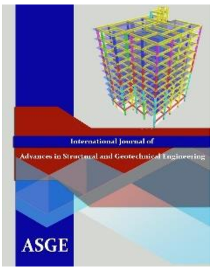

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FINITE ELEMENT MODELLING OF AXIALLY- AND ECCENTRICALLY-LOADED BUILT-UP LACED COLUMNS

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ABSTRACT

This paper presents the results of about seventeen tests, collected from literature, conducted on axially and eccentrically loaded built-up hot-rolled steel section laced columns. Four axially loaded specimens were constructed using hot-rolled channels connected together with lacing plates. The built-up column samples had various lengths and various distances between the main chords. Each test specimen was tested for several times. In each times the specimen was considered to be a column with different initial imperfection. The eccentrically loaded columns were constructed using two hot-rolled channels connected together with Z-lacing. The Z-lacing bars had an angle cross-section. For both axially and eccentrically loaded columns, the lacing bars were welded to the chords. A 3-D finite element model was created to simulate the behavior of these columns. The numerical modeling of the experiments is presented in detail by describing the numerical models, types of finite elements, end configurations and methods of analyses. The numerical analyses included the effects of initial geometric imperfections and used the material properties measured from test. 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.

Keywords: Built-up, Laced column, Finite element, Buckling.

INTRODUCTION

Built-up columns are often used in steel buildings and bridges to provide economical solutions in cases of large spans and/or heavy loads. Depending on the way that the chords 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 chords are connected with diagonal lacing bars, thus establishing truss-like action.

Johnston [1] studied the spaced steel columns. In this study the spaced columns were defined as the limiting case of a battened column in which the battens are attached to the longitudinal column elements by hinged connections. End tie plates in battened columns contributed significantly to the buckling strength. Their effect is accentuated by the study of a spaced column. The strengthening effect of the end tie plates is due to two factors: (1) a shortening of the length within which the column components can bend about their own axes and (2) the longitudinal components are forced to buckle in a modification of second mode shape and thus have elastic buckling coefficients that approach four times those of the first mode. In addition to their contribution to column strength, end tie plates perform their usual role of distributing the direct or moment applied loads to the component elements of either laced or battened columns. The out-of-plane buckling of battened columns under axial and/or moments was investigated by Toossi [2]. Temple and Elmahdy [3], carried out an experimental and theoretical study to investigate the behavior of battened columns made of standard channel steel sections. The number of connectors and the accompanying design strength for double-angle columns was determined by Zahn and Haaijer [4]. Temple and Elmahdy [5] investigated the buckling mode of built-up member. Their research also provides a brief derivation of the equivalent slenderness ratio equation and its applicability. Temple and Elmahdy [3] concluded that the slenderness ratio of the main member between connection points has a significant effect on the compressive resistance. Built-up columns experimental tests have been conducted by Dung et al. [6] and Liu et al. [7] using hot-rolled built-up columns. The slenderness ratio of built-up columns was investigated and the slenderness ratio formulas as specified in various design codes were discussed. Hashemi and Jafari [9, 10] investigated experimentally the elastic critical load of hot-rolled built-up columns. The two papers also provide an evaluation for some theoretical methods for predicting the elastic critical load and the compressive strength of built-up columns.

In the design of built-up columns additional effects should be considered, which characterize 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 [10]. The failure of Quebec Bridge in 1907 was attributed to the buckling of a built-up diagonal and pointed out the significance of shear deformation effect. Since then many researchers studied this problem, such as Nanni [11], Ziegler [12], Gjelsvik [13], Bazant [15,16], and more details can be found in many structural textbooks, such as the ones of Bleich [16], Timoshenko and Gere [17] and Bazant and Cedolin [18]. Banerjee and Williams [19] 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 [10] and used by Eurocode 3 [20] 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 [22] and experimental findings showed good agreement with analytical results [23]. Aristizabal-Ochoa [24] proposed a stability matrix for evaluating the elastic buckling load of Timoshenko members and second-order slope-deflection equations based on Haringx's approach [25]; he, then, compared available methods for the calculation of the elastic critical buckling load of Timoshenko members [26]. Gengshu et al. [27] used Engesser's method to investigate the buckling of dual shear–flexural systems. Razdolsky [28] proposed a method for the buckling of built-up columns considering them as statically indeterminate structures.

