A_w ranging from 0.33 to 1.02, with one outlier (1.88) for the longest panel with $\alpha = 3$ and $\lambda_w = 150$.

In 2007, Kim et al. [11] defined the required γ_{st2} as the rigidity beyond which further increase led to less than 3 % gain in ultimate shear

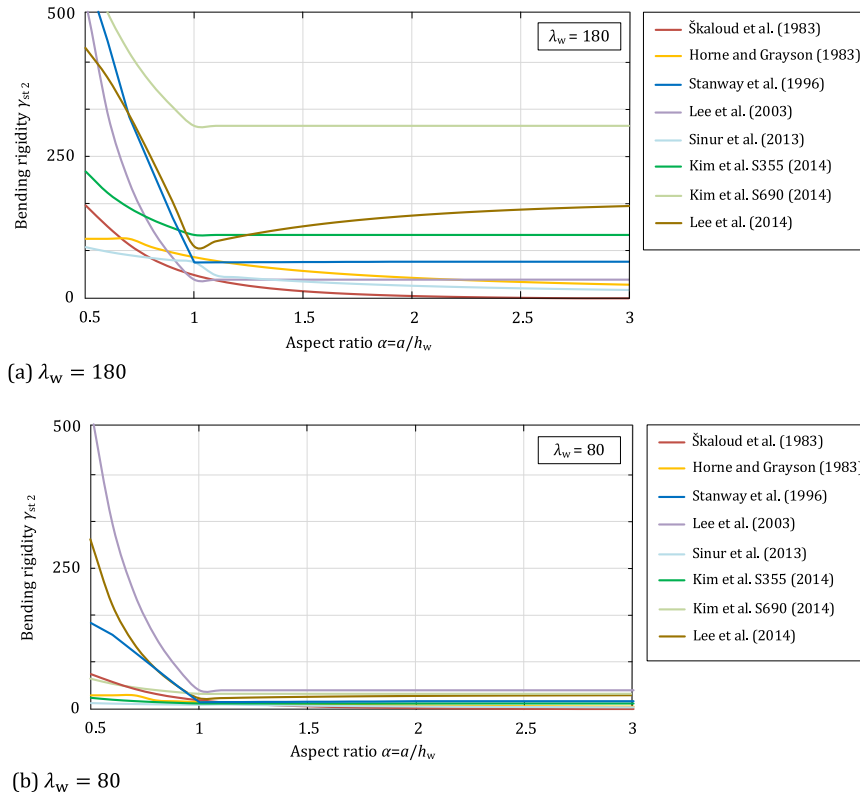


Fig. 4. Bending rigidity γ_{st2} of transverse stiffeners vs. α , as proposed by several authors, for: (a) $\lambda_w = 180$, (b) $\lambda_w = 80$.

resistance. They concluded that the condition for stiffener design corresponds to the web slenderness at which inelastic shear buckling (V_{cr}) equals the plastic shear resistance (V_{pl}), indicating minimal post-buckling tension field contribution.

Having the coefficient J from Eq. (30), and by substituting the web thickness t_w from Eq. (15), and $b = \min\{a; h_w\}$, into Eq. (16) (current AASHTO:2024 and AISC 360:2022 elastic design formula for transverse stiffeners – later presented in Section 4.2), Kim et al. proposed a simplified formula for the second moment of area I_{st2} (and corresponding bending rigidity γ_{st2}) defined in Eq. (17) [18].

$$\frac{h_w}{t_w} = 1.10 \sqrt{\frac{E k_\tau}{f_{y,w}}} \Leftrightarrow t_w = \frac{h_w}{1.10} \sqrt{\frac{f_{y,w}}{E k_\tau}} \quad (15)$$

$$\gamma_{st1} \geq 10.92 J_1 \Leftrightarrow I_{st1} \geq J b t_w^3 \quad (16)$$

$$\gamma_{st2} \geq \frac{10.92}{\beta} \lambda_w^3 \left(\frac{f_{y,w}}{E} \right)^{1.5} \Leftrightarrow I_{st2} \geq \frac{h_w^4}{\beta} \left(\frac{f_{y,w}}{E} \right)^{1.5} \quad (17)$$

The coefficient β in Eq. (18), was defined by Kim et al. [18], who recommended a constant value of $\beta = 40$. Additionally, the possibility of using stiffeners with yield strength lower than that of the web was addressed by introducing the parameter ρ_t , defined in Eq. (19), which adjusts the section modulus to ensure the same yield moment capacity as a stiffener with $f_{y,st} < f_{y,w}$.

$$\beta = \frac{1.33 k_\tau^{1.5}}{\max\left(\frac{2.5}{(a/h_w)^2} - 2.0; 0.5\right) \times \min(a/h_w; 1.0)} \quad (18)$$

$$\rho_t = \max(f_{y,w}/f_{y,st}; 1.0) \quad (19)$$

Accordingly, the second moment of area proposed by Kim et al. may be expressed in terms of I_{st2} and γ_{st2} as expressed in Eq. (32), and was adopted by AASHTO:2007 [47] and AISC 360:2010 [65].

3.2.7. Lee et al. [17]

In a continuation of earlier work [9,44], Lee et al. [17] proposed an updated expression for the required second moment of area of transverse stiffeners, accounting for post-buckling behaviour. They developed an extensive parametric study covering various web slenderness and aspect ratios, focusing on webs with $A_f/A_w = 0.9$, yield strengths of 245, 345 and 485 MPa, and two different stiffener plate widths. Based on its outcomes, a simplified design formula for I_{st2} was proposed as in Eq. (33), which is scaled by a factor n_{st2} given by Eq. (34). The improved J -value of Eq. (14) was employed where C is the ratio of the elastic buckling stress for a simply supported panel to the shear yield strength ($f_{y,w}/\sqrt{3}$). As C decreases, the contribution of post-buckling resistance increases, requiring greater stiffener rigidity than that needed for purely elastic buckling.

3.2.8. Xiao et al. [66]

In 2019, Xiao et al. [66] investigated the minimum stiffness requirements of intermediate transverse stiffeners in high-strength steel (HSS) plate girders considering post-buckling behaviour through FE analysis. Their results showed that the AASHTO design equations of I_{st2} underestimate the required moment of inertia for stiffeners in HSS girders and proposed a new formula to predict the stiffener inertia needed to fully develop the post-buckling resistance (in Eq. (35)), using an new empirical equation of J -value in Eq. (36).

3.2.9. Supplementary research

Early 1950s experimental work by Massonnet [31] indicated that, for stiffeners to remain effective up to girder failure, their flexural rigidity must be approximately 20 times higher than that required to maintain the elastic buckling nodal line. In 1989, Rahal [67], and later Rahal and Harding [68–70], considered out-of-plane stiffener forces and proposed an optimum γ_{st2} formulation, albeit with limited applicability in terms of panel aspect ratio. In 1999, Mijušković et al. [71] applied an energy-based large-deflection analysis to determine the optimum

Table 2
Summary of the key investigation on the design and requirements of intermediate transverse stiffeners reported in the literature.

