Buckling Analysis of Web Panel Girder in Steel Bridges Loaded in Shear

Esam M. Aziz
Sulaimani Polytechnic University

ABSTRACT

Plate girders in bridges are designed for many loading scenarios including; gravity, wind, impact, and earthquake. Under these loading scenarios, web plate of steel bridge girder is highly vulnerable for buckling due to high slenderness ratio of web plate. This paper presents a numerical modeling on the elastic and inelastic behavior of web panel girder subjected to shear loading using finite element program ABAQUS. The elastic buckling problem includes Eigen value analysis while, geometrical, material nonlinear analysis are performed to capture the inelastic response of web panel girder. Results from analysis show that, the influence of geometric nonlinearity can be higher than the material nonlinearity effect on the inelastic response of web panel girder.

Keywords: web buckling, steel bridges, elastic and inelastic behavior, finite element analysis, Eigen value problem.

INTRODUCTION

Steel plate girders are widely used and designed to carry heavy loads as in bridges, building floor, and cranes. In general, plate girder is fabricated by welding together two flanges, a web and a number of stiffeners. The flange plates design to resist bending moment web plates maintain the relative distance between flanges and resist shear. The most efficient way to increase the flexural capacity of the girder is by increasing the depth of the girder taking into account that the self-weight of the girder must be minimized for economic reasons. This can be achieved be reducing the thickness of the web plate. Furthermore, the web thickness has to be reduced to certain limit (the web proportions are normally expressed in terms of the web slenderness ratio, d/tw). However, reducing the web thickness causes various forms

Esam M. Aziz: Email. aziz.m.esam@gmail.com

of instability such as shear buckling of the web, compression buckling of the web, lateral-torsional buckling, flange-induced buckling of the web, and web crippling [1]. In this paper, the shear-induced buckling (elastic and post buckling) of the web is studied using finite element program ABAQUS considering elastic and inelastic analysis.

SHEAR RESISTANCE OF A WEB

When the applied shear force is increased, the failure mode of the web depends on the panel aspect ratio (d/b) and the slenderness ratio of the web (d/tw). Where d is the depth of the girder, tw is the thickness of the web; b is the distance between stiffeners. When the web is too thick then the web fails by shear yielding; and the shear yield strength is:

$$\tau_{yw} = \sigma_{yw}/\sqrt{3}$$

Where, σ_{yw} is the uniaxial tensile strength of the web but the web in general is thin and tends to buckle before yielding. The shear resistance of a web can be defined in three stages:

Pre-buckling behavior

By considering a rotated stress state of a point at the center of single panel of a plate girder subjected to shear forces. The applied shear force induces compressive and tensile principal stress as shown in Fig. 1.(a). With increasing the applied load, the web plate will buckle in the direction of the compressive principal stress and the plate will lose its capacity to resist more compressive stress. The corresponding shear stress at this point called the elastic critical shear stress. At this stage, the boundary conditions of the plate are very complicated because of the restrained offered by the flanges and stiffeners. Therefore the conservative assumption will be a simply support boundary condition. According to this assumption the elastic buckling stress can be calculated from [2]:

$$\tau_{cr} = \frac{\pi^2 E}{12(1-\mu^2)({}^{d}/_{t_w})^2} \leq \tau_{yw}$$

Where μ is poisson's ratio, E is elastic modulus of the web, k is buckling coefficient:

$$k = 5.35 + 4(\frac{d}{b})^2 \qquad , for\ \frac{b}{d} > 1$$

$$k = 5.35 + 4(\frac{d}{b})^2 \qquad , for\ \frac{b}{d} > 1$$

Post-buckling behavior

Once the critical shear stress reached, the web plate will not be able to resist any increase in shear force. The additional shear forces will be carried by development of tension field action (tensile membrane stress σ_t in the diagonal band of the web) as shown in Fig. 1.(b). For a web panel subjected to pure shear, the tensile membrane stress the cause yielding of the web is as follow [3]:

$$\sigma_{ty} = -\frac{3}{2}\tau_{cr}sin2\theta + \sqrt{\sigma_{yw}^2 + \tau_{cr}{}^2\left[(\frac{2}{3}sin2\theta)^2 - 3\right]}$$

