Technical Note

Strength of I-Girders with Narrow Panels Subjected to Concentrated Loads

Rolando Chacón and Luis B. Fargier-Gabaldon

ABSTRACT

This technical note deals with the strength of web panels under concentrated loads, with emphasis on girders with closely spaced transverse stiffeners (commonly referred to as narrow panels). A review of experimental data and data from simulations suggest that girders with closely spaced panels exhibit substantially higher strength to concentrated loads than that calculated in accordance with the AISC *Specification for Structural Steel Buildings* (AISC, 2016). A simple equation to account for a fraction of the excess in strength is proposed.

Keywords: patch loading, closely spaced stiffeners, web crippling, flange resistance.

INTRODUCTION

Concentrated loads often govern the design of built-up steel I-girders, for example when launching plate girders or lifting heavy structures. To optimize web thickness and to prevent failure, transverse stiffeners at a constant spacing along the axis of the member are often provided. The behavior of steel built-up I-girders subjected to concentrated loads has been studied experimentally and analytically over the past six decades by Bergfelt (1979), Roberts and Rockey (1979), Roberts (1981), Roberts and Markovic (1983), Elgaaly (1983), Shimizu et al. (1989), Lagerqvist and Johansson (1995), Roberts and Newmark (1997), Tryland et al. (2001), Graciano (2005, 2015), Carden et al. (2007), Chacón et al. (2010a, 2010b, 2012), Salkar et al. (2015), Kövesdi (2018), and Rodilla and Kowalkowski (2021a, 2021b). Results from these investigations indicate that stocky webs exhibit local yielding, while slender or deep webs tend to buckle and fold (referred to as web crippling in the AISC *Specification*). In either case, the length on which the load is applied spreads out through the flange to a wider portion of the web that contributed to the load-carrying capacity. The *load length* is the length of the web affected by the concentrated load in the absence of vertical stiffeners.

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In this technical note, the behavior of beams with narrow panels failing in crippling and web yielding is investigated. Other failures modes, including sidesway buckling of the web, are beyond the scope of this investigation. In beams with narrow panels, the nominal *load length* calculated with the design equations exceeds the spacing between transverse stiffeners, a (Figure 1). In this technical note, experimental results and numerical simulations of girders with narrow panels subjected to concentrated loads are compared with the calculated strength in accordance with the AISC *Specification for Structural Steel Buildings* (AISC, 2016), hereafter referred to as the AISC *Specification*. It should be noted that the AISC *Specification* does not differentiate between girders with narrow panels and girders with wide panels (in which the nominal *load length* is less than a).

AISC *SPECIFICATION*

The strength of the built-up I-girders under concentrated loads, $R_{n\,AISC}$, is the smallest calculated for web yielding (Equations 1 or 2) and web crippling (Equations 3, 4, or 5). (Note that the AISC *Specification* equation number is also given for reference.) For web yielding, when the concentrated load is applied at a distance from the member end that is greater than the depth of the member, d,

$$R_{nAISC} = f_{yw} t_w (5k + l_b) \quad \text{(AISC \textit{Spec}. Eq. J10-2) (1)}$$

When the concentrated load to be resisted is applied at a distance from the member end that is less than or equal to d,

$$R_{n\,AISC} = f_{yw} t_w (2.5k + l_b) \quad \text{(AISC \textit{Spec}. Eq. J10-3) (2)}$$

For web crippling, when the concentrated load to be resisted is applied at a distance from the member end that is greater than or equal to $d/2$,

$$R_{n\,AISC} = 0.80t_w^2\left[1+3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{Ef_{yw}t_f}{t_w}}Q_f$$

(AISC *Spec*. Eq. J10-4) (3)

When the concentrated compressive load to be resisted is applied at a distance from the member end that is less than $d/2$, two additional equations 4 and 5 are given for web crippling,

For $l_b/d \le 0.2$

$$R_{n\,AISC} = 0.40t_w^2\left[1+3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{Ef_{yw}t_f}{t_w}}Q_f$$

