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Experimental and numerical investigation of eccentrically loaded laced built-up steel columns

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ABSTRACT

The objective of the present study is the experimental and numerical investigation of the in-plane response of scaled Z-laced built-up columns under axial loading applied at the ends with significant eccentricities. The magnitude and direction of eccentricity, the profile of the chords and the density of the lacing are varied, in order to address their effect on the columns' capacity. Next, the corresponding numerical modeling of the experiments is presented in detail by describing the numerical models, types of finite elements and methods of analyses. The numerical analyses include the effects of initial geometric imperfections and thermally induced residual stresses. The numerical results showed a very good agreement with the experimental ones. The effect of end concentrated moments reduced significantly the axial capacity of the specimens.

1. Introduction

Built-up columns are often used in steel buildings and bridges providing economical solutions in cases of large spans and/or heavy loads. Depending on the way that the flanges are connected to each other, they can be grouped into laced and battened built-up columns. Laced columns are investigated in the present work, in which the flanges are connected with diagonal bars, thus establishing truss-like action.

In the design of built-up columns additional effects should be taken into account, which differentiate them from other structural members. The first one is the significant and detrimental influence of shear deformation, which is theoretically supported by the so-called Timoshenko beam theory, initially introduced by Engesser [16]. The Quebec Bridge failure in 1907 was attributed to the buckling of a built-up diagonal and pointed out the significance of this effect. Since then many researchers dealt with this problem, such as Nanni [35], Ziegler [43], Gjelsvik [22], Bazant [7,8], and more details can be found in many structural textbooks, such as the ones of Bleich [9], Timoshenko and Gere [41] and Bazant and Cedolin [6]. Banerjee and Williams [5] explained why the elastic buckling load of members with springs of different rotational stiffness at their ends cannot be obtained from the general equation proposed by Engesser [16] and used by Eurocode 3 [17] for the simply-supported case. The effect of end stay plates on simply-supported built-up columns was investigated by Gjelsvik [21] by considering a layered sandwich cross-section and using a sixth-order differential equation. This method was expanded for other possible boundary conditions by Paul [37] and experimental findings showed good agreement with analytical results [36]. Wang et al. [42] proposed an 8 × 8 stability matrix, providing exact stability criteria for Timoshenko columns with intermediate and end concentrated axial loads. Aristizabal-Ochoa [2] proposed a stability matrix for evaluating the elastic buckling load of Timoshenko members and second-order slope-deflection equations based on Haringx's approach [3]; he, then, compared available methods for the calculation of the elastic critical buckling load of Timoshenko members [4]. Gengshu et al. [20] used Engesser's method to investigate the buckling of dual shear–flexural systems. Razdolsky [38] proposed a method for the buckling of built-up columns considering them as statically-indeterminate structures. Kalochairetis and Gantes [27] proposed a method for calculating the elastic critical buckling load of multi-story frames consisting of Timoshenko members based on Engesser's approach. Gantes and Kalochairetis [19] proposed closed-form solutions for the evaluation of the second-order structural behavior of imperfect Timoshenko members with arbitrary supports under axial and various types of lateral loading, which can be used for calculating the capacity of imperfect laced built-up beam–columns with arbitrary boundary conditions. Comparison of the obtained analytical results with Geometrically and Materially Nonlinear Imperfection Analyses (GMNIA) confirmed a very good agreement between them.

The second issue differentiating built-up columns from other structural members is the interaction between global and local buckling modes. The former is associated with buckling of the built-up member as a whole, while the latter with local buckling of chord components between the points at which the chord and the shear system are connected. The effect of the interaction between global and local buckling in built-up members was investigated by Koiter and Kuiken [34], Bazant

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and Cedolin [6], Svensson and Kragerup [40] and Duan et al. [14]. It was concluded numerically by Kalochairetis and Gantes [26] that a laced built-up column can fail either due to elastic failure of the whole column or due to local inelastic failure of a part between joints of connectors under compression, and that in the first case EC3 may give unsafe results. Nevertheless, the predominant type of failure in the large majority of built-up members is expected to be the second one. All researchers mentioned above concluded that the existence of initial imperfections amplifies the reduction of the collapse load.

The experimental efforts related to built-up columns are limited. Hashemi and Jafari [23] compared the elastic buckling loads of battened columns with end stay plates obtained analytically with experimental results. They concluded that Engesser's method is always on the safe side. The same authors [24] compared experimental collapse loads of simply-supported battened built-up columns with the ones found analytically with the use of the Ayrton–Perry method and the ultimate capacity curve method, observing that a mean value of the two procedures can be both safe and economical. Lue et al. [32] performed experiments on built-up columns with back-to-back flanges connected with interconnectors. Sahoo and Rai [39] tested battened built-up columns under cyclic lateral loading concluding that more dense arrangement of battens at the expected plastic hinge region can be beneficial in terms of flexural strength, lateral stiffness, plastic hinge rotation and energy dissipation. Bonab and Hashemi investigated numerically [10] and experimentally [25] the cyclic behavior of laced built-up columns under a lateral concentrated load and different levels of axial loading. One of their conclusions was that high levels of axial load lead to poor ductility and that laced built-up columns are acceptable for use in moderately earthquake-prone areas. Additionally, they investigated the elastic critical buckling and compressive capacity of centrally loaded laced columns [11]. The seismic vulnerability of axially loaded laced built-up columns with the use of quasi-static loading was investigated by Lee and Bruneau [31]. Klöppel and Ramm [29] and Klöppel and Uhlmann [30] investigated built-up columns both analytically and experimentally, as they tested a large number of eccentrically loaded simply-supported laced and battened built-up columns and their work has been the basis of EC3 guidelines for such columns [33].