Author	Year	Experimental		FEA	Desing proposal		Eq.	
		Qty.			Criteria	Formula		
Basler [45,46]	1961	Yes	13	No	Strength	$N_{st} = \left(1 - \frac{\tau_{cr}}{\tau_y}\right) \times \frac{a/h_w}{2} \times \left(1 - \frac{a/h_w}{\sqrt{1 + (a/h_w)^2}}\right) h_w t_w f_y$	(20)	
Rockey et al. [50]	1981	Yes	24	No	Strength	$N_{st} = V_{ult} - V_{cr}$	(21)	
Xie [53]	2000	Yes	4	Yes	Strength	$N_{st} = \frac{\tau_{cr} h_w t_w}{\sqrt{1 - (a/h_w)^2}} \left(\frac{\tau}{\tau_{cr}} - 1\right)$	(22)	
						$N_{st} = \begin{cases} 1/2 \sqrt{a/h_w} \left(\frac{\tau}{\tau_{cr}} - 1\right) (1 + 0.27 + 0.05 \ln(a/h_w) \ln(\delta_{st})) & \text{for } V > V_{cr} \\ 0 & \text{for } V \leq V_{cr} \end{cases}$	(23)	
Horne and Grayson [60]	1983	Yes	30	Yes	Stiffness	$\gamma_{st1} \geq \begin{cases} 0.6 (\lambda_w - 60) / (a/h_w) & \text{for } a/h_w > 0.75 \\ 0.8 (\lambda_w - 50) & \text{for } a/h_w \leq 0.75 \end{cases}$	$I_{st2} \geq \begin{cases} (0.055 (\lambda_w - 60) / (a/h_w)) a t_w^3 \\ (0.073 (\lambda_w - 50) / a) t_w^3 \end{cases}$	(24)
Škaloud et al. [61]	1983	Yes	10	No	Stiffness	$\gamma_{st2} \geq k_{st2} \gamma_{st1} \text{ with}$	$I_{st2} \geq k_{st2} I_{st1}$	(25)
						$k_{st2} = \begin{cases} 1 & \text{for } h_w/t_w \leq 75 \\ 3 & \text{for } h_w/t_w \geq 150 \end{cases}$	(26)	
Stanway et al. [16]	1996	No	-	Yes	Deflection	$\omega_{st} = \frac{M_{st} h_w^2}{E I_{st} \pi^2} \leq \omega_{st,lim} = \frac{0.125 \omega_{w,ult}}{1 - \cos\left(\frac{\pi k_\lambda b}{2 h_w}\right)} \text{ with}$	(27)	
						$k_\lambda = \begin{cases} 0.625 + \frac{0.375 h_w}{b} & \text{for } a/h_w < 3 \\ 1.75 & \text{for } a/h_w \geq 3 \end{cases}$	(28)	
Lee et al. [9,44]	2003	Yes	6	Yes	Stiffness	$\gamma_{st2} \geq 65.52 J_1 \text{ with}$	$I_{st2} \geq 6 J_1 t_w^3 a$	(29)
						$J_1 = \frac{2.5}{(a/h_w)^2} - 2.0 \geq 0.5$	(30)	
Sinur et al. [62]	2013	Yes	6	Yes	Stiffness	$\gamma_{st2} \geq \begin{cases} \frac{21.84(h_w/20)^4 \sqrt{(a/h_w)/5}}{(h_w/t_w)^{1/3} a t_w^3} & \text{for } a/h_w \leq 1; h_w/t_w \leq 250 \\ \frac{273(h_w/10)^4 \sqrt{(a/h_w)/5}}{(h_w/t_w)^{4/3} a t_w^3} & \text{for } a/h_w \leq 1; h_w/t_w > 250 \\ \frac{10.92 h_w^4}{20^3 (h_w/t_w) a t_w^3} & \text{for } a/h_w > 1; h_w/t_w \leq 250 \\ \frac{10.92 h_w^4}{400 (h_w/t_w)^{3/2} \sqrt{(a/h_w)} a t_w^3} & \text{for } a/h_w > 1; h_w/t_w > 250 \end{cases}$	$I_{st2} \geq \begin{cases} \frac{2 (h_w/20)^4 \sqrt{(a/h_w)/5}}{(h_w/t_w)^{1/3}} \\ \frac{25 (h_w/10)^4 \sqrt{(a/h_w)/5}}{(h_w/t_w)^{4/3}} \\ \frac{h_w^4}{20^3 (h_w/t_w)} \\ \frac{h_w^4}{400 (h_w/t_w)^{3/2} \sqrt{(a/h_w)}} \end{cases}$	(31)
Kim et al. [11,18]	2014	No	-	Yes	Stiffness	$\gamma_{st2} \geq \frac{0.273}{a/h_w} \lambda_w^3 \left(\frac{f_{y,w}}{E}\right)^{1.5}$	$I_{st2} \geq \frac{h_w^4 \rho_t^{1.3}}{40} \left(\frac{f_{y,w}}{E}\right)^{1.5}$	(32)
Lee et al. [17]	2014	No	-	Yes	Stiffness	$\gamma_{st2} \geq 10.92 n_{st2} J_1 \text{ with } J_1 \text{ in Eq. (14) and}$	$I_{st2} \geq n_{st2} J_1 a t_w^3$	(33)
						$n_{st2} = \frac{3.4}{C^{6/5}}$	(34)	
Xiao et al. [66]	2019	No	-	Yes	Stiffness	$\gamma_{st2} \geq 10.92 J_2 \text{ with}$	$I_{st2} \geq J_2 a t_w^3$	(35)
						$J_2 = \left[162.98 - 218.65(a/h_w) + 104.59(a/h_w)^2 - 17.43(a/h_w)^3 + 0.031(h_w/t_w)\right] \left(\frac{f_{y,w}}{E}\right)^{0.4}$	(36)	

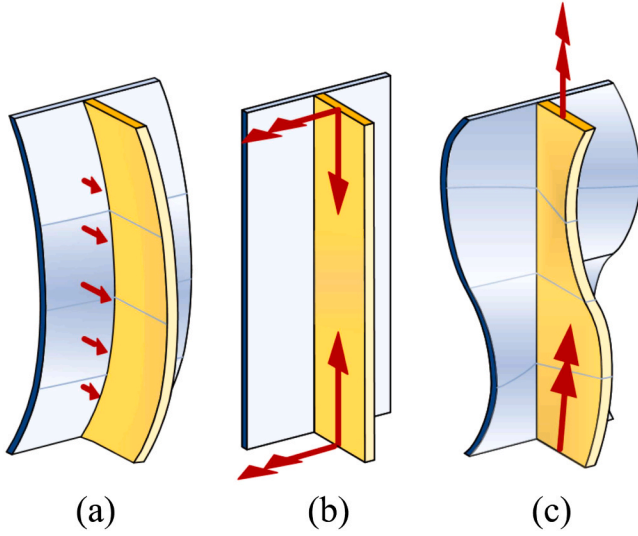


Fig. 5. Design check defined in [7].

stiffener rigidity under post-buckling shear. More recently, the topic has been further explored by Alinia [72] and Piculin [73].

3.2.10. Comparison between design models

Fig. 4 compares the principal proposals for the ultimate transverse stiffener bending rigidity, γ_{st2} , for web slenderness values $\lambda_w = 180$ and 80, respectively, where the horizontal axis represents the web panel aspect ratio a . The results indicate that the required γ_{st2} at ultimate load is generally more demanding than the elastic stiffener rigidity shown previously in Fig. 3. Several models show sensitivity to web slenderness (λ_w) and steel strength (f_y). Overall, γ_{st2} increases for shorter aspect ratios, more slender webs, and higher steel strengths.

Table 2 presents the summary of the key investigation on the design and requirements of intermediate transverse stiffeners reported in the literature.

4. Relevant standard design methods

4.1. EN 1993-1-5:2024 [7]

Intermediate transverse stiffeners must satisfy design criteria related to ultimate strength, bending rigidity, and torsional buckling rigidity, as illustrated in Fig. 5 (a-c).

4.1.1. Cross-section

The effective cross-section comprises the gross area of the stiffener plus a contributory web width of $15 \varepsilon t_w$ on each side, with $\varepsilon = \sqrt{235/f_y}$, as shown in Fig. 6 [7].

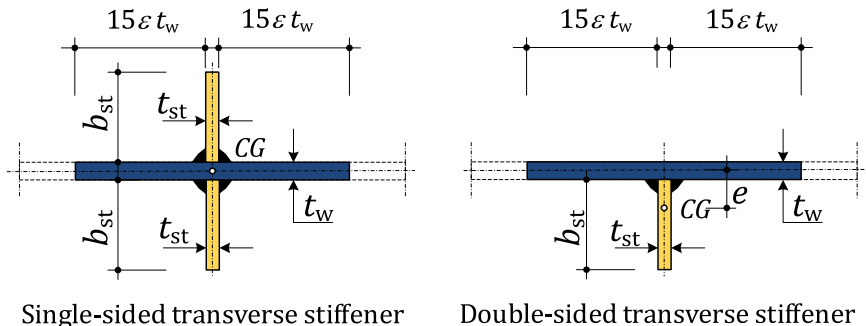


Fig. 6. Transverse stiffener effective cross-section [7].

4.1.2. Strength

Stiffeners are modelled as beam-columns subject to axial compression and lateral forces from the post-buckling support of the web. The ultimate strength is verified using the interaction Eq. (37) of EN 1993-1-1 [74], employing buckling curve c and an effective length $l \geq 0.75 h_w$. The axial force N_{st} results from TFA and is given by Eq. (38) which resembles the expression in Eq. (21). For asymmetric stiffeners, the induced bending moment M_{st} includes eccentricity e and the initial geometric imperfections $\omega_0 = s/200$:

$$\frac{N_{st}}{\chi_y N_{Rk} / \gamma_{M1}} + k_{yy} \frac{M_{st}}{\chi_{LT} M_{y,Rk} / \gamma_{M1}} \leq 1.0 \quad (37)$$

$$N_{st} = V_{Ed} - V_{cr} \quad (38)$$

$$M_{st} = N_{st} (e + \omega_0) \quad (39)$$

4.1.3. Bending rigidity

The minimum second moment of area I_{st1} (and respective γ_{st1}) of the effective section is defined by Eq. (40), that becomes independent of the panel aspect ratio for $a \geq \sqrt{2}$.

$$I_{st1} \geq \begin{cases} 1.5 h_w^3 t_w^3 / a^2 \\ 0.75 h_w t_w^3 \end{cases} \Leftrightarrow \gamma_{st1} \geq \begin{cases} 16.38 / (a/h_w)^3 & \text{for } a/h_w < \sqrt{2} \\ 8.19 / (a/h_w) & \text{for } a/h_w \geq \sqrt{2} \end{cases} \quad (40)$$

4.1.4. Torsional rigidity

For stiffeners without warping rigidity, the torsional stiffness is verified by limiting the relative slenderness to $\bar{\lambda}_{st} \leq 0.70$, where $\bar{\lambda}_{st} = \sqrt{f_{y,st} / \sigma_{cr,st}}$, which corresponds to have a critical elastic torsional stress of $\sigma_{cr,st} \geq 2 f_{y,st}$ (Eq.(41)). In the case of stiffeners with effective warping restraint, the limit is $\bar{\lambda}_{st} \leq 0.40$, equivalent to $\sigma_{cr,st} \geq 6 f_{y,st}$ (Eq. (41)):

$$\sigma_{cr,st} \geq \begin{cases} 2 f_{y,st}, & \text{no warping rigidity} \\ 6 f_{y,st}, & \text{with warping rigidity} \end{cases} \quad (41)$$

4.2. AASHTO:2024 [75] / AISC 360:2022 [76]

4.2.1. Cross-section

For second moment of area calculations, the transverse stiffener section is taken about the face in contact with the web (single-sided) or about the web mid-thickness (double-sided), without considering the web participation [75,76].

