Thin-Walled Structures 47 (2009) 1498–1506

Contents lists available at ScienceDirect

Thin-Walled Structures

journal homepage: www.elsevier.com/locate/tws

Shear failure characteristics of steel plate girders

M.M. Alinia *, Maryam Shakiba, H.R. Habashi

Department of Civil Engineering, Amirkabir University of Technology, 424 Hafez Avenue, Tehran 15875-4413, Iran

ARTICLE INFO

Article history:
Received 28 November 2008
Received in revised form
30 April 2009
Accepted 14 June 2009
Available online 16 July 2009

Keywords:
Steel plate girders
Shear failure mechanism
Plastic hinge
Ultimate capacity

ABSTRACT

A number of full-scale plate girders are modeled and analyzed to determine their shear failure mechanism characteristics. An objective of this numerical nonlinear large deflection elastoplastic finite element study is to clarify how, when, and why plastic hinges that emerge in experimental tests actually form. It is observed that shear-induced plastic hinges only develop in the end panels. These hinges are caused by the shear deformations near supports and not due to bending stresses arising from tension fields. Also, a comparison between the ultimate capacity of various plate girders and different codes and theories is presented. Finally, it is shown that simple shear panels, in the form of detached plates, do not accurately represent the failure mechanism of web plates.

1. Introduction

Plate girders are designed to support heavy loads over long spans such as building floors, bridges and cranes; where standard rolled sections or compound girders are not answerable. Modern plate girders are, in general, fabricated by welding together two flanges, a web and a series of transverse stiffeners. Flanges resist applied moment, while web plates maintain the relative distance between flanges and resist shear. In most practical ranges, the induced shearing force is relatively lower than the normal flange forces. Therefore, to obtain a high strength to weight ratio, it is common to choose deep girders. This entails a deep web whose weight is minimized by reducing its thickness. Various forms of instabilities, such as shear buckling of web plates, lateral-torsional buckling of girders, compression buckling of webs, flange-induced buckling of webs, and local buckling and crippling of webs are considered in design procedures.

Due to the slenderness of web plates, they buckle at early stages of loading. Therefore, one important design aspect of plate girders is the shear buckling and failure of web elements. Webs are often reinforced with transverse and in some cases with longitudinal stiffeners [1–3] to increase their buckling strength. A proper web design involves finding a combination of optimum plate thickness and stiffener spacing that renders economy in terms of material and fabrication cost. The design process of plate girder webs are commonly carried out within two categories: (i) allowable stress design based on elastic buckling as a limiting condition; and (ii) strength design based on ultimate strength, including postbuckling as a limit state. Till 1960s, the elastic buckling concept was basically used in the design of plate girders and the postbuckling strength was only indirectly accounted for by means of lowering safety factors.

Wilson [4] first discovered the postbuckling behavior in 1886, and Wagner [5] developed the theory of uniform diagonal tension for aircraft structures with very thin panels and rigid flanges in 1931. In late 1950s, Basler and Thurliman [6] took a different approach and carried out extensive studies on the postbuckling behavior of plate girder web panels. They assumed that tension field develops only in parts of the web and that flanges are too flexible to support normal stresses induced by the inclined tension field. In other words, yield zones form away from flanges and merely transverse stiffeners act as anchors. Their alleged assumption was in contrast to the Wagner's [5]; but later other researchers like Fujji [7] showed that the Basler's formula was given for complete tension field instead of limited band. Further research works by Basler [8–10] paved the way for the American Institute of Steel Construction (AISC) [11] and the American Association of Steel Highway and Transportation Officials (AASHTO) [12] to adopt the postbuckling strength of plates into their specifications. By moving towards applying the limit state design concept in the design of steel structures, SSRC [13] introduced a number of modified failure concepts to achieve a better correlation between theories and test results.

On the other side, the Cardiff model developed by Porter et al. [14] was adopted into the British Standards [15]. They also assumed that inclined tension fields only develop in a limited portion, but that flanges do contribute to the postbuckling strength by absorbing normal stresses from tension fields; and that as a result, girders collapse when plastic hinges form in their flanges.

* Corresponding author. Tel.: +98 21 6641 8008; fax: +98 21 6454 3268.
E-mail addresses: m.alinia@aut.ac.ir, alinia_aut@yahoo.com (M.M. Alinia).

0263-8231/$ - see front matter © 2009 Elsevier Ltd. All rights reserved.
doi:10.1016/j.tws.2009.06.002

Notations

A	area of end-post/stiffener
a	panel width
b_f	flange width
c	position of flange plastic hinge
E	elastic modulus
e	width of end stiffener
f_y	material yield stress
h_w	web height
k	shear buckling coefficient
L	girder span
t_f	flange thickness
t_s	thickness of intermediate stiffeners
t_{se}	thickness of end stiffeners
t_w	web plate thickness
Δ	in-plane deflection of girder
δ	out-of-plane displacement of web panels
υ	Poisson's ratio
σ_x, σ_y	normal stresses
τ_{cr}	critical shear stress
τ_{xy}	shear stress

Basler [10], Porter et al. [14], Takeuchi [16] and Herzog [17] assumed that the diagonal tension field develops in a limited portion of the web. In contrast, Fujji [18], Komatsu [19], Chern and Ostapenko [20] and Sharp and Clark [21] assumed that diagonal tension spreads all over the panel, but with different intensity. The Steinhardt and Schroter's [22] assumption, lies half way between the two previous assumptions. Hoglund [23–25] developed a theory for transversely stiffened and unstiffened plate girders. He used the system of diagonal tension and compression bars to model web plates. His theory later became the basis for Eurocode 3 [26].

Although these classical failure theories assumed different yield zone patterns, the fundamental assumption that "compressive stresses that develop in the direction perpendicular to the tension diagonal do not increase any further once elastic buckling has taken place" was common in all of them. The application of this fundamental assumption to the whole web panel led to the well-known theory that the tension field action in plate girders with transverse stiffeners needs to be anchored by flanges and stiffeners in order for the webs to develop their full postbuckling strength.

Takeuchi [16] was the first to make an allowance for the effect of flange stiffness on the yield zone of web plates. Among the previous researchers, Fujji [18], Komatsu [19], Porter et al. [14] and Hoglund [25] assumed that the normal stresses induced by the tension fields are anchored by the top and bottom flanges and/or the combination of transverse stiffeners and adjacent panels. These normal stresses, thus, produce a beam mechanism in flanges and the ultimate capacity of plate girder is accompanied by the formation of plastic hinges in flanges. Their proposed theories, it seems, were invented to justify the formation of plastic hinges that had materialized in extensive experiments.

