Composite Materials and Engineering, *Vol. 2, No. 1 (2020) 13-24*
DOI: https://doi.org/10.12989/cme.2020.2.1.013

Buckling of non-sway Euler composite frame with semi-rigid connection

Mostafa G. Ghadimi*

Department of Civil Engineering, Sarab Branch, Islamic Azad University, Sarab, Iran

(Received May 1, 2019, Revised September 20, 2019, Accepted October 5, 2019)

Abstract. The stability functions are calculated to obtain critical elastic buckling loads of asymmetric and axisymmetric one-span non-sway bending frames made up of laminated thin beams and columns with through-thickness mechanical properties variation subjected to axial compression. The shear and axial deformations are neglected. It is assumed that the members are perfect and axial compression is applied to neutral axis without eccentricity. The relative rotations of beams with respect to columns are occurred due to semi-rigid connections at joints of the bending frame. The perfect connection of two different rectangular thin plates with the same width and dissimilar elasticity modulus and thickness produces intact laminated members with similar curvature at junction of the plates in the buckled member. The mechanical and geometrical properties of laminated members in axial direction are invariant, as result the stiffness coefficient, modified stiffness coefficient, reduced stiffness coefficient and carry over factor are independent from thickness, length and layers' mechanical properties variations, but critical buckling loads of heterogeneous frames are dependent on these parameters. The results show that the dimensionless critical load of heterogeneous frame is same as the dimensionless critical load of homogeneous frame.

Keywords: buckling; non-sway bending frame; axisymmetric and asymmetric shape modes; semi-rigid connection; composite member; stability function

1. Introduction

The initial bending may change the lateral buckling load. In the previous studies, it is shown that the elastic buckling load of symmetric portal frame is affected by the initial bending, noticeably. In the case of anti-symmetric frames the changes are not so seriously like the symmetric frames (Chilver 1956). Lack of adequate information or conservative methods for the efficient design of steel structures against out-of-plane failure is an important issue. The method of design by buckling analysis can improve this situation. The design by buckling analysis can use member nominal design strengths in terms of the section moment capacities or compression capacities and the maximum moments at elastic buckling (Trahair 2009). In general, a two-step approach is applied to perform buckling analysis of steel frame. Firstly, a linear-elastic analysis is done to calculate the internal forces and moments; and secondly, an initial imperfection is considered to perform the buckling analysis and design for each individual element of the frame. In the steel design codes the element effective length factor *K*, depends on the buckling shape of that particular element within the

*Corresponding author, MSc, E-mail: mostafa.ghanizade.ghadimi@gmail.com

http://www.techno-press.org/?journal=cme&subpage=7 ISSN: 2671-4930 (Print), 2671-5120 (Online)

$N.A.$	Neutral axis
$\bar{y}$	Distance between $M.A.$ and $N.A.$
L	Member length
ρ	Curvature radius
w	Width
H	Total thickness
t_1	First layer thickness
t_2	Second layer thickness
E_1	Elasticity modulus of first layer
E_2	Elasticity modulus of second layer
$\bar{Y}$	Ordinate of $N.A.$
P	Axial compression load
V	Shear load in buckled member
M_z	Bending moment in buckled member
$\bar{E}$	Dimensionless elasticity modulus (E_2/E_1)
$\bar{t}$	Dimensionless thickness ($t_1(H-t_1)/H^2$)
$\overline{EI}$	Buckling load ratio of heterogeneous frame to homogeneous frame
EI_{eq}	Equivalent bending rigidity
y	Buckling deflection
θ	Rotation at pinned end
s	Stiffness coefficient
c	Carry over factor
α	Reduction stiffness parameter
$\tilde{s}$	Modified stiffness coefficient
$\bar{s}$	Reduced stiffness coefficient
n	Dimensionless buckling load