The objective of the experimental investigation presented here was to study the behavior of laced built-up columns subjected to combined axial and transverse loading. Concentrated bending moments at the ends were adopted as a first case of transverse loading, by introducing eccentricities in the applied axial load and associated boundary conditions. The cross-section of the chords, the magnitude and direction of eccentricity, and the density of the lacing were varied, in order to address their effect on the columns' capacity. Then, the experimental results were used to calibrate nonlinear finite element models for describing the response of such structural members. A preliminary presentation of the experimental and numerical findings has been provided by Kalochairetis and Gantes [28]. Details of both the tests and their numerical modeling are presented in the following sections.

2. Description of experiments and experimental set-up

2.1. Specimen groups

A total number of 10 simply-supported eccentrically loaded columns have been tested at the Institute of Steel Structures in the School of Civil Engineering of the National Technical University of Athens, grouped in five pairs of similar columns for repeatability purposes. The selection of geometry and cross-section of specimens was made by balancing between the desirable research aspects and the limitations imposed by the existing equipment in the laboratory regarding the dimensions of the specimens and the maximum load that the actuator could apply. Namely, the total available height under the testing frame is approximately equal to 3.9 m, within which the actuator, the top and bottom supports and the specimen should be placed, thus leading to an available height of 234.5 cm for the specimens. Moreover, the maximum load that can safely be applied by the actuator is equal to 500 kN.

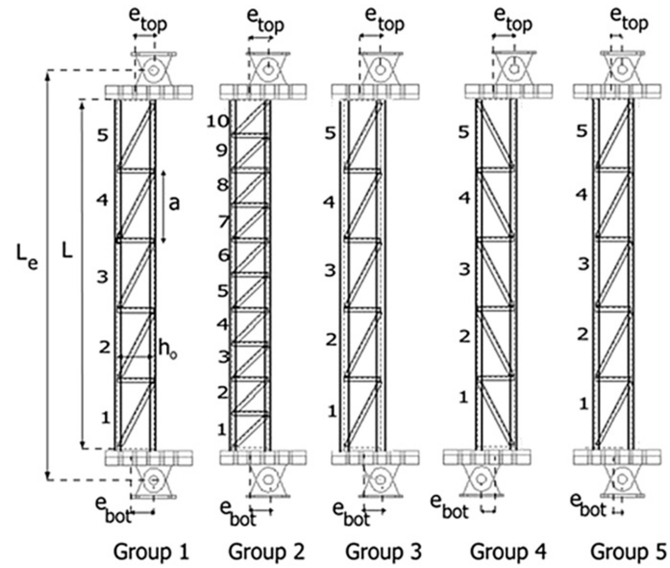

Fig. 1. Specimen groups.

Each specimen was assigned a name according to the group it belonged to, followed by the number 1 or 2 (i.e. the two specimens of Group 1 were named 1(1) and 1(2)). Indicative sketches of the column specimens in the five groups as well as their top and bottom hinged supports and eccentricities (e_{top} and e_{bot}) are shown in Fig. 1. The numbering of the panels is also shown, as measured starting from the bottom support. The actual length L of all specimens was equal to 202 cm, while their effective length L_e, as measured from the top pinned support to the bottom one, was equal to 234.5 cm, satisfying marginally the maximum available height limitation. In all cases the lacing bars had an angle cross-section L25 × 25 × 3 and were welded on the chords. Additional characteristics for each specimen group, including panels' lengths, chords' cross-sections, magnitude and direction of eccentricities of the loads at the ends (considered as positive when resulting in clockwise end concentrated moment), are listed in Table 1.

The design of the specimens was based on allowing failure to take place only along the chords. Thus, the specimens' diagonal bars were selected to have sufficient over-strength, in accordance with the common practice. This was one of the main reasons for selecting angle cross-sections for the diagonal bars, as they are characterized by larger buckling strengths when compared to rectangular cross-sections of the same area. Observing the eccentricities of the loads at the top and bottom of the specimens, it can be concluded that in Groups 1, 2, 3 and 5, the specimens were subjected to end moments of opposite direction leading to single curvature deformation, while in Group 4 the end moments were of the same direction, thus enforcing double curvature deformation. The first two groups differed only as far as the density of the lacing is concerned, in order to investigate the effect of a stiffer lacing, of the total number of welds and of the smaller length of panels on the structural response and bearing capacity of the columns. Group

Table 1
Characteristics of the five specimen groups.

Group	Panel's length a	Flanges' cross-section	e_{top}	e_{bot}
1	40	UNP60	10	−10
2	20	UNP60	10	−10
3	40	IPE80	10	−10
4	40	UNP60	10	8
5	40	UNP60	5	−5