World Applied Sciences Journal 8 (5): 527-530, 2010
ISSN 1818-4952

The Effects of the Corrugation Profiles of the Web on the Lateral-Torsional Buckling Strength of the Inelastic I-Girder

[1]H.R. Kazemi Nia Korrani and [2]S. Molanaei

[1]Department of Environmental and Civil Engineering, Amirkabir University of Technology Tehran, Iran
[2]Department of Civil Engineering, Kurdistan University, Kurdistan, Iran

Abstract: Lateral-torsional buckling is one of the modes of instability of steel beams that happened when steel beams are slender. Due to the widespread application of I-girder in bridges, this paper develops a three-dimensional finite-element model using ANSYS for the inelastic nonlinear flexural-torsional analysis of I-girder and uses it to investigate the effects of the corrugation profiles of the web on the lateral-torsional buckling strength. In this paper, all I-girders are simply supported and models flange are constant but webs used of I-girder are different, critical moment of beams have calculated and compared. Results show that critical moment increases with change of web from constant to corrugation. Also shape of corrugation is very effective to increased critical moment.

Key words: Inelastic lateral-torsional buckling . Corrugated web I-girder . finite element method

INTRODUCTION

If the beam length between two supports exceeds a given threshold, compression flange becomes unstable and tends to buckle laterally prior to reaching maximum flexural strength. Therefore, the moment of inertia around the x-axis should be much greater than that of around the y-axis to reduce flexural stress [1, 2]. Low moment of inertia around the y-axis in the beams with high distance between lateral supports can cause a further increase in the lateral bending in the compression flange and torsion in the section. Accordingly, if the compression flange is not properly braced laterally, the possibility of sudden failure of compression flange will increase. Such failure resulting from increase in the compression stress in the flange due to the beam bending and lateral bending due to lack of laterally bracing is called lateral-torsional buckling. Lateral-torsional buckling of beam is associated with the rotation of section. This rotation is a combination of pure torsion and warping torsion. Therefore, compression stress produced in the flange is the result of three types of stress (stress due to bending around x-axis, lateral bending and warping) and leads to decrease in the bearing capacity [3, 4].

AISC considers pure bending as a base in the calculation of buckling moment. This is because the beam buckling under pure bending is the most critical condition and can be simply determined. The determination of buckling moment of a simply supported beam under pure bending has been addressed in many references. The relation is as follows [5, 6, 7]:

$$(M_0)_{cr} = \sqrt{\frac{\pi^4}{L^4} EC_w EI_y + \frac{\pi^2}{L^2} EI_y GJ} \qquad (1)$$

where, $(M_o)_{cr}$ is critical moment in pure bending, L is unbraced length assumed to equal to free span length, E is elasticity modulus of materials, Cw is warping constant, Iy is the weak-axis moment of inertia, G is shear modulus of materials and J is polar moment of inertia of the section.

The above equation may be applied when equal end moments are imposed on the beam and consequently no shear occurs in the beam. However, in practice, end moments are not often equal. In order to consider bending moment changes along distance between two lateral supports, moment-gradient factor was multiplied by $(M_o)_{cr}$:

$$M_n = C_b .(M_0)_{cr} \qquad (2)$$

M_n is nominal moment for beams under impure bending and C_b is moment-gradient factor. The following relation was suggested by AISC to determine C_b [8, 9].

$$C_b = 1.75 + 1.05\frac{M_1}{M_2} + 0.3\left(\frac{M_1}{M_2}\right)^2 \le 2.3 \qquad (3)$$

Corresponding Author: Mr. H.R. Kazemi Nia Korrani, Department of Environmental and Civil Engineering, Amirkabir University of Technology Tehran, Iran

Beams show three ranges of behavior: (1) elastic buckling, which governs for long unbraced beams (of importance during construction); (2) inelastic buckling, when instability occurs after some portions of the beam have yielded; and (3) plastic behavior, where the unbraced length is short enough so the yielding of whole section occurs before any type of buckling happens [10, 11].

When a beam behavior is elastic, the nominal moment is given by Eq. (2) and for a beam with a low slenderness that buckles inelastically is as follows:

$$M_n = C_b \left[M_p - (M_p - M_r)\left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \le M_p \qquad (4)$$

Eq. (4) implies that AISC [8] proposes a constant value of C_b for all ranges of inelastic beam's slenderness. And also, it should be mentioned that the moment capacity obtained by multiplying the moment by C_b in Eq. (4) may not be larger than the plastic moment (Mp= Fy.Z).

NONLINEAR FINITE ELEMENT MODEL

To investigate the inelastic flexural-torsional buckling behavior of I-girder with corrugated webs, a nonlinear inelastic finite element model has been developed based on the assumptions that the cross section of the beam is doubly symmetric. Dimensions of croup1 models are shown in Table 1.