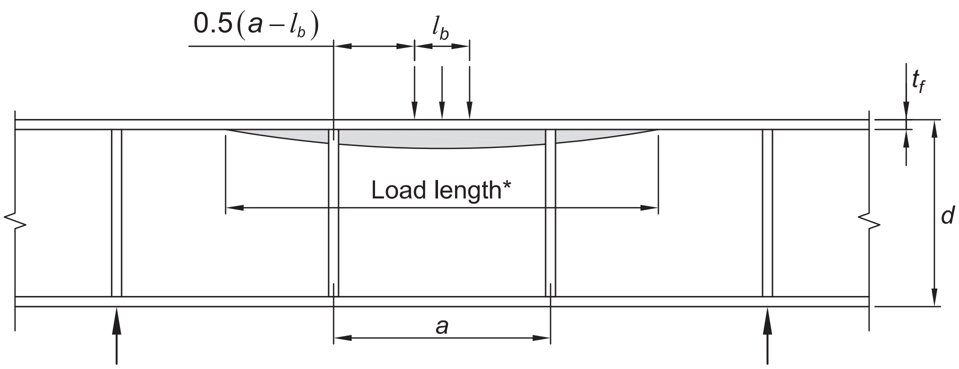
(AISC *Spec*. J10-5a) (4)

For $l_b/d \ge 0.2$

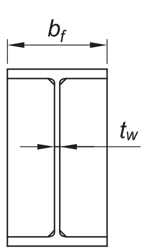
$$R_{n\,AISC} = 0.40t_w^2\left[1+\left(\frac{4l_b}{d}-0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{Ef_{yw}t_f}{t_w}}Q_f$$

(AISC *Spec*. J10-5b) (5)

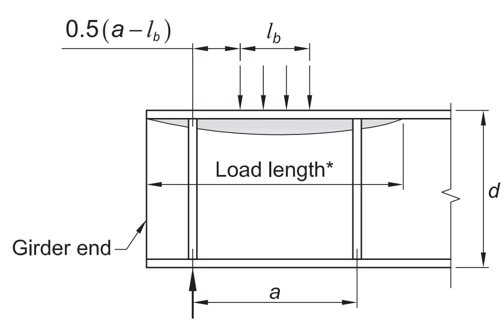
When the concentrated compressive force is a live load (e.g., during launching of a steel plate girder, or a steel shape that serves as the rail of an industrial crane), the location of the force changes over time, and it is not possible to have a stiffener at every potential force location. In this case, the design strength calculated with the force acting between transverse stiffeners will exceed the demand. Equations 1 and 2 were proposed by Roberts (1981). In these equations, the term in parentheses may be interpreted as the nominal *load length*. For a concentrated load applied at a distance less than d from the end of the girder, a nominal *load length* equal to $2.5k + l_b$ is inferred from Equation 2. In Equations 3 to 5, however, the *load length* is not explicit. Roberts (1981) already found that Equations 4 and 5 tend to underestimate the strength of girders with narrow panels. He suggested that the calculated strength from these equations can be taken as the maximum concentrated load recommended under service conditions. Other design equations to estimate the strength at ultimate load for these cases have been proposed, including the Eurocode, EN1993-1-5 (CEN, 2006), and the work of Chacón et al. (2013a, 2013b, 2017). It should be noted that the design equations proposed by Roberts (1981) were derived from a set of experimental tests on simply supported single panels.