In other series of analytical and experimental works, Lee and Yoo [27–31] showed that flanges and transverse stiffeners do not necessarily behave as anchors. Their studies confirmed that intermediate transverse stiffeners are not subjected to compressive forces and that flanges are not subjected to lateral loadings. They further introduced an approach that was referred to as the shear cell analogy to resolve the discrepancy between their previous understandings and new findings. However, on reexamining, they noticed that the shear cell analogy does in fact contain a serious flaw. An important stress component was inadvertently omitted during the transformation process from a two-dimensional stress to an assembly of one-dimensional bar element.

Ever since Wagner [5] proposed the pre-mentioned fundamental assumption, no one has examined it critically. Although Marsh et al. [32] found that the diagonal compression at the tension corners of the web increased after buckling, they still concluded that flanges contribute to the shear capacity of panels due to their bending strength, which permits the development of some diagonal tension.

The assumed failure mechanisms in Basler, Cardiff and other mentioned models probably do not accurately represent the ultimate shear behavior of web panels, since they are significantly affected by bending stresses when panels undergo large postbuckling deformations and the pattern of yield zones at one face is different from the other [33]. In short, although the classical theories underestimate the buckling strength due to the negligence of torsional rigidity of boundary members, they give higher values for the ultimate shear strength, because of their overestimation in the postbuckling strength [31,34,35].

The nonlinear shear stress and normal stress interaction that takes place from the onset of elastic shear buckling to the ultimate strength state is so complex that any attempt to address this phenomenon using classical closed form solutions appear to be unsuccessful. The fact that there have been many theories for explaining this occurrence is evidence to the complexity of tension field action. The objective of this nonlinear large deflection elastoplastic finite element (FE) study is to clarify the mechanism of shear failure in steel plate girders; and to answer why, how, when, and where plastic hinges form. Other aspects of shear plate behaviors, such as their deformability and rigidity and strength degradation due to fatigue-induced cracks have previously been reported by the present first author and his colleagues [34–38].

2. Method of study

2.1. General

A detached web panel simulation model, a simply supported web plate in shear, or even single-panel experimental tests cannot truly represent the behavior of plate girder web plates, since:

(a) A web plate is bound to have some bending moments due to lateral loadings.
(b) The torsional rigidity of girder flanges must be accounted for in the rotational stiffness of panel boundary conditions. The true behavior of flange–web junction is neither simply supported nor clamped.
(c) In reality, flanges are allowed to move towards or apart from each other, and their weak axis second moment of area becomes an important factor in this regard. A free or restrained in-plane movement of panel edges cannot represent the real behavior of web plates.
(d) The number of sub-plates created by intermediate transverse stiffeners and conditions of end-posts (end stiffeners) have considerable effects on the behavior of plate girders.

Therefore, in order to investigate the explicit shear failure mechanism of plate girders, complete girders with appropriate boundary restraints must be simulated. In this research, simple girder beam models subjected to point loads at their mid-spans are considered. The loading and end support conditions guarantee constant shear and a relatively small flexure throughout web panels. The FE modeling and the corresponding boundary conditions are illustrated in Fig. 1.

The mild steel material properties, with the elastic modulus $E = 210\,\text{GPa}$, normal yield stress $f_y = 345\,\text{MPa}$ and the Poisson's ratio $\upsilon = 0.3$ are used throughout the work. The material is assumed to be elastic perfectly plastic with no strain hardening.

2.2. FE modeling and validations

The four-noded-reduced-integrated element S4R of the ABAQUS software [39] is selected for all Eigenvalue and incremental nonlinear analysis. For convergence studies on mesh numbering, simple-detached panels were meshed into sufficient number of elements to allow the development of shear buckling modes and displacements. The elastic shear buckling stress values obtained via numerical analyses were compared to those obtained from the theoretical formula (1).

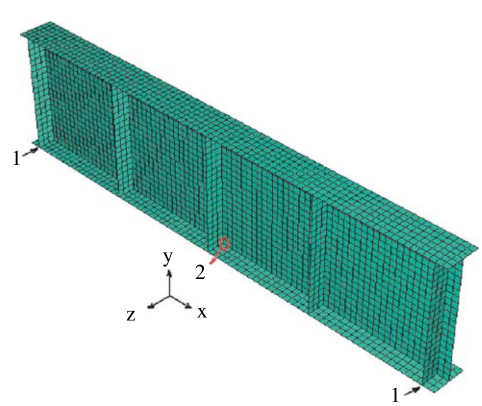
$$\tau_{cr} = \frac{k\pi^2 E}{12(1-\upsilon^2)}\left(\frac{t_w}{h_w}\right)^2 \tag{1}$$

Fig. 2 shows the variation of percentage errors obtained by comparing the finite element analysis results to the theoretical value for different numbers of incorporated mesh elements. Based on this figure, the model with a mesh refinement of 30×30 elements produced results which had good agreement with the theory and was, therefore, taken as the minimum requirement in the analyses.

To validate the overall modeling, boundary conditions and loading procedure, test results reported by Real et al. [40] were remodeled and analyzed. Fig. 3 presents the comparison between the mid-span in-plane deflections of a typical experimental data and the current FEA procedures.

2.3. Models

Several transversely stiffened plate girders having identical depth and panel width of 1000 mm in spans of 2, 4 and 6 m, such as the one given in Fig. 4, were considered and parametric studies regarding web thickness, flange dimensions and end-posts were carried out. Initial comparative analyses on the 2-, 4- and 6-panel



Restraint points	u_x	u_y	u_z	θ_x	θ_y	θ_z
1	○	✸	✸	✸	✸	○
2	✸	○	○	○	○	○

○= free
✸= restrained

Fig. 1. FE modeling of a typical plate girder.

