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Local and post-local buckling behavior of welded steel shapes in partially encased composite columns

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ABSTRACT

This paper presents a theoretical study on both local and post-local buckling behaviors of partially encased composite (PEC) columns, made with thin-walled, welded H-shapes and concrete encasement between flanges; transverse links are welded between flange tips to reinforce the section. Nonlinear finite element analysis (FEA) was conducted to predict buckling behaviors and strengths of steel shapes. Finite element models were verified through a comparison of FEA results with experimental results. A parametric study was then performed using validated FEA models to investigate the effect of several parameters on the buckling behavior of PEC columns. The residual stress of steel shapes, which is introduced through welding process, was discussed in detail. Based on the parametric study, a series of expressions was developed for predicting critical strength, post-buckling strength, as well as the entire stress-strain relationship of steel shapes in PEC columns under concentric loading.

1. Introduction

The partially encased composite (PEC) beam-column is a type of structural element for steel-concrete composite structures, which is mainly designed to carry vertical load in high-rise buildings. A typical configuration of such section is illustrated in Eurocode 4 [1], consists of hot-rolled steel H-shape and concrete encasement between two flanges, which could provide additional capacity, stiffness and fire resistance. The steel shape in this section is designed to avoid local buckling before material yield stress is obtained, and width-to-thickness ratio of steel plates is therefore limited, which is known as "compact section", with a maximum flange width-to-thickness ratio less than 20 in most cases. A new type of PEC column was developed by Canam Manac Group in late 90s. Such a structural element, as shown in Fig. 1, built up with welded H-shape, concrete encasement and transverse links between flanges, is focused in this study. This section could be efficiently fabricated with steel plates cut to any desired dimension. Owing to the reinforcement of transverse links, plates are less susceptible to local buckling compared with those of Eurocode 4 section. As a result, much thinner plates could be applied to this thin-walled section. Typical width-to-thickness ratio range from 20 to 35 for flange and 40–70 for web [2].

A number of experimental studies have been conducted on welded thin-walled PEC columns. Tremblay et al. [3] and Chicoine et al. [4] carried out experimental works on PEC stub columns under monotonic concentric loads. Specimens with various geometric properties showed similar failure mechanism, which was described as local buckling of flanges and crushing of concrete encasement. Web buckling was not observed in these tests. Effect of several parameters on ultimate capacity was also studied in these articles. PEC columns subjected to combine axial and flexural loading were tested [5]. It was observed that the strength and stiffness of PEC columns under minor-axis bending was significantly lower than those under major-axis bending. Extensive experimental studies were performed on long-term behavior [6] and columns with high strength concrete [7]. Cyclic and dynamic performances of PEC columns were studied experimentally by Elnashai et al. [8] and Chen et al. [9]. These experimental works indicated an ideal hysteretic performance of PEC columns.

A series of analytical works have been carried out using finite element method on PEC columns. Elnashai and Elghazouli [10] developed a beam-column fiber section model for PEC sections, which considered material inelasticity. The effects of flange local buckling and concrete confinement were also investigated in this work. Chicoine et al. [11] made use of Riks method to simulate the behavior of PEC columns. This method showed acceptable

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Notation

A_c	area of concrete in PEC section
A_w	area of steel web in PEC section
b	half-width of flange
b_f	full width of flange
d	depth of PEC section
E	elastic modulus of steel
E_c	elastic modulus of concrete
$f_c^{'}$	cylindrical strength of concrete
f_{co}	ultimate compressive stress of concrete
f_{to}	ultimate tensile stress of concrete
f_u	ultimate stress of steel
f_y	yield stress of steel
h	height of entire column
P_{ex}	peak load of PEC columns in experiments
P_{fe}	peak load of PEC columns carried out by FEA
P_{or}	peak load of steel sections in overall models
P_{sp}	peak load of steel sections in single-plate models
R_{ex}	residual load of PEC columns in experiments
R_{fe}	residual load of PEC columns carried out by FEA
R_{or}	residual load of steel sections in overall models
R_{sp}	residual load of steel sections in single-plate models
s	transverse link spacing
t	thickness of flange and web
u_h	deformation of flange in whole column
u_s	deformation of flange between two links
w_{f1}	tensile block width of residual stress at flange-web junction
w_{f2}	tensile block width of residual stress at flange tip
β_r	reduction factor of buckling coefficient considering residual stress
δ	average value of geometric imperfection
ε_{co}	strain at ultimate compressive stress of concrete
ε_h	hardening strain of steel
ε_{to}	strain at ultimate tensile stress of concrete
ε_{tu}	tensile strain at zero stress of concrete
ε_u	strain at ultimate stress of steel
ε_y	strain at yield stress of steel
λ_p	slenderness ratio of half flange
ν	Poisson's ratio of steel
σ_{cr}, $\bar{\sigma}_{cr}$	critical stress of flange and non-dimensional value
σ_{pb}, $\bar{\sigma}_{pb}$	post-buckling stress of flange and non-dimensional value
σ_{rc1}	compressive residual stress value (no sign)
σ_{rt1}	tensile residual stress value at flange-web junction
σ_{rt2}	tensile residual stress value at flange tip