Mesh and material properties: The nonlinear computations were performed using the commercial finite element software package ANSYS [12]. ANSYS has the ability to consider both geometric and material nonlinearities in a given model. However, all modeling reported here in only considers nonlinear material influences. Four side shell elements SHELL 43 from the ANSYS element library were used to model the web, flange (top and bottom).

The trilinear elastic-plastic strain-hardening stress-strain curve of Fig. 1 is assumed. Young' modulus E was set to 210 GPa and Poisson's ratio was set to 0.3. Residual stresses were not considered in this work, though it is relevant in this type of analysis.

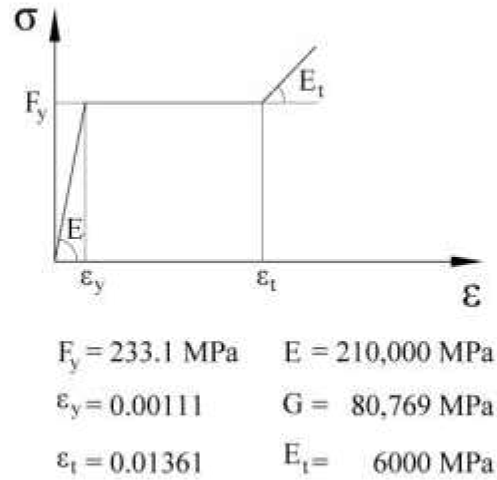

Fig. 1: Uniaxial constitutive model considered

Load and restraints: Simply supported I-girder with corrugated webs with different corrugation profiles are chosen under pure bending conditions in order to evaluate the critical moment for them. A typical configuration of the expanded beam and the notation adopted are shown in Fig. 2. All beams were provided with bearing stiffeners at support points. And also, it should be mentioned that in finite element analysis of beams subjected to pure bending, the end moments were applied as couples in order to prevent web local yielding as shown in Fig. 3. It should be noted that contrary to warping, twist may not occur in the two ends of the beam.

Model validation: Three different lengths for webbed beams have been selected to validate the model (IPE140, IPE180 and IPE220). Afterwards, the results obtained from the model have been compared with those from theoretical relation proposed by Timoshenko. This is illustrated in Fig. 4 (moments are in KN.m).

Table 1: Dimensions of analysis croup1

Model no.	a (mm)	b (mm)	c (mm)	d (mm)	t_w (mm)	h_w mm)	b_f(mm)	t_f (mm)	L (mm)
Type1	125	250	500	250	12	2000	500	40	15000
Type2	125	250	500	200	12	2000	500	40	15000
Type3	125	250	500	150	12	2000	500	40	15000
Type4	125	250	500	100	12	2000	500	40	15000
Type5	125	250	500	50	12	2000	500	40	15000
Type6	125	250	500	0	12	2000	500	40	15000
No corrugated	0	250	500	0	12	2000	500	40	15000