(a) Load away from the end of the girder



(b) Girder cross section



(c) Load close to the end of the girder

*Fig. 1. Built-up I-girder under concentrated loads (*nominal load length shown in absence of vertical stiffeners).*

TEST RESULTS AND NUMERICAL SIMULATIONS

Figure 2 shows the applied vertical load vs. displacement response of a typical built-up steel I-girder with closely spaced stiffeners tested by Chacón et al. (2013a, 2013b). The *load length* calculated for this girder, as defined, is not affected by the presence of vertical stiffeners. For girders with relatively closely spaced vertical stiffeners, the calculated *load length* may exceed the spacing between transverse stiffeners (see Figure 1 and Table A.1 in Appendix A). When transverse stiffeners are provided, however, the portion of the web resisting the concentrated load is bounded by the vertical stiffeners, and thus, the actual *load length* cannot exceed or extend beyond a. Multipanel built-up steel I-girders are more common in practice than single-panel built-up I-girders. The majority of tests from which design equations were derived have single panels and, thus, do not provide relevant information for the case of continuous deep girders with closely spaced transverse stiffeners. The last column of Table A.1 shows the ratio of the measured-to-calculated strength for two girders with three panels and closely spaced stiffeners (the strength obtained with the AISC *Specification*). The measured strength of both girders exceeds the calculated strength by a factor of 1.6. In this technical note, it is postulated that a fraction of the safety margin, or excess strength that is not captured by the current design equations, can be attributed to the reserve of strength in the loaded flange (that is engaged due to the presence of transverse stiffeners). Figure 2 shows a response curve of a tested specimen with three panels. A linear response up to approximately 250 kN (56 kips) is observed. This load is referred to as F_1 in Chacón et al. (2013a). At this load, the web starts to cripple, while the top flange remains elastic. A web folding mechanism was gradually observed from this point onward. For further load increases, the flanges and the stiffeners provided an additional load path allowing for some post-crippling capacity. A fraction of this post-crippling capacity, $\alpha\Delta F$ is provided by the reserve strength available in the loaded flange when the external load reaches F_1. Results from this investigation suggest that strength calculated based on the AISC *Specification*, $R_{n\,AISC}$, is a good approximation of the magnitude of F_1, as shown in Figure 2. The additional strength beyond $R_{n\,AISC}$ can be quantified and added, with certain assumptions. A database with test results from 62 steel built-up I-girders, some of which were collected by Lagerqvist (1994) and other specimens reported recently, was used to investigate the strength of beams with narrow panels. The measured strength, R_u, was normalized with respect to the strength calculated with Equations 2, 4, or 5, as shown in Figure 3. In addition, results from numerical simulations reported by Chacón et. al (2013a, 2013b, 2017) are included, and as it was done with the experimental data, normalized with respect to Equations 2, 4, or 5.

Data shown in Figure 3 suggest that Equations 2, 4, and 5 tend to underestimate the capacity of steel built-up I-girders girders with narrow panels (mean strength ratio is equal to 2.3).

There are at least two plausible and noncontradictory explanations for a fraction of the strength underestimation.

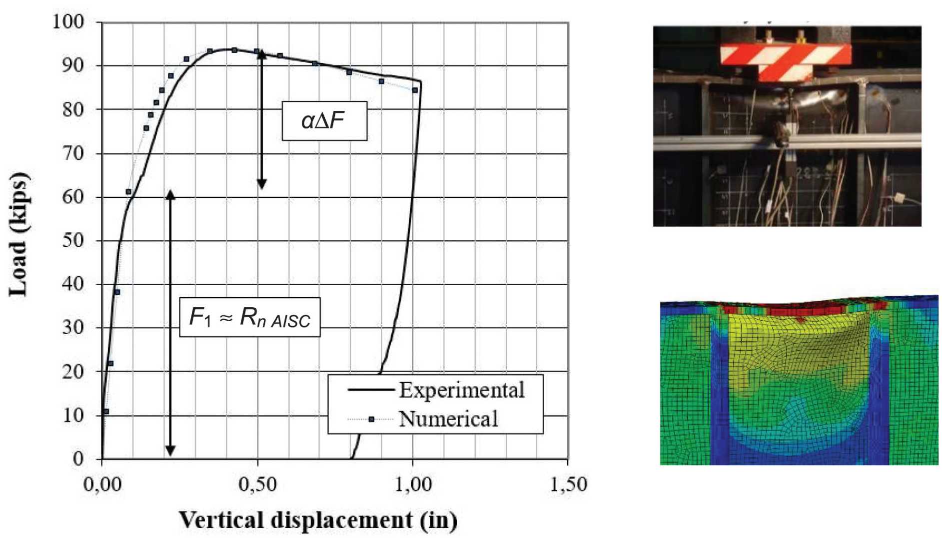


Fig. 2. Experimental and numerical results (beam VPL450, Chacón et al., 2013a).