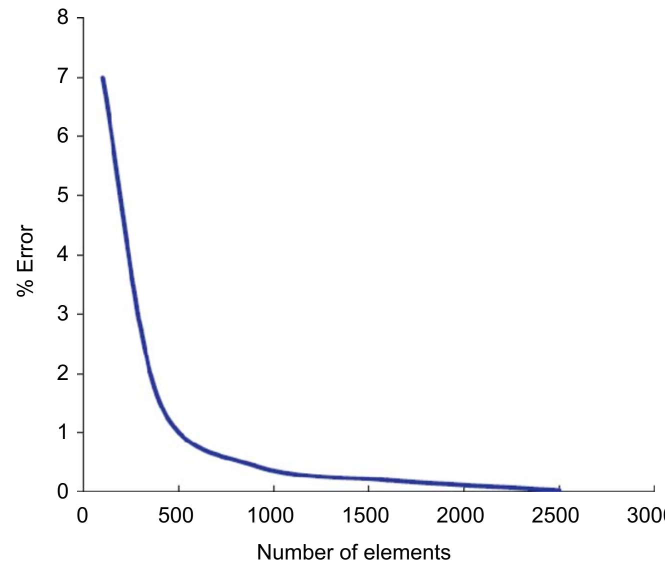


Fig. 2. Convergence study for the number of mesh elements.

girders showed good correlation for both deflections and stresses. Fig. 5 compares the out-of-plane displacement of the center of panels 1 or 3 of the four-panel girder in Fig. 4 to the corresponding 2- and 6-panel girders. The variation of the shear stress τ_{xy} at the central horizontal axis of the beams at the ultimate load is depicted in Fig. 6. The figure shows that the state of shear stresses is similar in the 2-, 4- and 6-panel girders.

An important element in the behavior and design of plate girders is the end-posts and therefore, a considerable part of this study is devoted to them. North American codes do not explicitly define end-posts; but in Eurocode 3 [26] three types of end stiffeners/posts, as shown in Fig. 7, are defined for steel plate girders. (a) Plate girders with no end-post, (b) plate girders with rigid end-posts and (c) plate girders with non-rigid end-posts.

3. Discussion of results

3.1. Shear vs. flexural plastic hinges

The FEM results of various plate girders indicate that the formation of plastic hinges in flanges may either have flexural or shear basis; as shown in Figs. 8 and 9. In the bending-initiated mechanism, plastic hinges form at mid-spans (Fig. 8); while in the shear-initiated mechanism, plastic hinges only occur in the end panels (Fig. 9).

If flange plates are not rigid enough to withstand bending-induced normal stresses, plastic hinges appear at the position of maximum-bending moment. In the typical model depicted in Fig. 8, plastic hinges are formed at the center-span adjacent to the central transverse stiffener. On the other hand, if the flange plates are rigid enough, shear-initiated mechanism becomes apparent by the formation of plastic hinges in the end panels next to the end stiffeners, as shown in Fig. 9. However, if both flange and web plates are strong enough to carry direct and shear stresses, other mechanisms such as local buckling and web crippling occur.

3.2. Failure modes

In order to characterize failure modes, some four-panel girders with non-rigid end-posts having various relative flange and web

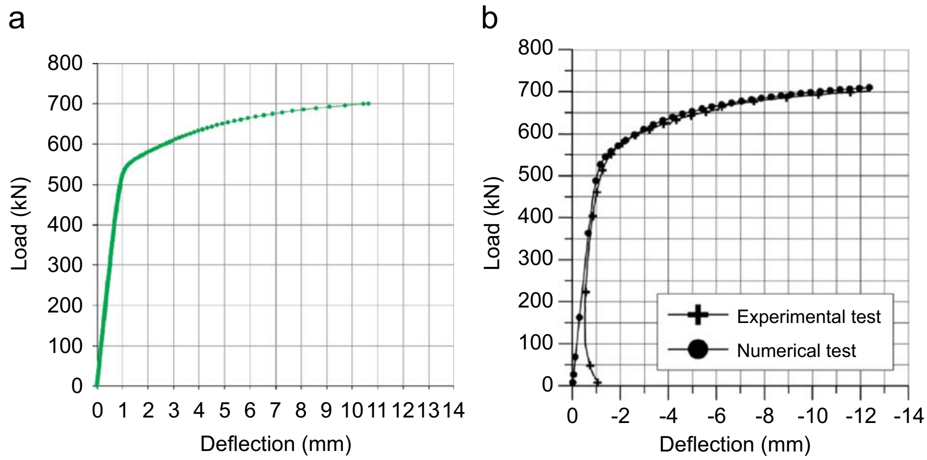


Fig. 3. FE model validation: (a) Numerical analysis and (b) test results [40].

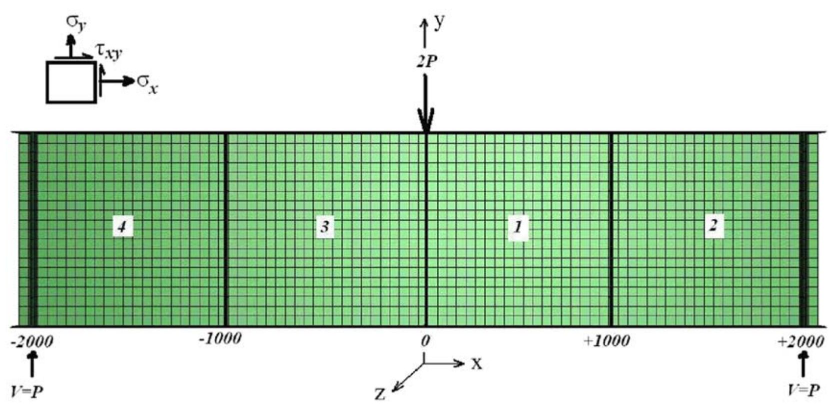


Fig. 4. Directions, panels and loading.

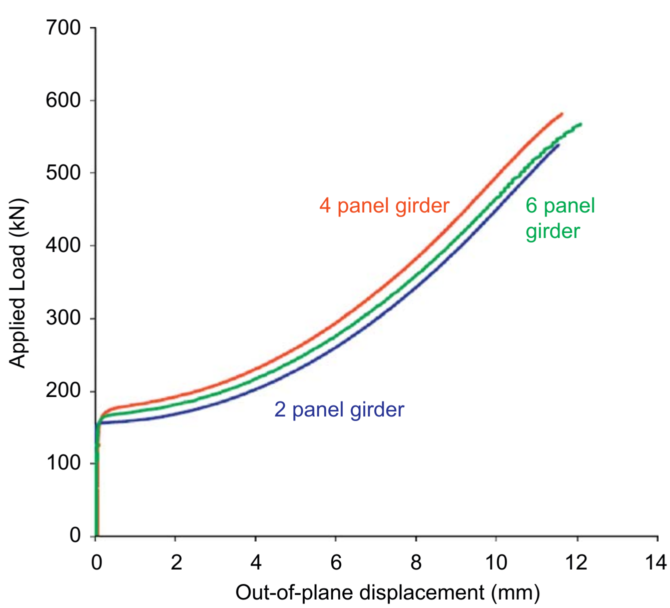


Fig. 5. Maximum out-of-plane deflection of the 2-, 4- and 6-panel girders.