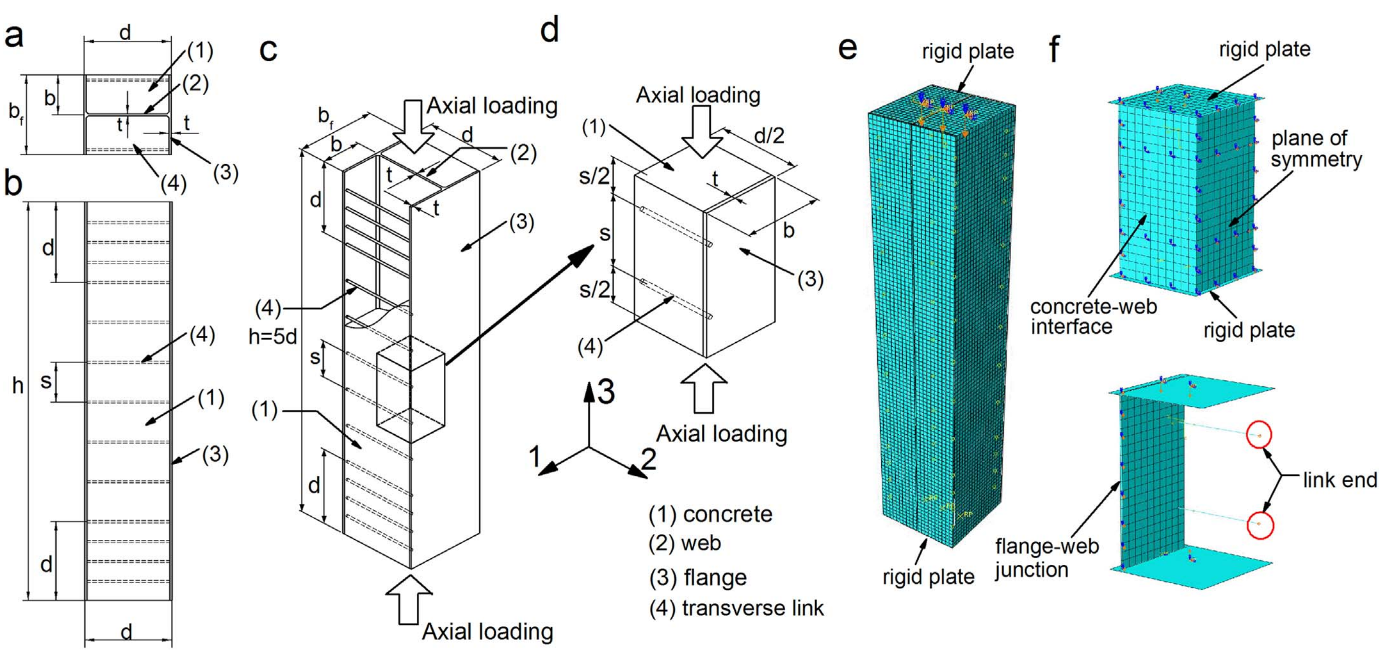

Fig. 1. Geometric configurations and FE models of PEC columns: (a) cross section; (b) elevation; (c) overall model configuration; (d) single-plate model configuration; (e) FE model (overall); (f) FE model (single-plate).

accuracy for predicting pre-peak behaviors. However, in the post-peak region, the Riks method had difficulty in convergence, due to the high nonlinearity of material and deformation. Begum et al. [12,13] proposed a finite element modeling method for PEC columns using the dynamic explicit algorithm. ABAQUS/Explicit module was applied in this work. Concrete was modeled by solid elements with concrete damaged plasticity model, while shell elements were adopted modeling steel shapes. This method was able to simulate main features like flange buckling and concrete expansion of PEC column. Results of numerical simulation showed efficient accuracy in predicting both near-peak and post-peak behaviors of PEC column under concentric and eccentric loadings.

Local buckling of steel shapes in steel structures and steel-concrete composite structures is a critical factor in design. A great deal of effort has been devoted to study local buckling of steel beam-columns [14–20]. In the past few decades, Local and post-local buckling behavior of steels in steel-concrete composite columns has raised growing concerns. Most studies focused on local buckling of concrete-filled steel tubes (CFT) [21–24], in which steel plates develop higher strength and ductility than bare steel tubes. Few works have been done on the local buckling behavior of encased and partially encased composite beam-columns [10,25,26].

The buckling strength of steel shapes in PEC columns is significantly higher than that of bare steel shapes, since flanges can