4.2.2. Strength

Since 1973, AASHTO standards recognised the need for sufficient stiffener area to resist the vertical component of the post-buckling tension field, based on Basler's [9]. However, given the relatively minor axial compression from the tension field, both AASHTO:2007 [47] and

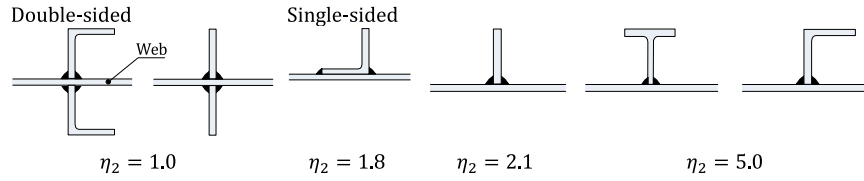


Fig. 7. Value of η_2 for the design of transverse stiffeners according to SIA 263.

AISC 360:2010 [65] removed the explicit strength (area) requirement from their provisions.

4.2.3. Bending rigidity

The current AASHTO:2024 [75] and AISC 360:2022 [76] standards define the minimum required bending rigidity I_{st1} (Eq. (42)) to ensure near-zero lateral deflection at the stiffener under the critical shear stress τ_{cr} . The panel width is defined as $b = \min(a; h_w)$, and the required stiffness J is expressed by Eq. (43).

$$I_{t1} \geq b t_w^3 J \quad (42)$$

$$J = \frac{2.5}{(a/h_w)^2} - 2.0 \geq 0.5 \quad (43)$$

For ultimate shear design, the enhanced requirement I_{st2} is given by Eq. (44), which incorporates post-buckling resistance. For stocky webs, I_{st2} tends to I_{st1} . The factor ρ_t is defined in Eq. (19), and the critical stress in the stiffener $\sigma_{cr,st}$ is limited by Eq. (45):

$$I_{t2} \geq \frac{h_w^4 \rho_t^{1.3}}{40} \left(\frac{f_{y,w}}{E} \right)^{1.5} \quad (44)$$

$$\sigma_{cr,st} = \frac{0.31 E}{(b_{st}/t_{st})^2} \leq f_{y,st} \quad (45)$$

4.2.4. Torsional rigidity

No explicit torsional rigidity requirements are provided. However, AASHTO [75] specifies minimum geometric limits for stiffeners:

$$b_{st} \geq 2.0 + \frac{h_w}{30} \quad (46)$$

$$16 t_{st} \geq b_{st} \geq \frac{b_{f,comp}}{4} \quad (47)$$

4.3. DNV-RP-C201: 2010 [77]

The Norwegian Standard DNV-RP-C201:2010 [77] provides general buckling verification rules for stiffened plates under shear, direct stresses, and lateral loads, but does not present specific provisions for transverse or longitudinal stiffeners, except regarding the required second moment of area for transverse stiffeners.

4.3.1. Cross-section

No guidance is given on the use of effective stiffener cross-sections or on the contribution of adjacent web width.

4.3.2. Strength

For stiffened plates under combined loading, the equivalent stiffener axial load, N_{st} , is defined by Eq. (48), based on the stiffener area A_{st} , the panel width b , and the combined direct, σ_x , and post-critical shear stresses, τ_{pc} of Eq. (48), where $\tau_{pc} = \tau_u - \tau_{cr,\infty}$, if post-buckling shear is considered.

$$N_{st} = \sigma_x (A_{st} + b t_w) + \tau_{pc} b t_w \quad (48)$$

4.3.3. Bending rigidity

The required second moment of area for a transverse stiffener ac-

counting for shear post-buckling is defined in Eq. (49). This expression is valid for aspect ratios up to $a/h_{st} = 1.118$ and assumes full flange participation over panel width a .

$$\begin{aligned} I_{st2} &\geq 0.3 h_{st} a^2 t_w \left(2.5 \frac{h_{st}}{a} - 2 \frac{a}{h_{st}} \right) \frac{f_{y,st}}{E} \Leftrightarrow \gamma_{st2} \\ &\geq 3.276 \frac{h_{st} a}{t_w^2} \left(2.5 \frac{h_{st}}{a} - 2 \frac{a}{h_{st}} \right) \frac{f_{y,st}}{E} \end{aligned} \quad (49)$$

4.3.4. Torsional rigidity

The characteristic buckling and torsional buckling capacities of transverse stiffeners are defined by Eqs. (50) and (51), though detailed expressions are not explicitly discussed.

$$\frac{\sigma_{cr,st}}{f_y} = \begin{cases} 1.0 & \text{for } \bar{\lambda}_{st} \leq 0.2 \\ \frac{1 + \mu + \bar{\lambda}_{st}^2 - \sqrt{(1 + \mu + \bar{\lambda}_{st}^2)^2 - 4 \bar{\lambda}_{st}^2}}{2 \bar{\lambda}_{st}^2} & \text{for } \bar{\lambda}_{st} > 0.2 \end{cases} \quad (50)$$

$$\frac{\sigma_{cr,st}}{f_y} = \begin{cases} 1.0 & \text{for } \bar{\lambda}_{st} \leq 0.6 \\ \frac{1 + \mu + \bar{\lambda}_{st}^2 - \sqrt{(1 + \mu + \bar{\lambda}_{st}^2)^2 - 4 \bar{\lambda}_{st}^2}}{2 \bar{\lambda}_{st}^2} & \text{for } \bar{\lambda}_{st} > 0.6 \end{cases} \quad (51)$$

4.4. SIA 263:2013 [48]

4.4.1. Cross-section

SIA 263-1:2013 [48] recommends that the second moment of area for both single- and double-sided transverse stiffeners should be calculated about an axis lying in the plane of the web. Only for bearing or support stiffeners, a portion of the adjoining web width may be considered as contributing to the strength verification.

4.4.2. Strength

SIA 263-1:2013 [48] defines the compressive force in transverse stiffeners based on Basler's formulation of Eq. (52). The required stiffener area depends on the material strength ratio η_1 (Eq. (53)). For single-sided stiffeners, the effect of eccentricity relative to the web is accounted for by the coefficient η_2 , which reflects the stiffener cross-section geometry (illustrated in Fig. 7). A reduction factor η_3 , related to the design shear resistance, is applied as defined in Eq. (54).

$$N_{st} = \left(1 - \frac{\tau_{cr}}{\tau_y} \right) \times \frac{a/h_w}{2} \times \left(1 - \frac{a/h_w}{\sqrt{1 + (a/h_w)^2}} \right) h_t t_w f_y \eta_1 \eta_2 \eta_3 \quad (52)$$

$$\eta_1 = f_{y,w} / f_{y,st} \quad (53)$$

$$\eta_3 = V_{Ed} / V_{Rd} \quad (54)$$

4.4.3. Bending rigidity

SIA 263-1:2013 [48] requires a second moment of area of the transverse stiffener higher than the value defined in Eq. (55). In this expression, η_1 is given by Eq. (53) and h_t denotes the vertical distance between the centroids of the flanges.

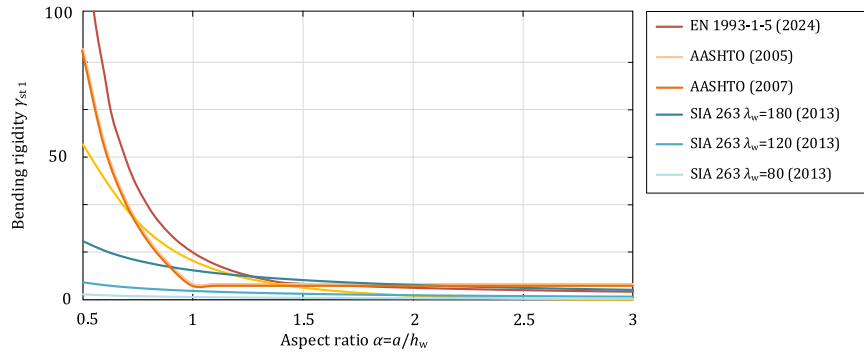


Fig. 8. Bending rigidity γ_{st1} of a transverse stiffener vs. α , in accordance with relevant standards.