First, the contribution of the flange to the web crippling strength as reported by Chacón et al. (2013a, 2013b, 2017) is not accounted for (in the derivation of Equation 4, only the mechanism associated with web crippling was adopted). Second, for the case of failures associated with web yielding, the AISC *Specification* requires a load length equal to $2.5k + l_b$ when the load is applied at a distance less than d from the end of the girder, regardless of the existence of stiffeners. In most cases, however, a bearing stiffener is present near the girder end [Figure 1(c)], leading to a significant strength increase.

CONTRIBUTION OF THE FLANGES TO THE STRENGTH, ΔF

The term ΔF can be obtained from a collapse mechanism involving four plastic hinges on the loaded flange (Chacón et al., 2013b). At an external concentrated load equal to F_1 (Figure 2) the web is assumed to cripple and its capacity exhausts. It is postulated that the strength increase, $\alpha\Delta F$, is resisted solely by the loaded flange (Figure 2). It should be noted that during the testing of the girder with narrow panels, a four-hinge collapse mechanism of the flange, as shown in Figure 4, is observed. The transverse stiffeners provide a load path to develop such hinges. Two plastic hinges develop in the vicinity of the flange-to-stiffener junction (outer hinges) while the other two plastic hinges develop under the applied load (inner hinges). The collapsed mechanism is idealized in Figure 4.

The strength contribution provided by the flanges, ΔF, can be estimated from a mechanism of the flanges as shown in Figure 4. The cross-sectional area of the flange is equal to $b_f t_f$ with length α between stiffeners, as shown in Figure 4. The reserve in flexural strength of the loaded flange at an applied external load corresponding to $R_{n\ AISC}$ is equal to Equation 6.

$$M_p = \frac{1}{4} b_f t_f^2 \left(f_{yf} - \sigma \right) \tag{6}$$

where σ can be taken as the peak flexural stress in the flange at the section under consideration, due to all loads acting on the plate girder (e.g., self-weight) when the magnitude of the external concentrated load is at the onset of