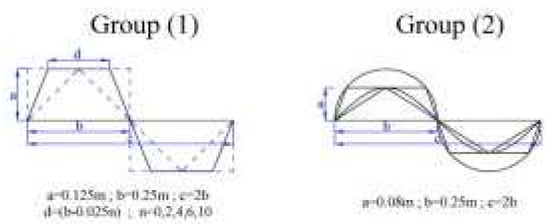

Fig. 2: Webs corrugation profiles of I-girder

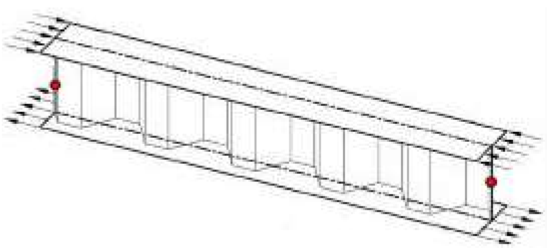

Fig. 3: Applying ending moments

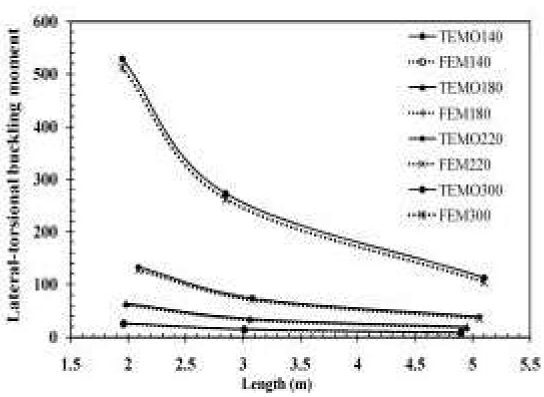

Fig. 4: Comparison of results

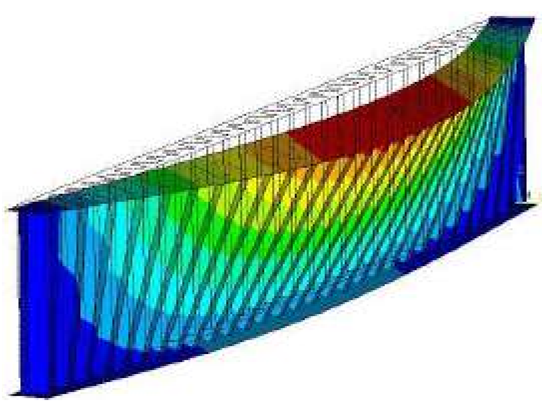

Fig. 5: Lateral-torsional buckling shape from finite element analyses

As cited in the Fig. 4, the difference between these two methods is low and is 3 percent in some cases. This low difference can be attributed to the approximations used in the Timoshenko method. Accordingly, beams modeled in the software can be accepted.

Table 2: Result of analysis croup1

Model no.	Mcr(MN.m) Mode 1	Mode 2	Mode 3	Mode 4	Mode 5
Type1	11.534	14.269	14.396	34.803	34.856
Type2	10.720	34.078	71.414	95.053	95.828
Type3	10.228	33.411	70.593	91.170	91.440
Type4	9.867	32.977	70.145	85.564	86.131
Type5	9.538	32.473	69.440	83.555	84.525
Type6	9.266	32.343	69.592	84.170	84.341
No corrugated	7.979	13.398	13.399	13.598	13.601

Table 3: Result of analysis croup2

Model no.	a (mm)	b (mm)	c (mm)	M_{cr} (MN.m)
Circle web	125	250	500	9.955
Trapezium web	80	250	500	9.271
Sin web	80	250	500	8.888
Triangle web	80	250	500	8.782
No corrugated	0	250	500	7.979

Determination to critical moment: In order to consider the effects of the corrugation profiles of the web on the lateral-torsional buckling strength, I-girder two-groups are considered (Fig. 3). Simply supported beams under and moments have been investigated. After applied end moments, compression flange buckled and is outside the page (Fig. 5). Result of I-girder two-groups are shown in Table 2 and 3.

To observe, if "d" decrease (corrugation profiles of the web change from rectangular to triangular) so does critical moment and I-girder with rectangular corrugation profiles of the web has the most critical moment.

Also other comparison is considered that relation to shape of corrugation profiles of the web of group 2. In this group, webs five has been used that are: 1) Circle; 2) Trapezium; 3) Sinusoidal; 4) Rectangular and 5) without corrugated. Results of this group are illustrated in Table 3.

CONCLUSIONS

In this study phenomenon of lateral-torsional buckling of the inelastic I-girder with corrugation profiles of the web has been considered. In order to better comparison, two groups of corrugation profiles of the web was considered and critical moment of no corrugated web was achieved. The results were shown that can increased critical moment to 40 percent using corrugation profiles of the web. Also, whatever "d" decrease, so does critical moment and I-girder with rectangular corrugation profiles of the web has the most critical moment. Critical moment of modes from 2 to 5 was closed in order to web buckling.

REFERENCES

1. Azhari, M. and R. Mirghaderi, 2003. Design of steel structures. Arkan publication, Esfahan, First volume, Second edition In Persian.
2. Tahooni, Sh., 2007. Design of steel structures on the basis of Iranian steel building code. Elmo Adab Publication, Tehran, In Persian.
3. Mahmoodian, B., 1995. lateral-torsional buckling of web in the castellated beams. M.Sc Thesis in Civil Engineering, Tehran University.
4. Nethercot, D.A. and D. Kerdal, 1982. Lateral-torsional buckling of castellated beams. Struct. Eng, Lond, pp: 53-61.
5. Chen, W.F. and E.M. Lui, 1987. Structural Stability. (Theory and Implementation), Elsevier Science.
6. Jiho, M., W. Jong, H.C. Byung and L. Hak-Eun, 2009. Lateral-torsional buckling of I-girder with corrugated webs under uniform bending. Thin-walled Structures, Elsevier, pp: 21-30.
7. Timoshenko, S.P. and J.M. Gere, 1961. Theory of Elastic Stability, Second Edition, McGraw-Hill.
8. American Institute of Steel Construction (AISC), 1999. Load and resistance factor design specification for structural steel buildings, Chicago (IL), AISC.
9. Iranian Steel building code, 2005 In Persian.
10. Kerdal, D. and Nethercot, 1984. DA. Failure modes for castellated beams. J. Construct. Steel Res., pp: 295-315.
11. Mohebkhah, A., 2004. The moment-gradient factor in lateral-torsional buckling on inelastic castellated beams. Constructional Steel Research, Elsevier, pp: 1481-1494.
12. ANSYS, User's manual, version 10.0.