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Shear strength and design of trapezoidally corrugated steel webs

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ABSTRACT

Due to the accordion effect, corrugated steel webs are only able to resist shear force. The shear force in the web can cause three different buckling modes: local, global and interactive shear buckling. Although several researchers have been investigating it, the shear buckling behavior of the corrugated webs has not yet been clearly explained, this leads to conservative design. This paper presents the shear strength and design of trapezoidally corrugated steel webs. Firstly, global shear buckling equations are rearranged in order to derive the global shear buckling coefficient. The interactive shear buckling coefficient and the shear buckling parameter for corrugated steel webs are then proposed based on the 1st order interactive buckling equation. The inelastic buckling strength is determined from the buckling curves based on the proposed shear buckling parameter. A series of tests are conducted to verify the proposed design equations. From the test results of this study and those provided by previous researchers, it was found that the proposed shear strengths provide good predictions for the shear strength of the corrugated steel webs.

1. Introduction

Trapezoidally corrugated steel webs, as shown in Fig. 1(a), provide enhanced shear buckling strength and weight savings due to elimination of the need for transverse stiffeners. While PSC box girder bridges with corrugated steel webs have been extensively constructed in France and Japan, the first PSC box girder bridge constructed with corrugated steel webs has recently been constructed in South Korea [1]. In addition, applications of corrugated steel webs have been extended to extra-dosed and cable-stayed bridges [2], and, there have been several attempts to apply corrugated webs to plate girder bridges [3–5].

For a corrugated steel web, it is assumed that the web carries only shear forces due to the accordion effect [6]. Because of this characteristic, the corrugated steel webs fail due to shear buckling or yielding. Three different shear buckling modes (local; global; and interactive) are possible, depending on the geometric characteristics of corrugated steel webs. Fig. 1(b) shows the geometric notations of the corrugated steel webs used in this study. In Fig. 1(b), a is the flat panel width; b is the horizontal projection of the inclined panel width; c is the inclined panel width; d is the corrugation depth; t_w is the web thickness; and θ is the corrugation angle.

Several researchers have conducted extensive studies on the shear buckling of corrugated webs [1,2,7–10]. Easley and McFarland [7] proposed the global shear buckling equation of corrugated webs by treating the corrugated web as an orthotropic flat web. Elgaaly et al. [8] and Yamazaki [9] conducted experimental studies on the buckling characteristics and strength of corrugated webs. Recently, Yi et al. [1] studied the nature of the interactive shear buckling of corrugated webs, and concluded that the 1st order interactive shear buckling equation that does not consider material inelasticity and material yielding provides good a estimation of the shear strength of corrugated steel webs. Based on test results and finite element analyses, Driver et al. [10] suggested shear design criteria for corrugated webs that are suitable for bridge design specification. However, despite the significant amount of research that has been carried out, the shear buckling designs of corrugated webs have been conducted with a large margin of safety.

This paper presents shear strength and design criteria of trapezoidally corrugated webs, based on the 1st order interactive equation proposed by Yi et al. [1], and test results for the shear buckling of corrugated webs. Global and interactive shear buckling equations are rearranged into a form of the classical plate buckling equation. The global and interactive shear buckling coefficients k_G

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Notations

The following symbols are used in this paper:

a	flat panel width;
b	horizontal projection of the inclined panel width;
b_f	width of flange;
c	inclined panel width;
D_x	bending stiffness per unit length about x axis (strong axis);
D_y	bending stiffness per unit length about y axis (weak axis);
d	corrugation depth;
d_{avg}	average corrugation depth;
$d_{\max}$	maximum corrugation depth;
$d_{\min}$	minimum corrugation depth;
E	Young's modulus of elasticity;
f_y	yield stress;
G	shear modulus of the flat plate;
G_{co}	shear modulus of the corrugated plate;
h_w	web height;
k_G	global shear buckling coefficient;
k_I	interactive shear buckling coefficient;
k_L	local shear buckling coefficient;
t_w	web thickness;
t_f	thickness of flange;
L	length of girder;
P	Applied load;
w	maximum fold width;
α	ratio of flat panel width to inclined panel width (a/c);
β	global buckling factor that depends upon the boundary condition;
δ	vertical displacement;
φ	factor which is introduced to provide a margin of safety on global buckling strength;
γ	shear strain;
η	length reduction factor defined as $(a+b)/(a+c)$;
λ_L	local buckling slenderness;
λ_s	shear buckling parameter;
θ	corrugation angle;
τ_{cr}	critical shear buckling stress;
$\tau_{cr,FEM}$	shear buckling strength obtained by finite element analysis;
$\tau^e_{cr,G}$	elastic local shear buckling stress;
$\tau^e_{cr,I}$	elastic interactive shear buckling stress;
$\tau^e_{cr,L}$	elastic global shear buckling stress;
τ_y	shear yielding stress;
υ	Poisson's ratio.