Where θ is the angle of the membrane tensile yielding stress σ_{ty}

Collapsed behavior

The failure of the girder starts when sufficient numbers of hinges will form in the top and bottom flanges at the diagonal yield zone prior to formation of plastic sway mechanism as shown in Fig. 1.(c). The additional resistance to applied shear force V_f can be written as [3]:

$$V_f = 2ct_w\sigma_{ty}sin^2\theta + \sigma_{ty}dt_wsin^2\theta(cot\theta - cot\theta_d)$$

Where, $\cot\theta_d = {}^{d}/_{b}$

$$c = \frac{2}{sin\theta}\sqrt{\frac{M_{pf}}{\sigma_{ty}t_w}}$$

Where, the first term of the equation is the contribution of the flanges to panel shear strength. Finally, the total shear resistance will be the summation of all the three stages.

$$V_u = V_{cr} + V_f$$

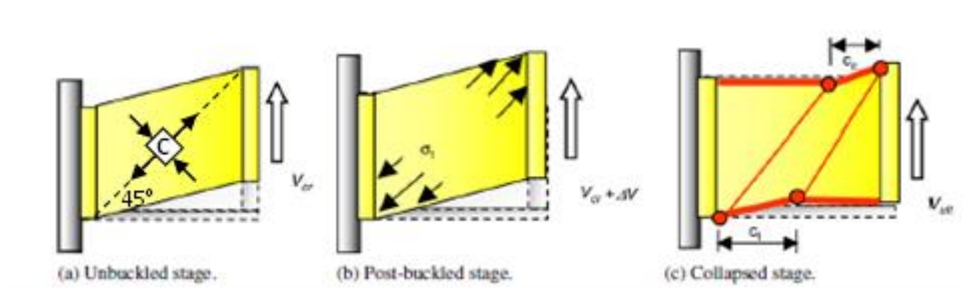
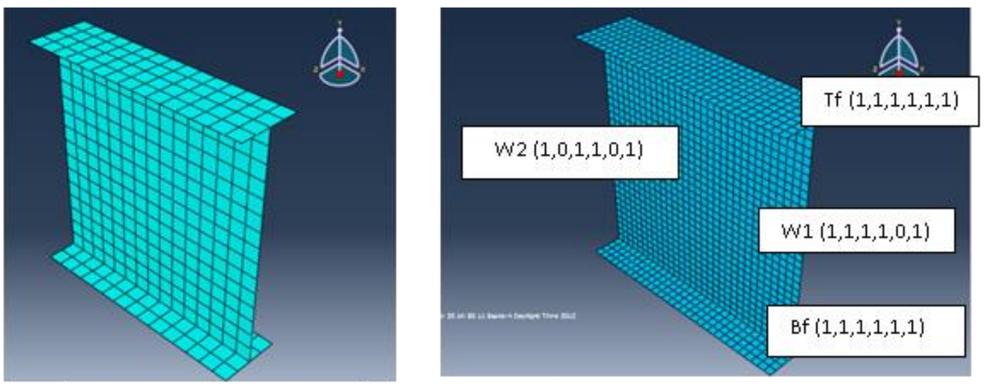


Fig. 1. Web Panel behavior under pure shear

CASE STUDY

To illustrate the response of a steel plate girder subjected to shear forces a set of analysis namely, elastic buckling (Eigen value problem), 1st order elastic, 2nd order elastic, 1st order inelastic, 2nd order inelastic analysis are carried out using the finite element computer program ABAQUS [4]. For the analysis, an isolated plate girder panel is used as shown in Fig. 2(a). The panel girder has aspect ratio (d/b) of 1 and slenderness ratio (d/tw) of 153. The boundary conditions of the girder panel setup in a way that the both ends of web remain rigid to represent the effect of stiffeners as illustrated in Fig. 2(b). The applied load considered as a pressure load on the top flange of the panel girder.