The second issue differentiating built-up columns from other structural steel 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 flange components between the points connecting the flanges to the shear system. The effect of the interaction between global and local buckling in built-up members was investigated by Koiter and Kuiken [29], Bazant and Cedolin [18], Svensson and Kragerup [30] and Duan et al. [31]. It was concluded numerically by Kalochairetis and Gantes [32] 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 tests related to built-up columns are limited and therefore the numerical investigations are limited as well. The experimental efforts related to built-up columns are limited. Hashemi and Jafari [8] 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 [9] 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. [6] performed experiments on built-up

columns with back-to-back flanges connected with interconnectors. Built-up cold-formed steel columns were investigated experimentally and numerically by Dabaon et al. [33-36] and Ramzy [37].
The objective of this paper is to provide a finite element model which is able to perfectly simulate the behavior of laced built-up columns subjected to axial and eccentric loading as a part from the extensive study conducted by Ramzy [38] on built-up CFS laced columns.

SUMMARY OF EXPERIMENTAL TESTS [39] and [40]

Axial Compression Tests [39]

The experimental tests conducted on built-up steel columns by [39], contained four built-up column samples with various lengths and various distances between the main chords. Each test specimen consisted of two main chords which were constructed using hot-rolled channel section having the profile of UNP 60. The main chords were connected using lacing plates having the section of 3 mm by 10 mm. Other specifications of the test specimens are given in Fig. 1 and Table 1.
All test specimens were pin-ended columns. To create the end conditions for a double-hinged column, a plate was welded to either end of each sample, and the plate was bolted to the hinge. The type of hinge part used was the bowl and sphere. The main concern in this study was buckling occurring about the y-axis of the built-up columns so that two roller-bearing supports were used at either side of the sample column at its mid-height to prevent the column from buckling about the x-axis. To obtain the column deformations in its three first modes of buckling, three LVDTs (linear variable displacement transducer) were installed along the height of the sample column at its mid-height and 1/4th of its length from each end. The axial force was applied by a displacement-control actuator, as described in [39]. Each test specimen was tested for several times. In each time the specimen was considered to be a column with different initial imperfection. The specimens were labeled as shown in the first column of Table 1, where the number after the letter L indicates the net length of the channel profile and the number after the letter B indicates the dimension of the column cross-section in the plane parallel to the lacing planes. The number after the letter R indicates the number of test repetitions. In Table 1, the effective length in the third column is the distance between the two pinned ends of the top and bottom hinges.
The material properties of tested columns were obtained from tensile test conducted on three samples taken from the web of the channels. The obtained specifications are listed in Table 2.

Eccentric Compression Tests [40]

A total number of 10 simply-supported eccentrically loaded columns, grouped in five pairs of similar columns for repeatability purposes, were tested in this experimental program. Each group's specimens consisted of two main chords which were constructed using hot-rolled channel section having the profile of UPN 60 except the specimens in Group 3 which were constructed using hot-rolled IPE 80 profiles. Only the groups constructed using UPN 60 were included in the current study. The actual length of the chords was 202 cm while the effective length of all specimens was 234.5 cm. The main chords were connected using lacing bars having an angle cross-section L25×25×3 and were welded on the chords. Other specifications of the test specimens are given in Table 3.
All the specimens were tested between eccentrically hinged supports at the top and the bottom of each specimen. Horizontal beams were placed at specimens' mid-height in order to restrict out-of-plane movement. The horizontal beams were at a very small distance from the specimens to avoid any undesirable contact between them. Figure 2 shows the locations of measuring displacements and strains. Linear Variable Differential Transformers (LVDTs) were used for measuring the lateral deflections at the mid-height (G1, G2, G5) and at a distance equal to 1/5 of the actual length from the bottom and top supports of the column (G4). These measurements were used for obtaining load-horizontal displacement graphs and identifying the global response of the columns. Each specimen was named 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)).

The material properties of the steel used for manufacturing the specimens were extracted through tensile tests with the use of displacement control in appropriately designed coupons obtained from the columns' chords. The average Young's modulus for all specimens was 210 GPa, which is in accordance with the value provided in many structural textbooks for steel. The yield stress for each coupon was taken as the 0.2% proof stress found in the plateau following the elastic branch. The mean yield stress for each group is given in Table 4.