dimensions were considered in a parametric study. The intermediate transverse stiffener thicknesses were presumed to be twice that of the web ($t_s = 2t_w$). The geometrical properties of selected plate girders and their predicted failure modes are given in Table 1. The results show that irrespective of the flange width to web height ratio (b_f/h_w), when the ratio of flange to web thickness is more than 3 ($t_f/t_w \geq 3$), the failure mode is always in shear; and if $t_f/t_w \leq 2$, the flexure mode governs. In the intermediate range ($2 < t_f/t_w < 3$), failure mode depends on the web slenderness parameter. Thicker webs ($h_w/t_w < 200$) result in flexural failure, while more slender webs fail in shear. AASHTO [12] has classified flange dimensions into three categories, as given in Table 2. Tables 1 and 2 deduce that girders made up with AASHTO's light flanges collapse in flexural mode, whereas moderate and heavy flange girders collapse in shear.

Further elaboration of results implies that the minimum-required flange thickness and width for girders not to collapse in flexure mode is in accordance with the simple beam theory by limiting the maximum stress to the material yield stress (f_y); as in Eq. (2). Therefore, the control of flexural mechanism is a simple procedure and easily preventive. In the next sections, the rather complex shear failure mechanism is discussed.

$$I^{\min}_{Girder} = \frac{M_{\max} h_w}{2f_y} \tag{2}$$

3.3. Shear-induced plastic hinges

In the shear failure mode, plastic hinges are formed in the flanges of end panels; after formation of web-inclined yield band.

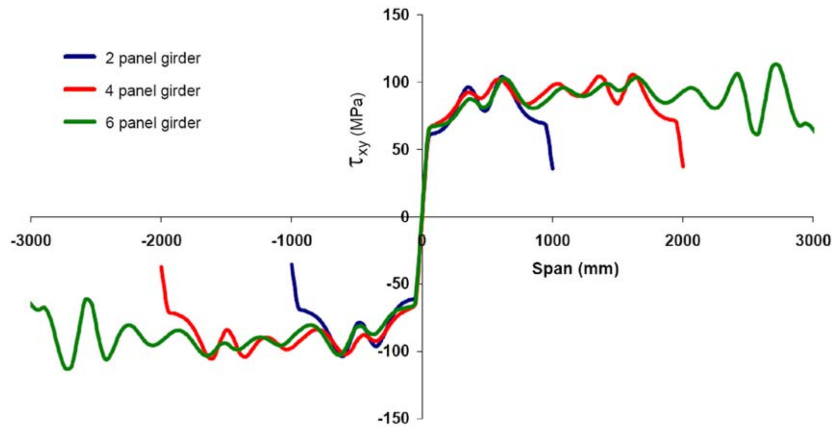


Fig. 6. Comparison of shear stresses in the 2-, 4- and 6-panel girders.

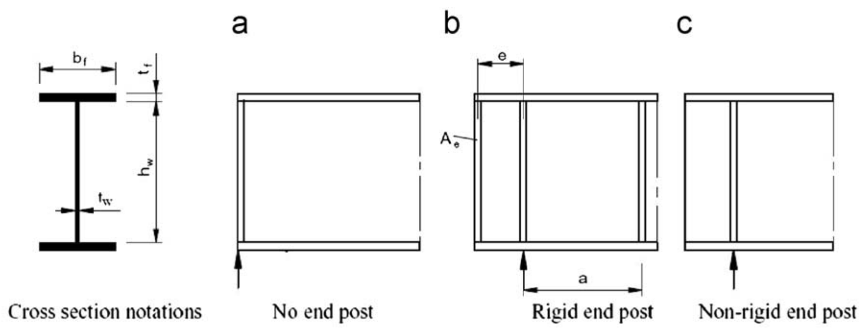


Fig. 7. Types of end stiffeners [26].

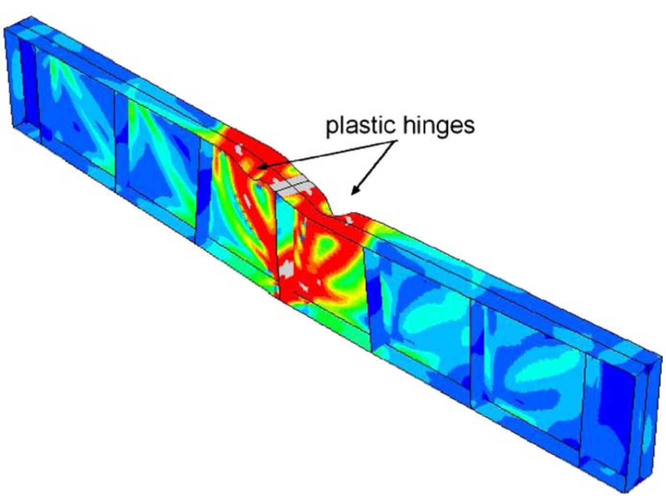


Fig. 8. Flexural failure mechanism.

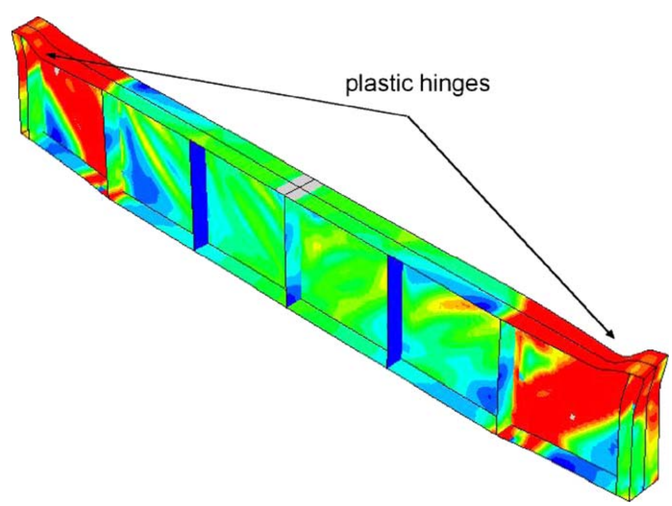


Fig. 9. Shear failure mechanism.