(a) Isolated web panel girder (b) Boundary conditions

Fig. 2. Web panel girder used in analysis

NUMERICAL MODELING

For the discretization of the web panel girder (web, and the flanges), shell elements S4R is used. S4R is a general purpose shell element with four nodes. Each node has six degrees of freedom per node, three translations in x, y, and z directions and three rotations about x, y, and z-axes with linear interpolation relation between the nodes. This element can capture buckling of web and it is well-suited for large rotation, large strain and nonlinear problems. Initial imperfection in the web is considered to perform 2nd order elastic, and 2nd order inelastic analysis. According to the AASHTO specification which limits the imperfection in the bridge girders by (d/120), therefore (d/240) imperfection is used in this project [5].

Stress-Strain Model for Steel

In analysis an A36 steel with elastic-plastic relationship between stress and strain is used as shown in Fig. 3. The given stress-strain model in Fig. 3, represent engineering stress and strain. All the stress and strain values are converted to true stress and plastic true strain in order to use them in ABAQUS program. The relationships between the engineering stress and strain (nominal stress and strain) are given as following:

$$\sigma_{true} = \sigma_E(1 + \varepsilon_E)$$

$$\varepsilon_{true} = \ln(1 + \varepsilon_E)$$

$$\varepsilon_{plastic} = \sigma_{true} - \frac{\sigma_{true}}{E}$$

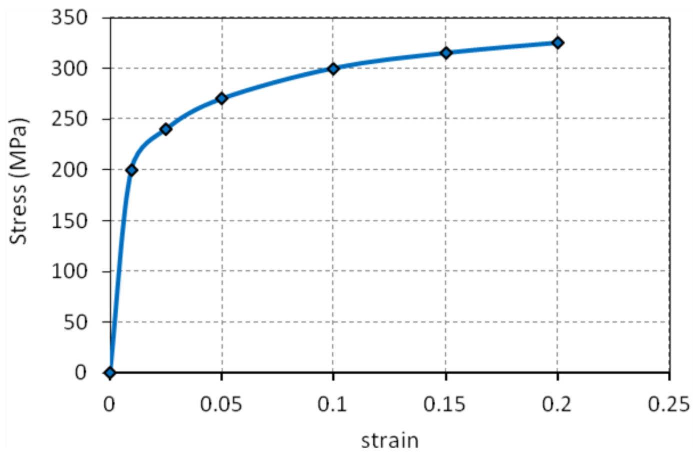


Fig. 3. Engineering stress-strain relationship for steel used in analysis

ANALYSIS AND DISCUSSION

Eigen value analysis

An Eigen value problem analysis is performed to find the elastic buckling load for the web panel girder. In the analysis five modes is selected for the buckling analysis. The effect of element size (element number) on the elastic buckling load is studied by repeating the analysis many times with different element numbers. The total numbers of the web plate that is considered are (31x31, 20x20, 15x15, 10x10, and 5x5). The result of buckling load for mode 5 is shown in Fig. 4 as an example while Fig. 5 shows some of the buckling modes. It can be seen that mesh size can affect the result significantly and with increase the number of the element the results get more stable. This can be attributed to the linear assumption for shape function of the deformation relationship between the nodes. Therefore, using more element reduce the error of the result.