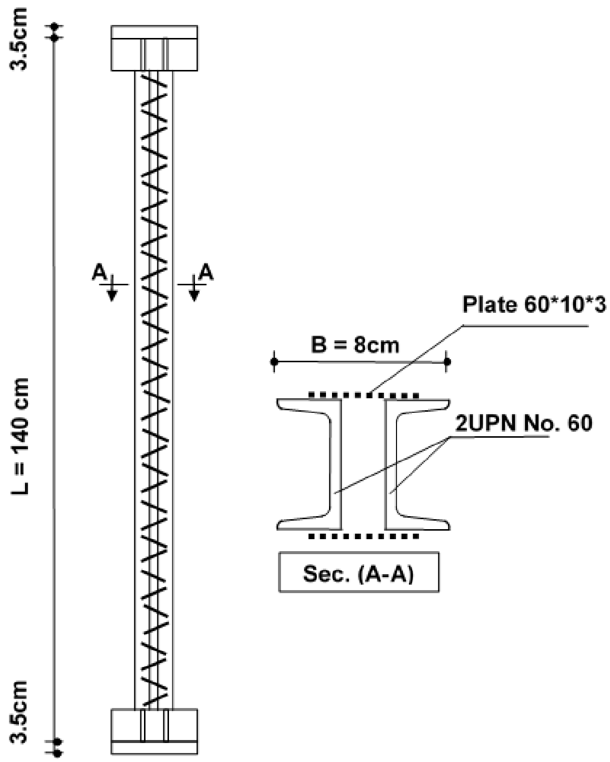


Fig. 1: Details of a typical specimen [39]

Table 1: Specifications of the specimens tested in [39]

Specimen	Length L (mm)	Effective length L_e (mm)	Chords' cross-section
L140B8(R1)	1400	1680	UNP60
L140B8(R2)	1400	1680	UNP60
L140B8(R3)	1400	1680	UNP60
L140B10(R1)	1400	1590	UNP60
L170B7(R1)	1700	1890	UNP60
L170B7(R2)	1700	1890	UNP60
L170B7(R3)	1700	1890	UNP60
L170B8(R2)	1700	1890	UNP60
L170B8(R3)	1700	1890	UNP60

Table 2: Material properties in [39]

Yield stress F_y (MPa)	Young's modulus E (GPa)	Ultimate stress F_u (MPa)
277.72	196.20	405.13

Table 3: Specifications of the specimens tested in [40]

Group	Actual length L (mm)	Effective length L_e (mm)	Panel's length (mm)	Eccentricities	
				e_{top} (mm)	e_{bot} (mm)
G1	2020	2345	400	10	-10
G2	2020	2345	200	10	-10
G4	2020	2345	400	10	8
G5	2020	2345	400	5	-5

Table 4: Material properties in [40]

Group	G1	G2	G4	G5
Yield stress F_y (MPa)	338	338	335	302
Young's modulus E (GPa)	210			