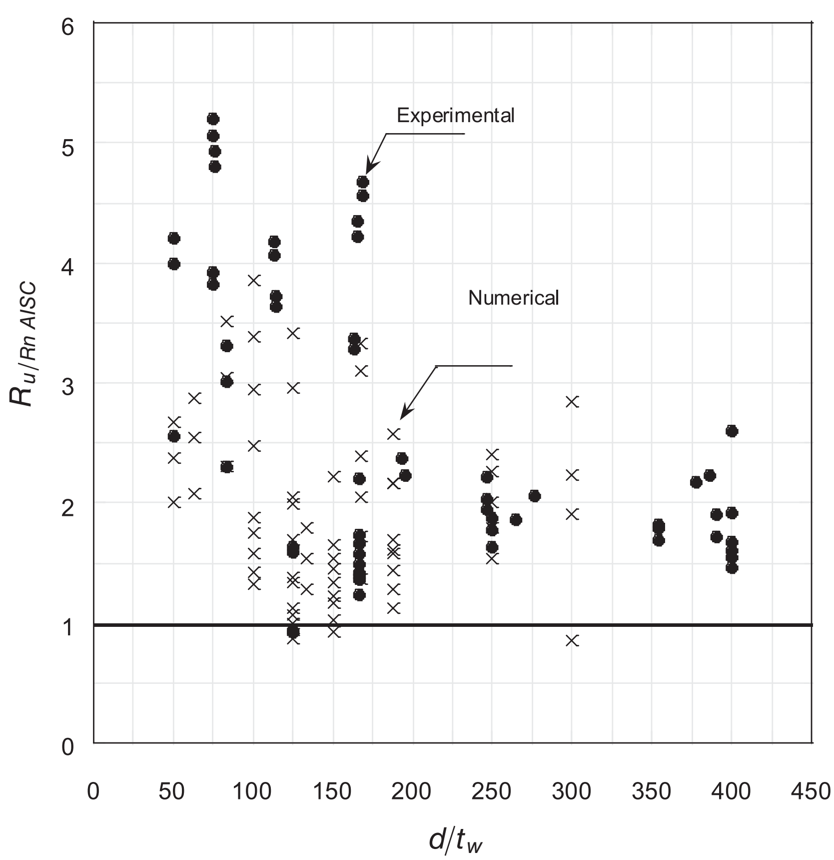


Fig. 3. Strength ratio vs. web slenderness ratio (using the AISC Specification*).*

web crippling, F_1, which can be approximated to $R_{n\,AISC}$, as shown in Figure 2. The peak stress can be obtained from a linear elastic beam model (Chacón et al., 2017). Based on the principle of virtual work and the idealized collapse mechanism shown in Figure 4, the additional strength can be calculated as shown in Equation 7.

$$\Delta F = \frac{4\left(M_{pi} + M_{po}\right)}{a - l_b} \tag{7}$$

where M_{pi} and M_{po} are the reserve in flexural strength of the inner and outer hinges. Incorporating Equation 6 into Equation 7, one obtains Equation 8.

$$\Delta F = \frac{b_f t_f^2}{a - l_b}\left(2f_{yf} - \sigma_i - \sigma_o\right) \tag{8}$$

where σ_o and σ_i are the peak flexural stresses in the flanges (both positive) due to external loads for outer and inner hinges, respectively. This accounts for the contribution of flexural stresses as well. The proposed expression to estimate the strength of the girder with narrow panels failing in crippling is given by Equation 9.

$$R_{n\,proposed} = R_{n\,AISC} + \alpha\Delta F \tag{9}$$

The term $R_{n\,AISC}$ is obtained from Equations 4 or 5, ΔF is the contribution of the flanges calculated from Equation 8, and α is a correction factor determined empirically as shown in Equation 10.

$$\alpha = 1 - \frac{d/t_w}{1000} \tag{10}$$

Strength ratios calculated with $R_{n\,proposed}$ are shown in Figure 5. Given that all girders had bearing stiffeners, a load length equal to $5k + l_b$ was used for failures controlled by web panel yielding—that is, based on Equation 1 instead of Equation 2 (even when the load was applied at a distance less than d from the end of the girder). An average strength ratio equal to 1.8 is obtained, compared with 2.3 as obtained with Equations 2, 4, and 5. Once more experimental data become available of multipanel beams with narrow panels, a refined reliability analysis is recommended prior to introducing code changes (note that strength ratios range between 0.83 and 5.17, with only a few below 1.0). The proposed methodology, however, provides additional tools to tackle design or rehabilitation projects with a better understanding of the mechanics.

CONCLUSIONS

The equations given in the AISC *Specification for Structural Steel Buildings* (AISC, 2016) to calculate the strength of girders under concentrated loads are safe, but they tend to be quite conservative when applied to beams with narrow panels. A fraction of the excess in crippling strength not captured by the current AISC *Specification* equations can be estimated by adding the term shown in Equation 11.

$$\alpha\Delta F = \alpha\frac{b_f t_f^2}{a - l_b}\left(2f_{yf} - \sigma_i - \sigma_o\right) \tag{11}$$

When a bearing stiffener is present, a load length equal to $5k + l_b$ seems reasonable to estimate the web yielding strength under concentrated loads.