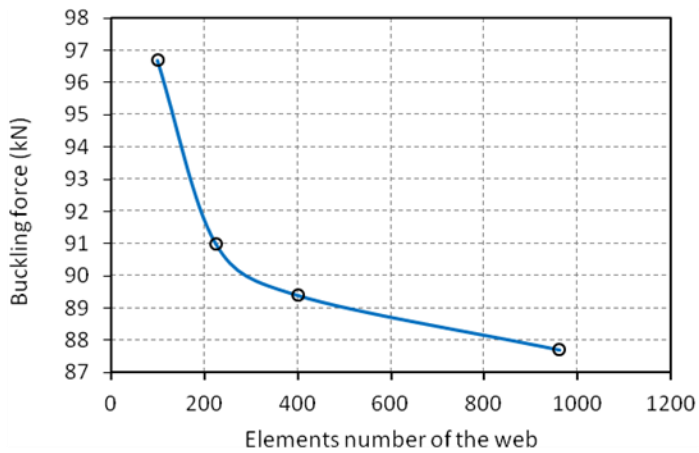


Fig. 4. Mesh size effect on buckling result for Mode 5

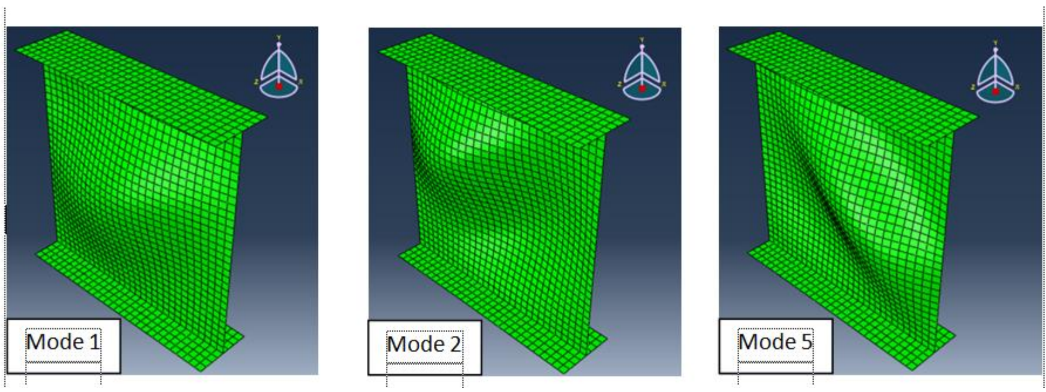


Fig. 5. Buckling modes of the web panel girder under shear force

Elastic and inelastic analysis

The linear elastic and inelastic response of web panel girder under shear force considering geometric and material nonlinearity are shown in Fig.6 where, the mid-panel out of plane displacement plotted with the applied shear force. For 2nd order elastic analysis, the web panel response experienced softening with increasing shear force due to initial imperfection till the point when the tension field action developed. After that the response gets stiffened because of tension field and since there is no yielding of the steel then the stiffened response increases up to instability situation. By comparing the response of 1st order inelastic and 2nd order elastic analysis, it can be seen that influence of the geometric nonlinearity on the response of the web panel girder is higher than the effect of material nonlinearity. This could be realistic for this type of problem due to the imperfection in the web and the critical instability of the slender web that soften the response upon the development of tension field action that support the web stability and resistance for shear forces. From the response of panel girder in 2nd order analysis, it can be seen that the out of plane displacement increase with increasing of applied shear force till the elastic buckling of the web which is not well define in this analysis. This is because of the high imperfection provided in analysis. After the elastic buckling, the web panel can resist more shear forces due to tension field action and finally the failure developed by instability of the web. The contour line result for out of plane displacement for different analysis cases are shown in Fig. 7.