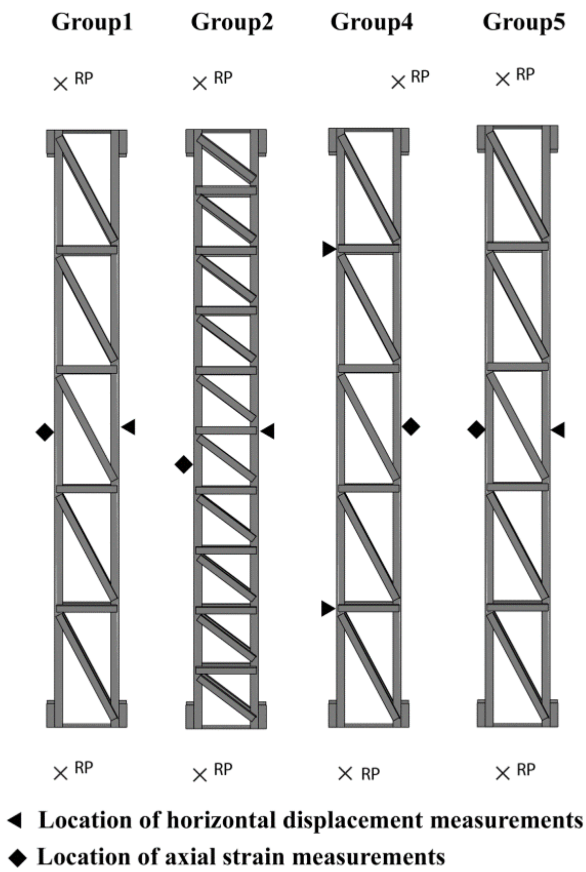


Fig. 2: Locations of measured displacements and strains

FINITE ELEMENT MODEL

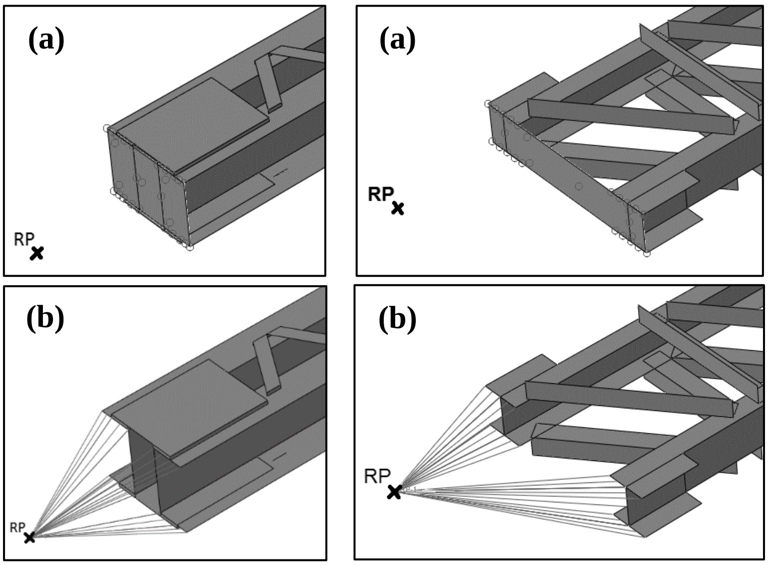
The finite element program ABAQUS [41] was used to simulate the behavior of the built-up steel laced columns. The model used the measured geometry and material properties. Finite element analysis for buckling requires two types of analyses. The buckling modes of the columns are estimated, first, through the Eigenvalue analysis. This is a linear elastic analysis performed using the (*BUCKLE) procedure available in the ABAQUS library with the load applied within the step. The Eigenvalue analysis were performed for a number of buckling modes and the adequate buckling mode predicted from Eigenvalue analysis was used. A load-displacement nonlinear analysis is, then, carried out. In this analysis, the initial imperfections and material nonlinearity are included. From this analysis, the ultimate loads, failure modes, lateral displacements, axial strains and axial shortenings are determined.

The S4R shell element is used to model the channels and lacing bars. The S4R element has six degrees of freedom per node and provides an accurate solution for most applications. The mesh that provided adequate accuracy and minimum computational time in modeling the steel built-up section laced columns was chosen by selecting approximate global size equals to 6mm. A finer mesh was used at the corners. Figure 3 shows the shape of the current finite element mesh.

Two different methods were tried to simulate the end plates and hinged supports. The first one contains upper and lower discrete rigid end plates. Fine mesh of three dimensional four-node bilinear rigid quadrilateral shell elements, so called R3D4, was used. A point at a distance perpendicular to the plane of each end plate and faces the center of the plate was selected to act as a reference point (RP) according to the effective length of each specimen between the two hinged supports. This loading plate is constrained to the channels by "Tie" constraint, to ensure that the displacements and rotations of the connected elements were kept the same in the whole loading process. Tie-constraint is a feature in ABAQUS by which three-dimensional shell meshes can be coupled automatically to three-dimensional shell meshes. In the second method, reference points were selected to simulate the centers of upper and lower hinged supports. These reference points were constrained to the channels by "Coupling" constraint. Coupling-constraint is a feature in ABAQUS by which a reference point can be coupled automatically to three-dimensional shell meshes, as shown in Fig. 4. In both methods, the boundary conditions were assigned to the RPs, while the load was assigned to the RP of the upper support. Generally, the second method showed the same results with less effort and less time consuming.