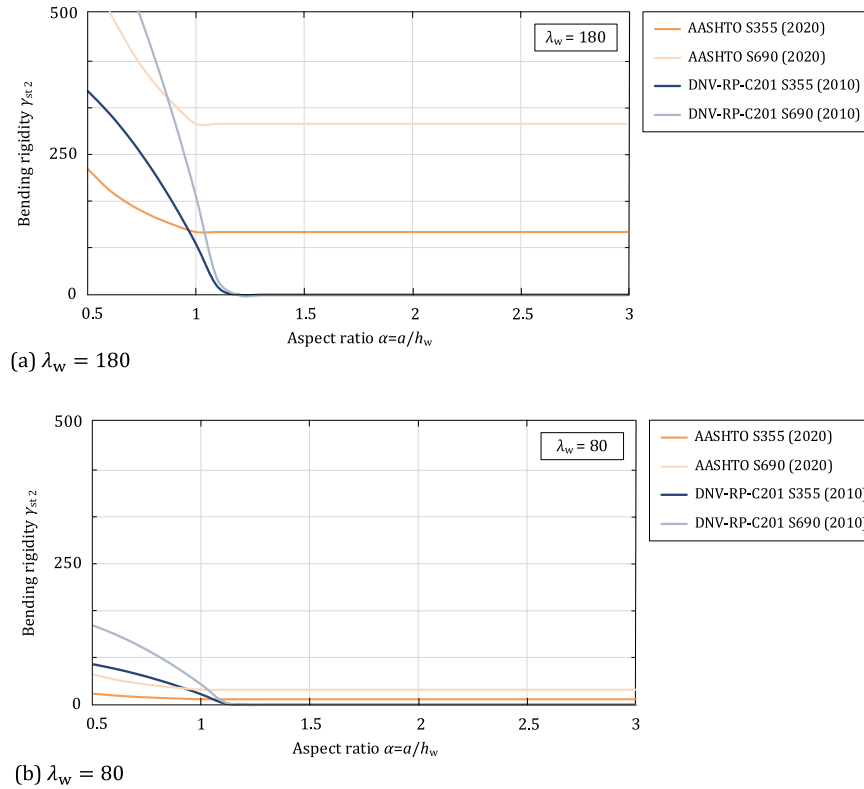


Fig. 9. γ_{st2} vs. α of a transverse stiffeners per AASHTO (2020) and DNV (2010) for: (a) $\lambda_w = 180$, (b) $\lambda_w = 80$.

$$I_{st1} \geq \left(\frac{h_f}{50}\right)^4 \eta_1^{3/2} \Leftrightarrow \gamma_{st1} \geq \frac{10.92}{a t_w^3} \left(\frac{h_f}{50}\right)^4 \eta_1^{3/2} \quad (55)$$

4.4.4. Torsional rigidity

No specific torsional rigidity requirements are specified in SIA 263–1:2013 [48].

4.5. Comparison between design models bending rigidity

The standard design models for intermediate transverse stiffeners, based on a rigidity criterion (γ_{st1}) for the web plate to attain its elastic critical web load, are compared in Fig. 8. According to this figure, the required critical stiffener bending rigidity γ_{st1} as prescribed by the standards is consistent with the values proposed by the main researchers previously discussed.

Fig. 9 compares the standard formulations for designing transverse stiffener (with γ_{st2}), for the web to reach its ultimate shear buckling load. As γ_{st2} depends on the web slenderness λ_w , Fig. 9 (a) and (b) correspond

to $\lambda_w = 180$ and 80 , respectively. As shown in this figure, the standard ultimate stiffener bending rigidity, γ_{st2} , is usually higher than the standard critical web stiffener bending rigidity, γ_{st1} . The Norwegian standard proved particularly stringent for aspect ratios $\alpha \leq 1.12$, while indicating that for $\alpha > 1.12$, no significant requirements on stiffener bending rigidity are necessary.

In contrast, the AASHTO specification of 2024, being based on the work of Kim et al. [11,18,63], yielded identical results, as expected, and consistent with those in Fig. 4. In line with the author's previous proposals, the standard ultimate stiffener bending rigidity γ_{st2} was found to depend on the web slenderness λ_w and the steel grade f_y . As illustrated in Fig. 9, γ_{st2} increased with decreasing aspect ratio, increasing web slenderness, and higher steel grade.

Table 3 presents the list of the most relevant standards contemplated in the current paper concerning alongside the related author work reference for the design of intermediate transverse stiffeners and respective design formula.

Table 3
Resume of the of relevant standard design verifications for intermediate transverse stiffeners.

Standard	Cross-section		Strength		Elastic stiffness γ_{st}		Ultimate stiffness γ_{st2}	
	Author	Formula	Author	Formula	Author	Formula	Author	Formula
EN 1993-1-5:2024 [7]	Rockey et al. [50]	$V_{Ed} - V_{cr}$	Kerensky et al. [24]	$\gamma_{st1} \geq \begin{cases} 16.38 & \text{for } a/h_w < \sqrt{2} \\ (a/h_w)^3 & \text{for } a/h_w \geq \sqrt{2} \end{cases}$	-	-	-	-
AASHTO:2024 [75]	- (*)	- (**)	Bleich [27]	$\gamma_{st1} \geq \begin{cases} 8.19 & \text{for } a/h_w < \sqrt{2} \\ a/h_w & \text{for } a/h_w \geq \sqrt{2} \end{cases}$	Kim et al. [11]	$\gamma_{st2} \geq \begin{cases} 0.273 \lambda_w^{-3} \rho_t^{1.3} \left(\frac{f_{yw}}{E} \right)^{1.5} & \text{for } a/h_w < 1 \\ 0.273 \lambda_w^{-3} \rho_t^{1.3} \left(\frac{f_{yw}}{E} \right)^{1.5} & \text{for } a/h_w \geq 1 \end{cases}$	-	-
AISC 360:2022 [76]	- (*)	- (**)	Bleich [27]	$\gamma_{st1} \geq \begin{cases} 27.3 & \text{for } a/h_w < 1 \\ (a/h_w)^2 - 21.3 & \text{for } a/h_w \geq 1 \end{cases}$	Kim et al. [11]	$\gamma_{st2} \geq \begin{cases} 0.273 \lambda_w^{-3} \rho_t^{1.3} \left(\frac{f_{yw}}{E} \right)^{1.5} & \text{for } a/h_w < 1 \\ 0.273 \lambda_w^{-3} \rho_t^{1.3} \left(\frac{f_{yw}}{E} \right)^{1.5} & \text{for } a/h_w \geq 1 \end{cases}$	-	-
DNV-RP-C201:2010 [77]	see [77]	$\sigma_x(A_{st} + b t_w) + \tau_{pc} b t_w$	-	-	Unknown Ref.	$\gamma_{st2} \geq 3.276 \lambda_w^{-2} \left(2.5 - 2 \left(\frac{a}{h_{st}} \right)^2 \right) \frac{f_{y, st}}{E}$	-	-
SIA 263-1:2013 [48]	25 ϵ t_w	$V_{Ed} - V_{cr}$	Basler [46]	$\gamma_{st1} \geq \frac{12(1 - \nu^2)}{50^3} \frac{h_t^3}{t_w^3} \frac{1}{a^{3/2}}$ (*)	-	-	-	-

(*) Effective web width and cross-section were recommended in former standard version before suppression

(**) Stiffener axial force was recommended in former standard version before suppression

5. Improved design method for transverse web stiffeners according to [1]

5.1. Basic conditions

This section introduces an improved design method for intermediate transverse stiffeners in steel plate girders, based on the following key assumptions:

1. The stiffener is designed to maintain stability up to the girder's ultimate shear resistance, giving the assumptions outlined in Section 5.2;
2. The design targets the maximum shear capacity of the web panel (assessed using the numerical shear capacity $V_{\max, \text{FEM}}$), rather than the applied shear force V_{Ed} ;
3. The stiffener cross-section is treated as effective, in accordance with EN 1993-1-5:2024.

The proposed design method is subject to the following constraints:

- It applies to intermediate transverse stiffeners engaged by TFA, excluding those at supports;
- It does not cover stiffeners subjected to external loads, such as those arising from cross-frames, flange-level loads, girder curvature, or slope discontinuities;
- The calibration was performed for single-sided flat plate stiffeners under pure shear, though the approach may be extended to other geometries and loading conditions as detailed in subsequent sections.

5.2. Procedure for determining the optimum bending rigidity

The effectiveness of the intermediate transverse stiffener is numerically evaluated by adjusting its cross-sectional bending rigidity, γ_{st} , for a given FE girder geometry – which incorporates a portion of the web that works in conjunction with the stiffener. This procedure is detailed in [1] and [78]. Moreover, the FE model validation is available in [1] and [79].

Fig. 10 (a) plots the normalized shear resistance numerically obtained using the ratio $V_{\max, \text{FEM}}/V_{pl}$ (circles marked in grey) for several stiffener bending rigidity, γ_{st} , which are represented on the horizontal axis. These results were interpolated, obtaining the blue line in Fig. 10 (a) which quantifies the increase in girder resistance due to the use of the transverse stiffener. The same figure highlights the $V_{\max, \text{FEM}}/V_{pl}$ scores for two plate girders examples ($\lambda_w=167$, $\alpha=1$, $A_f/A_w=1.0$, S355 longitudinally unstiffened): a flexible transverse stiffener that has $\gamma_{st}=5$ (circle in red) – which failed to obtained the maximum resistance of the web panel (about $0.76 V_{pl}$) – and for a rigid transverse stiffener designed with $\gamma_{st}=1000$ (circle in green) – which successfully attained the maximum resistance of the web panel of $0.76 V_{pl}$.

This behaviour is supported by the normalized load vs. vertical displacement plot of Fig. 10 (b) of two plate girders examples together with the corresponding numerical representation of the failure mode. This figure adds essential aspects concerning the stiffened girder behaviour such as the load stage at which one FE within the plate girder attained the strain limit of 5 % – from this point forward, the $P-\delta$ plot is drawn in grey. Moreover, the plate girder with a rigid transverse stiffener designed with $\gamma_{st}=1000$ (curve in green) shown a very similar behaviour to a reference plate girder with similar geometry and material but with a single web panel – which failed due to shear buckling in a single web panel without the influence of the transverse stiffener. On the other hand, the plate girder with the flexible stiffener was unable to sustain the vertical load after peak load – failing in both the web plate and stiffener together.