Data Availability Statement

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

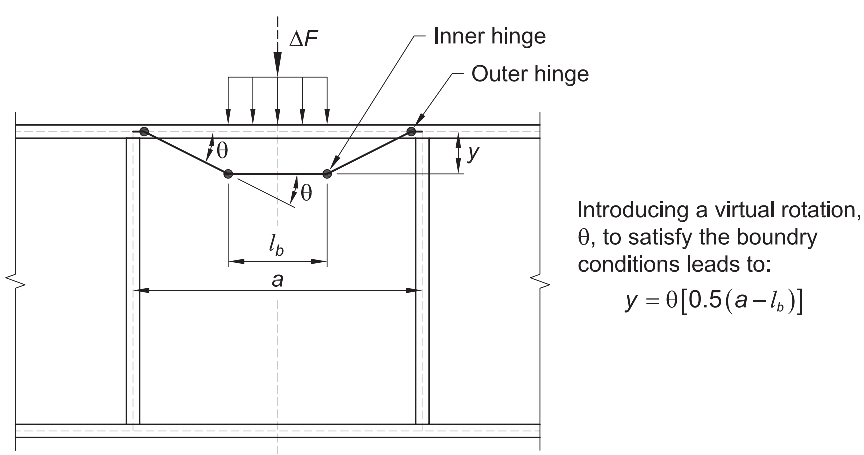


Fig. 4. Postulation of a four-hinge mechanism.

APPENDIX A EXPERIMENTAL RESULTS

Table A.1. Tests on Girders with Closely Spaced Stiffeners (Chacón, 2013a)													
	t_w	f_{yw}	f_{yf}	d_w	b	l_b	b_f	t_f	k	R_u	R_n Eq. 2	R_n Eq. 4	R_u/R_n
Girder	**in.**	**psi**	**psi**	**in.**	**in.**	**in.**	**in.**	**in.**	**in.**	**kips**	**kips**	**kips**	
1VPL450	0.157	51.49	65.85	19.69	17.72	7.87	7.87	0.79	0.79	95.77	63.85	30.35	1.6
2VPL450	0.157	30.46	65.12	19.68	17.72	7.87	7.87	0.79	0.79	75.76	37.77	23.38	1.6

NOTATION

E Young's modulus of steel

F_1 patch load at which the response curve changes slope

F_2 ultimate load capacity

R_{test} measured strength

Q_f Chord stress interaction parameter factor 1.0 for wide-flange sections and for HSS in tension