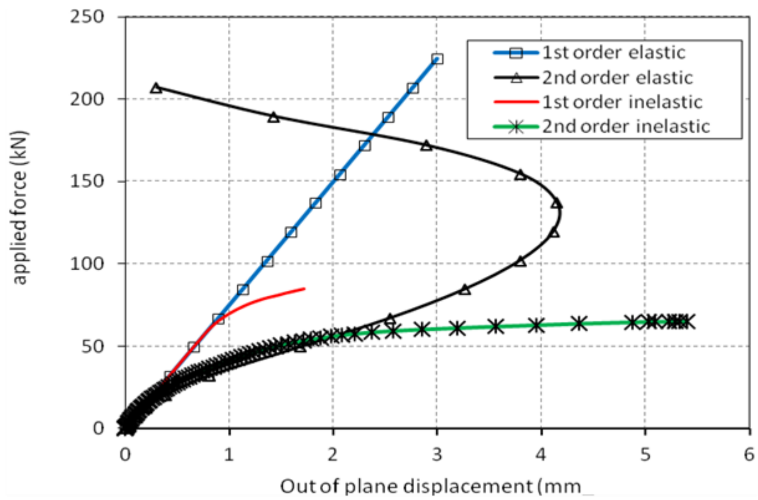


Fig. 6. Web panel girder response for different analysis cases

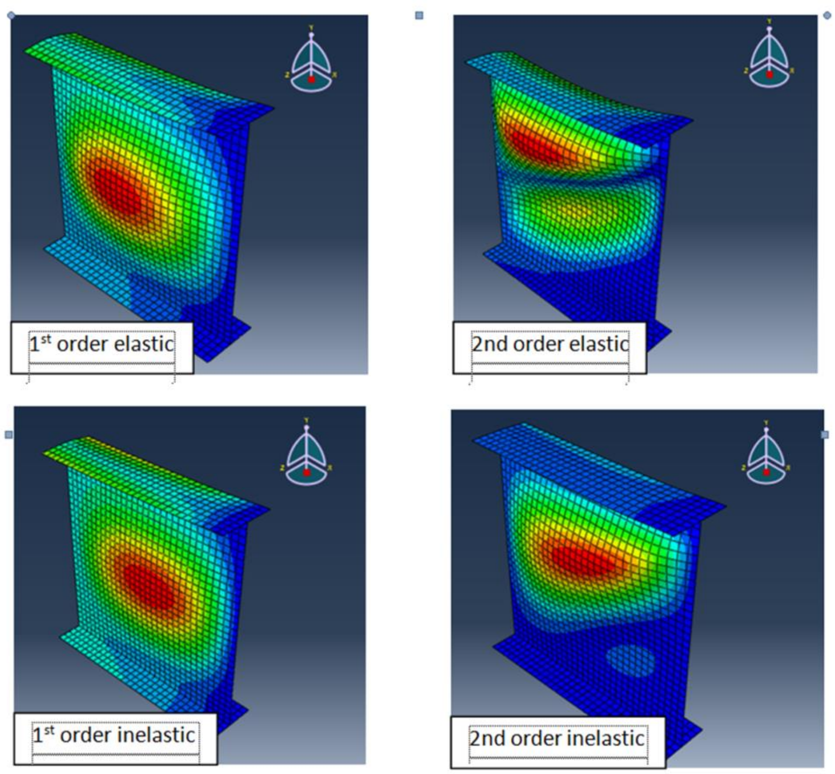


Fig. 7. Out of plane displacement contours for different analysis cases

CONCLUSIONS

Based on the results of numerical study the following conclusions can be drawn:

1. There are significant differences in the response results between all the type of analysis that are performed including; 1^{st} and 2^{nd} order elastic and inelastic analysis.
2. The mesh size has significant influence on reducing the error in finite element solution.
3. Using shell element can predict the buckling of the web panel successfully.
4. The effect of geometric nonlinearity could be higher than the material nonlinearity effect on the response of web panel girder.
5. The 2^{nd} elastic and inelastic analysis cannot be done in finite element software without providing initial imperfection.

REFERENCES

[1] M.M. Alinia, Maryam Shakiba, H.R. Habashi, 2009. “Shear failure characteristics of steel plate girders”.
[2] Rockey KC, Skaloud M. 1972. “The ultimate load behaviour of plate girders loaded in shear”. The Structural Engineering.
[3] Vanisson Vimonsatit, Kang-Hai Tan, Seng-Kiong Ting, 2010. “Shear strength of plate girder at elevated temperature”.
[4] Abaqus/CAE, Ver. 6.10, 2010. Standard user manual.
[5] AASHTO, 2009. LRFD Bridge design specifications. Washington D.C., USA.
[6] Pawel Woelke, 2000. “Elasto-plastic and damage analysis of plates and shells”. Poznan University of Technology, Ph.D dissertation.
[7] William McGuire, Richard H. Gallagher, Ronald D. Ziemian, 2000.” Matrix Structural Analysis” second edition.
[8] Chen and Han, 1987. “Plasticity for structural engineers”.