The in-plane vertical displacement of top and bottom flange plates in a typical girder near the end support is shown in Fig. 10. These displacement curves are extracted from the numerical analysis, and can be used as a guide to explain excessive shear deformations in plate girder end panels. It also demonstrates how end-posts act as clamped boundary conditions for top flange plates, whilst lower flange deforms in a manner similar to a simple beam.

The formation of plastic hinges is actually due to the differential shear deformation of end panels and they are not directly related to the stresses imposed by the inclined tension fields. That is why plastic hinges do not occur in mid-panels. It should be emphasized that the shear stress distributions in all panels are similar; that diagonal yield band occurs almost at the same time in all panels; and that the bending moment in the central panels are higher than the end ones. It should also be mentioned that most experiments on plate girders have been carried out on 2-panel girders, similar to the one provided by Shanmugam [41] in Fig. 11; and there are not many test reports on unrestrained multi-panel girders. However, experiments do

Table 1
Plate girder dimensions and failure modes.

b_f	t_w	t_s	t_f	t_f/t_w	Failure mode
300	3.33	6.66	3.33	1.00	Flexural
300	3.33	6.66	7.50	2.25	Shear
300	3.33	6.66	8.00	2.40	Shear
300	3.33	6.66	10.00	3.00	Shear
300	3.33	6.66	16.66	5.00	Shear
300	3.33	6.66	20.00	6.00	Shear
300	4.00	8.00	7.00	1.75	Flexural
300	4.00	8.00	9.00	2.25	Shear
300	4.00	8.00	12.00	3.00	Shear
300	6.66	13.33	6.66	1.00	Flexural
300	6.66	13.33	15.00	2.25	Flexural
300	6.66	13.33	20.00	3.00	Shear
350	6.66	13.33	6.66	1.00	Flexural
350	6.66	13.33	20	3.00	Shear
350	6.66	13.33	33.33	5.00	Shear
250	6.66	13.33	6.66	1.00	Flexural
250	6.66	13.33	20	3.00	Shear
250	6.66	13.33	33.33	5.00	Shear

$h_w = 1000$, $a = 1000$ and $L = 4000$ all dimensions are in mm.

Table 2
AASHTO classification for flange thickness.

Flange types	t_f/t_w	b_f/h_w
Light	1.0	0.25
Moderate	3.0	0.30
Heavy	5.0	0.35

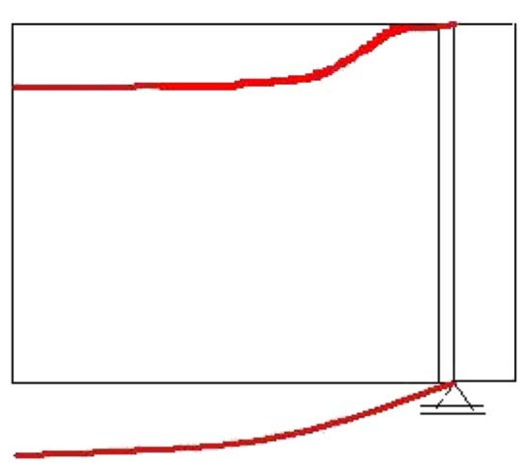


Fig. 10. Flange displacements at the occurrence of plastic hinges.

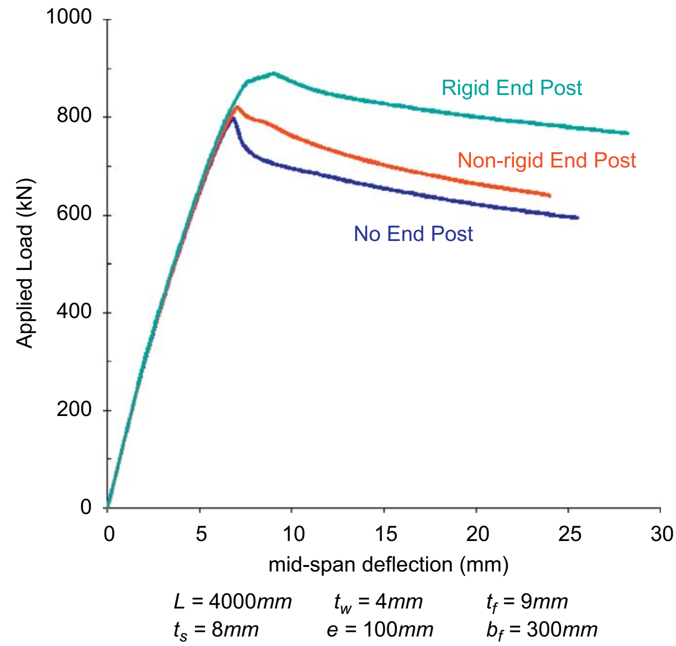


Fig. 12. Load vs. in-plane deflection curves for different end conditions.

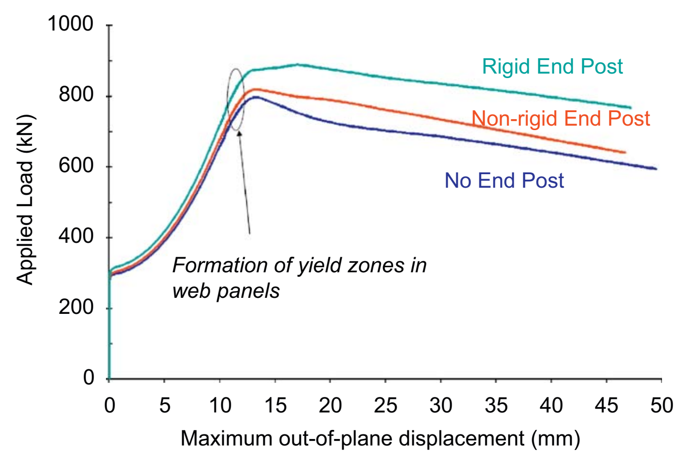


Fig. 13. Load vs. out-of-plane deflection curves for different end conditions.