$R_{n\ AISC}$ strength calculated in accordance with the AISC *Specification*

b panel width

b_f flange width

d member depth

f_{yf} flange yield stress

f_{yw} web yield stress

k distance from outer face of flange to the web toe of fillet

l_b bearing length of the patch load according to the AISC *Specification*

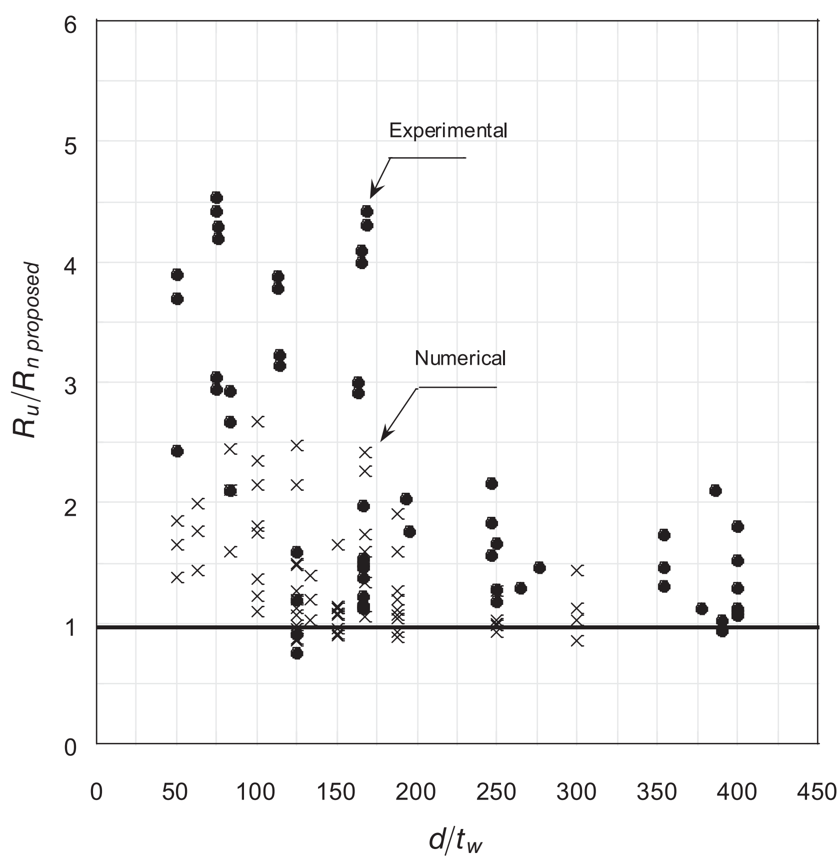


Fig. 5. Strength ratio from tests vs. web slenderness ratio using proposed equations.

t_f flange thickness

t_w web thickness

ΔF post-crippling capacity

REFERENCES

AISC (2016), *Specification for Structural Steel Buildings*, ANSI/AISC 360-16, American Institute of Steel Construction, Chicago, Ill.

Bergfelt, A. (1979), "Patch Loading on a Slender Web-Influence of Horizontal and Vertical Web Stiffeners on the Load Carrying Capacity," Report S79:1, Chalmers University of Technology, Gothenburg, Sweden.

Carden, L., Pekcan, G., and Itani, A. (2007), "Web Yielding, Crippling and Lateral Buckling under Post Loading," *Journal of Structural Engineering,* ASCE, Vol. 133, No. 5, pp. 665–673. https://doi.org/10.1061/(ASCE)0733-9445(2007)133:5(665)

CEN (2006), *Eurocode 3: Design of Steel Structures*, Part 1-5: Plated Structural Elements, EN 1993-1-5, Comite Européen de Normalisation, Brussels, Belgium.

Chacón, R., Braun, B., Kuhlmann, U., and Mirambell, E. (2012), "Statistical Evaluation of the New Resistance Model for Steel Girders Subjected to Patch Loading," *Steel Construction,* Vol. 5, No. 2, pp. 10–15. https://doi.org/10.1002/stco.201200002

Chacón, R., Herrera, J., and Fargier-Gabaldón, L. (2017), "Improved Design of Transversally Stiffened Steel Plate Girders Subjected to Patch Loading," *Engineering Structures,* Vol. 150, pp. 774–785. https://doi.org/10.1016/j.engstruct.2017.07.086

Chacón, R., Mirambell, E., and Real, E. (2010a), "Hybrid Steel Plate Girders Subjected to Patch Loading. Part 1: Numerical Study," *Journal of Constructional Steel Research*, Vol. 66, No. 5, pp. 695–798. https://doi.org/10.1016/j.jcsr.2009.12.005

Chacón, R., Mirambell, E., and Real, E. (2010b), "Hybrid Steel Plate Girders Subjected to Patch Loading, Part 2: Design Proposal," *Journal of Constructional Steel Research,* Vol. 66, No. 5, pp. 709–715. https://doi.org/10.1016/j.jcsr.2009.12.004

Chacón, R., Mirambell, E., and Real, E. (2013a), "Transversally Stiffened Plate Girders Subjected to Patch Loading, Part 1: Preliminary Study," *Journal of Constructional Steel Research,* Vol. 80, pp. 483–491. https://doi.org/10.1016/j.jcsr.2012.06.008

Chacón, R., Mirambell, E., and Real, E. (2013b), "Transversally Stiffened Plate Girders Subjected to Patch Loading, Part 2: Additional Numerical Study and Design Proposal," *Journal of Constructional Steel Research*, Vol. 80, pp. 492–504. https://doi.org/10.1016/j.jcsr.2012.06.001

Elgaaly, M. (1983), "Web Design under Compressive Edge Loads," *Engineering Journal*, AISC, Vol. 20, No. 4, pp. 153–171.