The optimum value of γ_{st} is indicated by the blue dot in Fig. 10 (a)

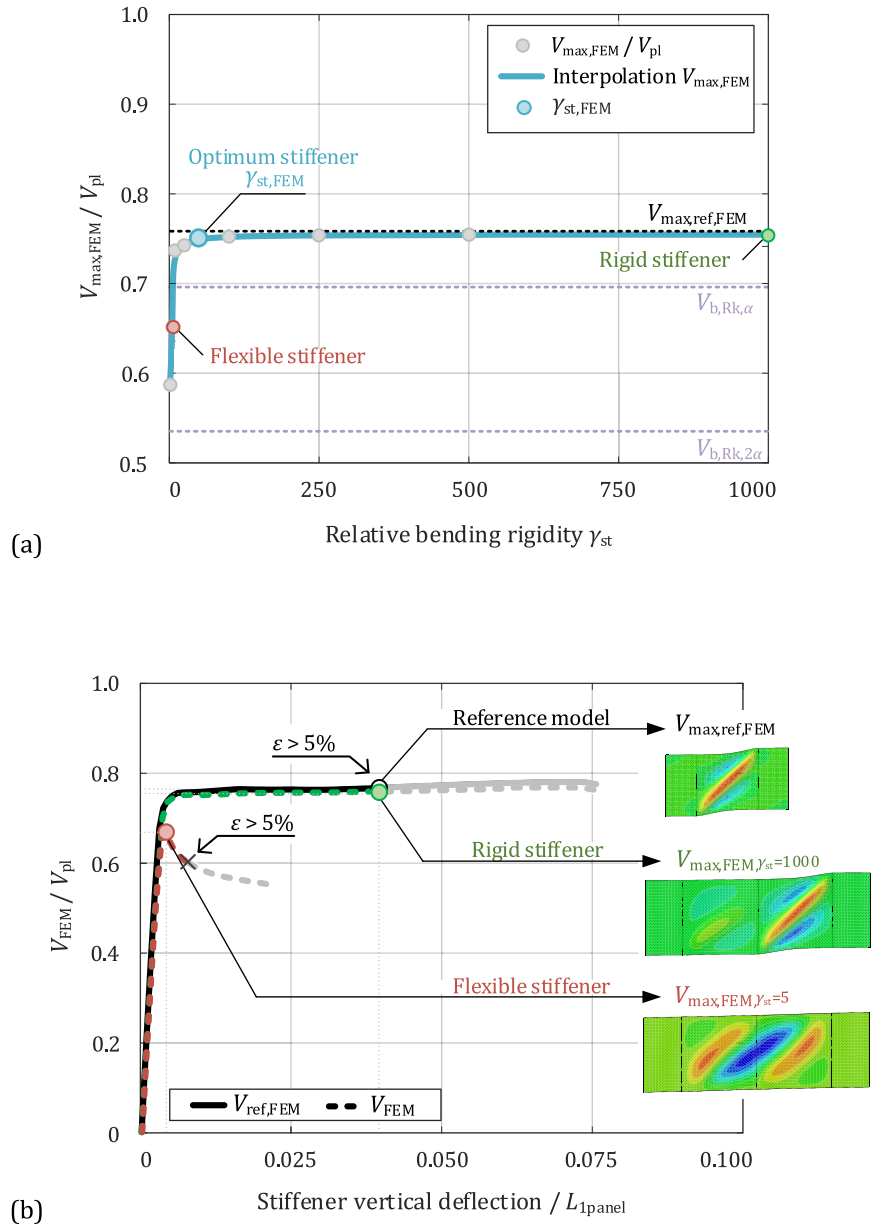


Fig. 10. Effectiveness of an intermediate transverse stiffener: (a) definition of $\gamma_{\text{st}, \text{FEM}}$, (b) load-displacement plot of two examples.

and is determined when $V_{\max, \text{FEM}}(i) / V_{\max, \text{FEM}}(i + 1) < 0.1\%$, which signifies the point at which further increases in γ_{st} no longer result in a significant increase in the relative resistance $V_{\max, \text{FEM}} / V_{\text{pl}}$. The resistance of a reference model with a single web panel without the stiffener influence is referred as $V_{\max, \text{ref}, \text{FEM}}$ in both figures and the optimum bending rigidity ensures a girder resistance of at least $V_{\max, \text{FEM}} \geq 0.9 V_{\max, \text{ref}, \text{FEM}}$.

In Fig. 10 (a), the characteristic shear buckling resistance, $V_{\text{b}, \text{Rk}, \alpha}$, for a plate girder with a web panel stiffened with a rigid intermediate transverse stiffener having the EN 1993–1–5:2024 rules are represented by a purple line. As a lower bond EN 1993–1–5:2024 reference, the characteristic shear buckling resistance, $V_{\text{b}, \text{Rk}, 2\alpha}$, of the scenario of a plate girder with a web panel stiffened with a very flexible intermediate transverse stiffener.

5.3. Design proposal development

To derive the required ultimate bending rigidity of a transverse stiffener, a procedure similar to that described in [18] is adopted. The

minimum second moment of area I_{st1} of an intermediate transverse stiffener to develop the critical shear resistance of a web panel with slenderness $\lambda_{\text{w}} = h_{\text{w}} / t_{\text{w}}$, as defined in EN 1993–1–5:2024, is calculated using Eq. (40). However, to develop the plastic shear resistance $V_{\text{pl}} = f_{\text{y}, \text{w}} h_{\text{w}} t_{\text{w}} / \sqrt{3}$, according to EN 1993–1–5:2024, the higher web panel slenderness is,

$$\frac{h_{\text{w}}}{t_{\text{w}}} = 31 \frac{\epsilon}{\eta} \sqrt{k_{\tau}} \quad (56)$$

Here, η is a hardening factor recommended to be 1.2 for steel grades up to S460 and 1.0 for grades above S460 [7]. The yield factor is given by $\epsilon = \sqrt{235 / f_{\text{y}}}$, and the shear buckling coefficient k_{τ} is defined in Eq. (57) assuming simple supported edges of the web panel:

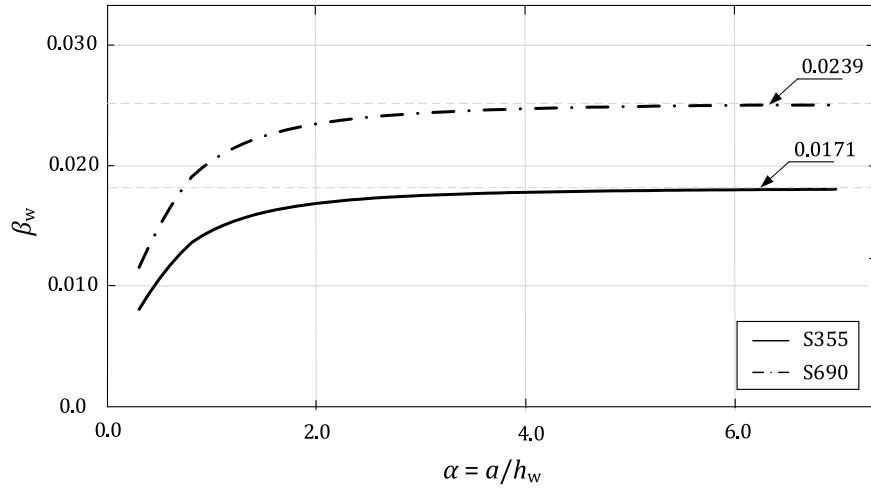


Fig. 11. Variation of factor β_w for $\eta = 1.0$ with respect to $\alpha = a/h_w$.

$$k_\tau = \begin{cases} 4.00 + \frac{5.34}{(a/h_w)^2} & \text{for } a/h_w < \sqrt{2} \\ 5.34 + \frac{4.00}{(a/h_w)^2} & \text{for } a/h_w \geq \sqrt{2} \end{cases} \quad (57)$$

Substituting Eq. (56) into Eq. (40) and solving for the resulting second moment of area I_{st2} and bending rigidity γ_{st2} are obtained by:

$$I_{st2} \geq \begin{cases} 1.50 \frac{h_w^6 \beta_w^3}{a^2} & \text{for } a/h_w < \sqrt{2} \\ 0.75 h_w^4 \beta_w^3 & \text{for } a/h_w \geq \sqrt{2} \end{cases} \Leftrightarrow \gamma_{st2} \quad (58)$$

$$\geq \begin{cases} \frac{16.38}{(a/h_w)^3} \lambda_w^3 \beta_w^3 & \text{for } a/h_w < \sqrt{2} \\ \frac{8.19}{a/h_w} \lambda_w^3 \beta_w^3 & \text{for } a/h_w \geq \sqrt{2} \end{cases}$$

The non-dimensional factor β_w , defined in Eq. (59) for $\eta = 1.0$, varies according to the steel grade and tends to a constant value as the panel aspect ratio increases (see Fig. 11):

$$\beta_w = \frac{1}{31\epsilon\sqrt{k_\tau}} \quad (59)$$

Considering γ_{st2} from Eq. (58), the following major conclusions can be drawn:

- The required γ_{st2} is identical to the required γ_{st1} (Eq. (40)) multiplied by the cubic exponential of the web slenderness λ_w and the factor β_w . Therefore, the term $\lambda_w^3 \beta_w^3$ may be interpreted as a post-buckling amplification factor applied to the critical bending rigidity γ_{st1} , leading to the ultimate bending rigidity γ_{st2} , assuming the web fails at the plastic shear resistance V_{pl} ;
- The required bending rigidity γ_{st2} increases with the steel grade f_y ;
- The required bending rigidity γ_{st2} decreases as the aspect ratio α increases;
- The factor β_w increases with the aspect ratio α and tends to a constant value for large panel width α .