The material properties and initial imperfections measured from the experimental tests were used in the model while the residual stresses were not included in order to avoid the complexity of the analysis.

Fig. 3: Current finite element mesh for axially and eccentrically loaded columns



Axially loaded columns **Eccentrically loaded columns**

Fig. 4: Different hinged end modeling methods (a) Rigid plate, RP and Tie constrain (b) RP and Coupling constrain

COMPARISON BETWEEN EXPERIMENTAL AND NUMERICAL RESULTS

Results of Axially Loaded Columns

Table 5 shows a good agreement between the experimental and the numerical results. All the tested specimens failed in an overall buckling mode of failure with different compressive capacities. Geometrical imperfections can drastically decrease the ultimate compressive capacity of laced columns. Although the amount of decrease is not the same for different cases with the same variation of imperfection, the amount of capacity reduction is remarkable for almost all the cases. In one case, when the amount of imperfection doubled, there was a 22% decrease in the column's capacity. The column capacities obtained from the experimental tests (P_{Test}) and finite element analysis (P_{FE}) were compared, as shown in Table 5 as well as presented in Fig. 5, and a good agreement was achieved. The mean value of the (P_{Test}/P_{FE}) ratio is 0.995 with the corresponding coefficient of variation (COV) of 0.019, as shown in Table 5 and presented in Fig. 5.

Results of Eccentrically Loaded Columns

The test and numerical results of eccentrically loaded column, in Table 6 and in the figures from Fig. 5 to Fig. 10, showed that a very good agreement was achieved. The model simulating Group 1 deformed in a single curvature configuration and the collapse load of the G1 numerical model was 195.37 kN. In the post-buckling range, the second panel buckled inwards as noticed in the tests. In Group 2 the model deformed in single curvature and the collapse load of the G2 numerical model was 200 kN, thus the denser lacing arrangement offered only a small increase in the capacity. Due to the stocky nature of the specimens at local level, no local buckling of the panels was observed. The collapse load was equal to 226.97 kN for the model simulating Group 4 and the model deformed in a double curvature leading to very small horizontal displacements. It should be noted that the eccentricity at the top was slightly larger than at the bottom (see Table 3) and for this reason the fifth panel buckled inwards. The collapse load of the G5 numerical model was 249.1 kN, larger than in Group 1 due to the smaller eccentricity at the ends. In the experimental tests, the panels buckled inwards were not the same in both specimens. As in Group 1, the intermediate panels were expected to have similar levels of stresses and therefore the different locations of local buckling in Group 5 indicate the different effect of initial imperfections.

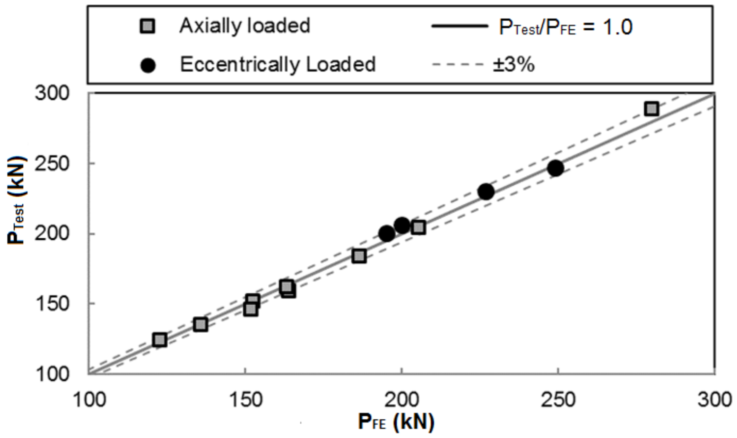
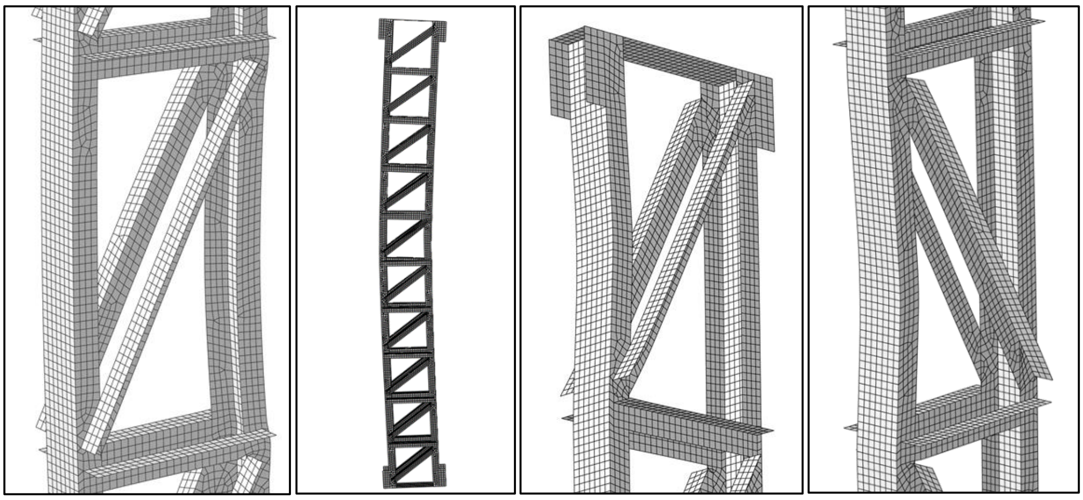