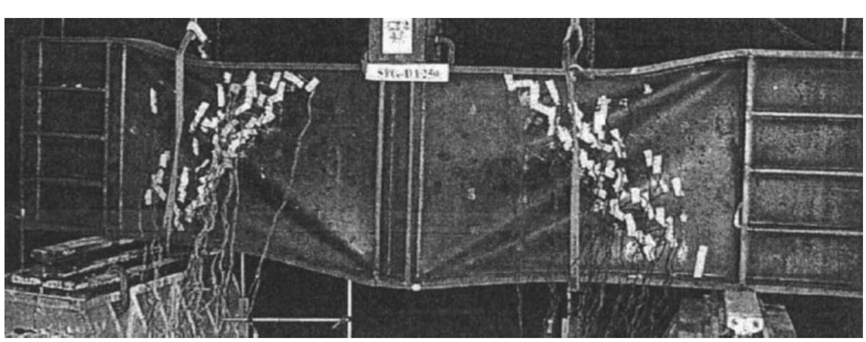


Fig. 11. View after failure of a 2-panel girder [41].

confirm that plastic hinges appear in flanges, but they do not demonstrate when, how and why they are formed.

3.4. Effect of end-posts/stiffeners

The addition of end-posts or reinforcing end stiffeners provides more fixity to flange plates and increases the ultimate resistance of plate girders. This is illustrated in the load-deflection curves of Fig. 12, where P–Δ curves of plate girders having different end-posts are compared. Evidently, end stiffeners have no effect on the initial stiffness of plate girders and only become effective after web panels yield in shear.

Fig. 13 depicts the web panel maximum out-of-plane displacement curves (P–δ) of girders. These out-of-plane displacements are measured at the center of panels. The apparent loss in the stiffness of girders is due to the formation of diagonal yield zones, as pointed out in the diagram. According to Fig. 13, the ultimate resistance of the plate girder with low end stiffness coincides with the load at which inclined yield zones form in panels. On the other hand, girders with rigid end-posts carry loads in excess of web yielding.

According to Eurocode 3 [26], a rigid end-post should act as a bearing stiffener resisting the reaction from bearing at the girder support, and as a short beam resisting the longitudinal membrane stresses in the plane of the web. A rigid end-post may comprise of two double-sided transverse stiffeners that form the flanges of a short beam of length h_w, see Fig. 7(b). The strip of web plate between the stiffeners forms the web of the short beam. Alternatively, an end-post may be in the form of a rolled section, connected to the end of the web plate. Also, each double-sided stiffener consisting of flat plates should have a cross-sectional area of at least

$$A = \frac{4h_w t_w^2}{e} \tag{3}$$

where A is the area of the two end stiffeners, and e ($>0.1h_w$) is the center-to-center distance between them (see Fig. 7). Therefore, for the plate girder with web height of $h_w = 1000$ mm and thickness of $t_w = 3.33$ mm, one would have $e \geq 100$ mm and $A \geq 444$ mm^2. Hence, the thickness of end stiffeners is derived as thin as $t_{se} \geq 1.5$ mm. To elaborate on the effect of the rigidity of end-posts, more models having different e and t_{se} were analyzed and their ultimate capacities are given in Fig. 14.

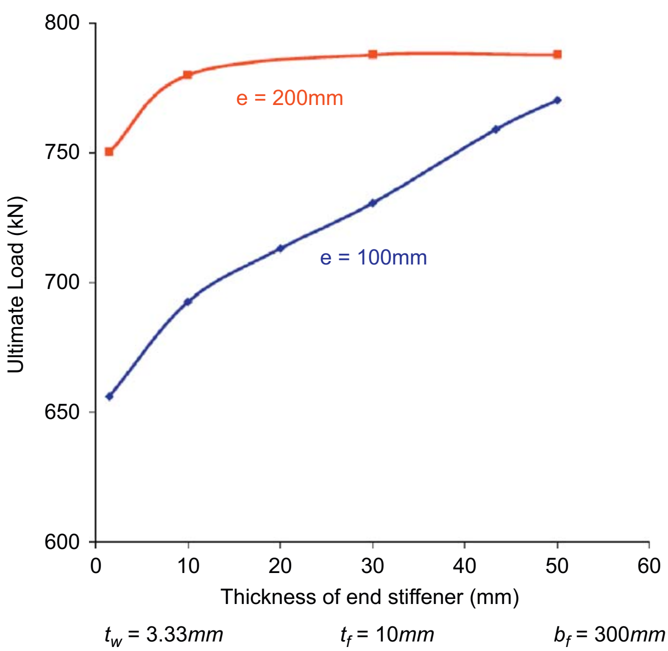


Fig. 14. Ultimate resistance of girders having different end-post dimensions.

Fig. 14 shows that for a constant e, the increase of the end stiffener thickness brings more fixity to the top flange and increases the ultimate capacity of girders. Furthermore, doubling the distance e, does considerably increase the ultimate capacity for small values of t_{se}; but this increase is not considerable in girders with thicker end stiffeners. Hence, the minimum-required thickness of the end stiffeners specified in Eurocode 3 seems to be too thin.

3.5. Occurrence of shear plastic hinges

The load vs. central out-of-plane displacement curves for a typical web panel in plate girders with various end-posts are depicted in Fig. 15. These curves are presented to demonstrate

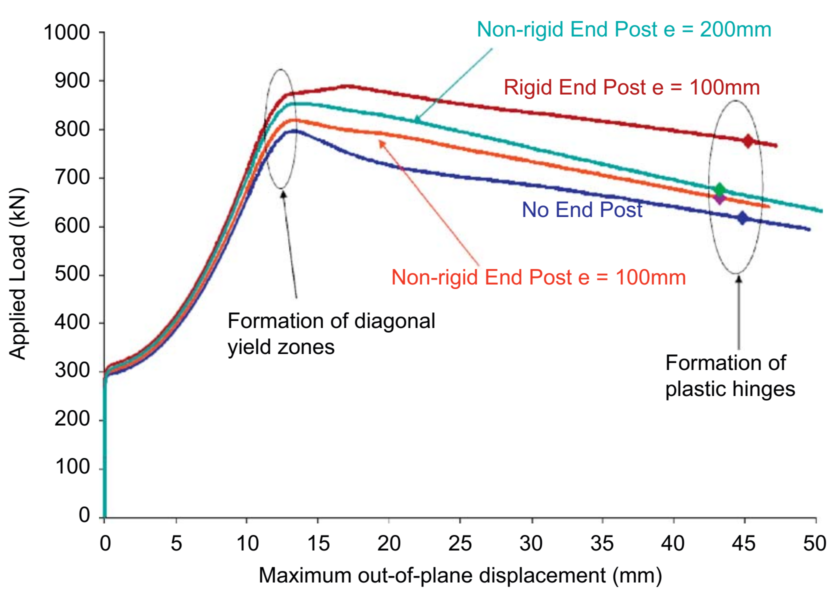


Fig. 15. Formation of plastic hinges.

how early the shear-oriented plastic hinges form in the plate girder web plates. They also clarify the relation between the plastic hinge occurrence with respect to the formation of inclined yield zone and the ultimate capacity of plate girders. The figure shows that plastic hinges occur much later than their ultimate capacities. In fact, plastic hinges do not occur before the ultimate load and at most they may take place concurrently [42]. The maximum load that a plate girder can resist is very close to the step at which diagonal yield zones form in panels. Therefore, once the web panel loses its shear capacity, the flanges and transverse stiffeners somehow act as a Vierendeel girder and ultimately flanges fail under shear deformation.