However, the required second moment of area I_{st2} for reaching the ultimate shear resistance of the web plate, as given in Eq. (58) (also expressed in terms of relative bending rigidity), should also be adjusted to account for the influence of the flange-to-web-area ratio, as evidenced by FEM results, see [78,1]. Accordingly, the following section presents a refined design proposal based on the adjusted values of I_{st2} and γ_{st2} .

5.4. New design proposal

An improved formulation for the ultimate bending rigidity of intermediate transverse stiffeners is proposed in Eq. (60), introducing an amplification factor Ψ to account for post-buckling behaviour and influence of the flanges (given that $I_{st2} = \gamma_{st2} \frac{t_w^3 a}{12(1-\nu^2)}$):

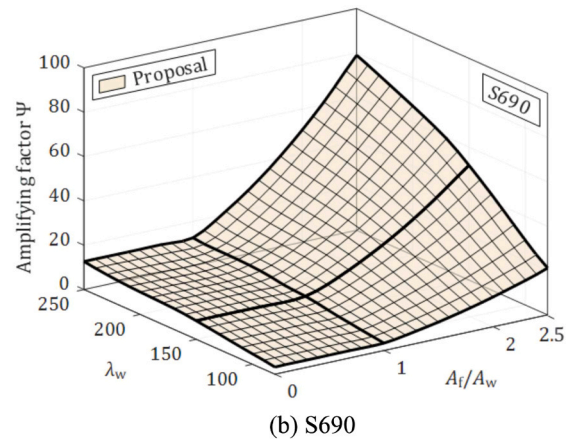
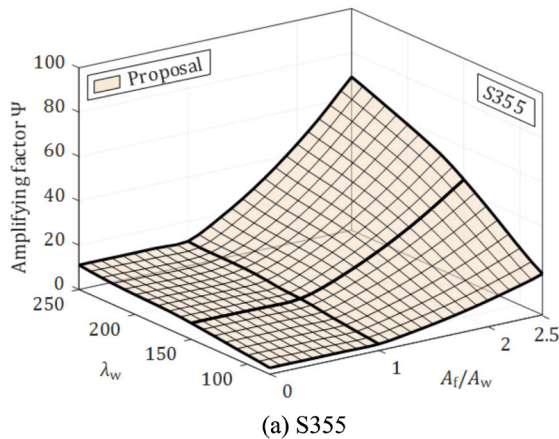


Fig. 12. Variation of the amplifying factor Ψ with respect to λ_w and A_f/A_w for: (a) S355, (b) S690.

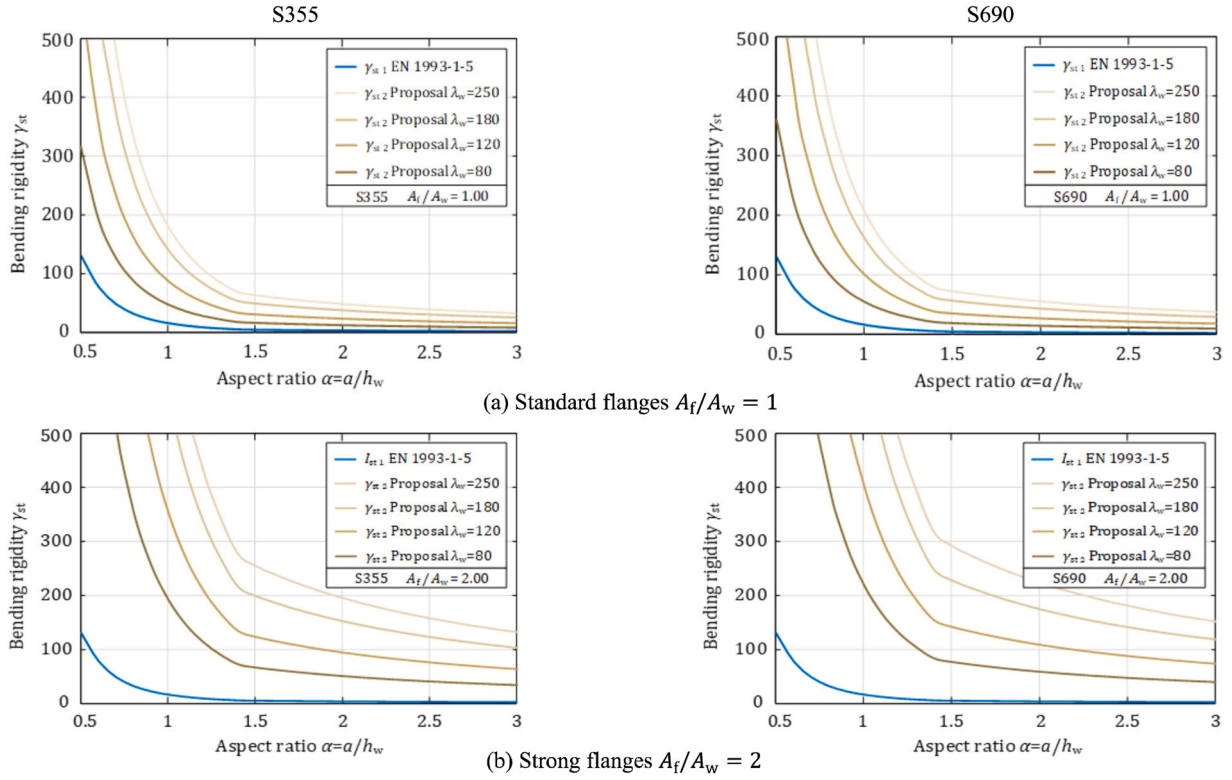


Fig. 13. Variation of the required bending rigidity γ_{st1} and γ_{st2} with respect to $\alpha = a/h_w$ for S355 and S690: (a) $A_f/A_w = 1$, (b) $A_f/A_w = 2$.

$$I_{st2} \geq \Psi I_{st1} \Leftrightarrow \gamma_{st2} \geq \Psi \gamma_{st1} \quad (60)$$

Here, γ_{st1} and I_{st1} refer to the actual EN 1993–1–5:2024 elastic design values from Eq. (40). The amplification factor Ψ reflects the post-buckling contribution of TFA and its general expression is provided in Eq. (61), which should be higher or equal to 1.0:

$$\Psi = \begin{cases} \left(\frac{\lambda_w}{\sqrt{150}} \right)^{1.5} \beta_w^{0.4} \zeta & \text{for } \lambda_w < 150 \\ \lambda_w^{0.75} \beta_w^{0.4} \zeta & \text{for } \lambda_w \geq 150 \end{cases} \quad (61)$$

The amplification factor Ψ was calibrated using numerical simulations, as described in Section 5.2, with the corresponding FE results presented in [1,78]. This factor is governed by the web slenderness λ_w , the flange factor ζ in Eq. (62) (dependent on the flange-to-web area ratio A_f/A_w), the steel-grade factor β_w in Eq. (59) (through ε), and the aspect ratio α (through k_r). These dependencies are consistent with recent investigations [1,78], which identified these parameters as the main factors governing the required bending rigidity of transverse stiffeners. Accordingly, the general expression for Ψ differentiates between two slenderness regimes ($\lambda_w < 150$ and $\lambda_w \geq 150$) and two flange-area-ratio ranges ($A_f/A_w < 1$ and $A_f/A_w \geq 1$), resulting in four design domains, as illustrated in Fig. 12. The relatively minor influences of the steel grade and aspect ratio are accounted for through the β_w factor.

$$\zeta = \max \left(\left(\frac{A_f}{A_w} \right)^2 ; 1.0 \right) \quad (62)$$

Fig. 13 compares the proposed γ_{st2} to EN 1993–1–5 design values of γ_{st1} , confirming that γ_{st2} is consistently higher due to additional post-buckling demands, particularly for slenderer webs, and larger flange-to-web ratios but also for higher steel grades.

5.5. Comparison to numerical results

The proposed intermediate stiffener requirement γ_{st2} was compared

with numerically derived demands γ_{FEM} , both normalised to the EN 1993–1–5:2024 elastic requirement γ_{st1} . The resulting amplification factor $\Psi = \gamma_{st2}/\gamma_{st1}$ reflects the increase needed to reach ultimate shear resistance. Figs. 14–16 show this comparison for steel grades S355 and S690 across $\alpha = 1.0$ – 2.0 , and for three web configurations: (a) unstiffened, (b) with a longitudinal stiffener at $0.25 h_w$, and (c) at $0.50 h_w$. For $\alpha \geq 2$, FE indicate low rigidity demands, suggesting the proposed values may be conservative. This aligns with the fact that the EN 1993–1–5:2024 expression for I_{st1} becomes constant beyond $\alpha \geq \sqrt{2}$. Across all cases, γ_{st2} consistently exceeds γ_{FEM} , confirming the adequacy and conservativeness of the proposed design method for various steel grades and web configurations.