Fig. 5: Verification of the finite element models

Table 5: Verification of the Axially loaded FEM

Specimen	P_{Test} (kN)	P_{FE} (kN)	P_{Test}/P_{FE}
L140B8(R1)	204.76	205.24	0.998
L140B8(R2)	183.86	186.51	0.986
L140B8(R3)	159.09	163.90	0.971
L140B10(R1)	289.05	279.86	1.033
L170B7(R1)	151.95	152.51	0.996
L170B7(R2)	135.19	135.84	0.995
L170B7(R3)	124.32	122.86	1.012
L170B8(R2)	162.40	163.09	0.996
L170B8(R3)	146.29	151.68	0.964
Mean			0.995
COV			0.019

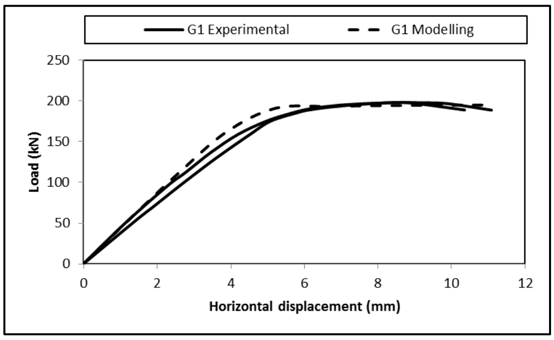


Group 1 **Group 2** **Group 4** **Group 5**

Fig. 6: Modes of failure found in the eccentrically loaded columns

Table 6: Verification of the Eccentrically loaded FEM

Specimen / Group	P_{Test} (kN)	P_{FE} (kN)	P_{Test}/P_{FE}
G1	200	195.37	1.024
G2	206	200.00	1.030
G4	230	226.97	1.013
G5	247	249.10	0.992
Mean			1.015
COV			0.014