and k_I are then derived as functions of d/t_w and w/h_w, where, w is the maximum fold width; and h_w is the web height. The shear buckling parameter of corrugated webs, λ_s, is then proposed. The shear buckling strength, considering material inelasticity, residual stresses, and initial imperfections, can be determined from the buckling curves using the proposed λ_s. A series of tests are performed with large corrugated webs in order to verify the proposed shear strength. Proposed shear strengths are also compared with those suggested by previous researchers [10,11]. From the results, the proposed shear strengths were successfully verified and provided a good estimation of the shear strength of the corrugated webs.

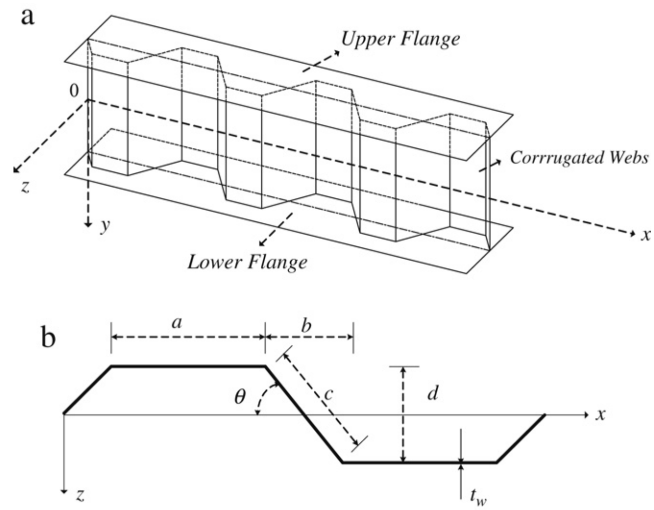

Fig. 1. I-girder with corrugated steel webs: (a) Profiles of I-girder with corrugated webs; (b) Geometric notations.

Table 1
Profiles of existing bridges with corrugated steel webs

Name of bridge	a (mm)	b (mm)	d (mm)	c (mm)	η	w/h_w	d/t_w
Sinkai bridge	250	200	150	250	0.90	0.21	16.67
Matunoki bridge	300	260	150	300	0.93	0.14	15
Hondani bridge	330	270	200	330	0.91	0.10	22.22
Cognac bridge	353	319	150	353	0.95	0.20	18.75
Maupre bridge	284	241	150	284	0.92	0.11	18.75
Dole bridge	430	370	220	430	0.93	0.17	22

2. Elastic shear buckling of trapezoidally corrugated steel webs

2.1. Local shear buckling

The presence of local shear buckling is characterized by the buckling of individual sub-panels. It is assumed that corrugated webs are treated as a series of flat rectangular sub-panels supporting each other along their vertical edges and by the flange along their horizontal edges. The elastic local shear buckling stress of the corrugated webs, $\tau^e_{cr,L}$, can be determined by the classical plate buckling theory [12] and expressed as

$$\tau^e_{cr,L} = k_L \frac{\pi^2 E}{12(1-\upsilon^2)} \left(\frac{t_w}{w}\right)^2 \tag{1}$$

where E is Young's modulus of elasticity; υ is Poisson's ratio; w is the maximum fold width (maximum of flat panel width a and inclined panel width c); and t_w is the web thickness. k_L is the local shear buckling coefficient. Assuming that the panel has simply supported edges, k_L is given by

$$k_L = 5.34 + 4\left(\frac{w}{h_w}\right)^2. \tag{2}$$

k_L is a function of the aspect ratio of the sub-panel, w/h_w. Table 1 represents profiles of existing bridges in France and Japan that have corrugated webs. It is found that the w/h_w on actual bridges that have been constructed to date, are generally smaller than 0.2, as shown in Table 1. Fig. 2 shows the variations in k_L with w/h_w. The difference between Eq. (2) and $k_L = 5.34$ is smaller than 2.9% when $w/h_w \leq 0.2$. Therefore, $k_L = 5.34$ is recommended for practical design purposes.