5.6. Other effects

5.6.1. Influence of longitudinal stiffeners

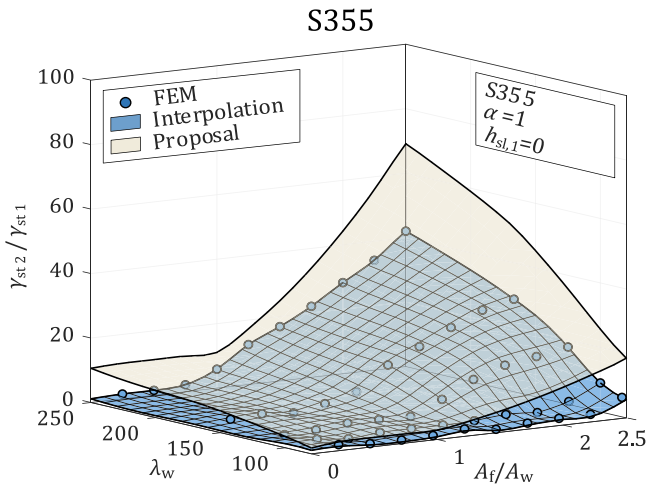
For webs with longitudinal stiffeners, EN 1993–1–5:2024 requires an additional verification of transverse stiffener rigidity. This is addressed through the amplification factor (Ψ) of Eq. (40) applied to the EN 1993–1–5:2024 minimum second moment of area, ensuring conservative and sufficient rigidity for longitudinally stiffened webs [1,78].

5.6.2. Influence of double-sided stiffeners

Studies [1,78] confirm that single- and double-sided transverse stiffeners provide equivalent out-of-plane restraint if they offer the same relative rigidity (γ_{st}). Thus, the proposed design method applies equally to both configurations.

5.6.3. Effect of plate girders with non-symmetric flanges

Non-symmetric flanges are common in practice due to construction and composite action with slabs. For typical flange-to-web area ratios ($A_{f,t}/A_w = 2.0$ top, $A_{f,b}/A_w = 1.0$ bottom), required stiffener rigidity falls between those for symmetric cases [78,1]. In the absence of detailed analysis, adopting the higher A_f/A_w ratio is recommended.

$\alpha = 1$ 

(a) no longitudinal stiffener

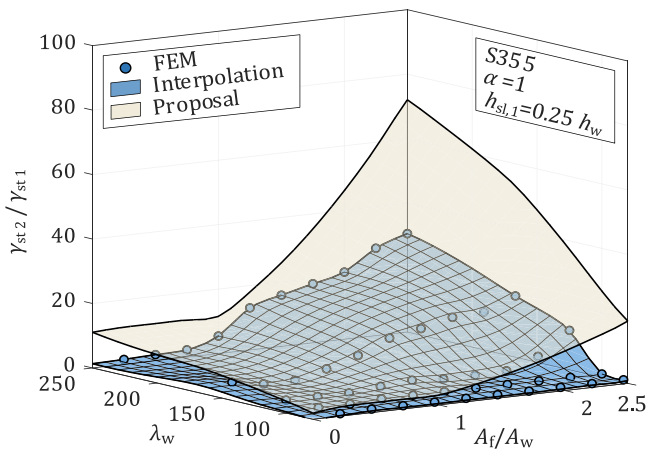
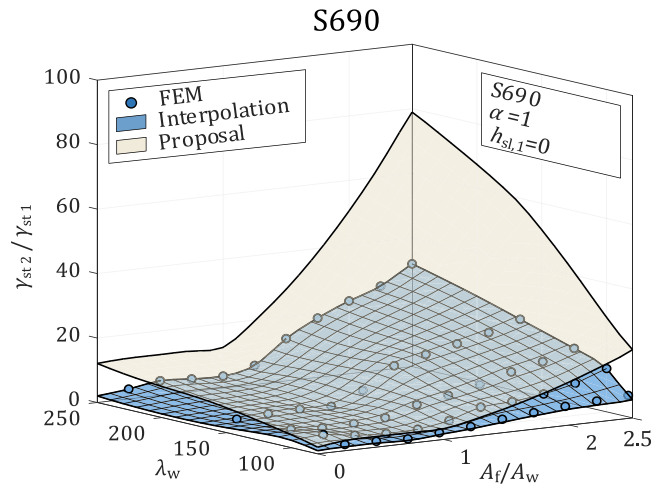
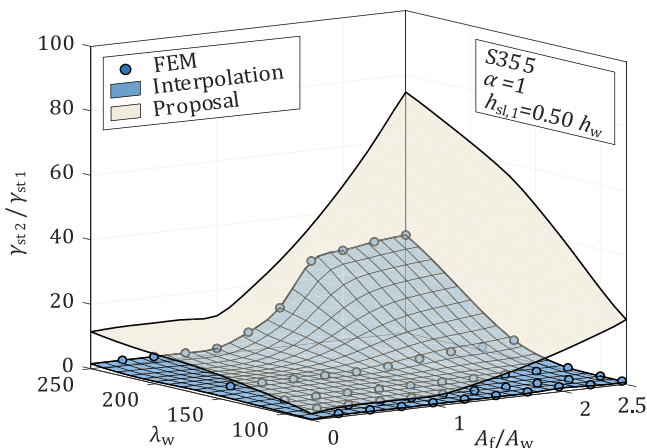
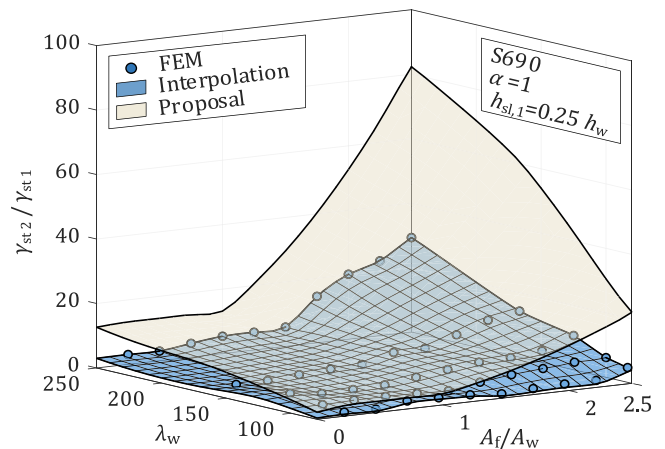
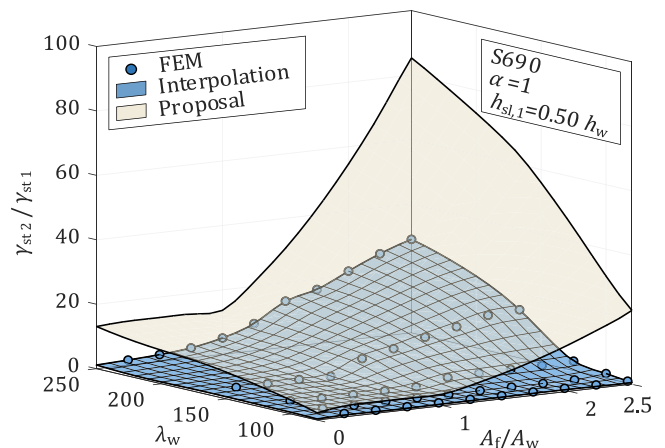
(b) longitudinal stiffener at $h_{sl,1} = 0.25 h_w$ (c) longitudinal stiffener at $h_{sl,1} = 0.50 h_w$ 

Fig. 14. Comparison of the required amplification factor $\Psi = \gamma_{st,2} / \gamma_{st,1}$ between numerical results and the proposed formulation for $\alpha = 1$: (a) no longitudinal stiffener, (b) $h_{sl,1} = 0.25 h_w$, (c) $h_{sl,1} = 0.50 h_w$ [78].

5.6.4. Influence of tee stiffeners layout

The effectiveness of tee-section stiffeners was assessed in [1,78], with geometric constraints ensuring structural class and flange restraint. Limits were set for flange width-to-thickness ($b_{st,f}/t_{st,f} \leq 10\epsilon$), flange width ($b_{st,f} = 2.5 b_{st,w}$), and stiffener properties ($b_{st}/t_{st} \leq 20$ and

$b_{st} \leq b_f/2$). Despite a minor ($<10\%$) reduction in shear resistance, it was concluded that the stiffener layout has little effect on required bending rigidity $\gamma_{st,FEM}$, confirming the method's applicability to tee stiffeners.