3.6. Location of plastic hinges

The position of plastic hinges in the end panel, regardless of the width and thickness of flange plates, is directly related to the rigidity of end stiffeners. According to the results, the position of plastic hinges in different girders varied from 0.20 to 0.35 of panel width measured from end stiffeners ($0.20a<c<0.35a$) and depends on the rigidity of end-posts as shown in Fig. 16. The location of plastic hinges does not directly depend on the width of the tension field. However, the hinge location and the width of tension fields are both related to the rigidity of end stiffeners and flange dimensions. Fig. 16 presents the stress distribution and the position of plastic hinges in two typical girders.

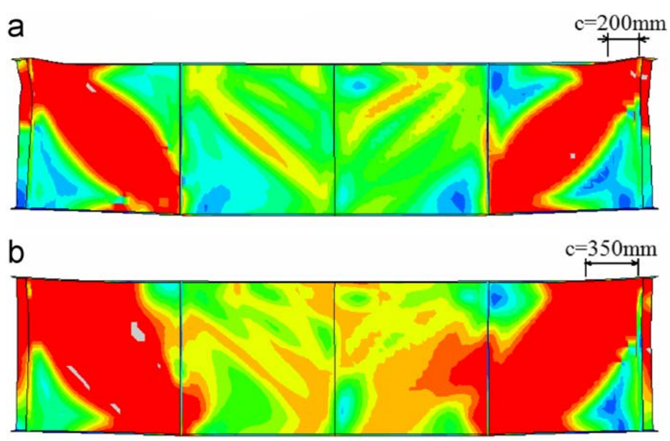


Fig. 16. The position of plastic hinges in two typical girders: (a) Girders with less rigid end stiffeners and (b) girders with more rigid end stiffeners.

3.7. Ultimate capacity of plate girders

Fig. 17 presents the comparison of results for the ultimate capacities of a number of plate girders extracted from the current FE analyses to those given by different codes and theories. It is observed that, in general, there is more divergence in the results of girders with thinner flanges. The Eurocode 3 gives the most conservative results, while the Porter's approach largely overestimates the girder capacities. The AISC results for the medium to thick flange plates best fit the FEM. The AASHTO and Basler results are very similar and they always overestimate the girders ultimate capacity. On the other hand, the Hoglund's theory for thicker flanges is always safe and reasonably close to FEM. Further elaborations on these results and the state of stresses in different elements of plate girders will be presented in the future paper.

4. Conclusions

Nonlinear large deflection finite element analyses of full-scale steel plate girders were performed to characterize their shear failure mechanism. The analyses concluded that:

- Detached plates simulation does not represent the true behavior of plate girder web panels.
- Shear-induced plastic hinges occur only in the flanges of end panels after the formation of partial-inclined yield zones in webs. They do not occur in mid-panels.
- The formation of plastic hinges is due to the shear deformation of girders, directly pertained to the stiffness of end-posts and flange dimensions. The location of plastic hinges is not directly related to the stresses imposed by the inclined tension fields.

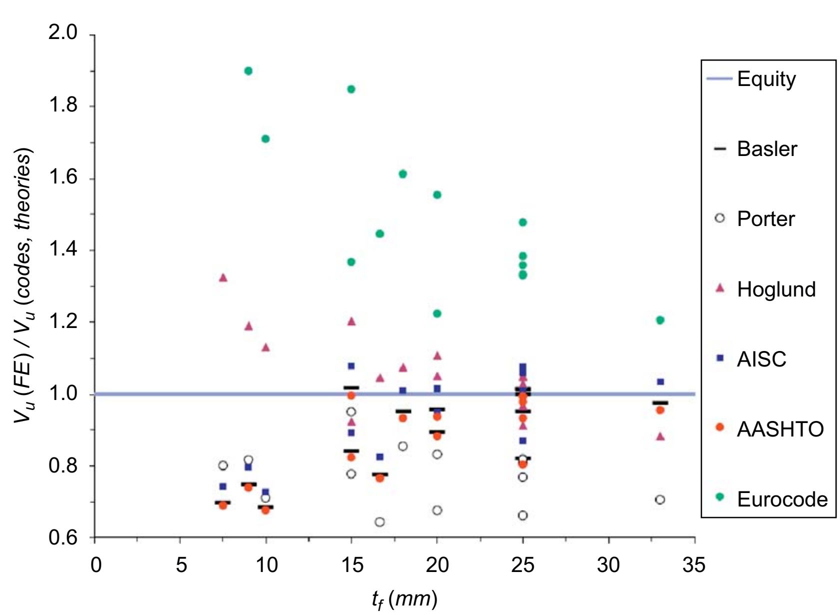


Fig. 17. Comparison of ultimate capacities of different codes and theories.

- When the flange thickness is more than three times the web thickness, the failure mode is always in shear and if this ratio is less than two, the flexure failure mode governs. In the intermediate ranges, the failure mode depends on the web slenderness ratio. Compact webs collapse in flexural mode, while slender webs fail in shear.
- The addition of end-posts provides more fixity to flange plates and increases the ultimate resistance of plate girders.
- Eurocode 3 gives the most conservative ultimate capacity for plate girders, while the Porter's model overestimates them. The AISC results for medium to stocky flanges produce closest results to the FEM. The AASHTO and Basler results are very similar and they always overestimate the capacity. The Hoglund's theory is always safe and reasonably close to FEM.