Graciano C. (2005), "Strength of Longitudinally Stiffened Webs Subjected to Concentrated Loading," *Journal of Structural Engineering*, ASCE, Vol. 131, No. 2, pp. 268–278. https://doi.org/10.1061/(ASCE)0733-9445(2005)131:2(268)

Graciano, C. (2015), "Patch Loading Resistance of Longitudinally Stiffened Girders—A Systematic Review," *Thin Walled Structures*, Vol. 95, No. 1–6. https://doi.org/10.1016/j.tws.2015.06.007

Kövesdi, B. (2018), "Patch Loading Resistance of Slender Plate Girders with Longitudinal Stiffeners," *Journal of Constructional Steel Research*, Vol. 140, pp. 237–246. https://doi.org/10.1016/j.jcsr.2017.10.031

Lagerqvist, O. (1994), "Patch Loading: Resistance of Steel Girders Subjected to Concentrated Forces," PhD Dissertation, Lulea University of Technology, Sweden. ISSN 0348-8373

Lagerqvist, O. and Johansson, B. (1995), "Resistance of Plate Edges to Concentrated Forces," *Journal of Constructional Steel Research*, Vol. 32, No. 1, pp. 69–105. https://doi.org/10.1016/0143-974X(94)00010-F

Roberts, T. (1981), "Slender Plate Girders Subjected to Edge Loading," *Proceedings of the Institution of Civil Engineers*, Vol. 71, No. 3, pp. 805–819.

Roberts, T. and Markovic, N. (1983), "Stocky Plate Girders Subjected to Edge Loading," *Proceedings of the Institution of Civil Engineers, Part 2*, Vol. 75, No. 3, pp. 539–550.

Roberts, T. and Newark, A. (1997), "Strength of Webs Subjected to Compressive Edge Loading," *Journal of Structural Engineering*, ASCE, Vol. 123, No. 2, pp. 176–186. https://doi.org/10.1061/(ASCE)0733-9445(1997)123:2(176)

Roberts, T. and Rockey, K. (1979), "A Mechanism Solution for Predicting the Collapse Loads of Slender Plate Girders when Subjected to In-Plane Patch Loading," *Proceedings of the Institution of Civil Engineers, Part 2*, Vol. 67, No. 1, pp. 155–175.

Rodilla, J.A. and Kowalkowski, K. (2021a), "Determination of Capacities of Eccentric Stiffeners Part 1: Experimental Studies," *Engineering Journal*, AISC, Vol. 58, No. 2.

Rodilla, J.A., and Kowalkowski, K, (2021b), "Determination of Capacities of Eccentric Stiffeners Part 2: Analytical Studies," *Engineering Journal*, AISC, Vol. 58, No. 2.

Salkar, R., Salkar, A., and Davids, W. (2015), "Crippling of Webs with Partial Depth Stiffeners under Patch Loading," *Engineering Journal*, AISC, Vol. 52, No. 4.

Shimizu, S., Yabana, H., and Yoshida, S. (1989), "A New Collapse Model for Patch Loaded Web Plates," *Journal of Constructional Steel Research*, Vol. 13, pp. 61–73. https://doi.org/10.1016/0143-974X(89)90005-9

Tryland, T., Hopperstad, O., and Langseth, M. (2001), "Finite-Element Modeling of Beams under Concentrated Loading," *Journal of Structural Engineering*, ASCE, Vol. 127, No. 2, pp. 176–185. https://doi.org/10.1061/(ASCE)0733-9445(2001)127:2(176)