$$\alpha = 1.5$$

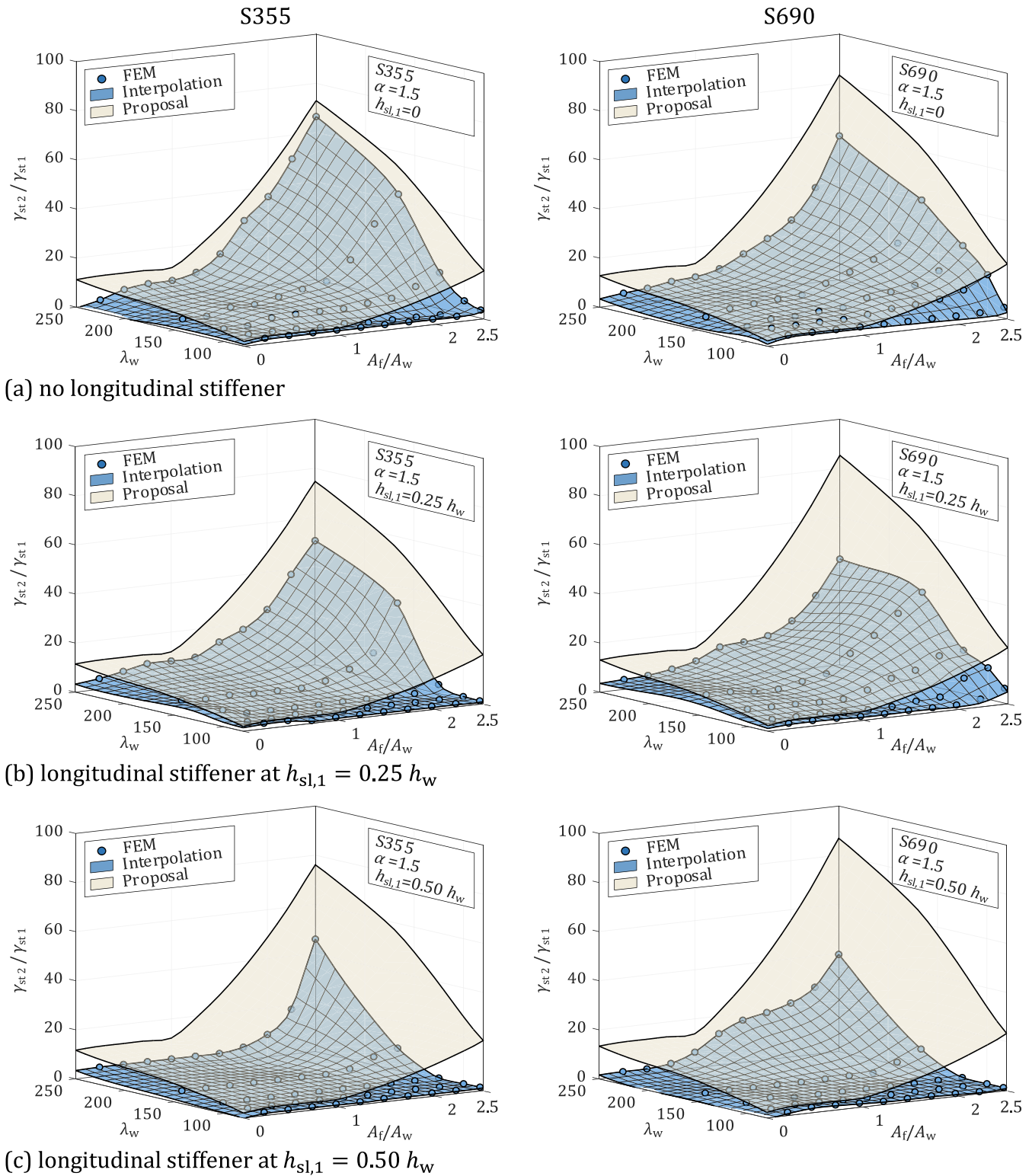


Fig. 15. Comparison of the required amplification factor $\Psi = \gamma_{st2}/\gamma_{st1}$ between numerical results and the proposed formulation for $\alpha = 1.5$: (a) no longitudinal stiffener, (b) $h_{sl,1} = 0.25 h_w$, (c) $h_{sl,1} = 0.50 h_w$ [78].

5.6.5. Effect of moment–shear interaction

Numerical analyses [1,78] of combined bending and shear ($M = 0.5\text{--}1.0 M_{pl,f}$ with shear) show only a slight reduction in shear capacity due to reduced flange contribution to post-buckling resistance. For these cases, stiffener demands are slightly lower than under pure shear,

supporting the proposition of using the same stiffeners design requirements under combined loadings M-V as a conservative approach.

$$\alpha = 2$$

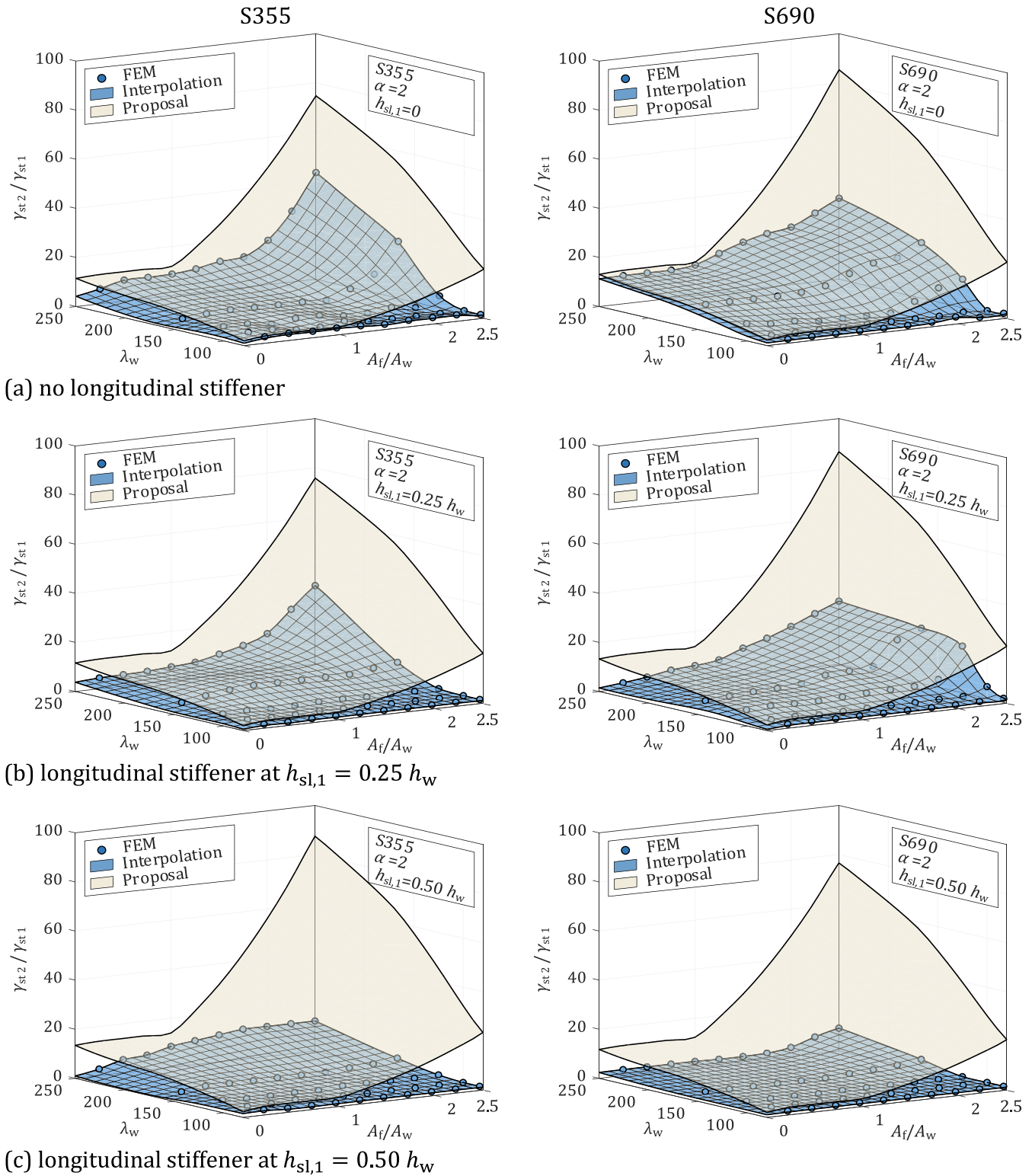


Fig. 16. Comparison of the required amplification factor $\Psi = \gamma_{st2}/\gamma_{st1}$ between numerical results and the proposed formulation for $\alpha=2$: (a) no longitudinal stiffener, (b) $h_{sl,1} = 0.25 h_w$, (c) $h_{sl,1} = 0.50 h_w$ [78].

6. Conclusions

This study investigated the role and design requirements of intermediate transverse stiffeners in achieving the ultimate shear buckling resistance of steel plate girders. An extensive literature review showed

that, in relevant codes and references, criteria related to bending rigidity have gained increased importance. This background as well as a comprehensive finite element (FE) parametric study have led to a new design approach which was proposed to ensure that the stiffener remains effective until the girder reaches its ultimate shear resistance.

Accordingly, the approach focuses on the web panel's post-buckling capacity rather than the applied design load. The method introduces an amplification factor to the current EN 1993–1–5 elastic bending requirement, thereby establishing a minimum bending rigidity criterion that accounts for the influence of web slenderness, flange contribution, and material strength. Although deliberately conservative, the method offers several advantages over existing procedures:

- it provides a direct and rational link between stiffener rigidity and the post-buckling web behaviour of steel plate girders;
- it eliminates the need for additional stiffener strength checks currently required by EN 1993–1–5:2024;
- it ensures consistent and reliable predictions across a wide range of geometries and material grades relevant to practical design.

By optimising the use of structural steel in transverse stiffeners, the proposed method enables more economical and sustainable design solutions, particularly for girders fabricated from high-strength steels (HSS).

CRedit authorship contribution statement

Sérgio Nascimento: Writing – original draft, Validation, Methodology, Investigation, Formal analysis. **Oliveira Pedro José:** Writing – review & editing, Supervision, Investigation, Formal analysis. **Ulrike Kuhlmann:** Writing – review & editing, Validation.

Declaration of Competing Interest

The authors declare the following financial interests/personal relationships which may be considered as potential competing interests: Sergio Nascimento reports financial support was provided by Foundation for Science and Technology. Sergio Nascimento reports financial support was provided by University of Lisbon Institute of Civil Engineering Research and Innovation for Sustainability. If there are other authors, they declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Data availability

Data will be made available on request.

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