References

[1] Alinia MM. A study into optimization of stiffeners in plates subjected to shear loading. Thin-Walled Struct 2005;43(5):845–60.
[2] Alinia MM, Moosavi SH. A parametric study on the longitudinal stiffeners of web panels. Thin-Walled Struct 2008;46(11):1213–23.
[3] Alinia MM, Moosavi SH. Stability of longitudinally stiffened web plates under interactive shear and bending forces. Thin-Walled Struct 2009;47(1):53–60.
[4] Wilson JM. On specifications for strength of iron bridges. Trans Am Soc Civ Eng 1886;15(1):401–3 489–490.
[5] Wagner H. Flat sheet metal girder with very thin metal web. Tech Memorandum 604–606, National Advisory Committee for Aeronautics (NACA), Hampton, Va, 1931.
[6] Basler K, Thurliman B. Plate girder research. In: Proc Nat Eng Conf., New York: American Institute of Steel Construction, 1959.
[7] Fujii T. On an improved theory for Dr. Basler's theory. Final Rep., IABSE 8th Congress, New York, 1968.
[8] Basler K. New provisions for plate girder design. In: Proc AISC Nat Eng Conf. New York: American Institute of Steel Construction. 1961, p. 65–74.
[9] Basler K. Strength of plate girders under combined bending and shear. J Struct Div, ASCE 1961;87(2):97–181.
[10] Basler K. Strength of plate girders in shear. Transaction of ASCE 1961 New York, American Society of Civil Engineers.
[11] American Institute of Steel Construction, AISC. Seismic provisions for structural steel buildings. Chicago, 1997.
[12] American Association of State Highway and Transportation Officials, AASHTO. LRFD bridge design specifications, 1st ed., Washington, D.C., 1994.
[13] Galambos TV, editor. SSRC guide to stability design criteria for metal structures, 5th Ed. New York: Wiley; 1998.
[14] Porter DM, Rockey KC, Evans HR. The collapse behavior of plate girders loaded in shear. Struct Eng 1975;53(8):313–25.
[15] British Standards Institution (BS). Code of practice for design of steel bridges. BS5400, steel, concrete and composite bridges, Part 3, London, 1982.
[16] Takeuchi, T. Investigation of the load-carrying capacity of plate girders. MS thesis, University of Kyoto, Kyoto, Japan, 1964.
[17] Herzog M. Ultimate strength of plate girders from tests. J Struct Div 1974;100(5):849–64.
[18] Fujii T. A comparison between the theoretical values and the experimental results for the ultimate shear strength of plate girders. IABSE Rep. of the working Commissions, Volume-Band 11, Colloquium, Design of Plate and Box Girders for Ultimate Strength, London, 1971, p. 161–71.
[19] Komatsu S. Ultimate strength of stiffened plate girders subjected to shear. IABSE Rep. of the working Commission, Volume-Band 11, Colloquium, Design of Plate and Box Girders for Ultimate Strength, London, 1971, p. 49–65.
[20] Chern C, Ostapenko A. Ultimate strength of plate girder under shear. Friz Engineering Laboratory Rep. no. 328.7, Lehigh Univ., Bethlehem, Pa, 1969.
[21] Sharp ML, Clark JW. Thin aluminum shear webs. J Struct Div 1971;97(4):1021–38.
[22] Steinhardt O, Schroter W. Postcritical behavior of aluminum plate girders with transverse stiffeners. IABSE Rep. of the Working Commissions, Volume-Band 11, Colloquium, Design of Plate and Box Girders for Ultimate Strength, London, 1971, p. 179–184.
[23] Hoglund T. Behavior and load-carrying capacity of thin plate I girders. R Inst Technol Bull, No. 93. Stockholm, 1971a.
[24] Hoglund T. Simply supported thin plate I girders without web stiffeners subjected to distributed transverse load. IABSE, Colloquium, Design of Plate and Box Girders for Ultimate Strength, London, 1971.
[25] Hoglund T. Design of thin plate I girders in shear and bending. R Inst Technol Bull No. 94, Stockholm, 1973.
[26] EUROCODE3. 1993-1-5: 2003. EUROCODE 3: Design of steel structures.
[27] Lee SC, Yoo CH. Strength of plate girder web panels under pure shear. J Struct Eng 1998;124(2):184–94.
[28] Lee SC, Yoo CH. Experimental study on ultimate shear strength of web panels. J Struct Eng 1999;125(8):838–46.
[29] Lee SC, Yoo CH, Yoon DY. Behavior of intermediate transverse stiffeners attached on web panels. J Struct Eng 2002;128(3):337–45.
[30] Lee SC, Yoo CH, Yoon DY. New design rule for intermediate transverse stiffeners attached on web panels. J Struct Eng 2003;129(12):1607–14.
[31] Yoo CH, Lee SC. Mechanics of web panel postbuckling Behavior in Shear. J Struct Eng 2006;132(10):1580–9.
[32] Marsh C, Ajam W, Ha H. Finite element analysis of postbuckled shear webs. J Struct Eng 1988;114(7):1571–87.
[33] Alinia MM, Habashi HR, Khorram A. Nonlinearity in the postbuckling behavior of thin steel shear panels. Thin-Walled Struct 2009;47(4):412–20.
[34] Alinia MM, Dastfan M. Cyclic behavior, deformability and rigidity of stiffened steel shear panels. J Constr Steel Res 2007;63(4):554–63.
[35] Alinia MM, Dastfan M. Behavior of thin steel plate shear walls regarding frame members. J Constr Steel Res 2006;62(7):730–8.
[36] Alinia MM, Hoseinzadeh SAA, Habashi HR. Buckling and postbuckling strength of shear panels degraded by near border cracks. J Constr Steel Res 2008;64(12):1483–94.
[37] Alinia MM, Hoseinzadeh SAA, Habashi HR. Influence of central cracks on buckling and postbuckling behavior of shear panels. Thin-Walled Struct 2007;45(4):422–31.
[38] Alinia MM, Hoseinzadeh SAA, Habashi HR. Numerical modeling for buckling analysis of cracked shear panels. Thin-Walled Struct 2007;45(12):1065–74.
[39] ABAQUS, Ver. 6.5, Standard user manual. Hibbit, Karlsson & Sorenson, Inc. (HKS), 2004.
[40] Real E, Mirambell E, Estrada I. Shear response of stainless steel plate girders. Eng Struct 2007;29:1626–40.
[41] Shanmugam NE, Baskar K. Steel–concrete composite plate girders subject to shear loading. ASCE J Struct Eng 2003;129(9).
[42] White DW, Barker MG. Shear resistance of transversely stiffened steel I-girders. ASCE J Struct Eng 2008;134(9):1425–36.