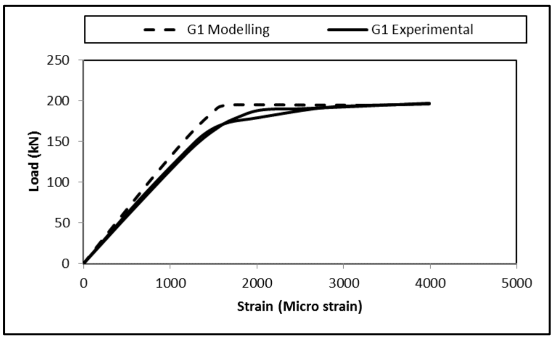


Fig. 7: Test Vs. Numerical results of Group 1

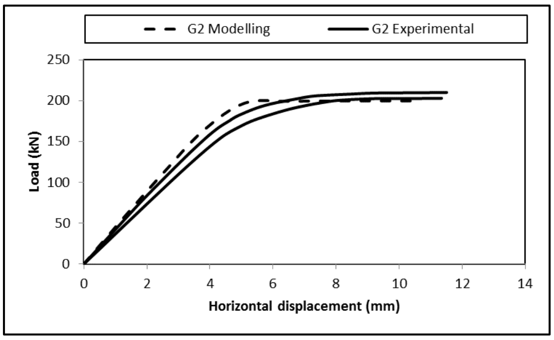
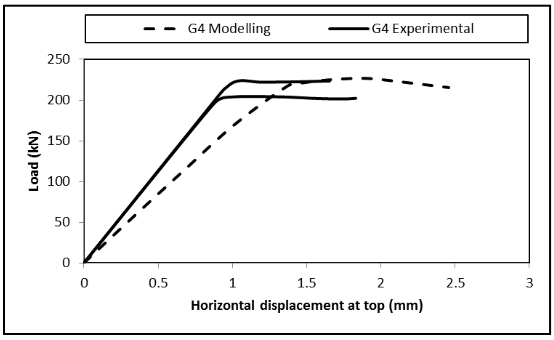


Fig. 8: Test Vs. Numerical results of Group 2



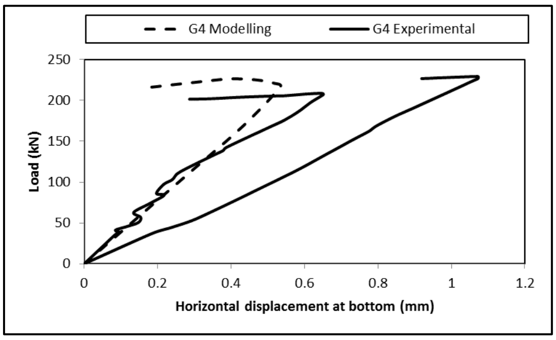


Fig. 9: Test Vs. Numerical results of Group 4

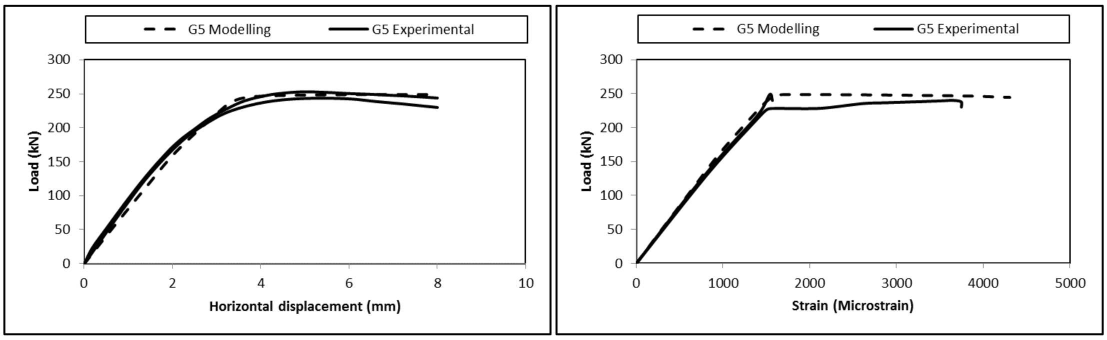


Fig. 10: Test Vs. Numerical results of Group 5

CONCLUSIONS

The data and results of tests done by [39] and [40] on a total number of nine axially loaded and four pairs of eccentrically loaded built-up hot-rolled steel section laced columns, respectively, were used to create a 3-D finite element model to simulate the behavior of built-up laced columns. Both axially and eccentrically loaded columns were compressed between hinged supports. In both cases, lateral supports near to the mid-height of columns were used to ensure a uniaxial deformation. The detailed finite element model results were presented and compared to the test results. A very good agreement between the test and the numerical results was achieved. All the axially loaded specimens failed in an overall buckling mode of failure with different compressive capacities. Geometrical imperfections can drastically decrease the ultimate compressive capacity of laced columns. The test and corresponding numerical results of eight specimens (four pairs of similar specimens) of laced built-up steel columns tested under combined axial load and end concentrated moments have been presented. The global response and collapse load were affected mainly by the cross-sectional area of the chords, the magnitude and direction of eccentricity at the columns' ends. The effect of end concentrated moments reduced significantly the axial capacity of the columns. The local buckling of chords in different intermediate panels even in specimens of the same group revealed that local initial imperfections had a significant local effect. In Group 4, the end panels were much more stressed than the other panels and initial imperfections could not modify the location of failure.

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